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# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-CR- 001 | |
| **Technical Contact/Email:** Jonathon R. Merk / jon@forrestassociate.com | |
| **Public Comment Number:** 2022 Comment # 49 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **10/26/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #49:**

We recently had a project where partial grout was used onsite as a bar positioner in select cells during construction in a toothed wall intersection, but the grout lift height is defined in TMS 602 commentary section 3.5 D as "the height to which grout is placed into masonry in one continuous operation." By that definition, grout should not be packed / used intermittently as a means of bar positioning. The grout lift definition appears only in the commentary of TMS 602. Specification TMS 602 3.4 B.1 states that bars must be "supported" to prevent displacement during grout placement, but it does not limit the ways that this can be accomplished. The accompanying commentary 3.4.c requires that "there is sufficient clearance for grout and mortar to surround reinforcement, ties, and anchors so stresses are properly transferred." Arguably, partial grouted bar positioning prevents proper consolidation for the final grout pour does not provide 'sufficient clearance' around the bars, but without a codified definition of grout lift height, there is nothing to prevent the contractor from packing grout to hold bars in place. Consider adding the definition of 'grout lift height' to chapter 2 to require grout to be placed in one continuous operation, as intended.

**Response/Rationale:**

To summarize the situation described in the Public Comment, a mason dry packed a fist sized clump of grout around a portion of the vertical reinforcement to position / support the bar.

After discussion, CR offers the following response. TMS 602 Article 2.6 B.2 requires grout be mixed to a consistency that has a slump between 8" and 11". The dry packed grout certainly failed to meet this requirement. CR also agrees with the commenter that the dry packed grout prevented proper consolidation around the bar when the mason placed the grout within that pour. CR believes the language within 2.6 B.2 and 3.4 B sufficiently address this situation.

Lastly, both grout lift and grout pour are defined in TMS 602 Article 1.2. Since grout lift and grout pour are not mentioned within TMS 402, it would be inappropriate to add these definitions to that document.

Therefore, no changes are proposed in response to this Public Comment.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:** N/A

**Code Commentary:** N/A

**Specification:** N/A

**Specification Commentary:** N/A

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 11 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 2 | *Did not vote* |

**Subcommittee Comments:** N/A

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| Item #: 20-CR- 002 | |
| Technical Contact/Email: Jonathon R. Merk / jon@forrestassociate.com | |
| Public Comment Number: 2022 Comment # 108 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 10/26/2021 |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #108:**

As a matter of clarification, the Specification indicates that grout pours 12 inches or less do not require reconsolidation, yet the commentary suggests that (all) grout needs to be reconsolidated. Please clarify so that Specification and commentary are consistent.

**Response/Rationale:**

CR concurs and offers the following revision.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:** N/A

**Code Commentary:** N/A

**Specification:** N/A

**Specification Commentary:**

**3.5 E.** *Consolidation* – Except for self-consolidating grout, consolidation<u>, and reconsolidation when the pour exceeds 12”,</u> is necessary to achieve complete filling of the grout space.

| Subcommittee Vote: | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:** N/A

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| Item #: 20-CR- 003 | |
| Technical Contact/Email: Jonathon R. Merk / jon@forrestassociate.com | |
| Public Comment Number: 2022 Comment # 134 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 10/26/2021 |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #134:**

Delete the two sentences after the first sentence. There are multiple responsible persons, (engineer, architect, building official, inspection agency). Individuals move and sometimes die. Projects continue often for years. Additionally, the first sentence identifies the requirement. The next two identify the procedure, which should be left to the design team to fit the needs of the project.

(Note: This comment is in regards to the last paragraph of the commentary on pg. 51 of the 10/26/21 draft.)

**Response/Rationale:**

CR appreciates the comment but are proposing no changes in response. While it is true that projects can last for years and individuals change positions / move / die / etc., the firm or entity employing the individual(s) will appoint a successor to fill the void left behind. CR believes it is important for all parties on a project to know who that is. Lastly, CR does not believe the existing language impedes a design team's ability to specify procedures to fit the needs of a project.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:** N/A

**Code Commentary:** N/A

**Specification:** N/A

**Specification Commentary:** N/A

| Subcommittee Vote: | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:** N/A

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|

**Item #:** **20-CR- 004**

**Technical Contact/Email:** Jonathon R. Merk / jon@forrestassociate.com

**Public Comment Number:** 2022 Comment # 138

| Public Comment Response Based on TMS 402/602 Draft Dated | 10/26/2021 |
|---|---|

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #138:**

The verbiage for the addition of water for ready-mixed grout is extremely unclear. After contacting The Masonry Society for clarification in June, we propose new verbiage for Section 3.5 A. The new verbiage proposed for the code provision is as follows:

3.5 A. Placing time - Place grout within 1½ hr from introducing water in the mixture and prior to initial set.

1. After the initial mixing of materials, discard site-mixed grout (grout prepared at the jobsite) that does not meet specified slump. Additional water shall not be added to the site-mixed grout after the completion of initial mixing to adjust slump.

2. For ready-mixed grout:

a. At truck arrival, check slump either visually or with a preliminary slump test (this does not satisfy the testing requirements of ASTM C1019) before commencing with grouting operations.

b. If slump is in conformance with the Construction Documents, commence with grouting operations. Grout shall maintain required slump throughout entire grouting operation(s).

c. If the slump is not in conformance with Construction Documents, the addition of water is permitted to adjust slump at onsite truck arrival prior to the commencement of grouting operations. Grout shall maintain minimum design compressive strength as outlined in the Construction Documents. Mix grout in accordance with ASTM C476.

d. After initial mixing and addition of water, re-check grout slump. If slump is in conformance with Construction Documents commence with grouting operations (see Article Section 3.5 A.2.b). Otherwise, reject grout truck and discard ready-mixed grout that does not meet the specified slump.

The time limitation is waived as long as the ready-mixed grout meets the specified slump.

The new verbiage proposed for the code commentary is as follows:

3.5 A. Placing time - Grout placement is often limited to 1½ hours after initial mixing, but this time period may be too long in hot weather (initial set may occur) and may be unduly restrictive in cooler weather. One indicator that the grout has not reached initial set is a stable and reasonable grout temperature. However, sophisticated equipment and experienced personnel are required to determine initial set with absolute certainty.
Article 3.5 A.2 permits water to be added to ready-mixed grout to compensate for evaporation that has occurred prior to discharge. Replacement of evaporated water is not detrimental to ready-mixed grout. However, water may not be added to the already discharged ready-mixed grout.

A flow-chart is to interpret the code section is also recommended. We have drafted a proposed flowchart. Since we cannot attach anything to this public comment, please email me for the flowchart if desired.

Thank you for your consideration!

**Response/Rationale:**

CR appreciates the comment but believes the existing language within the Specification and Specification Commentary Articles 3.5 A is sufficiently clear regarding the addition of water for ready-mix grout. Additionally, if one were to follow the suggested language for 3.5 A.2.a, one would not know whether the grout conforms to the construction documents as noted in the proposed 3.5 A.2.b and c. as even the commenter admits their method does not satisfy ASTM C1019 requirements. Lastly, the proposed commentary language largely reiterates the existing language with few editorial differences. Therefore, CR proposes no changes in response to this comment.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:** N/A

**Code Commentary:** N/A

**Specification:** N/A

**Specification Commentary:** N/A

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:** N/A

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-CR- 005 | |
| **Technical Contact/Email:** Jonathon R. Merk / jon@forrestassociate.com | |
| **Public Comment Number:** 2022 Comment # 152 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **10/26/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #152:**

Rebar positioners are not required by Code, therefore they should not be depicted or referenced in the Code Commentary. Their presence is often interpreted by design professionals (architects and engineers), building officials and special inspectors to imply necessity.

**Response/Rationale:**

This comment is in regards to the positioners depicted in Specification Commentary Figure SC-11. While the comment is correct that they are not required by Code, they do represent one way to comply with the requirements for the placement of reinforcement. Therefore, CR proposes adding a "disclaimer" to Figure SC-11 as a compromise.

---

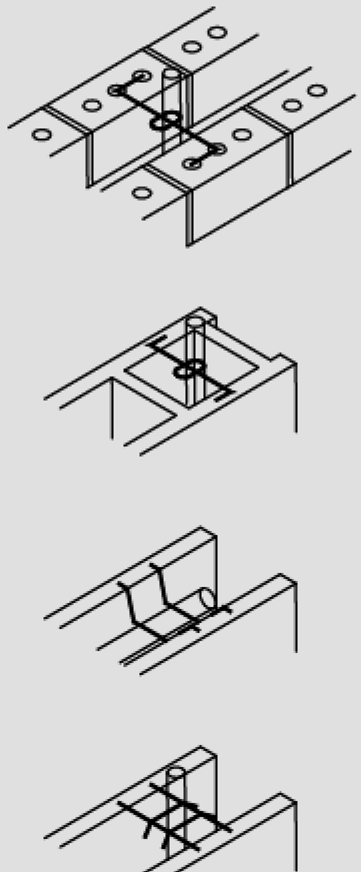
**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:** N/A

**Code Commentary:** N/A

**Specification:** N/A

**Specification Commentary:**



<u>While not required by Code, rebar positioners are one way to comply with reinforcement placement requirements.</u>

*Figure SC-11 — Typical positioners for reinforcement*

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 10 | *Affirmative* | 1 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 2 | *Did not vote* |

**Subcommittee Comments:**
The AWC response suggested an editorial correction that has been incorporated into this ballot.

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| Committee: Main Committee | | | | Ballot #: 20 | |
|---|---|---|---|---|---|
| Item #: 20-CR- 006 | | | | | |
| Technical Contact/Email: | Jonathon R. Merk / jon@forrestassociate.com | | | | |
| Draft Document Dated: | 10/26/2021 | | | | |
| Reballot of Main Committee Item No.: | N/A | Response to TAC Comment No.: | N/A | Response to Public Comment No.: | 5, 6, & 7 |

| Reference *(Choose from Drop-Down Menu)* | Section/Article |
|---|---|
| TMS 402 Code Section | 1.4 |

**Rationale:** *(Rationale is explanatory and not part of the proposed revision)*

Main Cmte. ballot item 19-CR-001 added references to ASTM C1714 within TMS 402 and 602. Four AWC responses to this ballot item pointed out that it is inappropriate to add a standard reference within TMS 402 when it is not cited within the Code. This ballot item seeks to make the necessary correction.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**TMS 402 Code Section 1.4:**

~~ASTM C1714/C1714M – Standard Specification for Preblended Dry Mortar Mix for Unit Masonry~~

**Code Commentary:** N/A

**Specification:** N/A

**Specification Commentary:** N/A

**Mandatory Requirements Checklist:** N/A

**Optional Requirements Checklist:** N/A

| Subcommittee Vote: | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:** N/A

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| Committee: Main Committee | | | | Ballot #: 20 |
|---|---|---|---|---|
| **Item #: 20-CR- 007** | | | | |
| **Technical Contact/Email:** | Jonathon R. Merk / jon@forrestassociate.com | | | |
| **Draft Document Dated:** | **10/26/2021** | | | |
| **Reballot of Main Committee Item No.:** N/A | | **Response to TAC Comment No.:** N/A | | **Response to Public Comment No.:** 5, 6, & 7 |

| **Reference** *(Choose from Drop-Down Menu)* | **Section/Article** |
|---|---|
| **TMS 602 Commentary Article** | 2.1 |

**Rationale:** *(Rationale is explanatory and not part of the proposed revision)*

Main Cmte. ballot item 19-CR-001 added a reference to ASTM C1714 within TMS 602. Two AWC responses to this ballot item requested additional language in the Specification Commentary as suggested by the AWC response at subcommittee level. This ballot item seeks to make the suggested addition.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown* <u>*underlined*</u> *and deletions are shown* ~~*struck-through*~~*.) Do not use 'Track Changes'*

**Code:** N/A

**Code Commentary:** N/A

**Specification:** N/A

**TMS 602 Specification Commentary 2.1:**

ASTM C270 contains standards for materials used to make mortar. Thus, component material specifications need not be listed. The Architect / Engineer may wish to include only certain types of materials, or exclude others, to gain better control.

<u>Mortars specified via C1714 / C1714M have materials and design requirements governed by C270, but are preblended in a factory instead of produced from individual raw materials delivered to the jobsite.</u>

Certain Applications …..

**Mandatory Requirements Checklist:** N/A

**Optional Requirements Checklist:** N/A

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:** N/A

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-CR- 008 | |
| **Technical Contact/Email:** Jonathon R. Merk / jon@forrestassociate.com | |
| **Public Comment Number:** 2022 Comment # 109 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **10/26/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #109:**

Article 3.5 E.b is clear that grout should be reconsolidated after initial water loss and settlement has occurred, but does not give any indication limiting how long after initial water loss and settlement. Previous codes used the term "before plasticity is lost". I would suggest some upper limitation, such as "loss of plasticity" since the attempt to reconsolidate grout that has lost plasticity does more damage than good.

**Response/Rationale:**

After negotiating language to satisfy a negative response on this item from Main ballot #19, CR offers the proposed revision in response to this comment.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:** N/A

**Code Commentary:** N/A

**TMS 602 Specification Article 3.5 E.b:**

b. Consolidate pours exceeding 12 in. (305 mm) in height by mechanical vibration, and reconsolidate by mechanical vibration after initial water loss and settlement has occurred<u>, but prior to the initial set and loss of plasticity</u>.

**Specification Commentary:** N/A

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:** N/A

# 2022 TMS 402/602 Committee
# Response to Public Comment

| **Committee: Main Committee** | **Ballot #: 20** |
|---|---|
| **Item #: 20-CR- 009** | |
| **Technical Contact/Email:** Jonathon R. Merk / jon@forrestassociate.com | |
| **Public Comment Number:** 2022 Comment # 31 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **10/26/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #31:**

Regarding TMS 602, Article 1.8.C.3.b.2. Language setting the minimum acceptable mixing temperature set to 70 degrees F, while requiring the minimum placement temperature be maintained above 70 degrees F does not make sense. Is the mason to apply heat on the way to the wall to raise the grout temperature above what is minimally required at the mixer? Either raise the minimum mixing temperature, or lower the minimum placement temperature, to account for a reasonable temperature drop between the mixer and the wall.

**Response/Rationale:**

Previous attempts to address the "minimum grout temperature at the time of placement" requirement during subcommittee, TAC, and previous PC ballots have failed due to insufficient data to support said change. In an attempt to compromise with the commenter, CR offers the following additional language for the Specification Commentary.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:** N/A

**Code Commentary:** N/A

**Specification Article:** N/A

**TMS 602 Specification Commentary Article 1.8 C.3.b.2:**

<u>Grout should be mixed to a temperature above the minimum mixing temperature to account for possible heat loss while transporting it between the mixing station and work area.</u>

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:** N/A

# 2022 TMS 402/602 Committee
# Response to Public Comment

| **Committee: Main Committee** | **Ballot #: 20** |
|---|---|

**Item #: 20-CR- 010**

**Technical Contact/Email:** Jonathon R. Merk / jon@forrestassociate.com

**Public Comment Number:** 2022 Comment # 182

| **Public Comment Response Based on TMS 402/602 Draft Dated** | **10/26/2021** |
|---|---|

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #182:**

The term "grout pour" is not understood by the design community and is too often confused with the pouring of grout into the wall which we call placement. The term should be deleted from the code and spec and described in another way. In many places in TMS 602, the phrase "maximum height of masonry prior to grouting" or "maximum height of the masonry to be grouted" can be used instead of grout pour to denote the maximum height the masonry may be built. This will eliminate the need to explain in great detail the difference between a lift and a pour.

**Response/Rationale:**

After discussion / reconsideration at our Nashville meeting, CR offers the following proposed revisions suggested by the commenter.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:** N/A

**Code Commentary:** N/A

**Specification:**

**3.5 C** *~~Grout pour height~~ Height of masonry prior to grouting* – Do not exceed the ~~maximum grout pour height~~ grout placement limits given in Table 7.

Table 7: Grout space requirements

| Grout type[1] | ~~Maximum grout pour height, ft (m)~~ Maximum height of masonry to be built prior to grouting, ft (m) | Minimum clear width of grout space,[2,3] in. (mm) | Minimum clear grout space dimensions for grouting cells of hollow units,[3,4] in. x in. (mm x mm) |
|---|---|---|---|
| Fine | 1 (0.30) | $^3/_4$ (19.1) | $1^1/_2$ x 2 (38.1 x 50.8) |
| Fine | 5.33 (1.63) | 2 (50.8) | 2 x 3 (50.8 x 76.2) |
| Fine | 12.67 (3.86) | $2^1/_2$ (63.5) | $2^1/_2$ x 3 (63.5 x 76.2) |
| Fine | 24 (7.32) | 3 (76.2) | 3 x 3 (76.2 x 76.2) |
| Coarse | 1 (0.30) | $1^1/_2$ (38.1) | $1^1/_2$ x 3 (38.1 x 76.2) |
| Coarse | 5.33 (1.63) | 2 (50.8) | $2^1/_2$ x 3 (63.5 x 76.2) |
| Coarse | 12.67 (3.86) | $2^1/_2$ (63.5) | 3 x 3 (76.2 x 76.2) |
| Coarse | 24 (7.32) | 3 (76.2) | 3 x 4 (76.2 x 102) |

1 Fine and coarse grouts are defined in ASTM C476.

2 For grouting between masonry wythes.

3 Minimum clear width of grout space and minimum clear grout space dimension are the net dimension of the space determined by subtracting masonry protrusions and the diameters of horizontal reinforcement from the as-built cross section of the grout space. Select the grout type and maximum grout ~~pour~~ height based on the minimum clear space.

4 Minimum grout space dimension for AAC masonry units shall be 3 in. (76.2 mm) x 3 in. (76.2 mm) or a 3 in. (76.2 mm) diameter cell.

**3.5 D** *Grout lift height* limits

1. For grout conforming to Article 2.2 A:
   a. Where the following conditions are met, place grout in lifts not exceeding 12 ft. 8 in. (3.86 m).
      i. The masonry has cured for at least 4 hours.
      ii. The grout slump is maintained between 10 and 11 in. (254 and 279 mm).
      iii. No intermediate reinforced bond beams are ~~placed~~ located between the top and the bottom of the ~~pour height~~ area to be grouted.
   b. When the conditions of Articles 3.5 D.1.a.i and 3.5 D.1.a.ii are met but there are intermediate bond beams within the ~~grout pour~~ area to be grouted, limit the grout lift height to the bottom of the lowest bond beam that is more than 5 ft. 4 in. (1.63 m) above the bottom of the lift, but do not exceed a grout lift height of 12 ft. 8 in. (3.86 m).
   c. When the conditions of Article 3.5 D.1.a.i or Article 3.5 D.1.a.ii are not met, place grout in lifts not exceeding 5 ft. 4 in. (1.63 m).
2. For self-consolidating grout conforming to Article 2.2:
   a. When placed in masonry that has cured for at least 4 hours, place in lifts not exceeding the ~~grout pour height~~ grouting height limits of Table 7.
   b. When placed in masonry that has not cured for at least 4 hours, place in lifts not exceeding 5 ft. 4 in. (1.63 m) or the ~~grout pour height~~ grouting height limit, whichever is less.

**3.5 E** *Consolidation*

1. Consolidate grout at the time of placement.
   a. Consolidate grout ~~pours~~ 12 in. (305 mm) or less in height by mechanical vibration or by puddling.
   b. Consolidate ~~pours~~ grout placed in lifts exceeding 12 in. (305 mm) in height by mechanical vibration, and reconsolidate by mechanical vibration after initial water loss and settlement has occurred.
2. Consolidation or reconsolidation is not required for self-consolidating grout.

**3.5 F** *Grout key* – When grouting, form ~~grout keys between pours. Form~~ grout keys between grout lifts when the first lift is permitted to set prior to placement of the subsequent lift.

1. Form a grout key by terminating the grout a minimum of $1^1/_2$ in. (38.1 mm) below a mortar joint.
2. Do not form grout keys within beams.
3. At beams or lintels laid with closed bottom units, terminate the grout ~~pour~~ placement at the bottom of the beam or lintel without forming a grout key.

**Specification Commentary:**

**3.5 – Grout Placement**
Grout may be placed by pumping or pouring from large or small buckets. The amount of grout to be placed and the contractor's experience influence the choice of placement method.

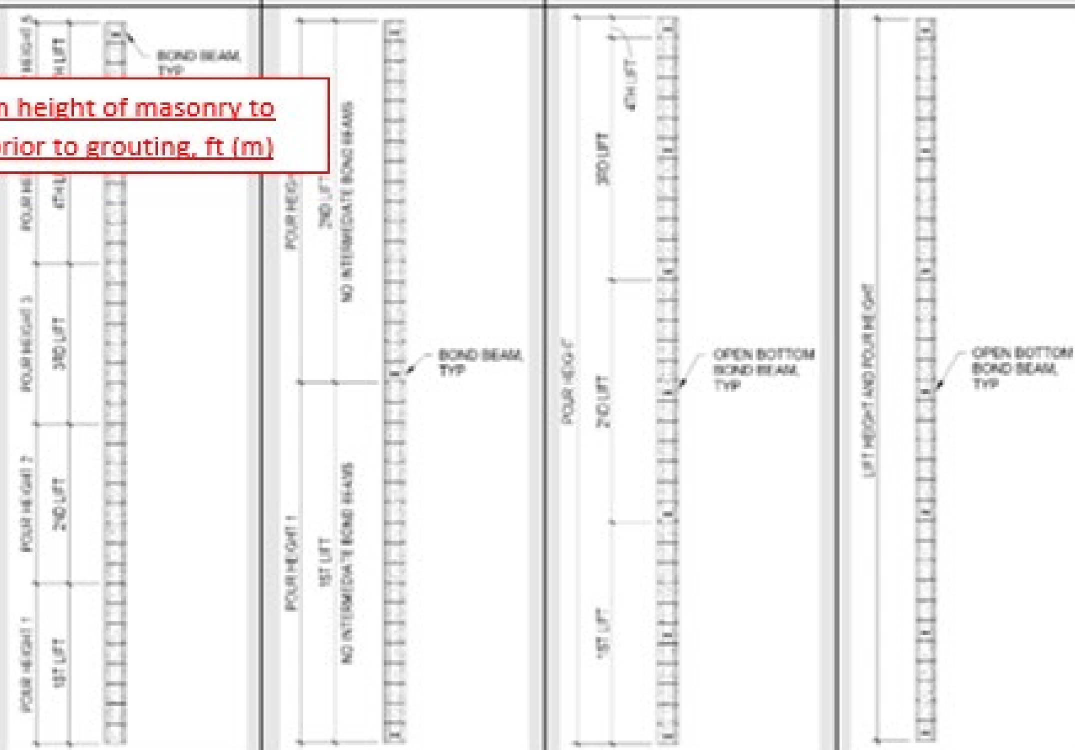
**3.5 C.** ~~*Grout pour height*~~ Grouting height limits – Table 7 in the Specification has been developed as a guide for grouting procedures. The designer can impose more stringent requirements if so desired. The recommended maximum height of ~~grout pour (see Figure SC-20)~~ masonry built prior to placement of grout corresponds with the least clear dimension of the grout space (see Figure SC-20). The minimum width of grout space is used when the grout is placed in collar joints. The minimum cell dimensions are used when grouting cells of hollow masonry units, including consideration of vertical alignment of cells. As the height of the ~~pour~~ masonry to be grouted increases, the minimum grout space increases. The grout space dimensions are the smallest clear dimensions, considering projections or obstructions into the grout space and the diameter or horizontal reinforcement, as illustrated in Figure SC-21. The grout space requirements of Table 7 are based on coarse and fine grouts as defined by ASTM C476, which defines aggregate size, and cleaning practices to permit the complete filling of grout spaces and adequate consolidation using typical methods of construction.

Grout ~~pour~~ placement heights and minimum dimensions that meet the requirements of Table 7 do not automatically mean that the grout space will be filled.

**3.5 D** *Grout lift height limits* – A lift is the height to which grout is placed into masonry in one continuous operation (see Figure SC-20). After placement of a grout lift, water is absorbed by the masonry units. Following this water loss, a subsequent lift may be placed on top of the still plastic grout.

Grouted construction develops fluid pressure in the grout space. Grout ~~pours~~ placement composed of several lifts may develop this fluid pressure for the full ~~pour~~ grout height. The faces of hollow units with unbraced ends can break out. Wythes may separate. The wire ties between wythes may not be sufficient to prevent this from occurring. Higher lifts may be used with self-consolidating grout because its fluidity and its lower initial water-cement ratio result in reduced potential for fluid pressure problems.

## COMMENTARY

| Type of Grouting* | Grouting with no cure time limit | Conventional grout with no intermediate bond beams | Conventional grout with intermediate bond beams | Self-consolidating grout with or without intermediate bond beams |
|---|---|---|---|---|
| TMS 602 Article | 3.5 D.1.c<br>3.5 D.2.b | 3.5 D.1.a | 3.5 D.1.b | 3.5 D.2.a |
| Lift Limit | 5 ft-4 in. | 12 ft-8 in. | See Limitation | Pour Height |
| ~~Pour Height~~ Maximum height of masonry to be built prior to grouting, ft (m) | Per Table 7 | Per Table 7 | Per Table 7 | Per Table 7 |
| Configuration |  | | | |
| Limitations | • Grout slump between 8 and 11 inches<br>• Conventional grout or self-consolidating grout<br>• ~~Lift height is 1-1/2 inches less than pour height for shear key, except at top of wall.~~ | • Masonry cured for at least 4 hours<br>• Grout slump between 10 and 11 inches | • Masonry cured for at least 4 hours<br>• Grout slump between 10 and 11 inches<br>• Lift cannot exceed ~~maximum~~ 12 ft-8 in.<br>• Limit grout lift to the bottom of lowest bond beam that is more than 5 ft-4 in. above bottom of grout lift<br>• ~~Lift height is 1-1/2 inches below the top of block for shear key, except at top of wall~~ | • Masonry cured for at least 4 hours |
| Cleanouts Required | No | No | Yes | Yes |

Stop grout 1-1/2 inches below top of lift to form grout key.

*Grout must conform to ASTM C476

*Figure SC-20 — Grout pour height and grout lift height*

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**3.5 F** *Grout key* – The top of ~~a~~ grout ~~pour~~ placement should not be located at the top of a unit, but at a minimum of $1^1/_2$ in. (38 mm) below the bed joint. If a lift of grout is permitted to set prior to placing the subsequent lift, a grout key is required~~ within the grout pour~~. This setting normally occurs if the grouting is stopped for more than one hour.

| Subcommittee Vote: | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 10 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 1 | *Abstain* | 2 | *Did not vote* |

**Subcommittee Comments:**
The abstention read: "As I read it, changing 'pour height' to 'maximum height of masonry to be built prior to grouting' is not equivalent. With pour height, I believe we could construct masonry higher than the pour height but only grout it up to the limit for pour height (meaning the maximum lift). If that is correct, we could have laid masonry up to 8 ft (for instance) but only grouted the first lift to 5 ft 4 in.

By changing to 'max ht of masonry to be built prior to grouting' it reads to me as if we have changed the requirement to not allow laying up masonry higher than we can grout in a lift. Is that really the intention?

This affects new proposed language in 3.5 C and Table 7."

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| **Committee: Main Committee** | | | | | **Ballot #: 20** |
|---|---|---|---|---|---|
| **Item #: 20-CR- 011** | | | | | |
| **Technical Contact/Email:** | Jonathon R. Merk / jon@forrestassociate.com | | | | |
| **Draft Document Dated:** | **10/26/2021** | | | | |
| **Reballot of Main Committee Item No.:** | N/A | **Response to TAC Comment No.:** | N/A | **Response to Public Comment No.:** | #159 |

| **Reference** *(Choose from Drop-Down Menu)* | **Section/Article** |
|---|---|
| **TMS 602 Commentary Article** | 1.4 |

**Rationale:** *(Rationale is explanatory and not part of the proposed revision)*

TMS staff pointed out an inconsistency between the Code and Specification as a result of language added in Main ballot item 19-CR-008. This ballot item seeks to correct that inconsistency.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)* *Do not use 'Track Changes'*

**Code:** N/A

**Code Commentary:** N/A

**Specification:** N/A

**Specification Commentary:**

*Dimension, nominal* – The permitted tolerances for units are given in the appropriate materials standards. Permitted tolerances for joints and masonry construction are given in this Specification. Nominal dimensions are usually used to identify the size of a masonry unit. Nominal dimensions are normally given in whole numbers nearest to the specified dimensions.

<u>*Dimension, specified –* </u>Specified dimensions are most often used for design calculations.

*Drainage space* – The drainage space may contain ……….

**Mandatory Requirements Checklist:** N/A

**Optional Requirements Checklist:** N/A

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:** N/A

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-CR-104 | |
| **Technical Contact/Email:** Jonathon R. Merk / jon@forrestassociate.com | |
| **Public Comment Number:** 2022 Comment # 32 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**NOTE TO VOTER—THIS BALLOT IS TO SUPPORT THE SUBCOMMITTEE RECOMMENDATION TO FIND THE NEGATIVE VOTER NON-PERSUASIVE. INFORMATION BETWEEN THE ASTERISKS (*) IS THE ORIGINAL BALLOT, FOR INFORMATION ONLY.**

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**Public Comment#32:**

When completing a low-lift wall, it would be helpful for the mason and / or inspector to have some wiggle room with respect to the cleanout requirement of TMS 602 3.2 F. For instance, if a mason wants to build 7’-4” above the last 5’-4” build, to top out the wall in one final step, and wishes to do so without cleanouts, or a grout demonstration panel, the inspector should still be able to adequately inspect the cells down to the last grout lift and then allow the mason to grout the 7’-4” height in two lifts. Please add language allowing conditions similar to the one described above.

**Response/Rationale:**

While we appreciate what the commenter is attempting to accomplish here, CR disagrees with the requested change. If we’re being brutally honest, some masons are lucky to go 2’-8” in height and keep the grout space clean enough to satisfy Code requirements for grout placement while other masons are capable of extending well beyond the current limitation of 5’-4”. The only legitimate way to determine that is through a demonstration panel. This could easily be accomplished with an enlarged sample panel reflecting the project conditions. Asking a mason to take this additional step in return for being allowed to deviate from Code does not constitute an onerous burden. Therefore, CR proposes no changes in response to this comment.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:** N/A

**Code Commentary:** N/A

**Specification:** N/A

**Specification Commentary:** N/A

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 7 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 6 *Did not vote* |

**Subcommittee Comments:** N/A

*****************************************************************************************

**Negative Vote (D McMillian)**

I disagree with the Response/Rationale to this public comment. The overall point of the public comment is not how clean a given contractor can keep a vertical cell column, but whether an inspector can still visually inspect a vertical cell column at a height slightly greater than 5'-4". The Response/Rationale does not address this. I would like to see the subcommittee consider that possibility further.

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**Subcommittee Meeting Discussion:**

19-CR-004 received one negative response. After discussion, a motion was made by Paul Scott and seconded by Kurt Siggard to find the McMillian negative response non-persuasive by a vote of 8-0-0 in a reaffirmation of our original response to this negative. Additional concerns raised during discussion included heavily reinforced walls where the amount of reinforcing within the cell would make inspection much more difficult if this were to pass.

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**Additional Comments From Subcommittee:**

CR maintains that you cannot ignore the potential consequences if this is allowed to pass as both cleanliness and the increased difficulty in cleaning / inspecting the cells due to the extra height will become issues. The best way to address this situation is through a demonstration panel prior to construction. Contractor / design team / inspector concerns can easily be addressed at that time. CR is also concerned that adding language to allow for this would easily be abused / used to justify inappropriate actions after the fact.

-------------

**Additional Comments From Negative Voter:**

Greetings main committee members, thanks for your consideration of this item. I disagree that the demonstration panel process is an option for the scenario posed by the public comment. TMS 602 Art. 1.6 E states that demonstration panels are an option to the requirements of Articles 3.5 C, 3.5 D and 3.5 E, but not the cleanout requirements of Art. 3.2 F. This is confirmed by the MDG-16 at the end of section 6.1.2, "Elimination of cleanouts for grout pour heights that exceed 5'-4" is not one of the construction procedures that TMS 602 permits to be waived via approval of a grout demonstration panel." To be clear, neither the public comment or my negative is advocating for the mason to be able to exceed the 5'-4" lift requirement for the scenario posed, but they do ask whether slightly exceeding the 5'-4" limit for when cleanouts become required could be allowed if the contractor and the inspector mutually agree that the grout space can still be cleaned and visually inspected. Obviously if the inspector is not comfortable with the potential grout space cleanliness or being able to visually inspect the grout space for a given wall, then he/she could deny the contractor's request and require cleanouts. This very well may be the case for heavily reinforced walls, as the subcommittee indicates was discussed during the recent meetings, however, I believe there are many walls that could still be cleaned and visually inspected at a couple courses higher than 5'-4". My negative asks the subcommittee to further consider that possibility. While I appreciate the subcommittee's concerns about changing the current language, I would point to a recent change to TMS 602 Art. 1.8 B, Masonry protection. Art. 1.8 B requires the tops of all unfinished masonry work be covered to protect it from moisture intrusion. However, a sentence was added to the commentary stating that in areas where dry weather is consistent, covering walls may not be required. This sets up a case where the decision to cover or not to cover must be made by mutual agreement between the contractor and the inspector. This is not unsimilar to what the public comment and my subsequent negative seeks.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-CR-105 | |
| **Technical Contact/Email:** Jonathon R. Merk / jon@forrestassociate.com | |
| **Public Comment Number:** 2022 Comment # 33 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**NOTE TO VOTER—THIS BALLOT IS TO SUPPORT THE SUBCOMMITTEE RECOMMENDATION TO FIND THE NEGATIVE VOTER NON-PERSUASIVE. INFORMATION BETWEEN THE ASTERISKS (*) IS THE ORIGINAL BALLOT, FOR INFORMATION ONLY.**

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**Public Comment #33:**

TMS 602, Table 4, Inspection Task 1.f, requires the special inspection of the sample panel construction for Levels 2 and 3, and lists Article 1.6 D for the inspection criteria. What is the purpose of these sample panels? So the mason and the inspector can practice the special inspection process before building and inspecting the actual walls? That does not seem beneficial since whatever might be established structurally by the completed sample panel would still have to be special inspected during the actual wall construction. Considerable code work has been done to require special inspections so that the actual construction agrees structurally with the approved construction documents, so why require it on a little piece of wall beforehand? If the structural engineer feels that a part of the construction warrants sampling for some structural reason, then he / she can always specify that outside of TMS 602, but sample panels should not be automatically required for every Level 2 or 3 masonry project. Please remove Inspection Task 1.f and let Article 1.6 D speak to aesthetic issues only, which most of the related commentary does anyway.

**Response/Rationale:**

Sample panels exist to help confirm the units match the design criteria, for the mason to demonstrate they are capable of installing the product within Code / project specification tolerances, and for the mason to demonstrate any difficult / unusual conditions the design team is concerned about, all of which establish a baseline for the quality of the masonry that extends well beyond aesthetics. Having a small sample panel rejected for a misunderstanding / etc. would have little impact on a project. Waiting to verify these items on "finished work" would yield terrible consequences. Therefore, CR proposes no changes in response to this comment.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown* <u>*underlined*</u> *and deletions are shown* ~~*struck-through*~~*.)* *Do not use 'Track Changes'*

**Code:** N/A

**Code Commentary:** N/A

**Specification:** N/A

**Specification Commentary:** N/A

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 7 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 6 *Did not vote* |

**Subcommittee Comments:** N/A

*******************************************************************************************

**Negative Vote (D McMillian)**

I disagree with the proposed subcommittee action to this public comment. I would prefer the subcommittee consider a new ballot, based on the current proposed Response/Rationale statement to PC 33, that would bring clarity to the exact role of the special inspector regarding the sample panel process. For instance, does the structural special inspector's role also include the project's aesthetic requirements as Art. 1.6 D commentary seems to imply?

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**Subcommittee Meeting Discussion:**

19-CR-005 received one negative response. After discussion, a motion was made by Kurt Siggard and seconded by Paul Scott to find the McMillian negative response non-persuasive by a vote of 9-0-0 in a reaffirmation of our original response to this negative. Additionally, while admittedly odd, an owner could also engage a special inspector to review the aesthetics the commenter is concerned about, but it is not our place to dictate contractual relationships.

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**Additional Comments From Subcommittee:**

While the first half of Specification Commentary Article 1.6 D discusses aesthetics and related items such as cleaning, the second half discusses items that have structural implications, including the tolerances in TMS 602 Article 3.3 F which are necessary for proper structural performance. Additionally, there are times when the special inspector absolutely needs to be involved in this process, especially when there are unique or otherwise heavily reinforced situations.

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**Additional Comments From Negative Voter:**

Greetings main committee members, thanks for your consideration of this item. I will let the original public comment and my subsequent negative speak for themselves. The following is in response to the recent subcommittee meeting discussions and the additional subcommittee comments above. I agree that it would be odd for the owner to engage the special inspector for aesthetic purposes, and I would add that it would also be a waste of time and expense to do so. Only the owner, through his designer representative (architect), can truly judge if the aesthetics shown in the sample panel match the intent of the project construction documents. It is also odd that the commentary for Art. 1.6 D says to construct the sample panels within the tolerances of Art. 3.3 F when the commentary for that article says those tolerances are not to be used for aesthetics. To me this just adds to the confusion of what Art. 1.6 D is trying to accomplish. I am a big proponent of special inspections, but I just don't feel that special inspection of sample panels should be dictated for all quality assurance Level 2 and 3 projects. If the engineer wants to require some type of structural sample panel for a given project, then he/she can certainly include that in the construction documents for that project. I also did a quick word study of Art. 1.6 D and of the approximately 200 words used for both the article and the commentary, only about 20 words are directly related to structural concepts. That's only 10%, with the rest being either general or more related to aesthetics. Personally, I would remove the sample panel requirement from the TMS 602 Table 4 special inspection list as suggested by the public comment. However, if we're going to keep it in, I believe there should be more guidance given to the special inspectors as to what their exact role in the sample panel process is. This is also the basic point of my negative which suggests the creation a new ballot to provide that guidance.

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-DE-004 | |
| **Technical Contact/Email:** | Dr. Richard Bennett (rmbennett@utk.edu) and Dr. Mark McGinley (m.mcginley@louisville.edu) |
| **Public Comment Number:** | 2022 Comment # 04 |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/4/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #4 (Kurt Siggard):** Chapter 9 has upper limits for design f'm found in 9.1.9.1.1. Chapter 8 does not have a provision with the same upper limits. Chapter 8 and Chapter 9 design provisions have been "harmonized" over the past couple of cycles, and we commonly say that "the wall doesn't know which design method is used".

I suggest that the limits found in 9.1.9.1.1 be moved to Chapter 4, or a similar provision be added to Chapter 8.

**Response/Rationale:** The committee agrees with the comment. The provisions are moved to Chapter 4 so they apply to both ASD and SD. For consistency, the provisions from Chapter 11 are also moved to Chapter 4. No significant changes are being made to the provisions.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)* *Do not use 'Track Changes'*

**Code:**

## 4.3 – Specified Compressive Strength

The specified compressive strength of masonry and grout shall meet the requirements of Table 4.3.1.

**Table 4.3.1: Specified Compressive Strength Requirements**

| **Type of masonry** | **Specified compressive strength of masonry** | **Specified compressive strength of grout** |
|---|---|---|
| Concrete masonry | $f'_m \leq$ 4,000 psi (27.58 MPa) | $f'_g \geq f'_m$<br>$f'_g \leq$ 5,000 psi (34.47 MPa) |
| Clay masonry | $f'_m \leq$ 6,000 psi (41.37 MPa) | $f'_g \leq$ 6,000 psi (41.37 MPa). |
| AAC masonry | $f'_{AAC} \geq$ 290 psi (2.0 MPa) | 2,000 psi (13.8 MPa) $\leq f'_g \leq$ 5,000 psi (34.47 MPa) |

Renumber subsequent sections.

**9.1.9** *Material properties*
~~**9.1.9.1** *Compressive strength*~~
~~**9.1.9.1.1** *Masonry compressive strength* — The value of $f'_m$ used to determine nominal strength values in this chapter shall not exceed 4,000 psi (27.58 MPa) for concrete masonry and shall not exceed 6,000 psi (41.37 MPa) for clay masonry.~~
~~**9.1.9.1.2** *Grout compressive strength* — For concrete masonry, the specified compressive strength of grout, $f'_g$, shall equal or exceed the specified compressive strength of masonry, $f'_m$, but shall not exceed 5,000 psi (34.47 MPa). For clay masonry, the specified compressive strength of grout, $f'_g$, shall not exceed 6,000 psi (41.37 MPa).~~

Renumber subsequent sections.

**11.1.8** *Material properties*
~~**11.1.8.1** *Compressive strength*~~
~~**11.1.8.1.1** *Masonry compressive strength* — The specified compressive strength of AAC masonry, $f'_{AAC}$, shall equal or exceed 290 psi (2.0 MPa).~~
~~**11.1.8.1.2** *Grout compressive strength* — The specified compressive strength of grout, $f'_g$, shall equal or exceed 2,000 psi (13.8 MPa) and shall not exceed 5,000 psi (34.5 MPa).~~

Renumber subsequent sections.

**Code Commentary:**

## 4.3 – Specified Compressive Strength

Most masonry research, including the design criteria based on TCCMaR research (Noland and Kingsley (1995)), has been conducted on structural masonry components having compressive strength in the range of 1,500 to 4,000 psi (10.34 to 27.58 MPa) for concrete masonry and 1,500 to 6,000 psi (10.34 to 41.37 MPa) for clay masonry. Thus, the upper limits given represent the upper values that were tested in the research.

Research (Varela et al (2006); Tanner et al (2005a)), Tanner et al (2005b); Argudo (2003)) has been conducted on structural components of AAC masonry with a compressive strength of 290 to 1,500 psi (2.0 to 10.3 MPa). Design criteria are based on these research results.

The code does not explicitly stipulate a minimum specified compressive strength for application with its design provisions. Compliance with the material requirements of TMS 602 implicitly establish a minimum masonry compressive strength. Care should be used when applying these provisions to materials and assemblies that do not conform to the requirements of TMS 602.

Because most empirically derived design equations calculate nominal strength as a function of the specified compressive strength of the masonry, the specified compressive strength of the grout is required to be at least equal to the specified compressive strength for concrete masonry. This requirement is an attempt to ensure that where the grout compressive strength controls the design (such as anchors embedded in grout), the nominal strength will not be affected. The limitation on the maximum grout compressive strength is due to the lack of available research using higher material strengths.

Due to the hydrophilic nature of AAC masonry, care should be taken to control grout shrinkage by pre-wetting cells to be grouted or by using other means, such as non-shrink admixtures. Bond between grout and AAC units is equivalent to bond between grout and other masonry units (Tanner et al (2005a), Tanner et al (2005b); Argudo (2003)).

Renumber subsequent sections.

**9.1.9** *Material properties* Commentary Section 4.2 provides additional information.
~~**9.1.9.1** *Compressive strength*~~
~~**9.1.9.1.1** *Masonry compressive strength* — Design criteria are based on TCCMaR research (Noland and Kingsley (1995)) conducted on structural masonry components having compressive strength in the range of 1,500 to 4,000 psi (10.34 to 27.58 MPa) for concrete masonry and 1,500 to 6,000 psi (10.34 to 41.37 MPa) for clay masonry. Thus, the upper limits given represent the upper values that were tested in the research. The code does not explicitly stipulate a minimum specified compressive strength for application with its design provisions. Compliance with the material requirements of TMS 602 implicitly establish a minimum masonry compressive strength. Care should be used when applying these provisions to materials and assemblies that do not conform to the requirements of TMS 602.~~
~~**9.1.9.1.2** *Grout compressive strength* — Because most empirically derived design equations calculate nominal strength as a function of the specified compressive strength of the masonry, the specified compressive strength of the grout is required to be at least equal to the specified compressive strength for concrete masonry. This requirement is an attempt to ensure that where the grout compressive strength controls the design (such as anchors embedded in grout), the nominal strength will not be affected. The limitation on the maximum grout compressive strength is due to the lack of available research using higher material strengths.~~

Renumber subsequent sections.

**11.1.8** *Material properties*
~~**11.1.8.1** *Compressive strength*~~
~~**11.1.8.1.1** *Masonry compressive strength* — Research (Varela et al (2006); Tanner et al (2005a)), Tanner et al (2005b); Argudo (2003)) has been conducted on structural components of AAC masonry with a compressive strength of 290 to 1,500 psi (2.0 to 10.3 MPa). Design criteria are based on these research results.~~
~~**11.1.8.1.2** *Grout compressive strength* — Because most empirically derived design equations relate the calculated nominal strength as a function of the specified compressive strength of the masonry, the specified compressive strength of the grout is required to be at least equal to the specified compressive strength. Additionally, due to the hydrophilic nature of AAC masonry, care should be taken to control grout shrinkage by pre-wetting cells to be grouted or by using other means, such as non-shrink admixtures. Bond between grout and AAC units is equivalent to bond between grout and other masonry units (Tanner et al (2005a), Tanner et al (2005b); Argudo (2003)).~~

Renumber subsequent sections.

**References, Chapter 4**

Argudo, J. (2003). "Evaluation and Synthesis of Experimental Data for Autoclaved Aerated Concrete," MS Thesis, The University of Texas at Austin. {Note to voters: this reference is already included in Chapter 4 so it does not need to be added.}

Noland, J., and Kingsley, G. (1995). "U.S. Coordinated Program for Masonry Building Research: Technology Transfer, Research Transformed into Practice", *Implementation of NSF Research*, Proceedings from the Conference, Arlington, Virginia, 360-371.

<u>Tanner, J.E., Varela, J.L., Klingner, R.E. (2005a). "Design and Seismic Testing of a Two-story Full-scale Autoclaved Aerated Concrete (AAC) Assemblage Specimen," *Structural Journal*, American Concrete Institute, 102(1), 114-119.</u>

<u>Tanner, J.E., Varela, J.L., Klingner, R.E., Brightman M. J. and Cancino, U. (2005b). "Seismic Testing of Autoclaved Aerated Concrete (AAC) Shear Walls: A Comprehensive Review," *Structural Journal*, American Concrete Institute, 102(3), 374-382.</u>

<u>Varela, J.L., Tanner, J.E. and Klingner, R.E. (2006). "Development of Seismic Force-Reduction and Displacement Amplification Factors for AAC Structures," *Earthquake Spectra*, Earthquake Engineering Research Institute, 22(1), 267-286.</u>

**References, Chapter 9**

Noland, J., and Kingsley, G. (1995). "U.S. Coordinated Program for Masonry Building Research: Technology Transfer, Research Transformed into Practice", *Implementation of NSF Research*, Proceedings from the Conference, Arlington, Virginia, 360-371. {Note to voters: this is referenced other places in Chapter 9 so it needs to stay}

**References, Chapter 11**

Argudo, J. (2003). "Evaluation and Synthesis of Experimental Data for Autoclaved Aerated Concrete," MS Thesis, The University of Texas at Austin. {Note to voters: this is referenced other places in Chapter 11 so it needs to stay}

Tanner, J.E., Varela, J.L., Klingner, R.E. (2005a). "Design and Seismic Testing of a Two-story Full-scale Autoclaved Aerated Concrete (AAC) Assemblage Specimen," *Structural Journal*, American Concrete Institute, 102(1), 114-119. {Note to voters: this is referenced other places in Chapter 11 so it needs to stay}

Tanner, J.E., Varela, J.L., Klingner, R.E., Brightman M. J. and Cancino, U. (2005b). "Seismic Testing of Autoclaved Aerated Concrete (AAC) Shear Walls: A Comprehensive Review," *Structural Journal*, American Concrete Institute, 102(3), 374-382. {Note to voters: this is referenced other places in Chapter 11 so it needs to stay}

Varela, J.L., Tanner, J.E. and Klingner, R.E. (2006). "Development of Seismic Force-Reduction and Displacement Amplification Factors for AAC Structures," *Earthquake Spectra*, Earthquake Engineering Research Institute, 22(1), 267-286. {Note to voters: this is referenced other places in Chapter 11 so it needs to stay}

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 9 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| Item #: 20-DE-037 | |
| Technical Contact/Email: Dr. Mark McGinley (m.mcginley@louisville.edu) | |
| Public Comment Number: 2022 Comment # 37 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 6/4/2021 |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☒ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 37 reads as follows:

> *This section (6.6.1(b)) states that joint reinforcing conforming to TMS 602 Article 2.4 D is within the scope of Chapter 6. It is unclear, however, whether stainless steel joint reinforcement is covered by this reference. While TMS 602 Article 2.4 D references ASTM A951 which in turn references ASTM 580 for stainless steel wire, the minimum yield strength requirements for wire in ASTM A951 (70 ksi) is incompatible with the yield strengths for ASTM 580 Grade 304 or 316 wire (30 to 45 ksi). This suggests that there may not be stainless steel joint reinforcement that is in conformance with ASTM A951 due to non-compliance with the minimum yield strength. Note that TMS 602 has a separate article that addresses stainless steel joint reinforcement (2.4 I) which only references ASTM A580; this is a wire specification, not a joint reinforcement specification.*
>
> *If the intent is to allow the use of stainless steel joint reinforcement for applications where conformance with Chapter 6 is required, several items need to be addressed.*
>
> *First, the specification of stainless steel joint reinforcement in TMS 602 needs to define a minimum yield strength of the wire. In addition it should be clarified that stainless steel joint reinforcement must be fabricated in accordance with ASTM A951, but using the lower strength ASTM A580 wire as permitted by TMS 602.*
>
> *Second, the provisions should be reviewed for the potential implications of the differing yield strengths of carbon steel and stainless steel joint reinforcement.*
> *(1) Are they equally as effective when used to meet the prescriptive requirements of Sections 7.3.2.2.1 and 7.4.3.1.1?*
> *(2) Are the minimum joint reinforcing areas for resisting shear of Sections 7.4.1.2.1 and 7.4.3.2.6 applicable regardless of wire type?*
> *(3) Is the allowable tensile stress of 30 ksi in Section 8.3.3.2 applicable to all wire types?*
> *(4) Can stainless steel joint reinforcement be used for conformance with Section 9.1.9.3.1?*

**RC subcommittee is addressing items 1 and 2 The DE Subcommittee has been asked to address items 3 and 4**

**Response/Rationale:**

Thank you for your public comment. The committee agrees that the current allowable stress value of 30ksi is appropriate and conservative for use with stainless steel joint reinforcing.

We also believe the fourth part of the comment relates to the use of stainless steel joint reinforcing for shear reinforcing, Section 9.1.9.3.2 . We agree that stainless steel joint reinforcing is appropriate for this use.  Note that Section 9.1.9.3.1 refers to in-plane flexural reinforcing and flexural tension perpendicular to the bed joint, both uses are not appropriate for joint reinforcing of any type.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 9 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee Response to Public Comment

| | |
|---|---|
| **Committee: Main Committee** | **Ballot #: 20** |
| **Item #: 20-DE-057** | |
| **Technical Contact/Email:** Dr. Richard Bennett (rmbennett@utk.edu) and Dr. Mark McGinley (m.mcginley@louisville.edu) | |
| **Public Comment Number:** 2022 Comment # 57 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/4/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☒ ***Public Comment only requires a response, no change to document***

**Public Comment:** where is the guidance for thru bolting for masonry. Say an all-thread bolt thru an 8" masonry.

**Response/Rationale:**
Thank you for your public comment. The committee agrees that thru bolt provisions should be added to TMS 402. However, there is limited research available, making it difficult to develop provisions. The committee will continue to review the research/literature, and develop provisions when adequate research on which to base the provisions is available.

For the present, the capacity of thru bolts could be obtained using TMS Section 8.1.3.2.1 or 9.1.6.2.1, which refers to determining the strength of anchors through testing using ASTM C1892. There are also a number of proprietary anchors, such as epoxy anchors with a screen tube, that could be used and are qualified with an ICC-ES report.

The commenter is advised to look at the limited information on the topic including: "Capacity of Masonry Loaded by Through-Bolts in Double Shear" by Gaur P. Johnson, Ian N. Robertson, and James Aoki published in TMS Journal, 2016 for in-plane loading, and "Testing of URM wall-to-diaphragm through-bolt plate anchor connections", Dmytro Dizhur, Shou Wei, Marta Giaretton, M.EERI, Arturo E. Schultz, M.EERI, Jason M. Ingham, M.EERI, Ivan Giongo, published in Earthquake Spectra, August 6, 2020 https://journals.sagepub.com/doi/10.1177/8755293020944187 for out-of-plane loading.

BIA Technical Note on Brick Construction 44, Anchor Bolts for Brick Masonry, states "However, based on the conservatism in the allowables for bent bar anchors and proprietary anchors, the allowable load equations should provide acceptable allowable load values for through bolts used in brick masonry. The embedment depth used to calculate the allowable load values should be taken as equal to the actual thickness of the masonry." It would be up to the design professional as to whether they are comfortable with the BIA suggestion.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 9 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| | |
|---|---|
| **Committee: Main Committee** | **Ballot #: 20** |
| **Item #: 20-DE-091** | |
| **Technical Contact/Email:** | Dr. Richard Bennett (rmbennett@utk.edu) and Dr. Mark McGinley (m.mcginley@louisville.edu) |
| **Public Comment Number:** | 2022 Comment # 91 |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/4/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** fu should be in italics and the "u" a subscript in the following. anchor bolt strength was changed to be based on fu

**Response/Rationale:** The Committee agrees with the comment and the change is made.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code: NONE**

**Code Commentary:**

**9.1.4.1** *Anchor bolts —*
In the 2022 edition of this Code, anchor bolt strength was changed to be based on ~~fu~~ <u>$f_u$</u> instead of $f_y$.

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 9 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| | | |
|---|---|---|
| **Committee: Main Committee** | | **Ballot #: 20** |
| **Item #: 20-DE-115** | | |
| **Technical Contact/Email:** | Dr. Richard Bennett (rmbennett@utk.edu) and Dr. Mark McGinley (m.mcginley@louisville.edu) | |
| **Public Comment Number:** | 2022 Comment # 115 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | | **6/4/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☒ ***Public Comment only requires a response, no change to document***

**Public Comment:**
While the compressive strength of grout in concrete masonry is required to equal or exceed f'm, there is not a corresponding requirement for clay masonry. Suggest either requiring a minimum grout strength for both materials or neither. Note TMS 602 2.2 B. only requires a minimum grout strength when f'm exceeds 2,000 psi.

**Response/Rationale:**
Thank you for your public comment. The committee discussed this at length and noted that concrete masonry and clay masonry have different behavior. Clay masonry units can have strengths in excess of 10,000 psi resulting in prism strengths of 5,000 psi or greater. Using a grout with this high of strength could be detrimental in clay as the high strength grout has potential for greater shrinkage while the clay is expanding. The committee recognizes that there is a potential conflict between the lack of a requirement for a minimum grout strength for clay masonry in TMS 402 Chapter 9 while TMS 602 Article 2.2. B requires the grout strength to equal or exceed f'm when f'm exceeds 2,000 psi for all masonry. The committee also recognizes that this is a larger issue, as the grout strength can affect lap splices and anchor bolt pullout. The committee will consider any appropriate code and specification changes and adding commentary as new business next code cycle.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 16 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 4 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| | |
|---|---|
| **Committee: Main Committee** | **Ballot #: 20** |
| **Item #: 20-DE-168** | |
| **Technical Contact/Email:** | Dr. Richard Bennett (rmbennett@utk.edu) and Dr. Mark McGinley (m.mcginley@louisville.edu) |
| **Public Comment Number:** | 2022 Comment # 168 |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/4/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** Section 9.3.3.2.2.1 makes sense for beams under gravity loads, but not for uplift. A singly reinforced beam over an opening and at the top of a wall may be subjected to a small amount of uplift from the roof that the reinforcement at the bottom of the beam can safely resist...but because the beam is bending about its weak vertical axis, it cannot meet the cracking moment check.

**Response/Rationale:** The Committee agrees with the comment. A small code change is proposed, and commentary is proposed to explain the provision.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**9.3.3.2.2** *Longitudinal reinforcement*

**9.3.3.2.2.1** <u>For gravity loading, the</u> ~~The~~ nominal flexural strength of a beam shall not be less than 1.3 multiplied by the nominal cracking moment of the beam, $M_{cr}$ . The modulus of rupture, $f_r$ , for this calculation shall be determined in accordance with Section 9.1.9.2.

**Code Commentary:**

**9.3.3.2.2** *Longitudinal reinforcement*

**9.3.3.2.2.1** The requirement that the nominal flexural strength of a beam not be less than 1.3 multiplied by the nominal cracking moment is imposed to prevent brittle failures. This situation may occur where a beam is so lightly reinforced that the bending moment required to cause yielding of the reinforcement is less than the bending moment required to cause cracking.

<u>This provision is only applicable to gravity loads. For example, a singly reinforced beam over an opening and at the top of a wall may be subjected to a small amount of uplift from the roof. The reinforcement at the bottom of the beam can safely resist these transient loads. This provision would not apply to the uplift loading.</u>

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 9 *Affirmative* | 1 *Affirmative w/ comment* | 1 *Negative* | 0 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:**

**Negative by A. Robinson** There is a chance that with a small increase in uplift, the entire beam could fail under gravity load with this proposal. I think we need to have at least some minimum strength greater than the cracking moment, maybe it does not need to be 1.3, but there should be some minimal capacity. See the attached example. The example may be extremely unlikely, but it shows a worst-case scenario. See following page.

