Minutes of ACI-ASCE Committee 352 Meeting

Joints and Connections in Monolithic Concrete Structures

ACI Spring 2013 Convention

Sunday, April 14, 2013; 2:00 pm – 5:00 pm

Meeting Room "C-M100", Hilton, Minneapolis, Minnesota

ATTENDANCE

Members Present: John Bonacci, Jim Cagley, Min Yuan Cheng, Damon Fick, Luis Garcia, Mary Beth Hueste, Shyh-Jiann Hwang, Mike Kreger, Jim LaFave, Roberto Leon, Jack Moehle, Gustavo Parra-Montesino, Myoungsu (James) Shin, Jim Wight, Loring Wyllie

Members Absent: Sergio Alcocer, Burcu Burak, Marvin Criswell, Jeff Dragovich, Catherine French, Kara Hartleib, Thomas Kang, Ted Krauthammer, Douglas Lee, Hung-Jen (Harry) Lee, Dawn Lehman, Cheng-Ming Lin, Nilanjan Mitra, Voula Pantazopoulou, Ian Robertson, M. Saiid Saiidi, Jorge Segura, Bahram Shahrooz, John Wallace

Associate Members: James Deaton, Bohwan Oh, Gangolu Appa Rao, Daniel Reider

Visitors: Anna Birely, Joanne Browning (TAC), Lou Colarusso, Remy Lesquene

1. Call to Order and Introductions

Committee 352 Chair Mary Beth Hueste called the meeting to order at approximately 2:05pm, after distributing copies of the meeting agenda. All individuals in the meeting room then introduced themselves.

2. Approval of Minutes from the Fall 2012 Committee 352 Meeting in Toronto, Ontario, CA

A motion (by Jim Wight, seconded by Luis Garcia) was made to approve the Fall 2012 meeting minutes (from the convention held in Toronto, Ontario, CA), which had previously been posted to the committee website and were also available in hard copy format at the meeting. No additional discussion was requested and the motion was approved by acclamation.

3. Results of the second ballot for slab-column connection design Example 2 to supplement ACI 352.1R-11 (D. Fick, M. Hueste).

A slab-column connection design example (Example 2 – Exterior Type 1 Connection) was balloted on March 11, 2013 (closed on April 11, 2013). This is the second ballot for the slab-column connection examples to accompany ACI 352.1R-11. The ballot passed with 16 Affirmative Votes, 5 Affirmative with Comment votes, 3 Negative votes, and 9 ballots not returned.

Damon Fick summarized the updated Example 2 based on the comments received from eight Committee 352 members. A complete list of comments is included as an Appendix to these meeting minutes. Specific comments discussed and recommendations made by committee members include:

- Lee negative – p. 1, line 10: The committee agreed to revise the calculated result of 6.8 in. to 6.75 in.
- Kang negative – p.2, line 22: The text "Because the connection is exterior, γ_f cannot be increased" has been removed from the example. This check will be illustrated for the interior connection example.
- Lee negative – p. 4, line 6: The committee agreed to keep 0.65 in. as shown in the example to reflect the least number of significant figures used in the calculation. Note: This value for *a* becomes 1.1 in. using #5 bars (see Moehle negative, p.4 line 21).
- Lee negative – p. 4, line 8: Result will be revised using three significant figures for *d* (6.75 in.).
- Lee negative – p. 4, line 15: Result will be revised using three significant figures for *d* (6.75 in.).
- Lee negative – p. 4, line 19: Result will be corrected using revised M_n value.
- Moehle negative – p. 4, line 21: The committee discussed the use of a reduced γ_f for the example and agreed with Jack's view that the example suggests that less steel in the effective transfer width is preferred. It was agreed to complete the reinforcement design using γ_f = 1.0 and then show the alternate procedure where part of the unbalanced moment is transferred through eccentric shear stress around the slab. Note: The increased moment using γ_f = 1.0 resulted in the use of #5 bars.
- Lee negative – p. 4, line 27: The committee agreed the subtitle should be revised per Doug's suggestion.
- Lee negative – p. 4, line 28-31: A note has been added to the example clarifying the reinforcement spacing requirement.
- Moehle negative – p. 6, line 28: Jack pointed out the 25 or 30% increase in development length was for top bars terminating without a standard hook. Committee members agreed to replace these calculations from Example 2 with text stating the required lap is l_d and because standard hooks are used the termination requirements for bars inside and outside the column core do not apply. Members also discussed options for clarifying the condition of standard hooks in 352.1R. A suggestion was made to check with TAC to see if this could be accomplished through an editorial change.
- Lee negatives – p. 8, Figure E2.2(c): The committee discussed the information shown in the figures and how much detail should be included. John Bonacci suggested a preamble to the example and the committee agreed to address some of Doug's concerns with a statement clarifying the intent of the figures. The committee found these 4 comments non-persuasive by a unanimous vote (14 voting members present) on the question called by Jim Wight and seconded by Jack Moehle.
- Moehle comment – p. 3, line 14: Committee agreed to add the calculation showing the transfer of the centerline column moment to the centroid of the critical section in addition to text stating it is conservative to use the centerline column moment.

