RCSC Executive Committee Meeting
June 6th, 2012 - Residence Inn, Deptford NJ
3:00 pm to 6:00 pm ET
Meeting Minutes

1. **Welcome - Call to order (Carter)**

   Carter called the meeting to order. It was requested that an update be added to the agenda for the status of the work of RCSC member Grondin.

   Carter also mentioned to add the resignation of Tide from the membership and funding committee.

2. **Approval of the Agenda (Carter)**

   Carter called for an approval of the agenda, with the added items. Motion to accept by Greenslade, Second by Harrold, all in favor.

3. **Approval of Minutes of the Previous Meeting (Carter)**

   Carter called for an approval of the minutes of the previous Executive meeting, January 19^{th}, 2012. Motion to accept by Greenslade, second by Harrold, all in favor.

4. **Nominating Committee Report (Carter)**

   Carter read the nominations and results of the April 29^{th} ballot for the open Executive Committee positions. The recommendations of the nominating committee were carried through by council vote. Three officer positions were to be filled by Carter (Chairman), Miazga (Vice-Chair) and Greenslade (Secretary/Treasurer). Two of the six director positions were up for election. Vissat was re-elected as director, and Ude was elected as a first term director.

   Motion to accept the nominating committee report by Greenslade, second by Hundley, all in favor.

5. **Bylaws Ballot (Carter)**

   Carter reviewed the pending changes and ballot results. This prompted a discussion on organizational voting. While it had not been a problem in the past there is the potential for confusion over voting privileges when an organization has more than one member. Greenslade or Carter will bring this topic up to the main meeting membership to preempt any future problems and reserve the right for the council to review the bylaws in the future on this topic.

6. **New Member Applications (Carter)**

   Carter reviewed open membership applications for Prchlik and Auer. Both new members were accepted. Carter will notify each of new membership status.

7. **Secretary Report (Larson)**

Larson gave the Secretary report. See included attachments.

New Members - 4 since January, 2 approved earlier this meeting.
Pending Membership Changes - none
Membership Summary 90 total - 30 new in past 5 years. Moved to 92 with the addition of the 2 approved at this meeting.
Unpaid Members - 20

There was a lengthy discussion on unpaid members. Once the new billing cycle completes, unpaid members will be dropped from the membership.

Motion to accept the Secretary report by Miazga, second by Hundley, all in favor.

8. **Treasurer Report (Larson)**

Larson gave a cash flow and financial summary of the council. See attached documents

-Cash Flow Summary
-Financial Statement
-Taxes are essentially ready to submit for 2012

Financial Base Move to AISC was previously approved and Larson reported that the physical change has happened.

Payment to video production company for Turn of Nut video.

Payment to identity and web design company for Logos and web design.

Larson made a formal motion to move the Council to a calendar fiscal year, second by Greenslade, all in favor.

Motion to accept Treasurers report by Greenslade, second by Hundley, all in favor.

9. **Research report (Ricles)**

Dusicka - Research status. Council had been waiting for an updated combined report and the consideration of a few comments and acknowledgements. The work is substantially complete and the Research Committee will make a motion to accept the report and issue final payment of $20,000 after comments from the council.

University of Toronto Phase II Progress. This work is on hold.

Brahimi - Rotational Capacity. This work is on hold and will be considered again in the future as new business.

Brahimi - Hydrogen Embrittlement. This work is ongoing and only very partially funded by RCSC. While we have no complete final report there is a significant synopsis article about the progress to date, which RCSC may consider as good enough for our involvement. Recommend final payment be made to Brahimi.

Motion by Miazga, second by Harrold to accept the research report, all in favor.

Larson made a formal motion to pay Dusicka and Brahimi and to cancel any research work which has not started if the researcher agrees and to cancel pending research which has not had any progress. Second by Hundley, all in favor. Larson will notify Brahimi, Carter will notify others.

10. Specification Items (Harrold)

Harrold gave the Executive committee a summary of the ballot item negatives and new ballot item proposals that will go before the specification committee the following day. See attached and Specification committee minutes for more information.

12. Old Business

Increase in dues for GI member to $100 will be tabled for now.

13. New Business

Tide resignation from Chair of membership and funding committee. Replacement will be considered once the needs and future of council spending is determined by the Greenslade task group. In general the financial and membership health of the council is in good condition. Larson will give interim M/F report at meeting. Carter to consider options for replacement of Tide.

Grondin update by Miazga. Work will be terminated as it is not likely to be brought to conclusion. A number of alternatives were discussed. A plan will be formalized to move forward with work that has been substantially completed, but is not in final form.

14. Adjourn

There was a motion by Harrold for adjournment, second by Larson, all in favor.

RCSC Executive Committee Meeting
June 6th, 2012 - Residence Inn, Deptford NJ
3:00 pm to 6:00 pm ET
Agenda

1. **Welcome - Call to order (Carter)**

2. **Approval of the Agenda (Carter)**

3. **Approval of Minutes of the Previous Meeting (Carter)**

4. **Nominating Committee Report (Carter)**

5. **Bylaws Ballot (Carter)**

6. **New Member Applications (Carter)**

7. **Secretary Report (Larson)**
   New Members
   Pending Membership Changes
   Membership Summary 90 total - 30 new in past 5 years
   Unpaid Members - 20

8. **Treasurer Report (Larson)**
   Cash Flow Summary
   Financial Statement
   Financial Base Move to AISC
   Payment to video production company
   Payment to identity and web design company

9. **Research report (Ricles)**
   Dusicka - Research report review

10. **Specification Items (Harrold)**
    2011-12 Ballot items
        Status of Ballot
        New Ballot Items

12. **Old Business**

13. **New Business**

14. **Adjourn**

RCSC Executive Committee Meeting

January 19th, 2012 – 8:00 am CT

AISC Office

Minutes

Welcome – (Carter)

Carter called the meeting to order at 8:10 and welcomed the executive committee members to Chicago. He covered the plan for the day and some general housekeeping items.

a) Approval of Minutes of the Previous Meeting (Carter)

Carter called for an approval of the minutes of the previous executive and main meetings. There was some review and a few comments and a requested change to the executive minutes. Larson would make the correction and post the minutes on the council website.

In review of the minutes and in clarifying the presentations that were given at the previous meeting, Larson summarized which presentations were given and which ones were left without adequate time to present. Greenslade offered some background information and ASTM task group status on DTI washers for review purposes. This topic was left on the shelf at the annual meeting due to time constraints. It was decided that the RCSC would not take a position in the middle of an ASTM standards debate, but rather wait to incorporate the outcome of the topic into our next standard.

Minutes were approved for both meetings with no objections.

b) Appointment of Nominating Committee and Open Positions (Carter)

After some discussion of available candidates, the committee settled on a nomination committee including McGormley, Rasatti, and Shoemaker. Carter will contact each of them to see if they are willing to serve on the nomination committee. Positions up for nomination this year include Carter (Chairman), Miazga (Vice Chair), Larson (Secretary/Treasurer), and two director positions, Greenslade and Vissat.

Assuming acceptance of the positions, the nominating committee will be asked to make appropriate nominations and create a ballot for the membership. Rassati will chair.

c) ***Bylaws - Ballot Status (Carter)***

Carter prepared a draft ballot and sent it to Larson and Harrold for review. Larson compared the wording to known previously approved yet un-balloted changes. Harrold gave Carter comments that he believed the wording for the handling of ballots and negatives was still circular in nature. Carter liked the proposal by Harrold and all agreed. The changes to meet his suggestions would be incorporated in the final draft of the ballot.

Janet at AISC will prepare the ballot and submit it to the membership in its entirety, but with each ballot item voted on separately. Miazga ask that we provide to the membership a brief rationale for each of the changes, as many of the rationale are known well to the executive committee, but not to the whole group. All agreed that this was a good idea.

The ballot will be sent out via AISC's electronic balloting system, a first for RCSC.

d) ***New Member Applications (Carter)***

Larson noted that Toby Anderson was approved in June, but he did not think a welcome letter had been sent out yet. Larson agreed to send Anderson a note.

Two new membership applications were received since the last meeting. See attached. Garret Byrne and Matthew Eatherton. After some review and discussion both applicants were approved as new general interest members with a motion by Harrold and a second by Miazga. Carter will send a welcome note to each.

Larson covered some requested replacements for new corporate representatives -

Tinney - Birmingham

Menke - Fastenal - attached

Bornstein - Skidmore – attached

Two of the three candidates had supplied information for review. Larson made a motion to accept the new corporate representatives for each company, with a second by Vissat, all in favor.

e) General (Larson)

(1) RCSC Facebook Page - Larson gave a demonstration and a preview of a proposed RCSC (Bolt Council) Facebook page that he put together. There was a lengthy debate on the pros and cons of moving the Council into social media. Ultimately it was moved by Greenslade with a second by Miazga that we create and post a Facebook page and make it known to our members at the June meeting.

It was decided that Larson would continue the work on a page and it was requested that he develop a policy as discussed at the meeting that the executive committee have control over the postings, that they not be of a commercial nature or to the direct benefit of any members, and that the information provided would be of general interest to the majority of our membership and that it be maintained in such a way as to provide the next generation of RCSC members some insight into our organization.

(2) Google Docs file storage for Key Documents – Larson gave a presentation of work he had done to move Council records and documents into Google Docs. There were number of reasons for the move, including maintenance and preservation of Council intellectual property, ease of use and access to all council documents, safe backup and storage, revision control of key documents, a central store for research papers, and continuity between executive position changes.

Larson had populated the document center with a number of key Council papers, forms, report, financial information and taxes and will continue to do so as he serves the balance of his final term of Secretary and Treasurer. He is also scanning and storing Council documents that were given to him from the previous Secretary/Treasurer.

All were in favor of the migration to Google docs after the presentation and there was a motion by Greenslade and a second by Miazga, with all in favor to move that direction.

(3) boltcouncil@gmail Address – Along with the Google Docs proposal, Larson explained the creation and benefit of a boltcouncil@gmail e-mail address. This will provide one address for use on Council documents, Facebook page, website, etc.

Having our own address will offer some continuity to the Council communications moving forward. This was also motioned for approval by Greenslade with a second by Miazga, all in favor.

(4) New identity for Council – Larson gave a presentation of the work that had been done to create some new identity pieces for the RCSC. Greenslade, Carter and Larson were on a committee to select the best package. The new logo and related style sheets and website design were all shown to the balance of the executive Committee. There was a motion for approval of the new identity pieces by Greenslade with a second by Miazga, all in favor. Larson will provide links to the new identity pieces to the executive committee and present to the full membership at the annual meeting.

f) Secretary Report (Larson)

(1) New Members - Anderson – Others – This was covered earlier in the meeting.

(2) Pending Membership Changes - Menke - Bornstein – Tinney – This was covered earlier in the meeting

(3) Membership Summary 90 total - 30 new in past 5 years – Larson gave a summary of the membership levels and status. He noted that of our record membership, 30 of the members were new in the last 5 years.

(4) Unpaid Members - 6 - no repeat offenders – There are 6 members with unpaid dues from last year. None of them are repeat offenders. The balance owed will be added to

the next invoice cycle and reviewed again for payment at the next meeting. It was noted again, that all unpaid members were of the general interest category.

g) Treasurer Report (Larson)

(1) Invoicing - Move to Dec 15th – Larson explained the pending move of the membership research contribution invoices to the month of December. This is being done to give corporate members the opportunity to pay in either year. It was later suggested that the fiscal year be changed and that may change the desired date of invoicing. This will be discussed further at the next meeting after discussion with the accountant regarding the possibility of changing the fiscal period

There was a motion by Greenslade and a 2nd by Harrold to change the RCSC fical year to a calendar year. This will require a change to the bylaws at that time.

(2) Cash Flow Summary – Larson presented the summary of the RCSC cash flo

(3) Financial Statement YTD – Larson presented the financial statement YTD.

(4) Financial Base Move to AISC – Larson explained the coming move of RCSC funds to AISC.

(5) Tax Return – Larson shoed the completed tax returns from the previous year and noted there were no problem or changes and no actions required. He stated that the current CPA has agreed to continue working on the taxes after the movement of the Treasurer at the next meeting.

(6) Payment to video production company

There was a motion to accept the secretary and treasurers report by Greenslade and a second by Harrold. Larson agreed to send all important financial information to David Slade at AISC in preparation of the movement of funds.

h) Research report (Ricles)

There was a general discussion on how best to manage the research process moving forward. With limited funds and limited resources it is a challenge to fund, manage and support large research projects. Other uses of RCSC funds, such as collaborative funding, education, grants, etc. were considered. It was agreed that a task group would be put together to take a look at the problems and options and come back to the Executive committee with ideas.

The task group includes Greenslade (chair), McGormley, Swanson, Schlafly, and Larson. They will consider ideas surrounding research, education and training.

(1) Dusicka - Research status – This work is done, but the council is awaiting a combined report from the researcher. This will be provided before the annual meeting.

(2) University of Toronto Phase II Progress. Peter will be contacted to ask if he will drop planned work on Phase 3 of this project. Larson will try to get in touch with him to see what his intentions are.

(3) Brahimi - Rotational Capacity – Larson reported that this project is in limbo waiting for financing and information for matching contributions. The scope has changed and he feels maybe it should start over with a full review. Larson will talk to Brahimi to see if there is interest in moving forward.

(4) Grondin – Progress work on the guide will be suspended.

i) Specification Items (Harrold)

(1) CSA, RCSC, AISC task group. (Miazga) Miazga presented the report from his task group. There were no significant changes since the annual meeting and the task group had no need for activity in the fall. He will report on any changes at the annual meeting.

(2) 2011-12 Ballot items – 4 Spec change items. Harrold commented and provided a brief description of the pending ballot items. The ballot will be sent out with the nominations and by-laws ballot.

(3) Pending Items from Tom Schlafly – Schlafly will propose a change to remove Dacromet from F1852 and F2280 in the RCSC specification. The items did not pass ASTM ballot.

(4) Pending Items from Rich Brown – Rich Brown provided a research report about hardened vs. non hardened DTI's. It was agreed that RCSC would again let ASTM take the lead on this.

j) New Business

(1) Meeting Location – Rowan University - June 7th and 8th (Larson) – Larson discussed plans already made for the annual meeting at Rowan University. It was agreed that RCSC

would charge a $100 fee for the meeting to help offset expenses. See below. Harrtold requested time at the meeting for task group meetings. It was agreed that this would be fit into the schedule. Larson gave a proposed schedule and tour information. Rich Brown is looking for sponsors.

(2) Meeting Rates and Expenses -(Larson)

(3) Website Move Plans -(Greenslade) Greenslade and Larson will move the RCSC website and domain to a new location.

(4) Marketing Ideas- (Larson)

(5) Dues Increase for GI to $100 - (Larson) This was tabled until the next meeting.

The meeting was adjourned.

Toby Anderson
Producer
A.1, A.4, B
Bay Bolt
4610 Malat Street
Oakland, CA 94601
Phone: 510-532-1188
Fax: 510-532-3947
e-mail: baybolt@pacbell.net

Abolhassan Astaneh-Asl
General Interest
A.1
University of California
781 Davis Hall
Berkeley, CA 94720-1710
Phone: 925-946-0903
Fax:
e-mail: astaneh@ce.berkeley.edu

Joseph G. Bahadrian
General Interest
3186 The Boulevard
Westmount, QC H3Y 1S3 Canada
Phone: 541-932-7339
Fax:
e-mail: bahadrian@sympatico.ca

Rodney L. Baxter
User
A.1
Schuff Steel Company
420 South 19th Avenue
Phoenix, AZ 85009
Phone: 602-417-8887
Fax: 602-256-0460
e-mail: rodney.baxter@schuff.com

Peter C. Birkemoe
General Interest
A.1, A.2
University of Toronto
35 St. George St., GB 231
Toronto, Ontario M5S 1A4 Canada
Phone: 416-232-2330
Fax: 416-978-7046
e-mail: pete.birkemoe@utoronto.ca

David W. Bogaty
General Interest
A.1, A.2
Spectra Tech., Inc.
5325 Hickory Hollow Road
Knoxville, TN 37919
Phone: 865-483-7210
Fax: 865-483-7262
e-mail: dbogaty@spectratechinc.com

David Bornstein
Producer
A.1
Skidmore Wilhelm
442 S. Green
South Euclid, OH 44121
Phone: 216-481-4774
Fax: 216-481-2427
e-mail: dbornstein@skidmore-wilhelm.com

Salim Brahimi
General Interest
IBECA Technologies, Corp.
4 Parkside Place
Montreal, QC H3H 1A8 Canada
Phone: 514-944-3358
Fax: 514-935-8918
e-mail: salimbrahimi@ibeca.ca

Richard C. Brown
Producer
A.1, A.2
TurnaSure LLC.
340 E. Maple Avenue
Suite 206
Langhorne, PA 19047
Phone: 215-750-1300
Fax: 215-750-6300
e-mail: rich.brown@turnasure.com

Frank Buck
Producer
Lindapter North America
3924A Varsity Drive
Ann Arbor, MI 48108
Phone: 888-724-2323
Fax: 734-677-2339
e-mail: fbuck@lindapterna.com

Bruce M. Butler
General Interest
Walt Disney Co.
10050 Honey Tree Court
Orlando, FL 32836
Phone: 407-824-6630
Fax: 407-824-7285
e-mail: bruce.butler@disney.com

Garret O. Byrne
General Interest
385 Copley Drive
Lancaster, PA 17601
Phone: 717-560-1971
Fax:
e-mail: garretbyrne@gmail.com

Charles J. Carter
Association
EX, A.4, B, A.3
AISC
One E. Wacker Drive
Suite 700
Chicago, IL 60601-1802
Phone: 312-670-5414
Fax: 312-896-9022
e-mail: carter@aisc.org

Helen Chen
Association
American Iron and Steel Institute
1140 Connecticut Avenue, NW
Washington, D.C. 20036
Phone: 202-452-7134
Fax: 202-452-1039
e-mail: hchen@steel.org

Robert J. Connor
General Interest
Purdue University-School of Civil Engineering
550 Stadium Mall Drive
West Lafayette, IN 47907-2051
Phone: 765-496-8272
Fax: 765-494-9886
e-mail: rconnor@purdue.edu

Bastiaan Cornelissen
General Interest
A.1, A.2
Structural Integrity Associates, Inc.
16154 Sandstone Drive
Morrison, CO 80465
Phone: 303-503-0411
Fax:
e-mail: bcornelissen@structint.com

Chris Curven
Producer
A.1, A.4
Applied Bolting Technology
1413 Rockingham Road
Bellows Falls, VT 05101
Phone: 802-460-3100
Fax: 802-460-3104
e-mail: chrisc@appliedbolting.com

Nick E. Deal General Interest A.1, A.2, A.4	828 Tulip Poplar Drive Birmingham, AL 35244-1671	Phone: 205-616-5734 Fax: 205-271-2482 e-mail: ndeal1140@aol.com
James M. Doyle General Interest	8778 Lawrenceburg Road Chaplin, KY 40012	Phone: 502-673-8778 Fax: 502-673-8778 e-mail: jmdoyle@bellsouth.net
Dean G. Droddy User A.1	National Steel City 14650 Jib Street Plymouth, MI 48170	Phone: 502-480-6068 Fax: e-mail: dean@nsc-us.com
Peter Dusicka General Interest	Portland State University Civil and Env. Eng. P.O. Box 751 Portland, OR 97207	Phone: 503-725-9558 Fax: 503-725-5950 e-mail: dusicka@pdx.edu
Matthew R. Eatherton General Interest A.2, A.4	Virginia Tech Patton Hall, Room 105D Blacksburg, VA 24061	Phone: Fax: e-mail: meather@vt.edu
Douglas B. Ferrell General Interest A.1	Ferrell Engineering, Inc. 15 Southlake Lane Suite 300 Birmingham, AL 35244	Phone: 205-879-2036 Fax: 205-879-5642 e-mail: doug.ferrell@ferrellengineering.com
John W. Fisher General Interest A.1	Lehigh University 117 Atlss Drive, H Bldg. Bethlehem, PA 18015-4729	Phone: 610-758-5537 Fax: 610-758-5553 e-mail: jwf2@lehigh.edu
Pat Fortney User A.1	Cives Steel Company 1825 Old Alabama Road Suite 200 Roswell, GA 30076	Phone: 678-925-1134 Fax: 678-287-3281 e-mail: pfortney@cives.com
Danilo M. Francisco General Interest	Tierra Nevada Subdivision Phase 6, Block 4, Lot 6 Bo. San Francisco General Trias, Cavite Philippines	Phone: 63-2-2511620 Fax: e-mail: danilo.francisco@aramco.com
Karl H. Frank User A.1, A.4	Hirschfeld Industries 5910 Courtyard Drive Suite 210 Austin, TX 78731	Phone: 325-486-4783 Fax: 325-486-4619 e-mail: karl.frank@hirschfeld.com
Michael C. Friel Producer	Haydon Bolts, Inc. 1181 Unity Street Philadelphia, PA 19124-3196	Phone: 215-537-8700 Fax: 215-537-5569 e-mail: mcfriel@haydonbolts.com
Bill Germuga Producer A.1	St. Louis Screw & Bolt 2000 Access Road Madison, IL 62060	Phone: 314-389-7500 Fax: 314-389-7510 e-mail: billg@stlouisscrewbolt.com
Jim Gialamas Producer A.1	Nucor Fastener Division PO Box 6100 6730 County Road 60 St. Joe, IN 46785	Phone: 260-337-1609 Fax: 260-337-1717 e-mail: james.gialamas@nucor-fastener.com
Rodney D. Gibble General Interest A.1	Rodney D. Gibble Consulting Engineers 19 West 21st Street Suite 501 New York, NY 10010	Phone: 212-989-2853 Fax: 212-989-4017 e-mail: rgibble@rdgengineers.com
Michael I. Gilmor General Interest A.1	24 Nadine Cre. Markham, ON L3R 7Y3 Canada	Phone: 905-479-0393 Fax: e-mail: migilmor@gilmor.ca
Joe Greenslade Association EX, A.1, A.2, A.3	Industrial Fasteners, Inst. 6363 Oak Tree Blvd. Independence, OH 44131-2500	Phone: 817-995-4685 Fax: e-mail: jgreenslade@indfast.org

Gilbert Y. Grondin General Interest A.2	University of Alberta Dept. of Civil Engineering Edmonton, AB T6G 2W2 Canada	Phone: 780-492-2794 Fax: 780-492-0249 e-mail: ggrondin@ualberta.ca
Jerome F. Hajjar General Interest	Northeastern University 360 Huntington Avenue Dept. of Civil and Env. Engineering Boston, MA 02115	Phone: 617-373-3242 Fax: 617-373-4419 e-mail: jf.hajjar@neu.edu
Allen J. Harrold User EX, A.1	BlueScope Building - North America P.O. Box 419917 Kansas City, MO 64141	Phone: 816-968-5719 Fax: 816-968-6512 e-mail: ajharrold@butlermfg.com
Robert A. Hay III General Interest	Flood Testing Laboratories 1945 E. 87th Street Chicago, IL 60617	Phone: 773-721-2200 Fax: 773-721-2206 e-mail: rahay@floodlabs.com
Todd Helwig General Interest A.1	University of Texas at Austin 10100 Burnet Road Building 177 Austin, TX 78758-4445	Phone: 512-232-2239 Fax: 512-471-1944 e-mail: thelwig@mail.utexas.edu
Ian C. Hodgson General Interest	Lehigh University - ATLSS Center 117 ATLSS Drive Bethlemem, PA 18015	Phone: 610-758-3293 Fax: 610-758-6840 e-mail: ich2@lehigh.edu
Charles E. Hundley Producer EX, A.1	Unytite, Inc. 1 Unytite Drive Peru, IL 61354-9710	Phone: 815-224-2221 Fax: 815-224-3434 e-mail: chundley@unytite.com
Kaushik A. Iyer General Interest	Exponent Inc. 17000 Science Drive Suite 200 Bowie, MD 20715	Phone: 301-291-2517 Fax: 301-291-2598 e-mail: kiyer@exponent.com
Emmanuel P. Jefferson General Interest A.4	The Hanna Group 1733 Waterstone Place San Ramon, CA 94582	Phone: 510-786-7965 Fax: 415-276-4784 e-mail: paul@hannagrp.com
Suja John General Interest	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: sjohn@cisc-icca.ca
Donald L. Johnson General Interest	Maus Engineering 10 Lary Road Wolfeboro, NH 03894-4121	Phone: 603-569-3337 Fax: e-mail: maus930@metrocast.net
Ronald B. Johnson General Interest	Skidmore, Owings & Merrill LLP 224 S. Michigan Ave. Chicago, IL 60604	Phone: 312-360-4088 Fax: 312-360-4553 e-mail: ronald.johnson@som.com
Charles J. Kanapicki User A.1	Fluor Enterprises, Inc. 375 Burma Road Oakland, CA 94607	Phone: 510-808-4609 Fax: 510-808-4601 e-mail: ckanapicki@abfjv.com
Peter F. Kasper Producer A.1	Ifastgroupe/Infasco/DSI P.O. Box 1452 St. Albans, VT 05478	Phone: 802-527-0341 Fax: 802-527-1087 e-mail: PKasper@ifastgroupe.com
Daniel J. Kaufman General Interest	One E. Wacker Drive Suite 700 Chicago, IL 60601-1802	Phone: 312-670-7523 Fax: e-mail: kaufman@aisc.org
James S. Kennedy General Interest A.1	4701 Fall Creek Drive San Angelo, TX 76904-7014	Phone: 325-227-9880 Fax: e-mail: jskthc@aol.com