**Affirmative with comment by B. Shing** The sentences added to the commentary are a bit unclear. The meaning of "safely resist" is not clear. Does this mean that the flexural demand is below the flexural strength of the beam? For seismic loads, if the beams are not designed as coupling beams in walls, this may not be a safety issue either even if the flexural strength is reached. Perhaps the sentence can be revised as" This provision is only applicable to gravity loads, where brittle failure imposes a safety issue."
The rest can be deleted.

**Chairs Response – Robinson Negative**
The proposed change is intended to limit the 1.3 cracking requirement to gravity loads where sustained loads need a minimum reserve strength sufficient to resist the release of energy when the section cracks and give warning of failure. The provision is indeed for gravity loads, which are sustained loads. If we have a sustained load and the beam has significantly greater cracking strength than nominal strength, then when it cracks the flexural strength is immediately exceeded, and there is no warning of failure. With wind, which is not a sustained load, the failure happens so fast that, no matter what, there is no warning. Wind is a 3 second gust.

**Chairs Response – Shing Affirm with Comment -** With just the suggested minimal commentary, it can be a bit confusing as to what the provision was saying without knowing the background discussion. Maybe the commentary could be written better, but I think the additional commentary is helpful.

# Robinson Beam Example

**Example Beam**

| | | | | | |
|---|---|---|---|---|---|
| Beam Width, b = | 5.625 in | | | | |
| Beam Depth, h = | 16 in | | | | |
| d Distance = | 12 in | | | | |
| d' Distance = | 4 in | | | | |
| Section Modulus, S = | 240 in$^3$ | | | | |
| Reinforcing, As = | 0.11 in$^2$ | | | | |
| Steel, fy = | 60,000 psi | | | | |
| Steel, E = | 29,000,000 psi | | | | |
| Masonry, f'm = | 2,500 psi | | | | |
| Modulus of Rupture, fr = | 100 psi | parallel to bed joints, running bond, type N masonry cement | | | |
| T = C = | 6600 lbs | | | | |
| a = | 0.55 in | | | | |
| c = | 0.69 in | | | | |
| εmu = | 0.0025 | | | | |
| εs + = | 0.0410 - Steel Yields | | | | |
| εs - = | 0.0120 - Steel Yields | | | | |
| Mn+ = | 77,378 lb-in | | | | |
| Mn- = | 24,578 lb-in | | | | |
| ΦMn+ = | 69,640 lb-in | | | | |
| ΦMn- = | 22,120 lb-in | | | | |
| Mcr = | 24,000 lb-in | | | | |
| 1.3Mcr = | 31,200 lb-in | | | | |
| Beam Length, L = | 8 ft | | | | |
| Tributary Width = | 15 ft | | | | |
| Dead Load, D = | 12 psf | | | | |
| Live Load, L = | 20 psf | | | | |
| Wind Uplift, W = | 25 psf | | Wind Uplift, W = | 28 psf | |
| Mu+ (1.4D) = | 24,192 lb-in | > Mcr | | | |
| Mu+ (1.2D + 1.6L) = | 66,816 lb-in | > Mcr | | | |
| Mu- (0.9D – W) = | -20,448 lb-in | < Mcr | Mu- (0.9D – W) = | -24,768 lb-in | > Mcr |

| | | |
|---|---|---|
| Mn > 1.3 Mcr - OK | | Mn > 1.3 Mcr - OK |
| Mn < 1.3 Mcr - NG | OK if proposal passes | Mn < 1.3 Mcr - NG |
| ΦMn > Mu - OK | | ΦMn > Mu - OK |
| ΦMn > Mu - OK | | ΦMn < Mu - NG |
| Beam OK? | | Beam NG? |

Change in uplift by 3 psf could make the beam fail under just gravity load.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-EX-001 | |
| **Technical Contact/Email:** Dr. Richard Bennett (rmbennett@utk.edu) | |
| **Public Comment Number:** 2022 Comment # 002 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #2:**

Please consider updating all standards if newer editions can be referenced. For example try to reference ASCE/SEI 7-22 if possible. Use this comment to make needed references throughout TMS 402, TMS 602, and Commentaries.

**Response/Rationale:**

We agree that all references should be reviewed and the newest ones used if possible.

This ballot item covers only the updating to ASCE/SEI 7-22. Other updates are considered in another ballot item.

ASCE/SEI 7-22 changed to strength level snow loads, which changed the load factors in the load combinations. Strength Design load combinations now have 1.0S and Allowable Stress Load Combinations now have 0.7S when the snow load is a primary load. There are two places in TMS 402 where the specific ASCE/SEI load combination is specified, and those are updated to the current ASCE/SEI load combination. The reference in TMS 402 is to ASCE/SEI 7-22 Strength Design Load Combination 6. For voter convenience, the load combination from the public comment version of ASCE/SEI 7-22 is:

6. $1.2D + E_v + E_h + L + 0.15S$
7. $0.9D - E_v + E_h$

After updates to references are approved, editors will make appropriate changes for entries in the body of the document.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**1.4 — Standards cited in this Code**

ASCE/SEI 7-~~16~~ 22— Minimum Design Loads and Associated
Criteria for Buildings and Other Structures

**9.3.5.6.2.3**
(a) Special boundary elements shall be provided over portions of compression zones where:

$$c \geq \frac{l_w}{600\left(1.5C_d\delta_{ne} / h_w\right)}$$

and $c$ is calculated for the $P_u$ given by ASCE/SEI 7 Strength Design Load Combination 6 (1.2$D$ + $E_v$ + $E_h$ + $L$ + ~~0.2~~0.15$S$) or the corresponding strength design load combination of the legally adopted building code, and the corresponding nominal moment strength, $M_n$ , at the base critical section. The load factor on $L$ in Combination 6 is reducible to 0.5, as per exceptions to Section 2.3.6 of ASCE/SEI

**11.3.6.6.2**
(a) Special boundary elements shall be provided over portions of compression zones where:

$$c \geq \frac{l_w}{600\left(1.5C_d\delta_{ne} / h_w\right)}$$

and $c$ is calculated for the $P_u$ given by ASCE/SEI 7 Strength Design Load Combination 6 (1.2$D$ + $E_v$ + $E_h$ + $L$ + ~~0.2~~0.15$S$) or the corresponding strength design load combination of the legally adopted building code, and the corresponding nominal moment strength, $M_n$ , at the base critical section. The load factor on $L$ in Combination 6 is reducible to 0.5, as per exceptions to Section 2.3.6 of ASCE/SEI

**Code Commentary:**

**7.3.2.9** *Ordinary plain prestressed masonry shear walls* — These shear walls are philosophically similar in concept to ordinary plain masonry shear walls. As such, prescriptive mild reinforcement is not required, but may actually be present. Seismic design factors provided for this type of prestressed masonry shear walls in ASCE/SEI 7~~-16~~ are in approximate agreement with the $R$ and $C_d$ factors recommended by Hassanli et al (2015) for ungrouted prestressed masonry shear walls.

**REFERENCES FOR THE CODE COMMENTARY**

Appendix D
ASCE/SEI 7 (~~2016~~ 2022). *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, American Society of Civil Engineers.

**Specification:** NONE

**Specification Commentary:** NONE

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 0 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:**

This ballot item submitted by Chair in accordance with Technical Committee Operations Manual Section 4.2.1. Technical input provided by Dick Bennett, John Hochwalt, Jamie Farny and Jon Merk.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-EX-002 | |
| **Technical Contact/Email:** James Farny / jfarny@cement.org | |
| **Public Comment Number:** 2022 Comment # 002 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #2:**

Please consider updating all standards if newer editions can be referenced. For example try to reference ASCE/SEI 7-22 if possible. Use this comment to make needed references throughout TMS 402, TMS 602, and Commentaries.

**Response/Rationale:**

We agree that all references should be reviewed and the newest ones used if possible.

For ease of voting, only those references suggested for changes are listed in this ballot. Each entry should be considered separately. If you disagree with a proposed change, please identify that item when submitting a negative or comment.

After updates to references are approved, editors will make appropriate changes for entries in the body of the document.

Note that ASCE/SEI 7 is being addressed on Ballot Item 20-EX-001.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**1.4 — Standards cited in this Code**

ANSI A137.1-19 — <u>American National</u> Standard Specifications for Ceramic Tile

ASTM A421/A421M-~~15~~ <u>21</u> — Standard Specification for Stress-Relieved Steel Wire for Prestressed Concrete

ASTM C90-~~16a~~ <u>21</u> — Standard Specification for Loadbearing

Concrete Masonry Units

ASTM C140/140M-~~20a~~ 21— Standard Test Methods for Sampling and Testing Concrete Masonry Units and Related Units

ASTM C212-~~20~~ 21— Standard Specification for Structural Clay Facing Tile

ASTM C216-~~19~~ 21— Standard Specification for Facing Brick (Solid Masonry Units Made from Clay or Shale)

ASTM C652-~~19b~~ 21— Standard Specification for Hollow Brick (Hollow Masonry Units Made from Clay or Shale)

ASTM C1006/C1006M-20a — Standard Test Method for Splitting Tensile Strength of Masonry Units

ASTM C1611/C1611M-~~18~~ 21 — Standard Test Method for Slump Flow of Self-Consolidating Concrete

ASTM C1634-~~17~~ 20 — Standard Specification for Concrete Facing Brick and Other Concrete Masonry Facing Units

ASTM C1670/C1670M-~~20a~~ 21a— Standard Specification for Adhered Manufactured Stone Masonry Veneer Units

ASTM C1892/~~C189220~~C1892M-20a — Standard Test Methods for Strength of Anchors in Masonry

AWS D 1.4/D1.4M: 2018 — Structural Welding Code — ~~Reinforcing~~ Steel Reinforcing Bars

**Code Commentary:**
**REFERENCES FOR THE CODE COMMENTARY**

Chapter 3
Chrysler, J. (~~2010~~ 2017). *Reinforced Concrete Masonry Construction Inspector's Handbook*, ~~7th~~ 10th Edition, Masonry Institute of America and International Code Council.

Chapter 4
NCMA TEK 10-~~2C~~ 2D (~~2010~~ 2019). "Control Joints for Concrete Masonry Walls – Empirical Method," *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

Chapter 5
CEB-FIP (~~1990~~ 2010). ~~*CEB-FIP Model Code 1990: Design Code*~~ *fib Model Code for Concrete Structures 2010*. Comité Euro-International du Béton (Euro-International Committee for Concrete, CEB) and the Fédération International de la Précontrainte (International Federation for Prestressing, FIP).

Chapter 10
ACI 318 (~~2014~~ 2019). Building Code Requirements for Reinforced Concrete, American Concrete Institute.

ASTM A416/A416M-~~12~~ 18 (~~2012~~ 2018). "Standard Specification for ~~Uncoated~~ Low-Relaxation, Seven-Wire Steel Strand for

Prestressed Concrete,” ASTM International, www.astm.org.

ASTM A421/A421M-~~10~~ 21 (~~2010~~ 2021). “Standard Specification for Stress-Relieved Steel Wire for Prestressed Concrete,” ASTM International, www.astm.org.

ASTM A722/A722M-~~07~~ 18 (~~2007~~ 2018). “Standard Specification for High-Strength Steel Bars for Prestressed Concrete,” ASTM International, www.astm.org.

Chapter 11
ASTM C78/C78M-~~02~~21 (~~2002~~ 2021). “Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading),” ASTM International, www.astm.org.

Chapter 12
ASCE 41 (~~2006~~ 2017). “~~Seismic Rehabilitation of Existing Buildings~~ Seismic Evaluation and Retrofit of Existing Buildings,” *ASCE 41-~~06~~17*, American Society of Civil Engineers.

Chapter 13
BIA TN 18A (~~2006~~ 2019). “Accommodating Expansion of Brickwork”, *Technical Notes on Brick Construction*, Brick Industry Association, www.gobrick.com.

IBC ~~2018~~ (2021). “International Building Code,” International Code Council, Washington D.C~~., 2018~~.

~~MVMA~~ NCMA (~~2017~~ 2021). *Installation Guide and Detailing Options for Compliance with ASTM C1780*, 5th edition, 5th printing, Masonry Veneer Manufacturers Association National Concrete Masonry Association.

**Specification:**
**1.3 — Reference standards**

*American Concrete Institute*
ACI 117-10 ~~Standard~~ Specifications for Tolerances for Concrete Construction and Materials (117-10) and Commentary-Reapproved 2015

*American National Standards Institute*
ANSI A118.4-19 American National Standard Specifications for Modified Dry-Set Cement Mortar

ANSI A118.15-19 American National Standard Specifications for Improved Modified Dry-Set Cement Mortar

ANSI A137.1-~~19~~21 American National Standard Specifications for Ceramic Tile

*American Wood Council*
AWC NDS-18 National Design Specification NDS for Wood Construction – with 2018 NDS ~~Design~~ Supplement

*ASTM International*

ASTM A240/A240M-20a Standard Specification for Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General

Applications

ASTM A307-~~14e1~~21 Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60000 PSI Tensile Strength

ASTM A421/A421M-~~15~~21 Standard Specification for Stress-Relieved Steel Wire for Prestressed Concrete

ASTM A510/A510M~~18~~-20 Standard Specification for General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel, and Alloy Steel

ASTM A899-91(~~2014~~2021) Standard Specification for Steel Wire, Epoxy-Coated

ASTM A1008/A1008M-~~20~~21a Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Required Hardness, Solution Hardened, and Bake Hardenable

ASTM C67/67M-~~20~~21 Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile

ASTM C90-~~16a~~21 Standard Specification for Loadbearing Concrete Masonry Units

ASTM C109/C109M-~~20b~~21 Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens)

ASTM C150/C150M-~~20~~21 Standard Specification for Portland Cement

ASTM C212-~~20~~21 Standard Specification for Structural Clay Facing Tile

ASTM C216-~~19~~21 Standard Specification for Facing Brick (Solid Masonry Units Made from Clay or Shale)

ASTM C652-~~19b~~21 Standard Specification for Hollow Brick (Hollow Masonry Units Made from Clay or Shale)

ASTM C744-~~16~~21 Standard Specification for Prefaced Concrete and Calcium Silicate Masonry Units

ASTM C926-~~20b~~21 Standard Specification for Application of Portland Cement-Based Plaster

ASTM C1019-~~19~~20 Standard Test Method for Sampling and Testing Grout for Masonry

ASTM C1063-~~20~~21 Standard Specification for Installation of Lathing and Furring to Receive Interior and Exterior Portland Cement-Based Plaster

ASTM C1314-~~18~~21 Standard Test Method for Compressive Strength of Masonry Prisms

ASTM C1325-~~19~~21 Standard Specification for Fiber-Mat

Reinforced Cementitious Backer Units

ASTM C1405-~~20a~~21 Standard Specification for Glazed Brick (Single Fired, Brick Units)

ASTM C1532/C1532M-~~20~~21 Standard Practice for Selection, Removal and Shipment of Manufactured Masonry Units and Masonry Specimens from Existing Construction

ASTM C1611/C1611M-~~18~~21 Standard Test Method for Slump Flow of Self-Consolidating Concrete

ASTM C1634-~~17~~20 Standard Specification for Concrete Facing Brick and Other Concrete Masonry Facing Units

ASTM C1670/C1670M-~~20a~~21a Standard Specification for Adhered Manufactured Stone Masonry Veneer Units

ASTM C1691-21 ~~(2017)~~ Standard Specification for Unreinforced Autoclaved Aerated Concrete (AAC) Masonry Units

ASTM C1788-~~14 (2019)~~20 Standard Specification for Non-Metallic Plaster Bases (Lath) Used with Portland Cement Based Plaster in Vertical Wall Applications

ASTM D1056-~~14~~20 Standard Specification for Flexible Cellular Materials — Sponge or Expanded Rubber

ASTM E328-~~13~~21 Standard Test Methods for Stress Relaxation Tests for Materials and Structures

ASTM E518/E518M-~~15~~21 Standard Test Methods for Flexural Bond Strength of Masonry

ASTM F1554-~~18~~20 Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength

*American Welding Society*

AWS D 1.4/D1.4M:2018 Structural Welding Code – Steel Reinforcing Bars ~~Steel~~

**Specification Commentary:**
**REFERENCES FOR THE SPECIFICATION COMMENTARY**

Part 1

BIA TN 1 (~~1992~~2018). "~~All~~ Hot and Cold Weather Construction," *Technical Notes on Brick Construction*, Brick Industry Association, www.gobrick.com.

Chrysler, J. (~~2010~~2017). *Reinforced Concrete Masonry Construction Inspector's Handbook*, ~~7th~~ 10th Edition, Masonry Institute of America and International Code Council.

Masonry Industry Council (1999) "~~hot~~ Hot & Cold Weather Masonry Construction", Masonry Contractors Association of America, ~~Lonbard~~ Lombard, IL.

Farny, J. A., Melander, J. M., and Panarese, W. C (2008). *Concrete Masonry Handbook for Architects, Engineers, and Builders*, Portland Cement Association~~, 121-123~~.

PCA (~~1993~~2006). “Hot Weather Masonry Construction,” *Trowel Tips* IS243, Portland Cement Association.

Part 2
ACI 315R (~~1999~~2018). ~~*Details and Detailing of Concrete Reinforcement*~~ *Guide to Presenting Reinforcing Steel Design Details*, American Concrete Institute.

BIA TN18A (~~2006~~2019). “Accommodating Expansion of Brickwork”, *Technical Notes on Brick Construction*, Brick Industry Association, www.gobrick.com.

BIA TN18 (~~2006~~2019). “Volume Changes – Analysis and Effects of Movement,” *Technical Notes on Brick Construction*, Brick Industry Association, www.gobrick.com.

NCMA TEK 10-~~2C~~2D (~~2010~~2019). “Control Joints for Concrete Masonry Walls-Empirical Method,” *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

Part 3
ACI 117 (~~1990~~2010). ~~*Standard*~~ *Specification~~s~~ for Tolerances for Concrete Construction and Materials*, American Concrete Institute.

ACI 440.1R (2015). *Guide for the Design and Construction of Structural Concrete Reinforced with Fiber-Reinforced Polymer ~~(FRP)~~ Bars*, American Concrete Institute.

ASTM C1780 (~~2017~~2020). “Standard Practice for Installation Methods for ~~Manufactured Stone~~ Cement-based Adhered Masonry Veneer”, ASTM International, www.astm.org.

BIA TN28C (2014). “Thin Brick Veneer,” *Technical Notes on Brick Construction*, ~~“28C Thin Brick Veneer,”~~ Brick Industry Association.~~”~~ www.gobrick.com

Chrysler, J. (~~2010~~2017). *Reinforced Concrete Masonry Construction Inspector's Handbook*, ~~7th~~ 10th Edition, Masonry Institute of America and International Code Council.

CRSI (2015), “Frequently Asked Questions (FAQ) About Reinforcing Bars,” *Construction Technical Note CTN-G-2-15*, Concrete Reinforcing Steel Institute. resources.crsi.org/index.cfm/_api/render/file/?method=inline&fileID=2A6B7DB0-D41F-D5F2-835D7F25685989A3

~~MVMA (2017)~~ NCMA (2021). *Installation Guide and Detailing Options for Compliance with ASTM C1780*, 5th edition, ~~Masonry Veneer Manufacturers Association.~~ 5th printing, National Concrete Masonry Association.

PTI (~~1994~~2016). *Field Procedures Manual for Unbonded Single Strand Tendons,* ~~2nd~~ 3rd Edition, Post-Tensioning Institute.

| Subcommittee Vote: | | | | |
|---|---|---|---|---|
| 0 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:** N/A

This ballot item submitted by Chair in accordance with Technical Committee Operations Manual Section 4.2.1. Technical input provided by Dick Bennett, John Hochwalt, Jamie Farny and Jon Merk.

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
| --- | --- |
| Item #: 20-GR-044 | |
| Technical Contact/Email: Charles Clark / cclark@bia.org | |
| Public Comment Number: 2022 Comment # 44 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 11/5/2021 |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment: Specification MANDATORY REQUIREMENTS CHECKLIST, Page 394, Lines 13 through 63.** The TMS 602 requires that the Architect/Engineer specify the location of movement joints on the project drawings. Frequently, many Architects/Engineers will include a general note such as "Provide control joints at 25'-0" maximum" without physically locating the joints in plan or elevation which can lead to issues at flanged shear walls, lintels designed based on arching action, and wall intersections. AISC 341 requires a restricted zone for moment frame connections and for braced frames. The mandatory checklist could be more specifically, such as: "Indicate type and location of movement joints on the project drawings and specifically show graphically in plan or elevation locations where movement joints are not permitted." This would allow the contractors flexibility to place the joints in the wall without worrying about compromising the structural intent.

**Response/Rationale:**
It is agreed that placement of movement joints that do not account for the location of structural elements such as flanged shear walls, lintels designed based on arching action, and wall intersections can be problematic. The commentary in Section 1.2.1 of TMS 402 discusses such elements and recommends that the project drawings accurately reflect the design so that masonry and movement joints can be constructed and placed as designed. Identifying locations where movement joints should be prohibited to maintain the intent of the structural design is a feasible approach.

The Foreword to the Specification Checklist states the following:

> **F3.** Checklists do not form a part of TMS 602. Checklists are provided to assist the Architect/Engineer in selecting and specifying project requirements in the Project Specification. The checklists identify the Articles and paragraphs of TMS 602 and the action required or available to the Architect/Engineer.

As such, text added to the checklist under the heading of “Notes to Architect/Engineer,” is considered commentary, and is not a code requirement, which is consistent with the current approach in Section 1.2.1 in TMS 402, where the recommendation to graphically depict movement joints on the drawings is part of the commentary for the Section and is not a code requirement.

A change is proposed to the Mandatory Requirements Checklist under the Notes to the Architect/Engineer for Movement joints. In the proposed change, the term "where necessary" is intended to apply in cases where the placement of movement joints is identified by note only and where movement joints are not already depicted on the project drawings. The proposal also changes the reference to the correct TMS 602 Article/Paragraph.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code: NONE**
**Code Commentary: NONE**
**Specification and Specification Commentary:** Article 3.3 is shown for user reference only and no changes are proposed. Changes to the Mandatory Requirements Checklist are proposed as shown.

### 3.3 — Masonry erection

3.3 E - *Embedded items and accessories* — Install embedded items and accessories as follows:
6. Install movement joints.

| MANDATORY REQUIREMENTS CHECKLIST (Continued) | | |
|---|---|---|
| **TMS 602 Article/Paragraph** | | **Notes to the Architect/Engineer** |
| **PART 3 — EXECUTION** | | |
| 3.3 ~~D~~ E.2-4 | Pipes and conduits | Specify sleeve sizes and spacing. |
| 3.3 ~~D~~ E.5 | Accessories | Specify accessories not indicated on the project drawings. |
| 3.3 ~~D~~ E.6 | Movement joints | Indicate type and location of movement joints on the project drawings and, where necessary, specifically show graphically in plan or elevation locations where movement joints are not permitted to maintain structural design intent. |

| Subcommittee Vote: | | | | |
|---|---|---|---|---|
| 6 *Affirmative* | 2 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 1 *Did not vote* |

**Subcommittee Comments:**

**Comment 1:** In the rationale, "forward" should be "Foreword." [This is corrected in the Main Committee ballot.]

**Comment 2:** If the designer is not going to show where movement joints are located, it is doubtful that the designer will state where they should not be located. I would recommend we keep the text as is.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| | |
|---|---|
| **Committee: Main Committee** | **Ballot #: 20** |
| **Item #: 20-GR-096** | |
| **Technical Contact/Email:** Charles Clark / cclark@bia.org | |
| **Public Comment Number:** 2022 Comment # 96 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **11/5/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** There is redundant language across Part 3 in regards to legally adopted load cases that should be consolidated in this section. In addition, IBC 2021 now adopts the ASCE 7 load combinations by reference, with the exception of retaining the alternate ASD load combinations. This change may not change how the legally adopted load combinations are referenced in TMS 402, but is brought to the committee's attention. Sections that should be looked at for potential consolidation with 4.1.2 include 9.1.2, 11.1.2, and 12.1.2.

It is anticipated that the individual chapters would still state whether ASD or SD load combinations should be used for a given chapter. Chapter 8 does not, but should, have a requirement to use allowable stress design load combinations.

Lastly, while Section 10.2.1 is already consistent with this comment, the wording of should be looked at for consistency across Part 3.

**Response/Rationale:**

The committee agrees with the comment and recommends modifying the text in Chapters 8, 9, 11, and 12 to better coordinate. For reference, the current text of 4.1.2 and 10.2.1 is provided below:

*4.1.2 Load provisions*
*Design loads shall be in accordance with the legally adopted building code of which this Code forms a part, with such live load reductions as are permitted in the legally adopted building code. In the absence of a legally adopted building code, or in the absence of design loads in the legally adopted building code, the load provisions of ASCE/SEI 7 shall be used, except as noted in this Code.*

*10.2.1 General*
*Members shall be designed to meet the strength provisions in this Chapter and checked for allowable stress level load requirements. The provisions of Section 10.4.3 shall apply for the calculation of nominal moment strength. Loading and load combinations shall be in accordance with the provisions of Section 4.1.2, except as noted in this Chapter.*

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)* *Do not use 'Track Changes'*

**Code:**

**Note that text in brackets is for clarity only and is not intended as proposed text.**

*Add new Section 8.1.2, and renumber subsequent sections:*

**<u>8.1.2 Required strength</u> [allowable stress design]**

<u>Required strength shall be determined in accordance with the allowable stress design load combinations as designated in Section 4.1.2, except as noted in this Chapter.</u>

*Modify Section 9.1.2:*

**9.1.2 Required strength [strength design]**

Required strength shall be determined in accordance with the strength design load combinations ~~of the legally adopted building code.~~ <u>as designated in Section 4.1.2, except as noted in this Chapter.</u> Members subject to compressive axial load shall be designed for the strength level moment accompanying the strength level axial load. The strength level moment, Mu, shall include the moment induced by relative lateral displacement.

*Modify Section 11.1.2:*

**11.1.2 Required strength [AAC masonry]**

Required strength shall be determined in accordance with the strength design load combinations ~~of the legally adopted building code.~~ <u>as designated in Section 4.1.2, except as noted in this Chapter.</u> Members subject to compressive axial load shall be designed for the strength level moment accompanying the strength level axial load. The strength level moment, Mu, shall include the moment induced by relative lateral displacement.

*Modify Section 12.1.2:*

**12.1.2 Required strength [masonry infills]**

Required strength shall be determined in accordance with the strength design load combinations ~~of the legally adopted building code.~~ <u>as designated in Section 4.1.2, except as noted in this Chapter.</u> ~~When the legally adopted building code does not provide load combinations, structures and members shall be designed to resist the combination of loads specified in ASCE/SEI 7 for strength design.~~

**Code Commentary: NONE**

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 7 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 3 | *Did not vote* |

**Subcommittee Comments: NONE**

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-GR-125 | |
| **Technical Contact/Email:** Charles Clark / cclark@bia.org | |
| **Public Comment Number:** 2022 Comment # 125 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **11/5/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** Implies TMS 402 governs when conflicting with the legally adopted building code. IBC-18 102.4.1 "Where conflicts occur between provisions of this code and referenced codes and standards, the provisions of this code shall apply."

**Response/Rationale:**

The committee respectfully disagrees with the comment and proposes no changes. The adopted building code already states that it governs when conflicts occur.

For reference only, the current text of Code Section 1.1.2 is provided below:

**1.1.2** *Governing building code*
This Code supplements the legally adopted building code and shall govern in matters pertaining to structural design and construction of masonry. In areas without a legally adopted building code, this Code defines the minimum acceptable standards of design and construction practice.

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 8 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 1 | *Did not vote* |

**Subcommittee Comments: NONE**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| | |
|---|---|
| **Committee: Main Committee** | **Ballot #: 20** |
| **Item #: 20-GR-128** | |
| **Technical Contact/Email:** Charles Clark / cclark@bia.org | |
| **Public Comment Number:** 2022 Comment # 128 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **11/5/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** Don't understand the meaning of "in other documents."

**Response/Rationale:**

The committee agrees that the phrase "in other documents" is vague. The intent was to convey that a similar term is also used in other codes and standards. For example, the International Building Code uses the term "Registered Design Professional." This information is more appropriate in the Code Commentary than the Code definition itself. This change proposes to move the phrase from the Code and to the Code Commentary and replace "in other documents" with "in other codes and standards."

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown* <u>*underlined*</u> *and deletions are shown ~~*struck-through*~~*.)  Do not use 'Track Changes'*

**Code:**

**2.2 — Definitions**

*Licensed design professional* — An individual who is licensed to practice design as defined by the statutory requirements of the professional licensing laws of the state or jurisdiction in which the project is to be constructed and who is in responsible charge of the design.~~; in other documents, also referred to as *registered design professional*.~~

**Code Commentary:**

<u>*Licensed design professional* – also referred to as a "registered design professional" in other codes and standards.</u>

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 8 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 1 | *Did not vote* |

**Subcommittee Comments: NONE**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-GR-130 | |
| **Technical Contact/Email:** Charles Clark / cclark@bia.org | |
| **Public Comment Number:** 2022 Comment # 130 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **11/5/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** Add "in design" before "to resist forces"

**Response/Rationale:**

The committee respectfully disagrees with the comment. Adding "in design" as suggested does not add clarity to the definition and is not necessary.

For reference only, the definition of the term "reinforced masonry" in Code Section 2.2 is provided below:

**2.2 — Definitions**

*Masonry, reinforced* — Masonry in which reinforcement acting in conjunction with the masonry is used to resist forces.

| Subcommittee Vote: | | | | |
|---|---|---|---|---|
| 7 *Affirmative* | 1 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 1 *Did not vote* |

**Subcommittee Comments:**

1. **Affirmative with Comment:** I think the commenter may have meant "after" (instead of before "to resist forces"), because saying "…is used to resist forces in design" kind of clarifies this as the assumption in design or where/how it is "used". If comes after, I would be okay with adding the words for context, but I do not feel too strongly about it to vote negative.

2. **Comment [Non-Voting]**: I understand what the public comment was saying about the definitions for reinforced and unreinforced masonry not quite aligning. For reinforced, we say it's "used" and for unreinforced, we say it's "not taken into consideration."

For masonry, reinforced, I wonder if something like "masonry in which reinforcement and masonry act together and are both taken into consideration in resisting forces" would make the definitions more compatible.

# 2022 TMS 402/602 Committee Response to Public Comment

| | |
|---|---|
| **Committee: Main Committee** | **Ballot #: 20** |
| **Item #: 20-GR-131** | |
| **Technical Contact/Email:** Charles Clark / cclark@bia.org | |
| **Public Comment Number:** 2022 Comment # 131 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **11/5/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** Add "in design" after "is neglected"

**Response/Rationale:**

The committee respectfully disagrees with the comment. Adding "in design" as suggested does not add clarity to the definition and is not necessary.

For reference only, the definition of the term "unreinforced masonry" in Code Section 2.2 is provided below:

**2.2 — Definitions**

*Masonry, unreinforced* — Masonry in which the tensile resistance of masonry is taken into consideration and the resistance of reinforcing steel, if present, is neglected.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 7 *Affirmative* | 1 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 1 *Did not vote* |

**Subcommittee Comments:**

Again, I kind of see the merit in this public comment, but it is not a big issue for me.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20- GR-135 | |
| **Technical Contact/Email:** Charles Clark / cclark@bia.org | |
| **Public Comment Number:** 2022 Comment # 135 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **11/5/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** It is a long-time engineering practice to distribute lateral load by tributary area for low rise buildings with flexible diaphragms. It is more accurate for one- or two-story construction and as far as I know is still allowed by the IBC and ASCE 7. I suggest referencing ASCE 7. This is a complicated subject.

**Response/Rationale:**

The committee agrees with the comment, as the term "member stiffnesses" alone could be interpreted to exclude the cases in which distribution by tributary area is appropriate. Changes to the code are proposed based on phrasing used in the commentary.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**4.1.6 *Lateral load distribution***

Lateral loads shall be distributed to the structural system in accordance <u>with the rigidities of the structural system and of the horizontal diaphragms</u> ~~with member stiffnesses~~ and shall comply with the requirements of this section.

**Code Commentary:**

**4.1.6** *Lateral load distribution*

The design assumptions for masonry buildings include the use of a lateral-force-resisting system. The distribution of lateral loads to the members of the lateral-force-resisting system is a function of the rigidities of the structural system and of the horizontal diaphragms. Refer to ASCE 7 for more information about the methods used to distribute load to the lateral force-resisting system. The method of connection at intersecting walls and between walls and floor and roof diaphragms determines if the wall participates in the lateral-force-resisting system. Lateral loads from wind and seismic forces are normally considered to act in the direction of the principal axes of the structure. Lateral loads may cause forces in walls both perpendicular and parallel to the direction of the load. Horizontal torsion can be developed due to eccentricity of the applied load with respect to the center of rigidity. The analysis of lateral load distribution should be in accordance with accepted engineering procedures.

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| | | | | |
| 8 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 1 *Did not vote* |

**Subcommittee Comments: NONE**

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-GR-169 | |
| **Technical Contact/Email:** Charles Clark / cclark@bia.org | |
| **Public Comment Number:** 2022 Comment # 169 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **11/5/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** With the deletion of Section 3.2 the following commentary was deleted: "The TMS 602 Specification addresses material and construction requirements. It is an integral part of the Code in terms of minimum requirements relative to the composition, quality, storage, handling, and placement of materials for masonry structures."

It is unclear what provision this commentary was intended to address. Regardless, this is an important requirement for designers to be aware of and to require the compliance of contractors with. As a result, it is suggested that compliance with TMS 602 be listed as a required item on the contract documents in Section 1.2.1. The commentary that was deleted in Section 3.2 would then be restored at that location. Note that the commentary to the preface for TMS 602 makes a similar statement: "Part 1 of the Building Code Requirements for Masonry Structures (TMS 402) makes the Specification for Masonry Structures (TMS 602) an integral part of TMS 402."

**Response/Rationale:**
The committee respectfully disagrees with the comment and no changes are proposed. TMS 602 is a standard cited in Section 1.4. As such, the requirements in TMS 602 are declared to be part of TMS 402 "as if fully set forth" in the document. In addition, the quality assurance program in Section 3.1 requires that all masonry meet the requirements of TMS 602. The text in the last sentence of Section 3.1 is nearly the same text that was cited and deleted from Code Commentary Section 3.2.

For reference only, the current text of Code Section 3.1 is provided below:

### 3.1 — Quality Assurance program

The quality assurance program shall comply with the Level defined in Table 3.1, depending on how the masonry was designed and the Risk Category, as defined in ASCE/SEI 7 or the legally adopted building code. The quality assurance program shall itemize the requirements for verifying conformance of material composition, quality, storage, handling, preparation, and placement with the requirements of TMS 602, and shall comply with the minimum requirements of TMS 602, Tables 3 and 4, for the required Level.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 8 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 1 *Did not vote* |

**Subcommittee Comments: NONE**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| **Committee: Main Committee** | **Ballot #: 20** |
|---|---|
| **Item #: 20-GR-198** | |
| **Technical Contact/Email:** Charles Clark / cclark@bia.org | |
| **Public Comment Number:** 2022 Comment # 198 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **10/26/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** With respect to (h)...Other engineering involvement, for example, design of cladding on the structure, requires statements (not necessarily prescribed provisions) about movements of the structure and backing so that the cladding design is able to be designed to accommodate differential movements.

**Response/Rationale:**
The committee respectfully disagrees with the comment. Code Section 1.2.1 (h) already requires that the design address dimensional changes. As explained in Code Commentary Section 1.2.1 (h), one of the primary methods of accommodated differential movement is to incorporate movement joints. This would apply to masonry cladding as well as other masonry construction.

For reference only, the current text of Code and Code Commentary for Section 1.2.1 (h) is provided below:
**Code:**
**1.2.1** Show or indicate all information required by TMS 402 on the project drawings or in the project specifications, including:

(h) Provision for dimensional changes resulting from elastic deformation, creep, shrinkage, temperature, and moisture.

**Code Commentary:**
**1.2.1** This Code lists some of the more important items of information that must be included in the project drawings or project specifications. This is not an all-inclusive list, and additional items may be required by the building official.

(h) Control joints, expansion joints, and other movement joints are the primary means of accommodating dimensional changes and differential movement. Joint placement can influence structural design and performance in many ways, including, but not limited to, shear wall length, flange behavior at corners and/or intersecting walls, and potential interference with lintel bearing. Therefore, it is recommended that the drawings accurately reflect design assumptions so that the masonry and movement joints can be constructed and placed as intended. Graphic depictions of movement joints may provide greater clarity than notes.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 8 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 1 *Did not vote* |

**Subcommittee Comments: NONE**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|

**Item #:** **20-GR-199**

**Technical Contact/Email:** Charles Clark / cclark@bia.org

**Public Comment Number:** 2022 Comment # 199

| Public Comment Response Based on TMS 402/602 Draft Dated | 10/26/2021 |
|---|---|

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** Is the following statement really true??? "Masonry design by prescriptive approaches relies on rules and masonry compressive strength need not be verified."

**Response/Rationale:**
It is true that the prescriptive design methods do not require verification of masonry compressive strength. Propose change to Code Commentary to limit text to indicate this.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)* *Do not use 'Track Changes'*

**Code: NONE (Code text shown below is for voter convenience only.)**

**1.2.2** Each portion of the structure shall be designed based on the specified compressive strength of masonry for that part of the structure, except for portions designed in accordance with Part 4.

**Code Commentary:**

**1.2.2** Masonry design performed in accordance with engineered methods is based on the specified compressive strength of the masonry. For engineered masonry, structural adequacy of masonry construction requires that the compressive strength of masonry equals or exceeds the specified strength. Masonry <u>compressive strength need not be verified when masonry is designed</u> ~~design~~ by prescriptive <u>methods.</u> ~~approaches relies on rules and masonry compressive strength need not be verified.~~

**Specification: NONE**

**Specification Commentary: NONE**

| Subcommittee Vote: | | | | |
|---|---|---|---|---|
| 8 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 1 *Did not vote* |

**Subcommittee Comments: NONE**

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|

**Item #:** **20-GR-200**

**Technical Contact/Email:** Charles Clark / cclark@bia.org

**Public Comment Number:** 2022 Comment # 200

| Public Comment Response Based on TMS 402/602 Draft Dated | 10/26/2021 |
|---|---|

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit, but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** "...joint and opening locations assumed in the design..." Use of the term "assumed" is not appropriate. The design must be concluded...nothing about the design should be assumed. All that is needed to construct the structure in accordance with the design should be suitably communicated by the architect and/or engineer within the contract documents.

**Response/Rationale:**

The committee agrees. Changes are proposed to Code Section 1.2.3 and Code Commentary Section 1.2.3 to replace "assumptions" and "assumed".

If this item passes the Main Committee, it will supersede action taken by 19-GR-200.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**1.2.3** The contract documents shall be consistent with the basis for the design ~~assumptions~~.

**Code Commentary:**

**1.2.3** The contract documents must accurately reflect the basis for the design ~~requirements~~. For example, joint and opening locations ~~assumed~~ used in the design should be coordinated with locations shown on the drawings.

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 7 | *Affirmative* | 0 | *Affirmative w/ comment* | 1 | *Negative* | 0 | *Abstain* | 1 | *Did not vote* |

**Subcommittee Comments:**

**Comment from Negative Vote:** I have two issues with the public comment and the response:

1. The proposed change/sentence strange to me.
2. I strongly disagree with the dismissal of the concept of "design assumptions". We do have to make a lot of assumptions to design structures. We list some of these in the code as “assumptions”. We even have sections titled Design Assumptions: 8.2.3, 8.3.2., 9.2.3., 9.3.2, and 11.3.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| **Committee: Main Committee** | **Ballot #: 20** |
|---|---|
| **Item #: 20-GR-217** | |
| **Technical Contact/Email:** Charles Clark / cclark@bia.org | |
| **Public Comment Number:** 2022 Comment # 217 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **11/5/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit, but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** Sub-Section (h) is very important and also seems to be one of the most vague and misunderstood sections of code. Sometimes architects take responsibility for all movement provisions, sometimes engineers do so for engineered masonry elements, sometimes neither one does or neither does it very well. At a minimum, it seems that the sub-section could be modified to say 'Provision, including vertical and/or horizontal movement joints and other detailing as necessary, for dimensional changes...'. It is my opinion that the movement joints should be located in the drawings, either in plan or elevation view, and they should be detailed for proper performance including dimensions and materials. Or, at a minimum add Commentary to clarify what 'Provision' may actually entail in the drawings.

Also, it would be good to add Commentary non-engineered veneer and non/engineered masonry movement provisions should be included in the architectural but may require input from the engineer in the case of horizontal joints below relief angles; and that joints in any engineered masonry (in my opinion, anything that's not veneer and has a prescriptive or engineered basis of design) should be developed and shown by the engineer. And that engineered veneers should have provisions developed and shown by the design engineer.

**Response/Rationale:**

The committee agrees with modifying the Code Commentary to more clearly convey that movement joints are recommended on drawings. However, the committee respectfully disagrees with explicitly adding text to the Code that would require that movement joints be placed on drawings as there are projects where this would not be required. For example, when masonry is appropriately reinforced, movement joints are not required. The committee respectfully disagrees with adding Code Commentary regarding assigning responsibilities for movement joint design and placement because the code has not historically assigned such responsibilities. These responsibilities vary on a project-to-project basis.

Ballot Item 20-GR-217 was developed to address a negative vote that was found persuasive on 19-GR-217. The negative vote indicated that including a Code requirement for vertical and/or horizontal movement joints was not appropriate.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code: NONE (Code text shown below is for voter convenience only.)**

**1.2.1** Show or indicate all information required by TMS 402 on the project drawings or in the project specifications, including:

(h) Provision for dimensional changes resulting from elastic deformation, creep, shrinkage, temperature, and moisture.

**Code Commentary:**

**1.2.1** This Code lists some of the more important items of information that must be included in the project drawings or project specifications. This is not an all-inclusive list, and additional items may be required by the building official.

(h) Control joints, expansion joints, and other movement joints are the primary means of accommodating dimensional changes and differential movement. Movement joint locations are recommended to be included on the project drawings as they may provide greater clarity than notes. Joint placement can influence structural design and performance in many ways, including, but not limited to, shear wall length, flange behavior at corners and/or intersecting walls, and potential interference with lintel bearing. Therefore, it is recommended that the drawings accurately reflect design assumptions so that the masonry and movement joints can be constructed and placed as intended. ~~Graphic depictions of movement joints may provide greater clarity than notes.~~

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 8 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 1 *Did not vote* |

**Subcommittee Comments: NONE**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| Item #: 20-GR-219 | |
| Technical Contact/Email: Charles Clark / cclark@bia.org | |
| Public Comment Number: 2022 Comment # 219 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 10/26/2021 |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☒ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:** The Commentary for Section 4.5 is good and the information is getting better and better. Consider expanding the Commentary discussion to include discussion of dead load and which dead load or how much should be considered. If the goal is to prevent long-term visible deflection and serviceability problems (I read that as objectionable crack size), then maybe all dead loads should be considered but this is kind of like a pre-stressed concrete design or a deck design - do we care about deflections that occur before the masonry is laid and should the pre-masonry dead loads be considered or not? If we it takes larger deflections to become visible then the L/600 seems more about cracking and therefore it seems that the dead load considered should be the masonry self-weight and that dead load that is applied after the masonry is placed. Please consider what is appropriate and add Commentary, possible modify the Code language if needed, if mandatory language should be added to properly address the issue.

**Response/Rationale:**
The committee does not have time to conduct the research and hold extensive discussions on this topic before the end of this cycle.

For reference only, the Code and Code Commentary for Section 4.5 is provided below:

**Code:**
**4.5 — Deflection of beams supporting unreinforced masonry**
The calculated deflection of beams of any material providing vertical support to masonry designed in accordance with Section 8.2, Section 9.2, Section 11.2, or Chapter 15 shall not exceed *l*/600 under allowable stress level dead plus live loads.

**Code Commentary:**
**4.5 — Deflection of beams supporting unreinforced masonry**
The deflection limits apply to beams and lintels of any material that supports unreinforced masonry. The deflection requirements may also be applicable to supported reinforced masonry that has vertical reinforcement only.

The deflection limit of *l*/600 should prevent long-term visible deflections and serviceability problems. In most cases, deflections of approximately twice this amount, or *l*/300, are required before the deflection becomes visible (Galambos and Ellinwood (1986)). This deflection limit is for immediate deflections. Creep will cause additional long-term deflections. A larger deflection limit of $l/480$ has been used when considering long-term deflections (CSA (2014)).

| Subcommittee Vote: | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 8 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 1 | *Did not vote* |

**Subcommittee Comments: NONE**

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-PI-149 | |
| **Technical Contact/Email:** Charles Tucker / ctucker@fhu.edu | |
| **Public Comment Number:** 2022 Comment # 149 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | 6/1/2021 |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment # 149 read as follows:

Please consider adding provisions to allow small openings in masonry infills.

**Response/Rationale:**

The Committee agrees that the allowance of small openings in masonry infills is appropriate. Small openings have negligible impact on the strength or performance of the infill wall in which they are located as long as reinforcement, when required, is not displaced by the opening and the equivalent diagonal strut is not interrupted. A new Figure CC-12.1-1 provides a schematic of allowable opening locations. Research by Dawe and Seah (1989a) indicated the first crack load was essentially unaffected by even large openings within the infill wall, so the suggested limitations on opening size and location are conservative.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)  Do not use 'Track Changes'*

**Code:**

**12.1.5** *Limitations*
Partial infills and infills with openings <u>larger than those permitted by Section 12.1.5.1</u> shall not be considered as part of the lateral force-resisting system. Their effect on the bounding frame, however, shall be considered.

<u>**12.1.5.1** *Maximum opening size* – Openings in infills shall not exceed 6 in. (152 mm) in any dimension at the face of the wall and shall not interrupt reinforcement required by Section 7.4.</u>

**12.1.5.1.1** *Maximum cumulative area of openings* – The cumulative area of openings shall not exceed 144 in.$^2$ (0.093 m$^2$) in any 10 ft$^2$ (0.93 m$^2$) of wall surface area.

**12.1.5.1.2** *Location of openings* – Openings adjacent to the bounding columns shall be permitted only in the middle third of the infill height and the exterior quarter of the infill length. Openings adjacent to the bounding beam or slab shall be permitted only in the middle third of the infill length and the exterior quarter of the infill height.

**12.1.5.1.3** *Spacing of openings* – Clear spacing between openings shall not be less than 16”.

**Code Commentary:**

**12.1.5.1** *Limitations*
Structures with partial-height infills have generally performed very poorly during seismic events. Partial-height infills create short columns, which attract additional load due to their increased stiffness. This has led to premature column failure. Concrete columns bounding partial-height infills are particularly vulnerable to shear failure (Chiou et al, 1999).

Small openings have negligible impact on the strength or performance of the infill wall in which they are located as long as reinforcement, when required, is not displaced by the opening and the equivalent diagonal strut is not interrupted. See Figure CC-12.1-1 for a schematic of allowable opening locations. Openings in excess of those permitted by Sections 12.1.5.1 have the potential to impact structural performance of the equivalent diagonal strut. Research by Dawe and Seah (1989a) indicated the first crack load was essentially unaffected by even large openings within the infill wall, so the limitations on opening size and location are conservative. Infill walls with excessive openings are required to be designed as non-participating infills per Section 12.2. For the purposes of this Chapter, the term openings includes penetrations.

**12.1.5.1.2** *Location of openings* – The limitations of Section 12.1.5.1.2 are sufficient for typical strut widths; however, the designer should verify an opening does not interrupt the equivalent diagonal strut width.

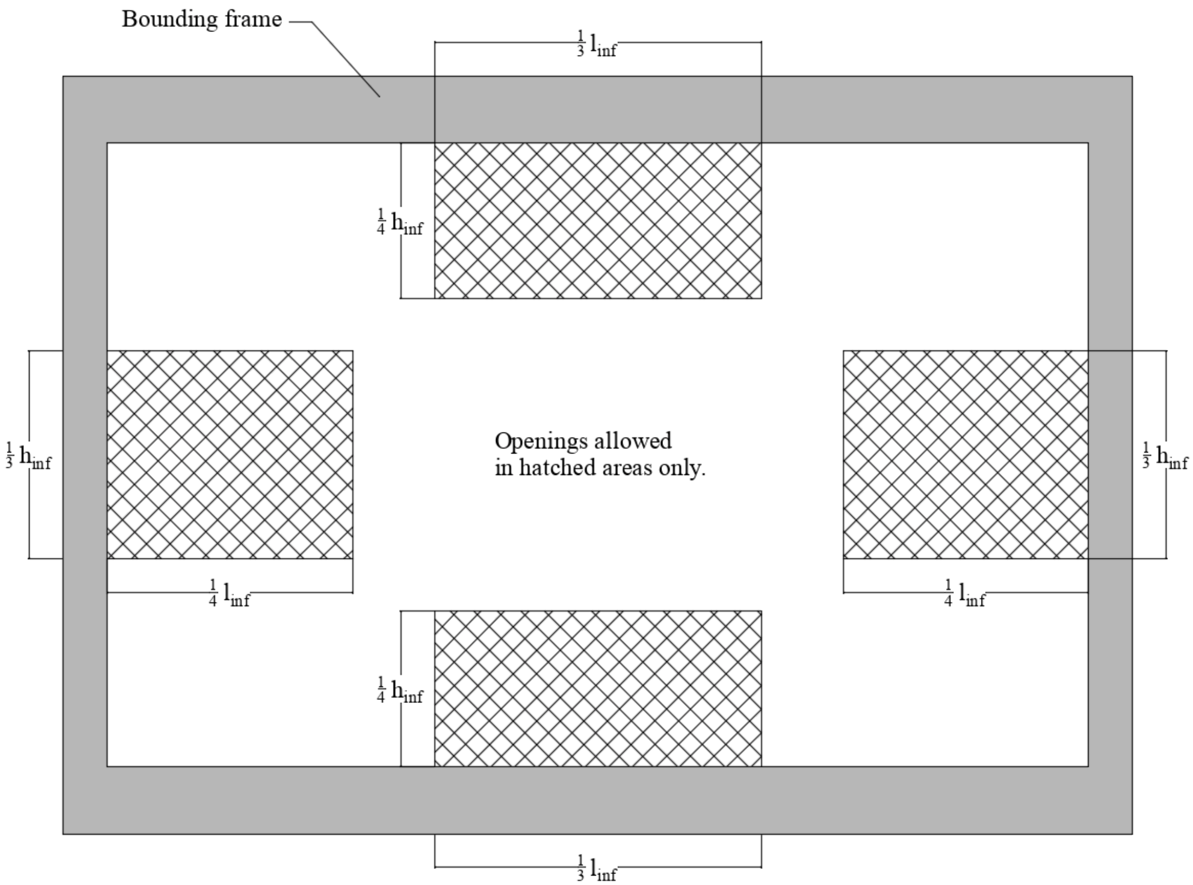


**Figure CC-12.1-1**

**Specification: None**

**Specification Commentary: None**

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 4 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 2 | *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| | |
|---|---|
| **Committee: Main Committee** | **Ballot #: 20** |
| **Item #: 20-RC-002** | |
| **Technical Contact/Email:** Heather Sustersic, hsustersic@colbycoengineering.com | |
| **Public Comment Number:** 2022 Comment # 45 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

*Consider balloting a change to Figure CC-6.1-8 to clarify that the lap shown is not a lap splice but rather the extension of negative moment reinforcement required by Section 6.1.10.*

**Response/Rationale:**

It is often difficult for masonry designers to interpret the intent of existing Figure CC6.1.8 as related to development of flexural reinforcement in a continuous masonry wall – is it a lap splice? Is it an extension of the development length for each bar? What happens when the bars above and below are the same size with respect to terminal ends of positive and negative moment reinforcement?

This ballot proposes modifications to the figure to help clarify the development lengths required for Bar a and Bar b as shown. The intent of the figure is to indicate the minimum required bar development length even though within the masonry code, the bar development and lap splice length equations are typically the same. It should be the decision of the designer as to how the wall should be reinforced at both positive and negative moment regions. Commentary language is proposed to provide direction when Bar a and Bar b are the same size.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Proposed changes are underlined in red or clouded in figure CC6.1.8 as shown below.**

**Code:**

None

**Code Commentary:**

**6.1.10.1.2** Critical sections for a typical continuous beam are indicated in Figure CC-6.1-7. Critical sections for a multi-span wall are indicated in Figure CC-6.1-8.

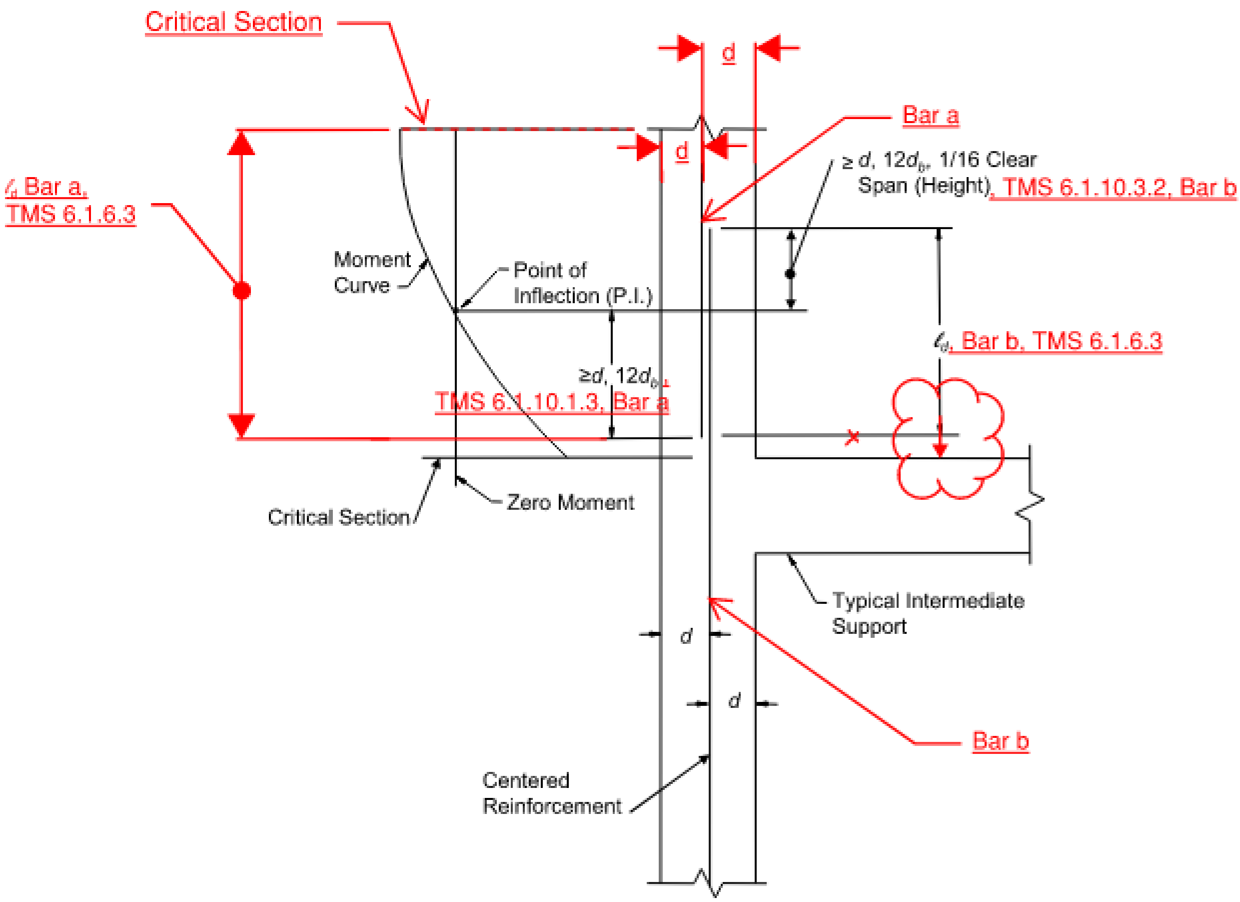


*Figure CC-6.1-8 — Development of flexural reinforcement in a continuous wall with centered reinforcement*

**6.1.10.1.3** The moment diagrams customarily used in design are approximate. Some shifting of the location of maximum moments may occur due to changes in loading, settlement of supports, lateral loads, or other causes. A diagonal tension crack in a flexural member without stirrups may shift the location of the calculated tensile stress approximately a distance d toward a point of zero moment. When stirrups are provided, this effect is less severe, although still present.

To provide for shifts in the location of maximum moments, this Code requires the extension of reinforcement a distance $d$ or $12d_b$ beyond the point at which it is theoretically no longer required to resist flexure, except as noted. <u>When terminal development lengths for positive and negative reinforcement occur coincidentally, the total lap length created need not exceed that required by Section 6.1.7.</u>

Cutoff points of bars or deformed wires to meet this requirement are illustrated in Figure CC-6.1-7.

