International Journal of Concrete Structures and Materials
Vol.5, No.1, pp.57~64, June 2011
DOI 10.4334/IJCSM.2011.5.1.057

Joint Shear Behavior Prediction for RC Beam-Column Connections

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(Received February 26, 2011, Revised May 30, 2011, Accepted May 31, 2011)

Abstract: An extensive database has been constructed of reinforced concrete (RC) beam-column connection tests subjected to cyclic lateral loading. All cases within the database experienced joint shear failure, either in conjunction with or without yielding of longitudinal beam reinforcement. Using the experimental database, envelope curves of joint shear stress vs. joint shear strain behavior have been created by connecting key points such as cracking, yielding, and peak loading. Various prediction approaches for RC joint shear behavior are discussed using the constructed experimental database. RC joint shear strength and deformation models are first presented using the database in conjunction with a Bayesian parameter estimation method, and then a complete model applicable to the full range of RC joint shear behavior is suggested. An RC joint shear prediction model following a U.S. standard is next summarized and evaluated. Finally, a particular joint shear prediction model using basic joint shear resistance mechanisms is described and for the first time critically assessed.

Keywords: experimental database, joint shear behavior, prediction model, bayesian parameter estimation, shear resistance mechanism.

1. Introduction

Properly managing joint shear behavior in reinforced concrete (RC) beam-column connections is required for maintaining reasonable structural response when RC moment resisting frames are subjected to lateral earthquake loading. Numerous experimental and analytical studies have been conducted over the years to improve the understanding of this RC joint shear behavior for structural design, with several approaches having been proposed to predict RC joint shear response. In this paper, a few recent developments related to these modeling approaches are presented, based in part on using an extensive experimental database reflecting much of that previous testing work. First, a joint shear prediction model employing a probabilistic methodology is discussed. This approach follows from Kim and LaFave,[1] who assessed influence parameters at the key points of RC joint shear behavior based on their constructed database, and then Kim et al.,[2] who suggested a procedure to develop RC joint shear strength models in conjunction with a Bayesian parameter estimation method.

As a second approach, a joint shear prediction model per the ASCE/SEI 41-06 standard[3] is described and evaluated. This follows from the FEMA 274[4] guidelines for seismic rehabilitation of existing structures, which was updated to a national pre-standard in FEMA 356[5] and to a standard in the form of ASCE/SEI 41-06[3]. More recently, some parts of ASCE/SEI 41 have been further updated, per Supplement 1.[6]

A third approach to RC joint shear behavior modeling, based on basic joint shear resistance mechanisms, is then discussed. In modeling of RC beam-column connection behavior, several researchers[7-9] have assumed that a joint panel is a cracked RC two-dimensional (2-D) membrane element and applied the modified compression field theory (MCFT)[10] to describe joint shear stress vs. strain. They then also considered strength and energy degradation in order to fully simulate the cyclic response of RC joint shear behavior. However, it has been identified that employing the MCFT may not be appropriate to predict RC joint shear behavior in some conditions, such as when there is poor joint confinement.[8,9,11] Parra-Montesinos and Wight[12] and Mitra and Lowes[11] have suggested RC joint shear models by assuming that joint shear is transferred into a joint panel via designated struts. Here the suggested model of Parra-Montesinos and Wight is briefly presented and then critically evaluated in a comprehensive fashion for the first time.

In brief, then, three approaches to predicting joint shear behavior in RC beam-column connections are explained and evaluated in this paper; those are by using a probabilistic methodology, prescribed code expressions, and basic mechanics considerations. This paper can be beneficial to improving overall understanding of the relative merits across these diverse approaches for predicting joint shear behavior, and for better characterizing joint shear behavior in general.

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2. Experimental database

A consistent set of inclusion criteria has been employed to construct an extensive experimental database for RC beam-column connection subassemblies (laboratory test specimens). Those inclusion criteria can be summarized as follows: (1) all cases within the database were subjected to quasi-static reverse cyclic lateral loading, (2) all cases experienced joint shear failure, either in conjunction with or without yielding of longitudinal reinforcement, (3) all specimens were at least one-third scale, (4) only deformed bars were used for longitudinal beam and column reinforcement, (5) all specimens had only conventional anchorage types (no headed bars or anchorage plates), and (6) all cases had proper seismic hooks. (While headed bars have been excluded from this particular study due to their potential for having different connection behavior, there is an emerging wealth of knowledge and test data about their use in and around joints that could be incorporated in the future.[13]) Then, qualified experimental data were classified according to in-plane geometry, out-of-plane geometry, and failure mode sequence. In the total database, 341 RC beam-column connection experimental cases were included. There is no limit on the degree of joint confinement; some specimens have no joint transverse reinforcement.

