CALIFORNIA COUNCIL OF TESTING AND INSPECTION AGENCIES

March 24, 2016
Four Points by Sheraton
5115 Hopyard Road
Pleasanton, CA 94588

Minutes - General Business Meeting

1. Call to Order – Tim Rodriguez
   a. Time
      • *The meeting was called to order at 3:10 p.m. by President Tim Rodriguez.*
   b. Self-introductions
      • *The following members were in attendance:*

Jim Auser (BSK Associates)
Tim Casey (Construction Testing Services)
Elizabeth Clarke (Structure Groups)
Cliff Craig (Structure Groups)
Miki Craig (CCTIA)
Terry Egland (Testing Engineers, Inc.)
Tim Rodriguez (BSK Associates)
August Smarkel (Mid Pacific Engineering, Inc.)
Colin Stock (Terracon)

2. Approval of Minutes
   a. February 25, 2016
      • *The minutes were approved as corrected (typographical errors).*

3. Financial Report
   a. Income Statement *(handout)*
      • *Executive Secretary Miki Craig provided a copy of the Income Statement through February 29, 2016, evidencing receipts totaling $14,300.00 and expenses of $2,953.74, leaving net reserves of $11,346.26.*
   b. Balance of Account
      • *The balance in the checking account at February 29th was $22,459.41.*
   c. Status of 2016 Dues Payments
      • *Executive Secretary Miki Craig reported she had emailed reminders to ENGEO Incorporated and Reliant Testing Engineers, Inc. Julia Moriarty (ENGEO) responded she would put the item before the Executive Committee for approval.*

4. Committee Reports
   a. ASTM – Jeffry Cannon *(handouts)*
      • *President Rodriguez reported he had connected with Chair Cannon, who indicated he would attend next month's meeting. Chair Cannon provided a listing of revised ASTM Standard, and wanted to bring specific attention to D421 and D422. His recap included the following comments:*

Handout of ASTM Updates

- *Since D421 and D422 have been withdrawn, sieve analysis of soils will have to be performed in accordance with D6913. It is recommended that labs become accredited for this method. Even though D6913 has been in existence for twelve years, there are very few labs who have this method in their scope of accreditation. AMRL has been providing assessment and accreditation for this test method for a number of years.*
- *There is a "new" standard on soil density that most people don't know about: D7263, Laboratory Determination of Density (Unit Weight) of Soil Specimens. This is a method for performing what a lot of us call "moisture/density" from tube samples (from soil investigation drilling). There are two methods in D7263: Method A and Method B. Method A is for wax coating soil and weighing it under water (similar to a Caltrans AC core specific gravity); probably something most of us will rarely do. But Method B is exactly what a lot of labs do for determining the moisture content and dry density (unit weight) of soil out of brass, stainless steel,*

and shelby tubes. AMRL does not provide assessments for this method yet, but it will likely come in the future. The standard was first published in 2009, but few people know of it.

- *Director Terry Egland reported all six (6) high strength bolt standards are been replaced by ASTM F3125-15. He provided an excerpt for review.*
- *Director Egland reported ASTM C1077 is once again under revision. The committee is working on terms, including removing "registered" engineer and replacing with "licensed" engineer. He also noted certifiers are having difficulty meeting the current language of the standard, so revisions are in the works.*
- *ASTM C1798 is a new standard under development, and relates to the reuse of concrete returned to the batch plant. In order for this new standard to apply, it must be referenced in C94. It is still a work item, so Director Egland has not seen the proposed language yet.*
- *ASTM is currently balloting changes address the inspection requires addressed in E7134 on firestopping.*
- *Last on his list, Director Egland noted ASTM E2265 has a work item addressing the terminology of anchors and fasteners in concrete. Some new terms will be added, and one needs to be revised.*

b. SEAONC CQA – Terry Egland *(handouts)*
   - *Director Egland provided copies of four (4) FAQs the CQA Committee has submitted to the SEAONC Board for approval to publish in upcoming newsletters.*
   - *The committee is still discussing the various Statement of Special Inspections in use, and analyzing where to go with the information. They spent a considerable amount of time reviewing the City of San Francisco's version, which is very old and outdated - most jurisdictions have the same issue. Member Cliff Craig commented CCTIA member's best course of action is just to follow the CBC and ICC Special Inspection Manual. He strongly recommended obtaining a copy of the ICC document if a firm did not have one.*
   - *Mr. Art Dell will be giving a presentation on special inspection and structural observation in San Francisco on April 13th. Information is available on SEAONC's website.*

c. FAQs – Colin Stock
   - *Director Egland reported he has a listing of 14 questions – all pertaining to bolting. He would like to get member's input, including volunteers to take on addressing and/or collecting responses.*

d. NCAWNV ACI Certification – Tim Casey/Cliff Craig
   - *No report*

e. Caltrans JTP Work Group – Jim Auser
   - *Chair Auser is working on getting to the correct person to express interest in joining the work group.*

f. DSA – Augie Smarkel
   - *Chair Augie Smarkel reported he had been in contact with Eric France, who had requested samples of the criteria local jurisdictions were using to recognize agencies. Executive Secretary Craig had provided this for him to forward. Mr. France noted he would review the information to determine if there was someplace that DSA's program could fit in. It was noted that since the increase in the fee, the LEA program is ridiculously expensive and no longer of value as a program. Additionally, the compliance criteria are not consistent between local jurisdictions and DSA, forcing the testing agencies to consistently cope with the burden of greater requirements. Substituting local programs with the DSA program would also necessitate getting multiple offices LEA-approved at the exorbitant fee. A member commented that the jurisdictional criteria is still the least expensive way for any testing agency to receive recognition to provide services. A member in attendance noted his opposition to jurisdictions utilizing the LEA program, as they may no longer accept alternative submittals, thereby creating unnecessary higher standards of practice. Several attendees agreed with keeping the regulations down as much as possible. Discussion turned to justification of DSA's increased fee, and who is really getting value for it. Most felt the cost of DSA's accreditation program should be included in the building permit fees. Chair Smarkel was requested to respond to Mr. France that the CCTIA membership does not see any value in promoting the LEA program outside of DSA's arena.*

g. Membership – Jim Backman/Elizabeth Clarke *(handout)*

- *President Rodriguez reported Vice President Mark Hahle had requested a membership application for Condor Earth Technologies.*
- *Past President Elizabeth Clarke provided a first page draft of a two-page flyer, focusing on getting people to the new website and the value of membership (networking, PDHs, etc.) It was suggested the document be turned to a portrait layout, rather than landscape.*

h. Communications – Tim Rodriguez

- *President Rodriguez inquired if anyone had looked at the new website. Feedback is welcome for everyone on all parts of it. Past President Clarke liked the pop-up to enter an email address to be added to the mailing list. Members may login using their LinkedIn account, and it will make the user a subscriber so they can post on the discussions. There is also a button under each post to share on the user's LinkedIn account.*
- *The updated site is based on WordPress, and allows for event registration (including non-members), and will add the event to the registrant's calendar, provide a Google map, as well as limit the number of attendees.*
- *The Member Tools page is currently restricted to registered members.*
- *An online "Contact Us" page has been added to the site. President Rodriguez would also like to add an FAQ submittal form.*
- *Past President Clarke suggested the "About Us" posting should be placed higher in the hierarchy.*
- *It was noted the 2016 membership application should be added to the site. President Rodriguez will look into the possibility of that being an online fillable form. Discussion included the option to accept credit card payments; however, Executive Secretary Craig cautioned that doing so through PayPal is an expensive proposition for the small amount of activity the Council would generate. She was asked to research PayPal and other potential providers.*

i. Professional Development – Elizabeth Clarke

- *No activity at this time*

j. Programs – Elizabeth Clarke

- *Past President Clarke will prepare a flyer advertising Mr. Tim Hart's seminar on special inspection of wood construction. Executive Secretary Craig will provide a mailing list for local jurisdictions, and Past President Clarke will take care of the mailing.*
- *Executive Secretary Craig reported State Architect, Chet Widom, had committed to being the guest speaker for the June and September meetings.*