When bars or deformed wires of different sizes are used, the extension should be in accordance with the diameter of reinforcement being terminated. A bar or deformed wire bent to the far face of a beam and continued there may logically be considered effective in satisfying this section, to the point where the bar or deformed wire crosses the middepth of the member.

**6.1.10.3** *Development of negative moment reinforcement* — Negative reinforcement must be properly anchored beyond the support faces by extending the reinforcement $\ell_d$ into the support or by anchoring of the reinforcement with a standard hook or suitable mechanical device.

Section 6.1.10.3.2 provides for possible shifting of the moment diagram at a point of inflection, as discussed under Commentary Section 6.1.10.1.3. This requirement may exceed that of Section 6.1.10.1.3 and the more restrictive governs. <u>When terminal development lengths for positive and negative reinforcement occur coincidentally, the total lap length created need not exceed that required by Section 6.1.7.</u>

**Specification:**

None

**Specification Commentary:**

None

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 8 | *Affirmative* | 1 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 6 | *Did not vote* |

**Subcommittee Comments:**

The affirmative comment from Hochwalt is as follows:

*I find the phrase "When terminal development lengths for positive and negative reinforcement occur coincidentally", which is used twice in the commentary, confusing. First, "terminal development length" sounds like it is a defined term that we should know the meaning of, but it is not. Second, "coincidentally" sounds they are happening in parallel, but I believe the situation we describing is where the two lengths are happening in sequence.*

*What if we the commentary said instead "In lieu or providing the development lengths and bar extensions shown in Figure CC-6.1-8, the reinforcing may be made continuous with an appropriate splice."*

*For extra credit, we could to attempt to address what happens if the size and/or spacing of bars "a" and "b" are different. It seems like the lesser intensity of reinforcement would be made continuous, and the remaining reinforcement would be treated as terminating per CC-6.1-8. Maybe next cycle!*

This comment has not been addressed by the subcommittee and is provided here for Main Committee voter information.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| Item #: 20-RC-003 | |
| Technical Contact/Email: John M. Hochwalt / john.hochwalt@kpff.com | |
| Public Comment Number: 2022 Comment # 37 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 6/1/2021 |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

See attached ballot 19-RC-003 for the full public comment.

**Response/Rationale:**

Public Comment 37 requests that additional language be added to TMS 402 and TMS 602 to clarify provisions for stainless steel joint reinforcement. Ballot 19-RC-003 addressed the first part of the public comment by adding a stainless steel wire material strength exception to Specification section 2.4D. The remainder of the public comment is being addressed by the SL and DE subcommittees. Ballot 19-RC-003 passed at Main with two affirmative comments and one additional comment as follows:

| | | |
|---|---|---|
| Affirmative With Comment | Mr. Charles B. Clark Jr. cclark@bia.org | The exception could be written more clearly to indicate that it is an exception to requirements within ASTM A951. |
| | Mr. Paul G. Scott pscott@ctsaz.com | Is the intent to say maximum of 90 ksi instead of minimum of 90 ksi? |
| Comment Non-Voting | Ms. Cortney Fried cfried@bia.org | While I agree with the content of the change, the placement of the exception could be interpreted to negate other requirements in the paragraph when using stainless steel joint reinforcing. Consider reorganizing this paragraph so that the exception is associated only with the material change. Suggested text:<br>2.4 D. Joint reinforcement â€" Provide joint reinforcement <u>in accordance with the following:</u><br>1. ~~that~~ Conforms to ASTM A951 or shall be <u>fabricated with AISI Type 304 or Type 316 stainless steel wire conforming to ASTM A580/A580M</u> ~~and~~ <u>having a minimum yield strength of 45 ksi (310 MPa) and a minimum ultimate tensile strength of 90 ksi (620 MPa)</u><br>2. ~~with~~ Maximum wire size <u>shall not exceed</u> one-half the specified mortar joint thickness. Do not use joint reinforcement with stacked wires whose total thickness exceeds one-half the specified mortar joint thickness.<br>3. Maximum spacing of cross wires in ladder-type joint reinforcement and of points of connection of cross wires to longitudinal wires of truss-type joint reinforcement shall be 16 in. (400 mm).<br>~~Exception: Joint reinforcement may be .~~ |

The RC subcommittee discussed these comments, confirming the intent to state a minimum ultimate tensile strength of 90 ksi and agreeing with Clark and Fried.

This ballot proposes to reposition the joint reinforcement exception to clarify that the exception pertains only to material specifications.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)* *Do not use 'Track Changes'*

**Please note that the text below reflects the passage of 19-RC-003. All underlines and strike-throughs are relative to that ballot.**

**Code:**

None.

**Code Commentary:**

None.

**Specification:**

**2.4 D.** *Joint reinforcement* — Provide joint reinforcement in accordance with the following:

1. ~~that c~~Conforms to ASTM A951 or is fabricated with AISI Type 304 or Type 316 stainless steel wire conforming to ASTM A580/A580M, having a minimum yield strength of 45 ksi (310 MPa) and a minimum ultimate tensile strength of 90 ksi (620 MPa).

2. ~~with m~~Maximum wire size shall not exceed one-half the specified mortar joint thickness. Do not use joint reinforcement with stacked wires whose total thickness exceeds one-half the specified mortar joint thickness.

3. Maximum spacing of cross wires in ladder-type joint reinforcement and of points of connection of cross wires to longitudinal wires of truss-type joint reinforcement shall be 16 in. (400 mm).

~~Exception: Joint reinforcement may be fabricated with AISI Type 304 or Type 316 stainless steel wire conforming to ASTM A580/A580M and having a minimum yield strength of 45 ksi (310 MPa) and a minimum ultimate tensile strength of 90 ksi (620 MPa).~~

**2.4 I.** *Stainless steel* — Stainless steel items shall be AISI Type 304 or Type 316, and shall conform to the following:

1. Plate and bent-bar anchors ..................ASTM A480/A480M and ASTM A666
2. Sheet-metal anchors and ties...............ASTM A480/A480M and ASTM A240/A240M
3. Wire ties .............................................ASTM A580/A580M

**Specification Commentary:**

**2.4 D.** *Joint reinforcement* — Code Section 9.1.9.3.2 limits the specified yield strength of joint reinforcement used to resist in-plane shear and flexural tension parallel to bed joints in strength design.

Where vertical reinforcement is present in a masonry wall, diagonal wires in the truss type joint reinforcement will conflict with placement of the vertical reinforcement. Mortar droppings on the diagonal cross wires also make quality grouting more difficult. Consequently, truss-type joint reinforcement should not be specified when the masonry contains vertical reinforcement.

Some manufacturers fabricate joint reinforcement with cross wires spaced at less than 16 in. (400 mm) on center. Joint reinforcement with non-modular dimensioned cross wires can interfere with placement of vertical reinforcement.

Commonly available ASTM A580/A580M stainless steel wire does not conform to the minimum yield and tensile strengths required by ASTM A951. The exception allows the use of this wire and requires that it meet the minimum strength requirements for Type 304 or Type 316 cold-finished wire.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 9 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 6 *Did not vote* |

**Subcommittee Comments:**

**THE FOLLOWING PAGES ARE ATTACHED FOR VOTER INFORMATION**
(NOT PART OF BALLOT)

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 19 |
|---|---|
| Item #: 19-RC-003 | |
| Technical Contact/Email: John M. Hochwalt / john.hochwalt@kpff.com | |
| Public Comment Number: 2022 Comment # 37 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 6/1/2021 |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 37 reads as follows:

> *This section (6.6.1(b)) states that joint reinforcing conforming to TMS 602 Article 2.4 D is within the scope of Chapter 6. It is unclear, however, whether stainless steel joint reinforcement is covered by this reference. While TMS 602 Article 2.4 D references ASTM A951 which in turn references ASTM 580 for stainless steel wire, the minimum yield strength requirements for wire in ASTM A951 (70 ksi) is incompatible with the yield strengths for ASTM 580 Grade 304 or 316 wire (30 to 45 ksi). This suggests that there may not be stainless steel joint reinforcement that is in conformance with ASTM A951 due to non-compliance with the minimum yield strength. Note that TMS 602 has a separate article that addresses stainless steel joint reinforcement (2.4 I) which only references ASTM A580; this is a wire specification, not a joint reinforcement specification.*
>
> *If the intent is to allow the use of stainless steel joint reinforcement for applications where conformance with Chapter 6 is required, several items need to be addressed.*
>
> *First, the specification of stainless steel joint reinforcement in TMS 602 needs to define a minimum yield strength of the wire. In addition it should be clarified that stainless steel joint reinforcement must be fabricated in accordance with ASTM A951, but using the lower strength ASTM A580 wire as permitted by TMS 602.*
>
> *Second, the provisions should be reviewed for the potential implications of the differing yield strengths of carbon steel and stainless steel joint reinforcement.*
> *(1) Are they equally as effective when used to meet the prescriptive requirements of Sections 7.3.2.2.1 and 7.4.3.1.1?*
> *(2) Are the minimum joint reinforcing areas for resisting shear of Sections 7.4.1.2.1 and 7.4.3.2.6 applicable regardless of wire type?*
> *(3) Is the allowable tensile stress of 30 ksi in Section 8.3.3.2 applicable to all wire types?*
> *(4) Can stainless steel joint reinforcement be used for conformance with Section 9.1.9.3.1?*

**Response/Rationale:**
This ballot is intended to address the first part of the comment, related to Chapter 6.

This ballot proposes to combine the requirements for stainless steel joint reinforcement in Article 2.4 I of TMS 602 with the provisions in Article 2.4 D for carbon steel joint reinforcement. Doing this clarifies that references to joint reinforcement in Chapter 6 are intended to apply to joint reinforcement manufactured from both carbon and stainless steel.

We have confirmed with manufacturers of joint reinforcement that the materials properties proposed for stainless steel joint reinforcement are consistent with materials that have been historically used and that are currently being used.

Given that the yield strength of stainless steel joint reinforcement is less than carbon steel joint reinforcement, the existing provisions in Chapter 6 for the development, splicing and anchorage of joint reinforcement can continue to be safely applied to joint reinforcement.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck through~~.)* *Do not use 'Track Changes'*

**Code:**

None.

**Code Commentary:**

None.

**Specification:**

**2.4 D.** *Joint reinforcement* — Provide joint reinforcement that conforms to ASTM A951 with maximum wire size one-half the specified mortar joint thickness. Do not use joint reinforcement with stacked wires whose total thickness exceeds one-half the specified mortar joint thickness. Maximum spacing of cross wires in ladder-type joint reinforcement and of points of connection of cross wires to longitudinal wires of truss-type joint reinforcement shall be 16 in. (400 mm).

Exception: Joint reinforcement may be fabricated with AISI Type 304 or Type 316 stainless steel wire conforming to ASTM A580/A580M and having a minimum yield strength of 45 ksi (310 MPa) and a minimum ultimate tensile strength of 90 ksi (620 MPa).

**2.4 I.** *Stainless steel* — Stainless steel items shall be AISI Type 304 or Type 316, and shall conform to the following:

~~1. Joint reinforcement ............................ASTM A580/A580M~~
2. Plate and bent-bar anchors ..................ASTM A480/A480M and ASTM A666
3. Sheet-metal anchors and ties...............ASTM A480/A480M and ASTM A240/A240M
4. Wire ties ..............................................ASTM A580/A580M

**Specification Commentary:**

**2.4 D.** *Joint reinforcement* — Code Section 9.1.9.3.2 limits the specified yield strength of joint reinforcement used to resist in-plane shear and flexural tension parallel to bed joints in strength design.

Where vertical reinforcement is present in a masonry wall, diagonal wires in the truss type joint reinforcement will conflict with placement of the vertical reinforcement. Mortar droppings on the diagonal cross wires also make quality grouting more difficult. Consequently, truss-type joint reinforcement should not be specified when the masonry contains vertical reinforcement.

Some manufacturers fabricate joint reinforcement with cross wires spaced at less than 16 in. (400 mm) on center. Joint reinforcement with non-modular dimensioned cross wires can interfere with placement of vertical reinforcement.

Commonly available ASTM A580/A580M stainless steel wire does not conform to the minimum yield and tensile strengths required by ASTM A951. The exception allows the use of this wire and requires that it meet the minimum strength requirements for Type 304 or Type 316 cold-finished wire.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 1 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 3 *Did not vote* |

**Subcommittee Comments:**

The affirmative comment from Walcowicz expressed concern regarding product availability for stainless steel meeting the proposed yield strength minimum. This ballot was developed in consultation with reinforcement manufacturers for consistency with products historically and currently available.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
| --- | --- |
| Item #: 20-RC-012 | |
| Technical Contact/Email: John M. Hochwalt / john.hochwalt@kpff.com | |
| Public Comment Number: 2022 Comment # 95 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 6/1/2021 |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

*Public Comment 95 read as follows:*

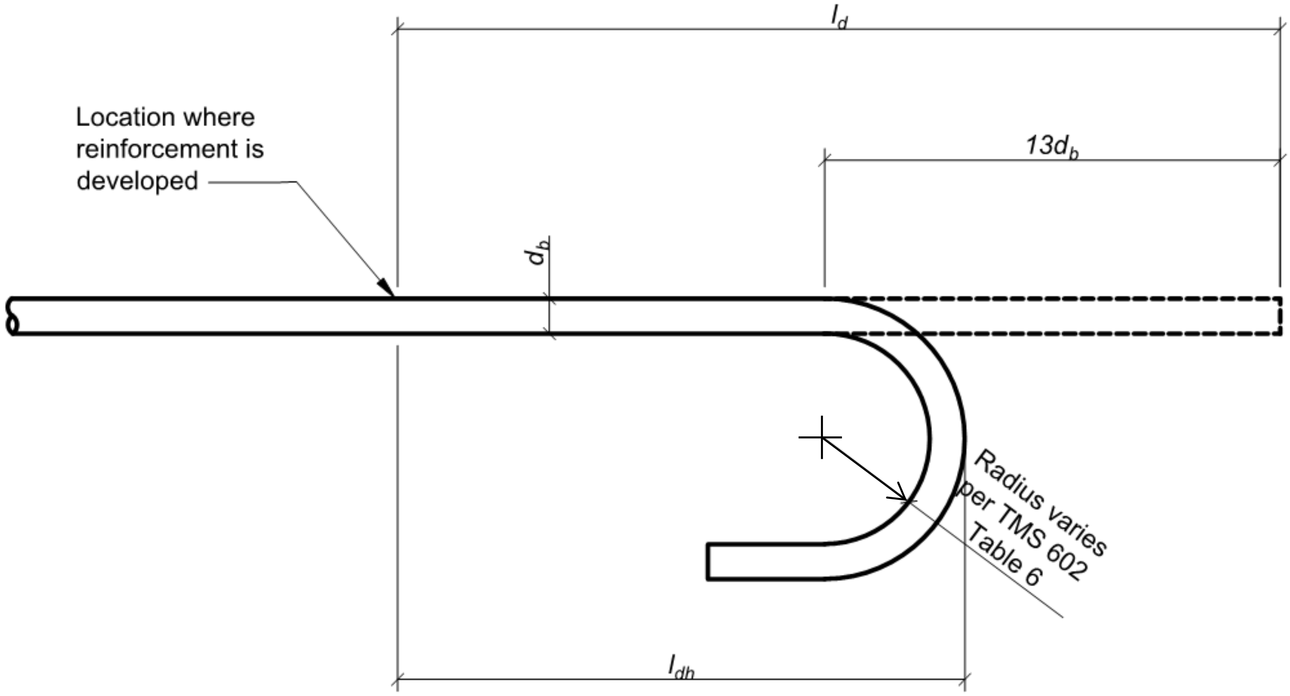
> *In talking with designers, there seems to be confusion about the application of the provision for development of hooked bars in Section 6.1.6.3.3, with some designers believing that $l_e$ is the development length of a hooked bar, and others believing that the development length of a hooked bar is $l_d$ - $l_e$. Can this be clarified?*

**Response/Rationale:**

The intent of the code is that the equivalent length, $\ell_e$, represents the amount by which the straight development length determined from equation 6-1 for deformed wire and Equation 6-2 for deformed bars can be reduced due to the presence of a standard hook. It is for this reason that the code refers to as an "equivalent embedment length" and not a development length.

This ballot proposes to define the development length for a hooked bar, $\ell_{dh}$ , that is measured consistently with the way the development length of hooked bars is measured in ACI 318 – it is measured starting at the outside of the bar or wire at the hook. See the figure proposed for the commentary below.

*Proposed Figure CC-6.1-3 — Hooked development length*




The dimensions at the top of the figure show how development length is intended to be determined by the code as currently written – the development length from equations 6-1 or 6-2 may be reduced when a bar or wire is hooked. The hook is assumed to provide a development length that is equivalent to $13d_b$; i.e. $\ell_e = 13d_b$. As defined in Section 2.1, $\ell_e$ is measured starting at the tangent point of the hook.

The reason that the code is written this way – with the hook allowing a reduction in the straight development length – is that in their earliest incarnation, the code provision for development required that the bar be developed by bond between the steel and the grout for the design stress in the bar. A hook was considered to be able to develop a stress of up to 10,000 psi in the bar. Any stress above 10,000 psi, up to the 20,000 psi allowable, had to be developed in bond stress on the straight portion of the bar. The allowable stress that could be developed in a bar by a hook was later reduced to 7,500 psi.

When development of straight bars was changed from a bond stress model to the current empirical equation, the beneficial effect of the hook was converted from being able to develop a stress in the bar of 7,500 psi to an equivalent embedment length. The equivalent embedment length provided by the hook was determined by taking the length of bar that would be required to develop 7,500 psi in the bar, assuming a 160 psi bond stress.

A more complete discussion of the evolution of these provisions follows.

History of the Provisions for the Development of Masonry Reinforcement

The 1961 UBC referenced the concrete provisions for the development of masonry reinforcement. Those provisions used a bond stress model - the development length was determined based on the force in the bar and an allowable bond stress. Hooks were allowed to carry a load which would produce a stress in the bar equal to 10,000 psi. Since the allowable stress in reinforcement at that time was 20,000 psi, the hook was assumed to be able to reduce the required embedment of a fully stressed bar by ½.

Subsequent versions of the UBC adopted provisions specific to masonry reinforcement, which retained, however, the same basic form as the 1961 UBC. By the 1976 UBC, the stress assumed to be developed by hooks of masonry reinforcement had been reduced to 7,500 psi. By the 1988 UBC, the bond stress provision for the development of straight bars had been replaced by an equation, $l_d = 0.002\ d_b\ F_s$. Although bond stress is no longer explicitly checked by this equation, this equation is still based on bond stress. Hooks were still considered capable of developing 7,500 psi of stress, equivalent to a length of 15 $d_b$.

These provisions remained essentially the same until TMS 402-05. At that time the provision for the development of straight bars departed from the bond stress model and took its present empirical form based on testing of spliced reinforcing bars. Since deformed wires had not been tested, they retained the historical bond stress model, with $l_d = 0.0015\ d_b\ F_s$ based on an allowable bond stress of 160 psi. As a side note, the present equation of $48d_b$ for development of deformed wires was derived by setting $F_s$ equal to 32,000 psi; the development length of deformed wire is still based on a bond stress model assuming an allowable bond stress of 160 psi.

With the development length of bars moving away from approach based on developing stress in the bar, the provision stating that a hook could develop a stress in the bar of 7,500 psi was no longer viable. The commentary to Section 2.1.10.5.1 of TMS 402-05 describes how this dilemma was resolved:

> The allowable stress developed by a standard hook, 7,500 psi, is the accepted permissible value in masonry design. Substituting this value into Eq. (2-8) yields the equivalent embedment length given. This value is less than half of that given in Reference 1.14.

Equation 2-8 was the equation for the development of deformed wire, $l_d = 0.0015\ d_b\ F_s$. The resulting equivalent embedment length was $11.25d_b$ for allowable stress design. For strength design, the value was set at $13d_b$. We have not found an explanation for the difference between ASD and SD, but by TMS 402-13, both methods were using the $13d_b$ value as the amount by which the development length of a bar could be reduced by the presence of a hook.

The following excerpt from the 6th Edition of Amrhein and Porter(2009), which was based on TMS 402-05, illustrates this understanding:

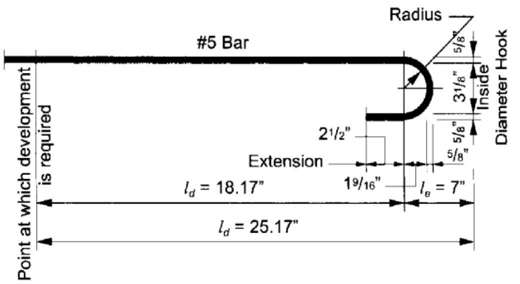
c) Development length provided by hooks:

$l_e = 11.25\ d_b$ (MSJC Code Section 2.1.10.5.1)

$= 11.25(0.625) = 7$ in.

Thus, the remaining development length required for a hooked bar is:

$l_{d\ balance\ required} = l_d - l_e = 25.17 - 7 \cong 18$ in.
(development length in addition to the hook)



The $\gamma_h$ factor in the proposed Equation 6-3 was determined by subtracting from 13$d_b$ the inside radius of hook determined from TMS 602 Table 6 and one bar or wire diameter, resulting in a value for $\ell_{dh}$ measured to the outside of the bar at the hook. This is illustrated in the proposed Figure CC-6.1-3.

Suggested Future Business

As a future business item, it is recommended that the committee next cycle consider whether the existing hooked bar provision is unduly conservative. It is likely that the beneficial effect of the hook is much greater than has been historically assumed.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

$\ell_{dh}$ = development length of hooked reinforcement measured from the outside of the bar or wire at the hook, in. (mm)

~~$\ell_e$ = equivalent embedment length provided by standard hooks measured from the start of the hook (point of tangency), in. (mm)~~

$\gamma$ = reinforcement size factor for straight development length

$\gamma_h$ = reinforcement size factor for hooked development length

**6.1.6.3.3** *Standard hooks* —~~Standard hooks~~The required development length $\ell_{dh}$ of bars and deformed wires terminating in a standard hook in grout subject to tension shall be ~~considered to develop an equivalent embedment length, $\ell_e$ , as~~ determined by Equation 6-3. Hooks shall not be used to develop bars or deformed wires in compression.

~~$\ell_e = 13 d_b$ (Equation 6-3)~~
$\ell_{dh} = \ell_d - \gamma_h d_b$ (Equation 6-3)

$\gamma_h$ = 9.0 for No. 3 (M#10) through No. 8 (M#25) bars and deformed wires; and
$\gamma_h$ = 8.0 for No. 9 (M#29) through No. 11 (M#36) bars

**Code Commentary:**

6.1.6.3.3 Standard hooks — Historically, standard hooks were considered to be able to develop a stress in the bar or wire of 7,500 psi. The remainder of the stress in the bar due to design loads was required to be developed in bond along the straight length of bar starting at the tangent point of the hook. When the bond stress model for development of bars was replaced by Equation 6-2, the 7,500 psi was converted into an equivalent embedment length of 13$d_b$. The minimum distance from the point where the bar needed to be developed to the tangent point of the hook, was determined by subtracting 13$d_b$ from Equation 6-1 or Equation 6-2. Equation 6-3 now defines a hooked development length, $\ell_{dh}$, in a manner consistent with ACI 318. The $\gamma_h$ factor in Equation 6-3 was determined by subtracting from $13d_b$ the inside radius of hook determined from TMS 602 Table 6 and one bar or wire diameter, resulting in a value for $\ell_{dh}$ measured to the outside of the bar at the hook. This is illustrated in Figure CC-6.1-3. It is expected that a more refined and potentially less conservative equation for $\ell_{dh}$ will be developed for a future edition of this Code.

In compression, hooks are ineffective and cannot be used as anchorage.

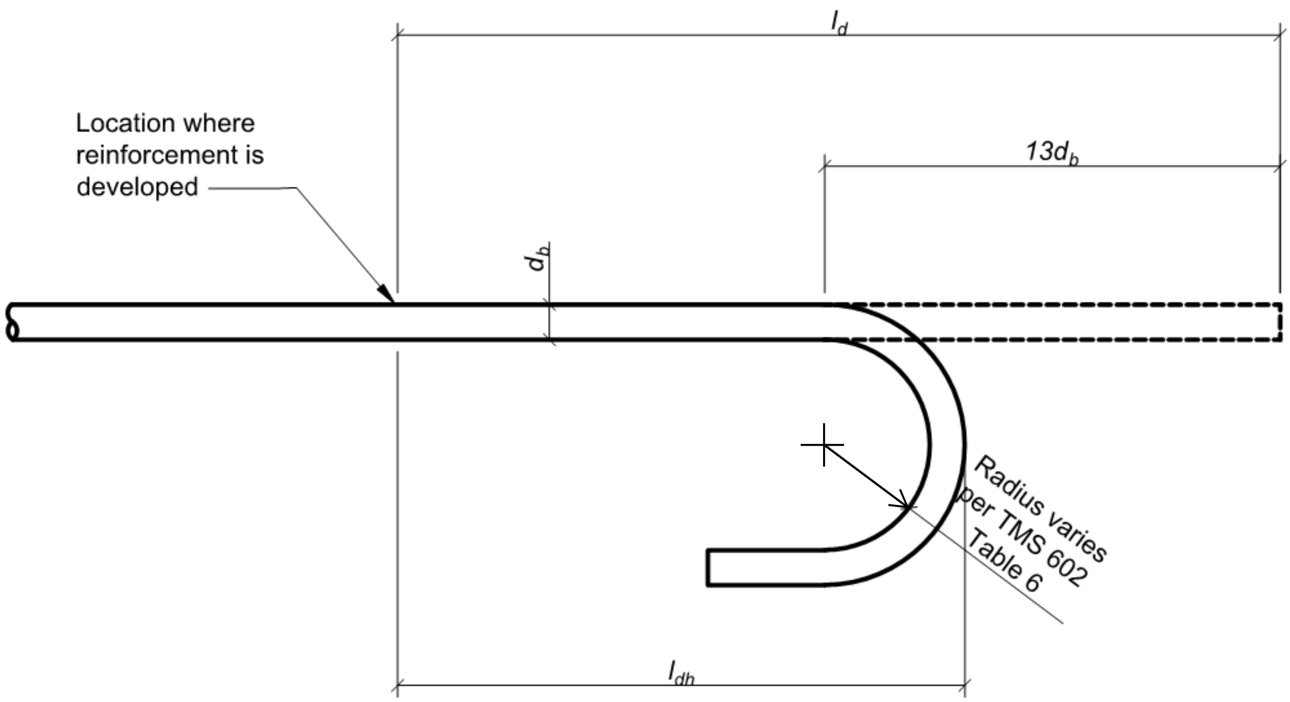


*Figure CC-6.1-3 — Hooked development length*

***Renumber subsequent figures***

**Specification:**

None

**Specification Commentary:**

None.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 9 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 6 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-RC-013 | |
| **Technical Contact/Email:** John M. Hochwalt / john.hochwalt@kpff.com | |
| **Public Comment Number:** 2022 Comment # 63 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 63 read as follows:

> There appear to be no provisions for the anchorage of deformed wire placed mortar and used as shear reinforcing. Can it be terminated with hook like joint reinforcing as illustrated in CC-6.1-4?

**Response/Rationale:**

The comment is correct that while the code allows the use of deformed wire placed in mortar to resist shear, no provisions are provided for the anchorage of deformed wire in that application.

It is proposed to require the same detailing for deformed wire placed in mortar as is used for joint reinforcement, including an option for a detail with enhanced ductility.

In preparing this ballot, it was noted that the deformed wire is not currently subject to the same restrictions as joint reinforcement in Chapter 7. It is suggested that the committee next cycle consider whether the restrictions on joint reinforcement should extended to deformed wire.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown* <u>*underlined*</u> *and deletions are shown* ~~*struck-through*~~*.)  Do not use 'Track Changes'*

**Code:**

**6.1.8** *Shear reinforcement*
Shear reinforcement shall extend to a distance d from the extreme compression face and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Shear reinforcement shall be anchored at both ends for its calculated stress.

**6.1.8.1** *Horizontal shear reinforcement* — Horizontal reinforcement shall meet the requirements of Sections 6.1.8.1.1 through 6.1.8.1.3<u>.</u>

**6.1.8.1.1** Except at wall intersections, the ends of horizontal reinforcing bar or deformed wire <u>embedded in grout</u> shall be bent around the edge vertical reinforcing bar or deformed wire with a 180-degree standard hook.

**6.1.8.1.2** At wall intersections, horizontal reinforcing bars or deformed wire <u>embedded in grout</u> shall be bent around the edge vertical reinforcing bar or deformed wire with a 90-degree standard hook and shall extend horizontally into the intersecting wall a minimum distance at least equal to the development length.

<u>**6.1.8.1.3** Deformed wire embedded in mortar and used as shear reinforcement shall be anchored by either:</u>
<u>(a) A 90-degree bend in longitudinal wires bent around the edge cell and with at least 3-in. (76-mm) bend extensions in mortar or grout, or</u>
<u>(b) A 90-degree bend in longitudinal wires bent around the edge cell and with at least 4-in. (102-mm) overlap of the wires in mortar or grout.</u>

**6.1.8.1.~~3~~<u>4</u>** Joint reinforcement used as shear reinforcement shall be anchored around the edge reinforcing bar or deformed wire in the edge cell, either by placement of the vertical reinforcement between adjacent cross-wires or with a 90-degree bend in longitudinal wires bent around the edge cell and with at least 3-in. (76-mm) bend extensions in mortar or grout.

**6.1.8.1.~~3~~<u>4</u>.1** Where the joint reinforcement consists of two longitudinal wires, both of the wires shall be anchored~~either~~ by one of the following:
(a) Placement of the vertical reinforcement between adjacent cross-wires, or
(b) A 90-degree bend in longitudinal wires bent around the edge cell and with at least 3-in. (76-mm) bend extensions in mortar or grout, or
(c) A 90-degree bend in longitudinal wires bent around the edge cell and with at least 4-in. (102-mm) overlap of the wires in mortar or grout.

**6.1.8.1.~~3~~<u>4</u>.2** Where the joint reinforcement consists of four longitudinal wires, all four of the wires shall be anchored by either:
(a) A 90-degree bend in the inner longitudinal wires bent around the edge cell and with at least 3-in. (76-mm) bend extensions in mortar or grout, and a 3/16 in. (5 mm) U-stirrup lapped at least 8-in. (205-mm) with the outer wires, or
(b) A 90-degree bend in both the inner and outer longitudinal wires bent around the edge cell and with at least 4-in. (102-mm) overlap of the wires in mortar or grout.

**Code Commentary:**

<u>**6.1.8.1.3** The options for the anchorage of deformed wire in mortar are based on the provisions for the anchorage of joint reinforcement - 6.1.8.1.3 (a) is equivalent to 6.1.8.4.1 (b) for joint reinforcement, and 6.1.8.1.3 (b) is equivalent to 6.1.8.4.1 (c) for joint reinforcement. The joint reinforcement options in Section 6.1.8.4.1 are depicted in Figure CC-6.1-4; deformed wire would appear the same except that no cross wire would be present.</u>

<u>6.1.8.1.3 (b) is intended for use in applications where enhanced ductility is desirable. As discussed in the Code Commentary Section 6.1.8.4, testing of the detail in four-wire joint reinforcing suggests it provides ductility suitable for use in Special Reinforced Masonry Shear Walls.</u>

**Renumber subsequent commentary sections accordingly.**

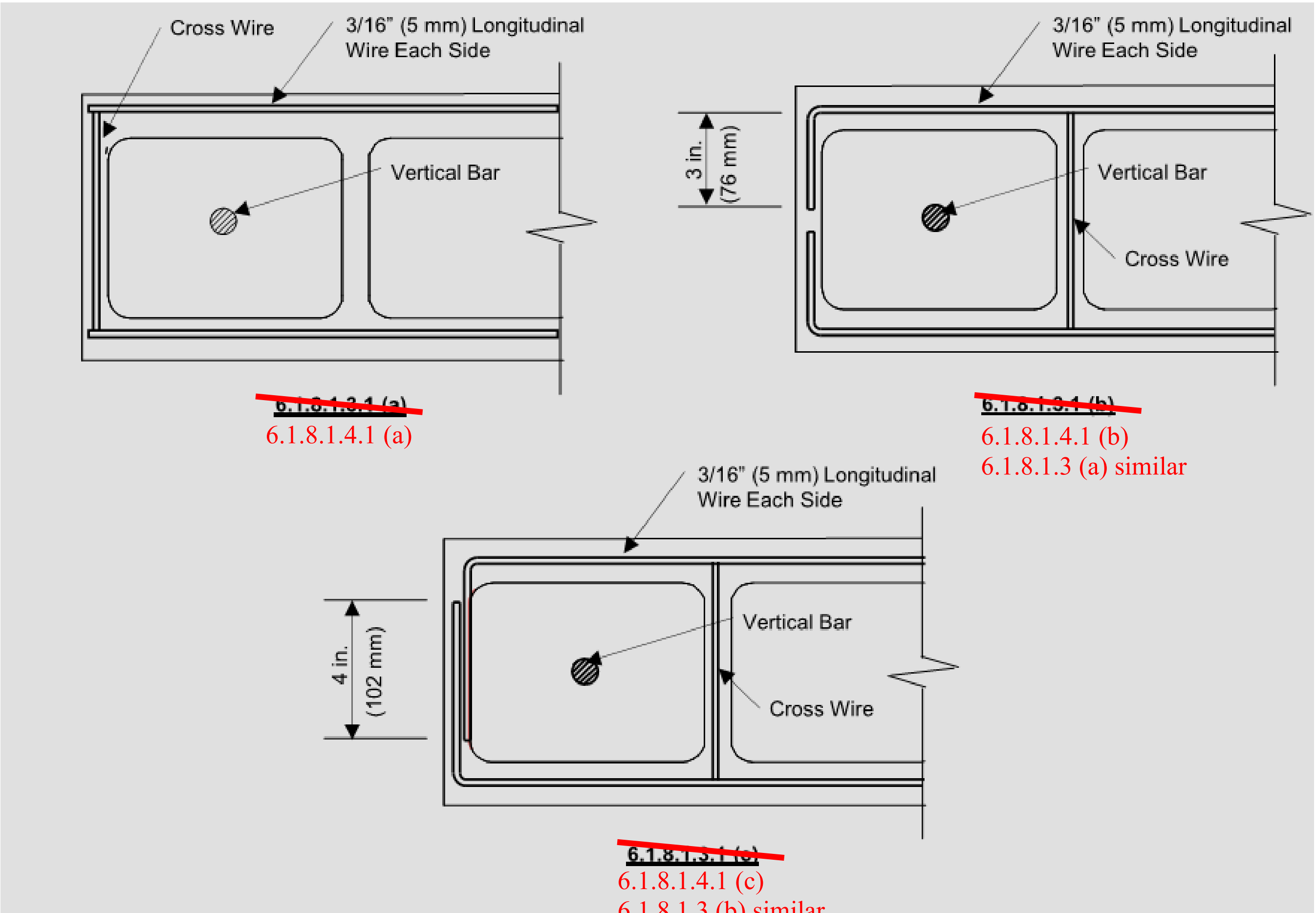


*Figure CC-6.1-4 — Anchorage of joint reinforcement with two longitudinal wires*

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**Specification:**

None.

**Specification Commentary:**

None.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 9 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 6 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| Item #: 20-RC-015 | |
| Technical Contact/Email: John M. Hochwalt / john.hochwalt@kpff.com | |
| Public Comment Number: 2022 Comment # 86 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 6/1/2021 |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

*Public Comment 86 reads as follows:*

> *There are no limitations on the size of mechanical splices or requirements for their placement and protection. It is suggested mechanical splices be subject to the size limits of 6.1.3.2.4 and 6.1.3.2.5 (laps included limit); the placing requirements of 6.1.4.3 and 6.1.4.5, and the protection requirements of 6.1.5.1.*
>
> *In addition, mechanical splices are not addressed in TMS 602. It is suggested to list mechanical splices as required submittal in Section 1.5, and to address the installation of mechanical splices (in accordance with manufacturer's instructions) in 3.4 B.7. The installation instructions should also reference compliance with other relevant requirements such as 3.4 B.3, 3.4 B.4, 3.4 B.5.*

**Response/Rationale:**

The committee agrees that the code and specification should provide additional criteria for mechanical couplers to ensure that their size does not create problems for grout placement. The following rationale is provided for selected proposed provisions:

> 6.1.7.2.3 (a) – Some mechanical splices have a non-circular cross section, such that diameter cannot be used to characterize the size of splices. The greatest cross-section dimension is proposed for consistency with the least grout space dimension that is used to establish the 1/3 limit.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**6.1.7.2** *Mechanical splices*

**6.1.7.2.1** *Bar reinforcement* — Mechanical splices shall have the bars connected to develop in tension or compression, as required, at least 125 percent of the specified yield strength of the bar.

**6.1.7.2.2** *Deformed wire reinforcement* — Mechanical splices shall have the deformed wires connected to develop the specified tensile strength of the wire. Mechanical splices shall not be used for deformed wire placed in mortar.

**6.1.7.2.3** *Size and placement* – Mechanical splices shall meet the following additional requirements:

(a) The greatest cross-sectional dimension of the mechanical splice shall not exceed one-third of the least dimension of the gross grout space in which it is placed.

(b) The cross-sectional area of the mechanical splice shall be treated as lapped reinforcement for the purpose of determining compliance with Section 6.1.3.2.5.

(c) The clear distance limitations between bars and between deformed wires required in Sections 6.1.4.1 and 6.1.4.2 shall also apply to the clear distance between a mechanical splice and adjacent splices or reinforcement. For the purpose of this provision, consider the nominal diameter of the splice to be the greatest cross-sectional dimension of the mechanical splice.

(d) The thickness of grout between the mechanical splice and the masonry units shall comply with Section 6.1.4.5.

(e) The mechanical splice shall have a masonry cover of 2 in. (50.8 mm) from any masonry face exposed to earth or weather and 1 ½ in. (38.1 mm) from all other masonry faces.

**Code Commentary:**

**6.1.7.2** *Mechanical splices*

**6.1.7.2.1** *Bar reinforcement* — ~~Full m~~Mechanical splices are also required to develop 125 percent of the specified yield strength in tension or compression as required, for the same reasons discussed for full welded splices.

**6.1.7.2.2** Deformed wire reinforcement — Mechanical splices of deformed wire are required to develop the specified tensile strength of the deformed wire instead of 125 percent of the yield strength as is required for reinforcing bars because the minimum specified tensile strength (85 ksi) of ASTM A1064 deformed wire is less than 125 percent of the minimum specified yield strength (75 ksi). Mechanical couplers that have been developed and tested for reinforcing bars may not be suitable for deformed wires due to differences in yield strength and deformations. Mechanical splices of deformed wires in mortar is not permitted because the coupler does not fit in the mortar joint.

**6.1.7.2.3** *Size and placement* – This section adapts the size limitations and placement requirements of Sections 6.1.3, 6.1.4 and 6.15 to mechanical splices, to maintain appropriate clearances for grouting and protection of the mechanical splice. If multiple bars are mechanically spliced in the same grout space, the splices may be staggered to achieve compliance with this section.

**Specification:**

**1.5 — Submittals**

. . .

**1.5 B.** Submit the following:

. . .

2. Material certificates — Material certificates for the following, certifying that each material is in compliance.

a. Reinforcement

b. Mechanical splices

c ~~b~~. Anchors, ties, fasteners, and metal accessories

d ~~c~~. Masonry units

e ~~d~~. Mortar, thin-bed mortar for AAC, and grout materials

f ~~e~~. Self-consolidating grout

g ~~f~~. Lath, scratch coat and setting bed mortar

. . .

**2.4 — Reinforcement, prestressing tendons, and metal accessories**

. . .

**2.4 G.** *Mechanical splices* - Provide mechanical splices that have been demonstrated to develop in tension or compression at least 125 percent of the specified yield strength of the reinforcement. Where indicated, provide mechanical splices that have been demonstrated to develop the specified tensile strength of the reinforcement. Mechanical splices shall be certified for compatibility with the type of reinforcement being spliced.

**Renumber subsequent sections**

. . .

**3.4 B**. Reinforcement

. . .

3. Maintain clear distance between reinforcing bars or mechanical splices and the interior of masonry unit or formed surface of at least 1/4 in. (6.4 mm) for fine grout and 1/2 in. (12.7 mm) for coarse grout, except where cross webs of hollow units are used as supports for horizontal reinforcement. Maintain the same clear distance when deformed wire is specified to be embedded in grout.

4. Place reinforcing bars, ~~and~~ deformed wire, and mechanical splices in grout maintaining the following minimum cover:

   a. Masonry face exposed to earth or weather: 2 in. (50.8 mm) for bars larger than No. 5 (M #16) and mechanical splices; 1½ in. (38.1 mm) for deformed wire and No. 5 (M #16) bars or smaller.
   b. Masonry not exposed to earth or weather: 1½ in. (38.1 mm).

5. Maintain minimum clear distance between parallel bars, ~~and~~ parallel deformed wires, and mechanical splices of the nominal reinforcement size or 1 in. (25.4 mm), whichever is greater. For mechanical splices, the reinforcement size is the greatest cross-sectional dimension of the mechanical splice.

. . .

7. Splice reinforcement only where indicated on the Project Drawings, unless otherwise acceptable. When splicing bars with mechanical splices, comply with manufacturer's installation requirements. When splicing bars by welding, provide welds in conformance with the provisions of AWS D 1.4. When splicing wire reinforcement by welding, provide welds as specified. When splicing reinforcement by lapping, provide lap length that meets or exceeds the lap length specified.

**Specification Commentary:**

**2.4 G.** *Mechanical splices* - The strength of mechanical splices is typically demonstrated through testing performed by an independent certification agency. Mechanical splices developing 125 percent of the specified yield strength are commonly referred to as Type 1 splices whereas mechanical splices developing the specified tensile strength are commonly referred to as Type 2 splices. The deformations of deformed bars and deformed wire are different; a splice developed for one type of reinforcement may not develop the intended capacity when used with the other type of reinforcement.

**Renumber subsequent sections**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 9 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 6 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|

**Item #:** **20-RC-016**

**Technical Contact/Email:** Scott Walkowicz / scott@walkowiczce.com

**Public Comment Number:** 2022 Comment # 127

| Public Comment Response Based on TMS 402/602 Draft Dated | 6/1/2021 |
|---|---|

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 127 reads as follows:

> Using the term net instead of gross would be more appropriate.

**Response/Rationale:**

This comment refers to the definition and commentary of the term "Grout space, gross" reproduced below for voter convenience:

> Code:
>
> *Grout space, gross* — The area or dimensions available within the continuous grouted cell, core, bond beam course, or collar joint, considering the effect of unit offset in adjacent courses but neglecting possible mortar protrusions and the presence of perpendicular reinforcement, if any.
>
> Commentary:
>
> *Grout space, gross* — The gross grout space is used to evaluate design limitations on reinforcement size and quantity. Depending upon the configuration of the masonry unit, the gross grout space in running bond may be smaller than the gross grout space in stack bond. Refer to Figure CC-2.2-2 for an example.

Response:

The Committee respectfully disagrees with the Commenter. The term, Gross Grout Space, is new to TMS 402 this code cycle. There were many discussions before settling on the term "gross" to describe the grout space actually available for units of varying web configurations and bond patterns. While the area noted could technically be considered a 'net' space, when recognizing that it is the remaining available space for grout, after considering unit dimensions, geometry and placement, it is noted as 'gross' because it is the 'gross' continuous space available in the masonry assembly and does not include deductions for mortar extrusions, vertical bars, horizontal bars or similar intrusions into the available grout space. A ballot in response to PC211 proposes clarifying language as to the exclusion of deductions for such intrusions and should help clarify the 'gross' terminology.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

None

**Code Commentary:**

None

**Specification:**

None.

**Specification Commentary:**

None.

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 9 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 6 | *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|

| | |
|---|---|
| **Item #:** | **20-RC-017** |
| **Technical Contact/Email:** | Scott Walkowicz / scott@walkowiczce.com,<br>Adam Hutchinson / ahutchinson@nwcma.org |
| **Public Comment Number:** | 2022 Comment # 211 |

| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |
|---|---|

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 211 reads as follows:

> *Commentary Figure CC-6.1-1 is a great aid in helping designers understand and then verify available gross grout space. It is, however, mostly representative of CMU although figure (b) may somewhat represent certain structural clay units. Please consider adding additional figures to show a couple generic structural clay unit configurations and their resulting gross grout area when laid in one-half running bond.*
>
> *Consider adding a sentence or two of verbal Commentary to accompany the figure and to remind users to consider their locally available unit geometry and/or the effects of different bond patterns, corbeling or other detailing that may affect the available gross grout space.*
>
> *Also consider adding a verbal Commentary that the Gross Grout Space does not include mortar extrusions, other vertical or horizontal bars, etc... and is based solely on the unit geometry and dimensions, while noting that concrete units are molded and commonly have a taper, being thicker at the top when laid, and that clay units are generally constant thickness due to being an extruded unit.*

**Response/Rationale:**

The subcommittee agrees that the commentary figures and language can be improved to incorporate clay units as follows:

1. Add figures to illustrate common structural clay units in one-half running bond to aid designers in understanding the effect of clay unit configuration on Gross Grout Space. Additional figures are proposed.
2. Add Commentary language to remind users to verify local (or selected) unit configurations when calculating the Gross Grout Space area. Commentary language has been proposed.

3. Add Commentary to clarify that Gross Grout Space includes only the unobstructed space available for grout in a continuous open core. Commentary language has been proposed.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

None

**Code Commentary:**

**6.1.3.2.5** The limitations on maximum reinforcement percentage are based on the gross grout space presented by the cell, bond beam course, collar joint, or AAC masonry core. These limitations are in contrast to the requirements for grout placement in TMS 602 Table 7, which are based on the net grout space per Footnote 3 and TMS 602 Figure SC-21. The limitations of Section 6.1.3.2.5 are intended to avoid overreinforcing, while the limitations of TMS 602 Article 3.5C are intended to prevent problems with grout consolidation. The alternative provisions presented in Table 6.1.3.2.5.1 and Table 6.1.3.2.5.2 provide a simplified method of determining the maximum vertical reinforcement permitted by TMS 402 when designing vertically reinforced two-celled hollow concrete masonry and hollow clay masonry even though the dimensions of the unit cross-section are unknown before the units have been ordered by the contractor. Because these provisions are simplified, they are also conservative. Designers who know the cross-sectional dimensions of the units to be used on the project may be able to specify greater amounts of reinforcement than those shown in these Tables, especially for units greater than 6-in. (152 mm) in thickness. The percentages in these Tables were correlated to the values in Table 6.1.3.2.5 and are based on "per 8-in. (203 mm) length" (per cell or core for two-celled units), with a footnote to address nominal 12-in. (305 mm) long clay units that have a 6-in. (152 mm) length per core or cell.~~.~~ Table 6.1.3.2.5.1 applies to units laid in one-half running bond (units overlap 50% of their length) and Table 6.1.3.2.5.2 applies to units laid in stack bond (unit overlap 100% of their length). Figure CC-6.1-1 illustrates two-celled flanged units, jamb units, and open-end units laid in one-half running bond<u> for typical CMU and clay units</u>.

<u>Concrete and clay masonry unit configurations can vary regionally and between manufacturers due to local production preferences. Consult producers local to the project to develop expected unit geometric parameters prior to calculating gross grout space. Other detailing aspects such as corbeling and varied unit overlap can also affect the available gross grout space. Include sufficient notes and/or details to illustrate necessary unit geometry and unit placement limits for compliance with the design basis. Refer to Figure CC-6.1-1 for illustrations of several unit possibilities in one-half running bond pattern. Other bond patterns and unit alignment should be considered when calculating the gross grout space.</u>

<u>Section 6.1.3.2.5, and Table 6.1.3.2.5, have been developed for use with a calculated gross grout space area and that space is the gross area available for grout based solely on the unit geometric properties and placement (bond, alignment, corbeling, etc.). Note that concrete masonry units typically include a taper for mold removal and, therefore, are thicker at their tops and the maximum thickness should be used when calculating the gross grout space. Structural clay units are typically extruded and maintain constant wall thickness throughout their depth. The effects of other items such as mortar extrusions, vertical and horizontal bars, etc., should not be included in the calculation of gross grout space.</u>

Table CC-6.1.3.2.5.1 shows the maximum size and quantity of vertical reinforcement permitted by Sections 6.1.3.2.5, 6.1.3.2.5.1, and 6.1.3.2.2 for two-celled masonry units laid in one-half running bond.

Table CC-6.1.3.2.5.2 shows the maximum size and quantity of vertical reinforcement permitted by Sections 6.1.3.2.5, 6.1.3.2.5.2, and 6.1.3.2.2 for two-celled masonry units laid in stack bond. Tables CC-6.1.3.2.5.1 and CC-6.1.3.2.5.2 do not include nominal unit thicknesses less than 6-in. (152 mm) as there are no commercially available two-celled units with an 8-in. (203 mm) module. Table CC-6.1.3.2.5.3 shows the maximum size and quantity of vertical reinforcement permitted by Sections 6.1.3.2.4, 6.1.3.2.5.1, and 6.1.3.4.4 for two-celled, 12-in. (305 mm) long clay masonry units. The maximum reinforcement listed in both tables may be doubled at lap splice locations.

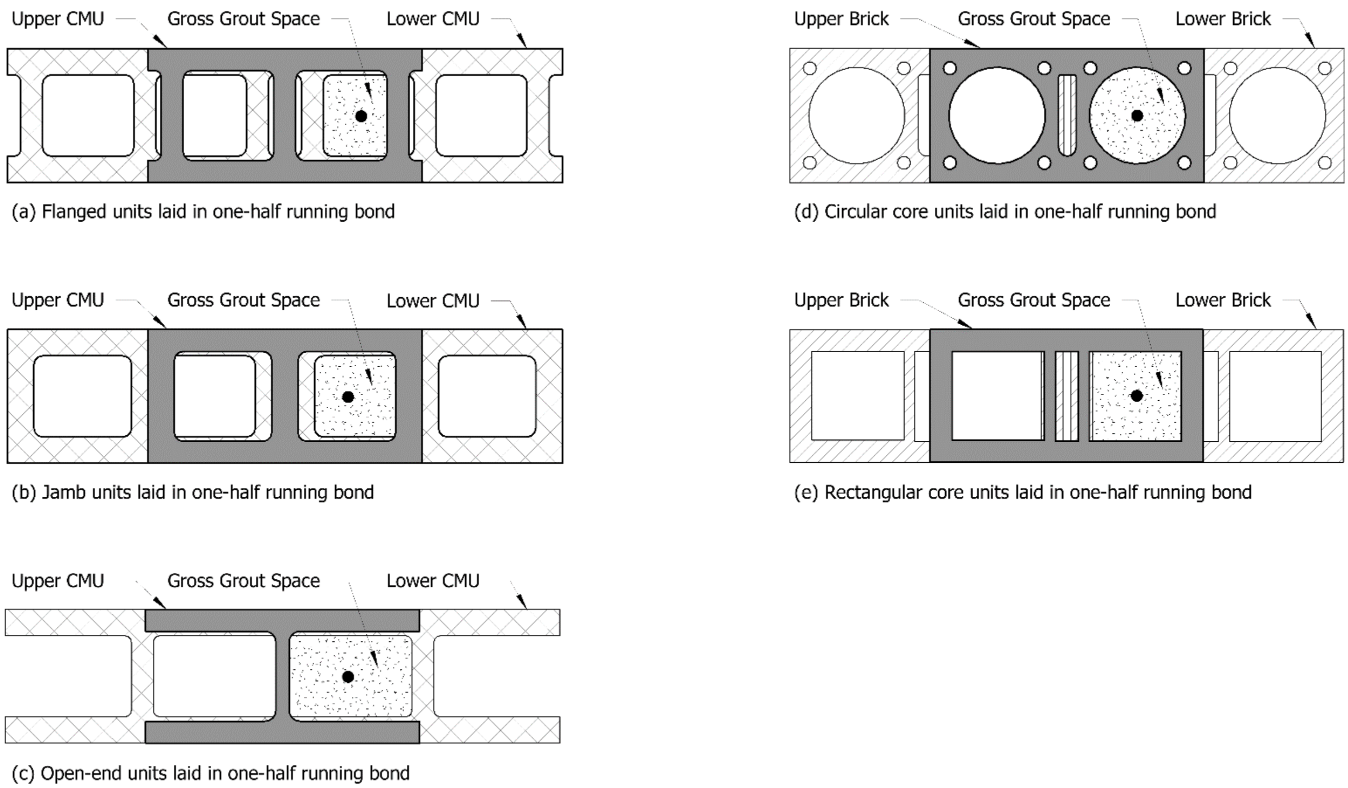


*Figure CC-6.1-1 – Two-celled flanged units, jamb units, and open-end units laid in one-half running bond* <u>*for concrete masonry units (a), (b), (c), and clay units (d), (e)*</u>

**Specification:**

None.

**Specification Commentary:**

None.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 8 *Affirmative* | 1 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 6 *Did not vote* |

**Subcommittee Comments:**

The affirmative with comment vote from Hochwalt is as follows:

*It seems like the phrase "Include sufficient notes and/or details to illustrate necessary unit geometry and unit placement limits for compliance with the design basis" belongs not in the commentary but in Section 1.2. Section 1.2.3 does provide a "catch all" requirement, but it would be good to make this specific to include bond pattern and either unit geometry or minimum gross grout space.*

*I also think the phrase "Other bond patterns and unit alignment should be considered when calculating the gross grout space" is potentially confusing. I think we are trying to say that if you are considering something other than stack bond or half unit running bond, that you need to look at what that means for gross grout space. As it reads, though, it might lead some to think that it is expected that the designer consider "what if" scenarios.*

This comment has not been addressed by the subcommittee and is provided here for Main Committee voter information.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-SL-003 | |
| **Technical Contact/Email:** John M. Hochwalt / john.hochwalt@kpff.com | |
| **Public Comment Number:** 2022 Comment # 87 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**NOTE TO VOTER—THIS BALLOT IS TO SUPPORT THE SUBCOMMITTEE RECOMMENDATION TO FIND THE NEGATIVE VOTER NON-PERSUASIVE. INFORMATION BETWEEN THE ASTERISKS (*) IS THE ORIGINAL BALLOT, FOR INFORMATION ONLY.**

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**Public Comment:**

Public Comment 87 read as follows:

> The prescriptive reinforcement for non-participating elements in SDC C+ is permitted to be placed in either the horizontal or vertical direction. Should this prescriptive reinforcement be required to be placed in the direction of span? Providing horizontal reinforcement, for example, in a wall spanning vertically would seem to offer little improvement to the integrity of the wall.

**Response/Rationale:**

While the current provisions allow the minimum prescriptive reinforcement to be placed in either the horizontal or vertical direction, regardless of the direction of the wall span, this may not achieve the intent of in enhancing the wall integrity in areas of higher seismic risk. Maintaining the post-cracking integrity of the wall requires the presence of a minimum amount of reinforcing that crosses the potential failure plane.

For example, consider a wall spanning in the vertical direction, ungrouted and with joint reinforcing in the bed joints only. The potential failure plane in such a wall is the bed joint. Should the modulus of rupture in the bed joint be exceeded, the wall will crack across the bed joint. There will be no post-cracking ductility because there is no reinforcing crossing the crack. Mandating vertical reinforcing in this condition provides a minimum amount of post-cracking ductility.

Lastly, the ballot proposes the deletion of the sentence of commentary that reads “If reinforcement is required, it must be provided in the direction of the span.” While this is addressing required reinforcement, not the prescriptive reinforcement which was the subject of the public comment, the commentary no longer has a purpose if the prescriptive reinforcement is required to be placed in the direction of the span. There is no longer a reason for the user to think that the required reinforcement would be placed in any direction other than the direction of the span.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)* *Do not use 'Track Changes'*

**Code:**

**7.4.3.1** *Design of nonparticipating elements* — Nonparticipating masonry elements shall comply with the requirements of Section 7.3.1 and Chapter 8, 9, 10, 11, 12, 15, or Appendix D. Nonparticipating masonry elements, except those constructed of AAC masonry, shall be reinforced in the direction of span ~~in either the horizontal or vertical direction~~ in accordance with Sections 7.4.3.1.1 and 7.4.3.1.2.

**7.4.3.1.1** *Horizontal reinforcement* — In walls spanning horizontally, h~~H~~orizontal reinforcement shall be provided within 16 in. (406 mm) of the top and bottom of nonparticipating masonry walls and shall consist of one of the following:

(a) Two longitudinal wires of W1.7 (MW11) joint reinforcement spaced not more than 16 in. (406 mm) on center. The space between these wires shall be the widest that the mortar joint will accommodate.

(b) Two D2 (MD13) deformed wires spaced not more than 16 in. (406 mm) on center for walls greater than 4 in. (102 mm) in width and at least one D2 (MD13) wire spaced not more than 16 in. (406 mm) on center for walls not exceeding 4 in. (102 mm) in width. Where two deformed wires are used, the space between these wires shall be the widest that the mortar joint will accommodate.

(c) One No. 4 (M #13) bar or one D20 (MD129) wire spaced not more than 48 in. (1219 mm) on center.

