DISCUSSION

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Factors Affecting Strength of Elements Designed Using Strut-and-Tie Models. Paper by Sergio F. Breña and Micah C. Morrison

Discussion by Emil de Souza Sánchez Filho, Júlio J. Holtz Silva Filho, and Maria Teresa Gomes Barbosa

ACI member, DSc, Professor, Fluminense Federal University, Rio de Janeiro, Brazil; DSc. Structural Engineer, Rio de Janeiro, Brazil; ACI member, DSc, Professor, Federal University of Juiz de Fora, Minas Gerais, Brazil.

The authors have made an interesting contribution to the experimental study of strut-and-tie models. However, the discussers would like to address some aspects in this study:

1. The discussers have reviewed several publications listed in the References section of the paper regarding the fundamental concepts of strut-and-tie models. An important parameter of these modes is the concrete effectiveness factor that depends on the strut type, reinforcement's arrangements, and so on, but the authors did not consider this parameter in the explanation of their modeling. The concrete effectiveness factor ν is an essential parameter that needs to be inserted into the development of the plasticity theory. The best agreement between theory and experimental data is obtained by the appropriate choice of ν, but the authors did not provide the value used in their study. The authors statement, "The strength of the nodes, struts, and ties was calculated using procedures in Appendix A of the 2002 ACI Code (ACI Committee 318 2002)" is very unclear because this strength depends on the quite a lot of parameters and this code provides several expressions to calculate the concrete effectiveness factor.

2. The authors adopted the strength reduction factor ϕ = 0.75 to find the ties armors, which is an inadequate and conservative approach for this type of research. The abundance of armors in several regions of the beams is corroborated by the low strain measured in the several ties. All specimens have unusual reinforcement arrangements. The secondary armors on Specimens 1A and 1B certainly are responsible for the great discrepancies among theoretical and experimental results. This fact is corroborated by the authors' approximate procedure to estimate the contributions of these secondary reinforcements, substantially reducing these differences.

3. Specimens 2A and 2B were designed for an ultimate load with very complicated models, and this is not the basic idea of the strut-and-tie model approach. Instead of very complex modeling for these specimens, it would be more interesting to use the simplest steel reinforcement arrangements, similar to what is currently done for reinforced concrete deep beams with openings.

4. All four models shown in Fig. 2 of the paper are composed by superimposed models, but this information is not given by the authors. Strut-and-tie modeling is a rational and simple method for analysis and design, but for Models 2A and 2B, it is not true. These two models are very complex and inappropriate for an engineering design.

5. All three possible different D regions are cataloged in Jennewein and Schäfer (1992), where the expressions for forces, angles of the struts, and ties are given for each D region, and it is a waste of time trying to compare with another special truss model for D regions near the openings, which is the case of models of Specimens 2A and 2B.

6. The authors failed to explain the theoretical considerations about the models. It would be interesting to know the values of the struts angles adopted in the analyses. Another consideration that requires a better explanation is the optimization of the models. Further precise information of the models conception is necessary—for example, struts lengths and widths, node dimensions, types of the basic models that are superimposed, and details about nonlinear analyses undertaken.

7. The discussers believe that it is impossible to check the theoretical values given in Tables 2 and 3 of the paper, and several topics of the paper are confusing. Therefore, the discussers would greatly appreciate if the authors could provide some complementary information about the research.

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Discussion by Rafael A. de Souza

ACI Member, PhD, Associate Professor, Universidade Estadual de Maringá, Brazil.

The experimental work conducted by the authors has shown that shrinkage and temperature reinforcement contributes significantly to the strength of reinforced concrete deep beams with web openings. It is a very important conclusion as it can substantially affect the design based on strut-and-tie models (STMs). The discusser would like to offer the following comments to emphasize this specific conclusion made by authors:

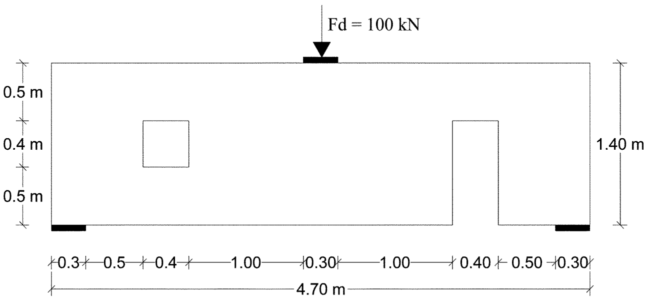

Fig. A—Complex deep-beam subjected to geometric irregularities.

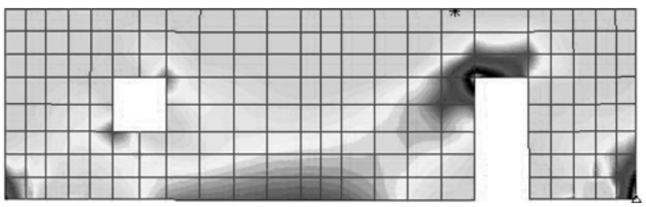

Fig. B—Principal tension stress for deep beam.

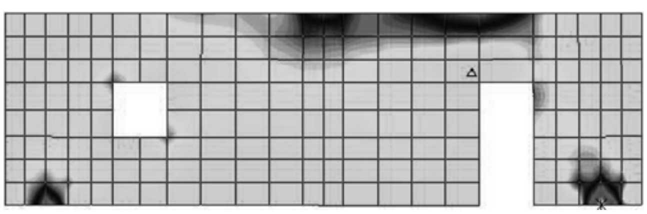

Fig. C—Principal compression stress for deep beam.

1. STMs have usually been taken as a panacea for solving any complex problem that arises when designing structural concrete. Undoubtedly, STM is a very powerful tool, but designers should be aware that sometimes it can lead to an exaggerated design, being that the collapse loads are much higher than the design loads;

2. Taking into account the mandatory recommendations of many normative codes about minimum reinforcement control for shrinkage and temperature (usually assumed as 10% of the concrete section area for each side of an structural element), some idealized truss models may be substantially changed from the original truss sketch. For these cases, minimum reinforcement may be higher than the reinforcement provided for the principal ties, and for this reason, an exaggerated collapse load may be found;

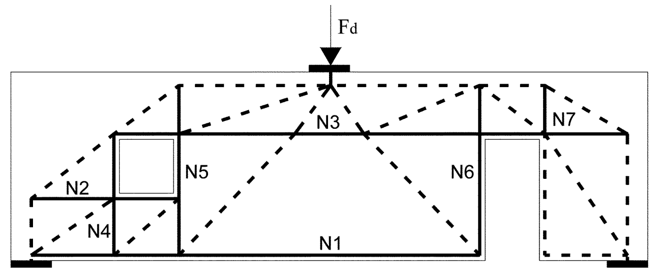

Fig. D—Strut-and-tie model proposed for complex deep beam.