5. Old Business

a. CCTIA Meeting Locations and Times – Tim Rodriguez

- *President Rodriguez reported Member Jeffry Cannon had suggested the Council have more Sacramento meetings, and hold the meetings earlier in the day. Stockton might also be a viable location.*
- *Executive Secretary Craig was instruction to research Four Points by Sheraton facility options in the Valley locations.*

b. ICC's Preferred Provider Program – Miki Craig *(handout)*

- *As requested, Executive Secretary Craig obtained a copy of the ICC Preferred Provider Program Manual for the attendee's review. Upon discussion, it was determined the program was not worthwhile for CCTIA to pursue.*

6. New Business

a. Coring of Concrete Masonry Walls: Is It Necessary? – Cliff Craig *(handouts)*

- *Member Cliff Craig led the discussion pertaining to two recently published articles. He noted masonry core testing applies to DSA work only – no one else requires it. These articles explain the background of the testing requirement, and the current belief that it is no longer of benefit and should be abandoned. SEAONC's CQA*

committee agreed to consider how it might support this as a commentary. SEAONC's Seismology Committee is looking into it to see how it might partner in this effort, and Member Craig is awaiting a response. A member questioned why we would want to give up the billing opportunity. Member Craig noted the problems that are created when there is a failure – including whether or not the coring process was performed correctly, how the testing was conducted, and other factors calling the testing agency into question. He reiterated that the test procedure was developed for masonry that is no longer in use. Director Egland commented DSA has not ignored the industry opinion, and it has moderated their enforcement of the requirement (excluded from smaller structures), but it is unwilling to give it up (once relinquished, it can never bring it back). Director Egland believed firing these articles at DSA would a good idea, as it might force it to finally abandon the requirement. If the Seismology Committee commits to taking up the fight, CCTIA will place the issue on its June or September agenda, with advance notification to DSA via Chair Augie Smarkel.

b. 2018 IBC Code Change Proposals – Terry Egland
   - *CCTIA members may view the proposed code changes via a link provided by Director Egland. Executive Secretary Craig was requested to send it via email to the membership, and President Rodriguez will add it to the website.*

c. New ICC Affiliate – Terry Egland *(handout)*
   - *Director Egland reported ICC has announced a new member of the ICC companies – the Solar Rating & Certification Corporation.*

7. Adjournment
   a. Time
      - *There being no further business, the meeting was adjourned at 5:30 p.m. by President Tim Rodriguez.*
   b. Next meeting
      - *The next meeting will be April 28, 2016, 1:00 p.m. at the Four Points by Sheraton in Pleasanton. Structural Engineer Tim Hart will present "Special Inspection Requirements for Wood Construction".*

Respectfully submitted,
Miki Craig
Executive Secretary

CCTIA
Operating Statement

	2016	
	Actual YTD 2/29/16	***Budget YE 12/31/16***
Revenues		
Dues & Initiation Revenues	$12,350.00	$13,650
ABM Registrations	1,950.00	900
ABM Sponsorships		500
General Meeting Guest Registrations		
Education Programs		
Total Revenues	*14,300.00*	*15,050*
Expenses		
Education Programs		
Executive Secretary Services		500
Hemsley Award Expenses		200
ABM Expenses	2,942.96	9,500
General Meetings		10,000
Newsletter		
Office Supplies		100
Postage	10.78	50
S I Guidelines		
Stationary & Printing		
Taxes & Licenses		20
Website		120
Total Expenses	*2,953.74*	*20,490*
Net Reserves/(Losses)	***$11,346.26***	***($5,440)***

Year-to-Date as of
February 29, 2016

Revised ASTM Standards

Standard	Title	Revisions Made
C11 – 15	Standard Terminology Relating to Gypsum and Related Building Materials and Systems	Revised definition of "All Purpose Compound"
C31 – 15	Standard Practice for Making and Curing Concrete Test Specimens in the Field	Revised rodding of beams, vibrating cylinders
C39 – 15a	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens	Clarified permissible time tolerance for testing specimens at different ages
C78 – 15a	Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)	Revised criteria for testing machine
C94 – 15	Standard Specification for Ready-Mixed Concrete	Many revisions throughout, especially in ordering requirements. Deleted "mild" as an exposure condition for air-entrained concrete. Revised Appendix X.1 on how to calculate average compressive strengths.
C114 – 15	Standard Test Methods for Chemical Analysis of Hydraulic Cement	One minor edit to Table 1, for permissible chloride
C125 – 15a	Standard Terminology Relating to Concrete and Concrete Aggregates	Revised the definitions for admixture, accelerating admixture, and retarding admixture. Added definition of concrete, pervious. Added terms age, equivalent; factor, temperature-time; and temperature, datum.
C127 – 15	Standard Test Method for Relative Density (Specific Gravity) and Absorption of Coarse Aggregate	Unknown changes
C128 – 15	Standard Test Method for Relative Density (Specific Gravity) and Absorption of Fine Aggregate	Unknown changes
C143 – 15	Standard Test Method for Slump of Hydraulic-Cement Concrete	Deleted the single comparative test to check suspected out of tolerance mold.
C151 – 15	Standard Test Method for Autoclave Expansion of Hydraulic Cement	Modification to autoclave apparatus, and safety section
C183 – 15	Standard Practice for Sampling and the Amount of Testing of Hydraulic Cement	Unknown changes
C186 – 15a	Standard Test Method for Heat of Hydration of Hydraulic Cement	Unknown changes
C192 – 15	Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory	Vibrator frequency has been increased to at least 9,000 vibrations per min. (Increased from 7,000 v/m)
C470 – 15	Standard Specification for Molds for Forming Concrete Test Cylinders Vertically	Added section on packaging. Manufacturer must label as single use, be marked with lot number or date, and have an arrow indicating orientation of the vertical axis. Added Note 9 saying shipping and storing single-use molds with their axis in the vertical position reduces the incidence of distortion.
C494 – 15	Standard Specification for Chemical Admixtures for Concrete	Sections 12.2.2, 14.4, 17.1.3, and 17.1.4 have been revised to indicate that the average results from the reference and test batches are to be compared. Section on specific gravity of liquid admixtures has been revised.

C617 – 15	Standard Practice for Capping Cylindrical Concrete Specimens	Revised temperatures of molten sulfur to 130 to 145°C (265 to 290°F); from 129 to 143 °C (265 to 290 °F)
C666 – 15	Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing	Added a section discussing the differences between Procedures A and B.
C670 – 15	Standard Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials	Unknown changes
C897 – 15	Standard Specification for Aggregate for Job-Mixed Portland Cement-Based Plasters	Minor revision to Scope
C900 – 15	Standard Test Method for Pullout Strength of Hardened Concrete	Revised "Significance and Use" section
C926 – 15	Standard Specification for Application of Portland Cement-Based Plaster	Minor edits throughout
C1077 – 15	Standard Practice for Agencies Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Testing Agency Evaluation	Revised referenced documents. Added lists of relevant tests laboratory supervisor and technicians must be certified for.
C1218 – 15	Standard Test Method for Water-Soluble Chloride in Mortar and Concrete	Apparatus section updated.
C1240 – 15	Standard Specification for Silica Fume Used in Cementitious Mixtures	Unknown changes
C1489 – 15	Standard Specification for Lime Putty for Structural Purposes	Unknown changes
C1708 – 15	Standard Test Methods for Self-leveling Mortars Containing Hydraulic Cements	Revised the section on physical property, length change
C1753 – 15	Standard Practice for Evaluating Early Hydration of Hydraulic Cementitious Mixtures Using Thermal Measurements	Unknown changes
C1758 – 15	Standard Practice for Fabricating Test Specimens with Self-Consolidating Concrete	Revised apparatus, sampling, and test procedures
C1761 – 15	Standard Specification for Lightweight Aggregate for Internal Curing of Concrete	Unknown changes
C1777 – 15	Standard Test Method for Rapid Determination of the Methylene Blue Value for Fine Aggregate or Mineral Filler Using a Colorimeter	Unknown changes
C1803 – 15	Standard Guide for Abrasion Resistance of Mortar Surfaces Using a Rotary Platform Abraser	Unknown changes
D421 – 85 (2007)	Standard Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants	Withdrawn in 2016 with no replacement
D422 – 85 (2007)	Standard Test Method for Particle-Size Analysis of Soils	Withdrawn in 2016. Sieve analysis portion has been replaced by D6913. Hydrometer portion is to be replaced by a future standard that is still in the process of final approval.