- Kang comment – p.4, line 6: Committee agreed to leave the calculation as is without including the compression reinforcement.
- Moehle comment – p. 5, line 20: Jack stated testing at Berkeley has shown that for slabs, reinforcement spacing larger than 0.75h did not influence the torsional response. Committee agreed that this should be looked into further for the next edition of 352.1R document.
- Burak comment – p. 5, line 38: Mary Beth suggested and the committee agreed to clarify the sentence to avoid adding two additional bars to the integrity reinforcement requirement of 6.1.5c.
- Moehle comment – p.5, line 42: Jack suggested and the committee agreed to use a dimension limit for the reinforcement parallel to the edge to illustrate the recommendation for extending these bars a minimum of 2h past the column face.
- Robertson comment – p.6, line 20: Committee suggested contacting Ian for clarification.
- Kreger comment – p.6, line 43: The lap length of the 180 hook is required to be 17.0 in. as calculated per S6.4.6. For convenience, the development length of the hook, l_{dh} is calculated to be 19.0 in. The committee agreed to use 19.0 in. on Figure E2.3c, showing the dimension string from the back of the hook.
- Wight comment – p.5, line 9 and p.6, line 13: The committee agreed that using 4-#5 bars instead of the 6-#4 bars was practical and would reduce bar congestion through the column.
- The committee agreed to remove the spacing text (i.e. @ 3in.) from the bar details.

<u>ACTION ITEMS:</u>

- The task group will incorporate the recommendations made by the committee and the example will be balloted again to resolve the negative votes.
- Chair Hueste on behalf of the committee will check with TAC to see if an editorial change can be made to ACI 352.1R-11 for clarification to the termination requirements of top bars in an exterior connection.
- The committee agreed to investigate the spacing requirements of Section 6.1.7b to see if larger spacing for slabs is appropriate. This item will be considered new business.

4. Preview of Slab-Column Connection Design Example 3 – Corner Type 1 Connection (M.-Y. Cheng)

M.-Y. Cheng presented a preview of the corner type design example. He asked the committee for an interpretation of “edge connection” in Section 6.1.5(b) and if it applied to the corner condition. Jack Moehle stated Section 6.1.5(b) is intended for interior connections and is not applicable for corner connections.

Jack Moehle inquired about the method used to combine Mu_1 and Mu_2. Min Yuan’s example considered each direction separately. The committee discussed briefly Jack’s question about thoughts on adding Mu_1 and Mu_2.

Jack asked if γ_f can be taken equal to 1.0 for corner connections, noting the intent was to include this provision.

Chair Hueste asked committee members to let her know if they are interested in helping with the remaining connection examples.

ACTION ITEMS:

- The task group will report back to the committee on permitted values γ_f of for corner columns.