3. Take, for example, the complex deep beam presented in Fig. A, subjected to a design load F_d = 100 kN (22 kips). This beam has a width of 0.25 m (9.8 in.) and is supposed to be molded with concrete with a compressive strength of 20 MPa (2900 psi). Based on the elastic analysis shown in Fig. B and C, the STM presented in Fig. D was developed. It is a very complex STM and the time required to sketch the proposed truss is a very demanding task. The time required for this activity is not compatible with the time available for the structural engineers at their offices;

4. Based on the last paragraph, for many complex situations, it is better to conduct a nonlinear analysis first rather than expend a great deal of time trying to develop a truss model based on the STM. Using some resources of nonlinear analysis available in many package software programs, the mandatory minimum reinforcement can be taken into account in the design, and only some tie positions will need to be strengthened. For the specific case presented in Fig. A, for example, it is easy to prove by using nonlinear analysis that minimum reinforcement control for shrinkage and temperature can carry approximately 60% of the design load. This design procedure is attractive for structural engineers because it is less time-consuming than STM;

5. Also, using nonlinear analysis resources for structural concrete, a quick estimate for both the ultimate and serviceability limit state (usually a problem when using STMs) is available. Until now, the development of STMs has been focused on the ultimate limit state, and the serviceability limit state is only implicitly considered through the selection of appropriate STMs; and

6. Finally, taking into account the powerful computational resources available today, STM could be used as a hand-made verification proof to certify the answers provided by many specific software programs. It seems to be a routine very close to that one required by structural offices. Besides, the minimum reinforcement for crack control could always be considered in design, providing a more economic answer for complex problems.

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Factors Affecting Strength of Elements Designed Using Strut-and-Tie Models. Paper by Sergio F. Breña and Micah C. Morrison

Discussion by A. Muttoni, N. Kostic, and M. Fernández Ruiz

École Polytechnique Fédérale de Lausanne, Lausanne, Switzerland.

The authors of the paper investigated the suitability of design of structural concrete members using strut-and-tie models inspired by linear elastic (uncracked) stress fields. The accuracy of this approach is checked against the results of four 1/4-scale tests. The methodology followed by the authors is much appreciated by the discussers. In the discussers' opinion, papers providing experimental data that can be compared with strut-and-tie models (or stress fields) are still necessary to advance the state of knowledge on this topic and to develop new strategies for development of suitable strut-and-tie models and stress fields.

In Table 2 of the paper, the theoretical strengths Q_{th} expected by the authors according to strut-and-tie models inspired by the uncracked stress field of the members are compared with actual values measured in the tests Q_{test}. The ratios between them (Q_{test}/Q_{th}) vary between 1.72 and 3.19 (summarized in the first row of Table A), showing too conservative estimates of the strength for the various specimens. The differences are, according to the authors of the paper, due to four phenomena:

1. The design method (strut-and-tie model) is a lower bound solution of the theory of plasticity, thus leading to conservative estimates of the actual failure load;
2. The contribution of the secondary (minimal) reinforcement is neglected in the strength of the strut-and-tie models;
3. Significant stress redistributions developed during tests (confirmed by test measurements); and
4. Also, according to the authors, concrete contribution to tie strength could have played a non-negligible role in the strength of the member.

Considering the influence of the minimal reinforcement and performing a nonlinear analysis to account for stress redistributions, the previous ratios (Q_{test}/Q_{th}) are improved by the authors and vary between 1.49 and 2.18 (refer to Table A, second row).

The discussers are in complete agreement with the first three phenomena mentioned by the authors. The differences between the measured and the predicted strengths can be mostly explained due to the fact that the selected strut-and-tie models, although licit for design (because they give a lower-bound solution), differ notably from the actual stress fields at failure. In this sense, the minimal reinforcement of Specimens 1A and 1B shows a significant influence on the actual stress field and, thus, on the strength of the members (refer to Table A). Also, stress redistributions from the uncracked stress field to the cracked stress field at failure (including yielding of the reinforcement and changes in the angle and the location of compression struts) has a clear influence on the strength of the member.

On the contrary, in the discussers' opinion, the fourth phenomenon (concrete contribution to tie strength) can be neglected in comparison with the other phenomena because significant crack widths develop at failure in RC members. Furthermore, this contribution is not reliable and should not be considered in plastic analyses (Muttoni et al. 1997).

In addition to the previous phenomena, the discussers think that the differences between the expected and the measured strengths are also due to the fact that the proposed strut-and-tie models do not account for a realistic kinematics at failure. An approach overcoming most of the previous problems (minimal reinforcement, stress redistributions, and suitable kinematics) can be easily developed, leading to satisfactory results both at failure and under serviceability conditions (Muttoni et al. 1997). According to this approach, a licit mechanism first has to be selected for the member (which yields an upper-bound solution of the theory of plasticity). Second, a stress field is developed in the free-bodies of the mechanism, respecting its kinematics and the plasticity criterion (which yields a lower-bound solution of