Lawrence A. Kloiber
General Interest

LeJeune Steel
P.O.Box 18070
118 W. 60th Street
Minneapolis, MN 55419-0070

Phone: 612-861-3321
Fax: 612-861-2724
e-mail: larry.kloiber@lejeunesteel.us

Richard F. Knobloch
Life Member

KM Consulting
9704 W. Ridgeway Court
Columbus, IN 47201-9291

Phone: 812-342-3774
Fax: 812-342-9478
e-mail: rknobloch@comcast.net

Lawrence Kruth
User
A.1

Douglas Steel Fabricating Corp.
1312 South Waverly Road
Lansing, MI 48917

Phone: 517-999-4113
Fax: 517-853-8119
e-mail: lkruth@douglassteel.com

Geoffrey L. Kulak
General Interest
B, A.1, A.2

University of Alberta
Dept. of Civil Engineering
Edmonton, AB T6G 2W2 Canada

Phone: 780-492-5809
Fax: 780-492-0249
e-mail: geoff.kulak@ualberta.ca

Chad M. Larson
Distributor
EX, A.1, A.3, A.5

LeJeune Bolt Company
3500 West Highway 13
Burnsville, MN 55337-1795

Phone: 952-890-7700
Fax: 952-890-3544
e-mail: clarson@lejeunebolt.com

Bill R. Lindley II
User
A.1, A.4

W W Steel, LLC
1730 West Reno
Oklahoma City, OK 73106-3299

Phone: 405-297-7541
Fax: 405-236-4842
e-mail: blindley@wwsteel.com

Kenneth B. Lohr
Distributor
A.1, A.4

Lohr Fasteners
2355 Wilson Rd
Humble, TX 77396

Phone: 281-446-6766
Fax: 281-446-7805
e-mail: klohr@aol.com

Hussam N. Mahmoud
General Interest
A.2

Colorado State University
Dept. of Civil and Environmental Eng.
1372 Campus Delivery
Fort Collins, CO 80523-1372

Phone: 970-491-6605
Fax: 970-491-7727
e-mail: hussam.mahmoud@colostate.edu

Curtis L. Mayes
User
A.1

L.P.R. Construction
1171 Des Moines Avenue
Loveland, CO 80537

Phone: 970-203-2591
Fax: 970-203-2596
e-mail: cmayes@lprconstruction.com

Carly McGee
User
A.1

KTA-Tator, Inc.
115 Technology Drive
Pittsburgh, PA 15275

Phone: 412-788-1300
Fax: 412-788-1306
e-mail: cmcgee@kta.com

Jonathan C. McGormley
General Interest
A.4

Wiss, Janney, Elstner Associates
330 Pfingsten Road
Northbrook, IL 60062-2095

Phone: 847-272-7400
Fax: 847-291-4813
e-mail: jmcgormley@wje.com

David L. McKenzie
General Interest

SP International
1423 Swift Avenue
No. Kansas City, MO 64116

Phone: 816-421-6449
Fax: 816-421-1715
e-mail: dmckenzie@spintlinc.com

Neil L. McMillan
General Interest
A.1, A.5

AECOM
300 Water Street
Whitby, ON L1N 9J2

Phone: 905-668-9363
Fax:
e-mail: neil.mcmillan@aecom.com

Jinesh K. Mehta
General Interest
A.1

Alta Vista Solutions
6475 Christie Ave.
Suite 425
Emeryville, CA 94608

Phone: 510-594-0510
Fax:
e-mail: jineshkmehta@gmail.com

Kevin Menke
Distributor
A.1

Fastenal
20345 County Road 23
Winona, MN 55987

Phone: 507-453-8188
Fax: 507-494-6450
e-mail: kmenke@fastenal.com

Greg Miazga
User
EX, A.1, A.4

Waiward Steel
10030 34th St.
Edmonton, AB T6B 2Y5 Canada

Phone: 780-485-3971
Fax: 780-485-3975
e-mail: greg.miazga@waiward.com

Heath E. Mitchell General Interest A.1	AISC 1250 Pacific Avenue Suite 701 Tacoma, WA 98402	Phone: 312-515-1714 Fax: e-mail: mitchell@aisc.org
Eugene R. Mitchell General Interest A.1, A.4	P.O. Box 282 Greenfield, NH 03047-0182	Phone: 603-562-9051 Fax: 603-547-3801 e-mail: mitch999@comcast.net
Scott Munter Association	Australian Steel Institute P.O. Box 6366 North Sydney, NSW 2059 Australia	Phone: 61-2-9929-6666 Fax: 61-2-9955-5406 e-mail: scottm@steel.org.au
Thomas M. Murray General Interest B	537 Wisteria Drive Radford, VA 24141	Phone: 540-731-3330 Fax: e-mail: thmurray@vt.edu
Gian A. Rassati General Interest A.1	University of Cincinnati 765 Baldwin Hall Cincinnati, OH 45221-0071	Phone: 513-556-3696 Fax: 513-556-2599 e-mail: gian.rassati@uc.edu
James M. Ricles General Interest EX, A.2	Lehigh University - ATLSS Center 117 ATLSS Drive Bethlemem, PA 18015-4729	Phone: 610-758-6252 Fax: 610-758-5902 e-mail: jmr5@lehigh.edu
Thomas J. Schlafly General Interest A.1, A.2	AISC One E. Wacker Drive Suite 700 Chicago, IL 60601-1802	Phone: 312-670-5412 Fax: e-mail: schlafly@aisc.org
Gerald E. Schroeder General Interest A.1	Fish & Associates, Inc. H. N398 HWY 58 LaValle, WI 53941	Phone: 608-985-7713 Fax: e-mail: gschroeder@mwt.net
David F. Sharp General Interest	GMS Engineers, LLP 129 W. 27th Street New York, NY 10001	Phone: 212-254-0030 Fax: 212-477-5978 e-mail: david.sharp@gmsllp.com
Robert E. Shaw Jr. General Interest A.1, B, A.4	Steel Structures Technology Center 5277 Leelanau Ct. Howell, MI 48843	Phone: 734-878-9560 Fax: 734-878-9571 e-mail: rshaw@steelstructures.com
Victor Shneur User A.1	LeJeune Steel Co. 118 West 60th Street Minneapolis, MN 55419-0070	Phone: 612-243-2358 Fax: 612-861-2724 e-mail: victor.shneur@lejeunesteel.us
W. Lee Shoemaker Association A.1	Metal Building Manufacturers Assoc. 1300 Sumner Avenue Cleveland, OH 44115-2851	Phone: 216-241-7333 Fax: 216-241-0105 e-mail: lshoemaker@mbma.com
James A. Swanson General Interest EX, A.1	University of Cincinnati P.O. Box 210071 765 Baldwin Hall Cincinnati, OH 45221-0071	Phone: 513-556-3774 Fax: 513-556-2599 e-mail: james.swanson@uc.edu
Arun A. Syam General Interest	Australian Tube Mills P.O. Box 246 Sunnybank, Queensland 4109 Australia	Phone: 61-7-3246-6600 Fax: 61-7-3246-6660 e-mail: aruns@austubemills.com
Thomas S. Tarpy Jr. General Interest A.1	Stanley D. Lindsey & Assoc. 5500 Maryland Way Suite 250 Brentwood, TN 37027	Phone: 615-320-1735 Fax: 615-320-0387 e-mail: ttarpy@sdl-nash.com
William A. Thornton User A.1, A.2	Cives Steel Company 1825 Old Alabama Road #200 Roswell, GA 30076-2201	Phone: 678-287-3241 Fax: 678-287-3281 e-mail: bthornton@cives.com

Raymond H.R. Tide
General Interest
A.1, A.3

Wiss, Janney, Elstner Assoc.
330 Pfingsten Road
Northbrook, IL 60062-2095

Phone: 847-272-7400
Fax: 847-291-4813
e-mail: rtide@wje.com

Brad Tinney
Producer

Birmingham Fastener
931 Avenue W
Birmingham, AL 35214

Phone: 205-595-3511
Fax: 205-591-0244
e-mail: brad.tinney@bhamfast.com

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

Amit H. Varma
General Interest

Purdue University
3363 Humbolt Street
West Lafayette, IN 47906

Phone: 765-496-3419
Fax: 765-496-1105
e-mail: ahvarma@purdue.edu

Floyd J. Vissat
General Interest
EX, A.1

URS
7800 East Union Avenue
Denver, CO 80237

Phone: 303-843-2079
Fax: 303-843-2684
e-mail: floyd.vissat@urs.com

Wayne Wallace
General Interest

Applied Bolting Technology
1413 Rockingham Road
Bellows Falls, VT 05101

Phone: 802-460-3100
Fax: 802-460-3104
e-mail: waynew@appliedbolting.com

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

boltcouncil
RESEARCH COUNCIL ON STRUCTURAL CONNECTIONS
rcsc

- HOME
- RESEARCH PROGRAMS
- DOCUMENTS
- MEMBERSHIP
- MEETING INFORMATION
- MEETING MINUTES
- BYLAWS AND EXECUTIVE COMMITTEE
- MEMBER LINKS

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

Portland State UNIVERSITY

Research Council on Structural Connections
7269 S Sundown Circle
Littleton, CO 80120

Invoice Number: 421675-2
Billing Period:
From:
To: Final Invoice

Award Number:

PI Name: Peter Dusicka

Project Title:
Effect of Fillers on Steel Girder Field Splice Performance

	Previously Billed Amount	Current Amount Due	Total Billed To-Date
Total	$60,000.00	$20,000.00	$80,000.00
Subtotal: Total	$60,000.00	$20,000.00	$80,000.00
Grant Total	**$60,000.00**	**$20,000.00**	**$80,000.00**

Please send payment to:

Portland State University
Research Accounting Office
PO Box 751 (BORA)
Portland, OR 97207-0751

Current Bill Amount:	$20,000.00
Past-Due Amount:	$0.00
Total Amount Due:	20000

Cost Share:
Current: *Cumulative:*

FINAL BILL

Huong Mai

SIGNATURE

18-MAY-12

DATE SUBMITTED

Huong Mai
Research Accounting

TYPED OR PRINTED NAME AND TITLE

PHONE NUMBER

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

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**

12. NAME	13. ROLE IN THIS CONTRACT	14. YEARS EXPERIENCE	
		a. TOTAL	b. WITH CURRENT FIRM
Aaron Prchlik, P.E.	Structural Material Representative	8	1

15. FIRM NAME AND LOCATION (City and State)

Alta Vista Solutions, Inc., Emeryville, CA

16. EDUCATION *(DEGREE AND SPECIALIZATION)*	17. CURRENT PROFESSIONAL REGISTRATION (STATE AND DISCIPLINE)
BSc, Civil & Environmental Engineering Univ. of Michigan-Ann Arbor Caltrans Training: •Introduction to Bridge Design •Fall Protection •Field Engineer Training •Confined Space •Retaining Wall-Seismic Design •Confined Space-Training for Trainers •Marine Construction Safety •Basic Surveying for Construction •Trenching and Shoring •Caltrans Building Construction •Introduction to Microstation	Professional Engineer, California, Civil (# 77562)

18. OTHER PROFESSIONAL QUALIFICATIONS *(Publications, Organizations, Training, Awards, Etc.)*

Mr. Prchlik is a Professional Engineer with over 8 years of experience in Civil Engineering and over 4 years of experience working with the California Department of Transportation (Caltrans) on projects both domestically and internationally as part of the China SAS Bridge Team. As Assistant SR, he performed various duties such as Field Testing and Reviews on Concrete, Welding, and SAS painting. He also reviewed RFIs and submittals for the SAS Tower Fabrication. Mr. Prchlik' s background encompasses advanced skills and knowledge of Structural Joints and high strength A325 and A490 Bolts, SSTC-Steel Structures Bolting Handbook, AWS D1.5 Bridge Welding Code, Caltrans Standard Specifications, AISC Steel Construction Manual, AASHTO LRFD Bridge Construction Specification, AutoCAD, Microstation, Microsoft Project, Primavera, ArcGIS, RISA, Solidworks, and 3Ds Max7. Mr. Prchlik has worked extensively with Caltrans Safety Manual and Code of Safe Practices as well as Bridge Construction records and Procedures (CRAP) Manual. He has assisted with review and response to RFIs and other submittals and with evaluation of CCOs, progress documents and pay estimates.

- Advance Knowledge of SAS Fabrication and Erection
- Experience in Heavy Transportation Projects
- International Fabrication Experience
- Experienced in Review of Submittals & RFIs
- Advanced Knowledge of
  - HS Bolting
  - AWS D1.5 Bridge Welding Code,
  - Caltrans Standard Specifications,
  - AISC Steel Construction,
  - AASHTO LRFD Bridge Construction Specification

19. RELEVANT PROJECTS

	(1) TITLE AND LOCATION (City and State)	(2) YEAR COMPLETED	
a.		PROFESSIONAL SERVICES	CONSTRUCTION (if applicable)
	Structural Material and Testing Inspection Services (METS), California Department of Transportation, Statewide, CA	Ongoing	Ongoing
	(3) BRIEF DESCRIPTION (Brief scope, size, cost, etc.) AND SPECIFIC ROLE	X	Check if project performed with current firm
	Structural Materials Representative - Mr. Prchlik is SMR for numerous Caltrans construction projects in D4. His duties include reviewing contractor submittals, structural material evaluations, and material quality management. He oversees quality assurance inspections for structural bridge component including steel and precast girders, columns and piles and other bridge components as well as signal and lighting poles. He also assists in the preparation of CCOs, identifies accurate labor and materials for the contract pay estimates, and develops detailed reports in responses to contractor claims.		
b.	(1) TITLE AND LOCATION (City and State)	(2) YEAR COMPLETED	
		PROFESSIONAL SERVICES	CONSTRUCTION (if applicable)
	SFOBB, Self-Anchored Suspension (SAS) Bridge, Oakland, CA	2007	2011
	(3) BRIEF DESCRIPTION (Brief scope, size, cost, etc.) AND SPECIFIC ROLE		Check if project performed with current firm
	Assistant Structure Representative - Mr. Prchlik reviewed RFIs and submittals for SAS-Tower and Orthotropic Box girder fabrication and worked closely with the Designer and METS personnel. He assisted METS personnel with development and implementation of a dimensional control plan. Mr. Prchlik inspected bolting materials and reviewed critical weld repair submittals and was responsible for final approval on all SAS painting issues and submittals. He worked closely with NACE-certified paint inspectors. He was also the designated Safety Officer for Team China and ensured that proper safety procedures including confined space training and proper use of safety equipment were enforced.		
c.	(1) TITLE AND LOCATION (City and State)	(2) YEAR COMPLETED	
		PROFESSIONAL SERVICES	CONSTRUCTION (if applicable)
	E2-T1 Foundation Project , California Department of Transportation (Caltrans), Oakland, CA	2008	2008
	(3) BRIEF DESCRIPTION (Brief scope, size, cost, etc.) AND SPECIFIC ROLE		Check if project performed with current firm
	Assistant Structure Representative - Mr. Prchlik reviewed mix designs for conventional concrete and self-consolidating concrete along with performing field concrete testing, design review of concrete formwork, and prepared concrete cylinders for QA testing. He also performed field review of contractor-submitted RFIs and CCOs, and provided solutions to ensure quality and timeliness of any change orders. He performed inspection of concrete operations at the jobsite and ensured all rebar was placed in accordance with the plans and specifications and reviewed and approved critical weld repairs during the welding of the T1 footing box.		

Alta Vista Solutions

Aaron J. Prchlik, P.E.

Civil Engineer

Education

BSc, Civil and Environmental Engineering

University of Michigan-Ann Arbor

Registration

Professional Engineer, California, Civil (77562)

Years in Practice

8 years

EXPERIENCE SUMMARY

Mr. Prchlik is a Professional Engineer with over 8 years of experience in Civil Engineering and over 4 years of experience working with the California Department of Transportation (Caltrans) on projects both domestically and internationally as part of the China SAS Bridge Team. He performed various duties as the Transportation Engineer for Caltrans such as Field Testing and Reviews on Concrete, Welding, and all SAS painting. He was the engineer in charge of reviewing RFIs and submittals for the SAS Tower Fabrication. Mr. Prchlik's background encompasses advanced skills and knowledge of RCSC Specification for Structural Joints using high strength A325 or A490 Bolts, SSTC-Steel Structures Bolting Handbook, AWS D1.5 Bridge Welding Code, Caltrans Standard Specifications, AISC Steel Construction Manual-13th Edition, AASHTO LRFD Bridge Construction Specification, AutoCAD, Microstation, Microsoft Project, Primavera, ArcGIS, RISA, Solidworks, and 3Ds Max7. Mr. Prchlik has worked extensively with Caltrans Safety Manual and Code of Safe Practices as well as Bridge Construction records and Procedures Manual. He has assisted with review and response to RFIs and other submittals and with evaluation of CCOs, progress documents and pay estimates.

RELEVANT PROJECT EXPERIENCE

San Francisco – Oakland Bay Bridge, Self-Anchor Suspension (SAS) Bridge-SAS Bridge Team Tower & Team China-Orthotropic Box Girders (Caltrans), Oakland, CA

Assistant Structure Representative, Mr. Prchlik reviewed RFIs and submittals for SAS-Tower and Orthotropic Box girder fabrication and worked closely with the Designer and METS personnel. He assisted METS personnel with development and implementation of a dimensional control plan. Mr. Prchlik inspected bolting installation and materials testing. He also reviewed critical weld repair submittals and was responsible for final approval on all SAS painting issues and submittals. He worked closely with NACE-certified paint inspectors. He was also the designated Safety Officer for Team China and ensured that proper safety procedures including confined space training and proper use of safety equipment were enforced.

E2-T1 Foundation Project (Caltrans), Oakland, CA

Assistant Structure Representative, Mr. Prchlik reviewed mix designs for conventional concrete and self-consolidating concrete along with performing field concrete testing, design review of concrete formwork, and prepared concrete cylinders for QA testing . He also performed field review of contractor-submitted RFIs and CCOs, and provided solutions to ensure quality and timeliness of any change orders. He performed inspection of concrete operations at the jobsite and ensured all rebar was placed in accordance with the plans and specifications and reviewed and approved critical weld repairs during the welding of the T1 footing box.

University of Michigan Construction Management – Renovation Projects, Ann Arbor, MI

Assistant Project Manager, Mr. Prchlik was responsible for conducting design and specification reviews for a large scale museum renovation. He coordinated several different contractors involving a several phase medical school renovation including reviewing contract change orders, project schedule review, and provided quality assurance.

RCSC

RESEARCH COUNCIL ON STRUCTURAL CONNECTIONS

APPLICATION FOR MEMBERSHIP

1. Full name: Aaron Prchlik Position: Structural Materials Representative

   Organization: Alta Vista Solutions

   Street Address: 6475 Christie Ave Suite 425 Res. ☐ or Bus. ☑

   City, State, Zip: Emeryville CA 94608

   Tel: 510-610-9822 Fax: 510-594-0511 E-mail: aprchlik@altavistasolutions.com

2. If applicable, organization representative you will replace: ____________

3. Voting privilege requested : ☑ Voting ☐ Non-voting

4. Voting classification requested: check appropriate category

   ☐ Association ☐ Distributor ☑ General Interest ☐ Producer ☐ User

5. Principle product or service offered by your organization: Quality management

6. Technical background: Attach relevant documents indicating interests and resume.

7. Particular committee preference, if any ____________

8. Does your organization have laboratory facilities or other resources which might be available for cooperative Council activities? No

9. Will applicant contribute to the work of the Council by attending meetings regularly and/or by correspondence and response to letter ballots? Yes

10. Comments, if any: I assisted with the organization of last years meeting in San Francisco.I have been involved with all bolting operations on the San Francisco Oakland Bay Bridge for Several years and have had contact with many members of the commitee

Applicant's signature Aaron Prchlik Digitally signed by Aaron Prchlik DN: cn=Aaron Prchlik, o, ou, email=aprchlik@altavistasolutions.com, c=US Date: 2012.05.10 16:21:20 -07'00' Date 5-10-12

FOR EXECUTIVE COMMITTEE USE ONLY

Accepted for membership by: ____________ Date: ____________

286453

Click to E-mail

RESEARCH COUNCIL ON STRUCTURAL CONNECTIONS

Background and Scope

The Research Council on Riveted and Bolted Structural Joints was established in January 1947 to provide a non-profit forum of individuals with special expertise and interest in the technology of and to advance the state of the art of civil engineering structural connections using threaded fasteners and rivets. In 1979, the name of the Council was changed to the Research Council on Structural Connections to more accurately reflect the scope of the Council's attention to all aspects of mechanically fastened structural connections and in recognition of the diminished importance of rivets as a fastener for structural connections.

The purpose of the Research Council on Structural Connections has remained unchanged since its first establishment, namely, to stimulate and support such investigations as may be deemed necessary and valuable to determine the suitability and capacity of various types of mechanically fastened structural connections, to promote the knowledge of reliable design criteria, to promote the use of reliable economical and efficient practices in the fabrication and assembly of such structural connections, and to prepare and publish related standards, educational bulletins and other documents necessary to achieving its purpose.

Committee Activities:

The Council accomplishes its purposes by assigning responsibility for specific interest topics, as they arise, to a committee for study and development of recommendations for action by the full Council. Over the years 34 different committees have been active. Many have completed their assignment and been dismissed. Beginning in 2002 the committee structure has been revised to consist of Task Groups under each of the five main committees. The current main committees are:

A.1 - Specifications	A.2 - Research	A.3 - Membership and Funding
A.4 - Education	A.5 - Organization Liaison	B. - Editorial

Types of Membership

The membership of the Council is composed of organizational members and individual members.

Organizational members are corporations, trade associations, technical associations or similar groups who make financial contributions of a minimum of $500 per year to the operation of the Council. Each organizational member shall designate one official representative to exercises the rights and responsibilities of that membership. Each organizational member may have more than one representative on the Council so long as those representatives represent different classifications of interest and responsibility within that organization.

Individual members are persons who apply for membership, and after acceptance, continue to fulfill the obligations of members as provided by the Bylaws.

Each organization and individual applying for membership shall furnish a completed application form containing information on their qualifications to serve and including information permitting the Executive Committee to determine the voting classification of the applicant.

The application of an organization or individual shall be accepted unless (1) acceptance will create an imbalance of voting interests as described below of (2) the applicant is not qualified or knowledgeable, in the area of the Council's purposes.

Voting Classification of Members:

Each voting member shall be classified by the Executive Committee according to interest as either Producer, Distributor, User, or General Interest. The combined number of members classified as Producers and Distributors are required to never exceed the sum of members classified as Users and General Interest.

A Producer member is a member who is employed by or represents an organization that produces non-fabricated structural steel material, fasteners, tools, coating systems, test devices or other similar materials or articles used in structural connections.

A Distributor member is a member who is employed by or represents an organization that distributes structural bolts, tools, coating systems, test devices or other similar materials or articles used in structural connections. A Distributor member is grouped with Producers for membership balance.

A User member (construction firm/fabricator/erector) is a member who is employed by or represents an organization that installs, or purchases or uses non-fabricated structural steel material, fasteners, tools, coating systems, test devices or other similar materials or articles used in structural connections, provided that the member could not also be classified as a producer

An Association Member is an Organizational member who is employed by, or represents a technical or non-technical association that is neither Producer, Distributor or a User of non-fabricated structural steel material, fasteners, tools, coating systems, test devices or other similar materials or articles used in structural connections.

A General Interest member is neither a Producer nor a Distributor nor a User nor an Association member as defined above.

A Life Member, is a long time member of the Council who has distinguished himself/herself by their contributions to the Council but is no longer active. Nominated Life Member recommendations by five members will be reviewed and acted upon by the Executive Committee.

A Consultant, retained under an indefinitely continuing arrangement with an organization, shall be classified the same as the classification of the organization by which the Consultant is retained. Consultants representing themselves or their employers but not engaged in the production or sale of the product or services of the Producer, Distributor or User, as applicable, shall be classified as User or General Interest, as appropriate.

Persons who are liaison representatives from other organizations shall be non-voting members unless they are also approved separately as voting members as described above.