ASTM F3125-15 Standard Specification for High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength, Inch and Metric Dimensions

Active Standard(Latest Version)

Abstract

Scope

1.1 This specification covers chemical, physical and mechanical requirements for quenched and tempered bolts manufactured from steel and alloy steel, in inch and metric dimensions, in two strength grades, two types and two styles.

1.1.1 This specification is a consolidation and replacement of six ASTM standards, including; A325, A325M, A490, A490M, F1852 and F2280.

1.1.2 This consolidated standard is to ensure alignment between standards with the same intended end use and to simplify the use and maintenance of structural bolt specifications.

1.2 Intended Use:

1.2.1 Bolts manufactured under this specification are intended for use in structural connections covered in the Specification for Structural Joints Using High-Strength Bolts, as approved by the Research Council on Structural Connections.

1.2.2 Bolts in this specification are furnished in sizes from ½ to 1-½ in. inclusive and from M12 to M36 inclusive.

1.3 Classification:

1.3.1 Bolts are designated by grade or property class, which indicates inch or metric respectively.

1.3.2 Bolts are designated by type denoting raw material chemical composition.

1.3.3 Bolts are designated by style denoting Heavy Hex bolts or "Twist-Off" Style assemblies.

Grade	Min. Strength	Type		Style
A325	120 ksi	1	3	Heavy Hex Head
A325M	830 MPa	1	3	Heavy Hex Head
F1852	120 ksi	1	3	Twist-Off
A490	150 ksi	1	3	Heavy Hex Head
A490M	1040 MPa	1	3	Heavy Hex Head
F2280	150 ksi	1	3	Twist-Off

Type 1 - 120 ksi (830 MPa) - carbon steel, carbon boron steel, alloy steel or alloy steel with boron addition

Type 3 - 120 ksi (830 MPa) or 150 ksi (1040 MPa) - weathering steel

Type 1 - 150 ksi (1040 MPa) - alloy steel or alloy steel with boron addition

C1077-15a Standard Practice for

Agencies Testing Concrete and Concrete Aggregates for Use in Construction and

Criteria for Testing Agency Evaluation

Work Item #: WK53421

Ballot Action: Revision of Section 6.1.1 and add a new 6.1.1.1 of C1077

Rationale: Licensed is the term used by the National Council of Examiners for Engineering and Surveying and 6.1.1.1 is being added to clarify duties of the professional engineer. The new Section 6.1.1.1 was rewritten based on negatives comments from previous ballot.

6. Personnel Qualifications

6.1 Information shall be made available to substantiate personnel qualifications as follows:

6.1.1 All relevant testing services are provided under the full-time technical direction of a ~~registered~~ licensed professional engineer with at least 5 years experience in construction materials testing.

6.1.1.1 As used in this standard, full-time technical direction means that the professional engineer, licensed according to applicable jurisdictional requirements, is in responsible charge of the work performed by the testing agency. The professional engineer shall provide objective evidence to the evaluation authority to substantiate the adequacy of the degree of technical direction and control being provided.

Work Item #: WK53459

Ballot Action: Revision of C1077, Sections 4 and 6

Rationale: 6.1.5's current language does not fit very well for certification programs that construct their written examinations using variable questions pulled from a pool of questions covering several different sections. By defining very short sections in the standards that must be included each time such as "significance," the standard makes it difficult to not use the same questions on all examinations. This makes it easier for individuals to share results, which would reduce the effectiveness of the certification programs. This section was reworded to better describe the most widely used certification programs that are accepted in concrete testing. The individual certification program can construct their exams in such a way as to satisfy the word "sufficient" in 6.1.5.1 for their own program. However, when an examination is constructed in a way that renders it ineffective, the user of the standard may add their own criteria to improve the effectiveness of an individual examination. A change was made to the significance and use section of this standard to widen the ability of users to add requirements to way this standard is being used.

The word "technician" was removed throughout because the certification program is being required for positions other than technicians. The word "body" was removed from 6.1.5.4 because the standard defines the use of a "program" rather than "body" throughout.

ASTM WK53818

Revision of C94 / C94M - 15b Standard Specification for Ready-Mixed Concrete

(What is a Work Item?)

Active Standard: C94 / C94M - 15b

Developed by Subcommittee: C09.40 | Committee C09 | Contact Staff Manager

WK53818

1. Rationale

ASTM C09 Main Committee recently approved new standard WK39876, Specification for Returned Fresh Concrete for Use in a New Batch of Ready-Mixed Concrete, to be published as ASTM C1798. This standard provides requirements for the use of returned fresh concrete, but does not itself permit the use of returned fresh concrete. Therefore, the following ballot item proposes to revise C94/94M to address permission to use returned fresh concrete.

Keywords

accuracy; blended hydraulic cement; certification; ready-mixed concrete; scales; testing ;

ASTM WK53495

Revision of E2174 - 14b Standard Practice for On-Site Inspection of Installed Firestops

(What is a Work Item?)

Active Standard: E2174 - 14b

Developed by Subcommittee: E06.21 | Committee E06 | Contact Staff Manager

WK53495

1. Rationale

There are several revisions developed by the task group that need to be balloted. As the industry learns more about inspections and the role of the inspector and inspection agency, the standard will need to be revised.

Keywords

firestop; inspection; inspector;;

Work Item Status

Date Initiated:
02-23-2016

Technical Contact:
William Mchugh

Status:
Draft Under Development

ASTM WK53498

Revision of E2265 - 09 Standard Terminology for Anchors and Fasteners in Concrete and Masonry

(What is a Work Item?)

Active Standard: E2265 - 09

Developed by Subcommittee: E06.13 | Committee E06 | Contact Staff Manager

WK53498

1. Rationale

The subcommittee has determined that a number of terms need to be added to the standard, and that one term currently found in the standard needs to be revised.

Keywords

Anchors/anchorage systems; Concrete; Concrete anchors/anchorage systems; Fasteners (structural); Masonry assemblages; Terminology–building materials/applications;

Work Item Status

Date Initiated:
02-23-2016

Technical Contact:
Chris Lavine

Item:
003

Ballot:
E06.13 (16-01)

Status:
In Balloting

CQA - FAQ

FAQ 10.003

PREQUALIFIED COLUMN SPLICE

Q Does anyone have an opinion on whether Figure 1 below would be a pre-qualified joint per AWS (American Welding Society) D1.1? Note there is only a weld access hole in the top tier column, and thus the backing bar does not actually lap past the joint. D1.1 is silent on the actual width of backing. All of the pre-qualified joints with backing shown in the Figures 3.3 and 3.4 show the backing lapping past the joint in butt joints (see prequalified joint B-U4a from Figure 3.4 shown below), but no minimum dimension is ever shown or discussed. In a Tee joint the backing of course stops right at the edge of the joint, but there is continuous base metal along that edge in that case. In the configuration shown below, except at the column web there is nothing behind the intersection of the corners of the backing and the bottom tier column flange.

Submitted by Art Dell, P.E. in San Francisco, California

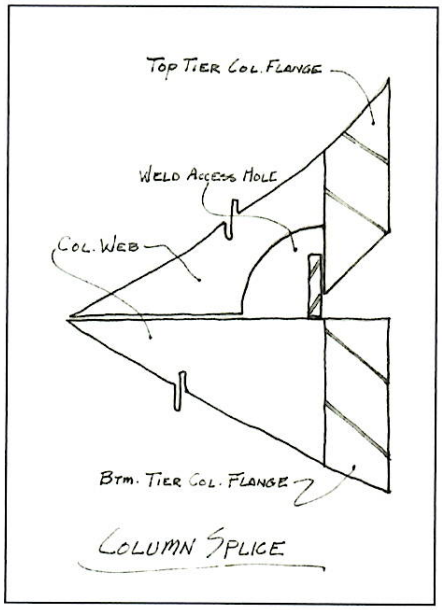


Figure 1

"Got a question, comment or tip?
E-mail it to: cqa@seaonc.org"

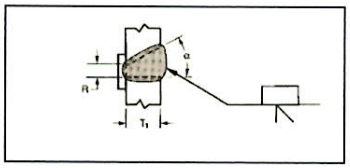
A **Informal Response Submitted by Andrew Davis**
"This does not constitute an official position of the American Welding Society or the D1 Committee on Structural Welding"

The code is silent on minimum edge overlap of backing. However, no overlap is not permitted. The overlap should be at least the thickness of the backing bar.