Key points displaying the most distinctive stiffness changes in the cyclic overall and local response were identified for each specimen using the constructed database, and then envelope curves were constructed by connecting those key points. The first key point is related to the initiation of diagonal cracking within the joint panel (point A); the second key point corresponds to yielding of longitudinal beam reinforcement or joint transverse reinforcement (point B); and the third key point is at peak response (point C). A descending branch is also needed to describe the full range of RC joint shear stress vs. joint shear strain behavior. The descending branch key point is called point D, with the vertical coordinate of point D simply assigned as 90% of the maximum joint shear stress (this point D ordinate value is typically similar to the level of point B stress). More detailed explanation about the constructed experimental database can be found elsewhere.[14]

3. Prediction approach using probabilistic methodology

3.1 Probabilistic methodology

Probabilistic methods have recently been applied to reduce prediction error (and scatter) for the behavior of RC members, especially for shear capacity. Gardoni et al.[15] suggested a probabilistic procedure to construct RC column shear capacity, relying on an existing deterministic model as a starting point. Then, this approach was updated to develop capacity models without relying on an existing model.[16] That is:

$$\ln[C(x, \Theta)] = \sum_{i=1}^{p} \theta_i h_i(x) + \sigma\varepsilon \quad (1)$$

where C is experimental shear capacity, x is the vector of input parameters that were measured during tests, $\Theta = \theta, \sigma$ denotes the set of unknown model parameters that are introduced to fit the model to the test results, θ is the uncertain model parameter, ε is the normal random variable (with zero mean and unit variance), and σ is the unknown model parameter representing the magnitude of model error that remains after bias-correction. A Bayesian parameter estimation method can be employed to find the distribution of uncertain parameters that makes the models in Eq. (1) best fit the test results.

Song et al.[16] introduced a constant bias-correction term, as shown in Eq. (2).

$$\ln[C(x, \Theta)] = \ln[c_d(x)] + \theta + \sigma\varepsilon \quad (2)$$

This equation can evaluate the overall bias and scatter of any particular used deterministic model (c_d); a deterministic model is less biased when the posterior mean of θ is more close to zero, and it has less scatter when the posterior mean of σ is smaller.

3.2 Joint shear strength models for the peak point (point C)

A rational procedure to develop RC joint shear strength models using the Bayesian parameter estimation method in conjunction with the constructed experimental database is as follows: (a) possible influence parameters were introduced to describe diverse conditions within joint panels of RC beam-column connections; (b) the Bayesian parameter estimation method was employed to find an unbiased joint shear strength model based on the experimental database; (c) at each stage, the least informative parameter was identified; (d) an unbiased model was again constructed after removing the least informative parameter; and (e) steps (c) and (d) were repeated until only the parameters that are most important to determining RC joint shear strength capacity remain.

Possible influence parameters were carefully introduced after consideration through literature review and qualitative assessment.[14] The following parameters were included: (1) concrete compressive strength (f'_c); (2) in-plane geometry (JP = 1.0 for interior, 0.75 for exterior, and 0.5 for knee joints); (3) beam-to-column width ratio (b_b / b_c); (4) beam height to column depth ratio (h_b / h_c); (5) beam reinforcement index (BI, defined as $(\rho_b \times f_{yb}) / f'_c$, in which ρ_b is the beam reinforcement ratio and f_{yb} is the yield stress of beam reinforcement); (6) joint transverse reinforcement index (JI, defined as $(\rho_j \times f_{yj}) / f'_c$, in which ρ_j is the volumetric joint transverse reinforcement ratio and f_{yj} is the yield stress of joint transverse reinforcement); (7) A_{sh} ratio (provided-to-recommended amount of joint transverse reinforcement per ACI 352R-02 design recommendations[17]); (8) spacing ratio (provided-to-recommended spacing of joint transverse reinforcement per ACI 352R-02); (9) out-of-plane geometry (TB = 1.0 for zero or one transverse beam, and 1.2 for two transverse beams); and (10) joint eccentricity ($1-e/b_c$, where e is the eccentricity between the centerlines of the beam and column). Normalized column axial stress and provided-to-required length of beam reinforcement (as a function of bar diameter) were not included in developing prediction models because the Bayesian parameter estimation method indicated that these parameters are not particularly informative, and including these parameters can reduce the efficiency of using the constructed database (due to certain complexities involved with consistently defining these parameters for use across the entire database).

Among the possible influence parameters, those representing out-of-plane geometry, joint eccentricity, joint confinement by transverse reinforcement, joint "confinement" provided by longitudinal beam reinforcement, in-plane geometry, and concrete compressive strength are more informative than the others.

Equation (3) shows the developed joint shear strength model only including the most informative parameters, and Eq. (4) is a simple and unified RC joint shear strength model based on Eq. (3). Concrete compressive strength is the most informative parameter, with its optimized contribution at around 0.75 for the power term, while the parameters become somewhat less informative as one moves from right to left in Eq. (3).

$$v_j(\mathrm{MPa}) = 1.21(\mathrm{TB})^{0.981}\left(1-\frac{e}{b_c}\right)^{0.679}(\mathrm{JI})^{0.136}(\mathrm{BI})^{0.301}(\mathrm{JP})^{1.33}(f_c')^{0.764} \quad (3)$$

$$v_j(\mathrm{MPa}) = \alpha_t\beta_t\eta_t\lambda_t(\mathrm{JI})^{0.15}(\mathrm{BI})^{0.30}(f_c')^{0.75} \quad (4)$$