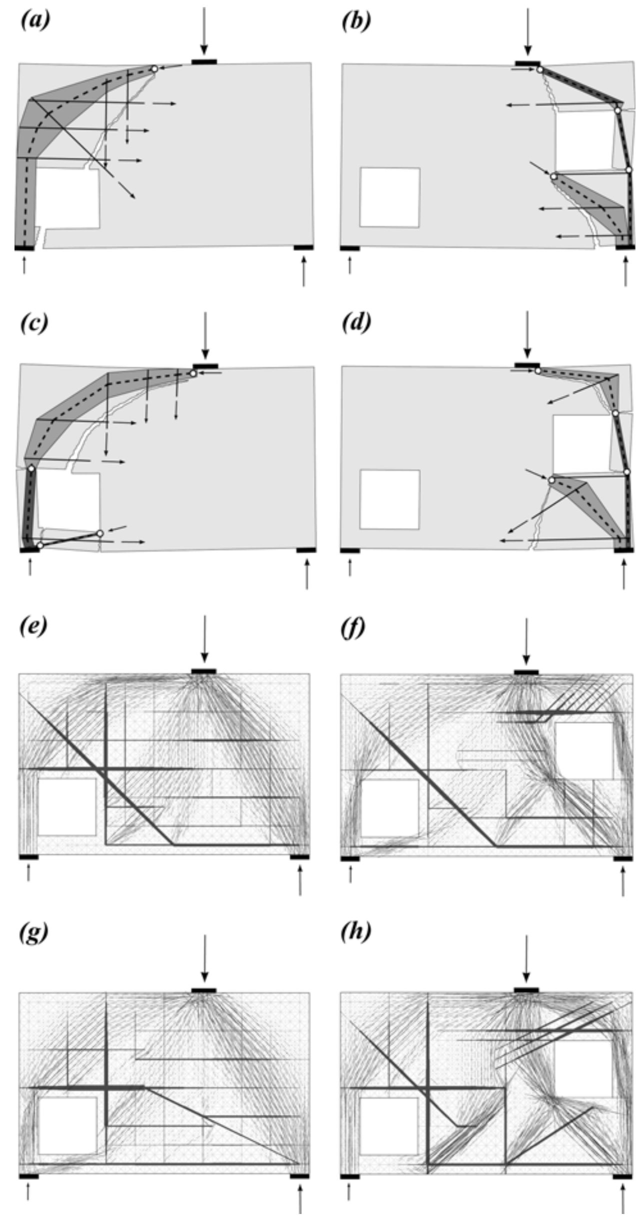

Fig. E—Development of strut-and-tie model and stress fields. Assumed kinematics at failure and discontinuous stress fields in critical free-bodies of Specimens: (a) 1A; (b) 2A; (c) 1B; and (d) 2B; and continuous stress fields of Specimens: (e) 1A; (f) 2A; (g) 1B; and (h) 2B.

Table A—Ratio between measured and estimated failure loads for various specimens

Q_{test}/Q_{th}	1A	1B	2A	2B	Average	Coefficient of variation
Breña and Morrison (strut-and-tie models inspired by linear-elastic uncracked stress field)	3.19	2.98	1.72	1.74	2.41	0.33
Breña and Morrison (strut-and-tie models where minimal reinforcement is considered and nonlinear analysis is performed)	1.72	2.18	1.49	1.49	1.72	0.19
Discontinuous stress field accounting for kinematics at failure	1.09	1.10	1.04	1.02	1.06	0.04
Continuous stress field	1.17	1.27	1.00	1.03	1.11	0.09

the theory of plasticity). Consequently, this approach leads to an exact solution according to the theory of plasticity.

For instance, Fig. E(a) to (d) show some possible failure mechanisms for the various specimens. According to these failure mechanisms, specimens with two openings fail on the right side, whereas specimens with one opening fail on the left side. Starting from such kinematics, and accounting for the reinforcement layout (including the minimal reinforcement) and concrete strength, discontinuous stress fields can be developed in the critical free-bodies (free-bodies governing the strength of the member), as shown in Fig. E(a) to (d). Such stress fields allow the location of the critical nodal regions to be determined and, thus, realistic angles and locations of the critical struts to be estimated (indicated in dark gray in Fig. E).

When checking the strength of a member, as in this case, the suitable failure mechanism is the one having the lowest strength. In doing so, the mechanisms shown in Fig. E(a) to (d) (which are found critical) lead to the failure loads detailed in Table A (third row) in excellent agreement with the test results. This methodology can also be followed for design purposes, leading to stress fields with a satisfactory behavior at SLS (crack control) and accounting for the kinematics at failure (Muttoni et al. 1997).

An alternative approach to account for the kinematics of structural concrete members is the development of continuous stress fields (Fernández Ruiz and Muttoni 2007), where compatibility conditions for concrete and for reinforcing steel are introduced. Figures E(e) to (h) show the continuous stress fields obtained for the four specimens. The ratios between the measured and the estimated failure loads according to continuous stress fields are also shown in Table A (fourth row). The results obtained are somewhat more conservative than those obtained with discontinuous stress fields accounting for kinematics at failure. This is due to the fact that the strength of concrete is reduced in continuous stress fields to account for transverse cracking.

In any case, both approaches accounting for kinematics provide a very good agreement with the actual failure loads and show to be more accurate than those obtained with elastically inspired strut-and-tie models.

To conclude, the discussers would like to highlight that simple, suitable, and satisfactory strut-and-tie models and stress fields can be developed if a realistic kinematics at failure is considered. This allows one to overcome most difficulties found when developing strut-and-tie models inspired by the linear elastic (uncracked) stress field of a member.

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AUTHORS' CLOSURE

The authors appreciate the interest expressed by three groups of discussers on our recently published paper. The response to each group will be addressed in a separate section as follows.

Closure to discussion by de Souza Sánchez Filho et al.

Using the same order that the discussers' used, the authors would like to clarify the issues brought up by the discussers:

1. As the discussers point out, the concrete effectiveness factor is an important parameter when computing the strength of struts in strut-and-tie models. Past experimental evidence suggests that the effective concrete strength is lower than the uniaxial compression strength when concrete is subjected to transverse tension that induces cracking in conditions of biaxial stresses. Because the intent was to use available design recommendations to develop strut-and-tie models in the design phase of the research, the authors chose to use the effective strength factors included in Appendix A of ACI 318-02. Effective strength factors in ACI 318-02 are calculated as $0.85\beta_s$, where β_s ranges between 1.0 and 0.4, depending primarily on the level of transverse tension that the strut will experience. The authors used factors corresponding to either prismatic or bottle-shaped struts depending on the location in the specimens as indicated in Table B.

2. The primary objective of the paper was to quantify sources of potential overstrength when using strut-and-tie models for design as stated in the Research Significance section of the paper. To achieve this goal, the authors used a strength reduction factor $\phi = 0.75$, in accordance with ACI 318-02 for design of the laboratory specimens. This factor was later removed, or in other words, $\phi = 1.0$ was used when strength evaluation of the specimens was being conducted after the tests. The procedure used to estimate the strength of the as-built specimens is discussed in detail in the paper.

3 and 4. The authors agree that the strut-and-tie models that resulted in some cases were more complicated than would be desired. The intent was to examine whether differences in models resulting in different reinforcement patterns would affect measured strength of the specimens. Even if nontraditional models are used for design, the reinforcement can be resolved in two orthogonal directions, as is commonly done in practice. As can be observed from the reported test results, the specimens failed at approximately the same load, so no apparent effect on reinforcement pattern was identified for these specimens.