I am told that accepted practice would be to put a weld access hole in the lower shaft web and center the backing. One could also cut a tight notch in the backing to "swallow" the web and center the rest of backing bar at the root opening (see Figure 2 below). However, the EOR (Engineer of Record) should review and approve this approach.

The joint as shown in Figure1 is not prequalified.

Andrew Davis is the Director, International Activities with AWS American Welding Society. He can be reached at adavis@aws.org



Prequalified Joint B-U4a from Figure 3.4

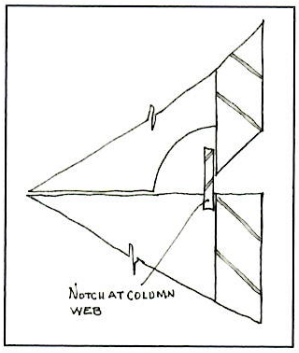


Figure 2

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Construction Quality Assurance Committee in cooperation with California Council of Testing and Inspection Agencies

Committee approved March 16, 2016

CQA - FAQ

FAQ 10.004

WELDING INSPECTION OF TEMPORARY ERECTION/BRACING AIDE

Q On a school project the DSE (District Structural Engineer) issued a non-compliance report stating that we did not provide welding inspection during the welding of temporary erection/bracing aides. The temporary aides were not shown on the approved drawing. The DSE's concern was that since we did not provide welding inspection including verifying any required preheat, the welding might have damaged the integrity of the structural members. While the DSE had a valid concern does the code require temporary welding to be inspected the same as permanent welding?

Submitted by Inspection Agency in Southern California

A **Response Submitted by Mike Clarke**

This happens quite often during the erection of a steel building and is typically guide aides for the setting and connecting of column splices between floors. It also takes place at the perimeter of the buildings at each floor level when a ring is welded to the exterior column to retain the safety cables for fall prevention. There are two things to think about here. First, I believe that the DSE's concern wasn't the amount of heat input but rather the fact there these areas were not properly preheated in accordance with any WPS (Welding Procedure Specification) or the AWS (American Welding Society) D1.1 at a minimum. The second item is that the D1.1 does not state that temporary welds do not require inspection. AWS D1.1 states that temporary welds are subject to the same WPS requirements as the final welds. That statement in itself requires the inspector to check that the WPS requirements are met i.e. the welder is certified, the amps, volts and other essential variables are followed including preheat. If the member was a thicker section that requires 150 or 225 degree preheat and the preheating was not performed this could cause some delayed cracking issues in the future even if the temporary welds are removed and ground flush with the base metal. Tack welds are very small and will cool almost instantly due to their small size and are very prone to cracking. One fact that I constantly need to point out to other project inspectors, special inspectors, contractors and especially welding shops is that the special inspector must be present during fabrication including the tack welding of parts. Most shops will begin fabrication and tack pieces together prior to the special inspector being present, this is not allowed, especially on a school project with DSA (Division of State Architect) oversight.

Mike Clarke is President of Structure Consultants, an AWS Certified Welding Inspector and a member of CCTIA. He can be reached at mike@structureconsultants.com

Published by STRUCTURAL ENGINEERS ASSOCIATION OF NORTHERN CALIFORNIA
Construction Quality Assurance Committee in cooperation with California Council of Testing and Inspection Agencies

Committee approval March 16, 2016

"Got a question, comment or tip?
E-mail it to: cqa@SEAONC.org"

CQA - FAQ

FAQ10.005

SPECIAL INSPECTION FOR R-3 SHEAR WALL

Q Does a 4 in. and 12 in. nailing pattern for R-3 construction (dwellings, including lodging houses; and congregate residences and large family day-care homes) require Special Inspection for wind resistance as specified in CBC 1705.10.1 and seismic 1705.11.2?

Submitted by David Knell, wilsonknell@gmail.com

Response Submitted by Tim Hart, S.E.

A It depends on how the building was designed. Many R-3 buildings are designed using the California Residential Code rather than the California Building Code, in which case the provisions of Chapter 17 (including wood special inspections) would not apply. In addition, wood structures that are designed per the conventional light frame provisions in Section 2308 of the California Building Code (as some R-3 buildings are) are also exempt from special inspections. There is an additional exemption from seismic special inspections in CBC Section 1705.11 for one and two family detached dwellings with no more than 2 stories and no structural irregularities. R-3 buildings could be one of these as well. Note that there is not a similar exemption for the other special inspections required for wood construction, including those required for buildings in high wind areas in CBC Section 1705.10. Finally, there are the exemptions in CBC Sections 1705.10 and 1705.11 for buildings in low wind areas and buildings with Seismic Design Categories of A or B.

If the building is not exempt from special inspection per these provisions, then special inspection of the shear wall would be required. The exemption for nail spacing is for shear walls and diaphragms with nail spacing greater than 4 inches. The special inspection would include more than just the 4 inch nailing. It would also include the wall sheathing, the anchor bolts, the holdowns, the chords and collectors in the wall, and the connections to the floor and roof diaphragms.

Tim Hart is a Civil/Structural Engineer at Lawrence Berkeley National Laboratory and a registered Structural Engineer in California. He can be reached at thart@lbl.gov

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Construction Quality Assurance Committee in cooperation with California Council of Testing and Inspection Agencies

Approved March 16, 2016

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CQA - FAQ

FAQ 10.006

WELDER QUAIFICATIONS for AWS D1.4-11

Q AWS D1.4-11 (rebar welding code) requires a Procedure Qualification Record (PQR) for different positions of weld. The question is: can the 3G vertical position qualification be used for Welding Procedure Specification (WPS) that includes 1G flat position and 2G horizontal positions without further qualification. The reading of Table 6.2 states that that 3G qualifies for 1G, 2G and 3G. Section 6.3.4, the welder qualification section, does not directly apply to WPS's, so Table 6.2 may not apply for the requirement in Section 6.2.3 that states a WPS shall be required for each production welding position. Please provide guidance for interpreting the apparent discrepancies in these two sections consistent with the intent of the code.

Submitted by Dan Watanabe with Testing Engineers, Inc. in San Leandro, California

A **Response Submitted by Steve Borrero**

I received a response from the Chairman of the subcommittee responsible for the maintenance of the D1.4 code.

Here's what he had to say:

A Procedure Qualification Record (PQR) qualified in the 3G position, qualifies WPS for the 1G, 2G and 3G positions.

Please note that this is not an official response from AWS and is his technical opinion. If you wish to receive an official interpretation, please follow the annex in the back of the code that describes how to request for an official interpretation.

Steve Borrero is Program Manager II Technical Services Division at AWS. He can be reached at sborrero@aws.org

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E-mail it to: cqa@seaonc.org"

TERMINOLOGY

PQR - Procedure Qualification Record
WPS - Welding Procedure Specification

1G – Welding in a flat position
2G – Welding in a horizontal position
3G – Welding in a vertical position
4G – Welding in an overhead position (not discussed in this article)

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Bolting Questions

1. In a weather exposed slip critical connection can the steel be hot-dip-galvanized?
2. Where do I find the torque required to fully tension a high-strength bolt?
3. Can you explain the need for a slip-critical connection?
4. What is the definition of snug-tight bolt installation?
5. Can you describe a "Calibration device" utilized for bolt tensioning?
6. What are the various methods for fully tensioning high-strength bolts?
7. How is the head of a high-strength bolt attached?
8. How are the threads of a high-strength bolt formed?
9. I have an anchor bolt application which requires a high tensile strength. Should ASTM F1554 or A449 be specified?
10. Is the turn-of-the-nut method the correct installation for snug-tight connection?

11. Does the code require different hole diameters for a tension controlled bolt (TC), normal bolt, or slip-critical bolt (SC)?