5. Task Group Updates to Revise and Update *ACI 352R-02 (M. Hueste)*

Chair Hueste asked members of the task groups to provide a summary of progress on efforts to update ACI 352R-02. Additional volunteers to help with the work of the task groups were also requested [* indicates task group leader(s)].

- TG1: Overall Technical and Editorial Review and General Updates (J. LaFave*, M. Hueste*, J. Bonacci, D. Fick, T. Kang, V. Pantazopoulou)
  - Chair Hueste presented a proposed table of contents based on Jack Moehle's suggestions from the Fall 2012 Toronto committee meeting.
  - Gustavo Parra-Montesino inquired about the beam strength, column strength, and joint shear. Jack Moehle clarified that beam strength should actually be calculated earlier (Ch. 3.) Gustavo suggested fiber reinforcement should be included under anchorage of beam and column reinforcement. S. J. Hwang asked how committee felt about also including high strength materials with anchorage of beam reinforcement. Chair Hueste stated the committee is flexible at this point.
  - Jack Moehle said column strength is typically done at the end. Loring Wyllie pointed out however, in the previous document, all the examples had to be re-worked because the columns weren't big enough when shear was checked at the end. Loring suggested a preliminary design in the beginning, then working through the details. Gustavo Parra-Montesino pointed out some of these issues should be handled with preliminary statements about design considerations. Gustavo also suggested fiber reinforcement should be a separate section under detailing considerations.
  - Jack Moehle noted the old 352 document has poor details. Mary Beth suggested a comment to clarify transverse reinforcement spacing/requirements/details could be added.
  - Gustavo Parra-Montesino noted the current approach is strength-based. He suggested starting the process of considering designs that may reduce damage. Perhaps suggesting minimums, but then providing other recommendations. Jim LaFave asked if this would be qualitative. Chair Hueste noted some comments suggest 352/ACI is becoming too empirical, but agreed that appropriate guidance would be helpful.

- Loring inquired about the offset beam from column centerline case – Jim Lafave thought it should be in Section 4.1.
- Chair Hueste suggested that the research needs section should be reviewed at some point. Jack asked if research needs appendix is necessary, since current funding trends don't typically rely on this information. Jack suggested a research section for joints with different aspect ratios (i.e. very deep and narrow joints).

- TG2A: Headed reinforcement applications in beam-column joints & connections (T. Kang*, H. Lee, M. Shin)

  - John Bonacci provided a brief summary of previous research that was presented in Toronto.
  - Loring Wyllie reviewed previous literature and indicated data was scant. Gustavo suggested reviewing data/information related to compressive strength of joints to avoid local crushing of concrete.

- TG2B: Effective joint area/strength reduction factors for beam-column joints for shear resistance (S.-J. Hwang*, J. LaFave*, H.-J. Lee, B. Burak, M. Shin)

  - Jim Lafave provided a summary of current related work and will present at the ASCE Structures Congress. Mary Beth suggested making presentation to 352 in the Fall.

- TG2C: High-strength materials in beam-column connections (H.-J. Lee*, S.-J. Hwang)

  - Shyh-Jiann Hwang presented recent work on high strength reinforcement research that TG2C related to previous research/database and the assessment of key design parameters. Current design provisions can be extended to cover high-strength reinforcement limitations of bar f_y and concrete f'c could be liberated. Ongoing progress is being made on a model to estimate the degradation of joint shear, and to construct a database.
  - A presentation is being prepared by Harry Lee and will be updated for the ASCE Structures Congress in Pittsburgh. Gustavo suggested focusing on ductility instead of drift and that joint behavior may be dependent on other factors (shear stress, flexural capacity).

- TG2D: Fiber-reinforced beam-column connections (G. Parra-Montesinos*)
  - No new items to report.

- TG3: Updates and addition to the examples (task group leader to be determined)
  - No activities underway.

Chair Hueste concluded the discussion by suggesting the task group members should begin working toward balloting different chapters separately.