In Eq. (4), α_t is a parameter for in-plane geometry (1.0 for interior connections, 0.7 for exterior connections, and 0.4 for knee connections); β_t is a parameter for out-of-plane geometry (1.0 for subassemblies with 0 or 1 transverse beams, and 1.18 for subassemblies with 2 transverse beams); η_t $(=(1-e/b_c)^{0.67})$ describes joint eccentricity; and $\lambda_t = 1.31$, which simply makes the average ratio of Eq. (4) to Eq. (3) equal 1.0. Figure 1 plots experimental joint shear stress vs. the simple and unified RC joint shear strength model (Eq. (4)). The total database except specimens with no joint transverse reinforcement were used for the comparison in Fig. 1. Within the total database, 18 cases had no joint transverse reinforcement. For these cases, the experimental joint shear stress to Eq. (4) ratio can be computed by using a trial value of JI. The average of experimental joint shear stress to Eq. (4) is 1.0 when the trial JI is equal to 0.0128. This means that using a virtual JI of 0.0128 enables Eq. (4) to predict joint shear strength for cases with no joint transverse reinforcement. When Eq. (4) is used as a deterministic model in Eq. (2), the means of θ and σ are –0.011 and 0.153, respectively. The simple and unified model (Eq. (4)) is therefore an unbiased model, which means that roughly half of the experimental cases are below their respective Eq. (4) values. For safe application to joint shear strength design, Eq. (4) could for example be multiplied by 0.82 in order to have only about 10% of cases with lower experimental joint shear stress values than this adjusted joint shear strength model.

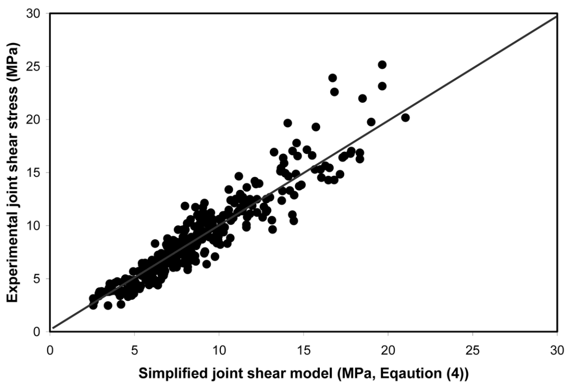


Fig. 1 Experimental joint shear stress vs. Simplified joint shear strength model (Eq. (4)).

3.3 Joint shear deformation models for the peak point (point C)

The same established procedure used to develop RC joint shear strength models was also employed to construct RC joint shear deformation capacity models. As before, possible influence parameters were first carefully determined.[14] To improve this model's accuracy, the ratio of the joint shear strength model (Eqs. (3) or (4)) to concrete compressive strength was even included as a possible influence parameter. Kim and LaFave[1] determined that minimum proper joint confinement is maintained when the A_{sh} ratio is equal to or above 0.70. For the data group maintaining minimum proper joint confinement, certain values for a parameter describing in-plane geometry (JPR) were determined to result in the strongest linear relation between normalized joint shear strain and normalized joint shear stress divided by JPR – 1.0, 0.59, and 0.32 for interior, exterior, and knee joints, respectively. For the total database, then, a new in-plane geometry parameter (JPRU) was determined by dividing JPR by 1.2 to reflect insufficient joint confinement – when the A_{sh} ratio is equal or above 0.7, JPRU is simply 1.0, 0.59, and 0.32, for interior, exterior, and knee joints, respectively, whereas when the A_{sh} ratio is below 0.7, JPRU is 1.0/1.2, 0.59/1.2, and 0.32/1.2 for interior, exterior, and knee joints. Except for these two parameters (joint shear strength model over concrete compressive strength and JPRU), the other included parameters are the same as for development of the RC joint shear strength model.

RC joint shear capacity (deformation, as well as strength) at the peak point is mainly dependent on out-of-plane geometry, in-plane geometry, joint eccentricity, confinement by joint transverse reinforcement, confinement by longitudinal beam reinforcement, and concrete compressive strength. Equation (5) is the developed joint shear deformation model only including the most informative parameters, and Eq. (6) is a simple and unified RC joint shear deformation model based on Eq. (5).

$$\gamma(\mathrm{Rad}) = 0.00565\mathrm{BI}\left(1-\frac{e}{b_c}\right)^{-0.628}(\mathrm{JI})^{0.0982}(\mathrm{TB})^{1.85}\left(\frac{v_j(\mathrm{Eq.(3)})}{f_c'}\right)^{-1.73}(\mathrm{JPRU})^{2.11} \quad (5)$$

$$\gamma(\mathrm{Rad}) = \alpha_{\gamma t}\beta_{\gamma t}\eta_{\gamma t}\lambda_{\gamma t}\mathrm{BI}(\mathrm{JI})^{0.10}\left(\frac{v_j(\mathrm{Eq.(4)})}{f_c'}\right)^{-1.75} \quad (6)$$

In Eq. (6), v_j (Eq.(4)) is the developed simple and unified RC joint shear strength model; α_{rt} $(=(\mathrm{JPRU})^{2.10})$ is a parameter for in-plane geometry; $\beta_{\gamma t}$ is a parameter for out-of-plane geometry (1.0 for subassemblies with zero or one transverse beam, and 1.4 for subassemblies with two transverse beams); $\eta_{\gamma t}$ $(=(1-e/b_c)^{-0.60})$ describes joint eccentricity (1.0 for no eccentricity); and λ_{rt} (= 0.00549) is a factor introduced to make the average ratio of the predictions by the models in Eqs. (6) and (5) equal to 1.0. Figure 2 plots experimental joint shear strain vs. the simple and unified RC joint shear deformation model (Eq. (6)). When Eq. (6) is used as a deterministic model in Eq. (2), the means of θ and σ are –0.117