**7.4.3.1.2** *Vertical reinforcement* — In walls spanning vertically, v~~V~~ertical reinforcement shall consist of at least one No. 4 (M #13) bar or one D20 (MD129) wire spaced not more than 120 in. (3048 mm). Vertical reinforcement shall be located within 16 in. (406 mm) of the ends of masonry walls.

**Code Commentary:**

**7.4.3.1** *Design of nonparticipating elements* — Reinforcement requirements of Section 7.4.3.1 ~~are traditional for conventional concrete and clay masonry. They~~ are prescriptive in nature. The intent of this requirement is to provide structural integrity for nonparticipating masonry walls by ensuring that a minimum amount of reinforcing is present in the direction of the span should the seismic induced moment exceed the cracking strength of the masonry. AAC masonry walls differ from concrete masonry walls and clay masonry walls in that the thin-bed mortar strength and associated bond strength is typically greater than that of the AAC units. Also, the unit weight of AAC masonry is typically less than one-third of the unit weight of clay or concrete masonry, reducing seismic inertial forces. This reduced load, combined with a tensile bond strength that is higher than the strength of the AAC material itself, provides a minimum level of structural integrity. Therefore, prescriptive reinforcement is not required. All masonry walls, including non-participating AAC masonry walls, are required to be designed to resist out-of-plane forces. ~~If reinforcement is required, it must be provided in the direction of the span.~~ Permitted types of reinforcement are defined in Section 6.1.1. Commentary Section 6.1.3 provides additional information.

**Specification:**

None

**Specification Commentary:**

None

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 10 | *Affirmative* | 0 | *Affirmative w/ comment* | 1 | *Negative* | 0 | *Abstain* | 9 | *Did not vote* |

**Subcommittee Comments:**

The negative voter, Robinson, commented as follows:

> *The line indicated to be deleted "If reinforcement is required, it must be provided in the direction of the span." is not about the prescriptive reinforcement. This is about reinforcement required to resist out-of-plane loads. Therefore, it should not be deleted.*

From discussion with the negative voter, the subcommittee chair came understands that the concern was that proposed change was outside the scope of the public comment. The subcommittee chair has added a third paragraph to the rationale to explain how the proposed change relates to the public comment.

*****************************************************************************************

**Negative Vote (J Thompson)**

This comment comes up every cycle or two. The purpose of this prescriptive seismic reinforcement for nonparticipating elements is not to add an undefined increase in strength to the element - nor is it to increase the ductility of these isolated elements. The initial design checks determine whether nonparticipating elements can be designed as reinforced or unreinforced - and if the latter, then these prescriptive reinforcement minimums kick in. Yet, many argue these provisions are already unnecessary - analogous to verifying that everything checks for an ordinary plain shear wall...but still requiring it to be reinforced for extra precaution. Might be an individual designer's take, but shouldn't be a code minimum.

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**Subcommittee Meeting Discussion:**

By a vote of 9 affirmative and two abstentions, the subcommittee voted to find Thompson's negative vote non-persuasive. The subcommittee agreed with the public commenter that reinforcing provided perpendicular to the direction of span will not enhance the integrity of the wall, which is the expressed intent of the prescriptive reinforcement.

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**Additional Comments From Subcommittee**

The subcommittee agrees with the negative voter that the prescriptive reinforcing is not intended to provide an increase in strength or an increase in ductility. As the commentary states, however, the prescriptive reinforcement is intended to maintain the integrity of the wall in Seismic Design Categories C and higher. To achieve this intent the reinforcement needs to be placed in the direction of the span.

It is a general principal of seismic design that the structures and non-structural components that pose a significant risk to life safety (like a masonry partition wall that can fall on building occupants or block intended egress routes) need to be withstand greater seismic shaking than currently predicted by code without collapse.

It is for the this reason that structures in higher seismic design categories are required to have ductile detailing, regardless of what the computed seismic demands are. It is for this reason that non-participating masonry walls should be required to have the minimum prescriptive integrity reinforcement placed in the direction of span.

--------------

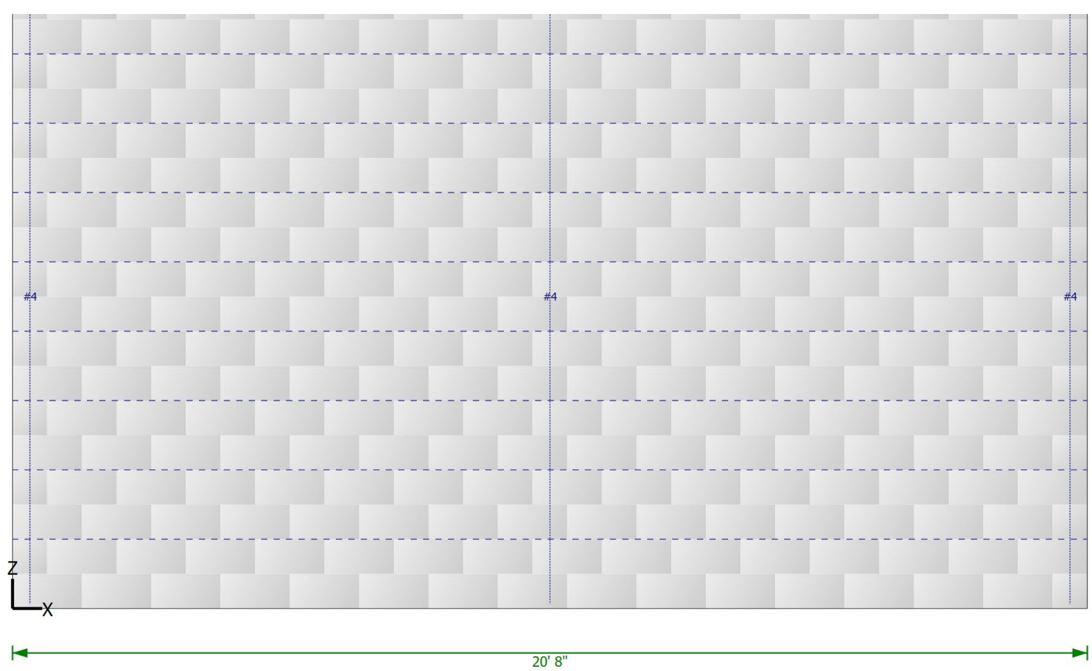
**Additional Comments From Negative Voter**

In general, if an assembly is properly designed and detailed (which in this context would require isolating the non participating element from the lateral force resisting system), then a minimum code shouldn't require extra reinforcement just because. If these elements can be designed as unreinforced to respond elastically, then adding the steel provides no benefits for strength or ductility.

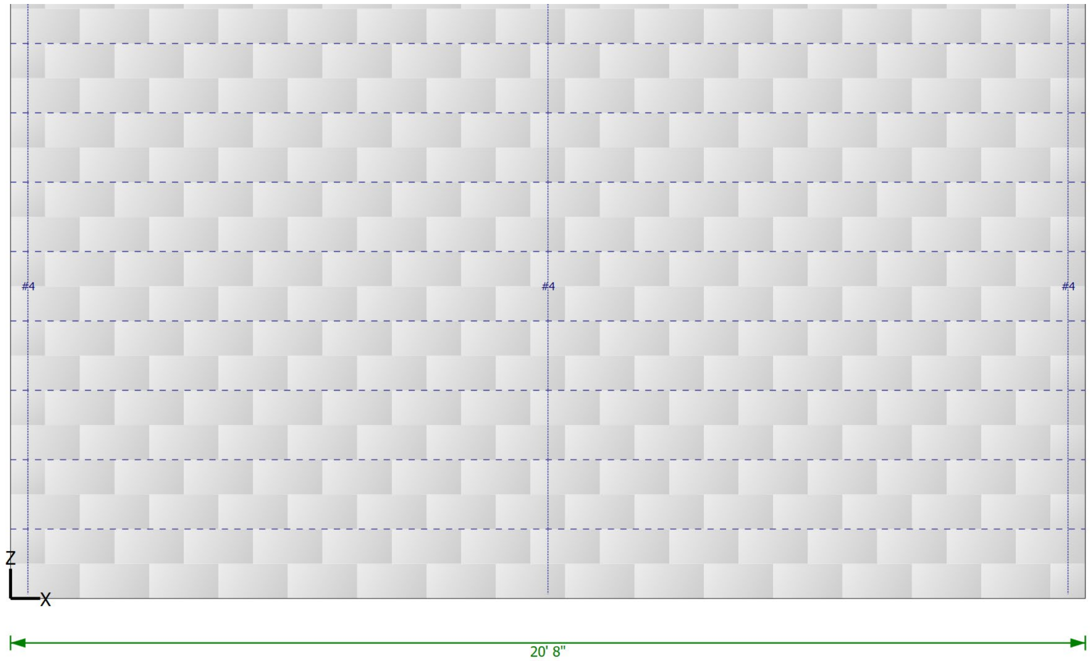
In the end, this is just a complicated way of requiring partitions to be reinforced masonry...but still designing them for R 1.5.

If the committee wants to mandate reinforced partitions, let's have that discussion rather than this approach of requiring reinforced masonry to be designed for an elastic response.

As a practical matter, if a lightly loaded, non-participating partition were spanning in the vertical direction and designed as reinforced, the least amount of vertical reinforcement would be spaced at 120 inches. For crack control, this wall would also likely contain bed joint reinforcement at 16 inches on center over its height to yield a design that may look like the following:




The same wall designed as unreinforced would still have bed joint reinforcement for crack control and with the proposed language, also contain vertical reinforcement at 120 inches…producing the following:




It’s a convoluted path, but the proposed language has the effect of prohibiting unreinforced non-participating elements.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-SL-004 | |
| **Technical Contact/Email:** John M. Hochwalt / john.hochwalt@kpff.com | |
| **Public Comment Number:** 2022 Comments # 90 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 90 read as follows:

> Since "shear reinforcements" is now a defined term, it is suggested to replace the phrase "reinforcement required to resist in-plane shear" in six locations in this section with "shear reinforcement."

**Response/Rationale:**

In Section 2.2, Shear reinforcement is defined as:

> *Reinforcement, shear* — Reinforcement required for compliance with Section 8.3.5, Section 9.3.3.1.2, or Section 11.3.4.1.2.

As the public comment notes, the provisions for Special Reinforced Masonry Shear Walls in Section 7.3.2.5 use the terminology "reinforcement required to resist in-plane shear." The only provisions in TMS 402 that require the use of reinforcement to resist shear are those referenced in the definition of "shear reinforcement" in Section 2.2. It is proposed to simplify the wording in Section 7.3.2.5 by using the defined term of "shear reinforcement."

This ballot was previously balloted as 19-SL-04 and received one negative vote on the main committee ballot from Pierson which read as follows:

> In my humble opinion, the provision as now written is clearer. I understand where we are trying to go here, but "in-plane shear reinforcement" does not seem as clear to me. Probably because "in-plane" is acting as an adjective and it could be interpreted to apply to either "shear" or to "reinforcement". I don't think we have "Out-of-plane" shear reinforcement, so one could argue that the descriptor "In-plane" is not really required.
>
> Also, vertical steel in shear walls does resist what we call shear forces (more correctly they are diagonal tension, I think) and could technically be considered "shear reinforcement". That's one reason I would like to keep "horizontal" in these provisions.

Also, what about a wall that is supported on piles every 30 ft? That wall will have in-plane shear forces applied in the vertical direction, so what is the "in-plane shear reinforcement" in that case?

The subcommittee agreed with Pierson about the potential confusion that could result from replacing “horizontal” with “in-plane” and was supported by the main committee in finding the negative vote persuasive. This ballot leaves the existing language alone aside from replacing “reinforcement required to resist in-plane shear” with “shear reinforcement.”

This ballot has includes a change proposed to the commentary for item (f) to reflect an affirmative with comment vote by Bennett on 19-SL-05:

> *By using the phrase "In previous editions of the Code," a change will need to be made in the 2028 Commentary. It is easy to forget to do that. I would suggest editorially changing to" Prior to the 2022 edition of this Code".*

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**Note: The text below reflects changes made to this section through ballots 19-SL-002, 19-SL-005 and 19-SL-007.**

**7.3.2.5** *Special reinforced masonry shear walls* — Design of special reinforced masonry shear walls shall comply with the requirements of Section 8.3, Section 9.3, or Appendix C. Reinforcement detailing shall also comply with the requirements of Section 7.3.2.2.1 and the following:

(a) In-plane flexural reinforcement shall be deformed reinforcing bars.

(b) The maximum spacing of vertical reinforcement shall be the smallest of one-third the length of the shear wall, one-third the height of the shear wall, and 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry not laid in running bond.

(c) The maximum spacing of horizontal reinforcement shall not exceed 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry not laid in running bond.

(d) The maximum spacing of horizontal <u>shear</u> reinforcement ~~required to resist in-plane shear~~ shall be the smaller of one-third the length of the shear wall and one-third the height of the shear wall. Horizontal <u>shear</u> reinforcement ~~required to resist in-plane shear~~ shall be uniformly distributed.

(e) Joint reinforcement and deformed wire placed in mortar <u>used as shear reinforcement</u>~~required to resist in-plane shear~~ shall be a single piece without splices for the length of the wall used for shear design, $d_v$.

(f) The sum of the horizontal reinforcement ratio and vertical reinforcement ratio shall be at least 0.002. Reinforcement ratios shall be based on the gross cross-sectional area of the wall, using specified dimensions and shall be not less than the following:

1. For masonry laid in running bond, the minimum reinforcement ratio in each direction shall be at least 0.0007.

2. For masonry not laid in running bond, the minimum vertical reinforcement ratio shall be at least 0.0007. The minimum horizontal reinforcement ratio shall be at least 0.0015.

Reinforcement used for compliance with these provisions shall be uniformly distributed.

(g) Joint reinforcement used as shear reinforcement shall be anchored in accordance with Section 6.1.8.1.3.1 (a) or (c) when two longitudinal wires are used and Section 6.1.8.1.3.2 when four longitudinal wires are used.

(h) Mechanical splices in flexural reinforcement in plastic hinge zones shall meet the requirements of Section 6.1.7.2.1 and develop the specified tensile strength of the spliced bar.

(i) Masonry not laid in running bond shall be fully grouted and shall be constructed of hollow open-end units or two wythes of solid units.

(j) Welded splices in reinforcement shall not be permitted in plastic hinge zones.

**Code Commentary:**

**Note: The text below reflects changes made to this section through ballots 19-SL-002, 19-SL-005 and 19-SL-007.**

**7.3.2.5** *Special reinforced masonry shear walls* — These shear walls are designed as reinforced masonry as noted in the referenced sections and are also required to meet restrictive reinforcement and material requirements. Accordingly, they are permitted to be used as part of the seismic-force-resisting system in any Seismic Design Category. Additionally, these walls have the most favorable seismic design parameters, including the highest response modification factor, R, of any of the masonry shear wall types.

(a) The reinforcing wire products – joint reinforcing, deformed wire and welded wire reinforcement – are cold worked and lack the ductility required for flexural reinforcement in special reinforced masonry shear walls.

Subsections (c), (d), and (f) stipulate a minimum level of in-plane shear reinforcement to improve ductility.

(e) At this time, splicing of joint reinforcing and deformed wire placed in mortar is not permitted as research has not been done on the performance of lap splices of reinforcement placed in mortar under cyclic loads, and in mortar joints that may be cracked due to in-plane or out-of-planeloads. Where a wall is divided into two or more segments by movement joints, each segment will have its own length, $d_v$, and the joint reinforcing or deformed wires can be terminated in accordance with (g) on either side of the joint.

Joint reinforcing is also subject to the minimum reinforcement requirements based on Seismic Design Category, see Sections 7.4.1.2.1 and 7.4.3.2.6.

(f) ~~In previous editions of the Code~~<u>Prior to the 2022 edition of this Code</u>, this section included a requirement for a minimum amount of vertical reinforcement based on the amount of horizontal <u>shear </u>reinforcement ~~required to resist shear~~. This requirement for a minimum amount of vertical reinforcement was redundant with provisions applicable to all reinforced masonry shear wall designs in Chapters 8 and 9 and has been removed from this section.

The minimum amount of wall reinforcement for special reinforced masonry shear walls has been a long-standing, standard empirical requirement in areas of high seismic loading. It is expressed as a percentage of gross cross-sectional area of the wall. It is intended to improve the ductile behavior of the wall under earthquake loading and assist in crack control.

(g) Option (b) in Section 6.1.8.1.3.1 is excluded from use in special reinforced masonry shear walls due to lack of testing. Section 6.1.8.1 also addresses the anchorage of reinforcing bars and deformed wires used as shear reinforcement in walls.

(h) In a structure undergoing inelastic deformations during an earthquake, the tensile stresses in flexural reinforcement in plastic hinge zones may approach the tensile strength of the reinforcement. This requirement is intended to avoid a splice failure in such reinforcement.

In a perforated or coupled shear wall, plastic hinge zones may form at locations other than at the base of the wall, such as at the interfaces between horizontal and vertical wall segments. Mechanical splices in these regions are required to develop the specified tensile strength of the bar.

For the purpose of this section, the plastic hinge zone may be assumed to extend at least half of the member depth from the plane where yielding is expected to initiate.

(j) Welding can adversely affect the ductility of the reinforcement, and is thus prohibited in plastic hinge zones. See commentary for item (h) for additional discussion of plastic hinge zones.

**Specification:**

None

**Specification Commentary:**

None

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 9 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 1 *Abstain* | 10 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| Item #: 20-SL-006 | |
| Technical Contact/Email: John M. Hochwalt / john.hochwalt@kpff.com | |
| Public Comment Number: 2022 Comments # 94 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 6/1/2021 |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 94 read as follows:

> The last sentence in 7.4.4.2.1 is redundant with the first sentence of 5.3.1.4 (d). Can it be deleted?

**Response/Rationale:**

The provision referenced by the comment reads follows:

> **5.3.1** *General column design*
> . . .
> **5.3.1.4** *Lateral ties* — Lateral ties shall conform to the following:
> . . .
> (d) Lateral ties shall be embedded in grout. When a lateral tie or combination of ties does not exceed the specified thickness of the mortar joint, the portion of the tie(s) that crosses a web or interior face shell shall be permitted to be embedded in mortar.

The requirement in 7.4.4.2.1 that column ties be embedded in grout is redundant and is proposed for deletion.

With deletion of the code provision, it is proposed to add commentary in 7.4.4.2.1 to remind the user of the commentary in Section 5.3.1.4 which may be especially relevant given the minimum 3/8" tie diameter required by 7.4.4.2.1. The commentary addresses the potential need to modify the units:

> When a lateral tie or combination of ties exceeds the specified mortar joint thickness, removal of part of the unexposed portion of the unit(s) or other modification is required to maintain proper clearance and grout coverage.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**7.4.4** *Seismic Design Category D requirements* — Masonry elements in structures assigned to Seismic Design Category D shall comply with the requirements of Section 7.4.3 and with the additional requirements of Sections 7.4.4.1 and 7.4.4.2.

. . .

**7.4.4.2** *Design of participating elements* — Masonry shear walls shall be designed to comply with the requirements of Section 7.3.2.5, 7.3.2.8, or 7.3.2.11.

**7.4.4.2.1** *Minimum reinforcement for masonry columns* — Lateral ties in masonry columns shall be spaced not more than 8 in. (203 mm) on center and shall be at least 3/8 in. (9.5 mm) diameter. ~~Lateral ties shall be embedded in grout.~~

**Code Commentary:**

**7.4.4.2.1** *Minimum reinforcement for masonry columns* — Adequate lateral restraint is important for column reinforcement subjected to overturning forces due to earthquakes. Many column failures during earthquakes have been attributed to inadequate lateral tying. For this reason, closer spacing of lateral ties than might otherwise be required is prudent. An arbitrary minimum spacing has been established through experience. Columns not involved in the seismic-force-resisting system should also be more heavily tied at the tops and bottoms for more ductile performance and better resistance to shear.

<u>The larger minimum tie diameter required by this provision makes it more likely that units may need to be modified to accommodate the ties as is discussed in the commentary to Section 5.3.1.4.</u>

**Specification:**

None

None

**Specification Commentary:**

None

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 16 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 4 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| Item #: 20-SL-009 | |
| Technical Contact/Email: John M. Hochwalt / john.hochwalt@kpff.com | |
| Public Comment Number: 2022 Comments # 114 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 6/1/2021 |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 114 read as follows:

> The notation and nomenclature used in TMS 402 to discuss lateral building movements is inconsistent and should be revised for clarity.
>
> The following nomenclature is used for story drifts:
> - Calculated story drift, $\Delta$. This notation is defined in Section 2.1. From Section 7.2.4 it can be inferred that that this is intended to include inelastic seismic displacements.
> - Design story drift, which includes inelastic displacements and is a defined term in Section 2.2.
>
> The notation $\Delta$ is not necessary as it is not used in any formulas; it is suggested to only use the term "design story drift." Alternatively, the notation $\underline{C_d}\Delta$ could be used in conjunction with "design story drift," to make the inclusion of inelastic effect more transparent and the notation more consistent with that used for system drifts.
>
> System (top of wall) drifts are defined using the notation $C_d\delta_{ne}$ where $\delta_{ne}$ is defined in Section 2.1 as "displacements calculated using code-prescribed seismic forces and assuming elastic behavior." While it can be inferred that this is measured at the top of wall, consider making that part of the definition.
>
> Some minor other suggestions related to drifts include:
> - Delete the reference to the "equivalent lateral force method" in the definition of design story drift in Section 2.1. This is applicable to all elastic analyses.
> - Delete the reference to "flexible frame systems" in the commentary to section 4.1.4 as the behavior described is not limited to flexible frame systems.
> - Reference the ASCE 7 provisions for building separations in the discussion of building separations in the commentary to Section 7.2.4.

**Response/Rationale:**

**This was previously passed by the subcommittee by a vote of 14 affirmative, 2 affirmative with comment, and no negatives or abstentions. This ballot is intended to address the comments from Shing and Lepage prior to submitting the ballot to main:**

- **The addition of "seismic" in front of "base" in the commentary has been deleted. Shing was correct that ASCE 7 only uses the term "base."**
- **A typo identified by Shing and Lepage has been corrected.**
- **A sentence in the commentary that attempted to put the revised displacements in context with the TMS 402-16 displacements has been deleted as suggested by Shing. There is not a simple correlation between the two and the sentence is not necessary to clarify the Code.**
- **Commentary about building separations was revised to delete the word "typically" as suggested by Lepage.**

TMS 402 relies on the building code / ASCE 7 to determine the movements that masonry structures will experience as a result of seismic events, and the movement limits that masonry structures must meet. As a result, addressing the public comment should favor nomenclature and definitions that are consistent with ASCE 7. For the voter's reference, the relevant provisions from the Public Comment version ASCE 7-22 are included at the end of this rationale.

Before discussing revisions to specific sections of code and commentary, three general observations should be made:

- In general, TMS 402 and ASCE 7 are consistent in using upper case $\Delta$ in reference to the displacements of one story relative to another and lower case $\delta$ to reference displacements relative to the base.
- Inelastic displacements. TMS 402 typically describes the design displacements as being the displacements from an elastic analysis multiplied by $C_d$. There are two problems with this.
  1. The accepted means of analysis are not limited elastic methods.
  2. The determination of inelastic displacements from an elastic analysis is more complicated than is currently presented in TMS 402. For example, when using the Equivalent Lateral Force procedure, the design displacement is determined using ASCE 7 equation 12.8-16:

     $$\delta_{DE} = \frac{C_d \delta_e}{I_e} + \delta_{di}$$

     The determination the design deflections is even more complicated when using other analysis procedures such as Modal Response Spectrum Analysis or Linear Response History Analysis.

     For this reason, it is proposed that TMS 402 no longer directly reference $C_d$ in the provisions.

     Note that references to $C_d$ in the commentary to Sections 7.3.2, 7.3.2.9, 7.3.2.10, 7.3.2.11, 9.3.5.6.1, and 12.1.1 will remain as they refer to the actual value of $C_d$, not a displacement calculated using $C_d$.

- Upper bound displacements. For structural components that whose capacity is deformation limited, it is important that the displacement capacity be checked again against an upper bound displacement. The two deformation limited components currently recognized by TMS 402 are cantilever shear walls designed using displacement based design (Section 9.3.5.6.2.3) and hinging components of limit mechanisms (Appendix C). A step towards using upper bound displacements was taken earlier this cycle when these provisions were revised to use $1.5C_d\,\delta_{ne}$ in lieu of $C_d\,\delta_{ne}$ as was used in TMS 402-16. This ballot proposes to take one additional step as is now required by ASCE 7, which is to use multiply the elastic displacement by $R$ rather than $C_d$ to provide a more reliable upper bound displacement for deformation controlled elements. The rationale is explained by the commentary to ASCE 7 Section 12.8.6:

> *Multiplying by R corrects for the fact that values of Cd less than R may substantially underestimate displacements for many seismic force resisting systems (Uang and Maarouf 1994). The degree of such underestimation and its variation among the various types of seismic-force-resisting systems is not known and R is substituted for Cd in the provision pending more detailed information.*

The specific proposed revisions and corresponding rationale is as follows:

Revisions to Chapter 2

- Section 2.1 Notation
    - The notation $C_d$ and $\delta_{ne}$ will no longer be used and are proposed to be deleted. Note that while $C_d$ will remain in the commentary, Section 2.1 only defines notation used in the provisions and not that used in the commentary.
    - The notation $\delta_{MCE}$ will be added with a definition matching that in ASCE 7.
    - The description of $\Delta$ currently refers to this as a "calculated" displacement; this is inconsistent with both ASCE 7 and the definition of *design story drift.* It is proposed to resolve these discrepancies by replacing "calculated" with "design."
    - The public commenter proposed deleting the notation $\Delta$*.* The committee felt it would be helpful to retain it for alignment with ASCE 7, and that it was needed for the deformation compatibility provision in Chapter 7.
- Section 2.2 Definitions
    - It is proposed to delete the second sentence of the definition of *design story drift.* The first sentence conveys the key ideas that this is an inelastic displacement and that the user needs to consult with ASCE 7 for the calculation of building displacements.

Revisions to Chapter 4:

- No changes are proposed to the code. Section 4.1.4 is provided in the ballot for the voter's reference.
- As suggested by the commenter, it is proposed to delete the reference to "flexible frame structures" in the discussion of forces induced by building displacements in the commentary for Section 4.1.4, as the referenced behavior occurs regardless of structure type.

Revisions to Chapter 7:

- Section 7.2.4 *Drift limits*:
    - In the code, it is proposed to change two references to "calculated story drift" to "design story drift" for consistency with definition of *design story drift* and ASCE 7.
    - In the commentary several changes are proposed.
        - It is proposed to underscore in the commentary that notation using upper case $\Delta$ indicates relative story displacements, and notation using lower case $\delta$ indicates displacements relative to the base.
        - Much of the commentary about building separations is proposed to be deleted and replaced with a reference to ASCE 7. This resolves inconsistencies between the discussion of building separations and ASCE 7.
        - NOTE: Several public comments were received addressing the last sentence of this commentary. A separate ballot has been prepared to address those comments.
- Section 7.3.1 *Nonparticipating elements*
    - In the code, the following changes are proposed:
        - Currently the provisions reference $C_d\delta_{ne}$ as the displacement that non-isolated, non-participating elements must accommodate. As discussed above, this notation

oversimplifies the calculation of inelastic displacements. Further, this is referencing a displacement relative to the base, whereas it is the relative story displacements that are critical for the performance on non-structural elements. As a result, it is proposed to replace $C_d\delta_{ne}$ with $\Delta$.
        - The load combinations to be used for deformation compatibility checks was based on ASCE 7 load combinations 6 and 7. It is proposed to change the load factor for snow load from 0.2 to 0.15 for consistency with the public comment version of ASCE 7-22.
    - No changes are proposed to the commentary; it is provided in the ballot for the voter's reference. NOTE: Several public comments about typos in this commentary, the correction of which is addressed by 19-FS-001. So as not to distract the voter, the typographical errors have been corrected in this ballot.

Revisions to Chapter 9:

- Section 9.3.5.6.2.3, in the section on alternate approaches to ductility:
    - The following changes are proposed in the code:
        - In the equation, it proposed to replace the term $1.5C_d\delta_{ne}$ with $\delta_{MCE}$ for the reasons discussed above.
        - It is proposed to correct the load factor on snow load in the load combination to reflect the revised load factor from the public comment version of ASCE 7-22.
    - The commentary is also revised to replace the term $1.5C_d\delta_{ne}$ with $\delta_{MCE}$, including the discussion of the rationale behind using $MCE_R$ level displacements and multiplying by $R$ instead of $C_d$.
- Section 9.3.5.6.2.5, in the section on alternate approaches to ductility:
    - No changes are proposed to the code. The code provisions are provided in the ballot for the voter's reference.
    - The following changes are proposed to the commentary:
        - In the equations, it proposed to replace the term $1.5C_d\delta_{ne}$ with $\delta_{MCE}$ for the reasons discussed above.
        - A phrase is added to clarify the origin of the assumed maximum $MCE_R$ drift ratio which may not be as clear with the new notation.

Revisions to Appendix C: The code and commentary are proposed to be revised to replace $1.5C_d\delta_{ne}$ with $\delta_{MCE}$ for the reasons discussed above

<u>Excerpts from ASCE 7-22, Public Comment version</u>

**11.2 DEFINITIONS**

. . .

**DISPLACEMENT AND DRIFT**

**Design Earthquake Displacement:** The displacement at a given location of the structure corresponding to the Design Earthquake.
**Design Story Drift:** The story drift corresponding to the Design Earthquake, taken at a representative plan location (center of mass or building perimeter, as required by Section 12.8.6).
**Maximum Considered Earthquake Displacement:** The displacement at a given location of the structure corresponding to the Risk-Targeted Maximum Considered Earthquake ($MCE_R$).
**Story Drift:** The horizontal displacement at the top of the story relative to the bottom of the story at vertically aligned points corresponding to the given loading.
**Story Drift Ratio:** The story drift divided by the story height, $h_{sx}$.

. . .

**11.3 SYMBOLS**

$C_d$ = Deflection amplification factor as given in Tables 12.2-1, 15.4-1, or 15.4-2

$\Delta$ = Design story drift as determined in Section 12.8.6
$\Delta_a$ = Allowable story drift as specified in Section 12.12.1
$\delta_{DE}$ = Design earthquake displacement as determined in Section 12.8.6
$\delta_e$ = Elastic displacement computed under design earthquake forces (Section 12.8.6)
$\delta_{MCE}$ = Maximum Considered Earthquake Displacement as determined in Section 12.8.6
. . .

**12.8 Equivalent Lateral Force (ELF) Procedure**
. . .

**12.8.6 Displacement and Drift Determination**
Displacements and drifts shall be determined as required by this section.

**12.8.6.1 Minimum Base Shear and Load Combination for Computing Displacement and Drift**
. . .
**12.8.6.2 Period for Computing Displacement and Drift**
. . .
**12.8.6.3 Design Earthquake Displacement**
The Design Earthquake Displacement, $\delta_{DE}$, shall be determined at the location of an element or component using Equation (12.8-16) or as permitted in Chapter 16, 17, or 18.

$$\delta_{DE} = \frac{C_d \delta_e}{I_e} + \delta_{di} \qquad (12.8\text{-}16)$$
where
$C_d$ = Deflection amplification factor in Table 12.2-1.
$I_e$ = Importance Factor determined in accordance with Section 11.5.1;
$\delta_e$ = Elastic displacement computed under design earthquake forces, including the effects of accidental torsion and torsional amplification as applicable; and
$\delta_{di}$ = Displacement due to diaphragm deformation corresponding to the design earthquake, including Section 12.10 diaphragm forces.

**12.8.6.4 Maximum Considered Earthquake Displacement**
The Maximum Considered Earthquake Displacement, $\delta_{MCE}$ , shall be determined at the location under consideration using Equation (12.8-17) or as permitted in Chapter 16, Chapter 17, or Chapter 18.

$$\delta_{MCE} = 1.5\left[\frac{R\delta_e}{I_e} + \delta_{di}\right] \qquad (12.8\text{-}17)$$

where $R$ is the response modification coefficient in Table 12.2-1.

**12.8.6.5 Design Story Drift Determination**
The Design Story Drift, $\Delta$, shall be computed as the difference of the Design Earthquake Displacements, $\delta_{DE}$, as determined in accordance with Section 12.8.6, at the centers of mass at the top and bottom of the story under consideration. Where centers of mass do not align vertically, it is permitted to compute the deflection at the bottom of the story based on the vertical projection of the center of mass at the top of the story.

Diaphragm deformation may be neglected in determining the design story drift. Diaphragm rotation shall be considered as follows. For structures assigned to Seismic Design Category C, D, E, or F that have horizontal irregularity Type 1 of Table 12.3-1, the design story drift, $\Delta$, shall be computed as the largest difference of the Design Earthquake Displacements of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure, including the effects of diaphragm rotation.

. . .

**12.9 Linear Dynamic Analysis**

**12.9.1 Modal Response Spectrum Analysis**

**12.9.1.1 Number of Modes**

. . .

**12.9.1.2 Modal Response Parameters**

The value for each force-related design parameter of interest, including story drifts, support forces, and individual member forces for each mode of response, shall be computed using the properties of each mode and the response spectra defined in either Section 11.4.6 or 21.2 21.3 divided by the quantity $R/I_e$. The value for displacement and drift quantities shall be multiplied by the quantity $C_d/I_e$.

. . .

**12.9.2 Linear Response History Analysis**

**12.9.2.1 General Requirements**

. . .

**12.9.2.2 General Modeling Requirements**

. . .

**12.9.2.3 Ground Motion Selection and Modification**

. . .

**12.9.2.4 Application of Ground Acceleration Histories**

. . .

**12.9.2.5 Modification of Response for Design**

*12.9.2.5.1 Determination of Maximum Elastic and Inelastic Base Shear*

. . .

*12.9.2.5.2 Determination of Base Shear Scale Factor*

. . .

*12.9.2.5.3 Determination of Combined Force Response.*

. . .

*12.9.2.5.4 Determination of Combined Displacement Response.* Response modification factors $C_{dX}$ and $C_{dY}$ shall be assigned in the X and Y directions, respectively. For each direction of response and for each ground motion analyzed, the combined displacement responses shall be determined as follows:

(a) The combined displacement response in the *X* direction shall be determined as $\eta_X C_{dX}/R_X$ times the computed elastic response in the X direction using the mathematical model with accidental torsion (where required), plus $\eta_Y C_{dY}/R_Y$ times the computed elastic response in the *Y* direction using the mathematical model without accidental torsion.

(b) The combined displacement response in the Y direction shall be determined as $\eta_Y C_{dY}/R_Y$ times the computed elastic response in the Y direction using the mathematical model with accidental torsion (where required), plus $\eta_X C_{dX}/R_X$ times the computed elastic response in the X direction using the mathematical model without accidental torsion.

**EXCEPTION:** Where the design base shear in the given direction is not controlled by Equation (12.8-7), the factors $\eta_X$ or $\eta_Y$ , as applicable, are permitted to be taken as 1.0 for the purpose of determining combined displacements.

**12.9.2.6 Enveloping of Force Response Quantities**

. . .

**12.9.2.7 Enveloping of Displacement Response Quantities**

Story drift quantities shall be determined for each ground motion analyzed and in each direction of response using the combined displacement responses defined in Section 12.9.2.5.4. For the purpose of complying with the drift limits specified in Section 12.12, the envelope of story drifts computed in both orthogonal directions and for all ground motions analyzed shall be used.

. . .

**12.12 Drift and Deformation**

**12.12.1 Story Drift Limit**

The design story drift, $\Delta$, as determined in Sections 12.8.6, 12.9.1, or 12.9.2 shall not exceed the allowable story drift, $\Delta_a$, as obtained from Table 12.12-1 for any story.

**Table 12.12-1. Allowable Story Drift, $\Delta_a$.**

| Structure | Risk Category | | |
|---|---|---|---|
| | I or II | III | IV |
| Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, and ceilings that have been designed to accommodate the drifts associated with the Design Earthquake Displacements | $0.025h_{sx}$[a] | $0.020h_{sx}$ | $0.015h_{sx}$ |
| Masonry cantilever shear wall structures[b] | $0.010h_{sx}$ | $0.010h_{sx}$ | $0.010h_{sx}$ |
| Other masonry shear wall structures | $0.007h_{sx}$ | $0.007h_{sx}$ | $0.007h_{sx}$ |
| All other structures | $0.020h_{sx}$ | $0.015h_{sx}$ | $0.010h_{sx}$ |

Notes: $h_{sx}$ is the story height below level x . For seismic force-resisting systems solely comprising moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.

[a]There shall be no drift limit for single-story structures in which the interior walls, partitions, and ceilings have been designed to accommodate story drifts associated with the Design Earthquake Displacement. The structural separation requirement of Section 12.12.3 is not waived.

[b]Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are so constructed that moment transfer between shear walls (coupling) is negligible.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)* *Do not use 'Track Changes'*

**Code:**

**2.1 — Notation**

~~$C_d$ = deflection amplification factor~~

$\Delta$ = ~~calculated~~ <u>design</u> story drift, in. (mm)

$\Delta_a$ = allowable story drift, in. (mm)

~~$\delta_{ne}$ = displacements calculated using code-prescribed seismic forces and assuming elastic behavior, in. (mm)~~

<u>$\delta_{MCE}$ = displacement due to Maximum Considered Earthquake as defined in ASCE/SEI 7, in. (mm)</u>

**2.2 — Definitions**

*Design story drift* — The difference of deflections at the top and bottom of the story under consideration, taking into account the possibility of inelastic deformations as defined in ASCE/SEI 7. ~~In the equivalent lateral force method, the story drift is calculated by multiplying the deflections determined from an elastic analysis by the appropriate deflection amplification factor, $C_d$, from ASCE/SEI 7.~~

. . .

**4.1.4** *Load transfer at horizontal connections*

**4.1.4.1** Walls, columns, and pilasters shall be designed to resist loads, moments, and shears applied at intersections with horizontal members.

**4.1.4.2** Effect of lateral deflection and translation of members providing lateral support shall be considered.

**4.1.4.3** Devices used for transferring lateral support from members that intersect walls, columns, or pilasters shall be designed to resist the forces involved.

. . .

**7.2.4** *Drift limits* — Under loading combinations that include earthquake, masonry structures shall be designed so the ~~calculated~~ <u>design</u> story drift, $\Delta$, does not exceed the allowable story drift, $\Delta_a$, obtained from the legally adopted building code. When the legally adopted building code does not prescribe allowable story drifts, structures shall be designed so the ~~calculated~~ <u>design</u> story drift, $\Delta$, does not exceed the allowable story drift, $\Delta_a$, obtained from ASCE/SEI 7.

. . .

**7.3.1** *Nonparticipating elements* — Masonry elements that are not part of the seismic-force-resisting system shall be classified as nonparticipating elements and shall be isolated in their own plane from the seismic-force-resisting system. Isolation joints and connectors shall be designed to accommodate the design story drift.

Exception: Isolation is not required if a deformation compatibility analysis demonstrates that the non-participating element can accommodate the inelastic <u>story drift</u> ~~displacement~~, ~~$C_d\delta_{ne}$~~ $\underline{\Delta}$, of the structure in a manner complying with the requirements of this code. Elements supporting gravity loads in addition their self-weight shall be evaluated for gravity load combinations of (1.2D + 1.0L + ~~0.2~~<u>0.15</u>S) or 0.9D, whichever is critical, acting simultaneously with the inelastic displacement and shall have a ductility compatible with the ductility of the lateral force resisting system.

. . .

**9.3.5.6.2.3** This Section applies to walls bending in single curvature in which the flexural limit state response is governed by yielding at the base of the wall. Walls not satisfying those requirements shall be designed in accordance with Section 9.3.5.6.2.4.

(a) Special boundary elements shall be provided over portions of compression zones where:

$$c \geq \frac{l_w}{600(\cancel{1.5\,C_d\delta_{ne}}\,\underline{\delta_{MCE}}/h_w)}$$

and c is calculated for the $P_u$ given by ASCE 7 Strength Design Load Combination 6 (1.2D + $E_v$ + $E_h$ + L + ~~0.2~~<u>0.15</u>S) or the corresponding strength design load combination of the legally adopted building code, and the corresponding nominal moment strength, $M_n$ , at the base critical section. The load factor on L in Combination 6 is reducible to 0.5, as per exceptions to Section 2.3.6 of ASCE 7.

. . .

**9.3.5.6.2.5** Where special boundary elements are required by Section 9.3.5.6.2.3 or 9.3.5.6.2.4, requirements (a) through (d) in this section shall be satisfied and tests shall be performed to verify the strain capacity of the element:

(a) The special boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of (c - 0.1lw) and c/2.

. . .

**C.3** *Mechanism deformation* — The rotational deformation demand on plastic hinges shall be determined by imposing the ~~design~~ displacement, ~~$1.5C_d\delta_{ne}$~~ $\underline{\delta_{MCE}}$, at the roof level of the yield mechanism. The rotational deformation capacity of plastic hinges shall satisfy C.3.1 to C.3.3.

**Code Commentary:**

**4.1.4** *Load transfer at horizontal connections*

Masonry walls, pilasters, and columns may be connected to horizontal members of the structure and may rely on the latter for lateral support and stability. The mechanism through which the interconnecting forces are transmitted may involve bond, mechanical anchorage, friction, bearing, or a combination thereof. The designer must assure that, regardless of the type of connection, the interacting forces are safely resisted.

~~In flexible frame construction, t~~The relative movement (drift) between floors may generate forces within the members and the connections. This Code requires the effects of these movements to be considered in design.

. . .

**7.2.4** *Drift limits* — Excessive deformation, particularly resulting from inelastic displacements, can potentially result in instability of the seismic-force-resisting system. This section provides procedures for the limitation of story drift. The term "drift" has two connotations:

1. "Story drift" is the maximum calculated lateral displacement within a story (the calculated displacement of one level relative to the level below caused by the effects of design seismic loads). In the Code, notation using an upper case delta ($\Delta$) is used to indicate relative story displacements.
2. The calculated lateral displacement or deflection due to design seismic loads is the absolute displacement of any point in the structure relative to the base. This is not "story drift" and is not to be used for drift control or stability considerations because it may give a false impression of the effects in critical stories. However, it is important when considering seismic separation requirements and is used in determining rotation demands on cantilevered walls and limit mechanisms. In the Code, notation using a lower case delta ($\delta$) is used to indicate displacements relative to the base.

Overall or total drift is the lateral displacement of the top of a building relative to the base. The overall drift ratio is the total drift divided by the building height. Story drift is the lateral displacement of one story relative to an adjacent story. The story drift ratio is the story drift divided by the corresponding story height. The overall drift ratio is usually an indication of moments in a structure and is also related to seismic separation demands. The story drift ratio is an indication of local seismic deformation, which relates to seismic separation demands within a story. The maximum story drift ratio could exceed the overall drift ratio.

There are many reasons for controlling drift in seismic design:

(a) To control the inelastic strain within the affected elements. Although the relationship between lateral drift and maximum nonlinear strain is imprecise, so is the current state of knowledge of what strain limitations should be.

(b) Under small lateral deformations, secondary stresses are normally within tolerable limits. However, larger deformations with heavy vertical loads can lead to significant secondary moments from P-delta effects in the design. The drift limits indirectly provide upper bounds for these effects.

(c) Buildings subjected to earthquakes need drift control to restrict damage to partitions, shaft and stair enclosures, glass, and other fragile nonstructural elements and, more importantly, to minimize differential movement demands on the seismic-force-resisting elements.

The designer must keep in mind that the allowable drift limits, $\Delta_a$, correspond to story drifts and, therefore, are applicable to each story. They must not be exceeded in any story even though the drift in other stories may be well below the limit.

Although the provisions of this Code do not give equations for calculating building separations, ASCE/SEI 7 can be used to determine the distance ~~should be~~ that would be sufficient to avoid damaging contact should such requirements be absent from the legally adopted building code. ~~under total calculated deflection for the design loading in order to avoid interference and possible destructive hammering between buildings. The distance should be equal to the total of the lateral deflections of the two units assumed deflecting toward each other (this involves increasing the separation with height). If the effects of hammering can be shown not to be~~

~~detrimental, these distances may be reduced.~~ For very rigid shear wall structures with rigid diaphragms whose lateral deflections are difficult to estimate, older code requirements for structural separations of at least 1 in. (25.4 mm) plus ½ in. (12.7 mm) for each 10 ft (3.1 m) of height above 20 ft (6.1 m) could be used as a guide.

Ordinary plain, detailed plain, ordinary reinforced, intermediate reinforced, ordinary plain AAC, and detailed plain AAC masonry shear walls are inherently designed to have relatively low inelastic deformations under seismic loads. As such, the Committee felt that requiring designers to check story drifts for these systems of low and moderate ductility was not exceeded.

. . .

**7.3.1** *Nonparticipating elements* — With regards to the exception, non-isolated, nonparticipating elements can influence a structure's strength and stiffness, and as a result the distribution of lateral loads and building irregularities. The influence of any non-isolated nonparticipating elements can inadvertently have on the performance of a structural system should be considered in design in accordance with Section 4.1.6 of this code, and other applicable provisions such as the modelling criteria of ASCE/SEI 7. Where partial height non-participating elements are constructed tight to building columns, this should include the consideration of short column effects.

The deformation compatibility analysis may consider the effect of cracking on element stiffness. Elements that are detailed to achieve ductile behavior may also develop plastic mechanisms. For example, elements detailed in accordance with the provisions for special reinforced masonry shear walls may be able to accommodate displacements through the development of plastic hinges. For such elements, Appendix C may be used to provide guidance on the determination of hinge rotation capacity. In addition to these provisions, other applicable provisions, such as the deformation limit and deformation compatibility provisions of ASCE/SEI 7 should be considered in design.

When the lateral force resisting system consists of masonry shear walls, a nonparticipating element can achieve a ductility compatible with the ductility of the lateral force resisting system by detailing the nonparticipating element to the same minimum requirements as the shear walls. For lateral force resisting systems constructed of other materials, a nonparticipating element can achieve a ductility compatible with the ductility of the lateral force resisting system can be achieved by detailing the nonparticipating element in accordance with the requirements for a masonry shear wall with an R value not less than that of the lateral force resisting system. If the lateral force resisting system has an R value in excess of that achievable for a special reinforced masonry shear wall, the non-participating element will not qualify for the exception.

. . .

**9.3.5.6.2.3** Section 9.3.5.6.2.3 is based on the assumption that inelastic response of the wall is dominated by flexural action at a critical, yielding section – typically at the base. The wall should be proportioned so that the critical section occurs where intended (at the base).

(a) The following explanation, including Figure CC-9.3-2, is adapted from a paper by Wallace and Orakcal (2002). The relationship between the wall top displacement and wall curvature for a wall of uniform cross-section with a single critical section at the base is presented in Figure CC-9.3-2. The provisions of this Code are based on a simplified version of the model presented in Figure CC-9.3-2(a). The simplified model, shown in Figure CC-9.3-2(b), neglects the contribution of elastic deformations to the top displacement, and moves the center of the plastic hinge to the base of the wall. Based on the model of Figure CC-9.3-2, the relationship between the top displacement and the curvature at the base of the wall is:

$$\cancel{1.5C_d\delta_{ne}}\underline{\delta_{MCE}} = \theta_p h_w = (\phi_u l_p) h_w = \left(\phi_u \frac{l_w}{2}\right) h_w \quad \text{(Equation 1)}$$

assuming that $l_p = l_w/2$ , as is permitted to be assumed by the 1997 UBC,

where $\phi_u$= ultimate curvature, and
$\theta_p$ = plastic rotation at the base of the wall.

~~The 1.5 factor in front of the term~~ $\cancel{C_d\delta_{ne}}$ ~~amplifies the displacement so that it corresponds to the displacement that would be expected for the Risk-Targeted Maximum Considered Earthquake (MCE~~$\cancel{_R}$~~) event. This is done~~ The displacement associated with the Risk-Targeted Maximum Considered Earthquake (MCE$_R$) is used to align the detailing requirements with the intent of the building code to have a low probability of collapse in the MCE$_R$ event.

If at the stage where the top deflection of the wall is $\delta_{\cancel{ne}MCE}$ , the extreme fiber compressive strain at the critical section at the base does not exceed $\varepsilon_{mu}$ , no special confinement would be required anywhere in the wall. Figure CC-9.3-3 illustrates such a strain distribution at the critical section. The neutral axis depth corresponding to this strain distribution is $c_{cr}$, and the corresponding ultimate curvature is $\phi_u = \varepsilon_{mu}/c_{cr}$.
From Equation 1,

$\cancel{1.5C_d\delta_{ne}}\underline{\delta_{MCE}} = \left(\frac{\varepsilon_{mu}}{c_{cr}}\frac{l_w}{2}\right)h_w$ (Equation 2a)

or, $c_{cr} = \frac{\varepsilon_{mu}}{2}\frac{l_w}{(\cancel{1.5C_d\delta_{ne}}\underline{\delta_{MCE}}/h_w)}$ (Equation 2b)

From the equations above (see Figure CC-9.3-3), special detailing would be required if:

$$c \geq \frac{\varepsilon_{mu}}{2}\frac{l_w}{(\cancel{1.5\,C_d\delta_{ne}}\underline{\delta_{MCE}}/h_w)} = \frac{0.003}{2}\frac{l_w}{(\cancel{1.5C_d\delta_{ne}}\underline{\delta_{MCE}}/h_w)}$$
$$= \frac{l_w}{667(\cancel{1.5\,C_d\delta_{ne}}\underline{\delta_{MCE}}/h_w)} \approx \frac{l_w}{600(\cancel{1.5C_d\delta_{ne}}\underline{\delta_{MCE}}/h_w)}$$

because if the neutral axis depth exceeded the critical value, the extreme fiber compressive strain would exceed the maximum usable strain $\varepsilon_{mu}$ . For purposes of this derivation, and to avoid having separate sets of drift-related requirements for clay and concrete masonry, a single useful strain of 0.003 is used, representing an average of the design values of 0.0025 for concrete masonry and 0.0035 for clay masonry.

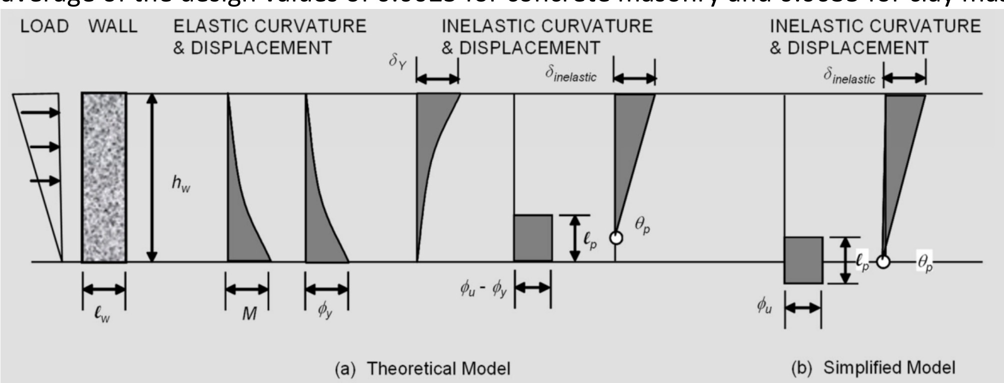


*Figure CC-9.3-2 — Wall curvature and displacement*

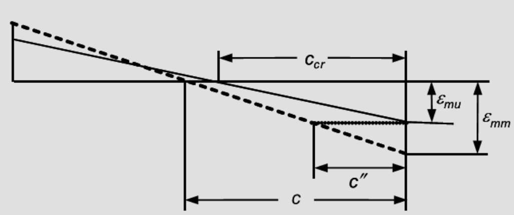


*Figure CC-9.3-3 —Strain distribution at critical section*

. . .

**9.3.5.6.2.5** This Code requires that testing be done to verify that the detailing provided is capable of developing a strain capacity in the boundary element that would be in excess of the maximum imposed strain. Reasonably extensive tests need to be conducted to develop prescriptive detailing requirements for specially confined boundary elements of intermediate as well as special reinforced masonry shear walls.

(a) Figure CC-9.3-3 shows that when the neutral axis depth $c$ exceeds the critical neutral axis depth $c_{cr}$, the extreme compression fiber strain in the masonry reaches a value $\varepsilon_{mm}$ in excess of the maximum usable strain

$\varepsilon_{mu}$. The corresponding ultimate curvature $\phi$ is $\varepsilon_{mu} / c$. Based on the model of Figure CC-9.3-2(b) with $l_p = l_w/2$ and assuming the wall experiences the Risk-Targeted Maximum Considered Earthquake (MCE$_R$) event.

$\cancel{1.5C_d\delta_{ne}}\underline{\delta_{MCE}} = \theta_p h_w = (\phi_p l_p)h_w = \left(\frac{\varepsilon_{mm}}{c}\frac{l_w}{2}\right) h_w$ (Equation 3)

From Equation 3:

$\varepsilon_{mm} = 2\left(\frac{\cancel{1.5C_d\delta_{ne}}\underline{\delta_{MCE}}}{h_w}\right)\left(\frac{c}{l_w}\right)$ (Equation 4)

The wall length over which the strains exceed the limiting value of $\varepsilon_{mu}$, denoted as c'', can be determined using similar triangles from Figure CC-9.3-3:

$c'' = c\left(1 - \frac{\varepsilon_{mu}}{\varepsilon_{mm}}\right)$ (Equation 5)

An expression for the required length of confinement can be developed by combining Equations 4~~2~~ and 5~~3~~:

$\frac{c''}{l_w} = \frac{c}{l_w} - \frac{(\varepsilon_{mu}/2)}{(1.5C_d\delta_{ne}/h_w)}$ (Equation 6)

The term $c/l_w$ in Equation 6 accounts for the influence of material properties ($f'_m$, $f_y$), axial load, geometry, and quantities and distribution of reinforcement, whereas the term $(\varepsilon_{mu}/2)/(\cancel{1.5C_d\delta_{ne}}\underline{\delta_{MCE}}/h_w)$ accounts for the influence of system response (roof displacement) and the maximum usable strain of masonry.

The wall length over which special transverse reinforcement is to be provided is based on Equation 6, with a value of $(\cancel{1.5\,C_d\delta_{ne}}\underline{\delta_{MCE}}/h_w)$ ~~=1.5(0.01)~~ =.015:

$\frac{c''}{l_w} = \frac{c}{l_w} - \frac{(0.003/2)}{0.015} = \frac{c}{l_w} - 0.1 \geq \frac{c}{2}$ (Equation 7)

The value of $(\cancel{1.5\,C_d\delta_{ne}}\underline{\delta_{MCE}}/h_w) = 0.015$ was selected to provide an upper-bound estimate of the mean drift ratio of typical masonry shear wall buildings designed in accordance with ASCE, based on a maximum permitted drift of 0.01 in the design earthquake, amplified by a 1.5 factor for the MCE$_R$ event. Thus, the length of the wall that must be confined is conservative for many buildings. The value of c/2 represents a minimum length of confinement, is adopted from ACI 318-99, and is arbitrary.

. . .

**C.3** *Mechanism deformation* — This section defines the ductility checks required by the limit design method. The deformation demands at locations of plastic hinges are determined by imposing the calculated inelastic roof displacement to the controlling yield mechanism. ~~The 1.5 factor in front of the term $C_d\delta_{ne}$ amplifies the displacement so that it corresponds to the displacement that would be expected for the Risk-Targeted Maximum Considered Earthquake (MCER) event. This is done~~ The displacement $\delta_{MCE}$ is used for this check to align the deformation capacity checks with the intent of ASCE/SEI 7 to have a low probability of collapse in the MCE$_R$ event. Additional commentary on $\delta_{MCE}$ is provided in the commentary for Section 9.3.5.6.2.3.

**Specification:**

None

**Specification Commentary:**

None

| Subcommittee Vote: | | | | |
|---|---|---|---|---|
| 7 *Affirmative* | 2 *Affirmative w/ comment* | 0 *Negative* | 1 *Abstain* | 10 *Did not vote* |

**Subcommittee Comments:**

Shing commented:

> *In commentary 9.3.5.6.2.3, the sentence "The 1.5 factor is the ratio....." can be deleted because the factor is no longer used in the commentary or code section.*

Shing is correct and the referenced sentence has been deleted in this ballot.

Dutta commented:

> *delta-MCE is not defined in ASCE 7 as stated in page 7/13.*

The chair believes this comment is based on ASCE 7-16. The proposed provisions are based on the public comment version of ASCE 7-22. The definition of $\delta_{MCE}$ from ASCE 7-22 can be found on page 5 of 13 of this ballot.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-SL-013 | |
| **Technical Contact/Email:** John M. Hochwalt / John.Hochwalt@kpff.com | |
| **Public Comment Number:** 2022 Comment # 120 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Comment regarding Commentary Section 7.3.1

> The commentary language "The influence of any non-isolated nonparticipating elements can inadvertently have on performance of a structural system should be considered in design in accordance with Section 4.1.6 of this code, and other applicable provisions such as the modeling criteria of ASCE /SEI 7." is language that should be mandatory and placed in the code, not the commentary. The reference to ASCE 7 can be left in the commentary, but the first part should be placed in the code as "The influence of any non-isolated nonparticipating elements can inadvertently have on performance of a structural system shall be considered in design in accordance with Section 4.1.6 of this code."