Submission of Applications:

Transmit applications for membership to the RCSC chairman:

Charlie Carter
AISC, 1 East Wacker Dr., Suite 700, Chicago, IL 60601
312/670-5414
carter@aisc.org

RCSC

RESEARCH COUNCIL ON STRUCTURAL CONNECTIONS

APPLICATION FOR MEMBERSHIP

1. Full name: Darlene Auer Position: Account Mgr

   Organization: NOF Metal Coatings

   Street Address: 26877 Northwestern Hwy. Res. ☐ or Bus. ☑

   City, State, Zip: Southfield, MI 48033

   Tel: 248-763-1064 Fax: 440-279-1480 E-mail: darlene-auer@nofmetalcoatings.com

2. If applicable, organization representative you will replace: ____________

3. Voting privilege requested : ☑ Voting ☐ Non-voting

4. Voting classification requested: check appropriate category

   ☐ Association ☐ Distributor ☑ General Interest ☐ Producer ☐ User

5. Principle product or service offered by your organization:

   Corrosion Resistant Coatings

6. Technical background: Attach relevant documents indicating interests and resume.

7. Particular committee preference, if any ____________

8. Does your organization have laboratory facilities or other resources which might be available for cooperative Council activities? Yes

9. Will applicant contribute to the work of the Council by attending meetings regularly and/or by correspondence and response to letter ballots? Yes

10. Comments, if any: ____________

Applicant's signature Darlene R. Auer Date 2/27/12

FOR EXECUTIVE COMMITTEE USE ONLY

Accepted for membership by: ____________ Date: ____________

286453

Click to E-mail

RCSC

RESEARCH COUNCIL ON STRUCTURAL CONNECTIONS

APPLICATION FOR MEMBERSHIP

1. Full name: Darlene Auer Position: Account Mgr

   Organization: NOF Metal Coatings

   Street Address: 26877 Northwestern Hwy. Res. ☐ or Bus. ☑

   City, State, Zip: Southfield, MI 48033

   Tel: 248-763-1064 Fax: 440-279-1480 E-mail: darlene-auer@nofmetalcoatings.com

2. If applicable, organization representative you will replace: ______

3. Voting privilege requested : ☑ Voting ☐ Non-voting

4. Voting classification requested: check appropriate category

   ☐ Association ☐ Distributor ☑ General Interest ☐ Producer ☐ User

5. Principle product or service offered by your organization: ______

   Corrosion Resistant Coatings

6. Technical background: Attach relevant documents indicating interests and resume.

7. Particular committee preference, if any ______

8. Does your organization have laboratory facilities or other resources which might be available for cooperative Council activities? Yes

9. Will applicant contribute to the work of the Council by attending meetings regularly and/or by correspondence and response to letter ballots? Yes

10. Comments, if any: ______

Applicant's signature Darlene R. Auer Date 2/27/12

FOR EXECUTIVE COMMITTEE USE ONLY

Accepted for membership by: ______ Date: ______

286453

Click to E-mail

***infra*Structure Testing & Applied Research (*i*STAR) Laboratory**
Department of Civil and Environmental Engineering
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

Peter Dusicka, Ph.D., P.E., Associate Professor
Department of Civil and Environmental Engineering, Portland State University
P.O. Box 751, Portland, OR 97207
email: dusicka@pdx.edu, phone: (503) 725-9558

Gregory Lewis and Cole Smith, Research Assistants
Department of Civil and Environmental Engineering, Portland State University
P.O. Box 751, Portland, OR 97207

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.

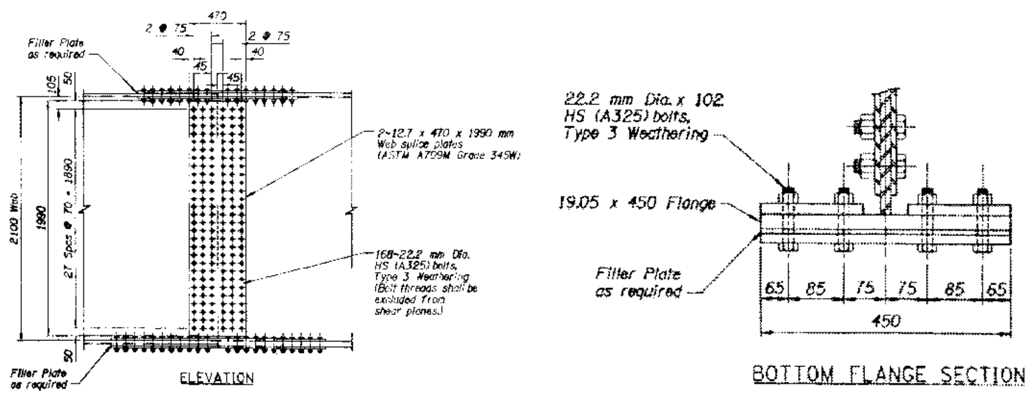
Comparisons of the onset of slip did not indicate reduction in slip coefficient with larger fillers or with presence of multiple fillers. The slip values themselves had been recorded to correlate well to those recorded in phase 1, but were found to be lower than expected for class B surface. A slip study 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.

TABLE OF CONTENTS

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.

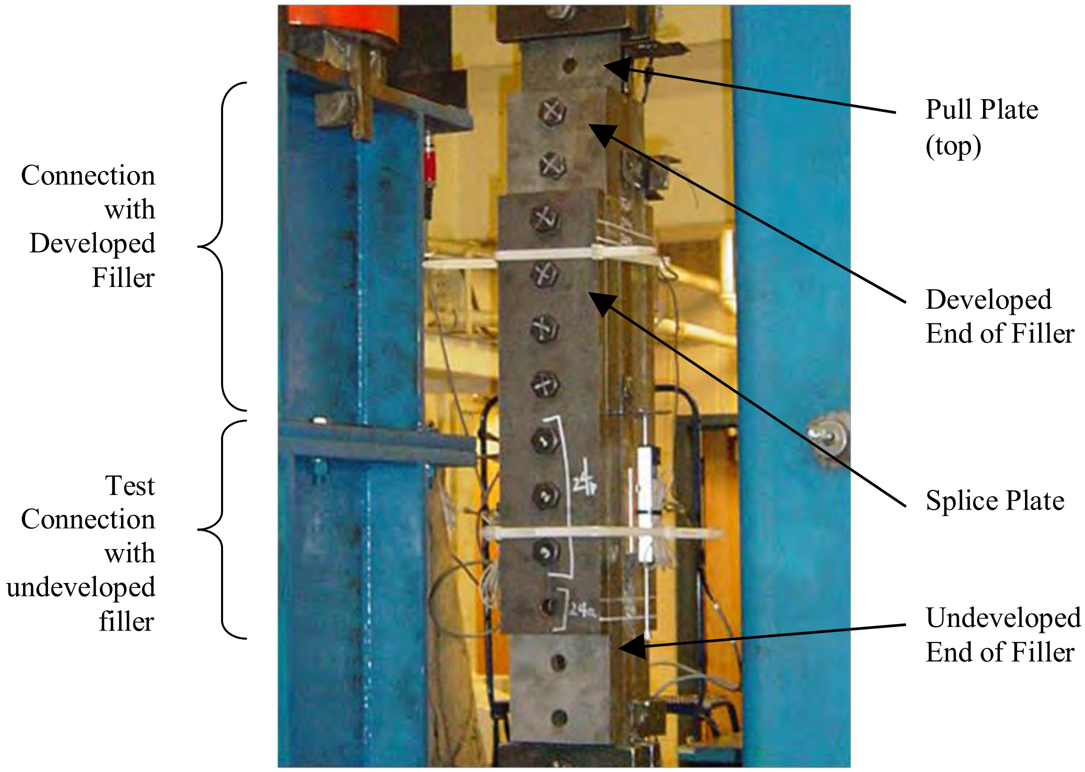
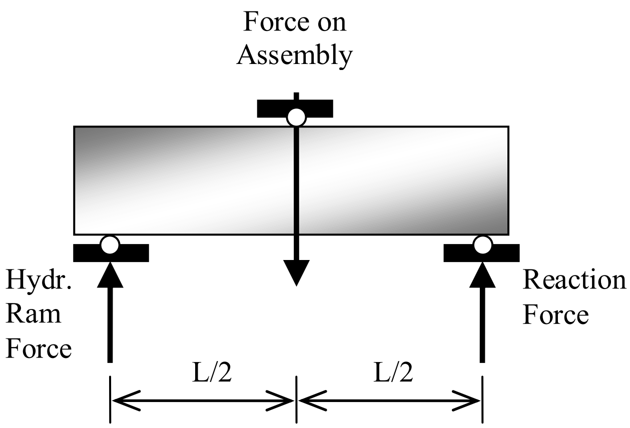


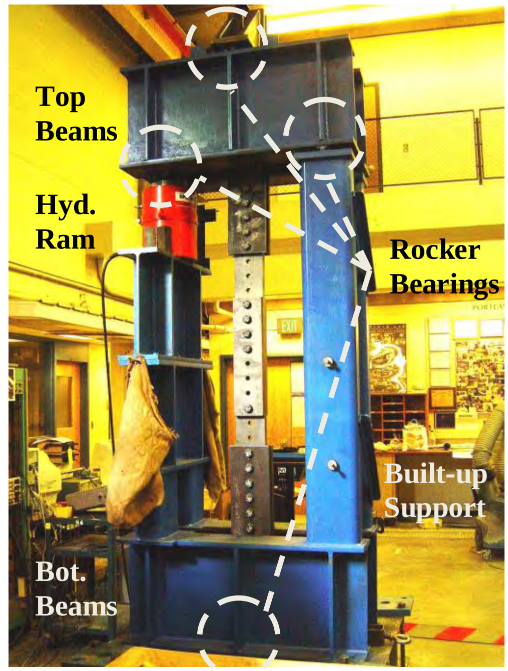
Figure 2: Typical Bolted Assembly Layout

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.

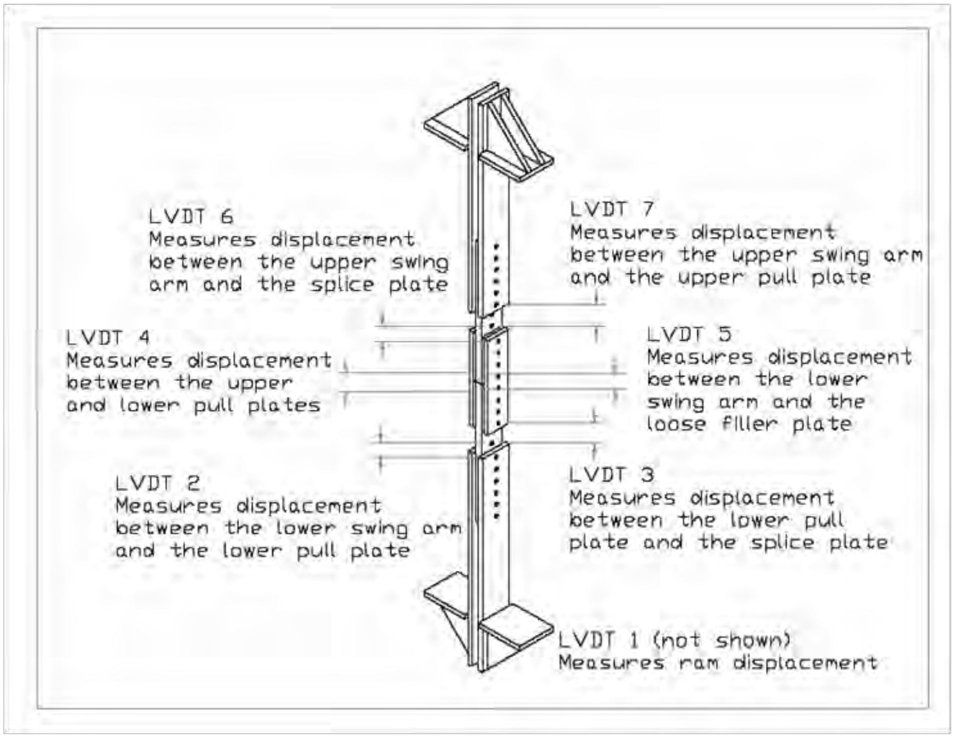


Figure 4: Schematic of Displacement Transducer Measurements

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.

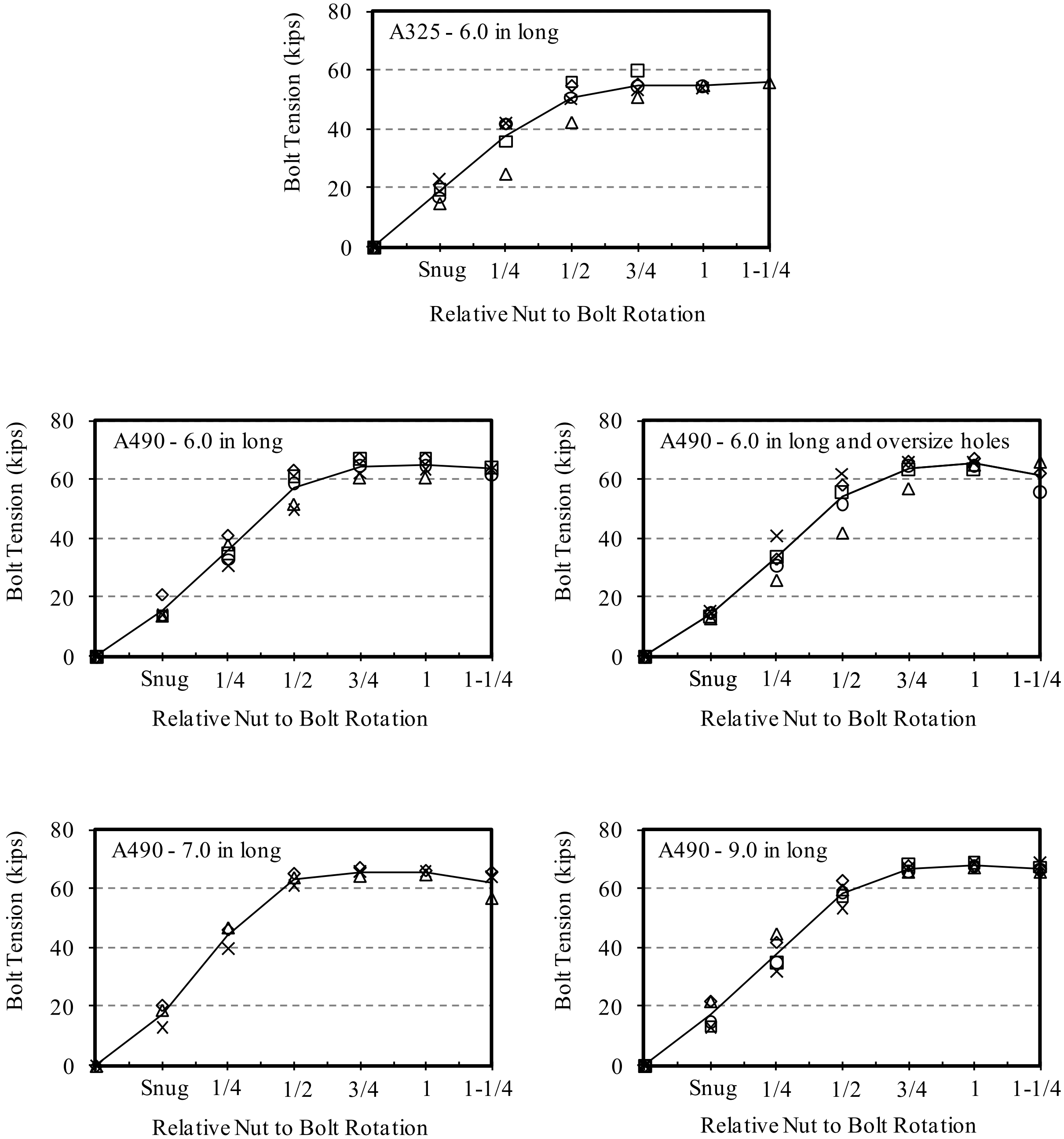


Figure 6: Achieved Tension in Bolts

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.

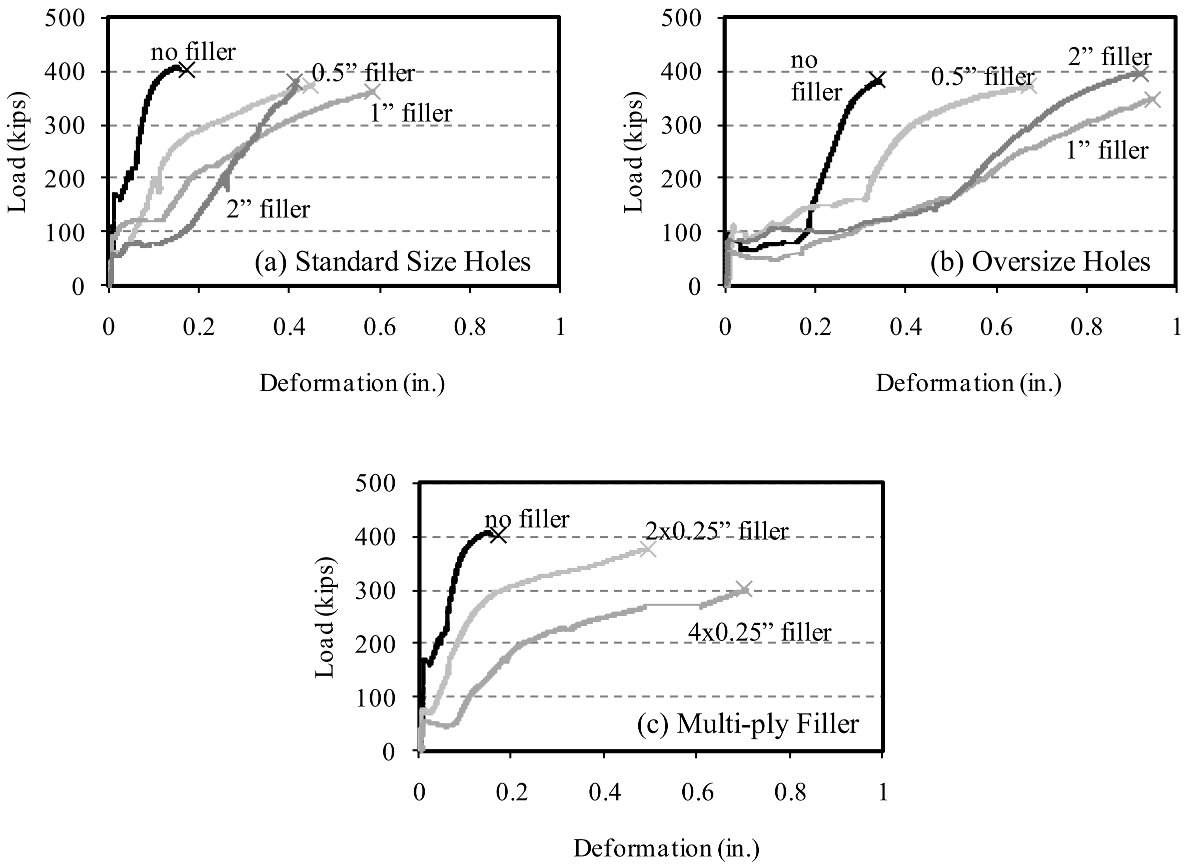


Figure 7: Examples of Three-bolt Assembly Force Deformation Results

Figure 8: Sample Deformed Bolts upon Removal From Connection Assembly

4.2 Ultimate Strength

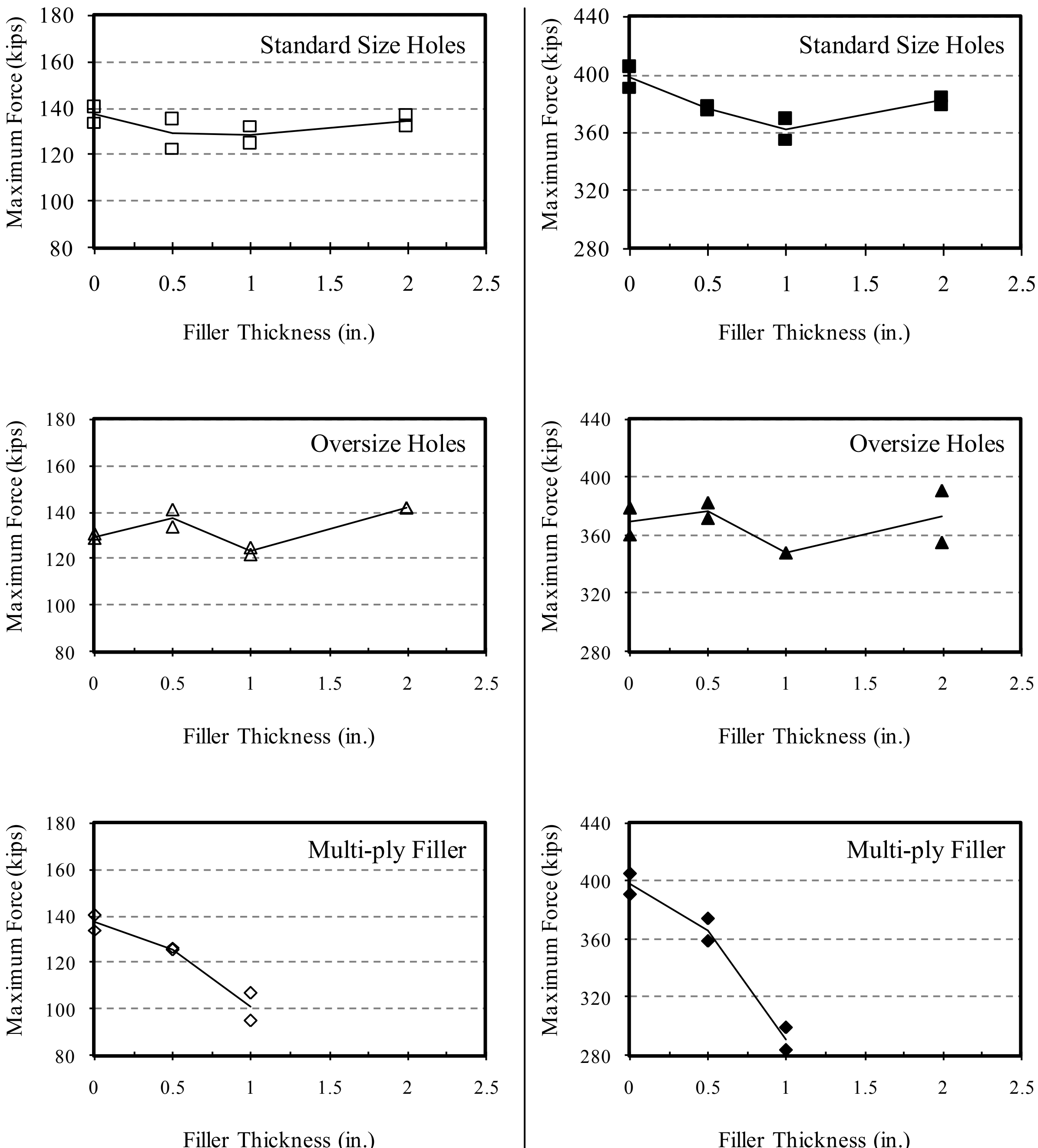
The maximum load recorded in each test was summarized in Figure 9 for the single bolt and three-bolt assemblies. The discrete data represent the recorded ultimate strengths, while the continuous line represents the average values for increasing filler thickness. The strengths from the one bolt assembly 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

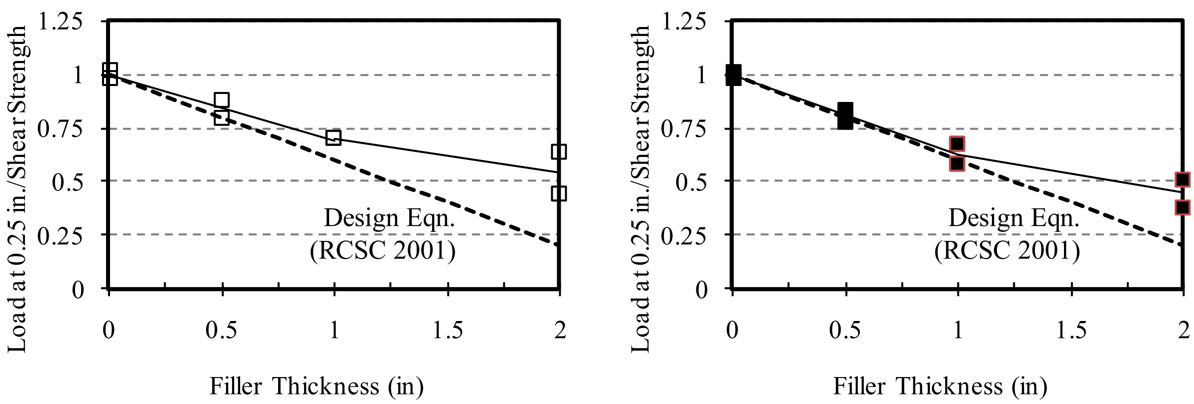
(b) Three Bolt Assembly

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.