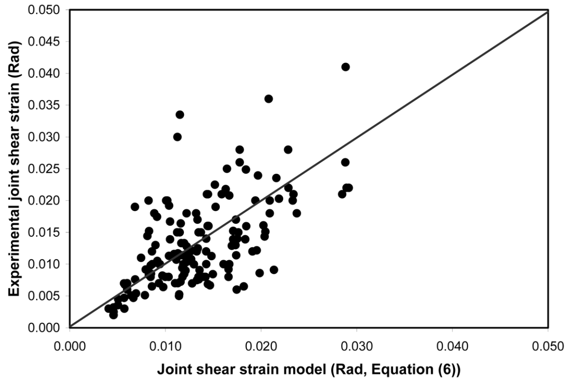
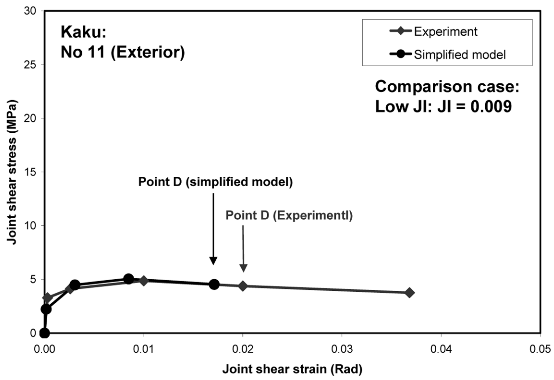


Fig. 2 Experimental joint shear strain vs. Simplified joint shear strain model (Eq. (6)).

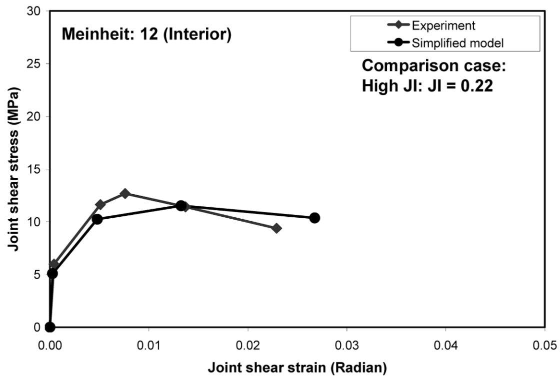
and 0.410, respectively. Model uncertainty for joint shear deformation is distinctively greater compared to that for joint shear strength. This increased scatter is in part due to the fact that experimental joint shear deformation data have not always been collected in exactly the same way across different testing programs, and it is perhaps also the result of having just one equation to address all cases of joint shear failure, including those with some beam longitudinal reinforcement yielding that might contribute to measured shear deformations in the joint.

3.4 Joint shear behavior model for other key points

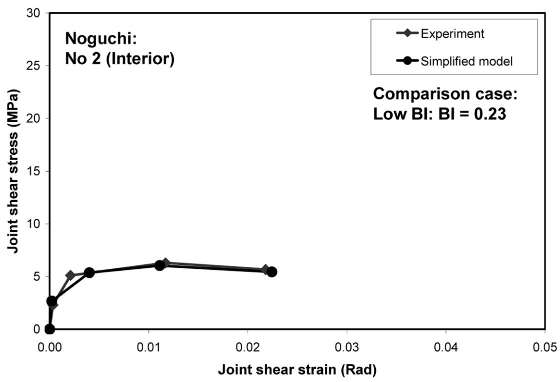
Kim and LaFave[14] have provided a detailed explanation about further RC joint shear stress and strain models employing the Bayesian parameter estimation method at the other key points (A, B, and D). Those developed models indicate that essentially the same key influence (most informative) parameters on joint shear stress and strain were maintained across all of the key points. Thus, at the other key points joint shear stress and strain models can also be suggested as simply the product of constant factors



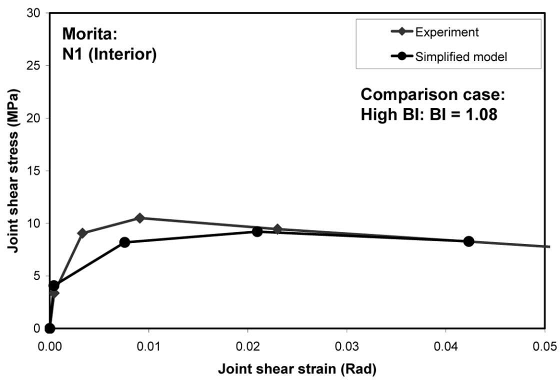
(a) Comparison case: low JI



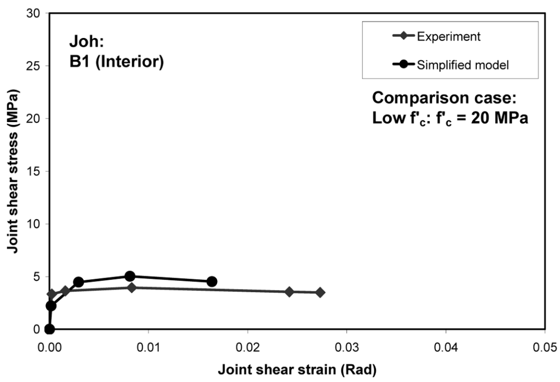
(b) Comparison case: high JI



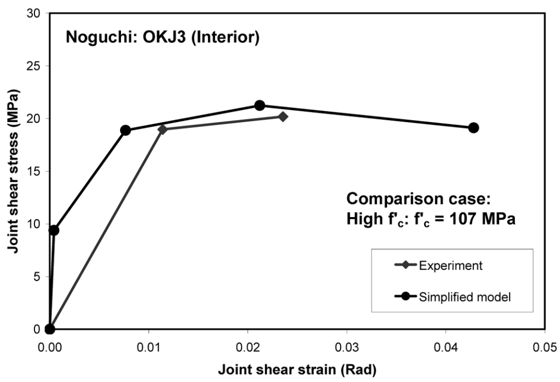
(c) Comparison case: low BI



(d) Comparison case: high BI



(e) Comparison case: low f'c



(f) Comparison case: high f'c

Fig. 3 Comparison of full range of experimental and suggested joint shear stress vs. strain.