5. The authors do not have a copy of the reference mentioned in this paragraph, so we cannot comment on the discussers' statement. It is difficult, however, to think of a catalog with only three different D-regions that would encompass all possible stress fields and boundary conditions that might be encountered in practice.

6 and 7. The discussers request additional information about the models used in the tests. This information was not provided in the original manuscript because of space constraints. Complementary information is summarized in Table B of this closure.

Closure to discussion by de Souza

In the authors' experience, designs based on the strut-and-tie method result in elements with higher strength than the design load. The authors also believe, as the discusser suggests, that minimum reinforcement required to control thermal and shrinkage cracking will contribute to strength of some types of structural elements that are designed using strut-and-tie models (such as deep beams). Secondary reinforcement is also used to prevent wide cracks from degrading the strength of struts after diagonal cracking occurs. The effect that this type of reinforcement has on element strength needs to be studied in more detail.

The discusser suggests using nonlinear finite element analysis to calculate effects of crack control reinforcement in structural concrete elements with complicated geometries.

Table B—Summary of strut geometry and forces for strut-and-tie design models

Specimen	Strut no.	Strut force, kN (kip)	β_s	Effective width, mm (in.)	Design strength, kN (kip)
1A	S1	–132 (–29.70)	1.000	168 (6.61)	330 (74.19)
	S2	–53 (–11.87)	0.750	102 (4.00)	150 (33.66)
	S3	–53 (–11.88)	0.750	102 (4.00)	150 (33.66)
	S4	–49 (–11.12)	0.750	102 (4.00)	150 (33.66)
	S5	–49 (–11.12)	0.750	102 (4.00)	150 (33.66)
	S6	–53 (–11.88)	0.750	114 (4.50)	169 (37.87)
	S7	–53 (–11.87)	0.750	114 (4.50)	169 (37.87)
	S8	–85 (–19.09)	1.000	127 (5.00)	250 (56.10)
	S9	–8 (–1.87)	1.000	15 (0.60)	30 (6.73)
	S10	–11 (–2.56)	1.000	25 (1.00)	50 (11.22)
	S11	–26 (–5.86)	1.000	38 (1.50)	75 (16.83)
	S12	–14 (–3.12)	1.000	25 (1.00)	50 (11.22)
	S13	–20 (–4.43)	1.000	38 (1.50)	75 (16.83)
	S14	–20 (–4.42)	1.000	51 (2.00)	100 (22.44)
	S15	–13 (–2.91)	1.000	25 (1.00)	50 (11.22)
	S16	–18 (–4.12)	1.000	38 (1.50)	75 (16.83)
	S17	–18 (–4.12)	1.000	32 (1.25)	62 (14.03)
	S18	–32 (–7.19)	1.000	51 (2.00)	100 (22.44)
	S19	–31 (–6.93)	1.000	51 (2.00)	100 (22.44)
	S20	–34 (–7.69)	1.000	57 (2.25)	112 (25.25)
	S21	–47 (–10.61)	1.000	76 (3.00)	150 (33.66)
	S22	–47 (–10.61)	1.000	102 (4.00)	200 (44.88)
1B	S1	–140 (–31.50)	1.000	168 (6.61)	330 (74.16)
	S2	–68 (–15.36)	0.750	127 (5.00)	187 (42.08)
	S3	–68 (–15.36)	0.750	127 (5.00)	187 (42.08)
	S4	–64 (–14.38)	0.750	108 (4.25)	159 (35.76)
	S5	–64 (–14.38)	0.750	108 (4.25)	159 (35.76)
	S6	–68 (–15.36)	0.750	140 (5.50)	206 (46.28)
	S7	–68 (–15.36)	0.750	140 (5.50)	206 (46.28)
	S8	–90 (–20.25)	1.000	127 (5.00)	250 (56.10)
	S9	–47 (–10.67)	1.000	76 (3.00)	150 (33.66)
	S10	–32 (–7.26)	1.000	51 (2.00)	100 (22.44)
	S11	–6 (–1.36)	1.000	13 (0.50)	25 (5.61)
	S12	–26 (–5.81)	1.000	38 (1.50)	75 (16.83)
	S13	–37 (–8.26)	1.000	76 (3.00)	150 (33.66)
	S14	–37 (–8.24)	1.000	76 (3.00)	150 (33.66)
	S15	–20 (–4.48)	1.000	51 (2.00)	100 (22.44)
	S16	–43 (–9.57)	1.000	76 (3.00)	150 (33.66)
	S17	–28 (–6.34)	1.000	51 (2.00)	100 (22.44)
	S18	–28 (–6.34)	1.000	76 (3.00)	150 (33.66)
	S19	–43 (–9.57)	1.000	76 (3.00)	150 (33.66)
	S20	–20 (–4.48)	1.000	38 (1.50)	75 (16.83)
	S21	–50 (–11.25)	1.000	76 (3.00)	150 (33.66)
	S22	–50 (–11.25)	1.000	102 (4.00)	200 (44.88)
2A	S1	–154 (–34.50)	1.000	164 (6.46)	336 (75.56)
	S2	–107 (–24.07)	1.000	127 (5.00)	260 (58.48)
	S3	–71 (–16.05)	1.000	127 (5.00)	260 (58.48)
	S4	–36 (–8.02)	1.000	76 (3.00)	156 (35.09)
	S5	–127 (–28.59)	1.000	159 (6.25)	325 (73.11)
	S6	–41 (–9.19)	1.000	127 (5.00)	260 (58.48)
	S7	–36 (–8.02)	1.000	76 (3.00)	156 (35.09)
	S8	–36 (–8.02)	1.000	76 (3.00)	156 (35.09)
	S9	–36 (–8.02)	1.000	76 (3.00)	156 (35.09)
	S10	–8 (–1.78)	1.000	25 (1.00)	52 (11.70)
	S11	–105 (–23.54)	1.000	127 (5.00)	260 (58.48)
	S12	–3 (–0.69)	1.000	25 (1.00)	52 (11.70)
	S13	–79 (–17.78)	1.000	102 (4.00)	208 (46.79)

Table B (cont.)—Summary of strut geometry and forces for strut-and-tie design models