6. Technical sessions

- Mini-track for 2013 Structures Congress (M. Hueste and J. LaFave)
    - Chair Hueste noted presentations by (1) B. Li, (2) Hwang and Lafave, (3) Hwang and Lee, and (4) Burak will be made from 8:00-9:30.
- Suggestions for future technical sessions
    - Suggestions for future technical sessions were not discussed.

7. Other business / presentations / new business (please notify M. Hueste in advance of any items in these categories)

Chair Hueste discussed the following other business items:

- Committee 421 Report Review – Doug Lee provided a review and comments on behalf of Committee 352 and these were passed along to TAC. Additional feedback provided by committee members was also submitted.
- Opportunities are available to develop educational products, including ACI online CEU program exam, ACI e-learning courses, or ACI seminar. Please let Chair Hueste know if you are interested.

8. Schedule for the next committee meeting

Committee discussed alternative meeting times and it was agreed current Sunday 2:00-5:00pm meeting times are reasonable. Chair Hueste encouraged task groups to be in communication between committee meetings and thanked everyone for attending and participating.

9. Adjournment

A motion (S.-J. Hwang, seconded by Jack Moehle) was made to adjourn the regular meeting. No additional discussion was requested and the motion was approved by acclamation. The regular meeting was adjourned at approximately 4:55pm.

Respectfully submitted,

Damon R. Fick	Mary Beth D. Hueste
Member, ACI-ASCE Committee 352	*Chair, ACI-ASCE Committee 352*
(Secretary, effective Feb. 13, 2013)	

Appendix – Summary of Ballot Comments

Slab-Column Connection Example 2 – Exterior Type 1 Connection: Comments from 3/11/2013 2nd ballot.

Member	Page	Line	Vote Y / C / N / A	Comment
Kreger	1	3	C	Some of the variables shown in the figure (d/2 and l_1/2) are distorted and appear too large. Also, dimension arrows are missing for dimension b_2.
Lee	1	10	N	Show 3 significant figures of 6.75 for d rather than 6.8, as shown in Line 18.
Moehle	1	10	C	This should be 6.75, as later appears on line 18.
Robertson	1	10	C	d should be 6.75 in, not 6.8 in. The 6.75 value is used in much of the rest of the example, but the 6.8 value appears a few times. One consistent value should be used throughout.
Burak	1	14	C	If d is used as 6.75 to be accurate, shouldn't "bo" be corrected as 85.6 throughout the text?
Kreger	1	18	C	6.8 has been used everywhere else in this example. Why is 6.75 used here?
Burak	1	10 & 18	C	In line 10, d should be corrected as 6.75, since it is used without rounding off in line 18.
Criswell	1	10-18	C	Page 1, lines 10 - 18. It seems awkward for d to be 6.8" in line 10 and 6.75" in line 18. Suggest d = 6.75" in line 10, ad then round the values for b sub 1 and 2 in the next two lines to the devised 3 significant digits.
Lee	1	*Fig. E2.1*	C	Range arrows are missing for b_2.
Lee	1	*Fig. E2.1*	C	Use the centerline line type rather than the broken lines for column centerlines in both directions.
Lee	1	*Fig. E2.1*	C	Show same lengths of boundary lines for c_t + c_2 +c_t.
Kang	2	22	N	Negative: pg 2. "Because the connection is exterior, gamma_f cannot be increased, and the portion of moment that...." [exterior connection is not the reason. It's because we chose the option of non-zero gamma_v.]
Criswell	2	22	C	Page 2, line 22 - Another small detail - is the intended word "cannot" or "may not" - as "can" addresses ability or capability to do, and "may" addresses if something is permissible - in this case, permitted by the design provisions.
Kreger	2	23	C	Suggest adding "using Eq. 5.2.1.2b" at the end of the line.