times the simple and unified models (Eqs. (4) or (6)). For joint shear stress, the reduction factors are 0.442, 0.889, and 0.9 for points A, B, and D, respectively. For joint shear strain, the factors are 0.0198, 0.361, and 2.02 for points A, B, and D, respectively.

Kim and LaFave[14] have compared experimental joint shear behavior vs. the full range of this relatively simple joint shear behavior model; the proposed model reasonably matches with experimental joint shear behavior (although some modest local biases do exist). For example, Fig. 3 compares experimental and suggested joint shear stress vs. strain for the extreme cases of JI, BI, and f_c'. Even in these extreme cases of JI, BI, and f_c', the suggested model shows quite good agreement for the envelope curve of joint shear stress vs. strain, when compared to the experimental results.

4. Prediction approach per U.S. reference standards

Figure 4 shows an envelope model of RC joint shear stress vs. joint shear strain behavior when subjected to lateral loading, which has been defined in Chapter 6 of ASCE/SEI 41-06[3]. In Fig. 4, joint shear stiffness (AB slope) is needed to determine the specific location of point B for joint shear behavior. However, there is little information in ASCE/SEI 41-06 about this; therefore, the X-coordinate of point B is not considered in this research, as represented in Fig. 5. In this model, ASCE/SEI 41-06 Supplement 1[6] has a similar approach to ACI 318-08[18] for defining joint shear strength. However, while ACI 318 only deals with well-confined joints, Supplement 1 covers all types of RC beam-column connections – it considers that an RC joint panel is confined as long as the spacing of joint transverse reinforcement is equal or less than one-half the column depth. Table 1 shows the RC joint shear strength factors defined in ASCE 41-06 Supplement 1, which are determined according to in-plane geometry, out-of-plane geometry, and confinement condition; for "well-confined" joints (with $s/h_c < 0.5$), these factors are identical to those in ACI 318-08 for use in design, except that ACI 318 does not have a reduced value specifically for knee joints. In Supplement 1, joint shear deformation is defined according to connection type, column axial compression, confinement condition, and ratio of design shear force to nominal shear capacity. In ASCE/SEI 41-06 Supplement 1, plastic joint shear strain (peak minus yielding strain, "a") is defined as 0.015 for confined interior joints, 0.010 for confined exterior and knee joints, and 0.005 for unconfined joints (as mainly determined by the provided spacing of joint transverse reinforcement over the column depth (s/h_c)).

Figure 6 plots experimental joint shear stress vs. joint shear models (Eq. (4) or the joint shear strength model of ASCE 41-06 Supplement 1[6]). As explained in the "Joint Shear Behavior Model for Other Key Points" section, point B joint shear deformation is the product of 0.361 and Eq. (6). So, plastic joint shear deformation

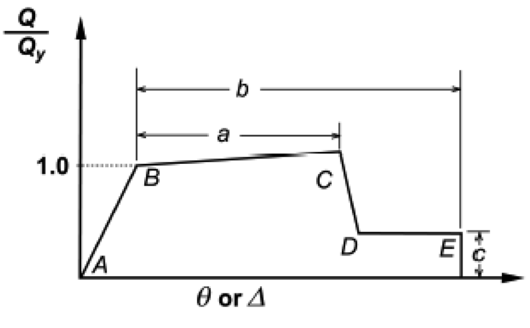


Fig. 4 FEMA 356 (ASCE/SEI 41-06) generic joint shear behavior model.

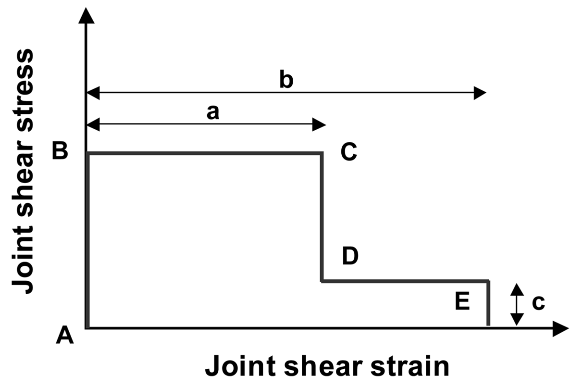


Fig. 5 Adjusted FEMA 356 (ASCE/SEI 41-06) joint shear behavior model.

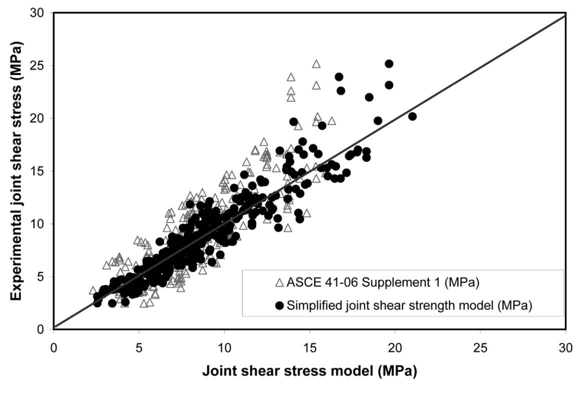


Fig. 6 Comparison of joint shear stress models: simplified model and ASCE 41-06 Supplement 1.