13. What guidelines should be followed for reusing HSB?

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CALIFORNIA COUNCIL OF TESTING AND INSPECTION AGENCIES

2016 Meeting Schedule

Date	*Description*	*Location*
February 25, 2016	BOD Meeting (1:00 pm)	Four Points, Pleasanton
	General Meeting (3:00 pm)	
March 24, 2016	General Meeting (3:00 pm)	Four Points, Pleasanton
April 28, 2016	General Meeting (3:00 pm)	Four Points, Pleasanton
May 19, 2016	General Meeting (3:00 pm)	Four Points, Pleasanton
June 23, 2016	Meeting w/DSA (12:00 pm)	Four Points, Sacramento
July 28, 2016	General Meeting (3:00 pm)	Four Points, Pleasanton
August 25, 2016	General Meeting (3:00 pm)	Four Points, Pleasanton
September 23, 2016 (Friday)	Meeting w/DSA (12:00 pm)	Four Points, LAX
October 27, 2016	BOD Meeting (12:00 pm)	Four Points, Pleasanton
	General Meeting (3:00 pm)	
November 17, 2016	General Meeting (3:00 pm)	Four Points, Pleasanton
December 15, 2016	General Meeting (3:00 pm)	Four Points, Pleasanton
January 21, 2017 (Saturday)	Annual Business Meeting	Four Points, Pleasanton
	Installation Dinner	McNamara's, Dublin

Note: *World of Concrete – January 17-20, 2017, Las Vegas, NV*
Super Bowl LI – February 5, 2017, Houston, TX

STRUCTURAL TESTING

issues and advances related to structural testing

Coring of Concrete Masonry Walls: Is it Necessary?

By Richard M. Bennett, Ph.D., P.E.

The 1933 Long Beach earthquake showed that unreinforced double-wythe masonry brick walls did not perform well. Consequently, California regulators imposed a requirement that double-wythe brick masonry be reinforced and grouted, and that the newly constructed masonry be destructively tested by drilling a core specimen horizontally through the wall to test the bond between the clay masonry unit and grout for shear capacity. The bond criteria for grout to masonry unit was arbitrarily set at 100 psi. In 1983, the bond criteria was changed to $2.5\sqrt{f'_m}$ psi, a value nearly equal to 100 psi.

Over the past 75 years, the requirement has morphed into application to single-wythe hollow unit masonry walls which was never the intent of the provision and ignores the benefit of webs and tapers in Concrete Masonry Units (CMU). Additionally, there is discussion at the national level on whether or not destructively coring and testing the masonry cores is a worthwhile effort. The following analysis is based on current code provisions and puts the discussion into a rational perspective.

When a reinforced masonry wall is subjected to out-of-plane loads, the tension is carried by the reinforcement and the compression by the masonry. In this context, the masonry is a combination of masonry units, mortar, and grout. There are also shear stresses in the wall. The shear stresses are both perpendicular to the face of the wall, as well as parallel to the face of the wall. The shear stresses parallel to the face of the wall are similar to those that develop between the structural steel and the concrete in a composite steel/concrete slab beam. The stresses in the cross-section are shown in *Figure 1*.

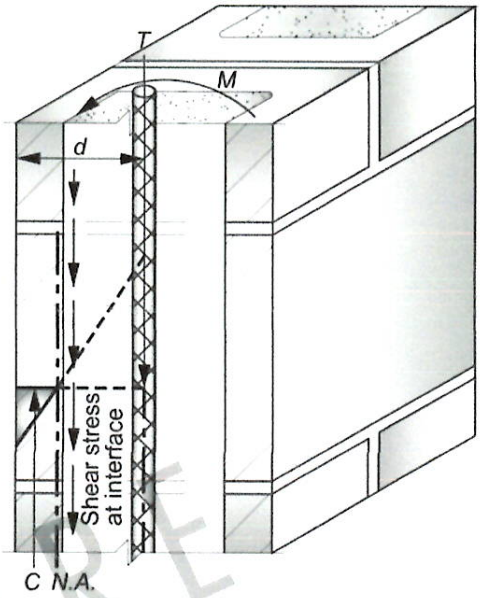

Figure 1. Stresses in a reinforced masonry wall.

Figure 2. Illustration of the destructive nature of coring. The first attempt hit reinforcement causing further damage.

The Masonry Society (TMS) 402 Code, *Building Code Requirements for Masonry Structures*, requires the wall to be designed to carry the shear forces perpendicular to the face of wall (2013 TMS 402 Section 8.3.5 for Allowable Stress Design, and Section 9.3.5.3 for Strength Design). There are no requirements in TMS 402 with regard to the shear stresses parallel to the face of the wall. However, the California Division of State Architect and the California Office of Statewide Health Planning and Development have requirements for core testing of masonry walls. The minimum average unit shear interface requirement between the grout and face shell has been arbitrarily set at $2.5\sqrt{f'_m}$ psi. This requirement is presumably to verify that there is sufficient bond between the grout and the masonry unit to carry the shear stresses. The coring, shown in *Figure 2*, demonstrates the destructive nature of the testing. The question is whether this coring is necessary, and whether TMS 402 should even consider a similar requirement.

To answer the question on the necessity of coring, a variety of wall configurations were analyzed. All walls were considered to be fully grouted and simply supported. The analysis procedure was as follows:

1) Select a wall height, block size, reinforcement bar size, reinforcement bar spacing, axial load, and a specified compressive strength, f'_m. Type S Portland cement-lime mortar was assumed for all

Richard M. Bennett is a professor of Civil and Environmental Engineering and the Director of Engineering Fundamentals at the University of Tennessee, Knoxville. He served as Chairman of the Flexural and Axial Loads Subcommittee of the TMS 402/602 Code Committee from 2004 to 2010. From 2010 to 2013, he was the Vice-Chair of the Main committee and is currently Chair of the 2016 TMS 402/602 Committee. He can be reached at ***rmbennett@utk.edu****.*

walls. Wall weights were determined based on 125 pcf units, although this assumption has a negligible effect on the results. The axial load was assumed to act concentric with the wall. Any eccentricity to the axial load would reduce the out-of-plane load the wall could carry.

In some cases, loads were unrealistically high, being several hundred psf, but the load was still used.

2) The wall was analyzed using the "slender wall procedure", Section 9.3.5.4.2 of the 2013 TMS 402 Code, to determine the maximum out-of-plane load the wall could carry. In some cases, loads were unrealistically high, being several hundred psf, but the load was still used.
3) Based on the maximum out-of-plane load, the maximum shear force was calculated. From the maximum shear force, the shear stress at the interface between the grout and face shell was calculated. If the wall is treated as a traditional composite section, and the equivalent rectangular stress block is in the face shell, the shear force at the grout/face shell interface will be based on the yield force of the steel. If part of the equivalent rectangular stress block were in the grouted core, the shear stress at the interface would be reduced. The shear stress can be obtained as the shear force divided by the shear area over half the wall height.

Typical results are shown in the *Table*. The first set of results is for the bar spacing being varied, increasing from 16 inches up to 32 inches. The second set of results is for different height walls. Many other conditions were examined, including varying the axial load, varying f'_m, varying the eccentricity of the axial load, and examining 12-inch walls with the bars offset from the center. Similar results were obtained in all cases.

A review of these results shows that the shear stress increases as wall height decreases. The highest shear stress is less than 20 psi for a 10-foot high wall, a typical story height. Most masonry walls are at least 10 feet high and the resultant shear stress was for an out-of-plane load

Typical results of analysis of a variety of wall configurations.

Height (ft)	t (inch)	Axial (k/ft)	w_u (psf)	Bar Size (#)	Bar Spacing (inch)	f'_m (psi)	Shear (lb)	Shear stress (psi)
12	7.625	1	195.8	5	16	2000	1175	16.0
12	7.625	1	139.1	5	24	2000	835	10.7
12	7.625	1	110.3	5	32	2000	662	8.0
10	7.625	1	270.5	5	16	2000	1353	19.2
12	7.625	1	195.8	5	16	2000	1175	16.0
16	7.625	1	114.0	5	16	2000	912	12.0
20	7.625	1	72.5	5	16	2000	725	9.6

of close to 400 psf, an unrealistically high out-of-plane load.

To summarize, the analyses made several conservative assumptions, resulting in a very conservative analysis. To review, the conservative assumptions were:

1) The axial load is considered to act concentrically, resulting in the largest shear force for a given moment capacity.
2) The wall is loaded to the maximum out-of-plane load that it can carry. Typically, due to discrete reinforcement sizes and spacings, and prescriptive reinforcement requirements, walls are not loaded to the maximum out-of-plane capacity.
3) Any interlocking due to offset webs, block taper, etc. was neglected. The shear surface was considered to be planar.