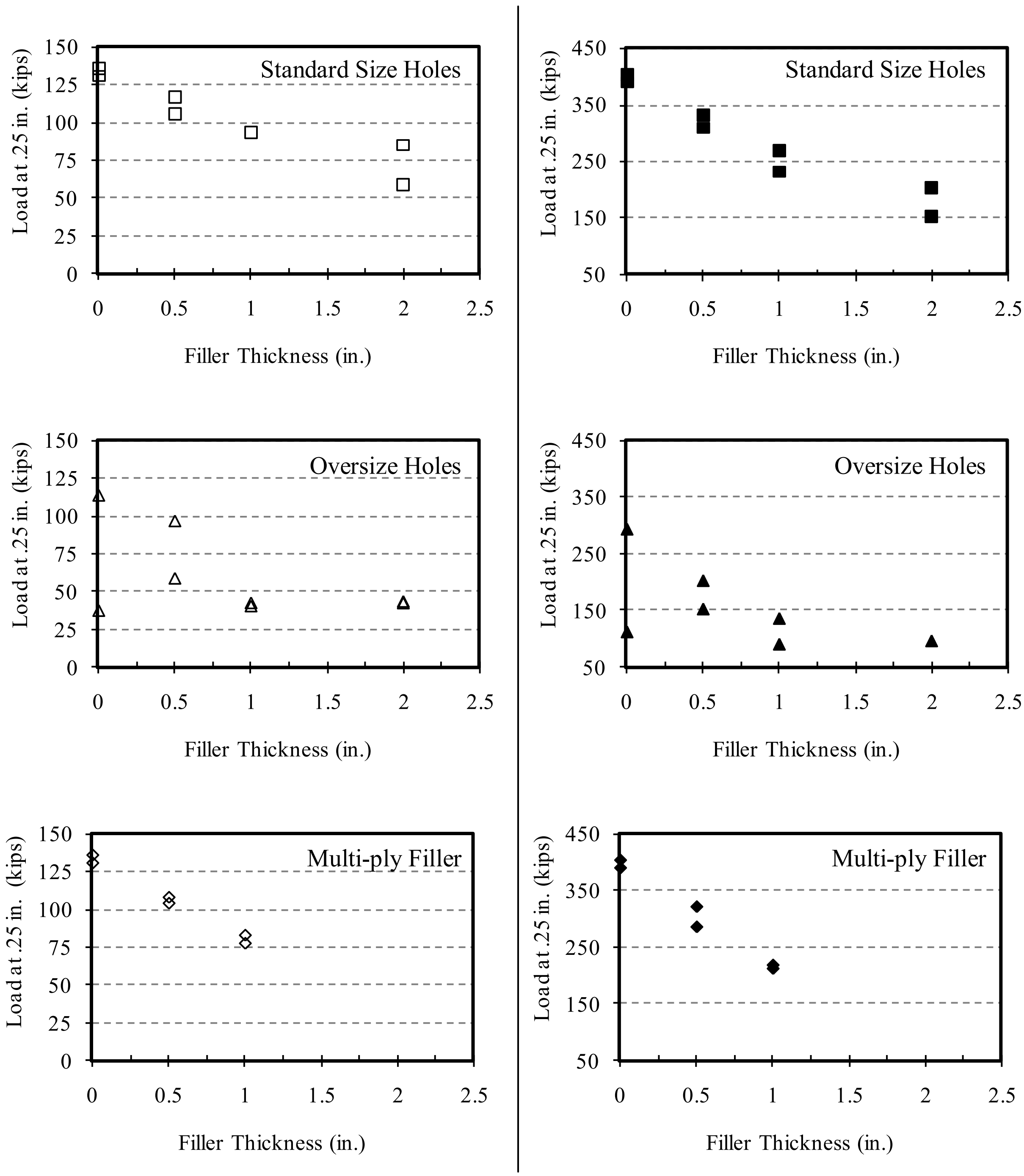
Current design guidelines for connections with fillers are primarily based on the limited dataset available in which the load being resisted at a deformation limit of 1/4" was used to establish the shear strength reduction design equations discussed in Section 1.3 (Yura et al 1982). This limit was chosen 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

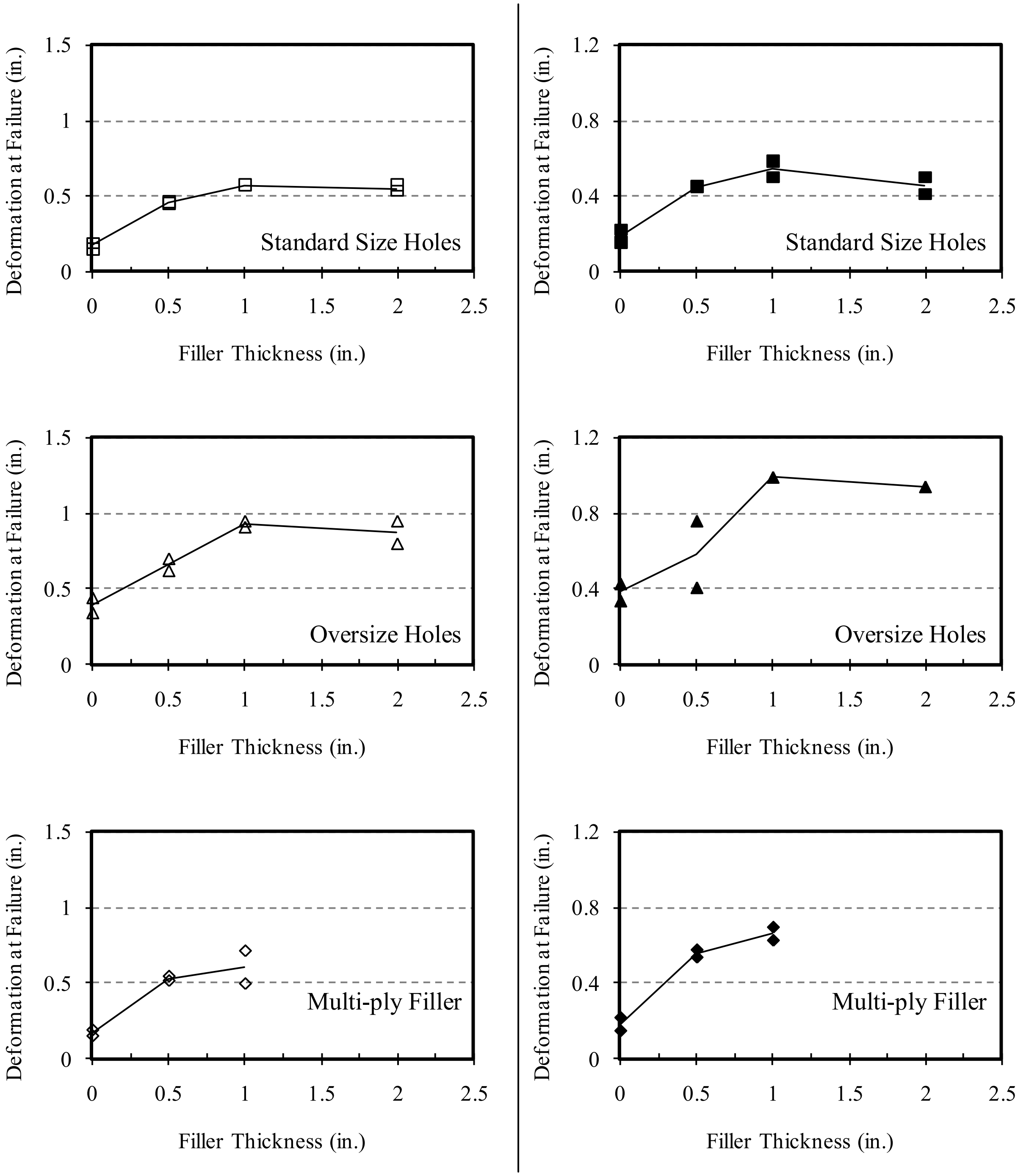
Figure 11: Resistance at 0.25 in Deformation

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.



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

Figure 12: Connection Deformation at Failure

4.4 Slip Resistance

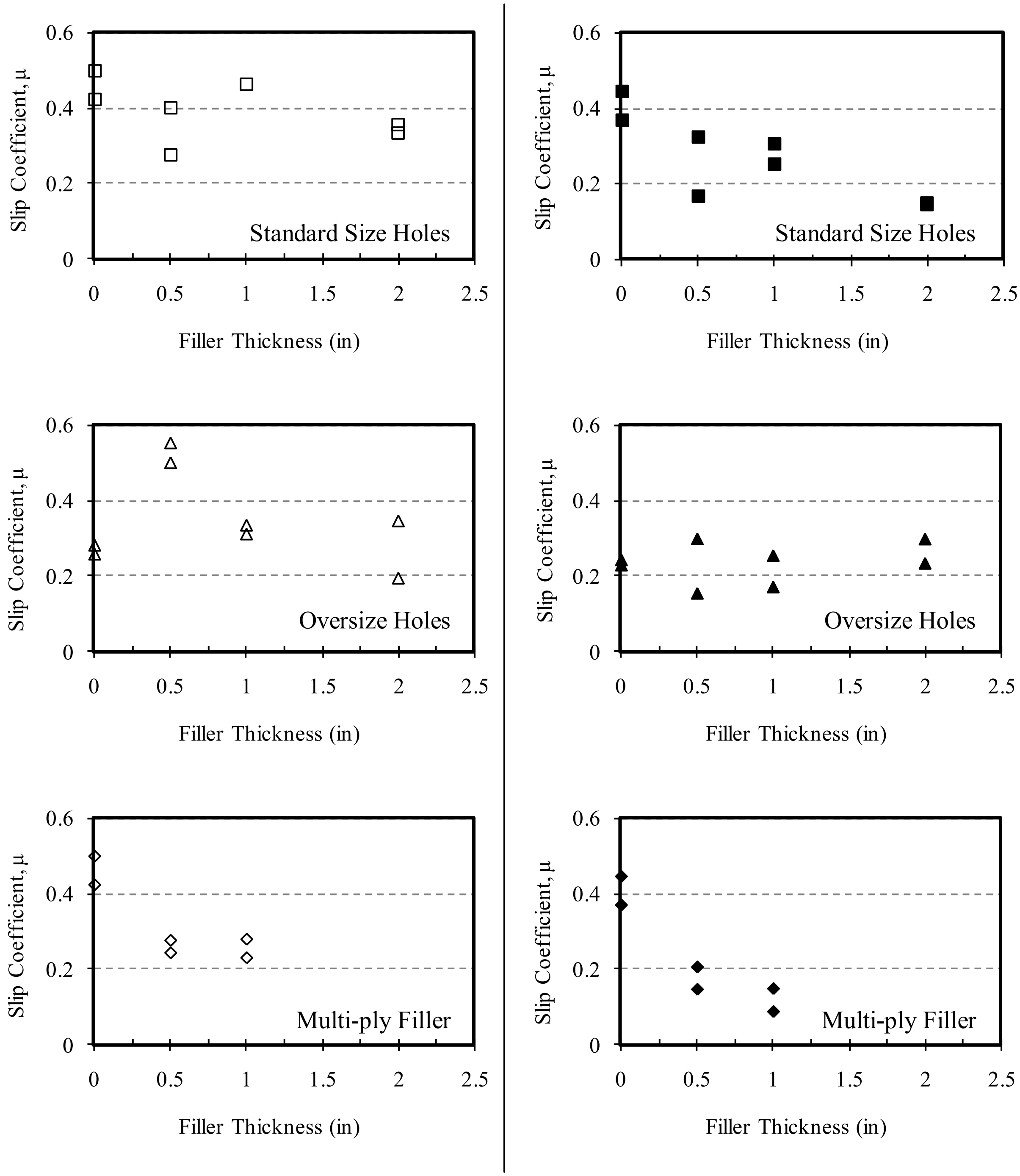
Slip critical joints rely on the slip resistance between the surfaces of the plates rather than the shear strength of the bolts in the connection. The slip was identified by the displacement transducer measuring deformation of the splice plate relative to the pull plate. The slip was not always accompanied 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.



(a) Single Bolt Assembly

(b) Three Bolt Assembly

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.

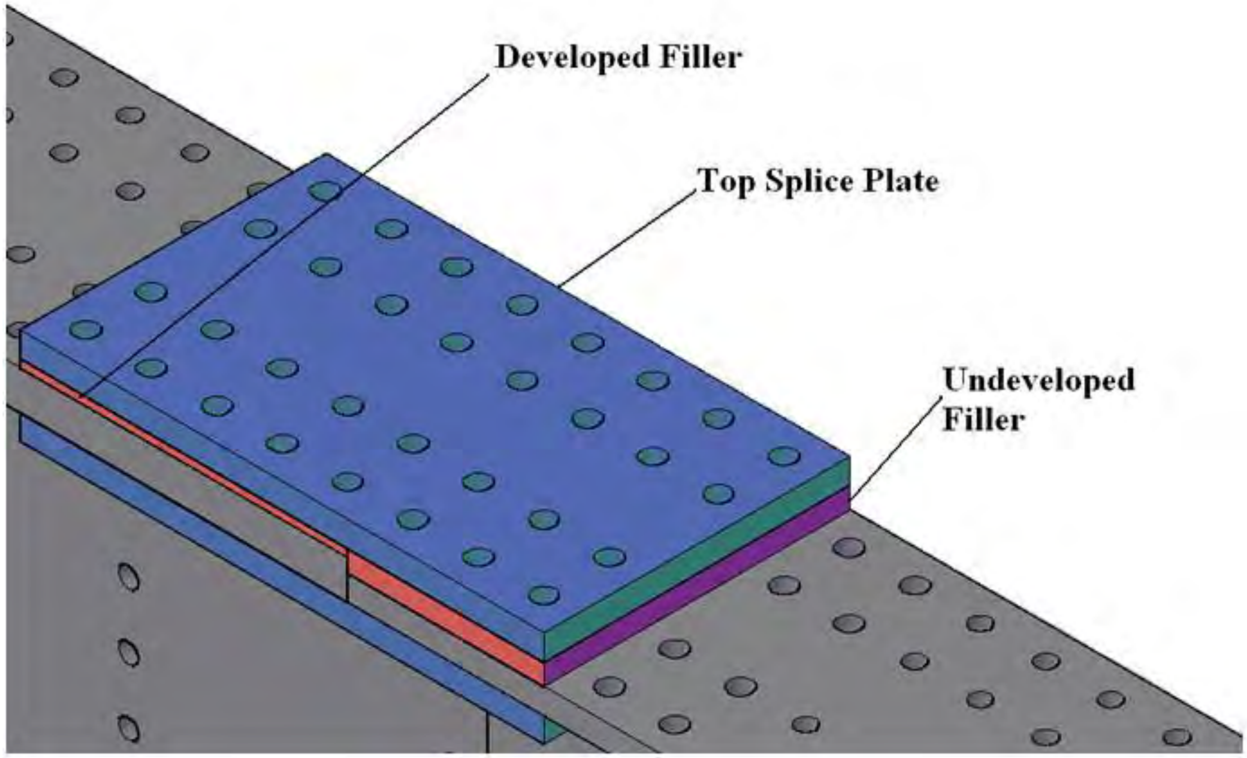


Figure 14: Developed and undeveloped fillers in a splice connection

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.

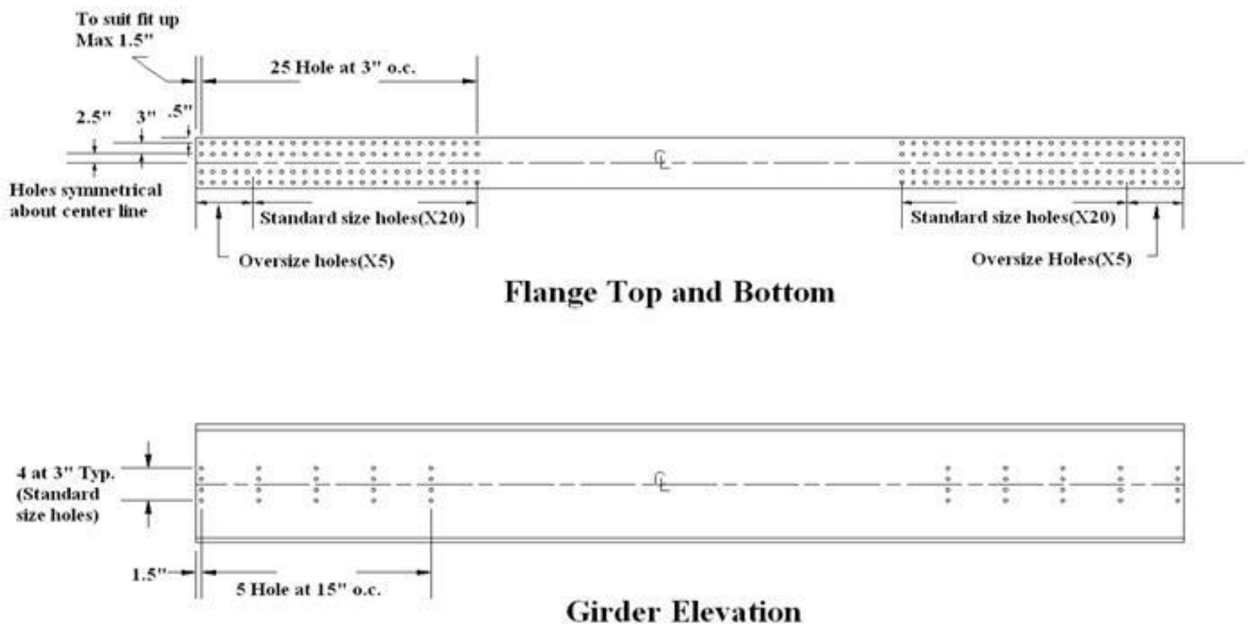


Figure 15: Geometry and hole schematic for the plate girders

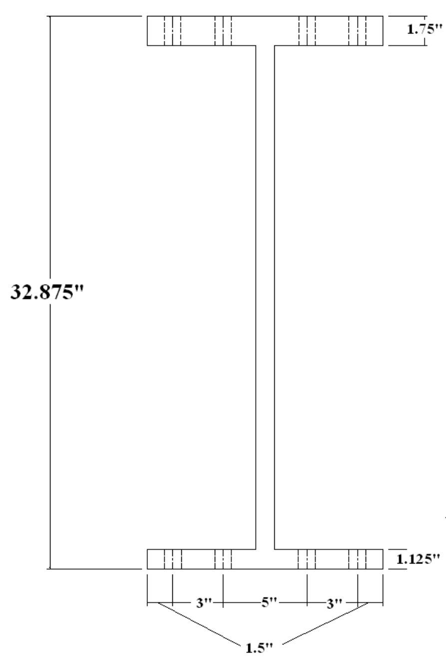


Figure 16: Front view of plate girder

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.

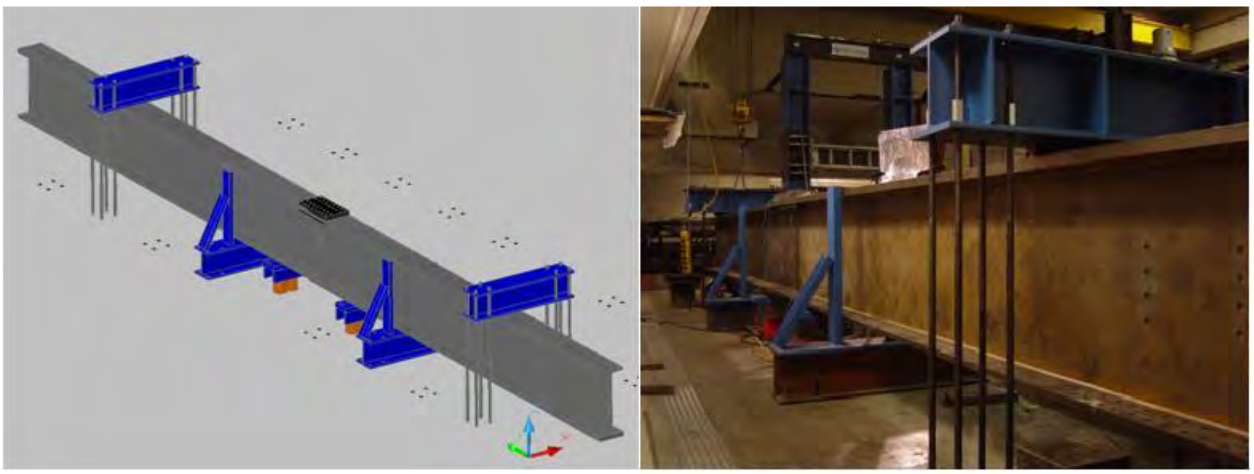


Figure 17: Girder splice test apparatus

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.

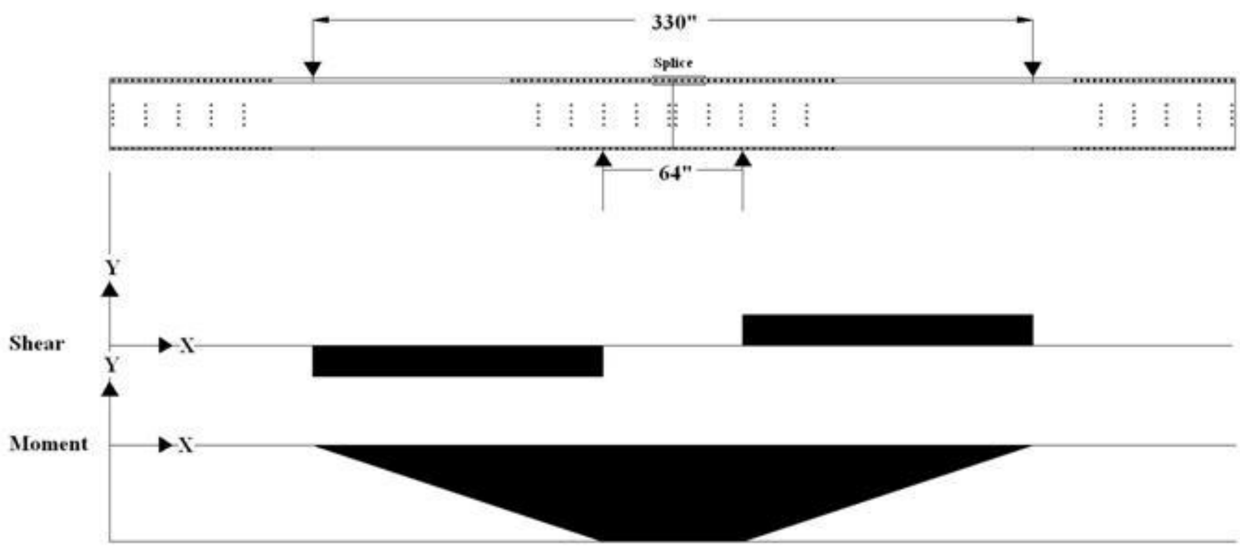


Figure 19: Analytical model for girder splice connection.

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.

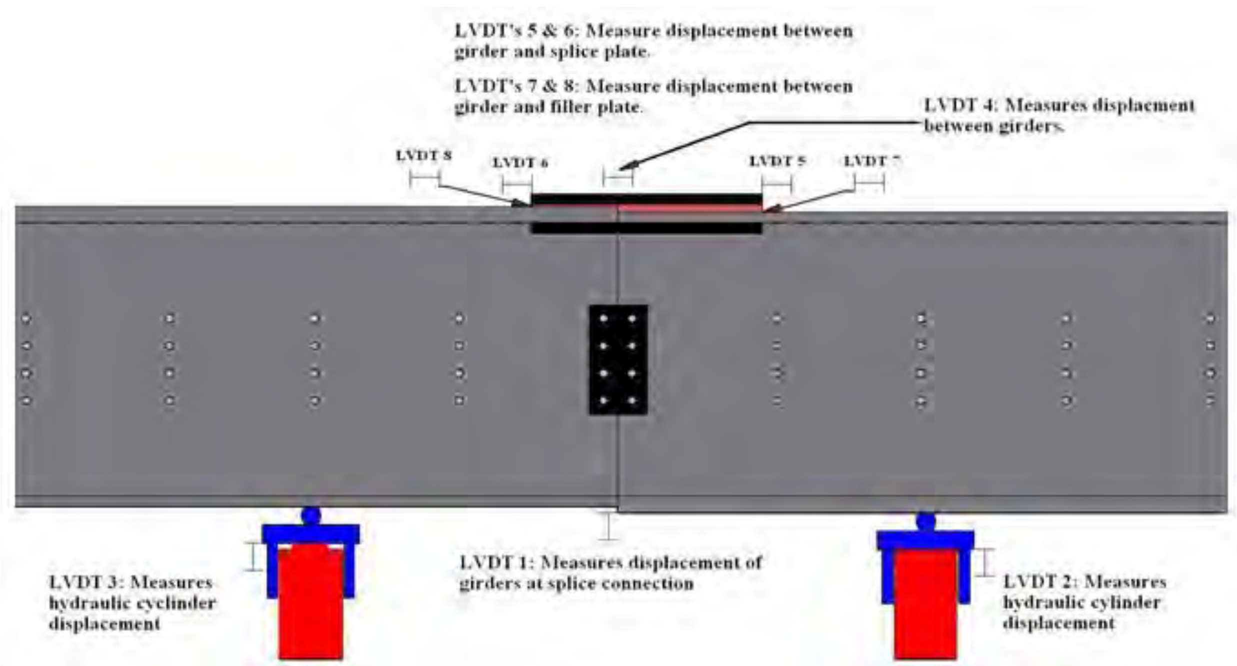


Figure 20: LVDT layout on the test apparatus

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).