Table 1 Joint shear strength factors (ASCE/SEI 41-06 Supplement 1).

Interior joints				Exterior joints				Knee joints	
0 or 1 TB*		2 TB		0 or 1 TB		2 TB			
s/h_c&	γ#	s/h_c	γ	s/h_c	γ	s/h_c	γ	s/h_c	γ
≤ 0.5	1.25	≤ 0.5	1.67	≤ 0.5	1.00	≤ 0.5	1.25	≤ 0.5	0.67
> 0.5	0.83	> 0.5	1.0	> 0.5	0.50	> 0.5	0.67	> 0.5	0.33

* : Transverse beam (s)
\# : Joint shear strength factor ($MPa^{0.5}$)
& : Spacing of joint transverse reinforcement / column depth

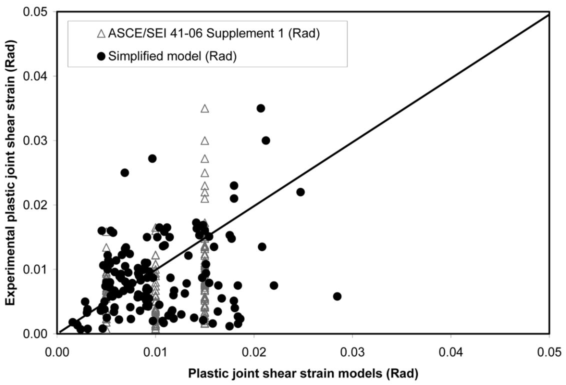


Fig. 7 Comparison of plastic joint shear strain: simplified model and ASCE 41-06 Supplement 1.

experimental plastic joint shear deformation vs. plastic joint shear deformation by ASCE/SEI 41-06 or the product of 0.639 and Eq. (6). For peak stress, the Supplement 1 model yields an average and coefficient of variation of the ratio of experiment to the model that are 1.08 and 0.26, respectively, which means the prediction accuracy of ASCE/SEI 41-06 Supplement 1 for strength is lower than for the suggested model (Eq. (4)). As shown in Fig. 7, the Supplement 1 constant values result in a large scatter when determining plastic joint shear deformation. Actual (experimental) plastic joint shear deformations show quite a wide range for the same X-value, so it is somewhat difficult to say that plastic joint shear deformation is clearly a function of s/h_c. These examination results indicate that even more improvement in prediction may be recommended for the ASCE 41-06 Supplement 1 model.

5. Prediction approach using basic joint shear resistance mechanisms

Parra-Montesinos[19] has suggested an innovative analytical approach for estimating joint shear behavior of composite RC column-to-steel beam connections. In that approach, principal joint shear strain is defined in a joint panel by relating principal tensile and compressive strains. Parra-Montesinos and Wight[12] then adjusted this analytical approach for estimating joint shear behavior of RC beam-column connections. Concrete struts and/or trusses are the generally accepted joint shear resistance mechanisms – a strut mechanism is formed from resisting the transferred force by concrete compression zones of adjacent beam(s) and column(s), whereas a truss mechanism is formed from resisting the transferred force via bond between reinforcement and surrounding concrete in the joint panel. Parra-Montesinos and Wight considered that RC joint shear strength is provided by an equivalent diagonal compression strut activated by force transfer to the joint by direct bearing from the beam and column compression zones and by bond between the beam and column reinforcement and the surrounding concrete. Figure 8 shows the equivalent concrete strut they suggested; the shape of this equivalent strut can be determined by the geometry and reinforcement array of the beams and columns. Parra-Montesinos and Wight further suggested a possible relationship between RC joint shear strain and principal strain ratio, based on their collected experimental test results; that is:

$$K_{tc} = 2 + k_s\gamma \tag{7}$$

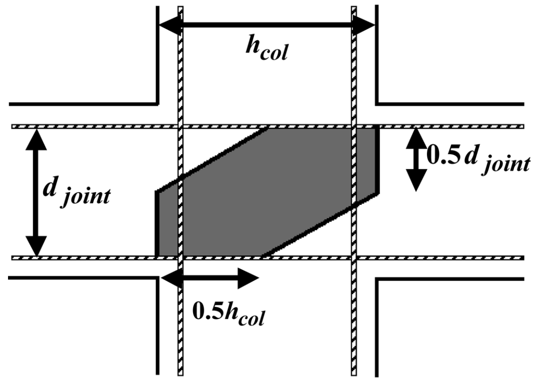


Fig. 8 Equivalent strut per Parra-Montesinos and wight.

where K_{tc} is the ratio of principal tensile strain to principal compressive strain ($-\varepsilon_t/\varepsilon_c$); γ is the joint shear strain; and k_s is a suggested slope for the K_{tc} vs. γ relationship ($= 500 + 2000\ e/b_c$). Parra-Montesinos and Wight commented that Eq. (7) is valid only up to about 1% joint shear deformation (for subassemblies maintaining proper confinement within the joint panel), because they derived this relation based only on their nine experimental results. They also recommended that more data are needed in order to derive general relations between joint shear deformation and the principal strain ratio.