Specimen	Strut no.	Strut force, kN (kip)	β_s	Effective width, mm (in.)	Design strength, kN (kips)
2A	S14	–92 (–20.60)	1.000	152 (6.00)	312 (70.18)
	S15	–80 (–18.05)	1.000	152 (6.00)	312 (70.18)
	S16	–61 (–13.60)	1.000	102 (4.00)	208 (46.79)
	S17	–24 (–5.38)	1.000	51 (2.00)	104 (23.39)
	S18	–39 (–8.67)	1.000	102 (4.00)	208 (46.79)
	S19	–74 (–16.60)	1.000	133 (5.25)	273 (61.41)
	S20	–52 (–11.63)	1.000	102 (4.00)	208 (46.79)
	S21	–34 (–7.60)	1.000	51 (2.00)	104 (23.39)
	S22	–9 (–1.97)	1.000	0 (0.00)	0 (0.00)
	S23	–43 (–9.57)	1.000	102 (4.00)	208 (46.79)
	S24	–16 (–3.69)	1.000	38 (1.50)	78 (17.55)
	S25	–40 (–9.07)	1.000	76 (3.00)	156 (35.09)
	S26	–60 (–13.43)	1.000	127 (5.00)	260 (58.48)
	S27	–60 (–13.40)	1.000	127 (5.00)	260 (58.48)
	S28	–99 (–22.18)	1.000	127 (5.00)	260 (58.48)
	S29	–42 (–9.36)	1.000	76 (3.00)	156 (35.09)
	S30	–9 (–2.02)	1.000	13 (0.50)	26 (5.85)
	S31	–11 (–2.56)	1.000	25 (1.00)	52 (11.70)
	S32	–26 (–5.83)	1.000	38 (1.50)	78 (17.55)
	S33	–13 (–2.97)	1.000	25 (1.00)	52 (11.70)
	S34	–19 (–4.20)	1.000	25 (1.00)	52 (11.70)
	S35	–19 (–4.20)	1.000	38 (1.50)	78 (17.55)
	S36	–13 (–2.94)	1.000	25 (1.00)	52 (11.70)
	S37	–21 (–4.65)	1.000	38 (1.50)	78 (17.55)
	S38	–20 (–4.41)	1.000	38 (1.50)	78 (17.55)
	S39	–42 (–9.35)	1.000	76 (3.00)	156 (35.09)
	S40	–55 (–12.32)	1.000	102 (4.00)	208 (46.79)
	S41	–55 (–12.32)	1.000	102 (4.00)	208 (46.79)
2B	S1	–147 (–33.00)	1.000	164 (6.46)	323 (72.48)
	S2	–102 (–23.02)	1.000	152 (6.00)	300 (67.32)
	S3	–32 (–7.19)	1.000	76 (3.00)	150 (33.66)
	S4	–26 (–5.91)	1.000	76 (3.00)	150 (33.66)
	S5	–26 (–5.91)	1.000	76 (3.00)	150 (33.66)
	S6	–72 (–16.29)	1.000	170 (6.70)	335 (75.17)
	S7	–168 (–37.70)	1.000	254 (10.00)	499 (112.20)
	S8	–3 (–0.69)	1.000	102 (4.00)	200 (44.88)
	S9	–52 (–11.72)	1.000	76 (3.00)	150 (33.66)
	S10	–131 (–29.51)	1.000	330 (13.00)	649 (145.86)
	S11	–83 (–18.65)	1.000	152 (6.00)	300 (67.32)
	S12	–28 (–6.20)	1.000	127 (5.00)	250 (56.10)
	S13	–94 (–21.21)	1.000	127 (5.00)	250 (56.10)
	S14	–33 (–7.33)	1.000	51 (2.00)	100 (22.44)
	S15	–20 (–4.49)	1.000	51 (2.00)	100 (22.44)
	S16	0 (0.00)	1.000	51 (2.00)	100 (22.44)
	S17	–55 (–12.36)	1.000	51 (2.00)	100 (22.44)
	S18	–36 (–7.99)	1.000	64 (2.50)	125 (28.05)
	S19	–25 (–5.65)	1.000	44 (1.75)	87 (19.64)
	S20	–36 (–7.99)	1.000	76 (3.01)	150 (33.77)
	S21	–45 (–10.11)	1.000	114 (4.50)	225 (50.49)
	S22	–16 (–3.62)	1.000	102 (4.00)	200 (44.88)
	S23	–10 (–2.29)	1.000	127 (5.00)	250 (56.10)
	S24	–15 (–3.43)	1.000	152 (6.00)	300 (67.32)
	S25	–27 (–6.13)	1.000	51 (2.00)	100 (22.44)
	S26	–57 (–12.71)	1.000	51 (2.00)	100 (22.44)
	S27	–52 (–11.79)	1.000	70 (2.75)	137 (30.86)
	S28	–52 (–11.79)	1.000	76 (3.00)	150 (33.66)

We believe that interpretation of nonlinear analysis results is quite complex and requires significant experience with these types of analyses. Adequate material models with appropriate parameters (often calibrated through experimental testing) must be used to achieve adequate solutions. Inadequate use of material properties may result in predicted loads that are higher than the actual force an element can carry. On the other hand, constructing and solving a relatively complicated strut-and-tie model can be performed with ease and will provide a lower bound to the true solution, which is desirable for design. Therefore, the authors still believe that strut-and-tie modeling techniques offers a viable option for safe design of members with discontinuities.

Closure to discussion by Muttoni et al.

As the discussers point out, the strut-and-tie models for this study were developed without consideration of kinematics because these are commonly neglected for design. The discussers have developed a method based on stress fields that accounts for realistic kinematic conditions at failure. The authors believe that these techniques are extremely promising and can provide much better estimates to actual strength of elements than current models based on strut-and-tie idealizations. The authors are extremely pleased that their research is able to contribute to development of sophisticated tools for structural design of elements with complex geometries.

Disc. 104-S29/From the May-June 2007 *ACI Structural Journal*, p. 294

Lattice Shear Reinforcement for Slab-Column Connections. Paper by Hong-gun Park, Kyung-soo Ahn, Kyoung-kyu Choi, and Lan Chung

Discussion by Ramez B. Gayed

ACI member, Research Associate, Civil Engineering Department, University of Calgary, Calgary, AB, Canada.

Several researchers[15-19] have suggested configurations of multi-leg prebent bars as shear reinforcement in flat plates. The ACI 318 Code[20] permits the use of shear reinforcement in the form of closed or multi-leg stirrups. Essential criteria for shear reinforcement in flat plates are effective anchorage and ease of constructibility. The results of the presented tests show adequacy of the anchorage achieved by welding of the lattice shear bars to the flexural bars. The constructibility of the proposed system is questionable, however, particularly in prestressed slabs due to the reinforcement congestion.