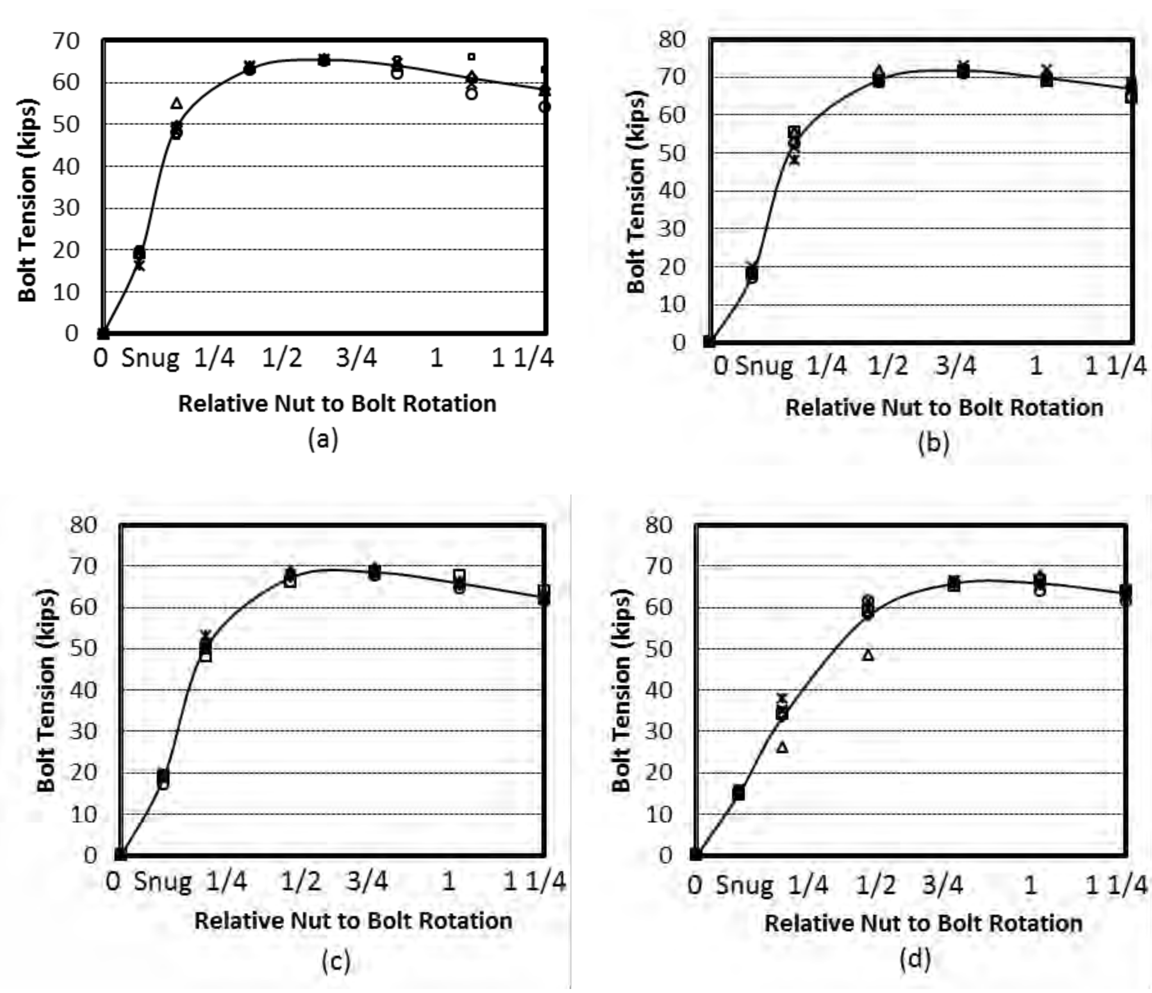


Figure 23: Tension created in bolts of varying lengths as turn-of-the-nut is performed (a) 4.5 in. A490, 5 tests (b) 5 in. A490, 5 tests (c) 5.5 in. A490, 5 tests (d) 6.5 in. A490, 5 tests

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.

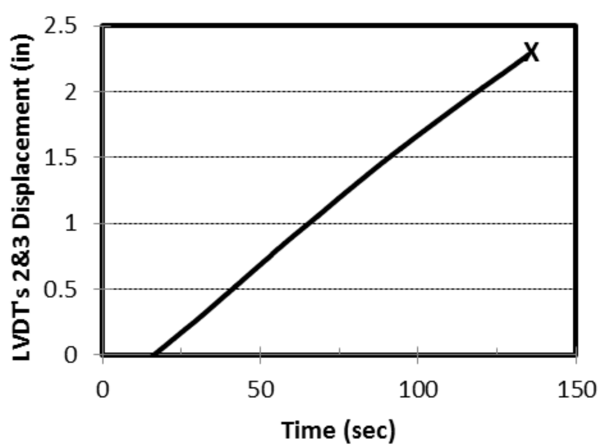


Figure 24: Typical ram velocity for girder splice testing

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.

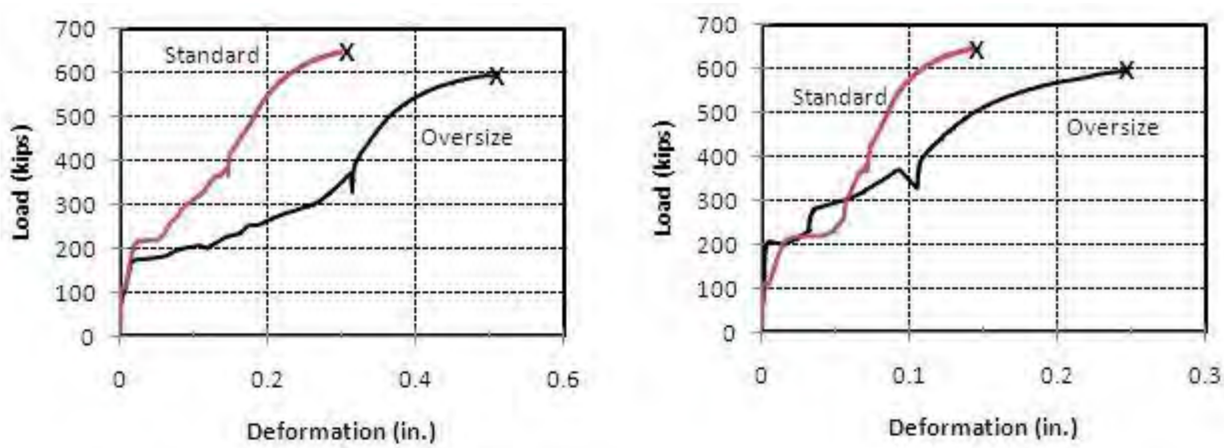


Figure 25: Undeveloped splice plate deformarion and undeveloped filler plate deformation for standard and oversize 1 inch filler tests

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).

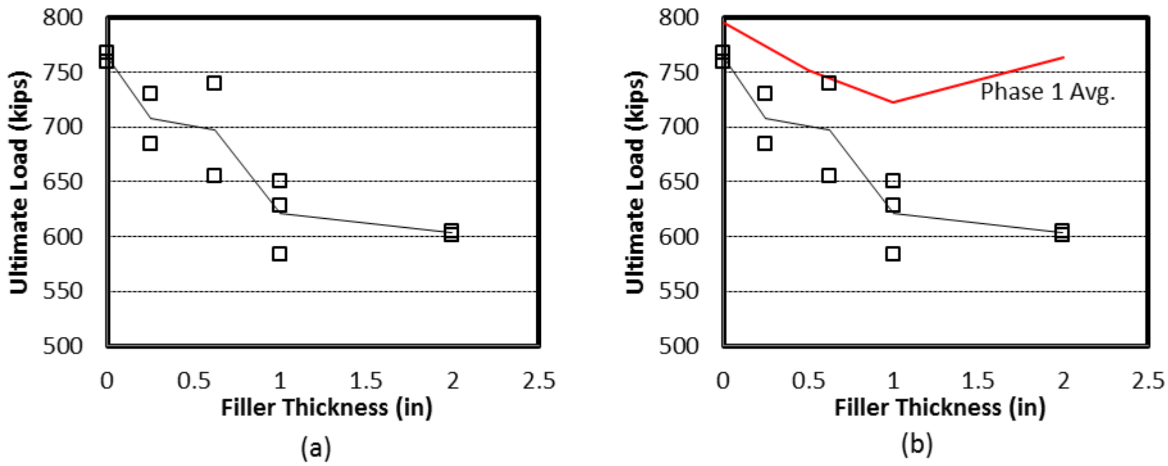


Figure 27: Ultimate loads reached during (a) standard hole tests for every filler thickness and (b) standard holes tests compared to 3 bolt standard hole tests from phase 1.

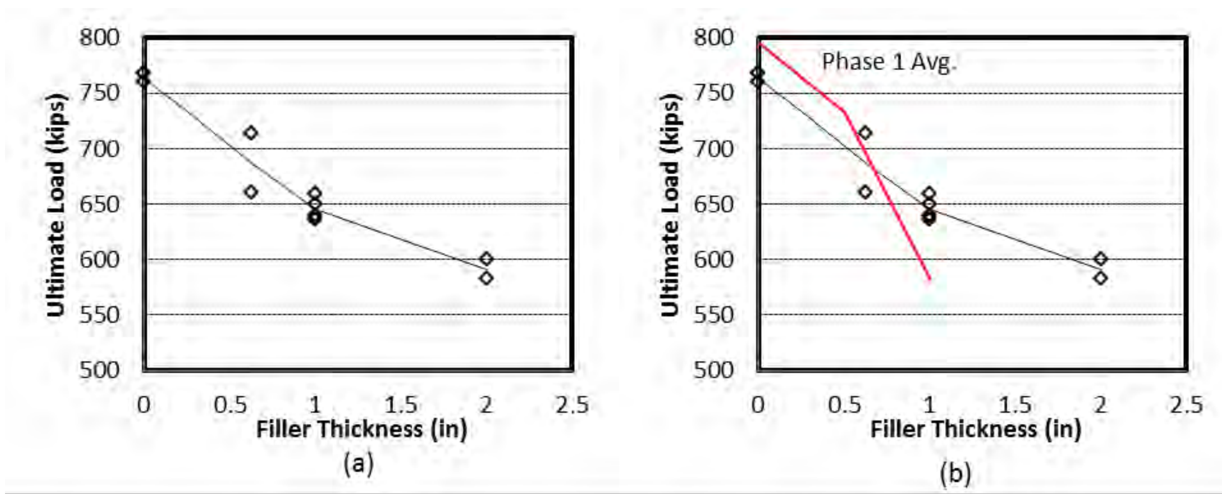


Figure 28: Ultimate loads reached during (a) Multi-ply tests for every filler thickness and (b) Multi-ply tests compared to 3 bolt multi-ply tests from phase 1.

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

<u>Ultimate Loads Reached for Oversize Hole Tests</u>

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.

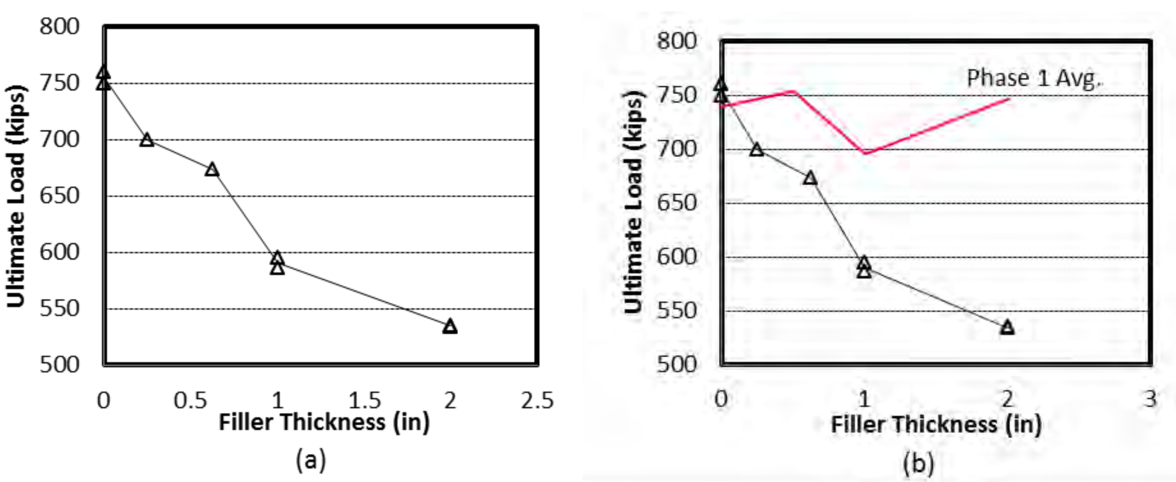


Figure 30: Ultimate loads reached during (a) oversize hole tests for every filler thickness and (b) oversize holes tests compared to 3 bolt oversize hole tests from phase 1.

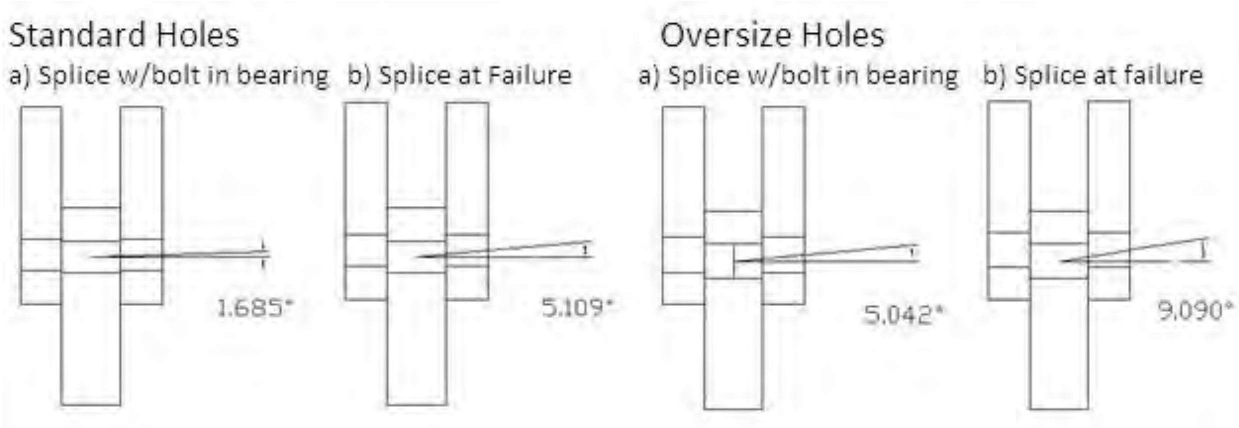


Figure 31: Theoretical bolt deformation angle at bearing and failure for standard and oversize cases.

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.

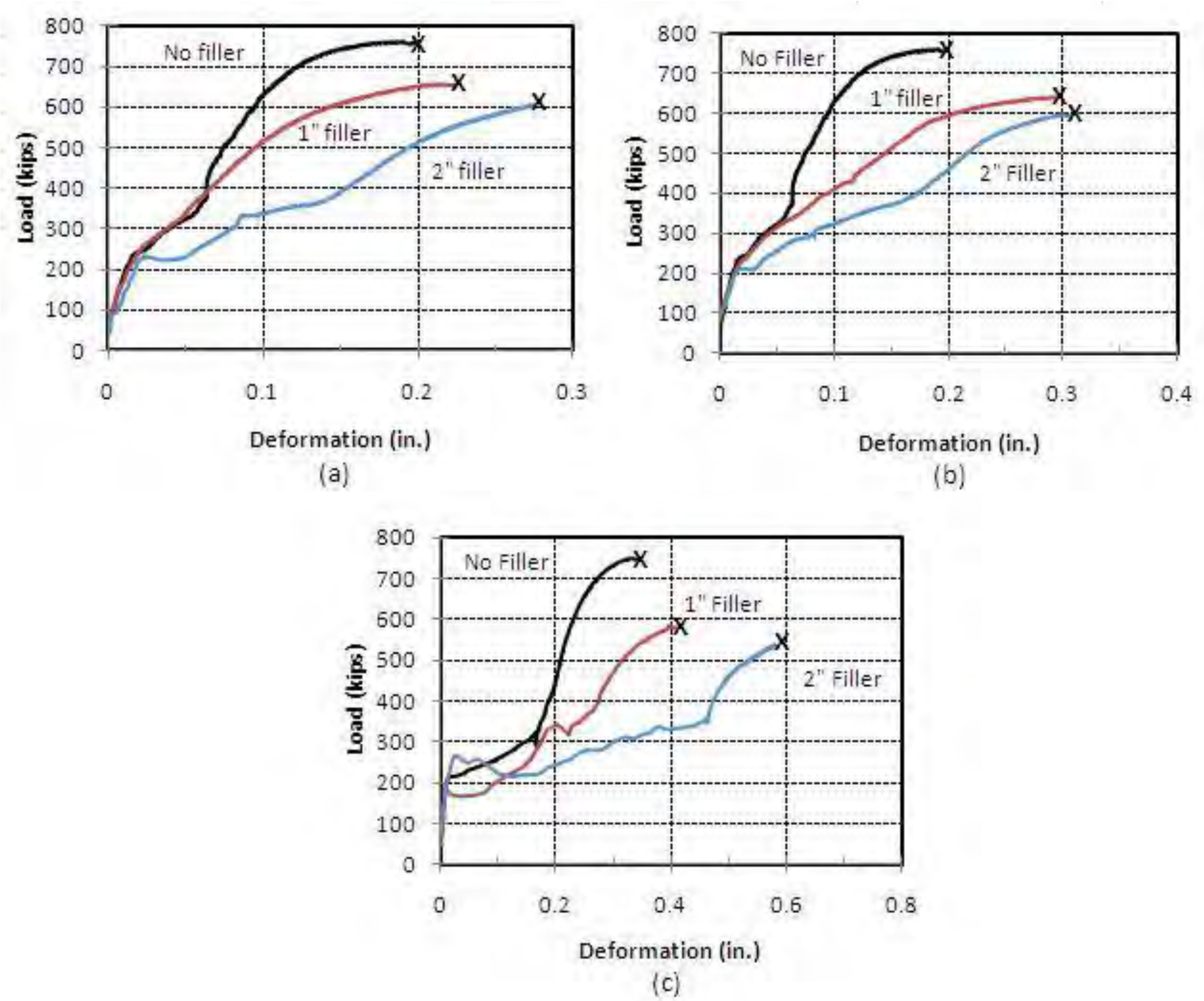


Figure 32: Representative force vs. displacement plots for (a) standard tests, (b) multi-ply tests and (c) oversize tests.

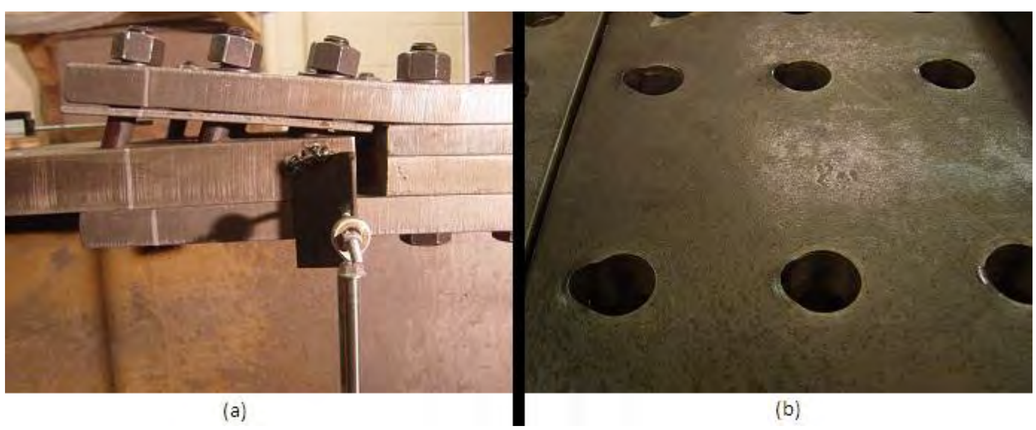


Figure 33: Regions of plastic deformation after girder tests in (a) the top splice plate/bolts and (b) the bolts holes.

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.

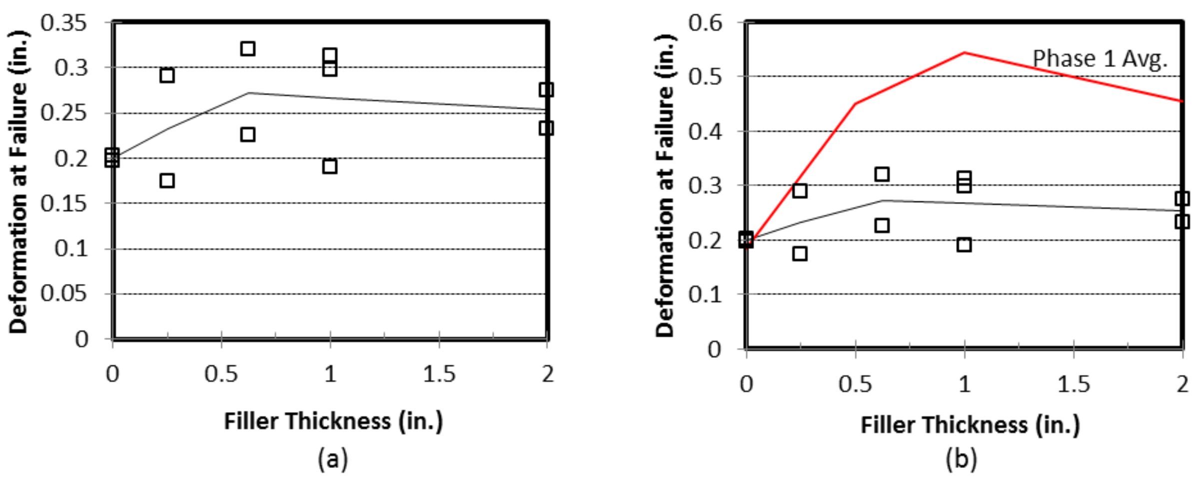


Figure 34: Plate deformation at failure for (a) standard tests and (b) standard tests compared to phase 1, 3 bolt standard tests

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.

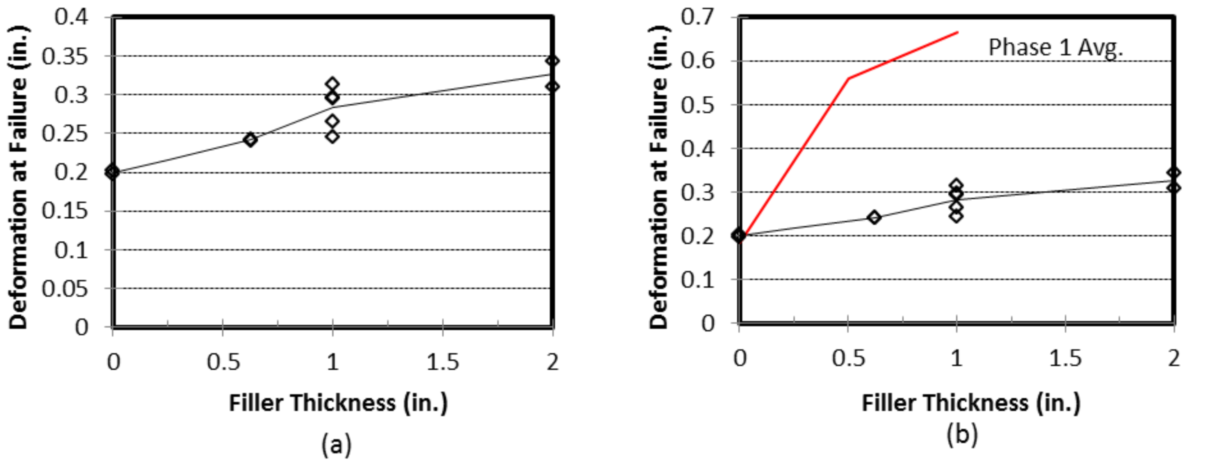


Figure 35: Plate deformation at failure for (a) Multi-ply tests and (b) Multi-ply tests compared to phase 1, 3 bolt multi-ply tests.

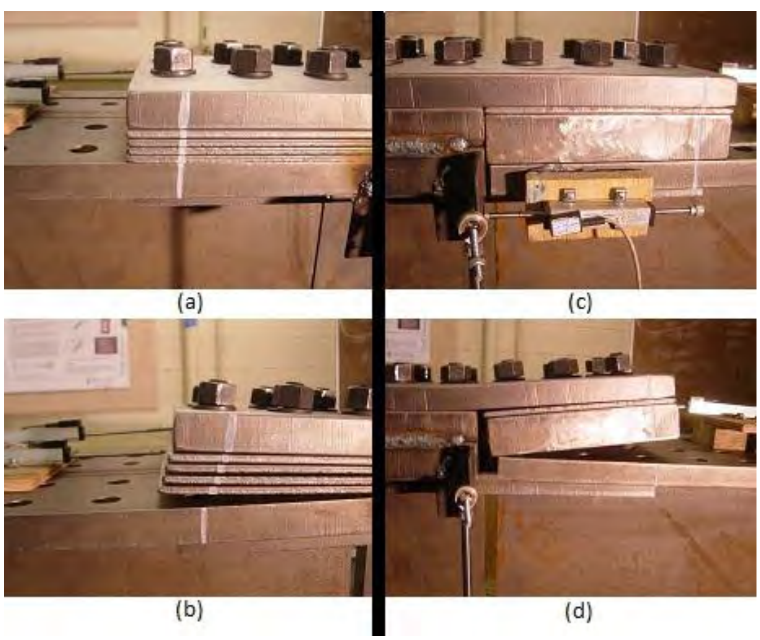


Figure 36: Multi-ply series individual filler plate deformation for 1 in. tests (a) before and (b) after testing; and 2 in. filler tests (c) before and (d) after testing.