Plane strain within a joint panel can be expressed as the following three equations, using strain coordinate transformation; that is:

$$\varepsilon_c = \frac{\varepsilon_x + \varepsilon_y}{2} + \frac{\varepsilon_x - \varepsilon_y}{2}\cos(2\theta) + \frac{\gamma}{2}\sin(2\theta) \tag{8}$$

$$\varepsilon_t = -K_{tc}\varepsilon_c = \frac{\varepsilon_x + \varepsilon_y}{2} + \frac{\varepsilon_x - \varepsilon_y}{2}\cos\left[2\left[\theta + \frac{\pi}{2}\right] + \frac{\gamma}{2}\sin\left[2\left(\theta + \frac{\pi}{2}\right)\right]\right] \tag{9}$$

$$\gamma = \tan(2\theta)(\varepsilon_x - \varepsilon_y) \tag{10}$$

where ε_x, ε_y, ε_c, and ε_t are the strain for the X, Y, principal compressive, and principal tensile directions, respectively. When joint shear strain (γ) is given and the principal strain direction (θ) is known, four plane strains (ε_x, ε_y, ε_c, and ε_t) can be determined by employing Eqs. (7), (8), (9), and (10).

Parra-Montesinos and Wight[12] employed used the concrete stress vs. strain model of Sheikh and Uzumeri[20] to find the principal compressive stress at the calculated principal compressive strain. Then, the horizontal component of the diagonal compressive stress can be taken as the horizontal joint shear stress for any given joint shear strain.

Kim and LaFave[14] have provided analytical joint shear stress vs. joint shear strain values, as proposed by Parra-Montesinos and Wight[12] (up to 1% joint shear deformation), for interior and exterior connections of confined joints with no out-of-plane members (due to limitations of the method). Figure 9 plots experimental joint shear stress vs. joint shear stress defined by the Parra-Montesinos and Wight[12] model, and this model is compared to the suggested simple and unified model (Eq. (4)). Joint shear strength (defined by Parra-Montesinos and Wight) appears to be

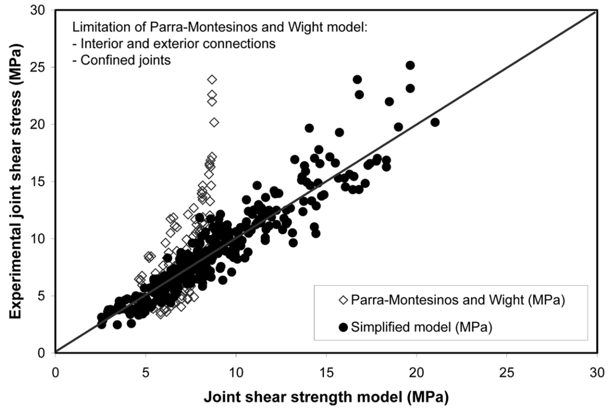


Fig. 9 Comparison of joint shear stress models: simplified model and Parra-Montesinos and Wight model.

determined too conservatively, in part due to Eq. (7) and the upper limit (1%) on joint shear deformation. The key point of the Parra-Montesinos and Wight model is Eq. (7), and this equation was based on only a few experimental data. By perhaps defining Eq. (7) using the total database, to consider diverse types of RC beam-column connections, the current limitations of the Parra-Montesinos and Wight model may be reduced (a possible modification to the Parra-Montesinos and Wight model will be discussed in a future paper.).

Figure 10 plots experimental joint shear strain vs. joint shear strain by the Parra-Montesinos and Wight model, and this model is also compared to the suggested simple and unified model (Eq. (6)). The overall means of θ and σ are computed as 0.442 and 0.410, respectively, for the Parra-Montesinos and Wight model. The constant upper limit (1%) of joint shear deformation is apparently not reasonable as a general approach for determining joint shear deformation at maximum response.

6. Conclusions and recommendations

Various prediction approaches for joint shear stress vs. strain behavior in RC beam-column connections subjected to seismic lateral loading have been discussed and assessed, in part by using an extensive experimental database. Key findings can be summarized as follows:

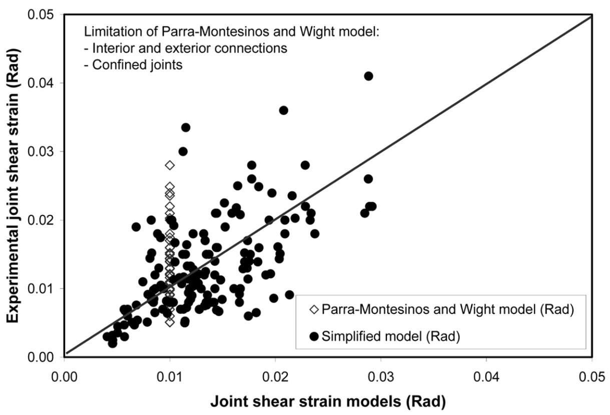


Fig. 10 Comparison of joint shear strain models: simplified model and Parra-Montesinos and Wight model.

1) RC joint shear strength and deformation (at peak stress) models have been developed by employing the Bayesian parameter estimation method. Joint shear stress and strain models similarly developed for other key points indicate that the most informative parameters are generally maintained across the full range of RC joint shear stress vs. strain behavior. Thus, a convenient full-range RC joint shear behavior model has been suggested by just using simple and unified RC joint shear strength and deformation models. Within the range of the constructed database, the suggested model predicts RC joint shear behavior in a very reliable manner.