It is well established that the shear reinforcement is most effective when it confines the maximum volume of concrete; thus, its overall height has to be as large as possible. However, all reinforcement must be protected with the concrete cover specified in codes. In the tested slabs, the lattice shear reinforcement had no cover at the bottom. To provide the required cover for the lattice shear reinforcement, its overall height has to be smaller than in the tests, resulting in an adverse effect on the observed strength and ductility.

In the lattice shear reinforcing system, the shear is resisted by the inclined legs running in almost two orthogonal directions. A leg intercepting a crack at a right angle is most effective in controlling its width; a leg parallel to a series of cracks may not intersect any of them. If half the legs are perpendicular to the cracks, the other half will be parallel to them. Thus, only one half of the web bars can be fully effective in intercepting and controlling the inclined shear cracks. Absence of web bars that intersect the shear crack at the critical section close to the column can induce failure at a low load level. For this reason, ACI 318-05[20] and ACI 421.1R-99[21] specify the distance between the column face and the first peripheral line of vertical shear reinforcement.

The comparison between the results of tests with the lattice shear reinforcement and tests having headed studs is unpersuasive because: 1) the volumetric ratio of the inclined legs of the lattice shear reinforcement was 13 to 19% higher than that of the stems of the headed studs; and 2) the compared tested slabs differed in f_c' (ρf_y) and the overall heights of the shear reinforcement assemblies.

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AUTHORS' CLOSURE

The authors thank the discusser for his interest in this paper. Each item of the questions and comments presented by the discusser is discussed separately, as follows.

1. Constructibility of proposed system

The lattice system that the authors tested is an existing commercial product that was originally developed as a part of a form deck system for slab construction. For better constructibility, however, the configuration and shape of the lattice bars can be changed according to the engineer's desire. For example, the number of the lattice bars that are installed at the slab-column connection can be significantly reduced by using large-diameter reinforcing bars for the lattice. Further, though the lattice system that was used in this study had a space truss configuration, it can be changed to a planar truss system for easy installation and avoiding congestion of the reinforcing bars.

2. Concrete cover for lattice shear reinforcement at the bottom

In the lattice system, the legs of the inclined web bars do not affect the structural capacity of the lattice, and play the role of bar-chairs. Therefore, like ordinary bar-chairs, the legs can be treated for corrosion protection. Otherwise, the lattice can be manufactured without the legs, and ordinary bar-chairs can be used to support the lattice bars during construction.

3. Distance between column face and first peripheral line of shear reinforcement

As the discusser mentioned, for effectiveness of shear reinforcement, the location of the first peripheral line of the shear reinforcement is important, and the specification of the ACI 318-05 Code[20] and ACI 421.1R-99[21] should be met. Unlike ordinary shear reinforcement including stirrups and shear studs, however, the lattice shear reinforcement is a continuous truss system going through the column and extending beyond the region of the critical section. In the authors' opinion, the load-transfer mechanism of the lattice reinforcement providing truss action and increased dowel action is different from that of the ordinary shear reinforcing bars. In this paper, the shear strength of the lattice reinforcement was calculated in the same manner as used for the stirrups. To clarify the load-transfer mechanism of the lattice, however, further intensive experimental and theoretical studies are required in the future.

4. Comparison between results of tests with lattice shear reinforcement and those with headed studs

As shown in Tables 1, 2, and 4, the specimens using shear stud rails have almost the same material properties as those of Specimens SL1, SL2, and SL4 tested by the authors. The compressive strengths of the authors' specimens are 25.9 to 28.0 MPa (3.8 to 4.1 ksi) whereas those using shear studs rails are 29.0 to 49.0 MPa (4.2 to 7.1 ksi). The effective shear strength ($\rho_v f_{yv}$) of the authors' specimens is 1.4 to 1.7 MPa (0.20 to 0.25 ksi) whereas those using shear studs rails are 1.3 to 5.0 MPa (0.19 to 0.73 ksi). As the discusser indicates, the amount of web shear reinforcement of the lattice is slightly greater than that of the stud rail. This is because a part of the inclined web bars of the lattice is ineffective in the shear contribution. However, the effective amount (the effective shear strength) of the web reinforcement of the lattice is similar to that of the stud rails. As such, because the material properties of the specimens using shear stud rails are not significantly different from those of the lattice specimens, the authors believe that the comparison presented in this paper is meaningful enough.

Though the discusser indicated the difference in the amount of web reinforcement of the lattice and the stud rails, from an economical standpoint, the cost for manufacturing the shear reinforcement is more important than the amount of the shear reinforcement. An application of the lattice reinforcement in Korea showed that the construction cost of the lattice was significantly less than that of the stud rails.

Disc. 104-S36/From the May-June 2007 *ACI Structural Journal*, p. 357

Distinction between Punching and Flexural Failure Modes of Flat Plates. Paper by Timm Stein, Amin Ghali, and Walter Dilger

Discussion by Carl Erik Broms

Senior Consultant, WSP Sweden AB, Stockholm, Sweden.

The authors maintain that experimental research to study the effectiveness of shear reinforcement in flat plates gives conclusive results only if tests are designed so that the predicted flexural capacity is at least 50% larger than the predicted punching capacity. As will be shown in the following, they thereby take advantage of a deficiency in the ACI 318 Code, where the nominal punching shear strength is considered to be independent of the provided amount of flexural reinforcement. It has been well known for more than 40 years, however, that the amount of flexural reinforcement does indeed have a major impact on the punching strength, which in fact is confirmed by the tests described in the paper. More severe misinterpretations of test results than those described by the authors may therefore be made if due respect is not paid to important factors that influence the punching strength. Such factors are, for instance, the flexural reinforcement ratio, the size effect, and the slenderness of the test specimen—none of them covered by ACI 318.

Kinnunen and Nylander (1960) showed that the punching shear strength of flat plates increases with increasing flexural reinforcement ratio and decreases with increasing slenderness of test specimens. Moe (1961) concluded that if the nominal punching shear strength is defined to be independent of the flexural reinforcement ratio (as by ACI 318-05), then the nominal strength level has to be chosen safely low. The level should allow the desirable structural behavior that all reinforcement will reach the yield stress without punching occurring if the nominal punching capacity of the slab exceeds the nominal flexural capacity. This sound engineering principle seems to have been ignored by the authors of the paper and also by ACI 421.1R-99 on design of shear studs in flat plates. In the latter, no indication is given that the proposed upper bound for the nominal punching strength with shear studs calls for more flexural reinforcement than required for the bending moment.