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.

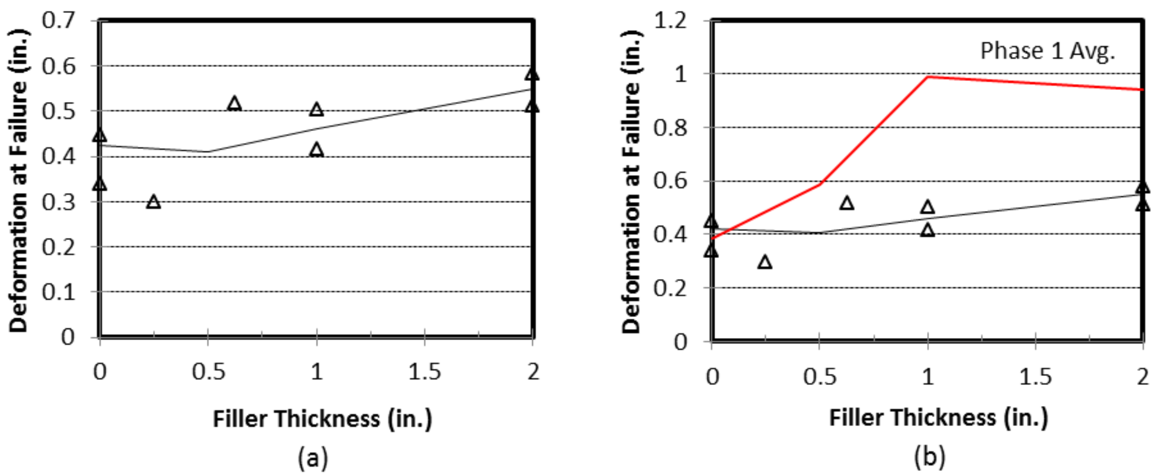


Figure 37: Plate deformation at failure for (a) oversize hole tests and (b) oversize hole tests compared to phase 1, 3 bolt oversize hole tests

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.

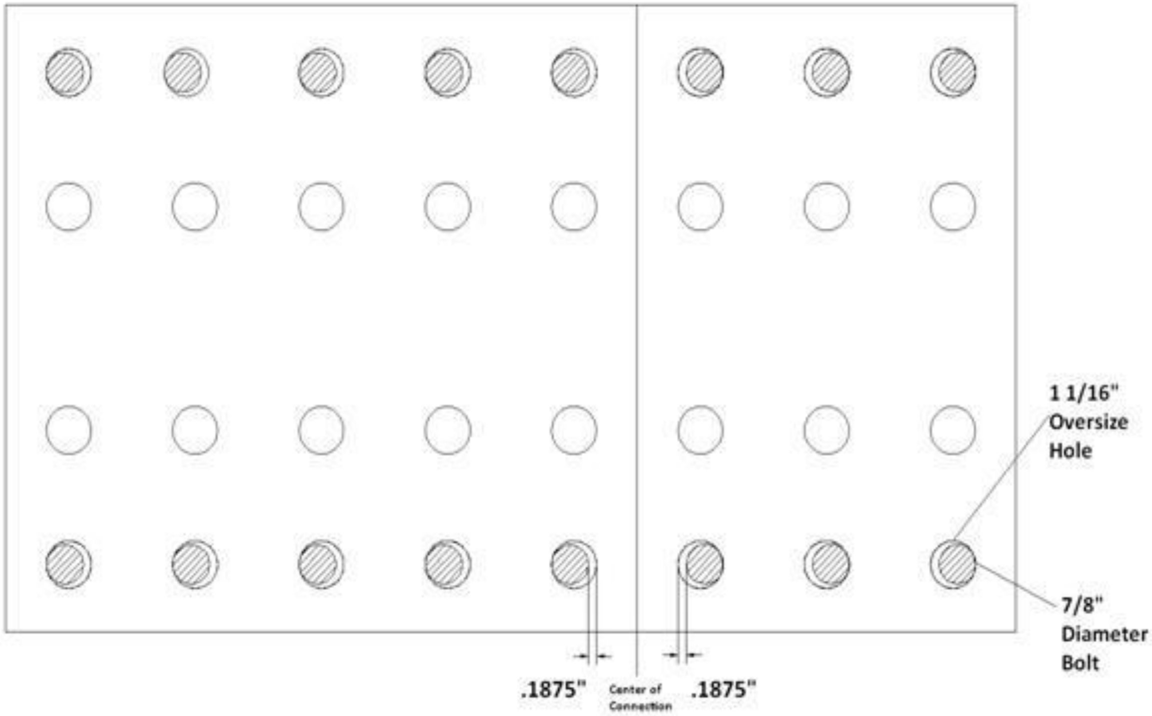


Figure 38: Potential connection displacement prior to complete bolt bearing for oversize holes.

<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.

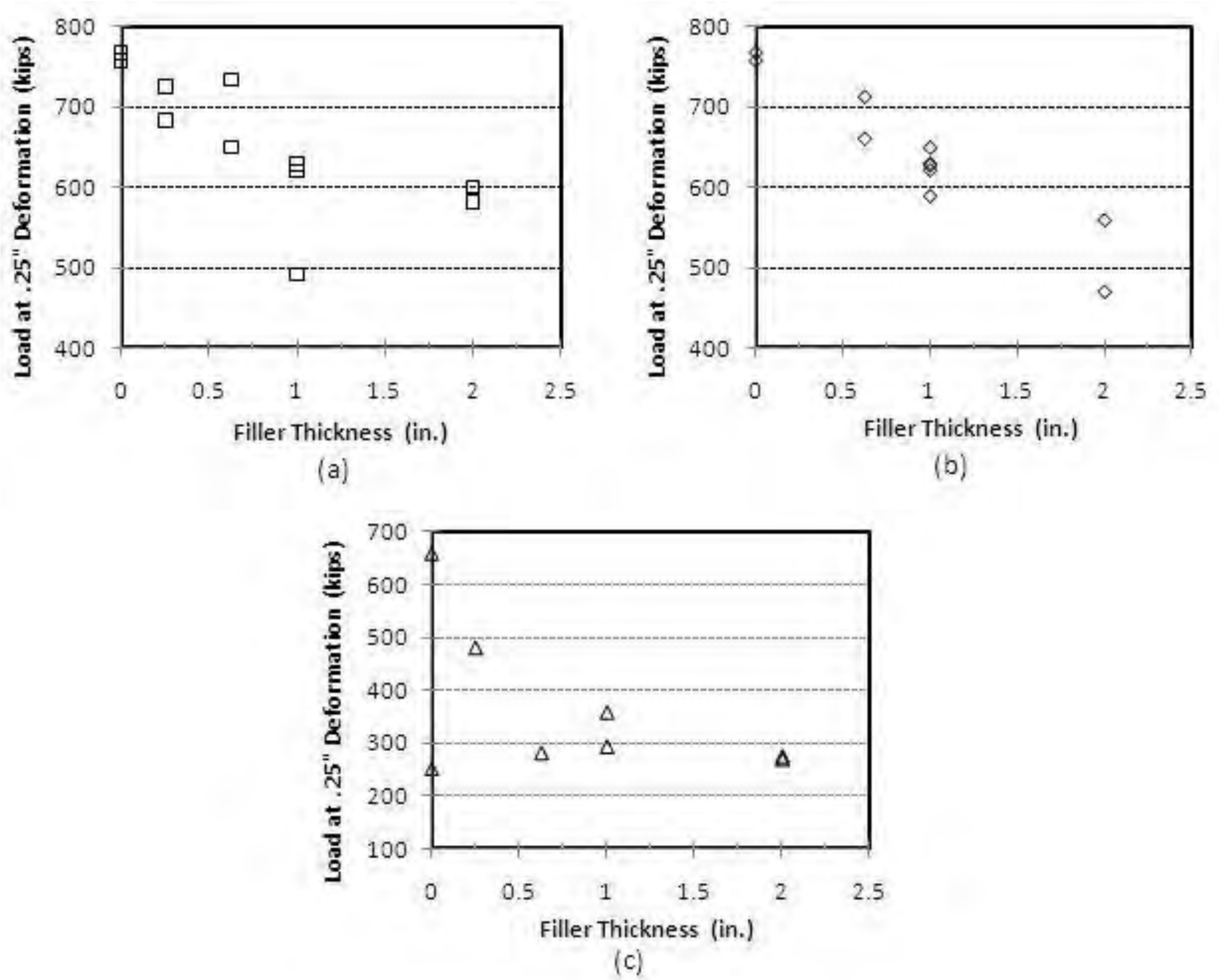


Figure 39: Load at 0.25 in. deformation for (a) standard tests (b) multi-ply tests and (c) oversize tests for every filler plate thickness

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.

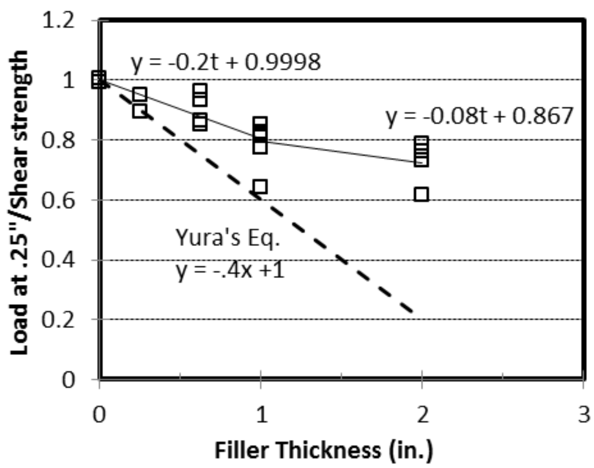


Figure 40: Shear strength reduction lines for girder splice testing compared to Yura's line for suggested shear strength reduction in standard hole tests for every filler plate thickness

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.

Girder Rotation

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.

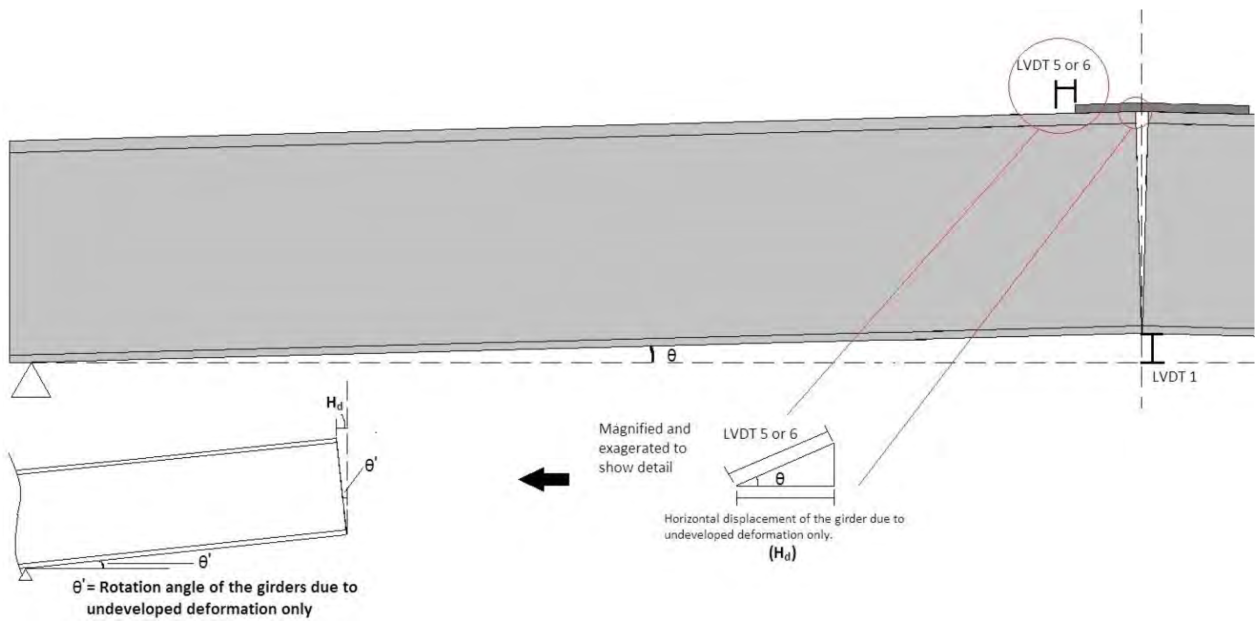


Figure 41: Girder rotation angle analysis.

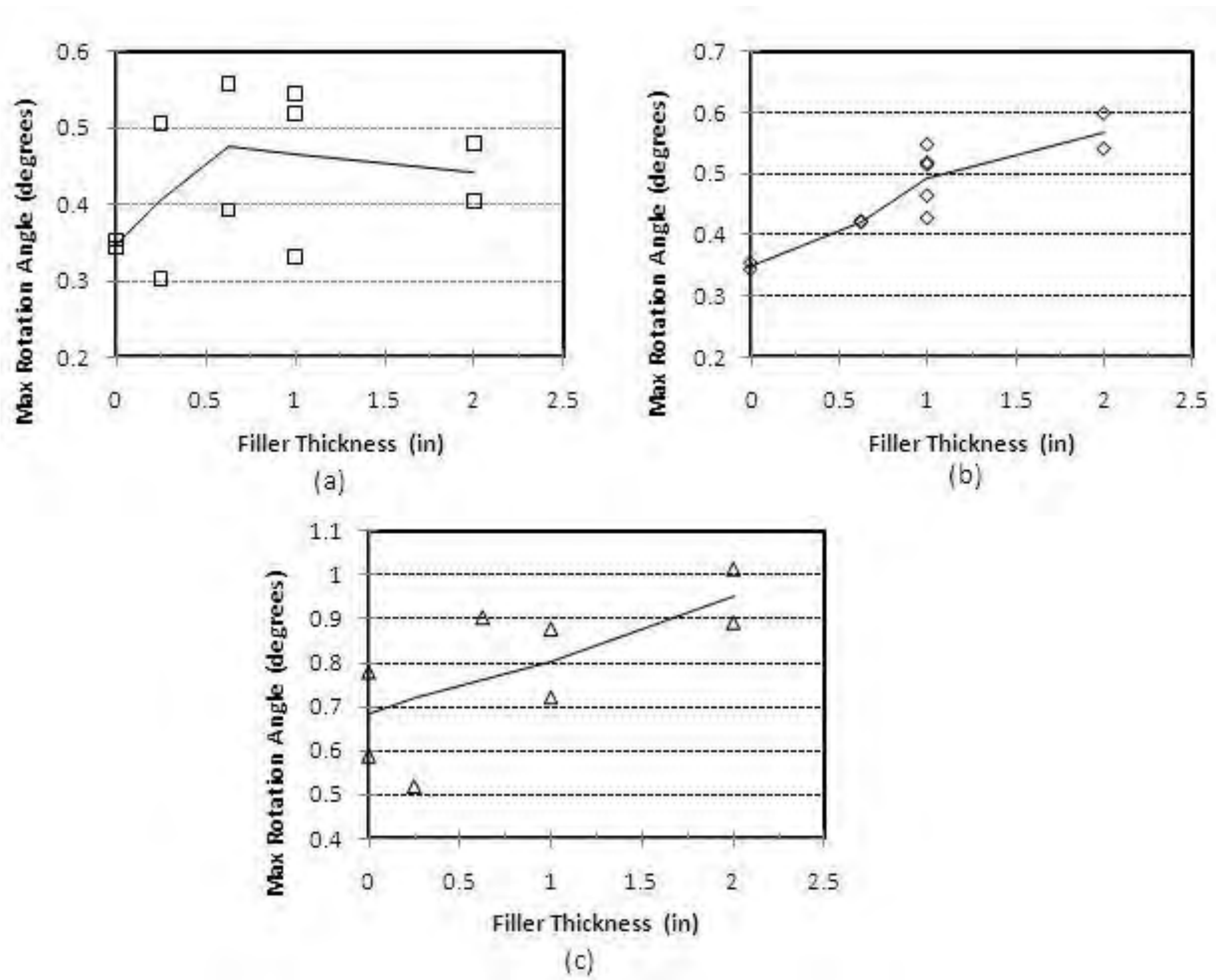


Figure 42: Maximum girder rotation angle for (a) standard tests (b) multi-ply tests and (c) oversize tests for every filler plate thickness.

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.

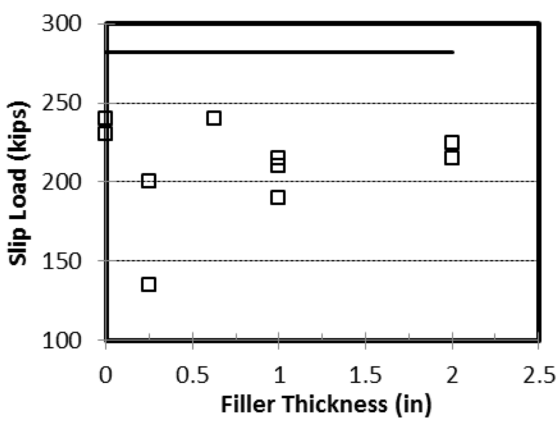


Figure 43: Slip loads (with the AISC slip critical line) and slip coefficients for standard tests.

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.

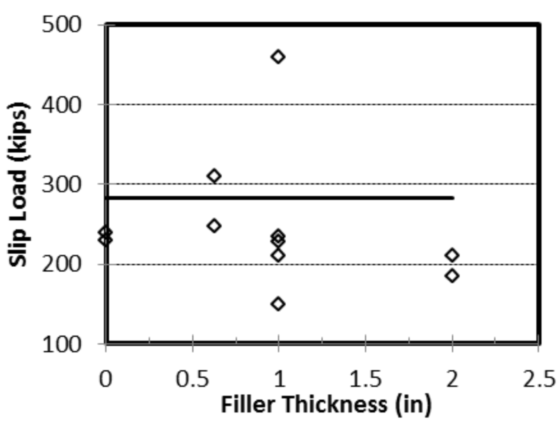


Figure 44: Slip loads (with the AISC slip critical line) and slip coefficients for multi-ply tests.

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.

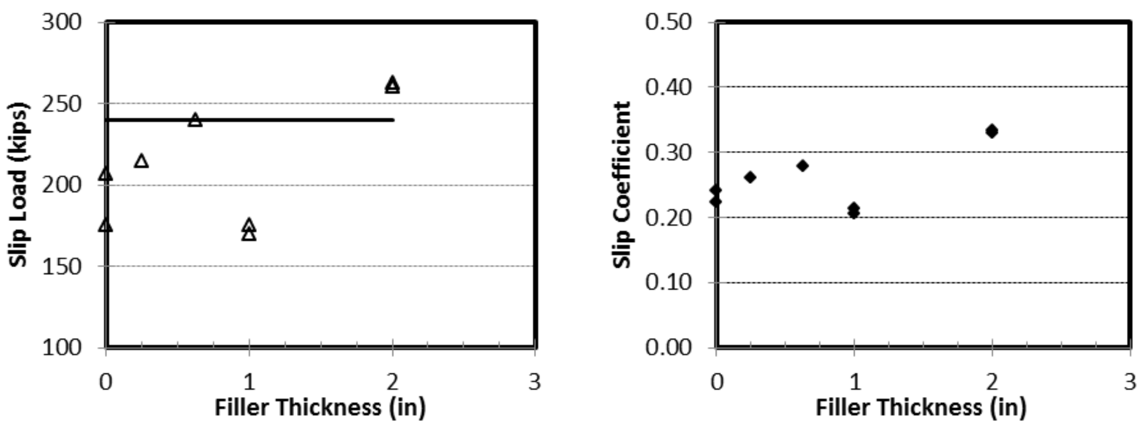


Figure 45: Slip loads (with the AISC slip critical line) and slip coefficients for oversize tests.

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.

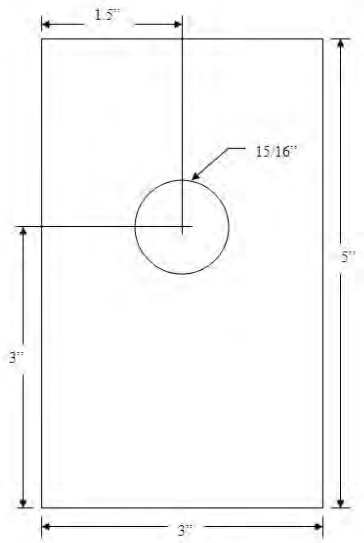


Figure 46: Geometry of coupons used in supplemental slip testing

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.

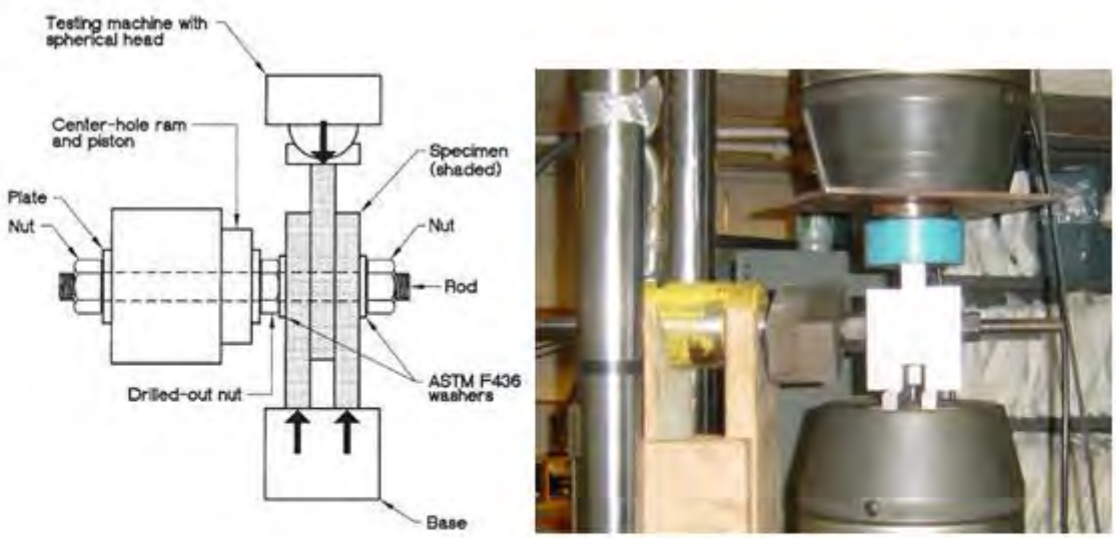


Figure 47: RCSC schematic of the slip test set up compared to actual set up.

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.

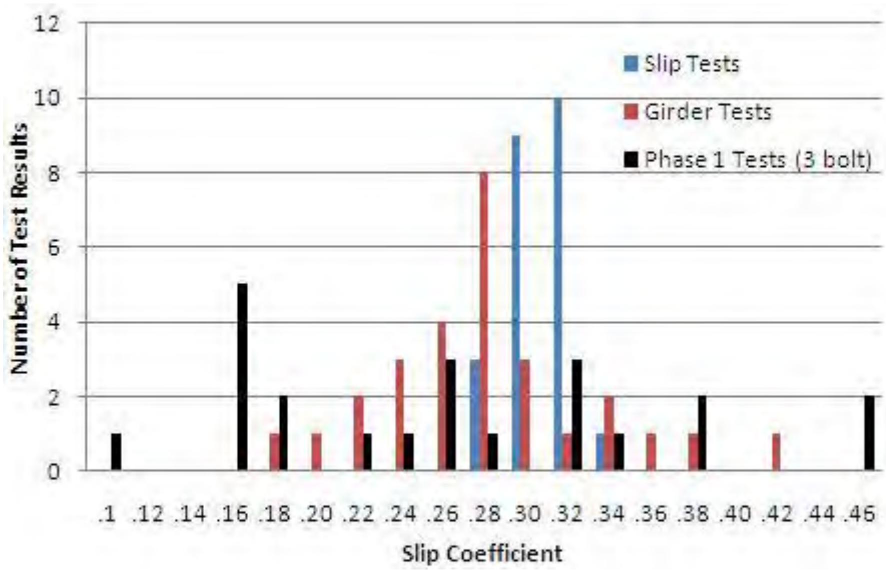


Figure 48: Frequency of slip coefficient values from slip coefficient tests, girder splice tests, and phase 1 tests (3 bolt)

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.

References

AASHTO (2008), *AASHTO LRFD Bridge Design Specifications – 4th Edition*, American Association of State Highway Officials

American Institute of Steel Construction Inc. (2005). Steel Construction Manual, 13th Ed.

Dusicka, P. and Lewis, G. (2007). "Effect of Fillers on Steel Girder Field Splice Performance Year 1 Progress Report", *Research Council on Structural Connections*, Department of Civil and Environmental Engineering, Portland State University, Portland, OR.

Kulak, G.L., Fisher J.W., and Struik, J.H.A. (2001). "Guide to Design Criteria for Bolted and Riveted Joints", Second Edition, Research Council on Structural Connections, Chicago, IL.

Lee, J.H., Fisher, J.W. (1968). "The Effect of Rectangular and Circular Fillers on the Behavior of Bolted Joints", Report 318.6, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania.