Even with a very conservative analysis, the maximum shear stress was only 19.2 psi. The 19.2 psi was for a 10-foot high wall with unrealistically high out-of-plane loads. Under typical load conditions, the shear stress was 16 psi or less. This shear stress is much less than the 100 psi that was the initial arbitrary California requirement, and also much less than $2.5\sqrt{f'_m}$, which would be about 97 psi for f'_m = 1500 psi and 112 psi for f'_m = 2000 psi.

Based on the above results, two conclusions can be drawn.

1) No core testing is required. The shear stresses are very low. Additionally, the above analysis does not consider the benefit of the homogeneous concrete masonry unit which has a continuous connection between the cross web and face shell, taper of the CMU or interlock of overhanging mortar fins.
2) TMS 402 is justified in not requiring designers to check the shear stress at the grout/face shell interface. That will not control the design.

The complete report along with calculations and expanded tables can be viewed online at **http://cmacn.org/PDF/Masonry_Chron_Winter_2016.pdf**.▪

Engineering Notes for Design
with Concrete Block Masonry

MASONRY CHRONICLES

Winter
2015-16

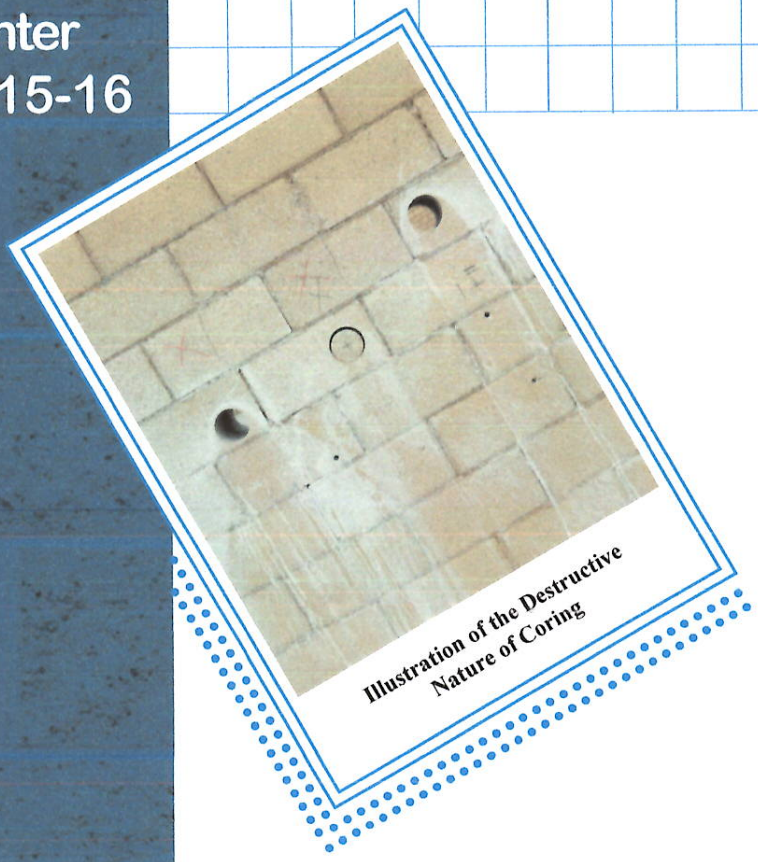

Illustration of the Destructive Nature of Coring

Coring of Concrete Masonry Walls: Is it Necessary?

Introduction

The 1933 Long Beach earthquake showed that unreinforced double-wythe masonry brick walls did not perform well. Consequently, California regulators imposed a requirement that double-wythe brick masonry be reinforced and grouted and that the newly constructed masonry be destructively tested by drilling a core specimen horizontally through the wall and that the bond between the clay masonry unit and grout be tested for shear capacity. The bond criteria for grout to masonry unit was arbitrarily set at 100 psi. In 1983, the bond criteria was changed to $2.5\sqrt{f'_m}$ psi, a value nearly equal to 100 psi.

Over the past 75 years, the requirement has morphed into application to single-wythe hollow unit masonry walls, which was never the intent of the provision and ignores the benefit of webs and tapers in Concrete Masonry Units. Additionally, there is discussion at the national level on whether or not destructively coring and testing the masonry cores is a worthwhile effort. The following analysis is based on current code provisions and puts the discussion into a rational perspective.

When a reinforced masonry wall is subjected to out-of-plane loads, the tension is carried by the reinforcement, and the compression by the masonry, Figure 1. In this context, the masonry is a combination of masonry units, mortar, and grout. There are also shear stresses in the wall. The shear stresses are both perpendicular to the face of the wall, as well as parallel to the face of the wall. The shear stresses parallel to the face of the wall are similar to those that develop between the structural steel and the concrete in a composite steel/concrete slab beam. The stresses in the cross-section are shown in Figure 2.

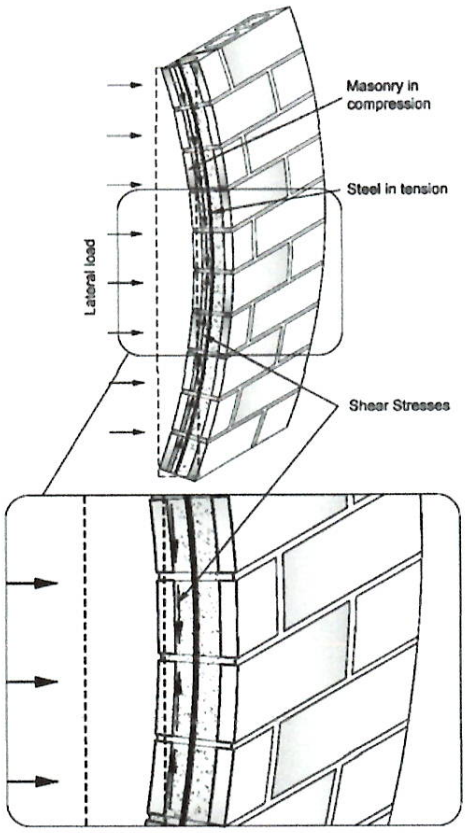

Figure 1. Masonry wall subjected to out-of-plane load

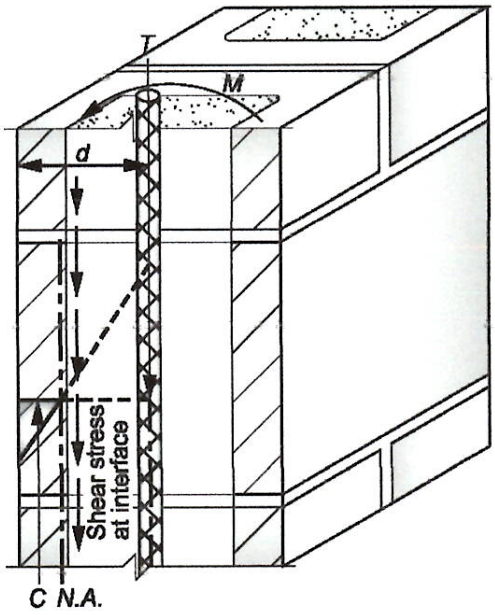

Figure 2. Stresses in a reinforced masonry wall

The TMS 402 Code, Building Code Requirements for Masonry Structures, requires the wall to be designed to carry the shear forces perpendicular to the face of wall (2013 TMS 402 Section 8.3.5 for Allowable Stress Design, and Section 9.3.5.3 for Strength Design). There are no requirements in TMS 402 with regard to the shear stresses parallel to the face of the wall. However, the California Division of State Architect and the California Office of Statewide Health Planning and Development have requirements for core testing of masonry walls. The minimum average unit shear interface requirement between the grout and face shell has been arbitrarily set at $2.5\sqrt{f'_m}$ psi. This requirement is presumably to verify that there is sufficient bond between the grout and the masonry unit to carry the shear stresses. The coring, shown in Figure 3, demonstrates the destructive nature of the testing. The question is whether this coring is necessary, and whether TMS 402 should even consider a similar requirement.

Figure 3. Illustration of the Destructive Nature of Coring
In many cases, as in photo 1 of 3, the first attempt hits reinforcement causing further damage.