2) ASCE/SEI 41-06 Supplement 1 employs a simplified tabular approach to define the full range of joint shear stress vs. strain behavior. However, it is difficult to say that this joint shear behavior model can reasonably describe various parameters' effects on RC joint shear behavior; some prediction improvement for this model could be warranted.

3) RC joint shear resistance mechanisms typically consist of concrete struts and/or trusses, so the mechanical modeling approach suggested by Parra-Montesinos and Wight is a promising one. However, their current model has several limitations, such as an upper limit on joint shear strain and applicability to only certain connection types. To overcome these limitations, and based on this first-ever critical assessment of the model, the approach of the Parra-Montesinos and Wight model is now being modified; these refinements will be discussed in a future publication.

References

1. Kim, J. and LaFave, J. M., "Key Influence Parameters for the Joint Shear Behaviour of Reinforced Concrete (RC) Beam-Column Connections," *Engineering Structures*, Vol. 29, No. 10, 2007, pp. 2523~2539.

2. Kim, J., LaFave, J. M., and Song, J., "A New Statistical Approach for Joint Shear Strength Determination of RC Beam-Column Connections Subjected to Lateral Earthquake Loading," *Structural Engineering and Mechanics*, Vol. 27, No. 4, 2007, pp. 439~456.

3. American Society of Civil Engineers, *Seismic Rehabilitation of Existing Buildings* (ASCE/SEI 41-06), ASCE, 2006.

4. Federal Emergency Management Agency, *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Building* (FEMA 274), FEMA, 1997.

5. Federal Emergency Management Agency, *Prestandard and Commentary for the Seismic Rehabilitation of Building* (FEMA 356), FEMA, 2000.

6. Elwood, K. J., Matamoros, A. B., Wallace, J. W., Lehman, D. E., Heintz, J. A., Mitchell, A. D., Moore, M. A., Valley, M. T., Lowes, L. N., Comartin, C. D., and Moehle, J. P., "Update to ASCE/SEI 41 Concrete Provisions," *Earthquake Spectra*, Vol. 23, No. 3, 2007, pp. 493~523.

7. Youssef, M. and Ghobarah, A., "Modeling of RC Beam-Column Joints and Structural Walls," *Journal of Earthquake Engineering*, Vol. 5, No. 1, 2001, pp. 93~111.

8. Lowes, L. N. and Altoontash, A., "Modeling Reinforced-Concrete Beam-Column Joints Subjected to Cyclic Loading," *Journal of Structural Engineering*, ASCE, Vol. 129, No. 12, 2003, pp. 1686~1697.

9. Shin, M. and LaFave, J. M., "Modeling of Cyclic Joint Shear Deformation Contribution in RC Beam-Column Connections to Overall Frame Behavior," *Structural Engineering and Mechanics*, Vol. 18, No. 5, 2004, pp. 645~669.

10. Vecchio, F. J. and Collins, M. P., "The Modified Compression-Field Theory for Reinforced Concrete Elements Subjected to Shear," *ACI Structural Journal*, Vol. 83, No. 2, 1986, pp. 219~231.

11. Mitra, N. and Lowes, L. N., "Evaluation, Calibration, and Verification of a Reinforced Concrete Beam-Column Joint Model," *Journal of Structural Engineering*, ASCE, Vol. 133, No. 1, 2007, pp. 105~120.

12. Parra-Montesinos, G. J. and Wight, J. K., "Prediction of Strength and Shear Distortion in R/C Beam-Column Joints," *Behavior and Design of Concrete Structures for Seismic Performance (SP-197)*, American Concrete Institute, Detroit, Michigan, 2002, pp. 191~214.

13. Choi, D.-U., "Test of Headed Reinforcement in Pullout II: Deep Embedment," *International Journal of Concrete Structures and Materials*, Vol. 18, No. 3E, 2006, pp. 151~159.

14. Kim, J. and LaFve, J. M., *Joint Shear Behavior of Reinforced Concrete Beam-Column Connections Subjected to Seismic Lateral Loading*, NSEL-20, Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, 2009.

15. Gardoni, P, Der Kiureghian, A., and Mosalam, K. M., "Probabilistic Capacity Models and Fragility Estimates for Reinforced Concrete Columns Based on Experimental Observations," *Journal of Engineering Mechanics*, Vol. 128, No. 10, 2002, pp. 1024~1038.

16. Song, J., Kang, W. H., Kim, K. S., and Jung, S., "Probabilistic Shear Strength Models for Reinforced Concrete Beams by Bayesian Updating Based on Experimental Observations," *5th Conference on Computational Stochastic Mechanics*, Rodos, Greece, 2007, pp. 623~632.

17. ACI-ASCE Committee 352, *Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structure* (ACI 352R-02), American Concrete Institute, Farmington Hills, MI, 2002.

18. ACI Committee 318, *Building Code Requirements for Reinforced Concrete (ACI 318-08) and Commentary (ACI 318R-08)*, American Concrete Institute, 2008.

19. Parra-Montesinos, G. J., *Seismic Behavior, Strength, and Retrofit or Exterior RC Column-to-Steel Beam Connections*, PhD Thesis, Department of Civil and Environmental Engineering, University of Michigan, 2000.

20. Sheikh, S. A. and Uzumeri, S. M., "Analysis Model for Concrete Confinement in Tied Columns," *Journal of Structural Engineering,* ASCE, Vol. 108, No. 12, 1982, pp. 2703~2722.