RCSC (2001) "Guide to Design Criteria for Bolted and Riveted Joints", by Kulak, G.L., Fisher J.W., and Struik, J.H.A., Second Edition, Research Council on Structural Connections, Chicago, IL.

RCSC (2004) "Testing Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints", *Specification for Structural Joints Using ASTM A325 or A460 Bolts*. Research Council on Structural Connections, Chicago, IL.

Sheikh-Ibrahim, F.I. (2002), "Design Methods for Bolts in Bearing-Type Connections with Fillers", Engineering Journal, Fourth Quarter, American Institute of Steel Connections.

Sugiyama, N., Kubo, M., Kondou, A., Ito, I., and Oda, H. (2001). "Slip Tests on High-Strength Bolted Friction Joints with Filler Plates", Vol. 41, Pp. 50-56., Reports of the Faculty of Science and Technology, Meijo University, Aichi, Japan

Wallaert, J.J., Fisher, J.W. (1965). "Shear Strength of High-Strength Bolts", *Journal of the Structural Division*, Vol. 91, No. ST3., American Society of Civil Engineers.

Yura, J.A., Hansen, M.A. and Frank, K.H (1982), "Bolted Splice Connections with Undeveloped Fillers", *Journal of the Structural Division*, Vol. 108, No. ST12, American Society of Civil Engineers.

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	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

Test Number		Tension (kips) Snug	Tension (kips) 1/4 turn	Tension (kips) 1/2 turn	Tension (kips) 3/4 turn	Tension (kips) 1 turn	Tension (kips) 1 1/4 turn	Tension (kips) 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

Research Council on Structural Connections

Procedure for Proposing Changes

Foreword: This procedure was created to establish a formal approach for the submission of ideas or proposed changes to the specification, educational documents or to the By-Laws of the Research Council on Structural Connections. The Executive Committee desires this process to be as straightforward as possible to enable members to communicate their ideas by an attachment to an e-mail, fax or mailed document method of their preference.

Proposed Change Process Summary: The process is a five step process:

1) All proposals for change are to be submitted to the Chairman of the Executive Committee for review by the Executive Committee for consideration and assignment to the appropriate committee chair for creation of a task group or to become an agenda item at the next committee meeting.
2) The task group or committee reviews the proposal, recommends an action back to the committee chair and may assist with final preparation of the letter ballot if that is the recommendation of the task group.
3) The committee chair then reviews the recommendation of the task group with the members of the committee and either forwards the recommendation onto the main council to be added as an agenda item for discussion or returns the item back to task group for further action.
4) The Council addresses the proposed revision as an agenda item of new business at their next meeting where the proposed change is debated, accepted and sent out for letter ballot, accepted and sent to committee for implementation, returned to committee for further action or found non-persuasive and terminated.
5) Proposals found non-persuasive may be appealed per Section 12.2 of the By-Laws to the Council Secretary which will be reviewed by the Executive Committee at their next meeting for a recommended action that is then placed on the agenda at the next Annual Meeting of the Council for final resolution.

Proposal Format: Each proposed change is to contain the following information and is to be submitted on separate documents:

1) Author's name and contact information, e-mail address (preferred) and phone number (as a minimum)
2) Proposed change
3) Rational or justification for change including references to research which justify the change
4) Commentary

Items without the above information may be considered inadequate and rejected by the Executive Committee from further consideration.

Use the attached page to submit proposals, contact a member of the Executive Committee or obtain a copy from the Web Site at www.boltcouncil.org.

RCSC Proposed Change

Name: Chris Curven **E-mail:** chrisc@appliedbolting.com

Phone: 802-460-3100 Fax: ______________________________________

Proposed Change:

8.2.3 does not actually state when the installer is to stop tightening or when the bolt is deemed tight. It states what type of installation tool to be used, but not what the installer is looking for.

For example, 8.2.1. states to rotate the head or nut as specified in table 8.2., 8.2.2. states to apply the installation torque determined by the pre-installation verification, and 8.2.4. has the installer making sure the achieved gap is less than the job inspection gap.

8.2.3. should read, "…Subsequently, all bolts in the joint shall be tightened with the twist-off-type tension-control bolt installation wrench until the splined-end shears off, progressing systematically from the most rigid part of the joint in a manner that will minimize relaxation of previously pretensioned bolts.

Also, Section 9.2.4. is the only installation method that has the inspector verify that snugging of the bolts and plies have taken place before the chosen pretensioning method takes place. 9.2.1., 9.2.2.,and 9.2.3. would obviously like to have inspection of the snug condition, but it is not listed.

For example, 9.2.4. …All bolts shall be installed in accordance with the requirements in Section 8.1, with washers positioned as required in Section 6.2. The installer shall verify that the direct-tension-indicator protrusions have not been compressed to a gap that is less than the job inspection gap during this operation, and if this has occurred, the direct tension indicator shall be removed and replaced.…

9.2.1. Turn-of-Nut Pretensioning: The inspector shall observe the pre-installation verification testing required in Section 8.2.1. Subsequently, but prior to pretensioning, it shall be ensured by routine observation that the plies have been brought into firm contact. Subsequently, it shall be ensured by routine observation that the bolting crew properly rotates the turned element relative to the unturned element by the amount specified in Table 8.2. Alternatively, when fastener assemblies are match-marked, the plies must be inspected for snug tight condition before match-marking may commence. After match-marking, the part turned relative to the unturned element must be inspected for rotation in accordance with Table 8.2.…

9.2.2. Calibrated Wrench Pretensioning: The inspector shall observe the daily preinstallation verification testing required in Section 8.2.2. Subsequently, but prior to pretensioning, it shall be ensured by routine observation that the plies have been brought into firm contact. Subsequently, it shall be ensured by routine observation that the bolting crew properly applies the calibrated wrench to the turned element. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection.

9.2.3. Twist-Off-Type Tension-Control Bolt Pretensioning: The inspector shall observe the pre-installation verification testing required in Section 8.2.3. <u>Subsequently, but prior to pretensioning, it shall be ensured by routine observation that the plies have been brought into firm contact without the splined-ends being severed. If the splined-end is severed, the bolt must be removed and replaced.</u> Subsequently, it shall be ensured by routine observation that the splined ends are properly severed during installation by the bolting crew. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection

Rational or Justification for Change (attach additional pages as needed):
Most of the rational is rolled into the "proposed changes" for clarity. Critical actions are not covered in the current specification. Since they are not mentioned, they are not required. This is an oversight that needs to be fixed so users can tighten bolts correctly.

Commentary (attach additional pages as needed): No commentary changes are needed.

-----------------------------------For Committee Use Below---

Date Received: ______ Exec Com Meeting: _______ Forwarded: Yes ☐ /No ☐

Committee Assignment: Executive -A. ☐ Editorial -B. ☐ Nominating -C. ☐

Specifications -A.1 ☐ Research -A.2 ☐ Membership & Funding -A.3 ☐ Education -A.4 ☐
Committee Chair: ____________ Task Group #: ___________ T.G. Chair: ________________
Date Sent to Main Committee: _______________Final Disposition: ______________________

RCSC Proposed Change: S11-038

Name: Chris Curven **E-mail: chrisc@appliedbolting.com**

Phone: 802-460-3100______ **Fax:** ______________________________________

Ballot History:

Proposed Change:

8.2. Pretensioned Joints and Slip-Critical Joints

One of the pretensioning methods in Sections 8.2.1 through 8.2.4 shall be used, except when alternative-design fasteners that meet the requirements of Section 2.8 or alternative washer-type indicating devices that meet the requirements of Section 2.6.2 are used, in which case, installation instructions provided by the *manufacturer* and approved by the *Engineer of Record* shall be followed.

{Table 8.1 "Minimum Bolt Pretension, Pretensioned and Slip-Critical Joints" is unchanged and will not be reproduced here.}

When it is impractical to turn the nut, pretensioning by turning the bolt head is permitted while rotation of the nut is prevented, provided that the washer requirements in Section 6.2 are met. A pretension that is equal to or greater than the value in Table 8.1 shall be provided. The pre-installation verification procedures specified in Section 7 shall be performed as indicated in Sections 8.2.1 through 8.2.4, using *fastener assemblies* that are representative of the condition of those that will be pretensioned in the work.

The required p~~P~~re-installation testing shall be performed for each fastener assembly lot prior to the use of that assembly lot in the work. The testing shall be done at the start of the work. For calibrated wrench pretensioning, this testing shall be performed daily for the calibration of the installation wrench.

Commentary:
{There are no proposed changes to the commentary for this subsection.}

8.2.1. Turn-of-Nut Pretensioning: The pre-installation verification procedures specified in Section 7 shall demonstrate that the required rotation from snug-tight shall reach at least the minimum required tension in Table 7.1. All bolts shall be installed in accordance with the requirements in Section 8.1, with washers positioned as required in Section 6.2. Subsequently, the nut or head rotation specified in Table 8.2 shall be applied to all *fastener assemblies* in the *joint*,

-----------------------------------For Committee Use Below--

Date Received: 6/10/11 Exec Com Meeting: _6/11___ Forwarded: Yes X /No ☐

Committee Assignment: Executive -A. ☐ Editorial -B. ☐ Nominating -C. ☐

Specifications -A.1 X Research -A.2 ☐ Membership & Funding -A.3 ☐ Education -A.4 ☐

Committee Chair: Harrold Task Group #: ___________ T.G. Chair: Curven

Date Sent to Main Committee: _______________Final Disposition: ______________________

progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts. The part not turned by the wrench shall be prevented from rotating during this operation. Upon completion of the application of the required nut rotation for pretensioning, it is not permitted to turn the nut in the loosening direction except for the purpose of complete removal of the individual fastener assembly. Such fastener assemblies shall not be reused except as permitted in Section 2.3.3.

{Table 8.2 "Nut Rotation from Snug-Tight Condition for Turn-of-Nut Pretensioning" is unchanged and will not be reproduced here.}

Commentary:
{There are no proposed changes to the commentary for this subsection.}

8.2.2. Calibrated Wrench Pretensioning:
{There are no proposed changes to this subsection.}

8.2.3. Twist-Off-Type Tension-Control Bolt Pretensioning: Twist-off-type tension-control bolt assemblies that meet the requirements of ASTM F1852 or F2280 shall be used. The pre-installation verification procedures specified in Section 7 shall demonstrate that, when the splined end is severed off with the required tool, the bolt tension shall be at least equal to that required in Table 7.1.

All *fastener assemblies* shall be installed in accordance with the requirements in Section 8.1 without severing the splined end and with washers positioned as required in Section 6.2. If a splined end is severed during this operation, the *fastener assembly* shall be removed and replaced. Subsequently, all bolts in the *joint* shall be pretensioned with the twist-off-type tension-control bolt installation wrench, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts.

Commentary:
{There are no proposed changes to the commentary for this subsection.}

8.2.4. Direct-Tension-Indicator Pretensioning:
{There are no proposed changes to this subsection.}

Rationale or Justification for Change (attach additional pages as needed):

Sections 9.2.1 and 9.2.3 make a reference to the pre-installation verification testing in Sections 8.2.1 and 8.2.3 respectively. There is currently no language in Sections 8.2.1 and 8.2.3 that refer to the pre-installation testing.

This proposal corrects that omission and makes all four subsections of Section 8.2 refer to Chapter 7 pre-installation requirements in an equivalent manner.

RCSC Proposed Change: S12-039

Name: Tom Schlafly
Phone: 312-670-5412
E-mail: schlafly@aisc.org
Fax:

Ballot History:

Proposed Change:
Table 2.1

Table 2.1. Acceptable ASTM A563 Nut Grade and Finish and ASTM F436 Washer Type and Finish

ASTM Desig.	Bolt Type	Bolt Finish[d]	ASTM A563 nut grade and finish[d]	ASTM F436 washer type and finish[a,d]
A325	1	**Plain (uncoated)**	C, C3, D, DH[c] and DH3; plain	1; plain
		Galvanized	DH[c]; galvanized and lubricated	1; galvanized
		Zn/Al Inorganic, per ASTM F1136 Grade 3	DH[c]; Zn/Al Inorganic, per ASTM F1136 Grade 5	1; Zn/Al Inorganic, per ASTM F1136 Grade 3[b]
	3	**Plain**	C3 and DH3; plain	3; plain
F1852	1	**Plain (uncoated)**	C, C3, DH[c] and DH3; plain	1; plain[b]
		Mechanically Galvanized	DH [c]; mechanically galvanized and lubricated	1; mechanically galvanized[b]
		~~**Zn/Al Inorganic, per ASTM F1136 Grade 3**~~	~~DH[c]; Zn/Al Inorganic, per ASTM F1136 Grade 5~~	~~1; Zn/Al Inorganic, per ASTM F1136 Grade 3[b]~~
	3	**Plain**	C3 and DH3; plain	3; plain[b]
A490	1	**Plain**	DH[c] and DH3; plain	1; plain
		Zn/Al Inorganic, per ASTM F1136 Grade 3	DH[c]; Zn/Al Inorganic, per ASTM F1136 Grade 5	1; Zn/Al Inorganic, per ASTM F1136 Grade 3[b]
	3	**Plain**	DH3; plain	3; plain
F2280	1	**Plain**	DH[c] and DH3; plain	1; plain
	3	**Plain**	DH3; plain	3; plain

-----------------------------------For Committee Use Below---

Date Received: 1/05/12 Exec Com Meeting: 1/19/12 Forwarded: Yes X /No ☐

Committee Assignment: Executive -A. ☐ Editorial -B. ☐ Nominating -C. ☐

Specifications -A.1 X Research -A.2 ☐ Membership & Funding -A.3 ☐ Education -A.4 ☐

Committee Chair: Harrold Task Group #: ___________ T.G. Chair: ________________

Date Sent to Main Committee: _______________Final Disposition: ______________________

[a] Applicable only if washer is required in Section 6.

[b] Required in all cases under nut per Section 6.

[c] The substitution of ASTM A194 grade 2H nuts in place of ASTM A563 grade DH nuts is permitted.

[d] "Galvanized" as used in this table refers to hot-dip galvanizing in accordance with ASTM F2329 or mechanical galvanizing in accordance with ASTM B695.

[e] "Zn/Al Inorganic" as used in this table refers to application of a Zn/Al Corrosion Protective Coating in accordance with ASTM F1136 which has met all the requirements of IFI-144.

Section 2.3.3 Commentary

{Modification is to the fourth paragraph of the commentary. All other portions of the Commentary are unchanged.}

An extensive investigation conducted in accordance with IFI-144 was completed in 2006 and presented to the ASTM F16 Committee on Fasteners (F16 Research Report RR: F16-1001). The investigation demonstrated that Zn/Al Inorganic Coating, when applied per ASTM F1136 Grade 3 to ASTM A490 bolts, does not cause delayed cracking by internal hydrogen embrittlement, nor does it accelerate environmental hydrogen embrittlement by cathodic hydrogen absorption. It was determined that this is an acceptable finish to be used on Type 1 ASTM A325 and A490 bolts~~ and F1852 and F2280 twist-off-type tension-control bolt assemblies~~.

Rationale or Justification for Change (attach additional pages as needed):

At the present time, ASTM has not accepted the use of the Zn/Al Inorganic coating on either the F1852 or F2280 tension-control bolt assemblies. There have been some concerns raised regarding the proper fabrication of the assembly parts given the significantly different coefficient of friction generated by the Zn/Al coating in comparison with normal lubricated assemblies. This difference could result in bolts that have not been properly tensioned.

RCSC Proposed Change: S12-041

Name: Tom Schlafly
E-mail: schalfly@aisc.org
Phone: 312-670-5412
Fax:

Proposed Change:

No changes proposed for the Specification itself.

Section 6.2.5 Commentary

{Only the third paragraph of the commentary has a change proposed.}

Heat-treated washers not less than 5/16 in. thick are required to cover oversized and short-slotted holes in external plies, when ASTM A490 or F2280 bolts of diameter larger than 1 in. are used, except as permitted by ~~per~~ Table 6.1 footnotes a and d. This was found necessary to distribute the high clamping pressure so as to prevent collapse of the hole perimeter and enable the development of the desired clamping force. Preliminary investigation has shown that a similar but less severe deformation occurs when oversized or slotted holes are in the interior plies. The reduction in clamping force may be offset by "keying," which tends to increase the resistance to slip. These effects are accentuated in *joints* of thin plies. When long-slotted holes occur in an outer ply, ⅜ in. thick plate washers or continuous bars and one ASTM F436 washer are required in Table 6.1. This requirement can be satisfied with material of any structural grade. Alternatively, either of the following options can be used:

Rationale or Justification for Change (attach additional pages as needed):
This is a continuation of change S09-028 that was approved in 2010 that added the applicability of Table 6.1 footnote "a" to the condition of A490 or F2280 bolts greater than 1" diameter when oversized or short-slotted holes exist in an outer ply. While the Specification language was updated, the inclusion of the footnote "a" reference in the supporting Commentary was missed.

------------------------------------For Committee Use Below---

Date Received: 5/3/2012 Exec Com Meeting: ________ Forwarded: Yes ☐ /No ☐

Committee Assignment: Executive -A. ☐ Editorial -B. ☐ Nominating -C. ☐

Specifications -A.1 ☐ Research -A.2 ☐ Membership & Funding -A.3 ☐ Education -A.4 ☐
Committee Chair: ____________ Task Group #: ___________ T.G. Chair: ________________
Date Sent to Main Committee: _______________Final Disposition: ______________________

RCSC Proposed Change: S12-042

Name: Tom Schlafly
E-mail: schlafly@aisc.org
Phone: 312-670-5412
Fax:

Ballot Actions:

Proposed Change:

5.4. Design Slip Resistance

Slip-critical connections shall be designed to prevent slip and for the limit states of bearing-type connections. When slip-critical bolts pass through fillers, all *faying surfaces* subject to slip shall be prepared to achieve design slip resistance.

At US LRFD or Canadian LSD load levels the design slip resistance is ϕR_n and at ASD load levels the allowable slip resistance is R_n/Ω where R_n,, ϕ and Ω are defined below.

The available slip resistance for the limit state of slip shall be determined as follows:

$$R_n = \mu D_u h_f T_b n_s k_{sc} \qquad \text{Equation 5.6}$$

For standard size and short-slotted holes perpendicular to the direction of the load

$\phi = 1.00$ (LRFD, LSD) $\Omega = 1.50$ (ASD)

For oversized and short-slotted holes parallel to the direction of the load

$\phi = 0.85$ (LRFD, LSD) $\Omega = 1.76$ (ASD)

For long-slotted holes

$\phi = 0.70$ (LRFD, LSD) $\Omega = 2.14$ (ASD)

where

μ = mean slip coefficient for Class A or B surfaces, as applicable, and determined as follows, or as established by tests:

(1) 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)

$\mu = 0.30$

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

-------------------------------------For Committee Use Below---

Date Received: ______ Exec Com Meeting: ________ Forwarded: Yes ☐ /No ☐

Committee Assignment: Executive -A. ☐ Editorial -B. ☐ Nominating -C. ☐

Specifications -A.1 X Research -A.2 ☐ Membership & Funding -A.3 ☐ Education -A.4 ☐

Committee Chair: Harrold Task Group #: ___________ T.G. Chair: Schlafly

Date Sent to Main Committee: ________________ Final Disposition: ______________________

$\mu = 0.50$

D_u = 1.13; a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension; the use of other values may be approved by the engineer of record.

T_b = minimum fastener tension given in Table 8.1, kips

h_f = factor for fillers, determined as follows:

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

$h_f = 1.0$

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

(i) For one filler between connected parts

$h_f = 1.0$

(ii) For two or more fillers between connected parts

$h_f = 0.85$

n_s = number of slip planes required to permit the connection to slip

$$k_{sc} = 1 - \frac{T_u}{D_u T_b n_b} \quad \text{(LRFD, LSD)}$$

$$= 1 - \frac{1.5T_a}{D_u T_b n_b} \quad \text{(ASD)}$$

where

T_a = required tension force using *ASD load combinations*, kips
T_u = required tension force using US *LRFD or Canadian LSD load combinations*, kips
n_b = number of bolts carrying the applied tension

~~5.4.1. At the Factored-Load Level: The design slip resistance is ϕR_n, where ϕ is as defined below and:~~

$$\cancel{R_n = \mu D_u T_m N_b \left(1 - \frac{T_u}{D_u T_m N_b}\right)} \quad \text{~~(Equation 5.6)~~}$$

~~where~~

~~ϕ = 1.0 for standard holes~~

~~= 0.85 for oversized and short-slotted holes~~

~~= 0.70 for long-slotted holes perpendicular to the direction of load~~

~~= 0.60 for long-slotted holes parallel to the direction of load;~~

~~R_n = *nominal strength* (slip resistance) of a slip plane, kips;~~

~~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);~~

~~D_u = 1.13, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension T_m; the use of other values of D_u shall be approved by the *Engineer of Record;*~~

~~T_m = specified minimum bolt pretension (for *pretensioned joints* as specified in Table 8.1), kips;~~

~~N_b = number of bolts in the *joint*; and,~~

~~T_u = *required strength* in tension (tensile component of applied factored load for combined shear and tension loading), kips~~

~~= zero if the *joint* is subject to shear only~~

~~5.4.2. At the Service-Load Level: The service-load slip resistance is ϕR_s, where ϕ is as defined in Section 5.4.1 and:~~

$$R_n = \mu D T_m N_b \left(1 - \frac{T}{D T_m N_b}\right) \qquad \text{(Equation 5.7)}$$

~~where~~

~~D = 0.80, a slip probability factor that reflects the distribution of actual slip coefficient values about the mean, the ratio of mean installed bolt pretension to the specified minimum bolt pretension, T_m, and a slip probability level; the use of other values of D must be approved by the *Engineer of Record*; and,~~

~~T = applied service load in tension (tensile component of applied service load for combined shear and tension loading), kips~~

~~= zero if the *joint* is subject to shear only~~

~~and all other variables are as defined for Equation 5.6.~~

Commentary:

~~The design check for slip resistance can be made either at the factored-load level (Section 5.4.1) or at the service-load level (Section 5.4.2). These alternatives are based upon different design philosophies, which are discussed below. They have been calibrated to produce results that are essentially the same. The factored-load level approach is~~

~~provided for the expedience of only working with factored loads. Irrespective of the approach, the limit state is based upon the prevention of slip at service-load levels.~~

~~If the factored-load provision is used, the~~ The *nominal strength* R_n represents the mean resistance, which is a function of the *mean slip coefficient* μ and the specified minimum bolt pretension (clamping force) T_m. The 1.13 multiplier in Equation 5.6 accounts for the ~~expected 13 percent higher mean value of the installed bolt pretension provided by the calibrated wrench pretensioning method compared to the specified minimum bolt pretension T_m used in the calculation.~~ statistical relationship between calculated slip resistance and historical measured test results. In the absence of other field test data, this value is used for all methods.

~~If the service-load approach is used, a probability of slip is identified. It implies that there is 90 percent reliability that slip will not occur at the calculated slip load if the calibrated wrench pretensioning method is used, or that there is 95 percent reliability that slip will not occur at the calculated slip load if the turn-of-nut pretensioning method is used. The probability of loading occurrence was not considered in developing these slip probabilities (Kulak et al., 1987; p. 135).~~

For most applications, the assumption that the slip resistance at each fastener is equal and additive with that at the other fasteners is based on the fact that all locations must develop the slip force before a total *joint* slip can occur at that plane. Similarly, the forces developed at various slip planes do not necessarily develop simultaneously, but one can assume that the full slip resistances must be mobilized at each plane before full *joint* slip can occur. ~~Equations 5.6 and 5.7 are formulated for the general case of a single slip plane. The total slip resistance of a *joint* with multiple slip planes can be calculated as that for a single slip plane multiplied by the number of slip planes.~~

The nominal resistance in 5.4 results in a reliability consistent with the reliability of structural member design. The engineer should not need to design to a higher reliability in normal structural applications. ~~Only the *Engineer of Record* can determine whether the potential slippage of a *joint* is critical at the service-load level as a serviceability consideration only or whether slippage could result in distortions of the frame such that the ability of the frame to resist the factored loads would be reduced.~~ The following comments reflect the collective thinking of the Council and are provided as guidance and an indication of the intent of the Specification (see also the Commentary to Sections 4.2 and 4.3):

(1) If *joints* with standard holes have only one or two bolts in the direction of the applied load, a small slip may occur. In this case, *joints* subject to vibration should be proportioned to resist slip ~~at the service-load level~~;
(2) 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~~to prevent slip, 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};

(3) In *joints* with long-slotted holes that are parallel to the direction of the applied load, the ~~designer has two alternatives. The~~ *joint* can be designed to prevent slip ~~in the service-load range using either the factored-load-level provision in Section 5.4.1 or the service-load-level provision in Section 5.4.2. In either case~~, however, the effect of the factored loads acting on the deformed structure (deformed by the maximum amount of slip in the long slots at all locations) must be included in the structural analysis; and,

(4) In *joints* subject to fatigue, design should be based upon service-load criteria and the design slip resistance of ~~Section 5.4.2~~ the governing cyclic design specification because fatigue is a function of the service load performance rather than that of the factored load.