The discusser therefore believes that the experimental principle described in the paper is of limited value if the tests are evaluated against the ACI 318 Code because the test specimens would then not reflect normal design. In a real case, the amount of flexural reinforcement would not be chosen to resist a bending moment that is more than 50% larger than the actual bending moment.

A consequence of ignoring the influence of the flexural reinforcement ratio on the punching capacity is demonstrated by the authors' own tests. Specimen V2, with a reinforcement ratio of 0.0098, failed in shear outside the zone with shear studs at the load 438 kN (98.5 kip). Specimen V3 with a reinforcement ratio of 0.0062 also failed in shear outside the studs at

365 kN (82.05 kip). The authors conclude that this latter failure is due to a combination of flexural and shear failure. This seems to be an erroneous conclusion. The radial bending moment is very low at the actual shear failure position outside the shear reinforcement and should therefore not influence the shear capacity of the slab. If the influence of the flexural reinforcement ratio on the shear strength is taken into account by the principle of Eurocode 2, then the theoretical relation between the shear capacities of Specimens V2 and V3 becomes $(0.0062/0.0098)^{1/3}$, which is close to the actual test relation 365/438 = 0.83. The lower shear capacity of Specimen V3 in relation to Specimen V2 is thus fully explained by its lower flexural reinforcement ratio. This demonstrates the danger with the authors' recommended testing principle in combination with evaluation according to ACI 318. If test results with high reinforcement ratios are taken as an indication of the shear strength of the slab outside the studs, the consequence will be unsafe design for real structures with less flexural reinforcement.

A similar evaluation mistake exists for Specimen V1. Its punching capacity without shear reinforcement is assessed by comparison with the previously tested Specimen AB1 with the compression strength of 36 MPa (5221.4 psi). Specimen AB1 failed in punching for the load of 408 kN (91.7 kip). The authors then assess the punching capacity V_c without shear reinforcement to 355 kN (80 kip) for Specimens V1 to V3 by proportioning to the square root of the concrete strengths in accordance with ACI 318. They thereby disregard the considerable difference in reinforcement ratios—0.013 for Specimen AB1 and 0.0045 for Specimen V1. If the principle of Eurocode 2 is used for proportioning in relation to Specimen AB1, the probable punching capacity without shear studs for Specimen V1 becomes

$$V_c = 408\left(\frac{0.0045}{0.013} \cdot \frac{29.7}{36}\right)^{\frac{1}{3}} = 269 \text{ kN (60.6 kip)}$$

instead of the authors' assessment of 355 kN (80 kip). It is then evident from Fig. 9 that both the capacity and the ductile behavior of Specimen V1 most probably should be attributed to the shear studs, contrary to the authors' conclusion. In addition, the size effect should be considered when evaluating test results. The obtained shear strengths (expressed in stress units) of slab specimens with the small effective depth of approximately 115 mm (4.5 in.) must be treated with caution when applied to real structures with larger effective depth. For instance, a real structure with an effective depth of 200 mm (8 in.) and a reinforcement ratio of 60% of the test specimen's ratio would, according to Eurocode 2, have a shear strength or punching strength of only

$$\frac{1 + \sqrt{\frac{200}{200}}}{1 + \sqrt{\frac{200}{115}}} \cdot (0.60)^{\frac{1}{3}}$$

= 0.73 times the strength (in stress units) of the test specimen.

Compact test specimens with a high flexural reinforcement ratio have often been used in Europe for testing the punching capacity of slabs with shear studs (Andrä 1981; DEHA 1996; Otto-Graf-Institut 1996), which is justified because the upper limit of the punching strength with shear reinforcement is, in Europe, usually expressed as a function of the flexural reinforcement ratio. The upper limit for the punching capacity with shear studs in ACI 421.1R-99 seems to be based on these tests, without mentioning that the flexural reinforcement thereby must be over-designed. Furthermore, slender test specimens simulating flat plates would display lower punching capacity than the tested compact test specimens, which has been reflected by, for instance, the Swedish Code for Concrete Structures since 1964.

The nominal punching strength without shear reinforcement according to ACI 318 is usually conservative when compared with test results as noted by the authors. This can be exemplified by the aforementioned Specimen AB1 with the nominal punching capacity of 333 kN (75 kip) according to ACI 318-05 to be compared with the test capacity of 408 kN (91.7 kip) (= 1.23 · 333). This conservatism, however, is only experienced for specimens with a high reinforcement ratio and small effective depth and is not sufficient to give reasonable safety for real structures with a low reinforcement ratio and large effective depth (Gardner et al. 2000). At least the same margin of approximately 1.25 for test results in relation to the nominal shear strength according to ACI 318-05 should be applied when evaluating test specimens with shear reinforcement failing in shear outside the shear-reinforced zone. This basic principle was not adhered to by, for instance, Mokhtar et al. (1985), Megally (1998), ACI Committee 421 (1999), and Gayed and Ghali (2006).

In summary, the authors of the paper have hit some tender spots of the ACI 318 Code. They have demonstrated that evaluation of punching experiments against ACI 318-05 or ACI 421.1R-99 may lead to false conclusions because due respect is not paid in those documents to important parameters that affect the punching strength. It is therefore high time that the ACI 318 provisions for shear and punching of slabs be modernized.

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Disc. 104-S36/From the May-June 2007 *ACI Structural Journal*, p. 357

Distinction between Punching and Flexural Failure Modes of Flat Plates. Paper by Timm Stein, Amin Ghali, and Walter Dilger

Discussion by Myoungsu Shin and Jacob Grossman

Project Engineer, Rosenwasser Grossman Consulting Engineers, New York, NY; President, Rosenwasser, Grossman Consulting Engineers.