**Response/Rationale:**

This comment was addressed by Ballot 19-SL-13.

This ballot is in response to an affirmative with comment vote on 19-SL-13 from Bennett who observed that the ballot used "lateral force resisting system" when "lateral-force-resisting system" is the more common, and preferred, usage.

Upon further review of Chapter 7, it was observed that "seismic-force-resisting system" is the most common description used.

In the section of code modified by 19-SL-13 there were two uses of "seismic-force-resisting system" and one use of "lateral force resisting system." Ballot 19-SL-13 added a second instance of "lateral force resisting system." It is proposed to change both instances of "lateral force resisting system" to "seismic-force-resisting system."

This ballot also adds a comma as suggested by Thompson.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)  Do not use 'Track Changes'*

**Code:**

**Code:**

**This text reflects the passage of 19-SL-013**

**7.3.1** *Nonparticipating elements* — Masonry elements that are not part of the seismic-force-resisting system shall be classified as nonparticipating elements and shall be isolated in their own plane from the seismic-force-resisting system. Isolation joints and connectors shall be designed to accommodate the design story drift.

Exception: Isolation is not required if a deformation compatibility analysis demonstrates that the non-participating element can accommodate the inelastic displacement, $C_d\delta_{ne}$, of the structure in a manner complying with the requirements of this code. Elements supporting gravity loads in addition their self-weight shall be evaluated for gravity load combinations of (1.2*D* + 1.0*L* + 0.2*S*) or 0.9*D*, whichever is critical, acting simultaneously with the inelastic displacement and shall have a ductility compatible with the ductility of the ~~lateral~~ <u>seismic-</u>force<u>-</u>resisting system.  The influence of any non-isolated<u>,</u> nonparticipating elements on the ~~lateral~~ <u>seismic-</u>force<u>-</u>resisting system shall be considered in design in accordance with Section 4.1.6 of this code.

**Code Commentary:  NONE**

**Specification:  NONE**

**Specification Commentary:  NONE**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 8 *Affirmative* | 1 *Affirmative w/ comment* | 0 *Negative* | 1 *Abstain* | 10 *Did not vote* |

**Subcommittee Comments:**

Dillon commented:

> *Should be a space, not a hyphen, between "seismic" and "force", i.e., should be "seismic force-resisting system." Compare to page 80 of ASCE/SEI 7-16.*

While there is merit in continuing to align the nomenclature and notation of TMS 402/602 with other building codes and standards, the subcommittee chair felt this would go beyond the original comment and has made no revision to the ballot based on this comment.

# 2022 TMS 402/602 Committee Response to Public Comment

| | |
|---|---|
| **Committee: Main Committee** | **Ballot #: 20** |
| **Item #: 20-SL-014** | |
| **Technical Contact/Email:** John M. Hochwalt / John.Hochwalt@kpff.com | |
| **Public Comment Number:** 2022 Comment # 163 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☒ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

ASCE 7-16 requires "6.1.6.1.1.4 Where $M/V_ud_v$ exceeds 1.5 and the seismic load associated with the development of the nominal shear capacity exceeds 80% of the seismic load associated with development of the nominal flexural capacity, lap splices shall not be used in plastic hinge zones of special reinforced masonry shear walls. The length of the plastic hinge zone shall be taken as at least 0.15 times the distance between the point of zero moment and the point of maximum moment."

TMS 402 should review this requirement and develop a more rational requirement for inclusion in TMS 402

**Response/Rationale:**

ASCE 7-10 Section 14.4.4.2.2 prohibited lap splices in plastic hinge zones of Special Reinforced Masonry Shear Walls. The current language in ASCE 7-16 (shown above) was an on the floor modification to a proposal to eliminate the prohibition of ASCE 7-10 Section 14.4.4.2.2. Dr. Richard Bennett posed a question as to what the current language in ASCE 7-16 is trying to achieve, and how it should be applied.

The vast majority of SRMSWs have a shear span, $M/V_ud_v$ less than 1.5. However, such walls may have a wall segment adjacent to one or more openings, or wall ends, that has a $M/V_ud_v$ which exceeds 1.5. These wall segments typically do not carry significant load, and would develop plastic hinges at the MCE.

The TMS 402/602 committee will undertake a review of this provision during the next cycle, with the intent of placing a requirement in Chapter 7, and then seeking the removal of this requirement from Chapter 14 of ASCE 7-28.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)* *Do not use 'Track Changes'*

**Code:**

**Code Commentary:**

**Specification:**

**Specification Commentary:**

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 10 | *Affirmative* | 1 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 9 | *Did not vote* |

**Subcommittee Comments:**

This was balloted as 19-SL-14 at the subcommittee.

Dillon commented:

> *Replace "The SL committee of TMS 402" with "The TMS 402 committee" because the changes are ultimately made by the committee, not the subcommittee. John C could probably explain this better.*

The recommended change has been made.

While this ballot passed in subcommittee ballot 19, it was not included in Main 19 so that the subcommittee could consider alternative ballots to address the comment during this code cycle. The committee was unable to formulate an alternative ballot, so this ballot is being forwarded for consideration by the main committee at this time.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| | |
|---|---|
| **Committee: Main Committee** | **Ballot #: 20** |
| **Item #: 20-SL-015** | |
| **Technical Contact/Email:** John M. Hochwalt / John.Hochwalt@kpff.com | |
| **Public Comment Number:** 2022 Comment # 166 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☒ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

ASCE 7-16 Chapter 14.4 contains the following provision. "9.3.4.2.5 Coupling Beams. Structural members that provide coupling between shear walls shall be designed to reach their moment or shear nominal strength before either shear wall reaches its moment or shear nominal strength. Analysis of coupled shear walls shall comply with accepted principles of mechanics.

The design shear strength, $\phi V_n$, of the coupling beams shall satisfy the following criterion:

$\phi V_n$=> $1.25(M_1+M_2)/L_c + 1.4V_g$
where
$M_1$, $M_2$ = Nominal moment strength at the ends of the beam;
$L_c$ = Length of the beam between the shear walls; and
$V_g$ = Unfactored shear force caused by gravity loads.

The calculation of the nominal flexural moment shall include the reinforcement in reinforced concrete roof and floor systems. The width of the reinforced concrete used for calculations of reinforcement shall be six times the floor or roof slab thickness.

ACI has similar requirements.

TMS 402 should consider this requirement and either adopt a similar provision, or prohibit coupling beams. This provision would also enhance Appendix C.

**Response/Rationale:**

Coupling beams are difficult to achieve in masonry walls or frames. However, if they are going to be permitted, the provisions for them should reside in TMS 402, not Chapter 14 of ASCE 7

The TMS 402/602 committee will undertake a review of this provision during the next cycle, with the intent of placing a requirement in Chapter 7, and then seeking the removal of this requirement from Chapter 14 of ASCE 7-28.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**Code Commentary:**

**Specification:**

**Specification Commentary:**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 10 *Affirmative* | 1 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 9 *Did not vote* |

**Subcommittee Comments:**

This was balloted 19-SL-15 at the subcommittee.

Dillon commented:

> *Replace "The SL committee of TMS 402" with "The TMS 402 committee" because the changes are ultimately made by the committee, not the subcommittee. John C could probably explain this better.*

The recommended change has been made.

While this ballot passed in subcommittee ballot 19, it was not included in Main 19 so that the subcommittee could consider alternative ballots to address the comment during this code cycle. This process resulted in subcommittee ballots 20-SL-22 and 20B-SL-22 with proposed coupling beam provisions. Unfortunately, that process was ultimately unsuccessful.

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-SL-018 | |
| **Technical Contact/Email:** Jason Thompson / jthompson@ncma.org | |
| **Public Comment Number:** 2022 Comment # 116 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

The requirement to prescriptively hook all horizontal reinforcement regardless of strength or ductility needs is too onerous. Consider the following revisions:

1) Remove the general requirement for hooking of horizontal shear reinforcement from Chapter 6. The broad rationalization for this revision is that shear reinforcement ($V_{sreq} > 0$) is required to be developed...and how that detail is to be accomplished should be left to the engineer and not prescriptively mandated to permit more flexibility in detailing.

2) Introduce a requirement into Chapter 7 requiring standard hooks around the end vertical bar in special reinforced shear walls for both prescriptive horizontal reinforcement ($V_{sreq} = 0$) and shear reinforcement (Vsreq > 0). Hooks are permitted to be 180° or 135° degree hooks at wall terminations or 180°, 135°, or 90° degree hooks at wall intersections. The rationalization for this change recognizes the potential high inelastic demand unique to special reinforced shear walls without specifically attributing the need to any performance objective (mitigating toe crushing, development of horizontal reinforcement, confinement of vertical reinforcement, etc.).

**Response/Rationale:**

As background for the Committee, the following is a brief summary of actions taking during the 2016 update cycle that resulted in the provisions that are the source of this public comment:

- From the 2016 cycle, Ballot Item 07-G-013 reorganized and consolidating the reinforcement detailing requirements. This action effectively relocated the prescriptive hook detailing requirements unique to seismic detailing and strength design and placed them within the general requirements of Chapter 6.
- Also from 2016 cycle, a TAC comment (comment 44) was received highlighting the potential issues associated with this aspect of the reorganization. The TAC comment was as follows:

  > Prior to the reorganization, the language of Section 6.1.7 was presented in the context of detailing stirrups. In its new location is explicitly applies to any type of shear reinforcement...which gets confusing to apply to shear walls. Is the intent in this case to use d or dv? What if the combination of axial and in-plane loads does not produce tension, where does one terminate the shear reinforcement relative to the tension face? Given the committee discussions over the past couple cycles on whether hooks are needed at the end of shear

reinforcement in special reinforced shear walls, I read the last sentences as simply prescriptively mandating such hooks for each and every application. This also could be interpreted as conflicting with provisions of Section 6.1.7.1 which stipulate anchoring only when the shear reinforcement is needed for shear strength. Is the calculated stress intended to be Vs…and if so, why require anchoring of shear reinforcement at the end of a shear wall where the stresses may be small? Difficult to do in some cases, impossible in others. The relocation of these provisions needs to be revisited.

The 2016 402/602 Committee proposed no revisions in response to this TAC comment.

As further background, the current language of 6.1.8 (working draft as of May 31, 2021) was pulled from Section 8.1.6.6.1.1 of TMS 402-13. While the language was indeed vague, Section 8.1.6.6.1 seemed to focus on stirrups rather than horizontal shear reinforcement in walls.

**8.1.6.6.1.1** Shear reinforcement shall extend to a distance *d* from the extreme compression face and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Shear reinforcement shall be anchored at both ends for its calculated stress.

**8.1.6.6.1.1** Stirrups must be carried as close to the compression face of the member as possible because near ultimate load, flexural tension cracks penetrate deeply.

Also from the 2013 TMS 402, detailing requirements for special reinforced masonry shear walls required shear reinforcement to be bent around the end vertical bars, but only using a standard hook. Per Section 7.3.2.6(d) of 402-13:

(d) Shear reinforcement shall be anchored around vertical reinforcing bars with a standard hook.

The TMS 402-13 language shown above no longer exists because TMS 402-16 Chapter 6 effectively requires all horizontal shear reinforcement to be hooked.

An important nuance here is how 'shear reinforcement' is currently defined by TMS 402 (the definition from Section 2.2 of the May 31, 2021 working draft is shown below). Reinforcement is only classified as 'shear reinforcement' when $V_s > 0$…that is, the reinforcement is needed to satisfy shear strength requirements.

*Reinforcement, shear* — Reinforcement required for compliance with Section 8.3.5, Section 9.3.3.1.2, or Section 11.3.4.1.2.

In the context of horizontal reinforcement, the current and historical TMS 402 provisions differ in two significant ways:

- The current working draft of TMS 402 requires hooks at the ends of horizontal shear reinforcement regardless of load level or ductility demand; and
- The current working draft of TMS 402 could produce a permitted design whereby special reinforced masonry shear walls do not have any hooks at the end of the horizontal reinforcement.

As further technical vetting, past Committee discussions also included the question of what type(s) of hooks should be permitted around the edge vertical reinforcement. Research conducted by Seif ElDin (see accompanying paper), investigated the performance of various shear reinforcement terminations, including specifically 180° and 90° hooks as well as non-hooked end terminations. As shown in the figure below, the impact on the peak lateral strength of the tested assemblies with various shear reinforcement terminations conditions was negligible. There was, however, a small but measurable difference in the displacement ductility of the systems with different shear reinforcement terminations.

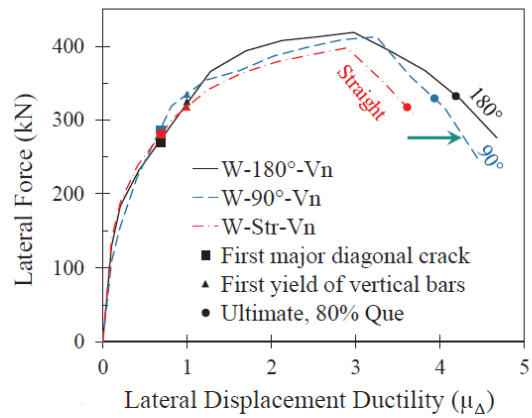
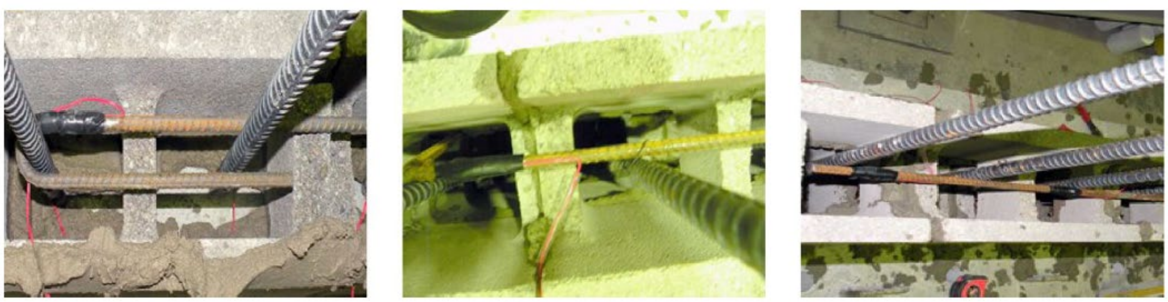



Fig. 8. Effect of horizontal reinforcement anchorage end detail on the in-plane shear strength of RM shear walls.

Of significance, however, is that while the 180° hooks were bent around the end vertical reinforcement, the 90° hooks were bent vertically into the end cells and not around the end vertical reinforcement. While it is difficult to parse out whether hooking around the edge vertical bar provides better engagement, more confinement, or some combination of these or other factors, hooking around the edge vertical bar does appear to provide some benefit.



(a) 180° hook (b) 90° hook (c) Str

For additional context, the provisions of ACI 318-19 for concrete shear walls were reviewed. While ASCE 7 Table 12.2-1 lists multiple types of concrete shear wall, ACI 318 recognizes three types: special (§18.10), intermediate precast (§18.5) and ordinary (Chapter 14).

Where special concrete shear walls have confined boundary elements the shear reinforcement is required to be developed within the confined core; hooks are required if the confined core is not long enough for the shear reinforcement to be developed with a straight bar (See §18.10.6.4 (k). Where there is not a boundary element present, the shear reinforcement is required to be hooked around a vertical bar or U-stirrups can be spliced with straight horizontal bars (See §18.10.6.5). In addition, ACI provides dimensional limits for wall piers that are intended to identify elements of the wall where the shear demand is expected to be limited by flexural yielding. In these elements the shear reinforcing is required to consist of hoops or single bar with 180 degree hooks around the vertical reinforcement at each end of the wall (§18.10.8).

Intermediate precast shear walls only require special detailing of shear reinforcing when the Seismic Design category is D or higher and the element is a wall pier. In that case the shear reinforcing for the wall pier must be detailed like a wall pier in a special wall (See §18.10.8 as referenced by §18.5).

There is no requirement for the anchorage of shear reinforcement in other types of concrete walls.

Based on the above, the following changes are proposed:

- Remove the prescriptive hook requirements from Chapter 6 for reinforcing bars and deformed wire placed in grout. (Maintain prescriptive hooks for joint reinforcement and deformed wire in placed in mortar pending the availability of more research data and subsequent analyses).
- Introduce a prescriptive requirement to hook horizontal reinforcement around the end vertical bar for special reinforced shear walls.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)* *Do not use 'Track Changes'*

**Code:**

**6.1.8** *Shear reinforcement*
Shear reinforcement shall extend to a distance *d* from the extreme compression face and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Shear reinforcement shall be anchored at both ends for its calculated stress.

~~**6.1.8.1** Horizontal shear reinforcement — Horizontal reinforcement shall meet the requirements of Sections 6.1.8.1.1 through 6.1.8.1.3~~

~~**6.1.8.1.1** Except at wall intersections, the ends of horizontal reinforcing bar or deformed wire shall be bent around the edge vertical reinforcing bar or deformed wire with a 180-degree standard hook.~~

~~**6.1.8.1.2** At wall intersections, horizontal reinforcing bars or deformed wire shall be bent around the edge vertical reinforcing bar or deformed wire with a 90- degree standard hook and shall extend horizontally into the intersecting wall a minimum distance at least equal to the development length.~~

~~**6.1.8.1.3**~~ <u>**6.1.8.1**</u> Joint reinforcement used as shear reinforcement shall be anchored around the edge reinforcing bar or deformed wire in the edge cell, either by placement of the vertical reinforcement between adjacent cross-wires or with a 90-degree bend in longitudinal wires bent around the edge cell and with at least 3-in. (76-mm) bend extensions in mortar or grout.

<u>**6.1.8.1.1**</u> ~~**6.1.8.1.3.1**~~ Where the joint reinforcement consists of two longitudinal wires, both of the wires shall be anchored either by one of the following:
(a) Placement of the vertical reinforcement between adjacent cross-wires, or
(b) A 90-degree bend in longitudinal wires bent around the edge cell and with at least 3-in. (76-mm) bend extensions in mortar or grout, or
(c) A 90-degree bend in longitudinal wires bent around the edge cell and with at least 4-in. (102-mm) overlap of the wires in mortar or grout.

<u>**6.1.8.1.2**</u> ~~**6.1.8.1.3.2**~~ Where the joint reinforcement consists of four longitudinal wires, all four of the wires shall be anchored by either:
(a) A 90-degree bend in the inner longitudinal wires bent around the edge cell and with at least 3-in. (76-mm) bend extensions in mortar or grout, and a 3/16 in. (5 mm) U-stirrup lapped at least 8-in. (205-mm) with the outer wires, or
(b) A 90-degree bend in both the inner and outer longitudinal wires bent around the edge cell and with at least 4-in. (102-mm) overlap of the wires in mortar or grout.

**7.3.2.5** Special reinforced masonry shear walls — Design of special reinforced masonry shear walls shall comply with the requirements of Section 8.3, Section 9.3, or Appendix C. Reinforcement detailing shall also comply with the requirements of Section 7.3.2.2.1 and the following:
(a) In-plane flexural reinforcement shall be deformed reinforcing bars.

(b) The maximum spacing of vertical reinforcement shall be the smallest of one-third the length of the shear wall, one-third the height of the shear wall, and 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry not laid in running bond.
(c) The maximum spacing of horizontal reinforcement shall not exceed 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry not laid in running bond.
(d) The maximum spacing of horizontal reinforcement required to resist in-plane shear shall be the smaller of one-third the length of the shear wall and one-third the height of the shear wall. Horizontal reinforcement required to resist in-plane shear shall be uniformly distributed.
(e) Joint reinforcement and deformed wire placed in mortar required to resist in-plane shear shall be a single piece without splices for the length of the wall used for shear design, dv.
(f) The vertical reinforcement ratio shall be at least one- third of the horizontal reinforcement ratio required to resist in-plane shear. The sum of the horizontal reinforcement ratio and vertical reinforcement ratio shall be at least 0.002. Reinforcement ratios shall be based on the gross cross-sectional area of the wall, using specified dimensions and shall be not less than the following.

1. For masonry laid in running bond, the minimum reinforcement ratio in each direction shall be at least 0.0007.
2. For masonry not laid in running bond, the minimum vertical reinforcement ratio shall be at least 0.0007. The minimum horizontal reinforcement ratio shall be at least 0.0015.

Reinforcement used for compliance with these provisions shall be uniformly distributed.
(g) Joint reinforcement used as shear reinforcement shall be anchored in accordance with Section 6.1.8.1.~~3.~~1 (a) or (c) when two longitudinal wires are used and Section 6.1.8.1.~~3.~~2 when four longitudinal wires are used.
(h) Mechanical splices in flexural reinforcement in plastic hinge zones shall meet the requirements of Section 6.1.7.2.1 and develop the specified tensile strength of the spliced bar.
(i) The termination of horizontal reinforcement embedded in grout shall meet one of the following:

1. Except at wall intersections, the ends of horizontal reinforcement shall be bent around the edge vertical reinforcement with a 180-degree standard hook.
2. At wall intersections, horizontal reinforcement shall be bent around the edge vertical reinforcement with a 90-degree standard hook and shall extend horizontally into the intersecting wall a minimum distance at least equal to the development length.

**Code Commentary:**

**6.1.8** *Shear reinforcement*
Design and detailing of shear reinforcement locations and anchorage in masonry requires consideration of the masonry module and reinforcement cover and clearance requirements.

~~**6.1.8.1** *Horizontal shear reinforcement* —~~ Given the definition of “shear reinforcement” in Section 2.2, the requirements of Section 6.1.8 ~~6.1.8.1~~ only apply to horizontal shear reinforcement required by analysis. The requirements do not apply to other horizontal reinforcement, such as prescriptive reinforcement or crack-control reinforcement, although there may be other requirements for these bars.

~~**6.1.8.1.1** In a wall without an intersecting wall at its end, the edge vertical bar or deformed wire is the bar or deformed wire closest to the end of the wall.~~

~~**6.1.8.1.2** When the wall has an intersecting wall at its end, the edge vertical bar or deformed wire is the bar or deformed wire at the intersection of walls. Hooking the horizontal reinforcement around a vertical bar or deformed wire located within the wall running parallel to the horizontal reinforcement would cause the reinforcement to protrude from the wall.~~

**6.1.8.1** ~~**6.1.8.1.3**~~ Wire reinforcement should be anchored around or beyond the edge reinforcing bar or deformed wire. Joint reinforcement longitudinal wires and wire bends are placed over masonry unit face shells in mortar and wire extensions can be placed in edge cell mortar or can extend into edge cell grout. Both joint reinforcement longitudinal wires and cross wires can be used to confine vertical reinforcing bars and deformed wires and grouted cells because wires are developed within a short length.

**6.1.8.1.1** ~~**6.1.8.1.3**~~ The options described for anchoring joint reinforcement are illustrated in Figure CC-6.1-4. Option (a) was used in the testing performed by Baenziger and Porter (2018) and demonstrated performance adequate for use in special reinforced masonry shear walls. While option (c) was not used in the testing, the good performance of overlapped wires in the four wire specimens demonstrated the adequacy of this detail. Option (b) has not been tested for use in special reinforced masonry shear walls.

**6.1.8.1.2** ~~**6.1.8.1.3**~~ The options described for anchoring joint reinforcement ~~reinfocement~~ are illustrated in Figure CC-6.1-4. Both options were used in the testing performed by Baenziger and Porter (2018) and demonstrated performance adequate for use in special reinforced masonry shear walls.

7.3.2.5 (i) Research (Seif Eldin (2017)) has shown an increase in the ductility of masonry piers where the horizontal reinforcement is hooked around the edge vertical bar.

*Add the following to the list of commentary references for Chapter 7:*

Seif ElDin, H.M., and Galal, K. (2017). "Effect of Reinforcement Anchorage End Detail and Spacing on Seismic Performance of Masonry Shear Walls," *Engineering Structures*, 157 (2018) 268-279.

**Specification:**

None.

**Specification Commentary:**

None.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 10 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 10 *Did not vote* |

**Subcommittee Comments:**

The above vote reflects the vote taken in the subcommittee meeting on October 15, 2021. This item had originally been balloted as a letter ballot; the version approved in the subcommittee meeting and presented here includes edits made to address comments received on the letter ballot.

Since this ballot also affects Chapter 6, the members of the RC subcommittee were invited and encourage to participate in the letter ballot voting. No comments were received from members of that subcommittee. The revised ballot 20-SL-18 was presented to the RC subcommittee in their meeting on October 15, 2021. No concerns were expressed about forwarding this ballot to the main committee.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| Item #: 20-SL-019 | |
| Technical Contact/Email: John M. Hochwalt / john.hochwalt@kpff.com | |
| Public Comment Number: 2022 Comment # 37 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 6/1/2021 |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☒ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 37 read as follows:

> This section (6.6.1(b)) states that joint reinforcing conforming to TMS 602 Article 2.4 D is within the scope of Chapter 6. It is unclear, however, whether stainless steel joint reinforcement is covered by this reference. While TMS 602 Article 2.4 D references ASTM A951 which in turn references ASTM 580 for stainless steel wire, the minimum yield strength requirements for wire in ASTM A951 (70 ksi) is incompatible with the yield strengths for ASTM 580 Grade 304 or 316 wire (30 to 45 ksi). This suggests that there may not be stainless steel joint reinforcement that is in conformance with ASTM A951 due to non-compliance with the minimum yield strength. Note that TMS 602 has a separate article that addresses stainless steel joint reinforcement (2.4 I) which only references ASTM A580; this is a wire specification, not a joint reinforcement specification.
>
> If the intent is to allow the use of stainless steel joint reinforcement for applications where conformance with Chapter 6 is required, several items need to be addressed.
>
> First, the specification of stainless steel joint reinforcement in TMS 602 needs to define a minimum yield strength of the wire. In addition it should be clarified that stainless steel joint reinforcement must be fabricated in accordance with ASTM A951, but using the lower strength ASTM A580 wire as permitted by TMS 602.
>
> Second, the provisions should be reviewed for the potential implications of the differing yield strengths of carbon steel and stainless steel joint reinforcement.
> (1) Are they equally as effective when used to meet the prescriptive requirements of Sections 7.3.2.2.1 and 7.4.3.1.1?
> (2) Are the minimum joint reinforcing areas for resisting shear of Sections 7.4.1.2.1 and 7.4.3.2.6 applicable regardless of wire type?
> (3) Is the allowable tensile stress of 30 ksi in Section 8.3.3.2 applicable to all wire types?
> (4) Can stainless steel joint reinforcement be used for conformance with Section 9.1.9.3.1?

**Response/Rationale:**

The Reinforcement and Connectors subcommittee have addressed the first part of this comment with their ballot 19-RC-003 that passed main committee. That ballot clarified the minimum mechanical properties that must be met by stainless steel joint reinforcement – a minimum yield strength of 45 ksi and a minimum ultimate tensile strength of 90 ksi. Ballot 19-RC-003 is attached for the voter's reference.

This ballot addresses items (1) and (2) of the comment which requests that the joint reinforcement provisions of Chapter 7 be reviewed to determine whether the provisions are equally applicable to stainless steel and carbon steel joint reinforcement.

We have concluded that the Chapter 7 provisions are equally applicable to carbon steel and stainless steel joint reinforcement and that no revisions are necessary.

The sections of the code cited by the comment and the committee's assessment of the effect of the differing material properties of carbon and stainless steel joint reinforcing on these provisions are as follows:

**Minimum Prescriptive Horizontal Reinforcement for Shear Walls**

Section 7.3.2.2.1 (Detailed plain reinforced masonry shear walls) establishes a minimum amount of horizontal reinforcement for shear walls that applies to all shear wall types. For special reinforced masonry shear walls, this minimum amount of reinforcement will need to be increased in order to meet the minimum prescriptive reinforcement ratios.

The code and commentary reads as follows.

| | |
|---|---|
| **7.3.2.2** *Detailed plain masonry shear walls* — Design of detailed plain masonry shear walls shall comply with the requirements of Section 8.2 or Section 9.2, and shall comply with the requirements of Section 7.3.2.2.1. | **7.3.2.2** *Detailed plain masonry shear walls* — These shear walls are designed as unreinforced masonry in accordance with the sections noted, but contain minimum reinforcement in the horizontal and vertical directions. Walls that are designed as unreinforced, but that contain minimum prescriptive reinforcement, have more favorable seismic design parameters, including higher response modification coefficients, R, than ordinary plain masonry shear walls. |
| **7.3.2.2.1** *Minimum reinforcement requirements* — Vertical reinforcement of at least 0.2 in.$^2$ (129 mm$^2$) in cross-sectional area shall be provided at corners, within 16 in. (406 mm) of each side of openings, within 8 in. (203 mm) of each side of movement joints, within 8 in. (203 mm) of the ends of walls, and at a maximum spacing of 120 in. (3048 mm) on center.<br><br>Vertical reinforcement adjacent to openings need not be provided for openings smaller than 16 in. (406 mm), unless the distributed reinforcement is interrupted by such openings.<br><br>Horizontal reinforcement shall consist of at least two longitudinal wires of W1.7 (MW11) joint reinforcement spaced not more than 16 in. (406 mm) on center, or at least 0.2 in.$^2$ (129 mm$^2$) in cross-sectional area of bond beam reinforcement spaced not more than 120 in. (3048 mm) on center. | **7.3.2.2.1** *Minimum reinforcement requirements* — The provisions of this section require a judgment-based minimum amount of reinforcement to be included in reinforced masonry wall construction. Tests reported in Gulkan et al (1979) have confirmed that masonry construction, reinforced as indicated, performs adequately considering the highest Seismic Design Category permitted for this shear wall type. This minimum required reinforcement may also be used to resist design loads. |

| | |
|---|---|
| Horizontal reinforcement shall also be provided: at the bottom and top of wall openings and shall extend at least 24 in. (610 mm) but not less than 40 reinforcement diameters past the opening; continuously at structurally connected roof and floor levels; and within 16 in. (406 mm) of the top of walls.<br><br>Horizontal reinforcement adjacent to openings need not be provided for openings smaller than 16 in. (406 mm), unless the distributed reinforcement is interrupted by such openings. | |

Since this is a prescriptive requirement, there is no clear basis to establish an equivalency between various types of reinforcement. Some possibilities would be:

- Elastic stiffness. In this case, there would be no difference between carbon steel and stainless steel joint reinforcement, and the stiffness would be proportional to the area:
    - Reinforcing bars: 0.20 $in^2$ x (12 in/ft / 120 in) = 0.02 $in^2$/ft
    - Joint reinforcement: 2 x 0.017 $in^2$ x (12 in/ft / 16 in) = 0.026 $in^2$/ft
- Yield strength. This would be affected by varying yield strengths. Comparing the existing requirements in terms of yield strength provided foot of wall height, we find:
    - Reinforcing bars: Note the Code allows either Grade 40 or Grade 60 bars.
        - Grade 40: 0.20 $in^2$ x 40 ksi x (12 in/ft / 120 in) = 0.8 k/ft
        - Grade 60: 0.20 $in^2$ x 60 ksi x (12 in/ft / 120 in) = 1.2 k/ft
    - Carbon steel joint reinforcement: 2 x 0.017 $in^2$ x 70 ksi x (12 in/ft / 16 in) = 1.8 k/ft
    - Stainless steel joint reinforcement: 2 x 0.017 $in^2$ x 45 ksi x (12 in/ft / 16 in) = 1.1 k/ft
- Tensile strength. Stainless steel joint reinforcement has a higher tensile strength than carbon steel joint reinforcement.
- Ductility. See discussion under "Minimum Area of Joint Reinforcement When Used As Shear Reinforcement in Shear Walls" below.

It seems mostly likely that the basis of these traditional prescriptive reinforcing requirements was to provide equivalent areas of steel, especially since no distinction in made between grades of reinforcing bars. Even considering other possible rationales, we do not find a compelling reason to treat stainless steel joint reinforcing different than carbon steel joint reinforcing. No change is proposed.

**Minimum Prescriptive Horizontal Reinforcement for Non-Participating Walls**

Section 7.4.3.1.1 (Seismic Design Category C) establishes a minimum amount of horizontal reinforcement for non-participating walls that applies to Seismic Design Categories C and higher. For non-participating walls not laid in running bond in Seismic Design Categories E and F, this minimum amount of reinforcement will need to be increased in order to meet the minimum prescriptive reinforcement ratios.

The code and commentary reads as follows.

| | |
|---|---|
| **7.4.3** *Seismic Design Category C requirements* — Masonry elements in structures assigned to Seismic Design Category C shall comply with the requirements of Section 7.4.2 and with the additional requirements of Section 7.4.3.1 and 7.4.3.2. | **7.4.3** *Seismic Design Category C requirements* — In addition to the requirements of Seismic Design Category B, minimum levels of reinforcement and detailing are required. The minimum provisions for improved performance of masonry construction in Seismic Design Category C must be met regardless of the method of design. Shear walls designed as part of the seismic-force-resisting system in Seismic Design |

| | |
|---|---|
| | Category C and higher must be designed using reinforced masonry methods because of the increased risk and expected intensity of seismic activity. Ordinary reinforced masonry shear walls, ordinary reinforced AAC masonry shear walls, intermediate reinforced masonry shear walls, special reinforced masonry shear walls, or masonry infills are required to be used. |
| **7.4.3.1** *Design of nonparticipating elements —* Nonparticipating masonry elements shall comply with the requirements of Section 7.3.1 and Chapter 8, 9, 10, 11, 12, 15, or Appendix D. Nonparticipating masonry elements, except those constructed of AAC masonry, shall be reinforced in either the horizontal or vertical direction in accordance with Sections 7.4.3.1.1 and 7.4.3.1.2.<br>**7.4.3.1.1** *Horizontal reinforcement* — Horizontal reinforcement shall be provided within 16 in. (406 mm) of the top and bottom of nonparticipating masonry walls and shall consist of one of the following:<br>(a) Two longitudinal wires of W1.7 (MW11) joint reinforcement spaced not more than 16 in. (406 mm) on center. The space between these wires shall be the widest that the mortar joint will accommodate.<br>(b) Two D2 (MD13) deformed wires spaced not more than 16 in. (406 mm) on center for walls greater than 4 in. (102 mm) in width and at least one D2 (MD13) wire spaced not more than 16 in. (406 mm) on center for walls not exceeding 4 in. (102 mm) in width. Where two deformed wires are used, the space between these wires shall be the widest that the mortar joint will accommodate.<br>(c) One No. 4 (M #13) bar or one D20 (MD129) wire spaced not more than 48 in. (1219 mm) on center. | **7.4.3.1** *Design of nonparticipating elements —* Reinforcement requirements of Section 7.4.3.1 are traditional for conventional concrete and clay masonry. They are prescriptive in nature. The intent of this requirement is to provide structural integrity for nonparticipating masonry walls. AAC masonry walls differ from concrete masonry walls and clay masonry walls in that the thin-bed mortar strength and associated bond strength is typically greater than that of the AAC units. Also, the unit weight of AAC masonry is typically less than one-third of the unit weight of clay or concrete masonry, reducing seismic inertial forces. This reduced load, combined with a tensile bond strength that is higher than the strength of the AAC material itself, provides a minimum level of structural integrity. Therefore, prescriptive reinforcement is not required. All masonry walls, including non-participating AAC masonry walls, are required to be designed to resist out-of-plane forces. If reinforcement is required, it must be provided in the direction of the span. Permitted types of reinforcement are defined in Section 6.1.1. Commentary Section 6.1.3 provides additional information. |

Like the prescriptive requirement for shear walls, there is no clear basis to establish an equivalency between various types of reinforcement. The only difference here is that the bar reinforcement in this situation has a maximum spacing of 48 inches. Oddly, by all of the measures considered above for prescriptive shear wall reinforcement, there appears to be no basis for equivalency between carbon steel bars and carbon steel joint reinforcement. For example, the comparison of reinforcing areas would be as follows:

- Reinforcing bars: 0.20 in$^2$ x (12 in/ft / 48 in) = 0.05 in$^2$/ft
- Joint reinforcement: 2 x 0.017 in$^2$ x (12 in/ft / 16 in) = 0.026 in$^2$/ft

Twice as much steel is required for the bar option as for the joint reinforcement option. As a result, it seems mostly likely that these traditional prescriptive reinforcing requirements were based on an engineering judgment about the efficacy of closely spaced reinforcing in maintaining wall integrity under extreme conditions. Material properties or strength do not appear to have been considerations.

Alternatively, if one considers the underlying intent of the provisions – the maintenance of wall integrity – reinforcing will be effective in maintaining integrity until tensile failure. On the basis of tensile strength, stainless steel joint reinforcement may be treated as equivalent to carbon steel joint reinforcement.

No change is proposed.

**Minimum Area of Joint Reinforcement When Used As Shear Reinforcement in Shear Walls**

Section 7.4.3.1.1 (Seismic Design Category C) establishes a minimum amount of horizontal reinforcement for non-participating walls that applies to Seismic Design Categories C and higher. For non-participating walls not laid in running bond in Seismic Design Categories E and F, this minimum amount of reinforcement will need to be increased in order to meet the minimum prescriptive reinforcement ratios.

The code and commentary reads as follows.

| | |
|---|---|
| **7.4.1** *Seismic Design Category A requirements*<br>. . .<br>**7.4.1.1** *Design of nonparticipating elements*<br>. . .<br>**7.4.1.2** *Design of participating elements* | |
| **7.4.1.2.1** *Joint reinforcement used as shear reinforcement* — Horizontal joint reinforcement used as shear reinforcement in walls shall consist of at least two 3/16 in. (4.8 mm) diameter longitudinal wires located within a bed joint and placed over the masonry unit face shells. The maximum spacing of joint reinforcement used as shear reinforcement shall not exceed 16 in. (406 mm). | **7.4.1.2.1** *Joint reinforcement used as shear reinforcement* — The quantities of joint reinforcement indicated are minimums and the designer should evaluate whether additional reinforcement is required to satisfy specific seismic conditions.<br><br>Studies of minimum shear reinforcement requirements (Schultz (1996); Baenziger and Porter (2018); Baenziger and Porter (2011); Porter and Baenziger (2007); Sveinsson et al (1985); Schultz and Hutchinson (2001)) have shown that when sufficient area, strength, and strain elongation properties of reinforcement are provided to resist the load transferred from the masonry after cracking, then the reinforcement does not rupture upon cracking of the masonry. Equivalent performance of shear walls with bond beams and shear walls with bed joint reinforcement under simulated seismic loading was observed in the laboratory tests (Baenziger and Porter (2011); Schultz and Hutchinson (2001)). Minimum Code requirements have been provided (Schultz (1996)) to satisfy both strength and energy criteria.<br><br>Joint reinforcement of at least 3/16 in. (4.8 mm) diameter longitudinal wire is deemed to have sufficient strain elongation and, thus, was selected as the minimum size when joint reinforcement is used as the primary shear and flexural reinforcement. The research (Baenziger and Porter (2011)) was for walls that contained a minimum of two 3/16 in. (4.8 mm) |

| | |
|---|---|
| | diameter longitudinal wires in a bed joint. Other research (Schultz and Hutchinson (2001)) contained two No. 9 gage (0.148 in. (3.76 mm)) diameter longitudinal wires or two No. 5 gage (0.207 in. (5.26 mm)) diameter longitudinal wires in a bed joint. The No. 5 gage longitudinal wires exhibited similar ductility to the joint reinforcement in the Baenziger/Porter research. |
| **7.4.3** *Seismic Design Category C requirement*<br>. . .<br>**7.4.3.2** *Design of participating elements*<br>. . . | |
| **7.4.3.2.6** *Joint reinforcement used as shear reinforcement* — The maximum spacing of horizontal joint reinforcement used as shear reinforcement in walls shall not exceed 8 in. (203 mm) in partially grouted walls. Joint reinforcement used as shear reinforcement in fully grouted walls shall consist of four 3/16 in. (4.8 mm) diameter longitudinal wires at a spacing not to exceed 8 in. (203 mm). | **7.4.3.2.6** *Joint reinforcement used as shear reinforcement* — See Commentary for Section 7.4.1.2.1. |

The key section of commentary that needs to be considered in evaluating stainless steel joint reinforcing reads as follows:

> *. . .when sufficient area, strength, and strain elongation properties of reinforcement are provided to resist the load transferred from the masonry after cracking, then the reinforcement does not rupture upon cracking of the masonry.*

Three factors are cited:

- Area. This is the same for carbon steel and stainless steel joint reinforcement.
- Strength. By referencing “rupture” the commentary makes clear that this is intended to be the tensile strength of the joint reinforcement. The minimum tensile strength of stainless steel joint reinforcement exceeds the minimum tensile strength of carbon steel joint reinforcement.
- Strain elongation properties. ASTM A951 does not establish requirements relative to material strains. The wire standards do, however:
    - ASTM A580 uses two metrics to quantify the strain elongation properties of stainless steel wire:
        - Elongation within a length equal to 4 times the wire gauge: 30% minimum.
        - Reduction in wire area at rupture: 40% minimum
    - ASTM A1064 only provides one metric to quantify the strain elongation properties of carbon steel wire:
        - Reduction in wire area at rupture: 30% minimum

  While ASTM A1064 does not establish a minimum elongation for carbon steel wire, there is available test data that addresses the elongation that can be achieved by carbon steel wire. As shown in Table 4.2(c) from the Wire Reinforcement Institute’s Manual of Standard Practice, the total elongation of reinforcing wire at failure has a mean value of 8.9%, with a low of 6%. Schultz (1996) reports elongations from 4% and 6%.

| TABLE 4.2(c) Summary of test Criteria in table 3(b) (27 Samples Tested) | | | |
|---|---|---|---|
| $f_y$ Range @ 0.35% of Strain | $f_t$ Range (ult) | %Elongation total* A370, **A4.4.2** | Permanent A370, **A4.4.1** |
| 73-88ksi | 91-102 ksi | 6-14%<br>Mean - 8.9% | 4-6%<br>5% |

* Maximum strength or maximum stretch is the full measure of extension before fracture. It is the true measure of elongation (total). Research background for this testing can be found in the ACI discussion paper, Disc.88-S60 in ACI Structural Journal, July - August 1992.

(Note: this table no longer appears in the 2021 Manual of Standard Practice, but was in prior editions.)

The conclusion is that the properties of stainless steel wires equal or exceed those of carbon steel wires for the factors cited in the commentary.

No change is proposed.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)  Do not use 'Track Changes'*

**Code:**

None.

**Code Commentary:**

None.

**Specification:**

None.

**Specification Commentary:**

None.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 12 *Affirmative* | 1 *Affirmative w/ comment* | 1 *Negative* | 0 *Abstain* | 6 *Did not vote* |

**Subcommittee Comments:**

The negative vote by Gangel was withdrawn based on a response by Schultz, but the response to the vote may be helpful to some voters so it is included here.

Withdrawn comment:

*I am sorry to say I am little behind on the data and/or testing associated with this provision. It seemed to me that we disallowed horizontal joint reinf from special shear walls years ago because of the brittle rupture behavior. Additional testing was done by Schultz and Porter and we allowed the joint reinf but for Special walls, it was to have a larger diameter. This larger diameter was to overcome a "gage length" deficiency. I will withdraw my negative given assurance that the "gage length" issue with stainless is the same as for the joint reinf that is currently allowed in the code.*

Response by Schultz:

*At NIST we used stainless wire reinforcement in a larger diameter because we wanted to have more steel area. It was NOT to make up for any deficiency in the stainless product regarding the gage length issue. We encountered no problems using this material, and the wire reinforcement performed very well. Stainless steel typically is more ductile than cold-drawn wire (or even hot-rolled products), and as a stainless steel it will offer better corrosion resistance. I also add that wire reinforcement today is better than the product we used at NIST in the mid-1990s, which was better than the stuff used in the Berkeley and Colorado tests in the 1970s and 1980s. I have no concern over the performance of this product relative to galvanized wire products.*

Porter emailed on behalf himself and his co-researcher Baenzinger in support of Schultz's position.

The affirmative with comment vote from Robinson suggested improvements and corrections to the rationale which have been incorporated into this ballot.

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-SL-020 | |
| **Technical Contact/Email:** John M. Hochwalt / john.hochwalt@kpff.com | |
| **Public Comment Number:** 2022 Comment # 104 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 104 read as follows:

The following suggestions are made relative to the treatment of prestressed shear walls in Chapter 7:

- 7.3.2.10 (a) and (e) have incorrect references to the special reinforced wall provisions. 7.3.2.5 (b), (c), and (d) should be referenced in lieu of 7.3.2.5 (a) and (b).
- In the first paragraph of the commentary for both 7.3.2.10 and 7.3.2.11, the commentary should state "bonded reinforcement" instead of "mild reinforcement" since 7.3.2.10 (e) allows the use of bonded prestressed reinforcement to meet the prescriptive requirements
- In the first paragraph of the commentary for both 7.3.2.10 and 7.3.2.11, the references to detailing requirements that are not required by the code should be deleted.
- It is suggested to delete 7.3.2.11 (a) as it is redundant relative to 7.3.2.10 (e).
- 7.3.2.11 (d) references 9.3.5.6 for ductility requirements. The classification of special reinforced prestressed walls in Table 9.3.5.6.1 should be clarified.
- In the commentary for Section 7.4.4, special prestressed walls should be added to the first sentence. This sentence should be moved to 7.4.4.2.

**Response/Rationale:**

For the most part the committee agrees with the public comment for the reasons provided by the commenter.

The changes proposed to the code provisions for 7.3.2.10 are required to reflect the reorganization of Section 7.3.2.5 in the course of this cycle.

The committee concurs with the commenter that the changes to the commentary for 7.3.2.10 and 7.3.2.11 to use “bonded” rather than “mild” are necessary for consistency with the code provisions which allow the use of bonded prestressed reinforcement for compliance with the prescriptive reinforcement requirements. The committee also agrees that the commentary to these sections should not reference broader compliance with 7.3.2.5 than is required by Code.

The committee does not agree with the commenter's suggestion to delete 7.3.2.11 (a) because it is redundant with 7.3.2.10 (e). 7.3.2.11 (a) is addressing different requirements than are addressed by 7.3.2.10 (e).

The committee agrees that Table 9.3.5.6.1 should include special prestressed walls since compliance with Section 9.3.5.6 is required for special prestressed walls.

The committee agrees that that commentary addressing permitted shear wall types in SDC D+ is better located at Section 7.4.4.2 where the Code limits the permissible wall types.

Lastly, the committee suggests as future business that it be considered whether compliance with Table 9.3.5.6.1 should be required for intermediate prestressed shear walls.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**7.3.2.5** *Special reinforced masonry shear walls* — Design of special reinforced masonry shear walls shall comply with the requirements of Section 8.3, Section 9.3, or Appendix C. Reinforcement detailing shall also comply with the requirements of Section 7.3.2.2.1 and the following:

(a) In-plane flexural reinforcement shall be deformed reinforcing bars.
(b) The maximum spacing of vertical reinforcement shall be the smallest of one-third the length of the shear wall, one-third the height of the shear wall, and 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry not laid in running bond.
(c) The maximum spacing of horizontal reinforcement shall not exceed 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry not laid in running bond.
(d) The maximum spacing of horizontal reinforcement required to resist in-plane shear shall be the smaller of one-third the length of the shear wall and one-third the height of the shear wall. Horizontal reinforcement required to resist in-plane shear shall be uniformly distributed.

. . .

**7.3.2.10** *Intermediate reinforced prestressed masonry shear walls* — Intermediate reinforced prestressed masonry shear walls shall comply with the requirements of Chapter 10, the reinforcement detailing requirements of Section 7.3.2.2.1, and the following:

(a) Reinforcement shall be provided in accordance with Sections 7.3.2.5(~~a~~<u>b</u>)<u>, 7.3.2.5 (c),</u> and 7.3.2.5(~~b~~<u>d</u>).
(b) The minimum area of horizontal reinforcement shall be 0.0007$bd_v$.
(c) Shear walls subjected to load reversals shall be symmetrically reinforced.
(d) The nominal moment strength at any section along the shear wall shall not be less than one-fourth the maximum moment strength.
(e) The cross-sectional area of bonded tendons shall be considered to contribute to the minimum reinforcement in Sections 7.3.2.2.1, 7.3.2.5(~~a~~<u>b</u>)<u>, 7.3.2.5 (c),</u> and 7.3.2.5(~~b~~<u>d</u>).
(f) Tendons shall be located in cells that are grouted the full height of the wall.

**7.3.2.11** *Special reinforced prestressed masonry shear walls* — Special reinforced prestressed masonry shear walls shall comply with the requirements of Chapter 10, the reinforcement detailing requirements of Sections 7.3.2.2.1 and 7.3.2.10 and the following:

(a) The cross-sectional area of bonded tendons shall be considered to contribute to the minimum reinforcement in Sections 7.3.2.2.1 and 7.3.2.10.
(b) Prestressing tendons shall consist of bars conforming to ASTM A722/A722M.

(c) All cells of the masonry wall shall be grouted.
(d) The requirements of Section 9.3.5.6 shall be met. Dead load axial forces shall include the effective prestress force, $A_{ps}f_{se}$.
(e) The design shear strength, $\phi V_n$, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, $M_n$, of the element, except that the design shear strength, $\phi V_n$, need not exceed 2.0 times required shear strength, $V_u$.

. . .

**7.4.4** *Seismic Design Category D requirements* — Masonry elements in structures assigned to Seismic Design Category D shall comply with the requirements of Section 7.4.3 and with the additional requirements of Sections 7.4.4.1 and 7.4.4.2.
Exception: Design of participating elements of AAC masonry shall comply with the requirements of Section 7.4.3.

**7.4.4.1** *Design of nonparticipating elements*
. . .
**7.4.4.2** *Design of participating elements* — Masonry shear walls shall be designed to comply with the requirements of Section 7.3.2.5, 7.3.2.8, or 7.3.2.11.
. . .

**Table 9.3.5.6.1: Strain in Extreme Tensile Reinforcement**

| Shear Wall | Tensile strain in reinforcement | |
|---|---|---|
| | $M_u/V_ud_v < 1$ | $M_u/V_ud_v \geq 1$ |
| Intermediate reinforced | $1.5\varepsilon_y$ | $3.0\ \varepsilon_y$ |
| Special reinforced, special prestressed | $1.5\ \varepsilon_y$ | $4.0\ \varepsilon_y$ |

**Code Commentary:**

**Note: The commentary shown for Section 7.4.4 reflects the passage of Ballot 19-SL-07.**

**7.3.2.10** *Intermediate reinforced prestressed masonry shear walls* — These shear walls are philosophically similar in concept to intermediate reinforced masonry shear walls. To provide the intended level of inelastic ductility, prescriptive ~~mild~~ bonded reinforcement is required. Intermediate reinforced prestressed masonry shear walls should include ~~the detailing requirements from Section 7.3.2.4 and~~ the sectional ductility (a/d) requirement in Section 10.5.3.

ASCE/SEI 7, Tables 12.2-1 and 12.14-1 conservatively combine all prestressed masonry shear walls into one category for seismic coefficients and structural system limitations on seismic design categories and height. The design limitations included in those tables are representative of ordinary plain prestressed masonry shear walls. Given that an intermediate prestressed masonry shear wall can be partially grouted, Hassanli et al (2015) recommend R and Cd factors of 2½ and 2.9, respectively. To utilize the seismic design factors proposed by Hassanli et al (2015), the structure would have to be accepted under Section 1.3, Alternative design or method of construction.

**7.3.2.11** *Special reinforced prestressed masonry shear walls* — These shear walls are philosophically similar in concept to special reinforced masonry shear walls. To provide the intended level of inelastic ductility, prescriptive ~~mild~~ bonded reinforcement is required. Special reinforced prestressed masonry shear walls should include ~~the detailing requirements from Section 7.3.2.5 and~~ the sectional ductility (a/d) requirement in Section 10.5.3.

ASCE/SEI 7, Table 12.2-1 and ASCE/SEI 7, Table 12.14-1 conservatively combine all prestressed masonry shear walls into one category for seismic coefficients and structural system limitations on seismic design categories and height. The design limitations included in those tables are representative of ordinary plain prestressed masonry shear walls. Given that a special prestressed masonry shear wall must be fully grouted, Hassanli et al (2015) recommend R and Cd factors of 3 and 3½, respectively. To utilize the seismic design factors proposed by Hassanli et al (2015), the structure would have to be accepted under Section 1.3, Alternative design or method of construction.

. . .

~~**7.4.4** *Seismic Design Category D requirements* — Masonry shear walls for structures assigned to Seismic Design Category D are required to meet the requirements of special reinforced masonry shear walls or ordinary reinforced AAC masonry shear walls because of the increased risk and expected intensity of seismic activity.~~

**7.4.4.2** *Design of participating elements* — Masonry shear walls for structures assigned to Seismic Design Category D are required to meet the requirements of special reinforced masonry shear walls, ordinary reinforced AAC masonry shear walls, or special reinforced prestressed masonry shear walls because of the increased risk and expected intensity of seismic activity.

**Specification:**

None.

**Specification Commentary:**

None.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 16 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 4 *Did not vote* |

**Subcommittee Comments:**

When balloted at the subcommittee, the outcome of ballot 19-SL-07 was unknown. As a result, the subcommittee ballot included the following comment:

> *Note: The Code Ballot 19-SL-07, which is currently under consideration by the main committee, would relocate the commentary for 7.4.4 which is shown to remain in this ballot. If both this ballot and 19-SL-07 pass, there would be no remaining commentary for 7.4.4 as all of the commentary will have been relocated.*

This ballot has been modified from that passed at the subcommittee to reflect the passage of ballot 19-SL-07 by the main committee, consistent with the above statement in the subcommittee ballot.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-SL-021 | |
| **Technical Contact/Email:** John M. Hochwalt / john.hochwalt@kpff.com | |
| **Public Comment Number:** 2022 Comment # 139 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☒ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 139 read as follows:

> It is unclear how the participating infills in Section 12.3 relate to Chapter 7. In what Seismic Design Categories is it anticipated that these would be used?

**Response/Rationale:**

The reference to Chapter 12 (Appendix B in TMS 402-16) in Section 7.4 for the design of participating wall types, allows the use of the participating infill design methodology in Section 12.3 to design walls resisting seismic loads in Seismic Design categories A, B, and C.

The commenter's confusion was due to a misunderstanding in thinking that a participating infill was a type of shear wall and thus needed to be listed in Section 7.3.2:

> **7.3.2** *Participating elements* — Masonry walls that are part of the seismic-force-resisting system shall be classified as participating elements and shall comply with the requirements of Section 7.3.2.1, 7.3.2.2, 7.3.2.3, 7.3.2.4, 7.3.2.5, 7.3.2.6, 7.3.2.7, 7.3.2.8, 7.3.2.9, 7.3.2.10, or 7.3.2.11.

The wall type used in a participating infill design is still required to be one of those listed in Section 7.3.2. This is established by Section 12.1.1 which requires that infills are compliant with Part 2, which includes Chapter 7. As explained in the Commentary to Section 12.1.1, this means that participating infills must be compliant with the detailing requirements for a wall type in Section 7.3.2.

In response to the commenter's question, the participating infill design methodology is permitted to be used in Seismic Design Categories A, B and C.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)  Do not use 'Track Changes'*

**Code:**

None.

**Code Commentary:**

None.

**Specification:**

None.

**Specification Commentary:**

None.

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 16 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 4 | *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| Item #: 20-SL-023 | |
| Technical Contact/Email: John M. Hochwalt / john.hochwalt@kpff.com | |
| Public Comment Number: 2022 Comments # 147 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 6/1/2021 |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

In reference to the commentary for 7.4.4.2.1, Public Comment 14 read as follows:

> Consider updating this commentary. Would it be clearer to refer to beneficial effects of column ties as "confinement"? Also, the last phrase "and better resistance to shear" is incorrect. Shear will be constant over the height of the column; when heavier ties are provided at the top and bottom of the column it is to provide enhanced confinement of potential hinge regions.
>
> Should enhanced confinement of potential hinge regions be made mandatory?

**Response/Rationale:**

The first part of the comment was addressed by Ballot 19-SL-11 which has been passed by the committee. This ballot addresses the second part of the comment that asks "Should enhanced confinement of potential hinge regions be made mandatory?"

At the time of the public comment, the question was referencing the following sentence:

> *Columns not involved in the seismic-force-resisting system should also be more heavily tied at the tops and bottoms for more ductile performance and better resistance to shear.*

With the passage of 19-SL-11 that sentence now reads:

> *Columns not involved in the seismic-force-resisting system should also be more heavily tied at the tops and bottoms for more ductile performance in potential plastic hinge regions.*

In either case, this commentary places users in a difficult position by suggesting that enhanced confinement should be provided in non-participating columns, but without providing guidance as to what that confinement should be.

Rather than create new requirements for non-participating columns, the ballot proposes to require the same confinement in the end regions of non-participating columns as is required for the full height of participating columns. The end region is defined as twice the maximum column dimension, which is consistent with the way plastic hinge regions are defined by ACI 318 for concrete columns.

The confinement required for participating columns by Section 7.4.4.2.1 is based on the columns being designed for an R value not greater than 1.5 in accordance Section 7.4.3.2.4; i.e. a condition with limited ductility demands. This level of confinement is likely not sufficient to allow the development of a stable plastic hinge; concrete columns that are designed to accommodate plastic hinging are required to have much greater confinement. For example, in Seismic Design Category D, multiple legs of #5 ties are often spaced at 4 to 6 inches on center in hinging concrete columns.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)  Do not use 'Track Changes'*

**Code:**

**7.3.1** *Nonparticipating elements* — Masonry elements that are not part of the seismic-force-resisting system shall be classified as nonparticipating elements and shall be isolated in their own plane from the seismic-force-resisting system. Isolation joints and connectors shall be designed to accommodate the design story drift.