To answer the question on the necessity of coring, a variety of wall configurations were analyzed. All walls were considered to be fully grouted and simply supported. The analysis procedure was as follows:

1. Select a wall height, block size, reinforcement bar size, reinforcement bar spacing, axial load, and a specified compressive strength, f'_m. Type S Portland cement-lime mortar was assumed for all walls. Wall weights were determined based on 125 pcf units, although this assumption has a negligible effect on the results. The axial load was assumed to act concentric with the wall. Any eccentricity to the axial load would reduce the out-of-plane load the wall could carry.
2. The wall was analyzed using the "slender wall procedure", Sections 9.3.5.4.2 of the 2013 TMS 402 Code, to determine the maximum out-of-plane load the wall could carry. In some cases, loads were unrealistically high, being several hundred psf, but the load was still used.
3. Based on the maximum out-of-plane load, the maximum shear force was calculated. From the maximum shear force, the shear stress at the interface between the grout and face shell was calculated. If the wall is treated as a traditional composite section, and the equivalent rectangular stress block is in the face shell, the shear force at the grout/face shell interface will be based on the yield force of the steel. If part of the equivalent rectangular stress block were in the grouted core, the shear stress at the interface would be reduced. The shear stress can be obtained as the shear force divided by the shear area over half the wall height.

In some cases, loads were unrealistically high, being several hundred psf, but the load was still used.

A sample calculation is shown below.

Given: 20 ft high 8 inch CMU fully grouted wall; concentric dead load of 0.2 k/ft; #5 Grade 60 bars at 16 inches; f'_m=2000 psi.

Required: Determine maximum out-of-plane load using 2013 TMS 402 Section 9.3.5.4.2. Calculate shear stress at grout/face shell interface.

Solution: Based on a spreadsheet calculation, the maximum out-of-plane load is 40.7 psf. Check this value. Use load combination 0.9D+E. The spreadsheet checks all load combinations and for higher axial loads, 1.2D+E will often control.

Use a wall weight of 81 psf (ASCE 7, 125 pcf units)
Based on an out-of-plane load of 72.6 psf, determine S_{DS}.

$0.4S_{DS}(\text{weight}_{\text{wall}}) = 72.6$ psf (ASCE 7, Section 12.11.1) $S_{DS} = 2.24$

$P_u = P_{uw} + P_{uf} = (0.9\text{-}0.2S_{DS})[(81\text{psf})(20/2)\text{ft} + 200\text{lb/ft}] = 456$ lb/ft

For fully grouted wall, $A_n = 91.5$ in^2/ft; $I_n = 443.3$ in^4/ft (from NCMA TEK 14-1B)
Find M_{cr}. Modulus of rupture, $f_r = 163$ psi.

$$M_{cr} = \left(f_r + \frac{P}{A_n}\right)\left(\frac{I_n}{t_{sp}/2}\right) = \left(163\text{ lb/in.}^2 + \frac{456\text{ lb/ft}}{91.5\text{ in.}^2/\text{ft}}\right)\left(\frac{443.3\text{ in}^4/\text{ft}}{7.63/2\text{ in.}}\right) = 19{,}520\text{ lb} - \text{in./ft}$$

Find I_{cr} (2013 TMS 402 Equations 9-34 and 9-35).

$A_s = 0.31\text{in}^2/16\text{in.}(12\text{in./ft}) = 0.232\text{in}^2/\text{ft}$

$n = E_s/E_m = 29000000\text{psi}/(900\times2000\text{psi}) = 16.1$

$$c = \frac{A_s f_y + P_u}{0.64 f_m' b} = \frac{0.232 \text{ in}^2/\text{ft} \times 60{,}000 \text{ psi} + 456 \text{ lb/ft}}{0.64 \times 2{,}000 \text{psi} \times 12 \text{in./ft}} = 0.936 in.$$

$$I_{cr} = n\left(A_s + \frac{P_u}{f_y}\frac{t_{sp}}{2d} \right)(d-c)^2 + \frac{bc^3}{3}$$

$$= 16.1\left(0.232 \tfrac{\text{in.}^2}{\text{ft}} + \frac{456 \tfrac{\text{lb}}{\text{ft}}}{60000 \tfrac{\text{lb}}{\text{in}^2}} \right)\left(\frac{7.63 \text{ in.}}{2} - 0.936 \text{ in.} \right)^2 + \frac{12 \tfrac{\text{in.}}{\text{ft}} \times 0.936^3 \text{ in.}^3}{3} = 35.2 \text{ in.}^4/\text{ft}$$

Use solution to simultaneous equations of 2013 TMS 402 Equations 9-27 and 9-29 to find M_u. Since the axial load is concentric, $e_u = 0$.

$$M_u = \frac{\frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + \frac{5 M_{cr} P_u h^2}{48 E_m}\left(\frac{1}{I_n} - \frac{1}{I_{cr}} \right)}{1 - \frac{5 P_u h^2}{48 E_m I_{cr}}}$$

$$= \frac{\frac{72.6\text{psf}(20\text{ft})^2}{8}\frac{12\text{in}}{\text{ft}} + \frac{5(19{,}520 \tfrac{\text{lb-in}}{\text{ft}})(456 \tfrac{\text{lb}}{\text{ft}})(240\text{in})^2}{48(1{,}800{,}000\text{psi})}\left(\frac{1}{443.3 \tfrac{\text{in}^4}{\text{ft}}} - \frac{1}{35.2 \tfrac{\text{in}^4}{\text{ft}}} \right)}{1 - \frac{5(456 \tfrac{\text{lb}}{\text{ft}})(240\text{in})^2}{48(1{,}800{,}000\text{psi})(35.2 \tfrac{\text{in}^4}{\text{ft}})}} = 44600 \frac{\text{lb} - \text{in}}{\text{ft}}$$

Compare to capacity, 2013 TMS Commentary 9.3.5.2.

$$a = \frac{A_s f_y + P_u/\phi}{0.80 f_m' b} = \frac{0.232 \tfrac{\text{in.}^2}{\text{ft}}(60000\text{psi}) + 456 \tfrac{\text{lb}}{\text{ft}}/0.9}{0.80(2000\text{psi})(12 \tfrac{\text{in}}{\text{ft}}.)} = 0.751\text{in.}$$

$$M_n = \left(P_u/\phi + A_s f_y \right)\left(d - \frac{a}{2} \right)$$

$$= \left(456 \tfrac{\text{lb}}{\text{ft}}/0.9 + 0.232 \tfrac{\text{in.}^2}{\text{ft}}(60000\text{psi}) \right)\left(3.81\text{in.} - \frac{0.751\text{in.}}{2} \right) = 49500 \frac{\text{lb} - \text{in}}{\text{ft}}$$

$\phi M_n = 0.9(49500 \text{ lb-in/ft}) = 44600 \text{ lb-in/ft} = M_u = 44600 \text{ lb-in/ft}$

This checks, and the maximum out-of-plane load the wall can carry is 72.6 psf.
Based on an out-of-plane load of 72.6 psf, the factored shear force is 72.6 psf(10ft) = 726 lb/ft.

Determine the shear stress.

$$f_v = \frac{A_s f_y}{b(h/2)} = \frac{0.232 \tfrac{\text{in}^3}{\text{ft}}(60000\text{psi})}{12 \tfrac{\text{in}}{\text{ft}}\left(\frac{240\text{in.}}{2} \right)} = 9.6\text{psi}$$

The first set of results examines an 8 inch CMU wall with #5@16 inches. The wall height and the wall axial load were varied. The maximum axial load was 5 kip/ft. This is a high axial load for most masonry structures, and there would typically only be a load this high in a multi-story bearing wall building. Above an axial load of 5 kip/ft, the equivalent rectangular stress block would no longer be in the face shell. If part of the equivalent rectangular stress block were in the grouted core, the shear stress at the interface would be reduced. Note that the shear stress is constant for a given height since the shear stress is just a function of the yield force in the reinforcement.

Height (ft)	t (inch)	Axial (k/ft)	w_u (psf)	Bar Size (#)	Bar Spacing (inch)	f'm (psi)	Shear (lb)	Shear stress (psi)
20	7.625	0.2	72.6	5	16	2000	726	9.6
20	7.625	1	72.5	5	16	2000	725	9.6
20	7.625	5	62.4	5	16	2000	624	9.6
16	7.625	0.2	113.4	5	16	2000	907	12.0
16	7.625	1	114.0	5	16	2000	912	12.0
16	7.625	5	116.5	5	16	2000	932	12.0
12	7.625	0.2	198.2	5	16	2000	1189	16.0
12	7.625	1	195.8	5	16	2000	1175	16.0
12	7.625	5	186.1	5	16	2000	1117	16.0
8	7.625	0.2	425.4	5	16	2000	1702	24.0
8	7.625	1	392.7	5	16	2000	1571	24.0
8	7.625	5	298.6	5	16	2000	1194	24.0

In looking at these results, analysis shows that the shear stress increases as wall height decreases. The highest shear stress is 24 psi, which is for an 8 ft tall wall. An 8 ft wall is very short, and most masonry walls are at least 10 ft high. This shear stress was also for an out-of-plane load of at least 299 psf, an unrealistically high out-of-plane load.