Member	Page	Line	Vote Y / C / N / A	Comment
Kreger	2	34	C	Dimension arrows are missing for dimension b_2.
Lee	2	*Fig. E2.2*	C	Range arrows are missing for b_2, and c_{CD}. The pointing arrow tip is missing for the critical section.
Robertson	2	Fig. E2.2	C	Fig E2.2 has some lines and arrowheads missing.
Kreger	3	2	C	Insert a comma following J_c.
Robertson	3	2	C	Line 2 should read "..., Jc are calculated."
Burak	3	10	C	d is taken as 6.8 in. I think this value could be rounded off, however it should be consistent throughout the example.
Moehle	3	14	C	If you really want to help the reader, do the math and transfer the moment to the centroid, and then make the comment that it would have been conservative to take the moment at the column centerline, as might be done in practice to save calculation time.
Kang	4	6	C	[calculation of "a" is assuming that there is no compression reinf., which is not true. eps_t is affected greatly by the value of "a", which is affected by the compression reinforcement.]
Lee	4	6	N	Show 3 significant figures of 0.647 for α.
Lee	4	8	N	Show 3 significant figures of 6.75 for d. So, results M_n = 70.7 k-ft.
Lee	4	15	N	Use 3 significant figures for d. So, results ε_t = 0.022.
Kreger	4	15	C	I calculate 0.022 instead of 0.021.
Lee	4	19	N	Replace 71.2 k-ft for M_n with 70.7 k-ft (noted above). So, results ϕM_n = 63.6 k-ft.

Member	Page	Line	Vote Y / C / N / A	Comment
Moehle	4	21	N	Earlier in this example, it was demonstrated that the design could use gamma sub f = 1, in which case all the moment transfer is by slab reinforcement for flexural strength. Then the designer decides to save a few No. 4 bars and demonstrates that some of the moment can be resisted by eccentric shear stress around the slab critical section. It is unclear to me how the remaining moment [of magnitude (1-gamma sub f) Mub] is going to get from the slab region outside the effective transfer width and into the column. A good design puts all the reinforcement for flexural strength within C2 +2Ct. I know that it is not what ACI 352 says, but I would not do it any other way. My N vote is from my view that the example is guiding the engineer in the wrong direction by not placing sufficient reinforcement within the effective transfer width. I would rather that you point out that it could be done the other way (the way shown in the example), but a smarter solution takes the full moment transfer in flexural moment.
Lee	4	27	N	Change the subtitle "**Additional reinforcement requirements**" to "**Additional requirements for reinforcement**". Reason for the change: Additional reinforcement requirements imply reinforcement in the subsequent discussion is additional to "**Reinforcement –exterior connections (Section 6.1.1(b))**" titled on page 3 but all reinforcement in the subsequent discussion are not additional at all. Therefore, the title is misleading and confusing. The subtitle "**Additional requirements for reinforcement**" is precise and more suitable for clarity.
Lee	4	28-31	N	The two top bars along the discontinuous slab edge from S6.1.2 are the same top bars required in S6.1.7(b) (See page 5, Parallel Reinforcement.) Make a note of this seemingly overlapping requirement for the designers to prevent a potential confusion.
Lee	5	1	C	Use the standard symbol of phi (ϕ) rather than φ.
Moehle	5	20	C	I think 0.75h, which recommended in ACI 352, is unnecessarily close. I understand the purpose is to resist torsion. In a beam, 0.75h might be too wide. But in a slab, h or even 1.5h or 2h would be fine. The tests by Hwang and Moehle (2000) showed that 2h worked just fine. Have any tests demonstrated that 0.75h is necessary? I recognize this would be new business.
Moehle	5	39	C	Change shall to can or may.
Moehle	5	42	C	What is the required length? Comm 352 report recommends that these extend not less than 2h past the column face.