Exception: Isolation is not required if a deformation compatibility analysis demonstrates that the non-participating element can accommodate the inelastic displacement, $C_d\delta_{ne}$, of the structure in a manner complying with the requirements of this code. Elements supporting gravity loads in addition their self-weight shall be evaluated for gravity load combinations of (1.2$D$ + 1.0$L$ + 0.2$S$) or 0.9$D$, whichever is critical, acting simultaneously with the inelastic displacement and shall have a ductility compatible with the ductility of the lateral force resisting system. The influence of any non-isolated nonparticipating elements on the lateral force resisting system shall be considered in design in accordance with Section 4.1.6 of this code.

. . .

**7.4.4** *Seismic Design Category D requirements*
Masonry elements in structures assigned to Seismic Design Category D shall comply with the requirements of Section 7.4.3 and with the additional requirements of Sections 7.4.4.1 and 7.4.4.2.
*Exception:* Design of participating elements of AAC masonry shall comply with the requirements of Section 7.4.3.

**7.4.4.1** *Design of nonparticipating elements* — Nonparticipating masonry elements shall comply with the requirements of Chapter 8, 9, 10, 11, or 12. Nonparticipating masonry elements, except those constructed of AAC masonry, shall be reinforced in either the horizontal or vertical direction in accordance with the following:
(a) Horizontal reinforcement — Horizontal reinforcement shall comply with Section 7.4.3.1.1.
(b) Vertical reinforcement — Vertical reinforcement shall consist of at least one No. 4 (M #13) bar or one D20 (MD 29) wire spaced not more than 48 in. (1219 mm). Vertical reinforcement shall be located within 16 in. (406 mm) of the ends of masonry walls.

**7.4.4.1.1** *Minimum reinforcement for non-participating masonry columns* — Lateral ties conforming to the requirements of Section 7.4.4.2.1 shall be provided for a length equal to twice the larger column dimension from the top and bottom of the column at each floor.

*Exception:* Compliance with this provision is not required if either of the following requirements are met:

(a) The column is isolated from building displacements in conformance with Section 7.3.1.
(b) An analysis complying with Section 7.3.1 demonstrates that the column will remain elastic when subjected to the required inelastic displacement.

**7.4.4.2** *Design of participating elements* — Masonry shear walls shall be designed to comply with the requirements of Section 7.3.2.5, 7.3.2.8, or 7.3.2.11.

**7.4.4.2.1** *Minimum reinforcement for participating masonry columns* — Lateral ties in masonry columns shall be spaced not more than 8 in. (203 mm) on center and shall be at least 3/8 in. (9.5 mm) diameter. Lateral ties shall be embedded in grout.

**Code Commentary:**

**7.3.1** *Nonparticipating elements* — With regards to the exception, non-isolated, nonparticipating elements can influence a structure's strength and stiffness, and as a result the distribution of lateral loads and building irregularities. Non-isolated nonparticipating elements can inadvertently have significant effects on the performance of a structural system and are to be considered in accordance with the code. This should also be considered in accordance with other applicable provisions such as the modeling criteria of ASCE/SEI 7. Where partial height non-participating elements are constructed tight to building columns, this should include the consideration of short column effects.

The deformation compatibility analysis may consider the effect of cracking on element stiffness. Elements that are detailed to achieve ductile behavior may also develop plastic mechanisms. For example, elements detailed in accordance with the provisions for special reinforced masonry shear walls may be able to accommodate displacements through the development of plastic hinges. For such elements, Appendix C may be used to provide guidance on the determination of hinge rotation capacity. In addition to these provisions, other applicable provisions, such as the deformation limit and deformation compatibility provisions of ASCE/SEI 7 should be considered in design.

. . .

**7.4.4.1.1** *Minimum reinforcement for non-participating masonry columns* — When columns are not isolated from building displacements, yielding of reinforcing steel or crushing of masonry may occur in response to those displacements. Providing a level of confinement consistent with that required for participating columns is intended to maintain column integrity in those conditions. The length of twice the larger column dimension represents the extent over which the inelastic behavior is expected to be concentrated.

This level of confinement may not be sufficient to allow the development of plastic hinges. If building displacements are to be accommodated through hinging of the non-participating columns, the rotation capacity of the columns will need to be assessed. See discussion in the commentary to Section 7.3.1 on the use of plastic hinges to accommodate building movements.

. . .

**7.4.4.2.1** *Minimum reinforcement for participating masonry columns* — Adequate lateral restraint is important for column longitudinal reinforcement resisting compression forces due to earthquakes. Many column failures during earthquakes have been attributed to buckling of longitudinal reinforcement and inadequate confinement of concrete or masonry in compression. For this reason, closer spacing of lateral ties than might otherwise be required is prudent. An arbitrary minimum spacing has been established through experience that is appropriate for columns designed with a R value not exceeding 1.5 per Section 7.4.3.2.4. ~~Columns not involved in the seismic-force-resisting system should also be more heavily tied at the tops and bottoms for more ductile performance in potential plastic hinge regions.~~

**Specification:**

None

**Specification Commentary:**

None

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 8 | *Affirmative* | 1 | *Affirmative w/ comment* | 0 | *Negative* | 1 | *Abstain* | 10 | *Did not vote* |

**Subcommittee Comments:**

Robinson commented:

> *Ballot item # is listed as 20-SL-11, but it should be 20-SL-23.*

This correction has been made.

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-SL-024 | |
| **Technical Contact/Email:** John M. Hochwalt / john.hochwalt@kpff.com | |
| **Public Comment Number:** 2022 Comment # 137 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 137 read as follows:

> *Foundation dowels add resilience for better long term performance, and also improve construction safety of masonry walls. The concrete code has had dowel requirements for several years. Is there any consideration to adding a dowel requirement to the masonry code?*

**Response/Rationale:**

Response to Commenter

Significant effort was made this cycle to consider a requirement for foundation dowels. In addition to this ballot, three main committee ballots on this topic were also considered in the course of this cycle.

As a Building Code, TMS 402 is intended to provide minimum standards relative to the life safety of the completed structure. TMS 602 supports TMS 402 in providing minimum standards of construction that have been assumed in the development of the TMS 402 Code. Neither document is intended to address construction site safety, which is the purview of regulatory agencies.

As such, this ballot does not address the possible safety benefits of requiring minimum foundation dowels. This ballot does however, consider the portion of the comment that suggests that minimum foundation dowels would add resilience and improved long term performance. It does so in the context of other building codes, specifically the concrete code (ACI 318).

In the context of current building codes, resilience is addressed through structural integrity provisions. The structural integrity provisions are intended to enhance the ability of the structure to survive unexpected conditions by providing minimum connections between parts of the structure and minimum continuous ties through the parts.

Extreme earthquakes – earthquakes stronger than anticipated by the building code – are a specific type of unexpected event that building codes are concerned about maintaining structural integrity through ductile detailing.

Thus, in response to this comment, it is proposed to add minimum dowel requirements to provide general structural integrity and resiliency in Seismic Design Categories A and higher, and more stringent requirements in Seismic Design Category D and higher that reflect the greater risk of extreme loading in these areas of higher seismic risk.

Previous Ballots

As noted above, three ballots related to minimum foundation dowels have been considered previously. Those ballots were:

- 2-SM-002: This ballot was in response to 2016 Public Comment 73 which proposed that the specification require the provision of foundation dowels. The ballot proposed to make no changes to the code or specification, with the rationale that this was a design issue and not a construction issue and that the design issue was adequately addressed by Section 4.1.1 which requires a continuous load path. This ballot received 2 negative votes. The negative votes were withdrawn, however, so that this ballot became the official response to 2016 Public Comment 73.
- 14-SM-008: This ballot proposed to require that foundation dowels be provided to match the vertical reinforcement of walls, columns and pilasters unless specifically designed otherwise. This ballot received 7 negative votes, one of which was found persuasive, terminating this ballot.
- 15B-SM-008: This ballot proposed a series of provisions addressing the interface of masonry with the foundations, based on the provisions in ACI 318-14. This ballot received 5 negative votes, one of which was found persuasive, terminating this ballot.

These ballots are included with this ballot for the voter's reference. A response to the comments on ballot 15B-SM-008 has also been provided.

Consistent with ballot 2-SM-002, this ballot treats the provision of foundation dowels as a design issue. Commentary is provided to acknowledge that the contractor may want additional dowels for reasons of safety or to limit external bracing of the walls during construction, and to identify for the user when those additional dowels could affect the design.

Consistent with ballot 15B-SM-008, the foundation dowel provisions of ACI 318 are used as a model for developing provisions for TMS 402. This ballot, however, proposes to place the provisions in Chapter 7 rather than Chapter 5. The reason for this is that in higher seismic design categories, having dowels that match the wall reinforcement is important for achieving the required level of ductility, as is discussed in more detail below. Given the need to have foundation dowels in higher seismic design categories in Chapter 7, it is proposed to place all of the foundation dowel provisions in Chapter 7 for ease of user reference.

The Case Against Friction

> *Generally, connections between structural components should be ductile and have a capacity for relatively large deformations and energy absorption under the effect of abnormal conditions. This criteria may be met in many different ways, depending on the structural system used.*
>
> Excerpt from Commentary to ASCE 7-16 Section 1.4 on General Structural Integrity.

If there are no dowels at the interface between the masonry members and the foundation, friction due to gravity load is the only mechanism available to resist the member sliding on the foundation. Friction cannot accommodate large deformations, nor can it absorb much energy. It is the responsibility of the TMS 402/602 committee to provide minimum requirements for the general structural integrity of masonry structural systems.

Examples of extreme events that the general structural integrity provisions are intended to address cited in the ASCE 7 commentary include:

- Explosions caused by ignition of gas or industrial liquids
- Boiler failures
- Vehicle impact
- Impact of falling objects
- Effects of adjacent excavations
- Gross construction errors
- Very high winds such as tornadoes
- Sabotage

In these events it is expected that the capacity of the structure will be exceeded. If, however, the structure is connected together it will have a chance to deform, remain intact and standing. A load path reliant on friction alone will not achieve this intent, even if calculation shows the adequacy of such a load path. As stated in the ASCE 7 commentary "because accidents, misuse and sabotage are normally unforeseeable events, they cannot be defined precisely." Since the events are by their nature undefined, they fall outside of our ability to demonstrate acceptable safety through calculations.

As suggested by the commenter, the concrete code provides an instructive example for the provision of structural integrity. ACI 318-19 does not permit the load path between the concrete structure and the foundation to be solely reliant on friction. Relevant to this discussion, Section 16.3 provides minimum connection requirements for general structural integrity at the structure to foundation. These provisions require minimum connections between concrete structural elements and the foundation even if calculations demonstrate that sufficient friction capacity is available to resist the code imposed loadings. The rationale for providing some minimal foundation dowels in areas of lower seismic demand is explained in the commentary to ACI 318-19 Section 16.3.4 which contains provisions for dowels or connections between concrete walls and foundations:

> The Code requires a minimum amount of reinforcement between all supported and supporting members to ensure ductile behavior. This reinforcement is required to provide a degree of structural integrity during the construction stage and during the life of the structure.

Similarly, the structural integrity provisions in ASCE 7, also require some minimum physical connections between structural components. For example, Section 1.4.4 in ASCE 7-16 for the anchorage of structural walls reads as follows:

> **1.4.4 Anchorage of Structural Walls.** Walls that provide vertical load bearing or lateral shear resistance for a portion of the structure shall be anchored to the roof and all floors and members that provide support for the for the wall or that are supported by the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resting a strength level horizontal force perpendicular to the plane of the wall equal to 0.2 times the weight of the wall tributary to the connection, but not less than 5 psf.

Note this provisions uses words like "anchorage" and "connection" which would preclude the use of friction provide a continuous load path for integrity.

<u>Development of this Ballot</u>

The balance of this rationale is broken into four sections:

- Minimum area of dowels: This section discusses the rationale behind the minimum area of steel crossing the interface between the masonry elements and the foundation.

- Embedment of dowels into the foundation: The section discusses the rationale for requiring the development of the dowels into the foundation and demonstrates that this should not affect the depth of typical wall foundations.
- Splicing of dowels with vertical reinforcement: This section discusses the rationale for requiring the dowels to be spliced with the vertical wall reinforcement.
- Future business: The section discusses issues identified in the development of this ballot that should be addressed during the next code development cycle.

*Minimum Area of Dowels*

In Seismic Design Categories D and higher, it is proposed to require that the wall dowels match the grade, size and spacing of the vertical wall reinforcement provided at the base of the wall. The reason for this is that that the seismic design forces in these seismic design categories are determined assuming that the lateral force resisting system can achieve a significant of inelastic behavior through yielding of the reinforcement. If the dowels do not match the wall reinforcement at the base of the wall, the inelastic behavior will be concentrated at the interface which may result in tensile rupture of the reinforcement rather than ductile yielding.

An exception is provided if the interface is evaluated for tension using forces determined with an R value not greater than 1.5. The rationale for the exception is that if there is no tension at the interface under essentially elastic loading that the inelastic demands at the interface will be quite limited.

The rationale for dowels for participating columns in high seismic regions is essentially the same. In accordance with Section 7.4.3.2.4, participating columns are required to be designed for R not greater than 1.5. As a result, it is not necessary to provide an exception.

In lower seismic design categories, there are two proposed minimum requirements for the dowels:

- That they equal or exceed the prescriptive reinforcement requirements for the masonry element, and
- That they be at least 25 percent of the vertical reinforcement provided at mid-height.

The first requirement follows the spirit of ACI 318 Section 16.3.4.2 that requires that area of reinforcement crossing the interface between the wall and foundation satisfy the minimum prescriptive wall reinforcement. For Grade 60 reinforcement, #5 or smaller, the minimum reinforcement ratio required for concrete walls is 0.0012. For an 8” concrete wall, this equates to roughly #4 @ 21”, or more than twice as many minimum dowels as would be required for a special reinforced masonry shear wall (#4 @ 48”) of any thickness.

Given the minimal amount of dowels mandated by the first requirement, the second requirement establishes a minimum dowel requirement that is proportionate to the design of the wall itself. The idea is that if the wall requires more than the minimum prescriptive reinforcement to resist the anticipated demands, the minimum integrity reinforcement should be proportionately increased. The 25% ratio was selected for consistency with Section 6.1.10.2 of TMS 402.

Code

**6.1.10** *Embedment of flexural reinforcement*

. . .

**6.1.10.2** *Development of positive moment reinforcement* — When a wall or other flexural member is part of the lateral-force-resisting system, at least 25 percent of the positive moment reinforcement shall extend into the support and be anchored to develop the yield strength of the reinforcement in tension.

Commentary

**6.1.10.2** *Development of positive moment reinforcement* — When a flexural member is part of the lateral-force-resisting system, loads greater than those anticipated in design may cause reversal of moment at supports. As a consequence, some positive reinforcement is required to be anchored into the support. This anchorage assures ductility of response in the event of serious overstress, such as from blast or earthquake. The use of more reinforcement at lower stresses is not sufficient. The full anchorage requirement need not be satisfied for reinforcement exceeding 25 percent of the total that is provided at the support.

One could argue that 6.1.10.2 already requires that 25% of the wall reinforcement be embedded into the foundation. Given the responses to previous dowel ballots, however, it seems that this is not a widely shared interpretation so making it apply explicitly to foundation supports seems appropriate. It is also unclear as to what reinforcement should be considered as "positive" in a wall; this ballot does not distinguish between positive and negative reinforcement.

In ACI 318, a parallel provision is found in Section 7.7.3.8.1 which requires that one third of the bottom reinforcement in one-way slabs be extended into the support. There are similar requirements at simple supports of concrete beams.

It should be noted that Section 6.1.10.2 seems to be in conflict with Section 6.1.10.1.3:

Code

**6.1.10.1.3** Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member or $12d_b$ , whichever is greater, except at supports of simple spans and at the free end of cantilevers.

This provision matches similar language in ACI 318 for one-way slabs and beams, but notably ACI 318 follows up this language with requirements for what should happen at simple supports. For example, for one-way slabs ACI 318 states:

**7.7.3.8.1** At simple supports, at least one-third of the maximum positive moment reinforcement shall extend along the slab bottom and into the support, except for precast slabs where such reinforcement shall extend at least to the center of the bearing length.

The language of Section 6.1.10.2 seems similar to that of ACI 318 for other than simple supports. For example, for one-way slabs ACI 318 states:

**7.7.3.8.2** At other supports, at least one-fourth of the maximum positive moment reinforcement shall extend along the slab bottom into the support at least 6 in.

Based on ACI 318, one could argue that providing dowels equal to one-third of the wall reinforcement would be more appropriate than the 25% that is being proposed. As is discussed in the next section, however, it is proposed to fully develop the dowels into the foundation; this improved anchorage justifies the use of 25% of the reinforcement rather than one-third.

*Embedment of Dowels into Foundation*

The provisions propose that all foundation dowels be developed for the yield strength into the foundation.

In high seismic regions, it is necessary that the dowels be developed for their yield strength in order to ensure that the intended ductile behavior is achieved. ACI 318 also requires development for yield strength for reinforcement resisting seismic loads in the seismic-force-resisting system in these seismic design categories.

In lower seismic regions, the first observation that should be made is that the size of the dowels is expected to be small. This expectation is based on the two limits proposed for minimum dowels. If the requirement to meet the minimum prescriptive reinforcement controls, then the largest dowel size required is #4. If the 25% limit controls, and a dowel is provided to match each vertical reinforcing bar, a #5 dowel can match a vertical bar as large as #9.

Given the small expected size of the dowels, a hooked dowel can be developed for its yield strength into a foundation that is 10 to 12" thick. Development for the yield strength is consistent with Section 6.1.10.2, ensures the intended integrity is achieved, and supports the rationale for only matching 25% of the wall reinforcement rather than one third. While there are other precedents for embedding dowels for less than their yield strength (ACI, for example, just requires that the bars cross the interface – no minimum embedment is provided), such vague guidance is not helpful to the user, is not enforceable, and, taken to the extreme, could result in the dowels being completely ineffective. Fully developing the dowels provides a connection that will meet the criteria suggested by ASCE 7 that the connection should have the "capacity for relatively large deformations and energy absorption."

While development of reinforcement into foundation concrete is the purview of ACI 318, several commenters on past ballots have expressed concerns about how dowel requirements in TMS 402 might affect the foundation design, so it worth discussing ACI 318-19 development length requirements in more detail. Specifically, the following items are noted:

- New in ACI 318-19, hooked bars must always be developed for the yield strength of the bar. This means that regardless of what TMS 402 requires, ACI 318 only provides a full development length. In addition, ACI 318-19 revises the equation for hooked development length. It appears that this equation results in slightly reduced embedment depths for masonry foundation dowels. The following table summarizes the hooked development length for typical foundation conditions:

| | **Hooked development length (in.)** | | | | | | |
|---|---|---|---|---|---|---|---|
| **f'c** | #3 | #4 | #5 | #6 | #7 | #8 | #9 |
| **3000** | 6.0 | 6.0 | 7.9 | 10.3 | 13.0 | 15.9 | 19.0 |
| **4000** | 6.0 | 6.0 | 7.4 | 9.7 | 12.2 | 14.9 | 17.8 |
| **5000** | 6.0 | 6.0 | 7.1 | 9.4 | 11.8 | 14.4 | 17.2 |

  Note that #5 bars and smaller can be fully developed in a 12" thick foundation with a hook and will achieve the 3" required cover.

- For straight bars, ACI 318-19 allows the development length to be reduced if more steel is provided than is required to resist the imposed demands. By requiring the development to be based on the yield strength of the reinforcement, we are preventing users from taking advantage of that provision. The resulting development lengths are, however, still modest for the expected size of the minimum dowels:

| | **Straight development length (in.)** | | | | | | |
|---|---|---|---|---|---|---|---|
| **f'c** | #3 | #4 | #5 | #6 | #7 | #8 | #9 |
| **3000** | 12.0 | 13.1 | 16.4 | 19.7 | 28.8 | 32.9 | 37.0 |
| **4000** | 12.0 | 12.0 | 14.2 | 17.1 | 24.9 | 28.5 | 32.0 |
| **5000** | 12.0 | 12.0 | 12.7 | 15.3 | 22.3 | 25.5 | 28.6 |

  Note #5 bars and smaller can be fully developed with a straight development length in foundations 15" to 20" thick.

*Splicing of Dowels with Vertical Reinforcement*

The provisions propose that all foundation dowels be spliced with the vertical wall reinforcement. These splices could be lap splices, mechanical splices, or welded splices, subject to the existing limitations of TMS 402.

In high seismic regions, it is necessary that the dowels be spliced with the vertical wall reinforcement in order to ensure that the intended ductile behavior is achieved.

As noted above, in lower seismic regions the required dowel size is expected to not exceed a #5 unless force transfer at the interface requires a larger bar. Splice lengths for these bars are not onerous and will ensure that the intended integrity is achieved.

*Future Business*

In preparing this ballot, a number of items were noted that may warrant consideration by the committee for the next cycle. These include the following:

- Mandating minimum dowels at all structural walls, even those not requiring prescriptive reinforcement, consistent with the ASCE 7 requirements to provide minimum anchorage or connection between structural masonry walls and supporting members.
- To achieve general structural integrity, should all structural masonry walls be required to have prescriptive reinforcement?
- Should there be a minimum dowel / anchorage / connection requirement for non-participating walls, either in high seismic regions or in all regions?
- How should out-of-plane force transfer be evaluated at the interface of walls and foundations? Can the shear friction provisions be used? In some cases, would it be preferable to a have a shear dowel provision which did not require that the tensile yield strength of the dowels be developed?
- Should requirements for post-installed foundation dowels be addressed? (While this is a concrete code issue, we could consider providing a minimum force per foot as alternate, like ACI requires 3000 lbs/foot vertical integrity ties in precast walls.)
- Should the provision be expanded to address other types of concrete supports such as concrete stem walls, concrete floor systems, and thickened slabs-on-grade?
- Are there interfaces between masonry elements and other materials that should be addressed to ensure that there are no gaps between TMS 402 and other codes?
- Generally review the code for current practice and peer codes with respect to integrity and resiliency.
- Clarify the intent of 6.1.10.2 – how is positive moment reinforcement intended to be understood in a wall?
- Resolve the apparent conflict between 6.1.10.1.3 and 6.1.10.2.
- Address effect of foundation restraint on out-of-plane wall design. Should we add provisions or commentary to Chapter 4 to address the effect of foundation restraint on out-of-plane wall design?

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)* *Do not use 'Track Changes'*

**Code:**

**6.1.10.2** *Development of positive moment reinforcement* — When a wall or other flexural member is part of the lateral-force-resisting system, at least 25 percent of the positive moment reinforcement shall extend into the support and be anchored to develop the yield strength of the reinforcement in tension.

*. . .*

**7.4.1** *Seismic Design Category A requirements*

*. . .*

**7.4.1.1** *Design of nonparticipating elements*

. . .

**7.4.1.2** Design *of participating elements* — Participating masonry elements shall be designed to comply with the requirements of Chapter 8, 9, 10, 11, or 12. Masonry shear walls shall be designed to comply with the requirements of Section 7.3.2.1, 7.3.2.2, 7.3.2.3, 7.3.2.4, 7.3.2.5, 7.3.2.6, 7.3.2.7, 7.3.2.8, 7.3.2.9, 7.3.2.10, or 7.3.2.11.

**7.4.1.2.1** *Foundation dowels* – Dowels crossing the interface between the masonry and the supporting foundation shall be provided for masonry elements that are required to have minimum prescriptive vertical reinforcement. The provided area of dowels shall equal or exceed the greater of the following:
(a) The area required for force transfer at the foundation interface,
(b) The area required to meet the prescriptive vertical reinforcement requirements for the masonry element,
(c) 25 percent of the area of the vertical reinforcement provided at mid-height at the level under consideration.
The dowels shall be developed into the foundation for their yield strength and shall be spliced with the vertical reinforcement in the masonry element. Where the dowels are a smaller size than the vertical reinforcement, the splice requirements may be determined based on the size of the dowel.

***Renumber subsequent sections***

**7.4.4** *Seismic Design Category D requirements*

. . .

**7.4.4.1** *Design of nonparticipating elements*

. . .

**7.4.4.2** *Design of participating elements*

. . .

**7.4.4.2.1** *Foundation dowels -* Dowels crossing the interface between the participating masonry elements and the foundations shall meet the following requirements:

(a) Dowels shall be provided matching the grade, size and spacing of the vertical wall reinforcement at the base of the wall. The dowels shall be developed into the foundation for their yield strength and shall be spliced with the vertical wall reinforcement.
Exception: Compliance with this provision is not required if there is no tension at the wall to foundation interface when in-plane forces at the interface are evaluated using R not greater than 1.5.

(b) Dowels matching the grade, size, and quantity of the vertical column reinforcement shall be provided for participating columns designed assuming a fixed-end condition at the foundation. The dowels shall be developed into the foundation for their yield strength and shall be spliced with the column vertical reinforcement.

***Renumber subsequent sections***

**Code Commentary:**

**6.1.10.2** *Development of positive moment reinforcement* — When a flexural member is part of the lateral-force-resisting system, loads greater than those anticipated in design may cause reversal of moment at supports. As a consequence, some positive reinforcement is required to be anchored into the support. This anchorage assures ductility of response in the event of serious overstress, such as from blast or earthquake. The use of more reinforcement at lower stresses is not sufficient. The full anchorage requirement need not be satisfied for reinforcement exceeding 25 percent of the total that is provided at the support.

*. . .*
**7.4.1.2.1** *Foundation dowels* – The rationale for this provision is discussed in the commentary to Section 6.1.10.2. Elements such as ordinary plain masonry shear walls that do not have prescriptive reinforcement requirements are not required by this Code to have dowels, as long as the design does not require dowels for force transfer at the foundation interface and the design does not require vertical reinforcement in the masonry element. There may be requirements of other applicable codes, such as the structural integrity provisions of ASCE 7 that mandate the use of dowels or other positive connections between structural masonry walls and foundations.

Where the dowels are a smaller size than the vertical reinforcement, it is permitted to base the splice length on the size of the dowel as this will fully develop the capacity of the dowel at the foundation while maintaining continuity of reinforcement through the splice.

Unless the foundation is proportioned to restrain out-of-plane rotation, most foundations can accommodate sufficient rotation to approximate a pinned support. The presence of dowels does not necessitate treating conditions that would otherwise be approximated as pinned as having a degree of fixity.

Some contractors may find it desirable to provide additional dowels for improved safety or reduced external bracing of the wall during construction. Such additional dowels are generally not detrimental to wall performance and also improve resiliency.

*. . .*
**7.4.4.2.1** *Foundation dowels* – The wall to foundation interface is one of the first places where inelastic behavior is expected to be experienced in a seismic event. Yielding of reinforcement at this interface is a key contributor to achieving the higher R values required in Seismic Design Category C and higher. If, however, there is no tension at the interface assuming a nearly elastic R value, the behavior will not rely on yielding and compliance with this provision is not required.

See commentary to Section 7.4.1.2.1 for discussion of out-of-plane restraint.

If additional dowels are proposed to be provided by the contractor as discussed in the commentary to Section 7.4.1.2.1, the increased flexural capacity may affect the shear capacity design of special reinforced masonry shear walls and mechanism limit states determined using the Appendix C provisions.

**Specification:**

None.

**Specification Commentary:**

None.

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 7 | *Affirmative* | 1 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 12 | *Did not vote* |

**Subcommittee Comments:**

Robinson commented:

> *I think this is probably a good start. I would prefer we addressed some of the issues you outlined in future business, specifically non-participating elements. I do not think we should be treating participating and non-participating elements differently when we consider doweling to foundations.*

*Some other minor comments:*

*1. Section 7.4.3.2.1 should be Section 7.4.4.2.1*

*2. Section 7.4.4.2.1(a) the phrase "... for participating walls..." is redundant as the charging language in 7.4.4.2.1 already says it is for participating masonry elements.*

*3. Section 7.4.4.2.1(a) what is the intent of the exception? Does that mean that no doweling is required or should doweling per 7.4.1.1.1 be required when there is no tension?*

*4. Commentary 7.4.1.2.1 there is a "to" missing between "this Code" and "have dowels" in the third line.*

*5. Commentary 7.4.1.2.1 third paragraph might be clearer if it was specified that it is out-of-plane rotation that is no restrained. The foundation will restrain rotation for in-plane loads.*

*Also, there are a few grammatical errors in the rationale.*

The subcommittee chair offers the following responses.

In developing the ballot, the decision was made that to focus on participating walls based on the assumption that the integrity of participating walls is more important to maintaining general structural integrity than the integrity of the nonparticipating walls.

1. Agreed - the suggested correction has been made.
2. Agreed - the suggested correction has been made.
3. The seismic design category provisions are additive, so the exception on alleviates the need to comply with 7.4.4.2.1 (a), not 7.4.1.1.1. Dowels complying with 7.4.1.1.1 would still be required.
4. Agreed - the suggested correction has been made.
5. Agreed – the phrase "out-of-plane" has been added to the commentary as proposed.

The text was reviewed for grammatical errors and those found have been corrected.

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| **Committee:** Main Committee | | | | **Ballot #:** 2 | |
|---|---|---|---|---|---|
| **Item #:** 2-SM-002 | | | | | |
| **Technical Contact:** | Fernando Fonseca, fonseca@byu.edu, 801-422-6329 | | | | |
| **Draft Document Dated:** | **3/2/2017** | | | | |
| **Reballot of Main Committee Item No.:** | N.A. | **Response to TAC Comment No.:** | N.A. | **Response to Public Comment No.:** | 2016-73 |

| Reference | Section/Article |
|---|---|
| **TMS 602 Specification Article** | 3.4.B.10 |

**Rationale:** *(Rationale is explanatory and not part of the proposed revision)*
2016 Public Comment 73 stated:

> For continuity and a positive connection between masonry walls and supporting foundations, reinforcing dowels should be provided at the base of reinforced walls, lapping with each vertical bar. The reinforcing dowels would enhance structural redundancy providing an alternate load path, added strength and stiffness. These enhancements each contribute to improving structural safety during abnormal loading events. There seems to be a growing awareness in this country in designing and constructing sustainable and resilient buildings, infrastructure and communities that are safe, secure and able to withstand and recover from natural and man-made disasters.
>
> Proposed Specification (add a new article and re-number subsequent articles)
> 3.4 B. 10. At each vertical reinforcing bar, provide a reinforcing dowel of the same size embedded into the foundation a minimum of 12 in. (305 mm), and projecting into the masonry a full lap splice length, unless otherwise required.
>
> Specification Commentary
> 3.4 B.10 Reinforcing dowels should be provided, except when calculations indicate that providing such dowels would adversely affect masonry performance.
>
> Optional Requirements Checklist
> Add a line item to the Optional Requirements Checklist as follows:
> 3.4 B.10 - Reinforcing dowels - Specify when reinforcing dowels should not be installed.

This ballot item proposes the following response to the referenced 2016 Public Comment.

The subcommittee appreciates your comment and we thank you for submitting it for consideration. Structural continuity requirements are a design topic and the code, not the specification, should address those requirements. In fact, code Section 4.1.1 states that a continuous load path or paths, with adequate strength and stiffness, shall be provided to transfer forces from the point of application to the final point of resistance. Thus, the designer determines what is needed for a load path to satisfy the continuity requirements of the code. Therefore, adding structural continuity provisions to TMS 602 is neither appropriate nor necessary.

An affirmative vote on this ballot item indicates agreement with the proposed response to the referenced 2016 Public Comment.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**Code Commentary:**

**Specification:**

**Specification Commentary:**

**Mandatory Requirements Checklist:**

**Optional Requirements Checklist:**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 4 *Affirmative* | 4 *Affirmative w/ comment* | -- *Negative* | -- *Abstain* | -- *Did not vote* |

**Subcommittee Comments:**

1. I think the last word of the response is supposed to be "appropriate"."
2. Please change last word in last sentence as follows: "Consequently, changes to TMS 602 are not appropriate."
3. In the last line of page 1, change "appropriated" to "appropriate."
4. I thought I would rephrase our response a little. We can use all/none or parts of it, it's up to the group. "The subcommittee appreciates your comment and we thank you for submitting it for consideration. Structural continuity requirements are a design topic contained in the code while the specification controls materials, labor, and construction. Code Section 4.1.1 states that a continuous load path or paths, with adequate strength and stiffness, shall be provided to transfer forces from the point of application to the final point of resistance. The designer determines the requirements for a continuous load path to meet the provisions of the code. Therefore, adding continuity provisions to TMS 602 are not necessary.")

The item has been corrected and modified as suggested by comments 1, 2, 3, and 4.

| Item Number | Comment Type | Commenter | Comment | Comment File |
|---|---|---|---|---|
| | | | | |
| 02-SM-002 | Comment Non-Voting | Mr. Daniel Zechmeister dan@masonryinfo.org | I agree with the public comment. The 2015 IBC requires tornado shelters for type E facilities in several states. A tornado is not biased towards any type of building. From recent tornado events we saw one of the big box stores lose their light weight roof and left the walls unsupported at the top. With a pinned base these walls collapsed. According to an article in Structures magazine, July 2012, Lessons Learned from the Joplin Tornado;<br><br>"...In consideration of the magnitude of devastation to the built environment, the Structural Engineers Association of Kansas & Missouri (SEAKM), a Member Organization of NCSEA, formed a committee to investigate the performance of | |

| Item Number | Comment Type | Commenter | Comment | Comment File |
|---|---|---|---|---|
| | | | strucutres affected by the tornado, whether directly or indirectly. This article offers some of the committee's observations and recommendations, which are based on site reconnaissance and other information...The intent of the following recommendations is to increase life safety for occupants and overall building integrity and robustness when impacted by tornado type winds. However, it should be understood that a structure built in accordance with them will not be "tornado proof"...<br><br>5) Develop code requirements for greater robustness or redundancy in hard wall buildings. These may be in the form of specifying: a defined base moment; a maximum length of continuous wall prior to a full-height lateral-load resisting member, wall or frame; or a system of cross-ties.<br><br>One of the buildings impacted by the Joplin Tornado experienced a near-total collapse of the tilt-up wall panel system except at the loading dock area, where the base of the panel was well below grade such that it behaved as a cantilever. Details could be designed and provided that would offer a fixed or partially restrained base condition..."<br><br>I beleive the committee should reconsider public comment 73, which states; "For continuity and a positive connection between masonry walls and supporting foundations, reinforcing dowels should be provided at the base of reinforced walls, lapping with each vertical bar. The reinforcing dowels would enhance structural redundancy providing an alternate load path, added strength and stiffness. These enhancements each contribute to | |

| Item Number | Comment Type | Commenter | Comment | Comment File |
|---|---|---|---|---|
| | | | improving structural safety during abnormal loading events..." | |
| | Negative | Mr. Todd A. Dailey<br>todddailey@me.com | Construction safety: Walls with foundation dowels that match the wall design reinforcement (rebar size and spacing) <u>and</u> that project sufficiently to achieve a full lap splice are over 200% stronger (in terms of resistance to wind during construction) compared to walls that do not have such dowels. As the amount of reinforcement in a wall increases, the strength increase will likely be in excess of 300%.<br><br>Walls without such dowels typically behave as a unreinforced free-standing cantilevered walls (up until the point that any external braces are installed). When walls in this mode are subjected to even moderate wind speeds, they can fail in a brittle and sudden fashion.<br><br>Additionally, this dowel requirement will allow the full potential for "internal bracing", which provides our industry a unique opportunity to both improve construction safety <u>and</u> significantly lower wall costs; leading to masonry becoming more cost effective and thus gain market share. Masonry is the only major building material that is capable of self-support, even to extreme wall heights. I believe this fact deserves more promotion.<br><br>This is a "means and methods" topic, and thus some will likely automatically object on this basis to its inclusion in the TMS 402/602 document. In a general sense, I agree that means and methods should not typically be addressed in design codes. However there are areas where exceptions should be made; and I believe this life safety related issue is one of them. | Dailey02-SM-002.doc |

| Item Number | Comment Type | Commenter | Comment | Comment File |
|---|---|---|---|---|
| | | | Other industries have already made very similar exceptions in their codes (such as the steel industry where designers are instructed to design structural steel columns with a degree of base fixity during construction to reduce collapse potential during erection). We already have "means and methods" content in the TMS 402/602 (such as grouting techniques); we have just gotten so used to it being included that we don't make a conscious connection to its nature. | |
| | | Ms. Rochelle C. Jaffe<br>jaffeconsulting@gmail.com | I don't think that we should be so quick to dismiss this concept. In addition to adding redundancy and resiliency, there are other benefits to reinforcing dowels.<br><br>Reinforcing dowels enhance structural capacity and benefit contractor safety during construction. Because the dowels provide partial fixity at the base, the reinforced masonry walls are more stable during construction and may not need external bracing. Other industries are recognizing that the design professional has some responsibility for designing-in safety during erection. Take for example, the steel industry. In the AISC Steel Construction Manual, the General Design Requirements include a requirement that design and detailing address aspects of the OSHA safety requirements (reference 13th edition, page 2-6). The most notable aspect is the requirement for four anchor bolts in all column base plates for stability during erection. If the masonry design community wants to be taken seriously, perhaps we should step up to that sort of consideration also.<br><br>Reinforcing dowels would also make the structure eligible for LEED points via Prevention through | Jaffe_R_C_2-SM-002.doc |

| Item Number | Comment Type | Commenter | Comment | Comment File |
|---|---|---|---|---|
| | | | Design (PtD) practices.<br><br>“PtD is an approach based on research and practice demonstrating that upstream design and planning decisions can influence and improve safety for construction workers and end users across the life cycle of a building or structure. . . . In building design and construction, the focus is on the construction process—both conventional and LEED-related topics. The pilot credit describes a cross-disciplinary “safety constructability review” to perform discovery and implementation and provides a list of topics to consider . . .”<br><br>http://www.usgbc.org/articles/new-leed-pilot-credit-prevention-through-design<br><br>Furthermore, TMS 602 Article 3.1 A.2 requires dowels to be correctly positioned. If the Code does not require dowels, then there is a conflict between Code and Specification. Consequently, it seems that some information about requirements for foundation dowels is needed.<br><br>If the Committee is not ready to make the change to requiring reinforcement dowels in the Code, then there should at least be a Commentary discussion that describes the potential benefits and why they may or may not be required by the design professional as well as why the contractor may want them. Regardless, a line item in the Optional Requirements Checklist should be added to specify when reinforcing dowels are required (or permitted?). | |

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| Committee: Main Committee | Ballot #: 14 |
|---|---|
| **Item #:** 14-SM-008 | |

| | |
|---|---|
| **Technical Contact/Email:** | Todd Dailey, todddailey@me.com, 517/467-9000 |
| **Draft Document Dated:** | **11/29/2019** |

| | | | | | |
|---|---|---|---|---|---|
| **Reballot of Main Committee Item No.:** | N/A | **Response to TAC Comment No.:** | N/A | **Response to Public Comment No.:** | N/A |

| **Reference** *(Choose from Drop-Down Menu)* | **Section/Article** |
|---|---|
| **TMS 402 Code Section** | 5.1.5 |

**Rationale:** *(Rationale is explanatory and not part of the proposed revision)*

Masonry must be designed to resist applicable loads. A continuous load path or paths, with adequate strength and stiffness, must be provided to transfer forces from the point of application to the final point of resistance. Thus, a new Code requirement for a foundation dowel is being proposed.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**5.1.5** *Foundation Dowels for Reinforced Masonry*
Foundation dowels of the same grade, size and spacing of the wall, column, or pilaster reinforcement are required unless specifically designed otherwise.

**Specification:** None

**Specification Commentary:** None

**Mandatory Requirements Checklist:** None

**Optional Requirements Checklist:** None

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 8 *Affirmative* | 0 *Affirmative w/ comment* | 1 *Negative* | 0 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:** The original subcommittee ballot was modified to address part of the negative vote, which was: the change only addresses reinforced masonry walls, what if the walls are unreinforced, are calculations required to show that dowels are not required for this condition?
The second part of the negative vote, which was "Are dowel lengths required to match the bar size development length into the footing and into the wall, should there be commentary to address this?" will be addressed in a subsequent ballot.

| | | | | | |
|---|---|---|---|---|---|
| 14-SM-008 | Negative | Dr. Andres Lepage<br>alepage@ku.edu | Without specifying an embedment length, the proposed change is incomplete. The embedment length should not be addressed in a subsequent letter ballot. | | 1 |
| | | Dr. Richard M. Bennett<br>rmbennett@utk.edu | The rationale provides no basis for the proposed provision. There are a hundred things that are required for providing a continuous load path or paths, with adequate strength and stiffness. What is the purpose of signaling out foundation dowels? Is there some other ulterior motive for this provision? | | 1 |
| | | Dr. William Mark McGinley<br>m.mcginley@louisville.edu | I have no problem requiring dowels at the base of reinforced masonry walls. I see no justification that they be the same size as the wall bars designed for peak flexural loadings. Often the base of the wall is designed as a pinned connection and would not need that many bars (if any). This provision also will likely force designers to lap the bars as well. | | 1 |
| | | Mr. John M. Hochwalt<br>johnh@kpff.com | For this provision to be enforceable, the proposed provision needs to address the required embedment of these dowels. Must the dowels be developed into the foundation? If not, what embedment is required?<br><br>For some context, ACI 318-19 Sections 16.3 and 18.13 address the anchorage of concrete elements to foundations and might provide a model to base TMS 402 provisions on.<br><br>The provision should also address whether these dowels are required to be spliced with the wall / column / pilaster reinforcement.<br><br>Lastly, some commentary should be provided to give some guidance on what one should consider if they decide to provide an engineered design of the foundation interface. That might include a reference to the shear friction | | 1 |

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| | | | provisions that mandate that any reinforcing required to resist sliding be fully developed. | | |
| | | Mr. Scott W. Walkowicz<br>scott@walkowiczce.com | I'm not convinced that this is a good thing to do as it may result in larger foundations being designed for masonry walls and particularly those that receive out of plane loads that would now be considered to have out of plane base fixity. The implication is that the dowels would be fully developed, or lapped with the member reinforcement thus precluding a 'pinned' base condition unless the designer specifically designs it that way or in some manner other than fully fixed. Some walls may have uplift requirements that will require full laps, some walls may rely on shear friction generated by fully lapped dowels and those conditions should be identified by the designer. It is my opinion that many walls are still designed that do not require a full lap splice at the base of the wall and that doweling may be done with shorter than development length laps for shear keying but not developing shear friction beyond that generated by the axial force at the base of the wall. I suggest reversing the Code application/intent to something along the lines of 'when axial tension, in-plane or out-of-plane flexural tension or shear friction requirements require reinforcement continuity between the masonry member and the foundation, then...' maybe that is more confusing but I'm concerned about making this the default approach when it may add to the foundation cost associated with masonry construction. Also, I understand that this may have a similar impact as the steel column design/detailing requirement when 4 bolt base plates became required... maybe there is empirical evidence that could be used and supplied to provide confidence that masonry foundations will not become more expensive if this were implemented. | | 1 |

| | | | | | |
|---|---|---|---|---|---|
| | | | If the Committee proceeds with this change, then I suggest that the laps be referenced to the appropriate Code section for clarity and that Commentary be added to discuss the base fixity consideration and to provide guidance to users as to when such fixity should be considered and when it may be neglected. | | |
| | | Ms. Diane B. Throop<br>diane@dtpeconsulting.com | 1.) I agree the load path must be established into the foundation. However, I think requiring the same grade, size, and spacing of the bars is too restrictive in many applications. It is common practice to size a vertical bar in a wall or column, etc based on the requirements at the maximum load. This is often not at the base of the wall. There is no need to extend all the bars into the foundations. This is just one example and I am sure there are others based on seisimic or wind requirements.<br><br>2.) Would ths requirement be applicable to partition walls?<br><br>3.) How would this requirement be applied to unreinforced masonry?<br><br>4.) In addition, while the ballot items says that things like length of dowel, etc will be addressed in future ballots, we cannot move forward with the language proposed without those details. Those propovisions are part-and-parcle of being able to design and construct to this proposed provisions so until it is complete it cannot move forward into the Code.<br><br>5.) Further, if included in the Code, companion requirements must be added to eh Specificiation in the same ballot | | 1 |

| | | | | | |
|---|---|---|---|---|---|
| | | | item. It cannot be in the Code without including companion requirements in the Specification. | | |
| | | Ms. Rochelle C. Jaffe jaffeconsulting@gmail.com | see attached | Jaffe_R_C_14-SM-008_N.doc | 1 |
| Totals | | | | | 110 |

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| Committee: Main Committee | | | | Ballot #: 15B | |
|---|---|---|---|---|---|
| Item #: 15B-SM-008 | | | | | |
| Technical Contact/Email: | Todd Dailey, todddailey@me.com, 517/467-9000 | | | | |
| Draft Document Dated: | 5/6/2020 | | | | |
| Reballot of Main Committee Item No.: | 14-SM-008 | Response to TAC Comment No.: | N/A | Response to Public Comment No.: | N/A |

| Reference *(Choose from Drop-Down Menu)* | Section/Article |
|---|---|
| TMS 402 Code Section | Section 5.1.5 |
| TMS 402 Commentary Section | Section 5.1.5 |
| TMS 602 Specification Article | |
| Section | |

**Rationale:** *(Rationale is explanatory and not part of the proposed revision)*

**General Rationale:**
There are two benefits that would result from adding minimum reinforcement provisions for connections between masonry and concrete foundations:

1) Increased structural integrity over the life of the building (greater resiliency and to promote ductile behavior).
2) To provide a degree of structural integrity and safety during construction.

Based on the comments and negative votes received on the previous Main Committee ballot and SM subcommittee ballot, this ballot proposal is increased in content (including commentary) to more fully address the comments and negative votes, including the following:

- Size and spacing of connection reinforcement
- Embedment requirements into the concrete foundation
- Lap splice requirements into the masonry element
- Alternate provisions for when connection reinforcement may be undesirable

This proposal is more closely patterned after the ACI 318 minimum reinforcement for connections between cast-in-place members and foundation requirements (see attachment #1), while also considering the differences between concrete and masonry.

**Specific Rationale** (Presented on a section-by-section basis of the proposed code and commentary changes):

1) **Proposed Code Change: "5.1.5.1** The connection between a reinforced masonry wall, column, or pilaster and a concrete foundation shall be designed with the vertical reinforcement crossing the interface. The minimum vertical reinforcement shall be of the same grade, size and spacing as that in the masonry member, except as permitted in section 5.1.5.3. "

   **Proposed Commentary Change: "5.1.5.1** The Code requires a minimum amount of reinforcement between structural masonry members and their foundation to promote ductile behavior. This reinforcement is required to provide a degree of structural integrity, and a load path during the construction stage and during the life of the structure."

This is a near-verbatim match to ACI 318 section 16.3.4 (see attachment #2)

There are two good reasons to add "foundation connection requirements" to the Code:
Increased robustness for buildings <u>and</u> improved jobsite safety. Either one of those reasons is reason enough on its own to warrant this Code change. Construction safety has sometimes been regarded as "off-limits" to TMS 402/602. However in both the steel (AISC) and concrete (ACI) industries, their Codes have called for some degree of base fixity during construction, which has benefited their respective industries. The steel industry has done this with the requirement for "4 bolt baseplates" (even for columns designed with a pinned base); while the concrete industry has done so with the minimum reinforcement requirements given in ACI 318 Chapter 16 Connections). It is time for masonry to move forward on this topic.

ACI 318 provisions for the amount of connection reinforcement required is based on a steel ratio. If the ACI provisions were correlated to masonry, it would equate approximately to #5's at 40" on center for an 8" CMU wall, and 5's at 24" on center for a 12" CMU wall. A minimum "steel ratio" is not the best route for masonry due to issues with masonry modularity and energy code compliance. Most design firms have already implemented "match size and spacing" callouts into their typical masonry designs.

<u>Economic impact</u>: This provision actually has the potential to significantly lower overall masonry costs. To illustrate, in the steel industry, designing to "least weight" is not recommended for economy if it triggers stiffeners, doubler plates, etc. The extra cost of labor rapidly offsets and exceeds the any material cost savings. In masonry, providing foundation dowels of the same size and spacing as the wall reinforcement will provide a high likelihood that the wall can qualify as internally braced, and thereby eliminate the cost of external bracing. Cost estimates for typical pipe brace/deadman assemblies range around $500-600 each. The typical bracing design calls for two pipe braces per panel (which typically are around 24' maximum in length). That corresponds to an approximate potential savings of $40 to $50 per foot of wall, which will dwarf the additional cost of either a few more dowels, or slightly longer dowels. For independent opinions/evaluations of these cost estimates, I encourage contacting any mason contractor from Michigan, where internal bracing has become the dominant temporary bracing method. Most TMS members generally stay clear of involvement into contractor "means and methods". However it is not a bad thing to understand the impact of design on construction costs.

<u>Construction safety</u>: So far, Michigan is the only state that has adopted "The Standard Practice for Bracing Masonry Walls Under Construction" as part of an official masonry wall bracing safety standard. The Michigan MIOSHA director commented to the MIM Executive Director that since its adoption, there has been a noticeable decrease in the number of masonry wall collapses in the state. The economic impact above does not consider any injury avoidance costs.

2) **Proposed Code Change: "5.1.5.2** Connection reinforcement shall be provided either by extending vertical bars from the supporting concrete foundation into the masonry, or by anchored and projecting reinforcement dowels."

   **Proposed Commentary Change: "5.1.5.2** See ACI 318 for design requirements for embedment/development of steel reinforcement in concrete foundations."

There are conditions where masonry interfaces with another building material, for which the design provisions of that other material are outside the scope of the Code. An existing example of this is in Section 5.2 where defection criteria of non-masonry beams providing vertical support to masonry is established, but no other provisions on how those beams should be designed is included. A second existing example is Section 6.1.6.1.2 which states: "Welding shall conform to AWS D1.4/D1.4M" for welded splices of bar reinforcement."

In the subject matter of this ballot, the "other building material" is specifically identified as concrete; for which embedment of foundation dowels is properly governed by the concrete Code (ACI 318). By the way, ACI 318 is already referenced in the TMS 402 Code (Chapter 6).

3) **Proposed Code Change: "5.1.5.3** Connection reinforcement shall satisfy the lap splice requirements of Section 6.1.6."

This section calls for a full lap splice on the foundation connection reinforcement. There are concerns by some about the impact of resulting base fixity on the foundation (i.e., Does it require the foundation to be designed for moment, or can it still be modeled as a pinned base for the permanent design condition?). To address those concerns, consider the following:

1) If the designer feels strongly that full lap splices would lead to moment that must be included in the foundation design, and wants to avoid that situation, then section 5.1.5.4 provides a design approach to design the connection reinforcement (if any) as desired.
2) Structural engineers often assume pinned base behavior for various connections, even though many of these connections modeled as pins transfer some moment in real life. In general, a pinned base assumption is conservative. Many designers would consider it a valid design assumption to regard the base as pinned, even with a full lap splice. If any moment transfer of the completed building were to occur at the footing, and the footing could not accommodate it, it would rotate slightly, and the system would thereby effectively behave as a pin.
3) Moment transfer from masonry walls, columns, and pilasters to footings during the construction period is inevitable (regardless of the presence or length of any connection reinforcement). Virtually every masonry wall will spend at least some of its life as "internally braced", even those walls for which external braces are eventually installed; until the external braces are installed, the wall will act as a vertical cantilever (shafts are an exception). And even unreinforced walls can transfer moment into the footing via mortar flexural tension. So during construction, footings experience moment transfer from walls. During construction, the only real problem to be concerned about is if the overturning capacity of the footing is exceeded.
4) Based on my review of over 100 plans per year at the request of mason contractors for temporary wall bracing designs, most structural engineers require some sort of a positive connection to the foundation (i.e., a foundation dowel). "Match size and spacing of wall reinforcement" and application of the typical lap splice criteria to apply to the dowels also is far and away the most common design practice I see.

5) **Proposed Code Change: "5.1.5.4** Masonry without vertical reinforcement and masonry intended to have a pinned base shall be designed with an alternate mechanism to transfer the applicable forces to the foundation."

   **Proposed Commentary Change: "5.1.5.4** There are situations where the connection reinforcement required by Sections 5.1.5.1 through 5.1.5.3 could be detrimental and/or undesirable to the design. These situations include "unreinforced" masonry, where foundation dowels could add unwarranted costs; and where a pinned base is a necessary or desired aspect of the design. For an example, see NCMA TEK 5-5B (2011) for pinned base guidance for masonry walls used for pre-engineered metal buildings. For these situations, the designer is required to design a connection between masonry and the foundation that is adequate to resist design loads."

While there is widespread support of a foundation connection requirement that would apply to most walls, there is also a need to provide an "opt-out provision". This section is patterned after ACI 318 Section 16.3.5.3, where when a pinned connection is used, the connection design just needs to satisfy basic load transfer requirements.

**CODE** / **COMMENTARY**

**16.3.3.7** At the base of a precast column, pedestal, or wall, mechanical connectors shall be designed to reach their design strength before anchorage failure or failure of surrounding concrete.

**16.3.4** *Minimum reinforcement for connections between cast-in-place members and foundation*

**R16.3.4** *Minimum reinforcement for connections between cast-in-place members and foundation*—The Code requires a minimum amount of reinforcement between all supported and supporting members to ensure ductile behavior. This reinforcement is required to provide a degree of structural integrity during the construction stage and during the life of the structure.

**16.3.4.1** For connections between a cast-in-place column or pedestal and foundation, $A_s$ crossing the interface shall be at least $\mathbf{0.005}A_g$, where $A_g$ is the gross area of the supported member.

**R16.3.4.1** The minimum area of reinforcement at the base of a column may be provided by extending the longitudinal bars and anchoring them into the footing or by providing properly anchored dowels.

**16.3.4.2** For connections between a cast-in-place wall and foundation, area of vertical reinforcement crossing the interface shall satisfy 11.6.1.

"Basic" Requirement

**16.3.5** *Details for connections between cast-in-place members and foundation*

**R16.3.5** *Details for connections between cast-in-place members and foundation*

**16.3.5.1** At the base of a cast-in-place column, pedestal, or wall, reinforcement required to satisfy 16.3.3 and 16.3.4 shall be provided either by extending longitudinal bars into supporting foundation or by dowels.

16

**16.3.5.2** Where moments are transferred to the foundation, reinforcement, dowels, or mechanical connectors shall satisfy 10.7.5 for splices.

**R16.3.5.2** If calculated moments are transferred from the column to the footing, the concrete in the compression zone of the column may be stressed to $\mathbf{0.85}\phi f_c'$ under factored load conditions and, as a result, all the reinforcement will generally have to be anchored into the footing.

**16.3.5.3** If a pinned or rocker connection is used at the base of a cast-in-place column or pedestal, the connection to foundation shall satisfy 16.3.3.

"Opt out provision" of the basic requirements if a pinned connection is used. Interestingly does not include walls.

**16.3.5.4** At footings, it shall be permitted to lap splice No. 14 and No. 18 longitudinal bars, in compression only, with dowels to satisfy 16.3.3.1. Dowels shall satisfy (a) through (c):

(a) Dowels shall not be larger than No. 11

(b) Dowels shall extend into supported member at least the greater of the development length of the longitudinal bars in compression, $\ell_{dc}$, and the compression lap splice length of the dowels, $\ell_{sc}$

(c) Dowels shall extend into the footing at least $\ell_{dc}$ of the dowels

**R16.3.5.4** Compression lap splices of large bars and dowels are permitted in accordance with 25.5.5.3. Satisfying 16.3.3.1 might require that each No. 14 or 18 bar be spliced to more than one dowel bar.

**16.3.6** *Details for connections between precast members and foundation*

CODE

COMMENTARY

The minimum area of wall reinforcement for precast walls has been used for many years and is recommended by the Precast/Prestressed Concrete Institute (PCI MNL-120) and the Canadian Concrete Design Standard (2009). Reduced minimum reinforcement and greater spacings in 11.7.2.2 are allowed recognizing that precast wall panels have very little restraint at their edges during early stages of curing and develop less shrinkage stress than comparable cast-in-place walls.

**Table 11.6.1—Minimum reinforcement for walls with in-plane $V_u \leq 0.5\phi V_c$**

| Wall type | Type of nonprestressed reinforcement | Bar/wire size | $f_y$, psi | Minimum longitudinal[1], $\rho_\ell$ | Minimum transverse, $\rho_t$ |
|---|---|---|---|---|---|
| Cast-in-place | Deformed bars | ≤ No. 5 | ≥ 60,000 | 0.0012 | 0.0020 |
| | | | < 60,000 | 0.0015 | 0.0025 |
| | | > No. 5 | Any | 0.0015 | 0.0025 |
| | Welded-wire reinforcement | ≤ W31 or D31 | Any | 0.0012 | 0.0020 |
| Precast[2] | Deformed bars or welded-wire reinforcement | Any | Any | 0.0010 | 0.0010 |

[1]Prestressed walls with an average effective compressive stress of at least 225 psi need not meet the requirement for minimum longitudinal reinforcement $\rho_\ell$.