The tests were performed with approximately 1/2-scale interior slab-column connection subassemblies subjected to both gravity and lateral loading. The discussers would like to deliberate several considerations on the test setup used in this study. There are several unrealistic features that could have directly affected the test results investigated, as summarized in the following:

1. The continuous simple supports along the four peripheral lines of the slab in the test were set up at approximately 1/3 of the span length apart from the column, assuming that the test specimen represented a roughly 1/2-scale model. There is no explanation, however, whether the supports were devised to simulate inflection points under gravity loads only or under combined gravity and lateral loads. The test specimen was subjected to subsequent lateral loads, with gravity loads applied first and sustained. In a real structure, inflection points for combined gravity and lateral loading would differ from those under gravity loading only. In general, the inflection points occurring in the prototype frame in an event of the design-level earthquake would typically be close to the midspan of the slab. In the test, however, the inflection points under the gravity loading were identical to those during the lateral loading by design (by applying compression on the column for simulating the gravity loads). Thus, vertical resultant forces due to the lateral loading at the slab edges perpendicular to the loading direction would have been larger in the test than those in the prototype frame, which in turn would have produced larger direct punching shear forces around the column. Also, slab moments generated in the slab-column interfaces due to the gravity loading applied first would have been smaller in the test than those in the prototype frame. In short, the magnitude of direct shear or unbalanced moment occurring at the slab-column interfaces depends on the locations of inflection points in the test subassembly.

2. Along with Item 1, data analysis related to the story drift ratio may not be valid if the subassembly dimensions were not detailed in proportion to the prototype slab-column frame. The discussers assumed that the test specimens were in roughly 1/2-scale in that the story height in a typical building with a flat plate system ranges from 2.75 to 3.35 m (9 to 11 ft).

3. During testing, the continuous simple supports played a function similar to continuous wall supports, so that relative vertical displacements along each of the four slab edges were restrained. When a slab span in a flat plate system is supported by a long wall at one end, moments in the slab section adjacent to the wall are much more evenly distributed across the wall, compared with moment distribution in the column and middle strips at a slab span supported by two columns at both ends; for an interior span, approximately 75 and 25% of the total negative moment are resisted in column and middle strips respectively, under gravity loads. In short, the moment distribution occurring in the test specimen would have been different from that in the prototype frame subjected to gravity and/or lateral loading. The middle strip reinforcement in the test specimen could have participated in force transfer significantly more than in the actual prototype frame.

4. In conjunction with Item 3, the support condition around the slab edges restrained relative rotational deformations (curvatures) along the slab edges parallel to the direction of lateral loading (x-direction) during testing, which would have occurred in the prototype structure.

Finally, the discussers would like to recommend that tests need to be conducted on three-dimensional frames with multiple spans in both principal directions that allow realistic moment distribution across slab sections and moment redistribution along spans in the nonlinear range, especially when two-way shear and flexural behaviors and their interaction are investigated as in this study.

AUTHORS' CLOSURE

Closure to discussion by Broms

The purpose of the presented tests was to show that test specimens, aiming to study the effectiveness of shear reinforcement in resisting punching of flat plates, have to be designed such that they fail by punching, not flexure. This requirement is obvious in a shear strength test of a simple beam subjected to gravity load. With a low flexural reinforcement ratio, the test beam can fail at midspan, in a ductile form, by yielding of the bottom flexural reinforcement combined with crushing of concrete at the top at large curvature; the flexural failure can occur before the shear strength is reached near the supports. The test results give information on the strength and the ductility in flexural failure; they can only indicate that the shear strength exceeds the maximum shear force that the beam has been exposed to during the test, without giving information on the shear strength or the ductility of the shear failure that has not occurred.

The issue is the same in punching shear tests of flat plates, although it is somewhat obscured by the fact that punching or flexural failure occurs at the same location—in the vicinity of the column. A test exhibiting flexural failure at a low load level that does not demand the full shear strength would not be indicative of the value of the shear strength, the ductility, or the brittleness of the shear failure that has not occurred. When searching for the strength and the ductility in punching shear of flat plates, premature failure by flexure has to be excluded. This can only be achieved by the provision of a sufficiently high flexural reinforcement ratio ρ. This logic does not appear acceptable to the discusser.

The discusser dwells on what he calls deficiencies of the ACI 318 Code and the recommendations of ACI 421.1R-99. In particular, he criticizes that the shear strengths' equations in these sources do not include ρ. The fact that the punching shear strength increases with the increase in ρ is well known and does not need the long explanation in the discussion. The calibration and the reasons behind the equations of the

ACI 318 Code or the recommendations of ACI 421.1R-99 are beyond the scope of the paper. Thus, a response to the claims by the discusser of “false conclusions” in the paper and the “high time” for ACI 318 to be “modernized” should not be in the authors’ closure of the paper.

On the amount of ρ that should be provided in practice, the discusser states, “The level should allow the desirable structural behavior that all reinforcement will reach the yield stress without punching occurring if the nominal punching capacity of the slab exceeds the nominal flexural capacity.” Then he argues, “In practice, ρ would not be chosen to resist a bending moment that is more than 50% larger than the actual bending moment.” The paper does not suggest or imply that ρ in practice should exceed the demand by 50%. The design steps in practice are: select the thickness of the flat plate as a ratio of the span to avoid excessive deflection; determine ρ values at midspans and at the supports, to resist the bending moments obtained by elastic analyses for the loading cases that produce maximum positive and maximum negative values due to factored loads; check the punching shear strength to ensure that it exceeds the maximum demand with the appropriate loading case; and provide shear reinforcing means if necessary. The discusser’s recommendation that all flexural reinforcements (top and bottom) reach their yield stress requires a higher load intensity than the factored design load intensity used in the elastic analyses is neither required by codes nor complied with in practice.

Closure to discussion by Shin and Grossman

The discussers propose an ideal test system that is relevant to all research on punching shear. They recommend a three-dimensional structure with multiple spans in two principal directions. Several structures of this type would be needed to study the effect of varying one or two parameters. The cost and the labor needed to test these ideal structures are prohibitive for practically all researchers.

The tests presented in the paper represented full-size isolated interior connections of a flat plate with spans of 4.8 m (15.75 ft) in two orthogonal directions. The supported edges were at approximately 1/5 the span representing the inflection location due to gravity loads. Under gravity loads combined with unbalanced moments, the zone of inflection in an actual structure would move away from the columns, but would practically not be at midspan.

International codes for punching shear design of flat plates are based on extensive experimental data. The majority of the tests are on isolated specimens, the type presented in the paper. These tests have the advantage that the magnitudes of the shear force V and the unbalanced moment M are accurately measured at all loading stages. Thus, the zone of the slab in the vicinity of the column is transferring forces V and M of known magnitudes, and in the tests, the behavior of the zone in the vicinity of the column is monitored with sufficient accuracy. Scarce tests on three-dimensional structures give information on the behavior of the entire structure (for example, Sherif [1996], Sherif and Dilger [2001], and Dechka [2001]), and at the same time confirm that the behavior of the zone of the slab-column connections can be safely predicted by tests on isolated specimens, for the same V, or V combined with M.

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