Member	Page	Line	Vote Y / C / N / A	Comment
Burak	5	38-40	C	"The two bottom bars shall be included in the structural integrity reinforcement specified in Section 6.1.5c." This statement may lead to a misunderstanding and use of 6+2 bars at the bottom. However, I think it is clearly shown in Fig. E2.3.
Moehle	6	9	C	Shall seems out of place.
Robertson	6	20	C	should add the development requirements for the integrity steel perpendicular to the slab edge.
Moehle	6	28	N	The requirement to increase lengths by 25% or 30% was intended for a design that terminates top bars without a standard hook. It was never intended for the tail on the 180-degree hook that laps with the bottom bars, which, frankly, was a nominal recommendation that never had any research backing that I am aware of. Instead, it was intended as a simple lap that the designer could specify without thinking twice. The example has way over-complicated the process by suggesting that all these calculations are justified and necessary. I recommend to modify the 352 recommendation so that this level of tinkering is avoided. (This example is a great illustration of why design examples should be run for every Code/Guideline change – the examples illustrate unintended consequences, and allow sensible changes to the Code/Guideline before it hits the streets.)
Kreger	6	43	C	I believe the wrong value has been used here for l_d. It should be 17.0", not 19.0".
Lee	6	8 –20	N	This part is a repetition of the structural integrity reinforcement previously calculated on page 5 (Lines 1- 10). Simply make a reference to it here without repeating.
Moehle	7	2	N	The effective beam within the slab depth and along the slab edge is recommended by ACI 352 to extend at least 2h either side of the column, resulting in a width of 24" + 4*8" = 56". The example shows it only 48 in.
Burak	7	3	C	Last figure should be renamed as Fig. E2.3.
Lee	7	3	C	Change "Fig. E2.2" to 'Fig. E2.3".
Kreger	8	1	C	The end of the tail on the 180-degree hook is not shown (i.e. the end of the bar looks like an open "pipe").
Lee	8	2	C	Change "Fig. E2.2 (cont.)" to "Fig. E2.3 (cont.)".

Member	Page	Line	Vote Y / C / N / A	Comment
Lee	8	*Fig. E2.2 (c)*	N	As shown, the columns below/above and the slab are monolithic. The current construction practices do not permit this kind of monolithic construction. It simply cannot be built as detailed. The construction joints below and above the slab must be provided. These construction joints are not optional by the contractors. The very reason for having the 180-degree hooks for slab reinforcement instead of 90-degree hooks is the presence of these horizontal construction joints. These non-optional construction joints should be regarded as the required part of structural design and detailing.
Lee	8	*Fig. E2.2 (c)*	N	As a Type 1 Connection, column vertical reinforcement should be spliced at the bottom of the upper column, not at the mid-height of the column as done for Type 2 Connection (without showing any splices, presently implying mid-height splices). Therefore, show the partial lower part of splice bars (graphically, two additional vertical bars above the slab.)
Lee	8	*Fig. E2.2 (c)*	N	The slab is a two-way slab. Therefore, show slab reinforcement in the other direction (say in E-W, i.e., parallel to the slab edge). (Graphically, the new top bar in E-W should be just below the existing top bar in N-S and the new bottom bar in E-W just above the existing bottom bar in N-S.)
Lee	8	*Fig. E2.2 (c)*	N	Show *Fig. E2.2(c)* Slab-Column Connection Detail in two separate sections: one passing the column core and another passing outside the column core but within ($c_t + c_2 + c_t$) as done in ACI 352.1R-89. The two separate sections would give a clearer detail of the connection reinforcement to the designers.
Lee, D.	8	*Fig. E2.2 (c)*	C	(Graphic touchups needed): The slab bar appears inside the column core but its 180-degree hook tail is outside the column core. Both should be inside the core. And close the end tip of the 180-degree hook tail.
Criswell			C	General comment: Although the previous ballot comments include the reason (reached in Toronto) why references in this example are to ACI 318-08 rather than -11, it would seem that the use of ACI 318-11 would make the example appear more current to the reader/user. Has any of the relevant provisions of ACI 318-08 been changed in ACI 318-l1? Could these changes be noted at the appropriate location in the example, and ACI 318-11 be generally used?
Wight			C	On line 9 of page 5 and on line 13 of page 6, I suggest the use of 4 #5 bars instead of 6 #4 bars. I think a lower number of larger bars is a better choice.

Member	Page	Line	Vote Y / C / N / A	Comment
Robertson			C	Fig E2.1 is missing some arrowheads and is "font-challenged"
Alcocer			A	