[2]In one-way precast, prestressed walls not wider than 12 ft and not mechanically connected to cause restraint in the transverse direction, the minimum reinforcement requirement in the direction normal to the flexural reinforcement need not be satisfied.

**11.6.2** If in-plane $V_u \geq 0.5\phi V_c$, (a) and (b) shall be satisfied:

(a) $\rho_\ell$ shall be at least the greater of the value calculated by Eq. (11.6.2) and 0.0025, but need not exceed $\rho_t$ in accordance with Table 11.6.1.

$$\rho_\ell \geq 0.0025 + 0.5(2.5 - h_w/\ell_w)(\rho_t - 0.0025) \qquad (11.6.2)$$

(b) $\rho_t$ shall be at least 0.0025

**R11.6.2** For monotonically loaded walls with low height-to-length ratios, test data (Barda et al. 1977) indicate that horizontal shear reinforcement becomes less effective for shear resistance than vertical reinforcement. This change in effectiveness of the horizontal versus vertical reinforcement is recognized in Eq. (11.6.2); if $h_w/\ell_w$ is less than 0.5, the amount of vertical reinforcement is equal to the amount of horizontal reinforcement. If $h_w/\ell_w$ is greater than 2.5, only a minimum amount of vertical reinforcement is required (**0.0025*sh***).

## 11.7—Reinforcement detailing

**11.7.1** *General*

**11.7.1.1** Concrete cover for reinforcement shall be in accordance with 20.6.1.

**11.7.1.2** Development lengths of deformed and prestressed reinforcement shall be in accordance with 25.4.

**11.7.1.3** Splice lengths of deformed reinforcement shall be in accordance with 25.5.

**11.7.2** *Spacing of longitudinal reinforcement*

**11.7.2.1** Spacing $s$ of longitudinal bars in cast-in-place walls shall not exceed the lesser of $3h$ and 18 in. If shear

## CODE

(b) Base of structural steel section shall be designed to transfer the factored forces from the steel core only, and the remainder of the total factored forces shall be transferred to the foundation by compression in the concrete and by reinforcement.

**16.3.2** *Required strength*

**16.3.2.1** Factored forces and moments transferred to foundations shall be calculated in accordance with the factored load combinations in Chapter 5 and analysis procedures in Chapter 6.

**16.3.3** *Design strength*

**16.3.3.1** Design strengths of connections between columns, walls, or pedestals and foundations shall satisfy Eq. (16.3.3.1) for each applicable load combination. For connections between precast members and foundations, requirements for vertical integrity ties in 16.2.4.3 or 16.2.5.2 shall be satisfied.

$$\phi S_n \geq U \quad (16.3.3.1)$$

where $S_n$ is the nominal flexural, shear, axial, torsional, or bearing strength of the connection.

**16.3.3.2** $\phi$ shall be determined in accordance with 21.2.

**16.3.3.3** Combined moment and axial strength of connections shall be calculated in accordance with 22.4.

**16.3.3.4** At the contact surface between a supported member and foundation, or between a supported member or foundation and an intermediate bearing element, nominal bearing strength $B_n$ shall be calculated in accordance with 22.8 for concrete surfaces. $B_n$ shall be the lesser of the nominal concrete bearing strengths for the supported member or foundation surface, and shall not exceed the strength of intermediate bearing elements, if present.

**16.3.3.5** At the contact surface between supported member and foundation, $V_n$ shall be calculated in accordance with the shear-friction provisions in 22.9 or by other appropriate means.

**16.3.3.6** At the base of a precast column, pedestal, or wall, anchor bolts and anchors for mechanical connections shall be designed in accordance with Chapter 17. Forces developed during erection shall be considered.

## COMMENTARY

**R16.3.3** *Design strength*

**R16.3.3.4** In the common case of a column bearing on a footing, where the area of the footing is larger than the area of the column, the bearing strength should be checked at the base of the column and the top of the footing. In the absence of dowels or column reinforcement that continue into the foundation, the strength of the lower part of the column should be checked using the strength of the concrete alone.

**R16.3.3.5** Shear-friction may be used to check for transfer of lateral forces to the supporting pedestal or footing. As an alternative to using shear-friction across a shear plane, shear keys may be used, provided that the reinforcement crossing the joint satisfies 16.3.4.1 for cast-in-place construction or 16.3.6.1 for precast construction. In precast construction, resistance to lateral forces may be provided by mechanical or welded connections.

**R16.3.3.6** Chapter 17 covers anchor design, including seismic design requirements. In precast concrete construction, erection considerations may control base connection design and need to be considered.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown* <u>*underlined*</u> *and deletions are shown* ~~*struck-through*~~*.)*  *Do not use 'Track Changes'*

**Code:**

<u>**5.1.5** *Minimum reinforcement for connections between masonry and foundation*</u>

<u>**5.1.5.1** The connection between a reinforced masonry wall, column, or pilaster and a concrete foundation shall be designed with vertical reinforcement crossing the interface. The minimum vertical reinforcement shall be of the same grade, size and spacing as that in the masonry member, except as permitted in section 5.1.5.4.</u>

<u>**5.1.5.2** Connection reinforcement shall be provided either by extending vertical bars from the supporting concrete foundation into the masonry, or by anchored and projecting reinforcement dowels.</u>

<u>**5.1.5.3** Connection reinforcement shall satisfy the lap splice requirements of Section 6.1.7.</u>

<u>**5.1.5.4** Masonry without vertical reinforcement and masonry intended to have a pinned base shall be designed with an alternate mechanism to transfer the applicable forces to the foundation.</u>

**Code Commentary:**

<u>**5.1.5** *Minimum reinforcement for connections between masonry and foundation*</u>

<u>**5.1.5.1** The Code requires a minimum amount of reinforcement between structural masonry members and their foundation to promote ductile behavior. This reinforcement is required to provide a degree of structural integrity, and a load path during the construction stage and during the life of the structure.</u>

<u>**5.1.5.2** See ACI 318 for design requirements for embedment/development of steel reinforcement in concrete foundations.</u>

<u>**5.1.5.4** There are situations where the connection reinforcement required by Sections 5.1.5.1 through 5.1.5.3 could be detrimental and/or undesirable to the design. These situations include "unreinforced" masonry, where foundation dowels could add unwarranted costs; and also where a pinned base is a necessary or desired aspect of the design. For an example, see NCMA TEK 5-5B (2011) for pinned base guidance for masonry walls used for pre-engineered metal buildings. For these situations, the designer is required to design a connection between masonry and the foundation that is adequate to resist design loads.</u>

**References**

<u>NCMA TEK 5-5B (2011) "Integrating Concrete Masonry Walls with Metal Building Systems," *e-TEK Notes,* National Concrete Masonry Association, www.ncma.org.</u>

**Specification:**
None

**Specification Commentary:**
None

**Mandatory Requirements Checklist:**

None

**Optional Requirements Checklist:**
None

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 2 *Affirmative* | 5 *Affirmative w/ comment* | 1 *Negative* | 0 *Abstain* | 1 *Did not vote* |

**Subcommittee Comments:**

The subcommittee ballot received several comments and 1 negative vote, which have been incorporated in the current ballot.

The comments and negative vote are listed below.

| Comment Type | Commenter | Comment |
|---|---|---|
| Affirmative With Comment | Dr. Ece Erdogmus | The following statement in the proposed language for 5.1.5.3 is unusal for code language "...and be close enough to the masonry reinforcement". I am not sure what will be considered acceptable for "close enough"? |
| | Mr. David L. Pierson | I still disagree with the idea of suggesting that the dowels or bars need to be developed into the footing. But unfortunately that may be how this is read by the Building Officials - especially based on the commentary language. Since a large majority of single story Masonry Buildings have continuous footings that are 24" wide and 12" thick, we must make sure we don't force thicker footings with this provision. The hooked bar development length for a vertical bar in 3000 psi concrete is 10" for a #5 bar and 12" for a #6 bar and 13" for a #7 bar. Add to this the 3" cover requirement, and the footings under every wall with #5 and larger footings will get thicker. Also, in Utah it is very common to have a 12" x 24" footing, with a 24" tall 8" wide concrete foundation wall above that, with masonry above that. The straight bar development length of #5 bars is 28" in 3000 psi concrete. So that very common construction condition would not be allowed if the dowels have to be developed into the foundation wall.<br>This is not acceptable. I will only vote affirm with comment now because there is no chance to resolve a negative at subcommittee. But I may vote negative at Main because this is a really big deal. To avoid a negative at the Main Ballot, there needs to be no implication that the dowel or bar must be developed into the member below. The word "anchored" in the code is also difficult unless the commentary indicates that being "anchored" does not mean the bar must be fully developed. |
| | Mr. David T. Biggs | I propose changing "5.1.5.3 Connection reinforcement shall be designed to project into the masonry and be close enough to the masonry reinforcement to satisfy the lap splice requirements of Section 6.1.6." "Close enough" is unneccessary. If the goal is to lap splice, just say so. Section 6.1.7 (not 6.1.6) already governs lap splices. |
| | Mr. Thomas Michael Corcoran | 1. Code section 5.1.5.1: I'm not sure what "except as permitted in section 5.1.5.3" means. It seems to me that section 5.1.5.3 is all about masonry splice requirements.<br>2. Code section 5.1.5.2: Should the word "longitudinal" be the word "vertical"?<br>3. Code section 5.1.5.3: Aren't the requirements for non-contact splices located in Code Section 6.1.6? If so, suggest rewriting 5.1.5.3 to "project into the masonry to satisfy the lap splice requirements of section 6.1.6". |
| | Ms. Heather A. Sustersic | the rationale does a great job of explaining the proposed changes. I am in favor of the change but would like to see 5.1.5.3 reworded with more definitive language. I'm voting AFC to help this get through, but think that "close enough" is too nebulous to be code language. I instead suggest rewording 5.1.5.3 as follows:<br>"5.1.5.3 Connection reinforcement shall satisfy the lap splice requirements of Section 6.1.7."<br>Note that I changed the section reference to 6.1.7 (instead of 6.1.6) based on the 5/6/2020 working draft wherein splices has been renumbered due to the addition of global scope language in 6.1.1. |
| Negative | Ms. Jamie L. Davis | 5.1.5.1 The wording is confusing to me. Suggest we modify this to more closely follow the ACI wording:<br>The FORCES AT THE BASE OF masonry wallS, columnS, or pilasterS SHALL BE TRANSFERRED TO THE foundation BY BEARING AND BY REINFORCEMENT OR DOWELS. THE MINIMUM vertical reinforcement crossing the interface shall be the same grade, size and spacing as thE VERTICAL REINFORCING in the masonry member, except as permitted in section 5.1.5.3.<br>5.1.5.2 This section doesn't make sense to me. The concrete foundation is already in place so the masonry longitudinal reinforcing cannot be extended into the concrete foundation. The reinforcing/dowels have to be in place before the masonry. Not sure this section is even needed. I suggest this section be deleted. |

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| | | Connection reinforcement shall be provided either by *extending longitudinal bars from the masonry into the supporting concrete* foundation or by anchored and projecting reinforcement dowels.<br>In general I don't like the nomenclature 'connective reinforcement' or 'connection reinforcement'. Prefer we just refer to vertical reinforcing and dowels. |

| Item Number | Comment Type | Commenter | Comment | Comment File |
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| This negative vote was intended for 15B-SM-008 but was submitted on item 15B-SM-005 | Negative | Mr. John M. Hochwalt<br>johnh@kpff.com | I am voting negative because the provisions as written are ambiguous as to whether the reinforcing steel must be fully developed in the foundation.<br><br>Some users may interpret the provision to only require a nominal engagement with the concrete, thereby defeating the purpose of the provision. As Pierson noted in his subcommittee vote, other users or building officials may interpret this to require that all reinforcing be fully developed, regardless of how much reinforcing that is.<br><br>I think the appropriate resolution would be to follow the ACI 318 model and provide a minimum quantity of reinforcing that must be developed. This could be a lesser amount than that required by ACI 318. | |
| 15B-SM-008 | Affirmative With Comment | Mr. Charles B. Clark Jr.<br>cclark@bia.org | In proposed Code Section 5.1.5.4, use of the term "alternative mechanism" seems very permissive. Unfortunately, I do not have an alternate text recommendation. | |
| | | Mr. David L. Pierson<br>davep@arwengineers.com | I think I can justify, with the language as is, that it is not necessary to fully develop the connection reinforcement into the foundation below. But it could take some back and forth with the building official. I really wish the following were added to the end of commentary section 5.1.5.2. | |

| Item Number | Comment Type | Commenter | Comment | Comment File |
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| | | | "Nothing in this code requires that the connection reinforcement be fully developed into the concrete foundation." | |
| | | Mr. Scott W. Walkowicz<br>scott@walkowiczce.com | I give up!!! But, either 5.1.5.3 needs Commentary developed per much of the rationale or some other Commentary section needs a discussion to guide designers, in particular, that a little foundation rotation will release the 'fixity' moments and so that foundations do not need to be designed for fixed base moments - dramatic increases in footing size could occur on some projects which may quickly add more cost than will be saved. Mark McGinley may be able to help with this and some input from a well qualified geotechnical engineer would be beneficial. I agree that many designers won't even think about this, but I know a few who would and these provisions as-is could result in higher cost/less competitive masonry projects. Please resolve this | |
| | | Ms. Rochelle C. Jaffe<br>jaffeconsulting@gmail.com | 1. Section 5.1.5.1: Capitalize "section 5.1.5.4" to "Section 5.1.5.4".<br><br>2. Section 5.1.5.3: Is there a technical reason why mechanical or welded splices are not permitted? If not, change the sentence to "Connection reinforcement shall be spliced with the masonry reinforcement in accordance with Section 6.1.7."<br><br>3. Commentary 5.1.5.1: Delete the comma between "integrity" and "and".<br><br>4. Commentary 5.1.5.4: Delete the quotation marks around "unreinforced" in the second sentence. Unreinforced masonry is a defined term in TMS 402. | Jaffe_R_C_15B-SM-008_AC.doc |
| | Negative | Dr. Richard M. Bennett<br>rmbennett@utk.edu | 1. It is not clear how the subcommittee comments and negatives were resolved, particularly the concern raised by Dave Pierson. I would interpret these provisions as requiring the reinforcement to be developed in the foundation.<br><br>2. Why is unreinforced in quotations in the commentary? Unreinforced masonry is an acceptable system. The following | |

| Item Number | Comment Type | Commenter | Comment | Comment File |
|---|---|---|---|---|
| | | | commentary statement is troublesome: For these situations, the designer is required to design a connection between masonry and the foundation that is adequate to resist design loads. What is a connection for unreinforced masonry? Typically unreinforced masonry is just laid on the footing with nothing additional. Is this a connection? I think most people would say no. Easy solution is to just make 5.1.5 applicable to reinforced masonry. | |
| | | Dr. William Mark McGinley<br>m.mcginley@louisville.edu | I do not think mandating all reinforced walls have all their vertical reinforcing extended into the foundation should be a minimum code requirement. If these bars are needed for uplift they should be designed for this, and the load path needs to be extended into the foundation<br><br>If you are trying to provide for internal bracing for construction that can be at the choice of the engineer and contractor. If it is not really required for life safety concerns it should not be in the code. The fact that you suggest allowing unreinforced masonry and other mechanisms to be used indicates that life safety is not an issue<br><br>Finally, unless the bars are fully developed into the concrete foundation, splicing them with wall bars will not ensure a sufficient connection. I did not see any requirement that this be done and calling the rebar footing dowel in the commentary suggests otherwise. | |
| | | Mr. Jason J. Thompson<br>jthompson@ncma.org | I prefer previous iterations of this ballot item to this one. If one wants to use reinforcement for internal bracing...great, but the code shouldn't have prescriptive detailing requirements that override judgement and mandate an optional practice. Internal bracing requires more than just development of the reinforcement at the intersection of the foundation. The footing will also need to be designed to accommodate the potential overturning forces. The way I read 5.1.5.4 would effectively invalidate 95% of wall designs...as that about the percentage that are designed assuming pinned supports. | |

| Item Number | Comment Type | Commenter | Comment | Comment File |
|---|---|---|---|---|
| | | Mr. John G. Tawresey<br>johntaw@aol.com | The wording is not clear.<br><br>What does it mean "with". Does it mean at the same time, or the same engineer or something else. Or, are you meaning "using" instead of with?<br><br>The connection between a reinforced masonry wall, column, or pilaster and a concrete foundation shall be designed with the vertical reinforcement crossing the interface.<br><br>The provision is unnecessary and overly conservative:<br><br>The provision and commentary are unclear regarding the requirement for in-plane or out-of-plane wall flexural reinforcement. The comment about ductility implies in-plan. For in-plane the requirements of the Code are clear. It is required to have continuous load paths between walls and foundations, which will require anchorage to the foundation for in-plane flexure.<br><br>For out-of-plane the provision is clearly too conservative. The dowels are required for shear capacity, not flexure. For a box store wall, or gym etc. the amount of wall flexural reinforcement, because of height, could far exceed the requirement for shear resistance at the base of the wall. Additionally, for out-of-plane ductility is not an issue. | |
| | | Ms. Diane B. Throop<br>diane@dtpeconsulting.com | I agree with the negative voter, Davis on all her comments. In addition, this is a potential interface problem and a provision that seems to go overboard. It is a constructability nightmare and there has been no evidence put forth that it is needed. | |
| | | | | |

RESPONSES TO COMMENTS ON BALLOT 15B-SM-008

| Vote | Voter | Comment | Response |
|---|---|---|---|
| AWC | Clark | In proposed Code Section 5.1.5.4, use of the term "alternative mechanism" seems very permissive. Unfortunately, I do not have an alternate text recommendation. | Reference to alternative mechanism has been removed. |
| AWC | Pierson | I think I can justify, with the language as is, that it is not necessary to fully develop the connection reinforcement into the foundation below. But it could take some back and forth with the building official. I really wish the following were added to the end of commentary section 5.1.5.2.<br><br>"Nothing in this code requires that the connection reinforcement be fully developed into the concrete foundation." | The previous ballot required that the dowels match the vertical reinforcement. This ballot allows much smaller area of dowels in SDC A, B, and C, but does require that the dowels be fully developed. Fully developing the dowels allows them to achieve large deformations and absorb energy in extreme events, which is the expressed intent of the general structural integrity provisions in ASCE 7 Section 1.4. |
| AWC | Walkowicz | I give up!!! But, either 5.1.5.3 needs Commentary developed per much of the rationale or some other Commentary section needs a discussion to guide designers, in particular, that a little foundation rotation will release the 'fixity' moments and so that foundations do not need to be designed for fixed base moments - dramatic increases in footing size could occur on some projects which may quickly add more cost than will be saved. Mark McGinley may be able to help with this and some input from a well qualified geotechnical engineer would be beneficial. I agree that many designers won't even think about this, but I know a few who would and these provisions as-is could result in higher cost/less competitive masonry projects. Please resolve this | The following sentence has been added to the commentary:<br><br>*Unless the foundation is proportioned to restrain out-of-plane rotation, most foundations can accommodate sufficient rotation to approximate a pinned support. The presence of dowels does not necessitate treating conditions that would otherwise be approximated as pinned as having a degree of fixity.*<br><br>Hopefully this is sufficient to address this concern. It may be appropriate next cycle to consider explicitly addressing soil-structure interaction in the Chapter 4 analysis and modeling provisions. |

RESPONSES TO COMMENTS ON BALLOT 15B-SM-008

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| AWC | Jaffe | 1. Section 5.1.5.1: Capitalize “section 5.1.5.4” to “Section 5.1.5.4”.<br>2. Section 5.1.5.3: Is there a technical reason why mechanical or welded splices are not permitted? If not, change the sentence to “Connection reinforcement shall be spliced with the masonry reinforcement in accordance with Section 6.1.7.”<br>3. Commentary 5.1.5.1: Delete the comma between “integrity” and “and”.<br>4. Commentary 5.1.5.4: Delete the quotation marks around “unreinforced” in the second sentence. Unreinforced masonry is a defined term in TMS 402. | On item 2, the ballot refers to splices generically.<br><br>The ballot has been completely rewritten so the other comments are no longer applicable. |
| Negative | Bennett | 1. It is not clear how the subcommittee comments and negatives were resolved, particularly the concern raised by Dave Pierson. I would interpret these provisions as requiring the reinforcement to be developed in the foundation.<br>2. Why is unreinforced in quotations in the commentary? Unreinforced masonry is an acceptable system. The following commentary statement is troublesome: For these situations, the designer is required to design a connection between masonry and the foundation that is adequate to resist design loads. What is a connection for unreinforced masonry? Typically unreinforced masonry is just laid on the footing with nothing additional. Is this a connection? I think most people would say no. Easy solution is to just | 1. The subcommittee negative by Pierson expressed a concern about the impact of dowels on foundation thickness. This ballot has addressed this in SDC A, B and C by permitting the use of smaller dowels. As is discussed in the rationale, this allows the use of footings as thin as 10”.<br>2. The ballot exempts ordinary plain masonry shear walls – the only wall type with no prescriptive reinforcement – from the minimum dowel requirements. It is suggested that general structural integrity provisions continue to be considered next code cycle, and it be considered whether walls with no reinforcing or dowels meet the expectations of modern building codes for general structural integrity. |

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| | | make 5.1.5 applicable to reinforced masonry. | |
| Negative | McGinley | I do not think mandating all reinforced walls have all their vertical reinforcing extended into the foundation should be a minimum code requirement. If these bars are needed for uplift they should be designed for this, and the load path needs to be extended into the foundation<br><br>If you are trying to provide for internal bracing for construction that can be at the choice of the engineer and contractor. If it is not really required for life safety concerns it should not be in the code. The fact that you suggest allowing unreinforced masonry and other mechanisms to be used indicates that life safety is not an issue<br><br>Finally, unless the bars are fully developed into the concrete foundation, splicing them with wall bars will not ensure a sufficient connection. I did not see any requirement that this be done and calling the rebar footing dowel in the commentary suggests otherwise. | This ballot does not require that all vertical reinforcing be developed into the foundation.<br><br>The purpose of the dowels in this ballot is to provide general structural integrity in SDC A+, and seismic ductility in SDC D+.<br><br>This ballot is consistent in that it requires development of the dowels and splicing with the wall vertical reinforcement. |
| Negative | Thompson | I prefer previous iterations of this ballot item to this one. If one wants to use reinforcement for internal bracing...great, but the code shouldn't have prescriptive detailing requirements that override judgement and mandate an optional practice. Internal bracing requires more than just development of the reinforcement at the intersection of the foundation. The footing will also need to | The dowels proposed in this ballot are unrelated to internal bracing, although the commentary acknowledges that some contractors may propose to increase the dowels for that purpose,<br><br>There is no longer a provision requiring a mechanism other than dowels to achieve a pinned base. Commentary has been added to note that the presence of dowels does not |

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| | | be designed to accommodate the potential overturning forces. The way I read 5.1.5.4 would effectively invalidate 95% of wall designs...as that about the percentage that are designed assuming pinned supports. | necessitate designing the foundation to achieve rotational restraint. |
| Negative | Tawresey | The wording is not clear.<br><br>What does it mean "with". Does it mean at the same time, or the same engineer or something else. Or, are you meaning "using" instead of with?<br><br>The connection between a reinforced masonry wall, column, or pilaster and a concrete foundation shall be designed with the vertical reinforcement crossing the interface.<br><br>The provision is unnecessary and overly conservative:<br><br>The provision and commentary are unclear regarding the requirement for in-plane or out-of-plane wall flexural reinforcement. The comment about ductility implies in-plan. For in-plane the requirements of the Code are clear. It is required to have continuous load paths between walls and foundations, which will require anchorage to the foundation for in-plane flexure.<br><br>For out-of-plane the provision is clearly too conservative. The dowels are required for shear capacity, not flexure. For a box store wall, or gym etc. the amount of wall flexural reinforcement, because of height, | The ballot has been completely rewritten.<br><br>The provisions have been rewritten to require fewer foundation dowels.<br><br>Some minimum dowels are necessary to provide general structural integrity. Please see the rationale for discussion of the necessity of integrity reinforcement. The amount of integrity dowels required is less than that required comparable concrete members. |

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| | | could far exceed the requirement for shear resistance at the base of the wall. Additionally, for out-of-plane ductility is not an issue. | |
| | Throop | I agree with the negative voter, Davis on all her comments. In addition, this is a potential interface problem and a provision that seems to go overboard. It is a constructability nightmare and there has been no evidence put forth that it is needed. | The subcommittee negative by Davis expressed concerns about the wording of the ballot; the ballot has been completely rewritten.<br><br>The evidence for the necessity of the ballot is in the poor performance of structures without minimum integrity connections. For example, the ASCE 7 Commentary discusses the Ronan Point collapse which was due to an accidental gas explosion and load bearing precast panels being insufficiently connected to the structure. A masonry wall with no foundation dowels may be similarly vulnerable to collapse in a freak event.<br><br>The ACI 318 provisions for structural integrity are also indicative that TMS 402 has fallen behind peer codes in addressing structural integrity. |

# 2022 TMS 402/602 Committee
# Response to Public Comment

| | |
|---|---|
| **Committee:** Main Committee | **Ballot #:** 20 |
| **Item #:** 20-SM-012 | |
| **Technical Contact/Email:** Ece.erdogmus@design.gatech.edu | |
| **Public Comment Number:** 2022 Comment # 12 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** 6/1/2021 | |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #12:** Defining Column, with the knowledge of the IBC's wall definition applicable to Masonry, would be helpful. This is important as TMS 402 requires specific detailing requirements for columns that are not present for walls. It is obvious to me that a jamb next to a door or window opening, is not intended to be considered a column. The scenario that can come up where this definition clarification would be helpful is this: two masonry walls intersect at 90 degrees. Both of those walls have openings right next to the intersection, leaving only a 8 inch by 16 inch section of wall between those openings, is that a column?

**Response:** The subcommittee disagrees with the comment and proposes no change.

***Rationale for the Response:*** IBC has a wall definition but not one for a column. There is a definition for columns in TMS 402, Chapter 2. Further related information is provided in Section 5.3.1. As such, the committee believes that there is sufficient description for a column in TMS402, without being too complex that it can lead to new interpretation issues. In the example the commenter provides, the elements described do not have to be designed as a column, but they can be if the designer chooses. There is also further guidance in the Masonry Designer's Guide.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 6 *Affirmative* | 1 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:** The subcommittee ballot's rationale had referenced the Strength Design of Masonry text book, but the commenter suggested changing that to Masonry Designer's guide because it covers both Strength Design and ASD. Subcommittee agreed with this suggestion and we have made this change to the rationale.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|

**Item #:** **20-SM-016**

**Technical Contact/Email:** Ece.erdogmus@design.gatech.edu

**Public Comment Number:** 2022 Comment # 16

**Public Comment Response Based on TMS 402/602 Draft Dated** **6/1/2021**

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #16:** The standard discusses lateral-torsional buckling of beams. However, there is nothing that provides guidance to designers as to the design of masonry beams for torsional effects. For example, masonry lintels/beams might have a shelf angle bolted to them for support of an anchored veneer. This induces torsion into the beam and its supporting wall jambs. ACI 318 has criteria for concrete beams but TMS 402 is silent on torsion. Masonry code criteria should be provided for torsion. Until that code criterion is provided, users should be warned of the torsional concerns through commentary.

**Response:** Committee agrees with the comment but no changes can be proposed at this time without further research.

***Rationale for the response:*** The committee acknowledges this is a topic that is not addressed in the Code but should be. However, the magnitude of the effort required is beyond the ability of the committee to address at this time in the current cycle. Particularly, published research on the topic needs to be identified and assessed. The committee proposes this Public Comment be left open and referred to the next code cycle.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 7 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-SM-017 | |
| **Technical Contact/Email:** David L. Pierson (davep@arwengineers.com) | |
| **Public Comment Number:** 2022 Comment # 17 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** 6/1/2021 | |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

The introductory statement of Section 5.2 indirectly prohibits unreinforced masonry beams, since the references are to sections 8.3. and 9.3 only. If this is the case, why not explicitly state this?

**Response:**

It is true that the code requirements are that all beams must be reinforced. This is indicated by the reference to the sections 8.3, 9.3, and 11.3 as noted in the comment. Use of pointer provisions such as this is common in the code.

For consistency with other code sections, the committee disagrees that it is necessary to explicitly state this. However, the committee does agree that commentary language would be appropriate.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:** (None)

**Code Commentary:**

**5.2 – Beams**

All masonry beams are reinforced to provide ductility.

**Specification:** (None)

**Specification Commentary:** (None)

**Mandatory Requirements Checklist:** (None)

**Optional Requirements Checklist:** (None)

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 7 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments: Meeting vote executed on 9-21-2021**

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-SM-018 & 019 | |
| **Technical Contact/Email:** David L. Pierson – davep@arwengineers.com | |
| **Public Comment Number:** 2022 Comment # 18 and 19 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

On Ballot 19, Item 19-SM-PC18-19 passed (the one negative vote was withdrawn). The ballot results in a new definition for span length. There were, however, comments that the committee agreed to address. Specifically, these two comments:

(Bennett): The first phrase "For design of beams other than those designed as deep beams per section 5.2.2," is not needed. Design is either by 5.2.1 or 5.2.2, and 5.2.2 does not include 5.2.1.1 (it includes other sections of 5.2.1, but not this one). Thus, the beginning phrase is not needed."

(Robinson): It might be better to indicate that this is the minimum span length. I could see a scenario where you have a 24 inch pier and taking the span to the center of the pier for calculation of the negative moment over the pier would be more conservative"

**Response/Rationale:**

The Committee agrees that the first phrase is not needed, as outlined by Bennett (and others). Also, the Committee agrees that adding "minimum" is a reasonable and clarifying addition.

The proposed language below shows the existing language as it would appear in the code with the passage of Ballot Item 19-SM-PC18-19. Strikethrough and Underline is based on that language.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)* *Do not use 'Track Changes'*

**Code:**

**5.2.1.1** *Span length* — ~~For design of beams other than those designed as deep beams per section 5.2.2,~~ <u>Minimum</u> span length shall be the distance from face-to-face of supports, plus ½ of the required bearing length at each end.

**Code Commentary:**

**Specification:**

**Specification Commentary:**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 7 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| **Committee: Main Committee** | **Ballot #: 20** |
|---|---|
| **Item #: 20-SM-020** | |
| **Technical Contact/Email:** Ece.erdogmus@design.gatech.edu | |
| **Public Comment Number:** 2022 Comment # 20 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated 6/1/2021** | |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #20:** Add commentary for 5.2.2.1 as follows: Design engineers commonly use the clear span or the distance between the centers of the bearing as the span length. It is the design engineer's responsibility to determine the span length.

**Response:** The subcommittee disagrees with the public comment and no changes are proposed.

***Rationale for the response***: Deep beams have special provisions that do not reflect conventional flexural member mechanics (e.g. direct specification of internal moment arm) and there is an associated approach to span length directly specified in the code. Adding commentary to state that the engineer has discretion to determine span length would contradict the mandatory code language.

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 7 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| Item #: 20-SM-021 | |
| Technical Contact/Email: Ece.erdogmus@design.gatech.edu | |
| Public Comment Number: 2022 Comment # 21 | |
| Public Comment Response Based on TMS 402/602 Draft Dated 6/1/2021 | |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #21:** Can a corbel (see Section 2.2) be a single course? Consider revising definition/requirements to clarify.

**Response:**
TMS 402, Section 2.2 defines a corbel as "*Corbel* — A projection of successive courses from the face of masonry."
TMS 402, Section 5.5 governs the design of corbels. Corbels can be either load bearing (Section 5.5.1) or non-loadbearing (Section 5.5.2).
Loadbearing corbels must be designed as reinforced (ASD, Strength or Prestress). Therefore, one course corbels are acceptable provided they are designed accordingly.
Non-loadbearing corbels can either be reinforced (ASD, Strength or Prestress) or detailed as noted in 5.5.2 and shown in Figures CC 5.5-1 or CC 5.5-2. As noted with the Loadbearing corbels, one course corbels are acceptable provided they are designed accordingly. In addition, one course corbels are acceptable when detailed per 5.5.2 and as shown by the commentary figures.
In short, given the design criteria already answers the question, **no change is proposed.**

| Subcommittee Vote: | | | | |
|---|---|---|---|---|
| 7 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** **20-SM-022A** | |
| **Technical Contact/Email:** Heather Sustersic, hsustersic@colbycoengineering.com | |
| **Public Comment Number:** 2022 Comment # 22 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #22:**
(Please note that this was a long comment with bullet items. Numbers in **bold** added by the ballot author for ease of reference)

> *Section 5.1.1. is nicely revised, but several things to consider:*
> ***1)*** *- Typo in heading "Intersecton" should be "Intersection".*
>
> ***2)*** *-After reviewing the new layout of all content in Section 5.1 as well as the rest of Chapter 5, I am wondering if we should title 5.1. Masonry Walls, instead of "Masonry Assemblages". Everything under 5.1. appears to relate to walls, and beams, columns, and Pilasters (which all could technically be called "assemblages") are in the subsequent sections 5.2, 5.3., and 5.4. Alternatively, we may need a Definition in Chapter 2 for "Assemblages" If this term is meant to refer to something other than a wall in Chapter 5.*
>
> ***3)*** *- In the first and second sentence, neither clearly indicates that the walls referred to are intersecting walls. In the first sentence, it is not clear that pilasters are needed for lateral support. Suggest changing first sentence to become, "Masonry walls that intersect and require lateral support from one another or from pilasters within those walls shall be ..." Suggest changing second sentence to become, "Masonry walls that intersect and do not require lateral support..."*
>
> ***4)*** *-Could we reverse the contents of 5.1.1.1 and 5.1.1.3 so that the shortest and simplest solution (structurally independent walls) comes first, then walls that support each other but are not considered composite, then finally composite walls and how to satisfy this condition?*
>
> ***5)*** *-The following sentence in the commentary is confusing. "Achieving stress transfer at a T intersection with running bond only is difficult." No recommendation, limitations or checks are given to ensure the stress transfer is successful- so what is the purpose of this sentence? What value does it bring to the code or the commentary?*

**Response/Rationale:**
The SM subcommittee agrees that Section 5.1 would greatly benefit from a reorganization of the content, as suggested. This ballot proposes such a reorganization to address items 2 and 4 of the public comment. Item 1 is also addressed herein. Items 3 and 5 will be addressed separately in 20-SM-PC22-B and 20-SM-PC22-C, respectively.

**Voter should note that Ballots 20-SM-PC22-A, PC22-B, and PC22-C are independent in that passing of each one will result ONLY in changes for the items covered in that ballot.**

The current organization of this Chapter is as follows:

**5.1** – Masonry Assemblies
- 5.1.1 Wall Intersections
  - 5.1.1.1 Design of masonry wall and pilaster intersections for composite action
  - 5.1.1.2 Design of lateral supports for walls, without composite action at the intersections
  - 5.1.1.3 Design of independent walls
- 5.1.2 Effective Compressive Width
- 5.1.3 Concentrated Loads
- 5.1.4 Multiwythe Masonry

**5.2** – Beams
**5.3** – Columns
**5.4** – Pilasters
**5.5** – Corbels

The proposed organization of section 5.1 is as follows:

| **Proposed New Section** | **Previous Section** |
|---|---|
| **5.1** – General | |
| **5.1.1 –** Concentrated Loads | **5.1.3** |
| **5.1.2** – Effective Compressive Width | **5.1.2** |
| **5.1.3** – Multiwythe Masonry | **5.1.4** |
| **5.2** – Walls | **5.1.1** |
| **5.2.1** Design of Independent Walls | **5.1.1.3** |
| **5.2.2** Design of lateral supports for walls, without composite action at the intersections | **5.1.1.2** |
| **5.2.3** Design of masonry wall and pilaster intersections for composite action | **5.1.1.1** |
| **5.3** – Beams | **5.2** |
| **5.4** – Columns | **5.3** |
| **5.5** – Pilasters | **5.4** |
| **5.6** – Corbels | **5.5** |

For voter ease, when a section is being relocated, the current section number is shown in strikethrough and the proposed section number is underlined. If no change other than in the section number is proposed, it is so indicated and the provisions are not included in the ballot. Where changes are proposed to the existing provision it is shown with changes indicated in strikethrough or underline.

Cross reference updates throughout the document are also proposed.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)  Do not use 'Track Changes'*

**Code:**

**4.1.6.1** Flanges of intersecting walls designed in accordance with Section 5.~~1.1.1~~2.3.1 shall be included in stiffness determination.

**5.1** — ~~Masonry assemblies~~ General

**5.1.~~3.~~1** – *Concentrated Loads* (No change)

**5.1.2** – *Effective compressive width* (No change)

**5.1.~~4.~~3** – *Multiwythe masonry*
Design of masonry composed of more than one wythe shall comply with the provisions of Section 5.1.~~4.~~3.1, and either 5.1.~~4.~~3.2 or 5.1.~~4.~~3.3.

**5.1.~~4.~~3.1** The provisions of Sections 5.1.~~4.~~3 2, and 5.1.~~4.~~3 3 shall not apply to AAC masonry units, masonry veneer, and glass masonry units.

**5.1.~~4.~~3.2** *Composite action* (No change)

**5.1.~~4.~~3.2.1** (No change)

**5.1.~~4.~~3.2.2** (No change)

**5.1.~~4.~~3.2.3** (No change)

**5.1.~~4.~~3.3** Wythes not bonded by headers shall meet the requirements of either Section 8.1.4.2 or Section 9.1.7.2 and shall be bonded by non-adjustable wall ties according to Table 5.1.~~4.~~3.2.3.

**Table 5.1.~~4.~~3.2.3**: *Maximum Spacing and Wall Area for Wall Ties in Multiwythe Masonry Designed per Chapter 8, Chapter 9, or Chapter 10*

**5.1.~~4.~~3.3** *Non-composite action*
The design of multiwythe masonry for non-composite action shall comply with Sections 5.1.~~4.~~3.3.1 and 5.1.~~4.~~3.3.2.

**5.1.~~4.~~3.3.1** (No change)

**5.1.~~4.~~3.3.2** Wythes of masonry designed for non-composite action shall be connected by ties meeting the requirements of Section 5.1.~~4.~~3.2.3 or by adjustable wall ties according to Table 5.1.~~4.~~3.2.3. Where the cross wires of joint reinforcement are used as wall ties, the joint reinforcement shall be ladder-type or tab-type and shall conform with spacing requirements of Table 5.1.~~4.~~3.2.3. Wall ties shall be without cavity drips.

**5.~~1.1.~~2** *Wall intersections*
Masonry walls depending upon one another for lateral support, or upon pilasters within those walls, shall be anchored or bonded at locations where they meet or intersect per Section 5.~~1.1.~~2.~~1.~~2 or 5.~~1.1.~~2.~~2.~~3. Masonry walls that do not require lateral support from other walls or pilasters within those walls shall be designed in accordance with Section 5.~~1.1.~~2.~~3.~~1

**5.~~1.1.3.~~2.1** *Design of Independent Walls*

5.~~1.1.3.~~2.1.1 (No change)

**5.~~1.1.~~2.2** *Design of lateral supports for walls, without composite action at the intersections*
Masonry walls depending upon masonry supporting walls or pilasters for lateral support, without composite action between those members, shall be anchored to the supporting walls or pilasters in accordance with sections 5.~~1.1.~~2.2.1 through 5.~~1.1.~~2.2.3.

**5.~~1.1.~~2.2.1** (No change)
**5.~~1.1.~~2.2.2** (No change)
**5.~~1.1.~~2.2.3** (No change)

**5.~~1.1.1~~.2.3** *Design of masonry wall and pilaster intersections for composite action*

**5.~~1.1.1~~.2.3.1** (No change)
**5.~~1.1.1~~.2.3.2** (No change)
**5.~~1.1.1~~.2.3.3** The width of flange considered effective on each side of the web shall be the smaller of the actual flange on either side of the web wall and the value shown in Table 5.~~1.1.~~2.3, based on the state of stress in the flange and whether or not the masonry is reinforced. The effective flange width shall not extend past a movement joint.
**Table 5.~~1.1.~~2.3:** *Effective Flange Width*
**5.~~1.1.1~~.2.3.4** (No change)
**5.~~1.1.1~~.2.3.5** (No change)

**5~~.2~~.3** – Beams (No change – renumber subsections)

**5~~.3~~.4** – Columns (No change – renumber subsections)

**5~~.4~~.5** – Pilasters

**5~~.4~~.5.1** Walls interfacing with projecting pilasters shall not be considered as flanges, unless the construction requirements of Sections 5.~~1.1.1~~.2.3.1 and 5.~~1.1.1~~.2.3.5 are met. When these construction requirements are met, the projecting pilaster's flanges shall be designed in accordance with Sections 5~~.1.1.1~~.2.3.2 through 5~~.1.1.1~~.2.3.4.
**5~~.4~~.5.2** (No change)

**5~~.5~~.6** – Corbels (No change – renumber subsections)

**8.3.2** *Design Assumptions*

…
(d) The compressive resistance of steel reinforcement does not contribute to the axial and flexural strengths unless lateral reinforcement is provided in compliance with the requirements of Section 5~~.3~~.4.1.4.
…

**8.3.3.3** When lateral reinforcement is provided in compliance with the requirements of Section 5~~.3~~.4.1.4, the compressive stress in bar reinforcement shall not exceed the values given in Section 8.3.3.1.

**9.3.2** *Design Assumptions*

…
(e) Compression and tension stress in reinforcement is Es multiplied by the steel strain, but not greater than fy. Except as permitted in Section 9.3.5.6.1 (e) for determination of maximum area of flexural reinforcement, the compressive stress of steel reinforcement does not contribute to the axial and flexural resistance unless lateral restraining reinforcement is provided in compliance with the requirements of Section 5~~.3~~.4.1.4.
…

**9.3.3.2** *Beams* — Design of beams shall meet the requirements of Section 5~~.2~~.3 and the additional requirements of Sections 9.3.3.2.1 through 9.3.3.2.4.

**9.3.5.6.2.4** Shear walls not designed by Section 9.3.5.6.2.3 shall have special boundary elements at boundaries and edges around openings in shear walls where the maximum extreme fiber compressive stress, corresponding to forces from strength level loads including earthquake effect, exceeds 0.2 f 'm . The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than 0.15 f m . Stresses shall be calculated from strength level loads using a linearly elastic model and net section properties. For walls with flanges, an effective flange width as defined in Section 5.~~1.1.1~~.2.3.3 shall be used.

**11.1.1.1** Except as stated elsewhere in this Chapter, design of AAC masonry shall comply with the requirements of Part 1 and Part 2, excluding Sections 5~~.5~~.6.1, 5~~.5~~.6.2(d) and 5~~.3~~.4.2.

**11.1.10** *Corbels* — Load-bearing corbels of AAC masonry shall not be permitted. Non-load-bearing corbels of AAC masonry shall conform to the requirements of Section 5~~.5~~.6.2(a) through 5~~.5~~.6.2(c). The back section of the corbelled section shall remain within ¼ in. (6.4 mm) of plane.

**11.3.2** *Design Assumptions*

(e) Tension and compression stresses in reinforcement shall be calculated as the product of steel modulus of elasticity, Es , and steel strain, s, but shall not be greater than fy . Except as permitted in Section 11.3.3 for determination of maximum area of flexural reinforcement, the compressive stress of steel reinforcement shall be neglected unless lateral restraining reinforcement is provided in compliance with the requirements of Section 5~~.3~~.4.1.4.

**11.3.4.2** *Beams* — Design of beams shall meet the requirements of Section 5~~.2~~.3 and the additional requirements of Sections 11.3.4.2.1 through 11.3.4.2.5.

**11.3.6.6.3** Shear walls not designed to the provisions of Section 11.3.6.6.2 shall have special boundary elements at boundaries and edges around openings in shear walls where the maximum extreme fiber compressive stress, corresponding to forces from strength level loads including earthquake effect, exceeds 0.2f'AAC . The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than 0.15f'AAC. Stresses shall be calculated from strength level loads using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in Section 5.~~1.1.1~~.2.3.3 shall be used.

**12.1.1** *Scope*

This chapter provides minimum requirements for the structural design of concrete masonry, clay masonry, and AAC masonry infills, either non-participating or participating. Infills shall comply with the requirements of Part 1, Part 2, excluding Sections ~~5.2,~~ 5.3, 5.4, ~~and~~ 5.5 and 5.6, Section 12.1, and either Section 12.2 or 12.3.

**13.2.2.3.**1 Veneer shall be designed for a vertical application. Out-of-plane corbelling shall meet the requirements of Section 5~~.5~~.6.2.

**D.1.1.1** Design of GFRP reinforced masonry shall comply with the following requirements:

(a) Part 1
(b) Chapter 4
(c) Chapter 5 excluding Sections 5~~.2~~.3.1.6.2, 5~~.2~~.3.2, and 5~~.3~~.4
(d) Sections 6.1.2, 6.1.3.3, 6.1.3.4, 6.1.3.5, 6.1.4, 6.1.5, and 6.1.10
(e) Chapter 7 excluding Section 7.3.2
(f) Chapter 9 excluding Sections 9.1.4.3, 9.1.4.4, 9.1.9.3, 9.3.2(e), 9.3.3.1.2, 9.3.3.2.4, 9.3.4.3, and 9.3.5.4.
....

**Code Commentary:**

**5.1** — ~~Masonry assemblies~~ General

**5.1.~~3.~~1** *Concentrated loads* (No content change – renumber figure CC-5.1-5 and cross-references to CC-5.1-5 a, b, and c)

**5.1.2** – *Effective compressive width* (No content change – renumber figure CC-5.1-4 and cross-reference)

**5.1.~~4.~~3** – *Multiwythe masonry* (No content change – renumber figures CC-5.1-6 thru CC-5.1-8 and cross-references)

**5.~~1.1.~~2** *Wall intersections*

Wall intersections may be designed and detailed as fully composite walls, as laterally supported walls, or as structurally independent walls in accordance with Sections 5.~~1.1.~~2.1, 5.~~1.1.~~2.2, and 5.~~1.1.~~2.3. Acceptable methods of detailing laterally supported walls may include the use of mesh ties, joint reinforcement, or anchors capable of transferring lateral loads only at the interface of laterally supported walls.

… (No content change – renumber figures CC-5.1-1 thru CC-5.1-3 and cross-references)

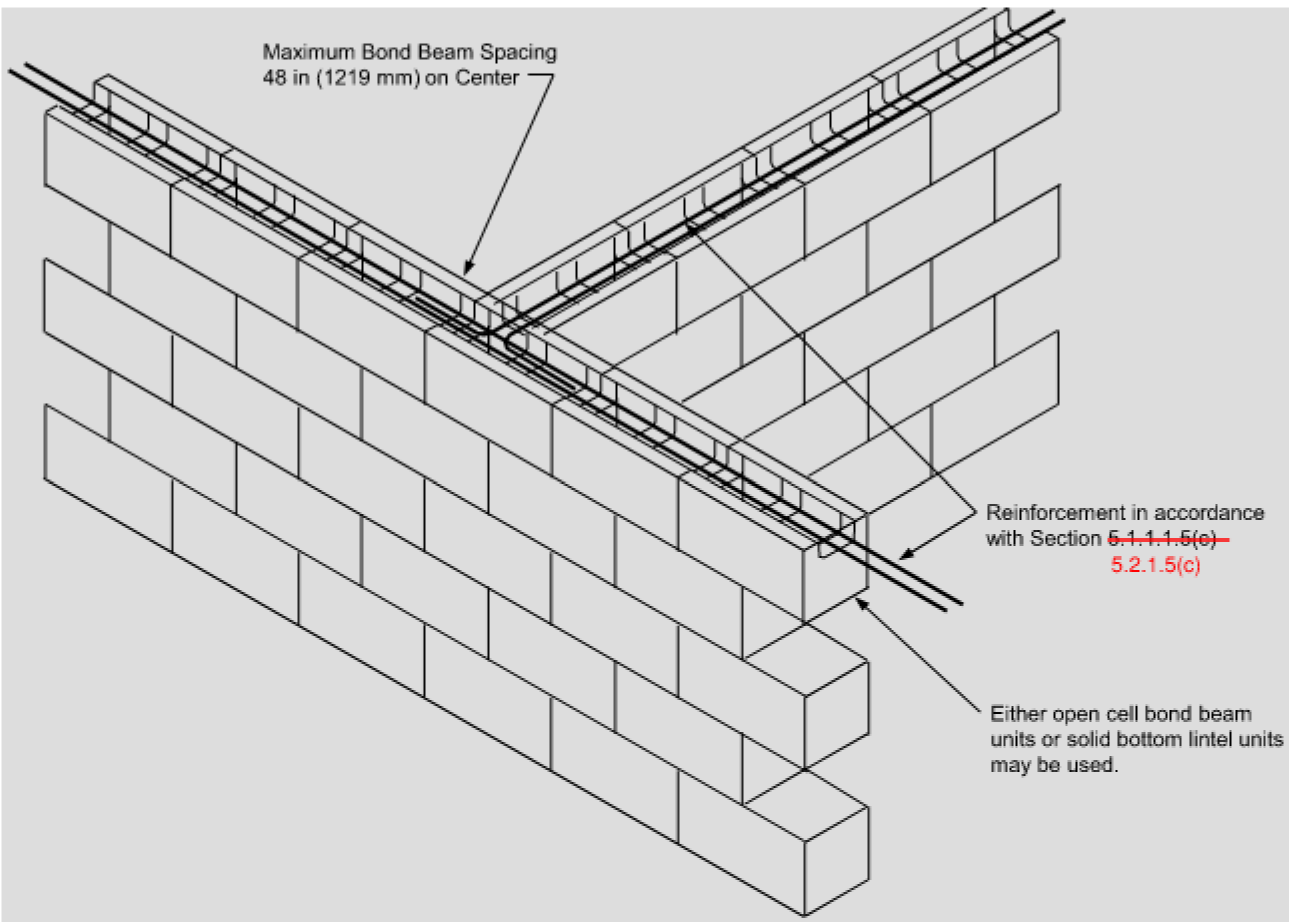



*Figure CC-5.1-3 — Bond beam at wall intersection*

**5.~~2.~~3** – Beams (No change – renumber subsections)

**5.~~3.~~4** – Columns (No change – renumber subsections)

**5.~~4.~~5** – Pilasters (No change – renumber subsections)

**5.~~5.~~6** – Corbels (No change – renumber subsections)

**8.3.4.3** Columns

…

Additional column design and detailing requirements are given in Section 5~~.3~~.4.

**13.2.2.3.1** Although anchored veneer can be installed in non-vertical applications, the veneer ties, fasteners and support need to be engineered as these unique loading conditions are not considered in the prescriptive requirements. Designs that exceed the prescriptive corbeling limitations of Section 5~~.5~~.6.2 would need to use modeling analysis method of Section 13.2.3.3.

**D.1.1.1** ….(No change)

In applying the prescriptive detailing provisions such as those of Sections 4.6, 5.~~1.1.1~~.2.3.5 (c) and 7.4.3.1.1, GFRP reinforcing bars can be treated the same as steel reinforcing bars, as the GFRP reinforcement has a similar capacity to the steel reinforcement, since the minimum $f_{fd}$ for straight GFRP reinforcement is 65,100 psi (449 MPa), and the GFRP reinforcement can accommodate larger strains.

**Specification:**

None

**Specification Commentary:**

None

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 7 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| | |
|---|---|
| **Committee: Main Committee** | **Ballot #: 20** |
| **Item #: 20-SM-022B** | |
| **Technical Contact/Email:** Heather Sustersic, hsustersic@colbycoengineering.com | |
| **Public Comment Number:** 2022 Comment # 22 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**
(Please note that this was a long comment with bullet items. Numbers in **bold** added by the ballot author for ease of reference)

*Section 5.1.1. is nicely revised, but several things to consider:*
***1)** - Typo in heading "Intersecton" should be "Intersection".*

***2)** -After reviewing the new layout of all content in Section 5.1 as well as the rest of Chapter 5, I am wondering if we should title 5.1. Masonry Walls, instead of "Masonry Assemblages". Everything under 5.1. appears to relate to walls, and beams, columns, and Pilasters (which all could technically be called "assemblages") are in the subsequent sections 5.2, 5.3., and 5.4. Alternatively, we may need a Definition in Chapter 2 for "Assemblages" If this term is meant to refer to something other than a wall in Chapter 5.*

***3)** - In the first and second sentence, neither clearly indicates that the walls referred to are intersecting walls. In the first sentence, it is not clear that pilasters are needed for lateral support. Suggest changing first sentence to become, "Masonry walls that intersect and require lateral support from one another or from pilasters within those walls shall be ..." Suggest changing second sentence to become, "Masonry walls that intersect and do not require lateral support..."*

***4)** -Could we reverse the contents of 5.1.1.1 and 5.1.1.3 so that the shortest and simplest solution (structurally independent walls) comes first, then walls that support each other but are not considered composite, then finally composite walls and how to satisfy this condition?*

***5)** -The following sentence in the commentary is confusing. "Achieving stress transfer at a T intersection with running bond only is difficult." No recommendation, limitations or checks are given to ensure the stress transfer is successful- so what is the purpose of this sentence? What value does it bring to the code or the commentary?*

**Response/Rationale:**
This ballot addresses item 3 of public comment #22, in which the wording of section 5.1.1 is adjusted for clarity. Section references and language shown under the proposed changes are from the current working draft of TMS 402.

If ballot 20-SM-PC22-B passes, this content will be located in the proposed section 5.2 as reorganized. That said, voter should note that Ballots 20-SM-PC22-A, PC22-B, and PC22-C are independent in that passing of each one will result ONLY in changes for the items covered in that ballot.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)  Do not use 'Track Changes'*

**Code:**
5.1 — Masonry assemblies
5.1.1 *Wall intersections*
Masonry walls that intersect and require ~~depending upon one another for~~ lateral support from one another or ~~upon~~ from pilasters within those walls, shall be anchored or bonded at locations where they meet or intersect per Section 5.1.1.1 or 5.1.1.2. Masonry walls that intersect and do not require lateral support from other walls or pilasters within those walls shall be designed in accordance with Section 5.1.1.3.

**Code Commentary:**
None

**Specification:**

None

**Specification Commentary:**

None

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 6 | *Affirmative* | 1 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 2 | *Did not vote* |

**Subcommittee Comments:** The comment corrected a typo. The ballot as presented to Main20 is the correct version.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| Item #: 20-SM-022C | |
| Technical Contact/Email: Heather Sustersic, hsustersic@colbycoengineering.com | |
| Public Comment Number: 2022 Comment # 22 | |
| Public Comment Response Based on TMS 402/602 Draft Dated | 6/1/2021 |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☒ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

(Please note that this was a long comment with bullet items. Numbers in **bold** added by the ballot author for ease of reference)

*Section 5.1.1. is nicely revised, but several things to consider:*

***1)*** *- Typo in heading "Intersecton" should be "Intersection".*

***2)*** *-After reviewing the new layout of all content in Section 5.1 as well as the rest of Chapter 5, I am wondering if we should title 5.1. Masonry Walls, instead of "Masonry Assemblages". Everything under 5.1. appears to relate to walls, and beams, columns, and Pilasters (which all could technically be called "assemblages") are in the subsequent sections 5.2, 5.3., and 5.4. Alternatively, we may need a Definition in Chapter 2 for "Assemblages" If this term is meant to refer to something other than a wall in Chapter 5.*

***3)*** *- In the first and second sentence, neither clearly indicates that the walls referred to are intersecting walls. In the first sentence, it is not clear that pilasters are needed for lateral support. Suggest changing first sentence to become, "Masonry walls that intersect and require lateral support from one another or from pilasters within those walls shall be ..." Suggest changing second sentence to become, "Masonry walls that intersect and do not require lateral support..."*

***4)*** *-Could we reverse the contents of 5.1.1.1 and 5.1.1.3 so that the shortest and simplest solution (structurally independent walls) comes first, then walls that support each other but are not considered composite, then finally composite walls and how to satisfy this condition?*

***5)*** *-The following sentence in the commentary is confusing. "Achieving stress transfer at a T intersection with running bond only is difficult." No recommendation, limitations or checks are given to ensure the stress transfer is successful- so what is the purpose of this sentence? What value does it bring to the code or the commentary?*

**Response/Rationale:**
This ballot addresses item 5 of public comment #22, in which the commentary to section 5.1.1 is improved for clarity. Section references are related to the current working draft of TMS 402.

If ballot 20-SM-PC22-A passes, this content will be located in the proposed section 5.2 as reorganized. That said, voter should note that Ballots 20-SM-PC22-A, PC22-B, and PC22-C are independent in that passing of each one will result ONLY in changes for the items covered in that ballot.

---

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**
None

**Code Commentary:**
**5.1** — Masonry assemblies
**5.1.1** *Wall intersections*
Wall intersections may be designed and detailed as fully composite walls, as laterally supported walls, or as structurally independent walls in accordance with Sections 5.1.1.1, 5.1.1.2, and 5.1.1.3. Acceptable methods of detailing laterally supported walls may include the use of mesh ties, joint reinforcement, or anchors capable of transferring lateral loads only at the interface of laterally supported walls.

Movement joints at structurally independent walls should be sized to prevent force transfer when the walls laterally deform.