Extensive data developed through research sponsored by the Council and others during the past twenty years has been statistically analyzed to provide improved information on slip probability of *joints* in which the bolts have been pretensioned to the requirements of Table 8.1. Two variables, the *mean slip coefficient* of the *faying surfaces* and the bolt pretension, were found to affect the slip resistance of *joints*. Field studies (Kulak and Birkemoe, 1993) of installed bolts in various structural applications indicate that the Table 8.1 pretensions have been achieved as anticipated in the laboratory research.

An examination of the slip-coefficient data for a wide range of surface conditions indicates that the data are distributed normally and the standard deviation is essentially the same for each surface condition class. This means that different reduction factors should be applied to classes of surfaces with different *mean slip coefficients*—the smaller the mean value of the coefficient of friction, the smaller (more severe) the appropriate reduction factor—to provide equivalent reliability of slip resistance.

The bolt clamping force data indicate that bolt pretensions are distributed normally for each pretensioning method. However, the data also indicate that the mean value of the bolt pretension is different for each method. ~~As noted previously, if~~ If the calibrated wrench method is used to pretension ASTM A325 bolts, the mean value of bolt pretension is about 1.13 times the specified minimum pretension in Table 8.1. If the turn-of-nut pretensioning method is used, the mean pretension is about 1.35 times the specified minimum pretension for ASTM A325 bolts and about 1.26 for ASTM A490 bolts.

The combined effects of the variability of the *mean slip coefficient* and bolt pretension have been accounted for approximately in the single value of the slip probability factor ~~*D*~~D_u in the equation for nominal slip resistance ~~in Section 5.4.2~~. This implies ~~90 percent reliability~~ that slip will not occur ~~if the calibrated wrench pretensioning method is used and 95 percent reliability if the turn-of-nut pretensioning method is used. For values of *D* that are appropriate for other *mean slip coefficients* and slip probabilities, refer to the *Guide* (Kulak et al., 1987; p. 135). The values given therein are suitable for direct substitution into the formula for slip resistance in Section 5.4.2.~~ with a beta of at least 2.6 regardless of the method of pretensioning.

The calibrated wrench installation method targets a specific bolt pretension, which is 5 percent greater than the specified minimum value given in Table 8.1. Thus, regardless of the actual strength of production bolts, this target value is unique for a given fastener grade. On the other hand, the turn-of-nut installation method imposes an elongation on the fastener. Consequently, the inherent strength of the bolts being installed will be reflected in the resulting pretension because this elongation will bring the fastener to its proportional limit under combined torsion and tension. As a result of these differences, the mean value and nature of the frequency distribution of pretensions for the two installation methods differ. Turn-of-nut installations result in higher mean levels of pretension than do calibrated wrench installations. Twist-off type tension control bolt and direct tension indicator pretensions are similar to those of calibrated wrench. These differences were taken into account when the design criteria for *slip-critical joints* were developed.

~~Statistical information on the pretension characteristics of bolts installed in the field using direct tension indicators and twist-off-type tension-control bolts is limited.~~

In any of the foregoing installation methods, it can be expected that a portion of the bolt assembly (the threaded portion of the bolt within the *grip* length and/or the engaged threads of the nut and bolt) will reach the inelastic region of behavior. This permanent distortion has no undesirable effect on the subsequent performance of the bolt.

Because of the greater likelihood that significant deformation can occur in *joints* with oversized or slotted holes, lower values of design slip resistance are provided for *joints* with these hole types through a modification of the resistance factor ϕ. For the case of long-slotted holes, even though the slip load is the same for loading transverse or parallel to the axis of the slot, the value for loading parallel to the axis has been further reduced, based upon judgment, in recognition of the greater consequences of slip.

Although the design philosophy for *slip-critical joints* presumes that they do not slip into bearing when subject to loads in the service range, it is mandatory that *slip-critical joints* also meet the requirements of Sections 5.1, 5.2 and 5.3. Thus, they must meet the strength requirements to resist the factored loads as *shear/bearing joints*.

Section 3.2.2(b) permits the *Engineer of Record* to authorize the use of *faying surfaces* with a *mean slip coefficient* μ that is less than 0.50 (Class B) and other than ~~0.33~~ 0.30 (Class A). This authorization requires that the *mean slip coefficient* μ must be determined in accordance with Appendix A. ~~following restrictions are met:~~

~~(1) The *mean slip coefficient* μ must be determined in accordance with Appendix A; and,~~
~~(2) The appropriate slip probability factor *D* must be selected from the *Guide* (Kulak et al., 1987) for design at the service-load level.~~

Prior to the 1994 edition of this Specification, μ for ~~Class C~~ galvanized surfaces was taken as 0.40. This value was reduced to 0.35 in the 1994 edition for better agreement with the available research (Kulak et al., 1987; pp. 78-82) and to 0.30 in the 2014 edition to be consistent with slip coefficients cited previously.

Rationale or Justification for Change (attach additional pages as needed):

Modify the RCSC equations to reflect the information that is contained in the AISC 2010 Specification. This reflects the most recent research on the subject.

Recent research by Hajjar et al, Dusicka et al and Grondin support changes to the formulation for slip resistance. The proposal results in reliability at levels acceptable for use in slip critical connections regardless of whether the slip limit state is considered to be a serviceability or strength limit.

Significant changes from the current specification include:

- The current specification includes three formulae for slip resistance
  - $R_n = \mu D_u T_m N_b \left(1 - \frac{T_u}{D_u T_m N_b}\right)$ *(Equation 5.6)*
  - $R_n = \mu D T_m N_b \left(1 - \frac{T}{D T_m N_b}\right)$ (Equation 5.7)
  - $R_n = H \mu D T_m N_b \left(1 - \frac{T}{D T_m N_b}\right)$ (Equation B5.5)

  All three equations should lead to the same number of bolts. Eq 5.6 uses LRFD loads. Eqs 5.7 and B5.5 use ASD loads. Bolt limit states can be formulated as a nominal resistance factored by a resistance factor (phi) or a safety factor (omega). This method is clear and concise and recommended as (near) future business. But it has not been adopted by RCSC so for consistency this proposal uses the nominal resistance formulation in the text and refers to it in Annex B.
- The basic slip coefficient is 0.30 instead of 0.33. This results in more uniform reliability across bolt strength levels and faying surface slip classes
- The current RCSC equation is for one slip plane but all bolts. The proposed is for any number of slip planes but one bolt.
- Caution has been added to commentary regarding galvanized surfaces because no research has been done subsequent to finding some surfaces with a low coefficient.

RCSC Proposed Change: S12-043

Name: Tom Schlafly
Phone: 312-670-5412
E-mail: schlafly@aisc.org
Fax:

Ballot Actions:

Proposed Change:

8.1. Snug-Tightened Joints

All bolt holes shall be aligned to permit insertion of the bolts without undue damage to the threads. Bolts shall be placed in all holes with washers positioned as required in Section 6.1 and nuts threaded to complete the assembly. Compacting the *joint* to the snug-tight condition shall progress systematically from the most rigid part of the *joint*. Snug tight is the condition that exists when all of the plies in a *connection* have been pulled into *firm contact* by the bolts in the *joint* and all of the bolts in the *joint* have been tightened sufficiently to prevent the removal of the nuts without the use of a wrench.

Commentary:

As discussed in the Commentary to Section 4, the bolted *joints* in most shear *connections* and in many tension *connections* can be specified as *snug-tightened joints*. The snug tightened condition is typically achieved with a few impacts of an impact wrench, application of an electric torque wrench until the wrench begins to slow or the full effort of a worker on an ordinary spud wrench. More than one cycle through the bolt pattern may be required to achieve the *snug-tightened joint*. The splines on twist-off type tension-control bolts may be twisted off or left in place in snug tightened joints.

The actual pretensions that result in individual fasteners in *snug-tightened joints* will vary from *joint* to *joint* depending upon the thickness, flatness, and degree of parallelism of the connected plies, as well as the effort applied. In most *joints*, plies of *joints* involving material of ordinary thickness and flatness can be drawn into complete contact at relatively low levels of pretension. However, in some *joints* in thick material or in material with large burrs, it may not be possible to reach continuous contact throughout the *faying surface* area as is commonly achieved in *joints* of thinner plates. This is generally not detrimental to the performance of the *joint*.

As used in Section 8.1, the term "undue damage" is intended to mean damage that would be sufficient to render the product unfit for its intended use.

-----------------------------------For Committee Use Below---

Date Received: 5/29/12 Exec Com Meeting: _______ Forwarded: Yes ☐ /No ☐

Committee Assignment: Executive -A. ☐   Editorial -B. ☐   Nominating -C. ☐

Specifications -A.1 ☐   Research -A.2 ☐ Membership & Funding -A.3 ☐   Education -A.4 ☐
Committee Chair: ____________ Task Group #: ___________ T.G. Chair: ________________
Date Sent to Main Committee: _______________Final Disposition: ______________________

Rationale or Justification for Change (attach additional pages as needed):

The proposed revision is in response to occasional inspector requirements to remove the splines of TC bolts even where they are to be snug tight.

RCSC Proposed Change: S12-044

Name: Tom Schlafly
E-mail: schlafly@aisc.org
Phone: 312-670-5412
Fax:

Ballot Actions:

Proposed Change:
{Note: Proposed Change S11-033 also makes modifications to this section. The proposed changes are mutually exclusive and should not impact technical decisions on this item.}

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 $\phi = 0.75$ and:

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

where

R_n = nominal strength (shear strength per shear plane or tensile strength) of a bolt, kips;

Table 5.1. Nominal Strengths per Unit Area of Bolts

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

[a] Except as required in Section 5.2.

[b] Reduction for values for $L_s > 38$ in. applies only when the joint is end loaded, such as splice plates on a beam or column flange.

------------------------------------For Committee Use Below---

Date Received: 5/31/12 Exec Com Meeting: 6/6/12 Forwarded: Yes ☐ /No ☐

Committee Assignment: Executive -A. ☐ Editorial -B. ☐ Nominating -C. ☐

Specifications -A.1 ☐ Research -A.2 ☐ Membership & Funding -A.3 ☐ Education -A.4 ☐
Committee Chair: ____________ Task Group #: ___________ T.G. Chair: ________________
Date Sent to Main Committee: _______________Final Disposition: ______________________

F_n = nominal strength per unit area from Table 5.1 for the appropriate applied load conditions, ksi, adjusted for the presence of fillers as required below, and,

A_b = cross-sectional area based upon the nominal diameter of bolt, in.^{2}

When a bolt that carries load passes through fillers or shims in a shear plane that are equal to or less than 1/4 in. thick, F_n from Table 5.1 shall be used without reduction. When a bolt that carries load passes through fillers or shims that are greater than 1/4 in. thick, they shall be designed in accordance with one of the following procedures:

(1) ~~For fillers or shims that are equal to or less than 3/4 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 3/4 in~~.;
(2) The fillers or shims shall be extended beyond the *joint* and the filler or shim extension shall be secured with enough bolts to uniformly distribute the total force in the connected element over the combined cross-section of the connected element and the fillers or shims;
(3) The size of the *joint* shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or,
(4) The *joint* shall be designed as a *slip-critical joint* using Class A surfaces with Turn-of-Nut pretensioning or Class B surfaces. ~~The slip resistance of the *joint* shall not be reduced for the presence of fillers or shims.~~

Commentary:
The nominal shear and tensile strengths of ASTM A325, F1852, A490 and F2280 bolts are given in Table 5.1. These values are based upon the work of a large number of researchers throughout the world, as reported in the *Guide* (Kulak et al., 1987; Tide, 2010). The *design strength* equals the *nominal strength* multiplied by a resistance factor ϕ.

The nominal shear strength is based upon the observation that the shear strength of a single *high-strength bolt* is about 0.62 times the tensile strength of that bolt (Kulak et al., 1987; pp. 44-50). In addition, a reduction factor of 0.90 is applied to joints up to 38 in. in length to account for an increase in bolt force due to minor secondary effects resulting from simplifying assumptions made in the modeling of structures that are commonly accepted in practice (e.g. truss bolted connections assumed pinned in the analysis model). Second order effects such as those resulting from the action of the applied loads on the deformed structure, should be accounted for through a second order analysis of the structure. As noted in Table 5.1, the average shear strength of bolts in *joints* longer than 38 in. in length is reduced by a factor of 0.75 instead of 0.90. This factor accounts for both the non-uniform force distribution between the bolts in a long joint and the minor secondary effects discussed above. Note that the 0.75 reduction factor does not apply in cases where the distribution of force is essentially uniform along the *joint*, such as the bolted *joints* in a shear *connection* at the end of a deep plate girder.

The average ratio of nominal shear strength for bolts with threads included in the shear plane to the nominal shear strength for bolts with threads excluded from the shear plane is 0.83 with a standard deviation of 0.03 (Frank and Yura, 1981). Conservatively, a reduction factor of 0.80 is used to account for the reduction in shear strength for a bolt with threads included in the shear plane but calculated with the area corresponding to the nominal bolt diameter. The case of a bolt in double shear with a non-threaded section in one shear plane and a threaded section in the other shear plane is not covered in this Specification for two reasons. First, the manner in which load is shared between these two dissimilar shear areas is uncertain. Second, the detailer's lack of certainty as to the orientation of the bolt placement might leave both shear planes in the threaded section. Thus, if threads are included in one shear plane, the conservative assumption is made that threads are included in all shear planes.

The tensile strength of a *high-strength bolt* is the product of its ultimate tensile strength per unit area and some area through the threaded portion. This area, called the tensile stress area, is a derived quantity that is a function of the relative thread size and pitch. For the usual sizes of structural bolts, it is about 75 percent of the nominal cross-sectional area of the bolt. Hence, the nominal tensile strengths per unit area given in Table 5.1 are 0.75 times the tensile strength of the bolt material. According to Equation 5.1, the nominal area of the bolt is then used to calculate the *design strength* in tension. The *nominal strengths* so-calculated are intended to form the basis for comparison with the externally applied bolt tension plus any additional tension that results from *prying action* that is produced by deformation of the connected elements.

If pretensioned bolts are used in a *joint* that loads the bolts in tension, the question arises as to whether the pretension and the applied tension are additive. Because the compressed parts are being unloaded during the application of the external tensile force, the increase in bolt tension is minimal until the parts separate (Kulak et al., 1987; pp. 263-266). Thus, there will be little increase in bolt force above the pretension load under service loads. After the parts separate, the bolt acts as a tension member, as expected, and its *design strength* is that given in Equation 5.1 multiplied by the resistance factor ϕ.

Pretensioned bolts have torsion present during the installation process. Once the installation is completed, any residual torsion is quite small and will disappear entirely when the fastener is loaded to the point of plate separation. Hence, there is no question of torsion-tension interaction when considering the ultimate tensile strength of a *high-strength bolt* (Kulak et al., 1987; pp. 41-47).

When required, pretension is induced in a bolt by imposing a small axial elongation during installation, as described in the Commentary to Section 8. When the *joint* is subsequently loaded in shear, tension or combined shear and tension, the bolts will undergo significant deformations prior to failure that have the effect of overriding the small axial elongation that was introduced during installation, thereby removing the pretension. Measurements taken in laboratory tests confirm that the pretension that would be sustained if the applied load were removed is essentially zero before the bolt fails in shear (Kulak et al., 1987; pp. 93-94). Thus, the shear and tensile strengths of a bolt are not affected by the presence of an initial pretension in the bolt.

See also the Commentary to Section 5.5.

Tests of 24 bolt A490 1 1/8 diameter connections indicated the reduction in bolt shear strength in connections with filler as required in section 5.1 (1) is limited to 85%. (Borello, Denavit, Hajjar Behavior of Bolted Steel Slip Critical Connections with Fillers UIUC August 2009) . Review of available data on slip critical connections revealed that connections with Class A surfaces pretensioned by Turn-of-Nut and connections with Class B surfaces provide a sufficient reliability against slip to eliminate the need to fasten the fills outside the connection or reduce the bolt shear capacity. Grondin, Ming, Josi Slip Critical Bolted Connections - A Reliability Analysis for Design at the Ultimate Limit State. University of Alberta, April 2008.

Rationale or Justification for Change (attach additional pages as needed):

The provisions governing fillers in Section 5.1 have limits and may be incorrect. Example issues include: The equation in (1) stops at ¾ in. Fillers can be thicker. There is a question about whether (4) can be considered valid if slip critical joints need to be checked for bearing. Dr Hajjar conducted a study of the effect of fillers on SC joints. Dr Grondin performed a statistical review of slip critical connections. The proposal is an outcome of those studies.

RCSC Proposed Change: S12-045

Name: Chris Curven **E-mail:** chrisc@appliedbolting.com
Phone: **802-460-3100** **Fax:**

Ballot Actions:

Proposed Changes:

8.2.3. Twist-Off-Type Tension-Control Bolt Pretensioning: Twist-off-type tension-control bolt assemblies that meet the requirements of ASTM F1852 or F2280 shall be used.

All *fastener assemblies* shall be installed in accordance with the requirements in Section 8.1 without severing the splined end and with washers positioned as required in Section 6.2. If a splined end is severed during this operation, the *fastener assembly* shall be removed and replaced. Subsequently, all bolts in the *joint* shall be ~~pretensioned~~ tightened with the twist-off-type tension-control bolt installation wrench until the splined-end shears off, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts.

Commentary:

ASTM F1852 and F2280 twist-off-type tension-control bolt assemblies have a splined end that extends beyond the threaded portion of the bolt. During installation, this splined end is gripped by a specially designed wrench chuck and provides a means for turning the nut relative to the bolt. This product is, in fact, based upon a torque-controlled installation method to which the *fastener assembly* variables affecting torque that were discussed in the Commentary to Section 8.2.2 apply, except for wrench calibration, because torque is controlled within the *fastener assembly*.

Twist-off-type tension-control bolt assemblies must be used in the as-delivered, clean, lubricated condition as specified in Section 2. Adherence to the requirements in this Specification, especially those for storage, cleanliness and verification, is necessary for their proper use.

9.2.1. Turn-of-Nut Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Section 8.2.1. Subsequently, but prior to pretensioning and optional match-marking, it shall be ensured by *routine observation* that the plies have been brought into *firm contact*. Subsequently, it shall be ensured by *routine observation* that the bolting crew properly rotates the turned element relative to the unturned element by the amount specified in Table 8.2. Alternatively, when *fastener assemblies* are match-marked after the initial

-----------------------------------For Committee Use Below---

Date Received: 5/31/12 Exec Com Meeting: 6/6/12 Forwarded: Yes ☐ /No ☐

Committee Assignment: Executive -A. ☐ Editorial -B. ☐ Nominating -C. ☐

Specifications -A.1 ☐ Research -A.2 ☐ Membership & Funding -A.3 ☐ Education -A.4 ☐
Committee Chair: ____________ Task Group #: ___________ T.G. Chair: ________________
Date Sent to Main Committee: _______________Final Disposition: ______________________

fit-up of the *joint* but prior to pretensioning, visual inspection after pretensioning is permitted in lieu of routine observation. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection.

Commentary:
Match-marking of the assembly during installation as discussed in the Commentary to Section 8.2.1 improves the ability to inspect bolts that have been pretensioned with the turn-of-nut pretensioning method. The sides of nuts and bolt heads that have been impacted sufficiently to induce the Table 8.1 minimum pretension will appear slightly peened.

The turn-of-nut pretensioning method, when properly applied and verified during the construction, provides more reliable installed pretensions than after-the-fact *inspection* testing. Therefore, proper inspection of the method is for the inspector to observe the required pre-installation verification testing of the *fastener assemblies* and the method to be used, followed by monitoring of the work in progress to ensure that the method is routinely and properly applied, or visual inspection of match-marked assemblies.

Some problems with the turn-of-nut pretensioning method have been encountered with hot-dip galvanized bolts. In some cases, the problems have been attributed to an especially effective lubricant applied by the *manufacturer* to ensure that bolts and nuts from stock will meet the ASTM Specification requirements for minimum turns testing of galvanized fasteners. Job-site testing in the *tension calibrator* demonstrated that the lubricant reduced the coefficient of friction between the bolt and nut to the degree that "the full effort of an ironworker using an ordinary spud wrench" to snug-tighten the *joint* actually induced the full required pretension. Also, because the nuts could be removed with an ordinary spud wrench, they were erroneously judged by the *inspector* to be improperly pretensioned. Excessively lubricated *high-strength bolts* may require significantly less torque to induce the specified pretension. The required pre-installation verification will reveal this potential problem.

Conversely, the absence of lubrication or lack of proper over-tapping can cause seizing of the nut and bolt threads, which will result in a twist failure of the bolt at less than the specified pretension. For such situations, the use of a *tension calibrator* to check the bolt assemblies to be installed will be helpful in establishing the need for lubrication.

9.2.2. Calibrated Wrench Pretensioning: The *inspector* shall observe the daily pre-installation verification testing required in Section 8.2.2. Subsequently, but prior to pretensioning, it shall be ensured by *routine observation* that the plies have been brought into *firm contact*. Subsequently, it shall be ensured by *routine observation* that the bolting crew properly applies the calibrated wrench to the turned element. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection.

Commentary:

For proper inspection of the method, it is necessary for the *inspector* to observe the required pre-installation verification testing of the *fastener assemblies* and the method to be used, followed by monitoring of the work in progress to ensure that the method is routinely and properly applied within the limits on time between removal from *protected storage* and final pretensioning.

9.2.3. Twist-Off-Type Tension-Control Bolt Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Section 8.2.3. Subsequently, but prior to pretensioning, it shall be ensured by *routine observation* that the plies have been brought into *firm contact* without the splined ends being severed. If the splined end is severed, the bolt must be removed and replaced. Subsequently, it shall be ensured by *routine observation* that the splined ends are properly severed during installation by the bolting crew. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection.

Commentary:
The sheared-off splined end of an installed twist-off-type tension-control bolt assembly merely signifies that at some time the bolt was subjected to a torque that was adequate to cause the shearing. If in fact all fasteners are individually pretensioned in a single continuous operation without first properly snug-tightening all fasteners, they may give a misleading indication that the bolts have been properly pretensioned. Therefore, it is necessary that the *inspector* observe the required pre-installation verification testing of the *fastener assemblies*, and the ability to apply partial tension prior to twist-off is demonstrated. This is followed by monitoring of the work in progress to ensure that the method is routinely and properly applied within the limits on time between removal from *protected storage* and final twist-off of the splined end.

Rationale or Justification for Change (attach additional pages as needed):

8.2.3 does not actually state when the installer is to stop tightening or when the bolt is deemed tight. It states what type of installation tool to be used, but not what the installer is looking for.
For example, 8.2.1. states to rotate the head or nut as specified in table 8.2., 8.2.2. states to apply the installation torque determined by the pre-installation verification, and 8.2.4. has the installer making sure the achieved gap is less than the job inspection gap.

Also, Section 9.2.4. is the only installation method that has the inspector verify that snugging of the bolts and plies have taken place before the chosen pretensioning method takes place. 9.2.1., 9.2.2.,and 9.2.3. would obviously like to have inspection of the snug condition, but it is not listed.
For example, 9.2.4. …All bolts shall be installed in accordance with the requirements in Section 8.1, with washers positioned as required in Section 6.2. The installer shall verify that the direct-tension-indicator protrusions have not been compressed to a gap that is less than the job inspection gap during this operation, and if this has occurred, the direct tension indicator shall be removed and replaced.…

RCSC Proposed Change: S12-046

Name: Chris Curven **E-mail:** _chrisc@appliedbolting.com
Phone: 802-460-3100 **Fax:**

Ballot History:

Proposed Change:

Glossary

{All existing terms in Glossary remain unchanged.}

Torque (noun). **1.** The moment of a force; the measure of a force's tendency to produce torsion and rotation about an axis, equal to the vector product of the radius vector from the axis of rotation to the point of application of the force and the force vector.
2. A turning or twisting force.
(Both copied from The Free Dictionary by Farlex)
3. A rotational moment; it is a measure of how much twisting is applied to a fastener.
(Copied from boltscience.com)

Torque (verb). to impart a twisting force. *(copied from The Free Dictionary by Farlex)*

Tension. A bolt resistance to elongation that provides a clamping in a bolted connection.

Rationale or Justification for Change:

Torque and tension are the two basic terms used in structural bolting with the term torque being used predominantly. However, in the field and in offices, their definitions and physical differences are not understood. The users of this specification would be well served if we provide them with a definition.

I am not committed to any of the definitions I have offered, but merely would like to use them as a starting point so we CAN include them in the glossary of the specification.

------------------------------------For Committee Use Below---

Date Received: 5/31/12 Exec Com Meeting: 6/6/12 Forwarded: Yes ☐ /No ☐

Committee Assignment: Executive -A. ☐   Editorial -B. ☐   Nominating -C. ☐

Specifications -A.1 ☐   Research -A.2 ☐ Membership & Funding -A.3 ☐ Education -A.4 ☐
Committee Chair: ____________ Task Group #: ___________ T.G. Chair: ________________
Date Sent to Main Committee: _______________Final Disposition: ______________________

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 ≤ 38	54	68
	L > 38	45	56
	Threads excluded from shear plane		
	L ≤ 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:

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

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

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

$$[\text{S.I.: } 1 - 0.0154(t - 6)]$$

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 $\phi = 0.75$ and:

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

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

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

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

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 $\phi = 0.75$ and:

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

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 $[1-0.4(t'-0.25)]$. 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).