The second set of results is for varying f'_m with the height and axial load held constant at 12 ft and 1 k/ft, respectively. The primary effect of f'_m is on the out-of-plane load. The shear stress remains constant as it is just a function of the yield strength of the reinforcement, and the height of the wall.

Height (ft)	t (inch)	Axial (k/ft)	w_u (psf)	Bar Size (#)	Bar Spacing (inch)	f'm (psi)	Shear (lb)	Shear stress (psi)
12	7.625	1	195.8	5	16	2000	1175	16.0
12	7.625	1	199.2	5	16	2500	1195	16.0
12	7.625	1	189.9	5	16	1500	1139	16.0

The third set of results is for the bar spacing increasing from 16 inches up to 32 inches with the height and axial load held constant at 12 ft and 1 k/ft, respectively.

Height (ft)	t (inch)	Axial (k/ft)	w_u (psf)	Bar Size (#)	Bar Spacing (inch)	f'm (psi)	Shear (lb)	Shear stress (psi)
12	7.625	1	195.8	5	16	2000	1175	16.0
12	7.625	1	139.1	5	24	2000	835	10.7
12	7.625	1	110.3	5	32	2000	662	8.0

The fourth set of results is for varying eccentricity, e, of the axial load at the top of the wall. The bar spacing is 16 inches and the height and axial load are held constant at 12 ft and 1 k/ft, respectively. Increasing eccentricity decreases the shear force.

Height (ft)	t (inch)	Axial (k/ft)	e (inch)	w_u (psf)	Bar Size (#)	Bar Spacing (inch)	f'm (psi)	Shear (lb)	Shear stress (psi)
12	7.625	1	0	195.8	5	16	2000	1175	16.0
12	7.625	1	3	190.1	5	16	2000	1141	16.0
12	7.625	1	12	172.9	5	16	2000	1037	16.0

The fifth set of results is for a 12 inch CMU wall with the bars offset (d=9.5 inches). This would increase the flexural strength and the out-of-plane load on the wall would increase. Again, a 12 ft high wall with 1 kip/ft axial load was used, and the reinforcement spacing was varied. The shear stress again is 16 psi for a 16 inch spacing of the reinforcement, but in this case a 489 psf (3.4 psi) out-of-plane load is required to develop the shear stress of 16 psi. There is no realistic scenario for that level of loading.

Height (ft)	t (inch)	Axial (k/ft)	w_u (psf)	Bar Size (#)	Bar Spacing (inch)	f'm (psi)	Shear (lb)	Shear stress (psi)
12	11.625	1	263.5	5	32	2000	1581	8.0
12	11.625	1	339	5	24	2000	2034	10.7
12	11.625	1	488.9	5	16	2000	2933	16.0

The final set of results is for a 32 ft high wall. Due to the height of the wall, #6 vertical reinforcement at 16 inches is used in order to carry the out-of-plane load. The shear stress is only 8.6 psi.

Height (ft)	t (inch)	Axial (k/ft)	w_u (psf)	Bar Size (#)	Bar Spacing (inch)	f'm (psi)	Shear (lb)	Shear stress (psi)
32	7.625	1	25.7	6	16	2000	411.2	8.6
32	7.625	1	29.8	6	16	3000	476.8	8.6

To summarize, the analyses made several conservative assumptions, resulting in a very conservative analysis. To review, the conservative assumptions were:

1. The axial load is considered to act concentrically, resulting in the largest shear force for a given moment capacity.
2. The wall is loaded to the maximum out-of-plane that it can carry. Typically, due to discrete reinforcement sizes and spacings, and prescriptive reinforcement requirements, walls are not loaded to the maximum out-of-plane capacity.
3. Any interlocking due to offset webs, block taper, etc. was neglected. The shear surface was considered to be planar.

Even with a very conservative analysis, the maximum shear stress was only 24 psi. The 24 psi was for an 8 ft high wall with unrealistically high out-of-plane loads. Under typical load conditions, the shear stress was 16 psi or less. This shear stress is much less than the 100 psi that was the initial arbitrary California requirement, and also much less than $2.5\sqrt{f'_m}$, which would be about 97 psi for f'_m=1500 psi and 112 psi for f'_m =2000 psi.

Based on the above results, two conclusions can be drawn.

1. No core testing is required. The shear stresses are very low. Additionally, the above analysis does not consider the benefit of the homogeneous concrete masonry unit which has a continuous connection between the cross web and face shell
2. TMS 402 is justified in not requiring designers to check the shear stress at the grout/face shell interface. That will not control the design.

This issue of "Masonry Chronicles" was written by:

Richard Bennett, PhD, PE
Professor, Civil and Environmental Engineering
The University of Tennessee, Knoxville
Chair, 2016 TMS 402/602 Code Committee

About the Author:

RICHARD M. BENNETT, PhD, PE, is a professor of Civil and Environmental Engineering, and the Director of Engineering Fundamentals, at the University of Tennessee, Knoxville. He received his PhD from the University of Illinois, Urbana-Champaign. He served as Chairman of the Flexural and Axial Loads Subcommittee of the TMS 402/602 Code Committee from 2004 to 2010. From 2010 to 2013, he was the Vice-Chair of the Main committee and is currently Chair of the 2016 TMS 402/602 Committee.

Concrete Masonry Association
of California and Nevada
6060 Sunrise Vista Drive, Suite 1990
Citrus Heights, CA 95610
(916) 722-1700
info@cmacn.org
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> HOME
> ABOUT US
> CERTIFICATION & LISTING PROGRAMS
> APPLY FOR CERTIFICATION
> CERTIFICATION & LISTING DIRECTORY
> CERTIFICATION MARKS
> STANDARDS
> GUIDELINES & POLICIES
> OTHER SERVICES
> RESOURCES
> TEST LABS
> CONTACT US

ABOUT US - GENERAL

The modern solar industry was founded in 1974, following the original oil embargo of the previous year. In the years that followed, energy in all its forms became a national priority.

While commercial and residential solar systems were available on a limited regional basis, a mechanism had to be put in place that would encourage consumers to purchase these relatively unknown products and new technology with confidence, showing that the technology was valid and that the products would perform in terms of energy and dollar savings.

States with potentially large solar markets, such as California and Florida, were the first to establish such a mechanism by establishing state testing and rating programs for solar collectors. However, since there was little consistency between each state's testing requirements and approach to rating solar equipment, such programs soon became an impediment to manufacturers who marketed in more than one state.

It became evident that there was a need for a single, national program that would allow manufacturers to rate and test the efficiency of their equipment. This would also benefit consumers by providing a uniform, national approach for rating and comparing solar equipment. In an unprecedented move, the trade association for the solar energy industry and a national consortium of state energy offices and regulatory bodies collaborated to lay the groundwork for such a program, which would soon lead to the founding of the Solar Rating & Certification Corporation (SRCC).

In 1980 the SRCC was incorporated as a non-profit organization whose primary purpose was the development and implementation of national rating standards and certification programs for solar energy equipment. On November 20, 2014, the **International Code Council** (ICC) and the

Solar Rating & Certification Corporation (SRCC) completed a period of due diligence and formalized the consolidation of the two associations. SRCC is now part of the ICC Family of Companies that includes the **ICC Evaluation Service** (ICC-ES) and the **International Accreditation Service** (IAS).

The corporation is a non-profit, independent third-party certification entity that is wholly funded through fees paid by participants and users. It is unique in that it is the only national certification program established solely for solar thermal products. It is also the only national certification organization whose programs are the direct result of the combined efforts of state organizations and an industry association involved in the administration of standards.

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500 New Jersey Avenue, NW - 6th Floor, Washington, DC 20001
Telephone: (888) 422-7233 Ext. 7735