Connections of webs to flanges of walls may be accomplished by running bond, metal connectors, or bond beams. Achieving stress transfer at a T intersection with running bond only is difficult<u> due to constructability and modularity of the units</u>. A running bond connection is shown in Figure CC-5.1-1 with a "T" geometry over their intersection.

**Specification:**
None

**Specification Commentary:**
None

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 6 *Affirmative* | 1 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:** The comment corrected a typo. The ballot as presented to Main20 is the correct version.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| | |
|---|---|
| **Committee: Main Committee** | **Ballot #: 20** |
| **Item #: 20-SM-028 & 029** | |
| **Technical Contact/Email:** David L. Pierson (davep@arwengineers.com) | |
| **Public Comment Number:** 2022 Comment #28 and #29 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

No 28: Consider revising section 5.3.2 as follows: "...gravity loads not exceeding 2,000 pounds (8,900 N) or 50 PSI..."

No 29: Consider revising commentary of section 5.3.2 as follows: "...load of 2,000 pounds (8,900 N) or 50 PSI..."

**Response/Rationale:**

The provisions referenced here are for lightly loaded columns, which are a special case for columns in low to medium seismic regions with light loads. The proposed language provides for a uniform pressure to be applied over the cross-sectional area (or any portion thereof), as may occur if a beam extends over the column on a bearing plate.

The committee has reviewed this and verified that the actual masonry stresses caused by a 50 psi uniform pressure are lower than the stresses caused by a 2000 pound point load (which may be located at any location in the cross section). Apologies for the handwriting but attached is a sheet showing this analysis.

ARW ENGINEERS
structural consultants

Project No. ________ Sheet No. ________
Project ________
Prepared By ________ Date ________

GRAVITY LOADS NOT EXCEEDING 2000 lb OR 50 psi
8×8 COLUMN   $A = 58\text{ in}^2$   $S = 73\text{ in}^3$


WORST CASE 2000# LOAD

$P = 2000\#$   $M = 2000(3.8) = 7600\ \#\text{-in}$

$f_a = \frac{2000}{58} = 34\text{ psi}$   $f_b = M/S = 104\text{ psi}$

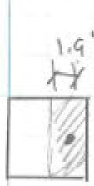
$\boxed{f_a + f_b = 138\text{ psi}}$

50 psi OVER ENTIRE SECTION

$P = 50(58) = 2900\#$   $f_a = 50\text{ psi}$

$M = 0$   $f_b = 0\text{ psi}$

$f_a + f_b = 50\text{ psi} < 138$

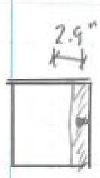



50 psi OVER 1/2 OF SECTION

$P = 50(29) = 1450\#$   $f_a = 25\text{ psi}$

$M = 1450(1.9) = 2755\ \#\text{-in}$   $f_b = 2755/73 = 38\text{ psi}$

$f_a + f_b = 63\text{ psi} < 138$

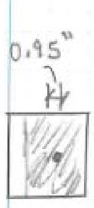



50 psi OVER 1/4 OF SECTION

$P = 50(15) = 750\#$   $f_a = 13\text{ psi}$

$M = 750(2.9") = 2175\ \#\text{-in}$   $f_b = M/S = 30\text{ psi}$

$f_a + f_b = 43\text{ psi} < 138$




50 psi OVER 3/4 OF SECTION

$P = 50(43.5) = 2175\#$   $f_a = 37.5\text{ psi}$

$M = 2175(0.95) = 2066\ \#\text{-in}$   $f_b = \frac{2066}{73} = 28\text{ psi}$

$f_a + f_b = 66\text{ psi} < 138$

IN ALL CASES, THE MOMENT DUE TO 50 psi IS MUCH LOWER, AND $f_a + f_b$ IS MUCH LOWER, SO 50 psi IS OKAY.

Note that the resultant of 50 psi across the entire section is 3000 pounds, but since it is the resultant of a uniform pressure, it must occur at the centroid of the section and therefore does not result in moment in the column.

The committee agrees that this change is appropriate.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**5.3.2** *Lightly loaded columns*

Masonry columns used only to support light frame roofs of carports, porches, sheds or similar structures assigned to Seismic Design Category A, B, or C, which are subject to allowable stress level gravity loads not exceeding 2,000 lbs (8,900 N) or 50 psi (345 kPa) acting within the cross-sectional dimensions of the column are permitted to be constructed as follows:

**Code Commentary:**

**5.3.2** *Lightly Loaded Columns*

…..The axial load limit of 2,000 pounds (8,900 N) or 50 psi (345 kPa) was developed based on the flexural strength of a nominal 8 in. (203 mm) by 8 in. (203 mm) by 12 ft high (3.66 m) column with one No. 4 (M#13) reinforcing bar in the center and f'm of 1350 psi (9.31 MPa)….

**Specification:**

No Changes

**Specification Commentary:**

No Changes

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 7 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments: Meeting vote executed on 9-21-2021**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-SM-078 | |
| **Technical Contact/Email:** Jamie Davis (jdavis@ryanbiggs.com) | |
| **Public Comment Number:** 2022 Comment # 078 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** 6/1/2021 | |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**
The code limit on column slenderness defines the slenderness in terms of the distance between lateral supports, not the effective height, yet the commentary uses the nomenclature "h" and the terminology "effective height." It is suggested to remove "h/r" from the first sentence of the commentary, and to move the second sentence, along with Figure CC-5.3-1, to Section 2.2 as commentary on the nomenclature "effective height." This would have the additional benefit of making this commentary applicable to walls as well as columns.

**Response:**

The Section in question is shown below:

**5.3.1** *General column design*

**5.3.1.1** *Dimensional limits* — Dimensions shall be in accordance with the following:

(a) The distance between lateral supports of a column shall not exceed 99 multiplied by the least radius of gyration, $r$.

**5.3.1** *General column design*

**5.3.1.1** *Dimensional limits* — The limit of 99 for the slenderness ratio, $h/r$, is judgment based. See Figure CC-5.3-1 for effective height determination. The minimum nominal side dimension of 8 in. (203 mm) results from practical considerations.

In Section 2.2, the Effective height is defined as the distance between lines of support. The definition states that it is to be used for calculating the slenderness ratio of a member.

*Effective height* — Clear height of a member between lines of support or points of support and used for calculating the slenderness ratio of a member. Effective height for unbraced members shall be calculated.

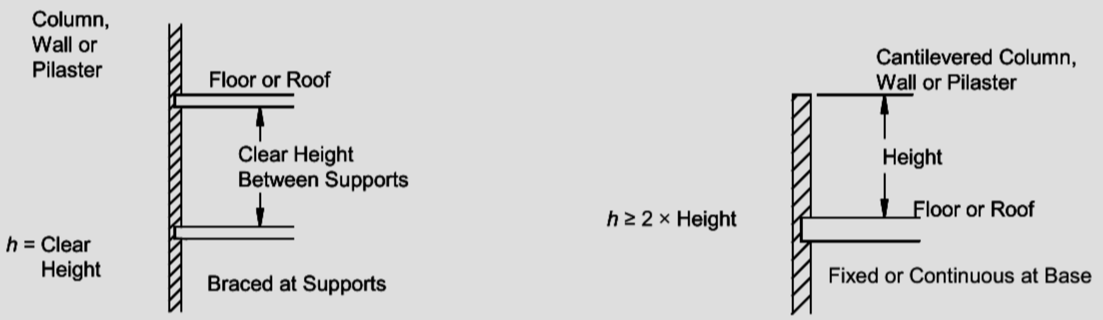
Section 2.2 also defines 'h' as the Effective height of a column, wall, or pilaster.

$h$ = ~~depth of the element~~effective height of column, wall, or pilaster, in. (mm)

Therefore the nomenclature ‘h’ and ‘effective height’ are interchangeable and are not in conflict.

The term h/r in the Commentary is a commonly used expression for the slenderness ratio that is used throughout the Code.

Figure CC-5.3-1 is a clarification of the term h and effective height and specifically addresses cantilevered conditions.




If data (see Section 1.3) show that there is reliable restraint against translation and rotation at the supports, the “effective height” may be taken as low as the distance between points of inflection for the loading case under consideration.

*Figure CC-5.3-1 — Effective height, h, of column, wall, or pilaster*

It is

The limit of h/r < 99 applies only to columns. This limit does not apply to other members such as walls and provisions for h/r > 99 are found in Chapter 8 and 9.

**8.3.4.2.1** The compressive force in reinforced masonry due to axial load only shall not exceed that given by Equation 8-16 or Equation 8-17:

(a) For members having an *h/r* ratio not greater than 99:

$$P_a = \left(0.30 f'_m A_n + 0.65 A_{st} F_s\right)\left[1 - \left(\frac{h}{140r}\right)^2\right]$$

(Equation 8-16)

(b) For members having an *h/r* ratio greater than 99:

$$P_a = \left(0.30 f'_m A_n + 0.65 A_{st} F_s\right)\left(\frac{70r}{h}\right)^2$$

(Equation 8-17)

The committee disagrees that it is necessary to change the wording of the Commentary.

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 7 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 7 | *Did not vote* |

**Subcommittee Comments: Meeting vote executed on 9-21-2021**

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-SM-111 | |
| **Technical Contact/Email:** Jamie Davis (jdavis@ryanbiggs.com) | |
| **Public Comment Number:** 2022 Comment # 111 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** 6/1/2021 | |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**
In 5.1.1.2, I believe it would remove redundancy of "supporting walls that support" and be more clear to describe walls that provide lateral support as "intersecting" rather than "supporting" walls. This occurs twice in the sentence. Proposed section would read: Masonry walls depending upon intersecting masonry walls or pilasters for lateral support, without composite action between those members, shall be anchored to the intersecting walls or pilasters in accordance with sections 5.1.1.2.1 through 5.1.1.2.3.

**Response:**

The Section in question is shown below:

**5.1.1.2** *Design of lateral supports for walls, without composite action at the intersections*

~~Laterally supported m~~Masonry walls depending upon masonry supporting walls~~,~~ or pilasters~~, or upon structural members~~ for lateral support, without composite action between those members, shall be anchored to the supporting walls or pilasters in accordance with sections 5.1.1.2.1 through 5.1.1.2.3.

The committee agrees that the wording would be clearer to state "Masonry walls depending upon intersecting masonry walls or pilasters for lateral support, without composite action between those members, shall be anchored to the intersecting walls or pilasters in accordance with sections 5.1.1.2.1 through 5.1.1.2.3."

This would be consistent with the wording of Section 5.1.1.1.5 which refers to 'intersecting walls'

**5.1.1.1.5** The connection of intersecting walls and walls to pilasters shall conform to one of the following requirements:

(a) At least fifty percent of the masonry units at the interface shall interlock.

(b) Walls shall be anchored by steel connectors grouted into the wall and meeting the following requirements:

(1) Minimum size: $^1/_4$ in. x $1^1/_2$ in. x 28 in. (6.4 mm x 38.1 mm x 711 mm) including 2-in. (50.8-mm) long, 90-degree bend at each end to form a U or Z shape.

(2) Maximum spacing: 48 in. (1219 mm).

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**5.1.1.2** *Design of lateral supports for walls, without composite action at the intersections* Masonry walls depending upon intersecting masonry ~~supporting~~ walls, or pilasters, or upon structural members for lateral support, without composite action between those members, shall be anchored to ~~the supporting walls~~ ~~or pilasters~~ those members in accordance with sections 5.1.1.2.1 through 5.1.1.2.3.

**Code Commentary: NA**

**Specification: NA**

**Specification Commentary: NA**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 7 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments: Meeting vote executed on 9-21-2021**

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-SM-PC136 | |
| **Technical Contact/Email:** Philippe Ledent (phil@masonryinfo.org) | |
| **Public Comment Number:** 2022 Comment # 136 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** 6/1/2021 | |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☒ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 136 is related to TMS 402 Section 5.3.1.4 (c), starting on page 81 at line 10 and states the following:

Needs clarification. Seems to say no longitudinal bar can be spaced more than 6 inches with out ties. Figure CC-5.3.3 seems to contradict this requirement.

**Response:**

The committee disagrees with the Commenter's statement that Figure CC-5.3-3 contradicts the requirements stated in TMS 402 Section 5.3.1.4 (c). TMS 402 Section 5.3.1.4 (c) does state that no longitudinal bars shall be spaced farther than 6 in. clear on each side without ties.

For reference, TMS 402 Section 5.3.1.4 (c) states the following: "Lateral ties shall be so arranged so that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a lateral tie with an included angle of not more than 135 degrees. No bar shall be farther than 6 in. (152 mm) clear on each side along the lateral tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular lateral tie is permitted. Lap length for circular ties shall be 48 bar diameters."

Figure CC-5.3-3 is copied below for reference and depicts a reinforced clay masonry column. Figure CC-5.3-3 shows a clay column where the clear space between bars is stated as being greater than 6 in., and it shows all bars laterally supported by ties with an included angle of not more than 135 degrees which is in agreement with TMS 402 Section 5.3.1.4 (c).

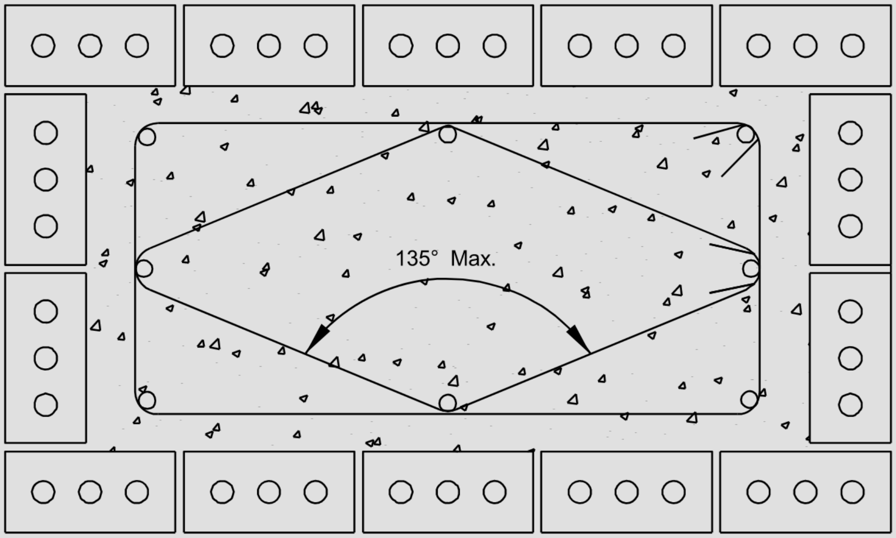



*Figure CC-5.3-3 — Example of a lateral tie included angle for a clay masonry column*

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 7 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 2 | *Did not vote* |

**Subcommittee Comments: Meeting vote executed on 9-21-2021**

# 2022 TMS 402/602 Committee Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| Item #: 20-SM-190 | |
| Technical Contact/Email: Ece.erdogmus@design.gatech.edu | |
| Public Comment Number: 2022 Comment # 190 | |
| Public Comment Response Based on TMS 402/602 Draft Dated 6/1/2021 | |

**This ballot item proposes the following response to the Public Comment:**

☐ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☒ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment #190:**
Page 79, Line 10, Section 5.2.2.3 The requirements for distribution of flexural reinforcement for deep beams appears to be excessive and makes designers less likely to use the deep beam provisions. The zone where distributed flexural reinforcement is required by code is based on dv. As shown in the figure on the previous page, dv is an arbitrary value selected by the designer during beam design and could vary from a single course to the full depth of the panel above the opening. The masonry panel does not know what beam depth was used in its design and will not behave differently for varying values of dv. If cracking in the bottom half of dv is a concern for deep beams, then it should be a similar concern for masonry supported on a shallow beam, because the masonry will perform the same either way. If you look up the original primary research on which the deep beam provision are based, you'll find that the depth from the bottom to the neutral axis for beams with l_eff / dv < 1 is dependent on l_eff, not dv. So, for a given span, once dv exceeds l_eff, the flexural tension zone does not get any deeper. And unlike what is inferred in the commentary, the depth of the flexural tension zone is only 0.28 l_eff for a simply supported beam. In addition, the resultant tension force changes very little and is nearly constant at these high depths. I recommend revising the provisions to make them align better with the research and remove the over-conservatism so that designer can better use the benefits of deep beams in their designs without unnecessary penalties.

**Response:** Committee agrees with the comment but no changes can be proposed at this time without further research.
***Rationale:*** The committee acknowledges this is a topic that is not addressed in the Code but should be. However, the magnitude of the effort required is beyond the ability of the committee to address at this time in the current cycle. Particularly, published research on the topic needs to be identified and assessed and possibly, new research needs to be conducted. The committee proposes this Public Comment be left open and referred to the next code cycle.

| Subcommittee Vote: | | | | |
|---|---|---|---|---|
| 6 *Affirmative* | 1 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 2 *Did not vote* |

**Subcommittee Comments:** Commenter reminded that a response option was not checked. Chair corrected this error in this version of the ballot.

# 2022 TMS 402/602 Committee
# Response to Public Comment

| Committee: Main Committee | Ballot #: 20 |
|---|---|
| **Item #:** 20-SM-197 | |
| **Technical Contact/Email:** Philippe Ledent (phil@masonryinfo.org) | |
| **Public Comment Number:** 2022 Comment # 197 | |
| **Public Comment Response Based on TMS 402/602 Draft Dated** | **6/1/2021** |

**This ballot item proposes the following response to the Public Comment:**

☒ ***Committee agrees with Public Comment, change is proposed***

☐ ***Committee agrees comment has merit but proposed changes are not completely consistent with Public Comment***

☐ ***Committee disagrees with Public Comment and no changes are proposed***

☐ ***Committee unable to fully develop a response to Public Comment***

☐ ***Public Comment only requires a response, no change to document***

**Public Comment:**

Public Comment 197 is related to TMS 402 Section 5.2.1.6, starting on page 76 at line 12 and states the following:

Delete the word "reinforced". All masonry beams must be reinforced per section 5.2.

**Response/Rationale:**

The Subcommittee agrees with the commenter. Although Section 5.2 does not specifically state that all masonry beams must be reinforced, Section 5.2 requires that the design of beams meet the requirements of Section 8.3, Section 9.3, or Section 11.3. These sections all relate to reinforced masonry for allowable stress design, strength design, and design of AAC masonry, respectively. Thus, Section 5.2.1.6 stating "…deflections of reinforced masonry beams…" is redundant since all beams must inherently be reinforced per the requirements of Section 5.2.

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**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.) Do not use 'Track Changes'*

**Code:**

**5.2.1.6.1** Deflections of ~~reinforced~~ masonry beams need not be checked when the span length does not exceed 8 multiplied by the effective depth to the reinforcement, *d*, in the masonry beam.

**Code Commentary:**

**Specification:**

**Specification Commentary:**

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 7 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 2 | *Did not vote* |

**Subcommittee Comments: Meeting vote executed on 9-21-2021**

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| | | |
|---|---|---|
| **Committee: Main Committee** | | **Ballot #: 20** |
| **Item #: 20-VG-039, 201** | | |
| **Technical Contact/Email:** | Brian E. Trimble, PE, btrimble@imiweb.org, (703) 300-0109 | |
| **Draft Document Dated:** | **10/26/2021** | |
| **Response to Public Comment No.:** 39 and 201 | | |
| **This ballot item proposes the following response to the Public comment:** | | |
| ☐ | Committee agrees with public comment, change is proposed | |
| ☒ | Committee agrees comment has merit, but proposed changes are not completely consistent with public comment | |
| ☐ | Committee disagrees with public comment and no changes are proposed | |
| ☐ | Committee unable to fully develop a response to public comment | |
| ☐ | Public comment only requires a response, no change to document | |

**Public comments:**

**39** – This comment has multiple parts related to the definition of Cavity. The definition listed in the public comment draft is as follows:

> Cavity - The space between wythes of non-composite masonry or between masonry veneer and it backing, which may contain insulation.

I request that the phrase, 'which may contain insulation.' be deleted so the definition would read, "Cavity - The space between wythes of non-composite masonry or between masonry veneer and it backing."

Reasons for this are:

1) the phrase 'may include insulation' is in effect including a code provision within a definition. The insultation statement should appear within the appropriate chapters not in the definition;

2) also, by including only insultation in the definition as a permissible material in the cavity, the definition excludes anything else that could be in the cavity space such as drainage mat, mortar droppings, parging, and so on.

3) The definition as written only permits insultation in the cavity -- this directly conflicts with the commentary. One or the other needs to be changed. *[Page 37, Line 10-13; Section 2.2]*

**201** - The definitions of "cavity" and "cavity wall" are somewhat inconsistent. Under "cavity", it states correctly that the cavity may contain insulation. Under "cavity wall", it states that the air space may contain insulation. These are contradictory. It is the "cavity" that may contain the insulation, not the air space. An air space IS the cavity, or forms part of the cavity where other components such as insulation are included (in the cavity). *[Page 37, Line 10; Section 2.2]*

**Response:** The Committee agrees with the comment and modifications are made to the definition of cavity and cavity wall as requested as well as to the commentary to cavity.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)*

**Code:**

**2.2 – Definitions**
*Cavity* — The space between wythes of non-composite masonry or between a masonry veneer and its backing~~, which may contain insulation~~.

*Cavity wall* — A non-composite masonry wall consisting of two or more wythes, at least two of which are separated by a continuous <u>cavity</u> ~~air space; air space(s) between wythes may contain insulation~~; and separated wythes must be connected by wall ties.

**Code Commentary:**

**2.2 – Definitions**
*Cavity* — A cavity can be part of a multiwythe masonry wall assuming non-composite action (Section 5.1.4.3) or a veneer wall (<u>Chapter 13</u>). The cavity ~~may be detailed as a~~ <u>includes a</u> drainage space and may contain sheathing, insulation, drainage mats, fasteners, veneer ties, wall ties, mortar collection devices, and ancillary accessories depending upon function and design intent. Cavities are not permitted to be spanned by headers and are not permitted to be filled solid with mortar or grout, except for cavities below the base flashing which should be filled solid with mortar or grout. Also see "Drainage Space" to differentiate between the two definitions.

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 12 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:** A comment from a non-voting member resulted in a change to the second sentence of the commentary. The comment noted that the "use of 'may' in reference to the drainage space implies that there could be a case where there is no drainage space in the cavity. While the other items in the list are optional, I don't think it is possible to have a cavity wall or veneer wall without a drainage space."

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| **Committee: Main Committee** | | **Ballot #: 20** |
|---|---|---|
| **Item #: 20-VG-040** | | |
| **Technical Contact/Email:** | Brian E. Trimble, PE, btrimble@imiweb.org, (703) 300-0109 | |
| **Draft Document Dated:** | **10/26/2021** | |
| **Response to Public Comment No.:** 40 | | |
| **This ballot item proposes the following response to the Public comment:** | | |
| ☐ | Committee agrees with public comment, change is proposed | |
| ☐ | Committee agrees comment has merit, but proposed changes are not completely consistent with public comment | |
| ☒ | Committee disagrees with public comment and no changes are proposed | |
| ☐ | Committee unable to fully develop a response to public comment | |
| ☐ | Public comment only requires a response, no change to document | |

**Public comment:**
Revise the definition of Cavity to exclude adhered veneer by inserting the word "anchored' in the public comment draft definition of Cavity so it reads, Cavity - The space between wythes of non-composite masonry or between anchored masonry veneer and its backing. (note the public comment draft also includes the phrase "which may contain insulation" but I have proposed that be deleted in a previous comment so I did not include it here).
I propose this as there is a fundamental difference between the way non-composite masonry walls and anchored veneer wall cavities function compared to adhered veneer. I find it confusing the think of a cavity in adhered veneer - which is intended to be mostly filled with adhesive, mortar or other materials. Limiting 'cavities' to non-composite and anchored veneer walls is consistent with the terminology the design community uses which was the primary reason I was given for changing the definition in the first place. If this change is accepted, Tables 13.3.2.5 and 12.3.2.6 will need some revision in terminology as will parts of the rest of the chapter. *[Page 37, line 11-13; Section 2.2]*

**Response:** The committee disagrees with the comment since a cavity can refer to any space within either an anchored or adhered masonry wall. Cavities are often integrated in adhered veneer wall assemblies to achieve drainage wall or rainscreen functionality. In these cases, the cavity is located behind the cement board or lath and scratch coat to which the masonry units are adhered. Cavities in an adhered veneer are not filled with adhesive or mortar but would have a drainage material in the cavity. Cavities often include other materials (drainage mesh, mortar collection devices, fasteners, etc.) as noted in the commentary to the definition. It is important to keep the term cavity for an adhered veneer wall so that the span of a fastener can be clearly stated as it is for veneer ties in an anchored veneer wall. No changes are made.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)*

**Code: NONE**

**Code Commentary: NONE**

**Specification: NONE**

**Specification Commentary: NONE**

| Subcommittee Vote: | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 12 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 0 | *Did not vote* |

**Subcommittee Comments:** A sentence was added to the rationale based on a comment from a non-voting member.

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| Committee: Main Committee | | Ballot #: 20 |
|---|---|---|
| **Item #: 20-VG-056,067** | | |
| **Technical Contact/Email:** | Brian E. Trimble, PE, btrimble@imiweb.org, (703) 300-0109 | |
| **Draft Document Dated:** | **10/26/2021** | |
| **Response to Public Comment No.:** 056 and 067 | | |
| **This ballot item proposes the following response to the Public comment:** | | |
| ☐ | Committee agrees with public comment, change is proposed | |
| ☒ | Committee agrees comment has merit, but proposed changes are not completely consistent with public comment | |
| ☐ | Committee disagrees with public comment and no changes are proposed | |
| ☐ | Committee unable to fully develop a response to public comment | |
| ☐ | Public comment only requires a response, no change to document | |

**Public comments:**
**67** – There are several uses of the term "backing" in the adhered veneer provisions that are inconsistent with the definition of backing in Section 2.2. Alternate terminology should be used at the following locations (noted as "page - line"): 242-66, 243-7, 243-54, 243-56, 243-30, 243-79, 248-56.

**56** – By including concrete, masonry, and light frame in the definition of backing, the code is requiring the backing to be one of these types. However, the commentary for 13.2.2.3 states that there could be other backings. The definition of backing should be limited to: Structural wall or surface to which veneer is attached. The rest of the definition should be moved to the commentary.

*For voter's convenience here are the sections that are mentioned in Public Comment 67:*

242-66

**13.3** *Adhered Veneer* – The designer should provide for proper means of bonding units to the backing, attachment of the lath and scratch coat or cement backer unit to the structure, control curvature of the backing, account for differential movement, consider freeze-thaw cycling, water penetration, air leakage, and vapor diffusion. There are proprietary systems that can demonstrate compliance with this section. Manufacturer documentation including submittals should be consulted and referenced as required in TMS 602 Article 1.5.

243-7

**13.3.2.1** *Permitted units* — Prescriptively-designed adhered veneer shall be constructed of units complying with ASTM C1088, ASTM C1364, ASTM C1670/C1670M, or ASTM C1877. Units complying with ASTM C73 or TMS 602 Article 2.3 C shall be permitted provided the bond developed between adhered veneer units and backing has a shear strength of at least 50 psi (345 kPa) based on gross unit bonded area when tested in accordance with ASTM C482.

243-30

**13.3.2.4** *Installation requirements* — Lath and scratch coat shall not be required when adhered masonry veneer units are applied directly to concrete, concrete masonry, or cement backer units free of coatings, debris, membranes, or similar materials that would inhibit bond to the backing.

243-54 to 56

**13.3.2.1** *Permitted units* — The design strengths are based on bond between the unit and the mortar, and the backing and the mortar. The strength of other components in the system also needs to be considered. The strength could be controlled by the backing, such as a shear failure in a cement backer unit or within other layers within the system.

243-79

**13.3.2.4** *Installation requirements* — Installation of adhered masonry veneer units must comply with TMS 602. Lath and scratch coat are not required when adhered masonry veneer units are applied directly to certain backings (concrete, concrete masonry, or cement backer units) due to adequate bond.

248-56

**13.3.3** *Engineered design of adhered masonry veneer* — The intent of Section 13.3.3 is to permit the designer to use alternative unit thicknesses, areas, installation techniques, and units for adhered veneer. The designer should provide for proper means of bonding units to the backing, attachment of the lath and scratch coat to the structure, control curvature of the backing, account for differential movement, consider freeze-thaw cycling, water penetration, air leakage, and vapor diffusion. If sheathing is present, it should only be considered part of the backing if it is shown to have appropriate strength and stiffness for all applied loads, including the additional vertical loads permitted by Section 13.1.2.4, and there is an established load path from the sheathing to the studs. However, rational design provisions for adhered veneer have not been fully developed. Hagel et al (2017) presented an initial design method.

**Response:** Changes are made consistent with public comment. Changes are made to the definition and then four uses of the term backing are changed.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)*

**Code:**

**2.2 – Definitions**

*Backing* — Structural wall or surface to which veneer is attached. ~~Backings include concrete, masonry, and light frame. Light frame backings consist of either wood studs or cold-formed metal studs with associated auxiliary members.~~

**13.3.2.4** *Installation requirements* — Lath and scratch coat shall not be required when adhered masonry veneer units are applied directly to concrete, concrete masonry, or cement backer units free of coatings, debris, membranes, or similar materials that would inhibit bond to <u>those surfaces</u> ~~the backing~~.

**Code Commentary:**

**2.2 — Definitions**

*Backing* — The structural role provided by the backing varies between adhered and anchored veneer systems. For anchored veneer, the backing provides lateral support. For adhered veneer, the backing provides lateral and vertical support.

<u>Backings typically are concrete, masonry, and light frame.</u> In the context of this code, the use of the term “light frame backing” refers to wood or cold-formed metal studs and other structural members, such as rim joists, used in light frame construction.

**13.3** *Adhered Veneer* – The designer should provide for proper means of bonding units to the backing, attachment of the lath and scratch coat or cement backer unit to the backing ~~structure~~, control curvature of the backing, account for differential movement, consider freeze-thaw cycling, water penetration, air leakage, and vapor diffusion. There are proprietary systems that can demonstrate compliance with this section. Manufacturer documentation including submittals should be consulted and referenced as required in TMS 602 Article 1.5.

**13.3.2.1** *Permitted units* — The design strengths are based on bond between the unit and the mortar, and the backing and the mortar. The strength of other components in the system also needs to be considered. The strength could be controlled ~~by the backing~~ within the assembly, such as a shear failure in a cement backer unit or within other layers within the system.

**13.3.2.4** *Installation requirements* — Installation of adhered masonry veneer units must comply with TMS 602. Lath and scratch coat are not required when adhered masonry veneer units are applied directly to ~~certain backings (~~concrete, concrete masonry, or cement backer units~~)~~ due to adequate bond.

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 12 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 0 | *Did not vote* |

**Subcommittee Comments:** None.

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| **Committee: Main Committee** | | **Ballot #: 20** |
|---|---|---|
| **Item #: 20-VG-066** | | |
| **Technical Contact/Email:** | Brian E. Trimble, PE, btrimble@imiweb.org, (703) 300-0109 | |
| **Draft Document Dated:** | **10/26/2021** | |
| **Response to Public Comment No.:** 66 | | |
| **This ballot item proposes the following response to the Public comment:** | | |
| ☒ | Committee agrees with public comment, change is proposed | |
| ☐ | Committee agrees comment has merit, but proposed changes are not completely consistent with public comment | |
| ☐ | Committee disagrees with public comment and no changes are proposed | |
| ☐ | Committee unable to fully develop a response to public comment | |
| ☐ | Public comment only requires a response, no change to document | |

**Public comment:**
A minimum factor of safety of 1.5 should be required for the stability analysis to maintain a level of safety consistent with Table 13.2.1.5.

**Response:** Changes are made consistent with public comment.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)*

**Code:**

**13.2.1.5** *Out-of-plane deflection* — For the purpose of maintaining veneer stability, backings shall have a calculated deflection less than or equal to those in Table 13.2.1.5, or a stability analysis shall be performed <u>to demonstrate a minimum factor of safety of 1.5 against loss of stability under strength level loading</u>. The veneer ties and backing shall be designed for any additional forces determined from the stability analysis.

**Code Commentary: NONE**

*[The following paragraph of 13.2.1.5 commentary is included for voter convenience to show that Table 13.2.1.5 is based on a factor of safety of 1.5. No changes are being proposed.]*

**13.2.1.5** *Out-of-plane deflection* — The deflection of the backing creates the potential for instability in the veneer by creating an eccentricity between the weight of the veneer and the geometric area of the veneer that is available to resist that load. Figure CC-13.2-1 depicts the condition producing the greatest eccentricity of the vertical load on the section at mid-height, which will occur when the veneer is cracked at mid height. Veneers that remain elastic or have multiple levels of cracking will have a reduced eccentricity at mid-height. If the strength level displacement of the veneer at mid-height due to the displacement of the backing, $\delta_u$, is expressed as a ratio of the height of the backing, $h_b/x$ (for example $h_b/360$), the resulting eccentricity of the veneer weight is $h_b/2x$. As shown in Figure CC-13.2-1, the maximum permitted eccentricity of the load before loss of stability is $t_{sp}/2$.

In order to maintain a factor of safety of 1.5 against loss of stability, the maximum eccentricity of the veneer gravity load is limited to $t_{sp}/3$. Equating $t_{sp}/3$ to $h_b/2x$ results in $x$ being equal to $3/2(h_b/t_{sp})$. The value of $h_b$ is the effective height of the backing, and not the distance between supports of the veneer, as shown in Figure CC-13.2-2.

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 12 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:** None.

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| Committee: Main Committee | | Ballot #: 20 |
|---|---|---|
| **Item #: 20-VG-097A** | | |
| **Technical Contact/Email:** | Brian E. Trimble, PE, btrimble@imiweb.org, (703) 300-0109 | |
| **Draft Document Dated:** | **10/26/2021** | |
| **Response to Public Comment No.:** 097 | | |
| **This ballot item proposes the following response to the Public comment:** | | |
| ☒ | Committee agrees with public comment, change is proposed | |
| ☐ | Committee agrees comment has merit, but proposed changes are not completely consistent with public comment | |
| ☐ | Committee disagrees with public comment and no changes are proposed | |
| ☐ | Committee unable to fully develop a response to public comment | |
| ☐ | Public comment only requires a response, no change to document | |

**Public comment:**
A withdrawn negative on Ballot item 17-VG-022A asked that the phrase "or, where sheathing is present, into the structural member behind the sheathing;" be added in four places after "penetration into backing." Although the withdrawal was unconditional, the negative voter did ask the VG subcommittee to consider the negative, which it never did. The addition of this phrase should be considered. *[Page 234, Line 1-27]*

**Response:** Changes are made consistent with public comment.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)*

**Code:**

**Table 13.2.2.3 – General prescriptive anchored veneer requirements**

| Backing | Veneer Tie Type | Maximum Specified Cavity Width | Other requirements |
|---|---|---|---|
| Wood Light Frame | Corrugated Sheet-metal | 1 in. (25.4 mm) [1] | Fastener: Minimum 2.5 in. (63.5 mm) x 0.131 in. (3.33 mm) ring-shank nail(s) with minimum 1.5 in. (38.1 mm) penetration into backing; or No. 10 screw(s) with ¾ in. (19.0 mm) penetration into backing. <u>Where sheathing is present, the minimum penetration shall be into the structural member behind the sheathing.</u><br><br>Locate fastener within ½ in. (12.7 mm) of the 90-degree bend in the veneer tie.<br><br>The limiting $p_{veneer}$ values for prescriptive design method shall be 75 percent of those listed in Table 13.2.1.1.<br><br>Corrugated ties shall not be used on veneers greater than 30 ft (9.14 mm), or 38 ft (11.58 m) at a gable, in height. |

| | | | |
|---|---|---|---|
| | Sheet Metal | 4 in. (101.6 mm) [1] | Fastener: Minimum #10 screw(s) with 1.5 in. (38.1 mm) penetration into backing<u>, or, where sheathing is present, into the structural member behind the sheathing</u>.<br><br>Exterior veneer exceeding 30 ft (9.1 m), or 38 ft (11.58 m) at a gable, in height above the vertical support shall be designed and detailed to provide for differential movement. |
| | Adjustable | 6 in. (152 mm) [1] | Fastener: Minimum #10 screw(s) with 1.5 in. (38.1 mm) penetration into backing<u>, or, where sheathing is present, into the structural member behind the sheathing</u>.<br><br>Exterior veneer exceeding 30 ft (9.1 m), or 38 ft (11.58 m) at a gable, in height above the vertical support shall be designed and detailed to provide for differential movement. |

**Code Commentary: NONE**

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 12 | *Affirmative* | 0 | *Affirmative w/ comment* | 0 | *Negative* | 0 | *Abstain* | 0 | *Did not vote* |

**Subcommittee Comments:** A sentence was added to the rationale based on a comment from a non-voting member.

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| **Committee: Main Committee** | | **Ballot #: 20** |
|---|---|---|
| **Item #: 20-VG-106, 143, 170** | | |
| **Technical Contact/Email:** | Brian E. Trimble, PE, btrimble@imiweb.org, (703) 300-0109 | |
| **Draft Document Dated:** | **10/26/2021** | |
| **Response to Public Comment No.:** 106, 143 and 170 | | |
| **This ballot item proposes the following response to the Public comment:** | | |
| ☒ | Committee agrees with public comment, change is proposed | |
| ☐ | Committee agrees comment has merit, but proposed changes are not completely consistent with public comment | |
| ☐ | Committee disagrees with public comment and no changes are proposed | |
| ☐ | Committee unable to fully develop a response to public comment | |
| ☐ | Public comment only requires a response, no change to document | |

**Public comment:**
**PC 170** (Page 243, Line 1; Section 13.3.2.1)
The new standard ASTM C1823 "Standard Test Method for Shear Bond Strength of Adhered Dimension Stone" has recently been adopted and should be incorporated into the code and commentary as appropriate.

**PC 106** (Page 243, Line 1-9; Section 13.3.2.1)
13.3.2.1 references ASTM C482 which is a laboratory shear bond test for adhered tile that cannot be performed in situ on an actual installation. It should be clearly stated that ASTM C482 is a quality assurance test performed prior to the intended installation. ASTM C482 protocol is based on using a fresh mortar bed at a certain ratio of sand, cement and water, and then bonding the tile to it with a portland cement paste. That is not a realistic representation of how tile is installed today.....

**PC 143** – Page 243, Line 1; Section 13.3.2.1
The wording of this section exempts most AMV units from any requirement for bond strength between units and backing. While compliance with the listed ASTM standards should provide a reasonable assurance for the bond strength between the unit and the setting mortar, the standards give no assurance of the bond strength between the setting bed and the backup.

**Response:** Changes are made consistent with the comments. ASTM C1823 is added which can provide a way to determine bond strength between the setting bed and the backup as noted in PC 143.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)*

**Code: NONE**

**Code Commentary:**

**13.3.2.1** *Permitted units* – The design strengths are based on bond between the unit and the mortar, and the backing and the mortar. The strength of other components in the system also needs to be considered. The

strength could be controlled by the backing, such as a shear failure in a cement backer unit or within other layers within the system.
ASTM C482 is a laboratory test method to qualify that an adhered masonry unit develops adequate bond strength at its bonding surface with a specified adhesive over a specified substrate. The method is often adapted to include materials that will be used in construction. ASTM C482 is not intended to evaluate the bond strength between various combinations of masonry units, setting bed mortar, membranes, and backings. Alternately, ASTM C1823 is a test method that is used in the field to measure the shear bond strength in situ. This test method includes failure modes beyond the normal unit and mortar bond, therefore failures that occur within the units or within the substate may not be appropriate for qualifying materials (Dillon and Dalrymple (2021)).

**References, Chapter 13**

ASTM C1823–20 (2020). "Standard Test Method for Shear Bond Strength of Adhered Dimension Stone," ASTM International, www.astm.org.

Dillon, P. B. and Dalrymple, G. A. (2021). "In-Field Shear Bond Strength Testing of Adhered Masonry Veneer." *14th Canadian Masonry Symposium*, Montreal, Canada.

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 10 *Affirmative* | 1 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 0 *Did not vote* |

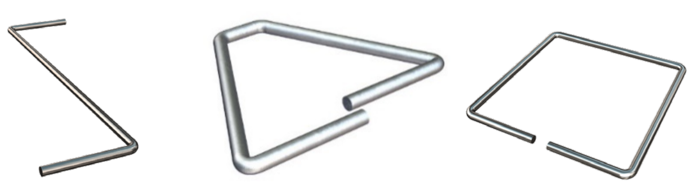
**Subcommittee Comments:** A correction to the reference was made based on the comment.

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| Committee: Main Committee | | Ballot #: 20 |
|---|---|---|
| **Item #:** 20-VG-151A | | |
| **Technical Contact/Email:** | Brian E. Trimble, PE, btrimble@imiweb.org, (703) 300-0109 | |
| **Draft Document Dated:** | **10/26/2021** | |
| **Response to Public Comment No.:** 151 | | |
| **This ballot item proposes the following response to the Public comment:** | | |
| ☒ | Committee agrees with public comment, change is proposed | |
| ☐ | Committee agrees comment has merit, but proposed changes are not completely consistent with public comment | |
| ☐ | Committee disagrees with public comment and no changes are proposed | |
| ☐ | Committee unable to fully develop a response to public comment | |
| ☐ | Public comment only requires a response, no change to document | |

**Public comment:**
Table 13.2.2.4 - Veneer Tie Requirements - The requirements for the Tie Type - Unit Wire appear to have been written for a "Z" shaped wire tie, which is in fact referenced in the diagram in the commentary, same section. The requirements call specifically to "....... have ends bent to form an extension from the bend at least 2" long". For a Z-shaped tie this is fine, as the 2" extension will develop the necessary pullout strength, however, Z-shaped ties are nearly non-existent today. Further compounding the confusion, later in the table, under the Tie Type - Adjustable, the requirement for wire components of adjustable ties is for those ties to conform with the requirements under the Tie Type - Unit Wire. The wire components of the vast majority of adjustable veneer ties are either pintles or triangular ties, neither of which unambiguously conform to the language found within Unit Wire. If the intention is to provide a minimum of 2" of wire to be embedded in a mortar joint, please reword the Unit Wire requirements to state that instead of having commonly used ties conform to non-existent product requirements. *[Page 235, Line 27]*

*For voter's convenience here are the unit type of wire ties mentioned in the Public Comment:*



**Response:** Changes are made consistent with public comment by providing requirements that apply to box and triangular unit ties.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)*

**Code:**

**Table 13.2.2.4** ***[remainder of Table is not changed]***

1) Minimum W1.7 (MW11) wire ~~and have ends bent to form an extension from the bend~~ where the length of the wire that is parallel to and within the veneer be at least 2 in. (50.8 mm) long.

2) Drips are not permitted.

**Code Commentary: NONE**

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 12 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:** None.

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| Committee: Main Committee | | Ballot #: 20 |
|---|---|---|
| **Item #: 20-VG-155** | | |
| **Technical Contact/Email:** | Brian E. Trimble, PE, btrimble@imiweb.org, (703) 300-0109 | |
| **Draft Document Dated:** | **10/26/2021** | |
| **Response to Public Comment No.:** 155 | | |
| **This ballot item proposes the following response to the Public comment:** | | |
| ☐ | Committee agrees with public comment, change is proposed | |
| ☐ | Committee agrees comment has merit, but proposed changes are not completely consistent with public comment | |
| ☒ | Committee disagrees with public comment and no changes are proposed | |
| ☐ | Committee unable to fully develop a response to public comment | |
| ☐ | Public comment only requires a response, no change to document | |

**Public comment:**
In TMS 602, sections 1.3, 2.3C, and Table SC-5, the document references ASTM Standard specifications C503 (Marble), C568 (Limestone) C615 (Granite), C616 (Quartz-Based), and C629 (Slate). Yet nowhere does it refence C1526 (Serpentine) or C1527 (Travertine). Why are these two standards omitted? *[Page 319, Line 3; Article 1.3, 2.3 C, and Table SC-5]*

**Response:** There are some concerns with the use of serpentine and travertine stone for both anchored and adhered masonry veneer. These materials may not have the appropriate durability and may contain asbestos as is the case with serpentine. The panel size of travertine is larger than used for adhered masonry veneer applications in this code. Based on these concerns, these materials are not included in this code at this time. No changes are made.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)*

**Code: NONE**

**Code Commentary: NONE**

**Specification: NONE**

**Specification Commentary: NONE**

| Subcommittee Vote: | | | | |
|---|---|---|---|---|
| 12 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:** None.

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| Committee: Main Committee | | Ballot #: 20 |
|---|---|---|
| Item #: 20-VG-158, 165 | | |
| Technical Contact/Email: | Brian E. Trimble, PE, btrimble@imiweb.org, (703) 300-0109 | |
| Draft Document Dated: | 10/26/2021 | |
| Response to Public Comment No.: 158 and 165 | | |
| This ballot item proposes the following response to the Public comment: | | |
| ☒ | Committee agrees with public comment, change is proposed | |
| ☐ | Committee agrees comment has merit, but proposed changes are not completely consistent with public comment | |
| ☐ | Committee disagrees with public comment and no changes are proposed | |
| ☐ | Committee unable to fully develop a response to public comment | |
| ☐ | Public comment only requires a response, no change to document | |

**Public comment:**
**PC 158**
A TAC comment suggested prohibiting open jointed adhered veneer in freeze-thaw climates. There was no action taken and the rationale noted, incorrectly, that the TAC comment only required a response - the TAC comment said 'do not allow open joints...' which seems like direction to remove, or consider removing, the allowed open joints in the freeze-thaw zones.... *[Page 369, Line 25-29 ; Article 3.3 D 4.c]*

**PC 165**
There has been considerable discussion about the appropriate applications for the use of dry stack or dry-fit joint applications for adhered veneers. Some additional language should be added that alerts users to possible issues in certain climates. Consider adding language to the commentary of Section 13.3.1.3 at the end:
"Since water penetration is a critical issue for adhered masonry veneer, consideration should be given to appropriate drainage layers within the adhered veneer system. Adhered masonry veneer with tight-fit joints (joints between adhered veneer units that are not purposely filled with mortar), also referred to as dry-stack veneer, should be carefully considered in wet climates that include freeze thaw conditions and should closely follow the installation requirements in TMS 602 Article 3.3 C." *[Page 242, Line 82-85 ; Section Commentary 13.3.1.3]*

**Response:** The Committee agrees with the comments and modifications are made.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)*

**Code: NONE**

**Code Commentary:**

**13.3.1.3** *Water penetration resistance* — Water penetration through the exterior veneer is expected. The wall system must be designed and constructed to prevent water from entering the building. Information and references on designing and detailing for water penetration resistance are located in Section 13.1.2.1.

Since water penetration is a critical issue for adhered masonry veneer, consideration should be given to appropriate drainage layers within the adhered veneer system. Use of adhered masonry veneers with tight-fit joints (joints between adhered veneer units that are not purposely filled with mortar), also referred to as dry-stack veneer, should be carefully considered in wet climates that include freeze thaw conditions and should closely follow the installation requirements in TMS 602 Article 3.3 C.

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 12 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:** The second sentence of the commentary was modified based on a comment from a non-voting member.

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| Committee: Main Committee | | Ballot #: 20 |
|---|---|---|
| **Item #:** 20-VG-174 | | |
| **Technical Contact/Email:** | Brian E. Trimble, PE, btrimble@imiweb.org, (703) 300-0109 | |
| **Draft Document Dated:** | **10/26/2021** | |
| **Response to Public Comment No.:** 174 | | |
| **This ballot item proposes the following response to the Public comment:** | | |
| ☐ | Committee agrees with public comment, change is proposed | |
| ☒ | Committee agrees comment has merit, but proposed changes are not completely consistent with public comment | |
| ☐ | Committee disagrees with public comment and no changes are proposed | |
| ☐ | Committee unable to fully develop a response to public comment | |
| ☐ | Public comment only requires a response, no change to document | |

**Public comment:**
Clay masonry walls should be included in Section 13.3.2.4 as an appropriate backing for adhered veneer without the need for lath and scratch coat. However, the section must include language that not all clay masonry backings are appropriate, for example an existing brick veneer wall or a brick that has a glazed or smooth face or an existing wall that is weathered and spalled. *[Page 243, Line 27-30]*

**Response:** This issue was partially addressed during TAC comments, but did not get fully resolved. The proposal is based on previous ballot items and any negatives or comments associated with those items.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)*

**Code:**

**13.3.2.4** *Installation requirements* – Lath and scratch coat shall not be required when adhered masonry veneer units are applied directly to concrete, ~~concrete~~ <u>unglazed clay or concrete</u> masonry, or cement backer units free of coatings, debris, membranes, or similar materials that would inhibit bond to the backing.

**Code Commentary:**

**13.3.2.4** *Installation requirements* – Installation of adhered masonry veneer units must comply with TMS 602. Lath and scratch coat are not required when adhered masonry veneer units are applied directly to certain backings ~~(concrete, concrete masonry, or cement backer units) due to~~ <u>that provide</u> adequate bond. <u>Differential movement between adhered veneer units and the backing should be considered as their incompatibility may result in cracks or debonding.</u>
When concrete<u>, clay masonry</u> or concrete masonry walls are smooth<u>, have a glazed coating,</u> or where good bond cannot be achieved<u>,</u> adhered veneer systems should be installed over lath. The surfaces intended to receive adhered units must have a rough texture to ensure good mortar bond. ICRI Technical Guideline 310.2 (ICRI 2013) provides information on ~~concrete~~ surface preparation, including information on Concrete Surface Profile, a

standardized method to measure concrete surface roughness. A Concrete Surface Profile equal to or greater than 2 is usually acceptable for the installation of adhered veneer over concrete and masonry assemblies but verification for specific project conditions may be required. When testing is warranted due to surface texture of the substrate or the presence of a membrane or coating that may inhibit bond, the procedures of Section 13.3.3 should be followed.

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 12 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:** None.

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| Committee: Main Committee | | Ballot #: 20 |
|---|---|---|
| Item #: 20-VG-209A | | |
| Technical Contact/Email: | Brian E. Trimble, PE, btrimble@imiweb.org, (703) 300-0109 | |
| Draft Document Dated: | 10/26/2021 | |
| Response to Public Comment No.: 209 | | |
| This ballot item proposes the following response to the Public comment: | | |
| ☒ | Committee agrees with public comment, change is proposed | |
| ☐ | Committee agrees comment has merit, but proposed changes are not completely consistent with public comment | |
| ☐ | Committee disagrees with public comment and no changes are proposed | |
| ☐ | Committee unable to fully develop a response to public comment | |
| ☐ | Public comment only requires a response, no change to document | |

**Public comment:**
This is far from being a comprehensive list and does not serve as a suitable introduction to the discussion under 13.1.2.2. *[Page 223, Line 75-80]*

**Response:** Changes are made consistent with the Public Comment.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)*

**Code: NONE**

**Code Commentary:**

**13.1.2.2** *Deformation and differential movement* — Deformations include<u>, but are not limited to,</u> out-of-plane deflection of the backing, vertical deflection of horizontally spanning support elements, and in-plane movement due to absolute and relative story drift. See Sections 13.2.1.5 and 13.3.1.2 for deflection requirements specific to anchored and adhered veneers, respectively.

**Specification: NONE**

**Specification Commentary: NONE**

| Subcommittee Vote: | | | | |
|---|---|---|---|---|
| 12 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:** None.

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| | | |
|---|---|---|
| **Committee: Main Committee** | | **Ballot #: 20** |
| **Item #: 20-VG-210, 212A** | | |
| **Technical Contact/Email:** | Brian E. Trimble, PE, btrimble@imiweb.org, (703) 300-0109 | |
| **Draft Document Dated:** | **10/26/2021** | |
| **Response to Public Comment No.:** 210 and 212 | | |
| **This ballot item proposes the following response to the Public comment:** | | |
| ☐ | Committee agrees with public comment, change is proposed | |
| ☒ | Committee agrees comment has merit, but proposed changes are not completely consistent with public comment | |
| ☐ | Committee disagrees with public comment and no changes are proposed | |
| ☐ | Committee unable to fully develop a response to public comment | |
| ☐ | Public comment only requires a response, no change to document | |

**Public comments:**
**Comment 210** – "water penetration into the building"...What exactly is the extent of "into the building"...into the backing??...into interior space?? This statement must be consistent with the extent of water penetration permitted by the applicable building code. *[Page 226, Line 66]*

**Comment 212** – "...entering into the building." What exactly is the extent of "into the building"...into the backing??...into interior space?? Such statements must be consistent with that permitted by the applicable building code. *[Page 230, Line 88]*

*For voter's convenience here are the paragraphs referenced in the Public Comment along with text from the 2021 IBC regarding weather protection as noted in the Public Comment that is required for all exterior walls:*

design, detailing, and construction.

e) Water will penetrate the veneer, and the wall system should be designed, detailed, and constructed to prevent water penetration into the building.

f) Corrosion and fire resistance should be considered as

**13.2.1.8** *Water Penetration Resistance* — Flashing and weep holes in exterior veneer wall systems shall be designed and detailed to resist penetration of water into the building interior. A minimum 1 in. (25.4 mm)

Page 230

**13.2.1.8** *Water Penetration Resistance* — Water penetration through the exterior veneer is expected. The wall system must be designed and constructed to prevent water from entering the building.

**1402.2 Weather protection.** *Exterior walls* shall provide the building with a weather-resistant *exterior wall envelope*. The *exterior wall envelope* shall include flashing, as described in Section 1404.4. The *exterior wall envelope* shall be designed and constructed in such a manner as to prevent the accumulation of water within the wall assembly by providing a *water-resistive barrier* behind the exterior *veneer*, as described in Section 1403.2, and a means for draining water that enters the assembly to the exterior. Protection against condensation in the *exterior wall* assembly shall be provided in accordance with Section 1404.3.

**Response:** A designer has the prerogative to determine what level of design and detailing is required for a particular building, especially for different climates. Therefore, having a general statement on water penetration is appropriate. It would be appropriate to specify a certain point or plane to resist water penetration (which we can define rather precisely) rather than "into the building interior," which is difficult to define. Therefore, "beyond the drainage space" is recommended to define that plane. This is consistent with requirements found in the building code (2021 IBC).

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown underlined and deletions are shown ~~struck-through~~.)*

**Code:**

**13.2.1.8** *Water Penetration Resistance* — Flashing and weep holes in exterior veneer wall systems shall be designed and detailed to resist penetration of water ~~into the building interior~~ beyond the drainage space and insulation. A minimum 1 in. (25.4 mm) ….

**Code Commentary:**

**13.2.1** *General requirements for anchored veneer*

….

e) Water will penetrate the veneer, and the wall system should be designed, detailed, and constructed to prevent water penetration ~~into the building~~ beyond the drainage space and insulation.

**13.2.1.8** *Water Penetration Resistance* — Water penetration through the exterior veneer is expected. The wall system must be designed and constructed to prevent water from ~~entering the building~~ passing beyond the drainage space and insulation.

**Specification: NONE**

**Specification Commentary: NONE**

| **Subcommittee Vote:** | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 1 *Affirmative w/ comment* | 0 *Negative* | 0 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:** The comment questioned whether this was the best wording and the subcommittee proposes that this is the most appropriate language.

# 2022 TMS 402/602 Committee
# Proposed Change to Masonry Standard

| Committee: Main Committee | | Ballot #: 20 |
|---|---|---|
| Item #: 20-VG-214A | | |
| Technical Contact/Email: | Brian E. Trimble, PE, btrimble@imiweb.org, (703) 300-0109 | |
| Draft Document Dated: | 10/26/2021 | |
| Response to Public Comment No.: 214 | | |
| This ballot item proposes the following response to the Public comment: | | |
| ☐ | Committee agrees with public comment, change is proposed | |
| ☐ | Committee agrees comment has merit, but proposed changes are not completely consistent with public comment | |
| ☒ | Committee disagrees with public comment and no changes are proposed | |
| ☐ | Committee unable to fully develop a response to public comment | |
| ☐ | Public comment only requires a response, no change to document | |

**Public comment:**
13.2.1.8...For water penetration resistance...it is interesting that so many redundancies, such as air space and weep holes, etc., are required for water management for conventional (anchored) masonry veneer systems, but so little is required for adhered veneer with respect to water management! How is this possibly rationalized???? *[Page 230, Line 38-40]*

**Response:** Adhered veneers require more analysis since they can be designed as a barrier wall or a drainage wall. Adhered veneer could also be considered as "newer" wall systems as compared to anchored veneer walls and thus don't have as many prescriptive requirements. This committee will consider more prescriptive requirements for adhered veneer as more research is conducted and experience is gained on this wall system but the requirements, especially in regard to water penetration, are deemed as minimum levels appropriate for a building code at this time. No changes are made.

**PROPOSED CHANGES:** *(Only the suggested change(s) being balloted are proposed for consideration. Supplementary text included for clarity, but not proposed for modification, is not part of this ballot item. Additions are shown <u>underlined</u> and deletions are shown ~~struck-through~~.)*

**Code: NONE**

**Code Commentary: NONE**

**Specification: NONE**

**Specification Commentary: NONE**

| Subcommittee Vote: | | | | |
|---|---|---|---|---|
| 11 *Affirmative* | 0 *Affirmative w/ comment* | 0 *Negative* | 1 *Abstain* | 0 *Did not vote* |

**Subcommittee Comments:** None.