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Investigation of shear strength models for exterior RC beam-column joint

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Abstract. Various models have been proposed by several researchers for predicting the exterior RC beam-column joint shear strength. Most of these models were calibrated and verified with some limited experimental database. From the models it has been identified that the joint shear strength majorly depends on ten governing parameters. In the present paper, detailed investigation of twelve analytical models for predicting shear strength of exterior beam-column joint has been carried out. The study shows the effect of each governing parameter on joint shear strength predicted by various models. It has been observed that the consensus on effect of few of the governing parameters amongst the considered analytical models has not been attained. Moreover, the predicted joint strength by different models varies significantly. Further, the prediction of joint shear strength by these analytical models has also been compared with a set of 200 experimental results from the literature. It has been observed that none of the twelve models are capable of predicting joint shear strength with sufficient accuracy for the complete range of experimental results. The research community has to reconsider the effect of each parameters based on larger set of test results and new improved analytical models should be proposed.

Keywords: RC exterior beam-column joint; seismic behavior; shear strength; parametric analysis; experimental database

1. Introduction

The study of behavior of RC beam column joints under seismic loading and endeavors for predicting the joint shear strength has started long back (Hanson and Connor 1967 as reported by Park and Pauley 1975), however, the collapse of many RC buildings in recent past earthquakes has been attributed to failure of beam-column joints and numerous of contemporary research on retrofitting of poorly detailed joints are also being conducted (Bansal *et al.* 2016, Li *et al.* 2015). The prediction of accurate joint shear strength is difficult task since it depends on complex interactions of various parameters. Many researchers have proposed analytical models for predicting joint shear strength; however, these analytical models are generally based on limited set of experimental results. From the literature, the crucial parameters affecting the joint shear strength have been identified as beam width, beam depth, column width, column depth, concrete

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compressive strength, yield strength of reinforcement, amount of column and beam reinforcement, shear reinforcement inside joint zone and axial compressive load from column. Considering some of the aforementioned parameters, various models for computing joint shear strength have been proposed by different researchers. It has been identified that the researchers used three different approaches viz. empirical, strut and tie and theoretical approach, in developing the equations for calculating joint shear strength. Empirical models are generally developed by assessing the effect of various parameters influencing the joint shear strength using experimental database. Zhang and Jirsa (1982) developed a model based on the mechanism of formation of plastic hinges in the beam. The model is based on the analysis of experimental database on exterior joints. Sarsam and Philips (1985) proposed a semi empirical model considering several influencing parameters i.e., concrete compressive strength, column longitudinal reinforcement ratio and column axial load based on the regression analysis of experimental results. Vollum and Newman (1999) developed a semi empirical model based on the experimental database. The model considers the effect of parameters like joint aspect ratio and beam reinforcement details. They have identified that the shear transfer occurs through a fixed angle softened strut-and-tie mechanism but did not consider the effect of diagonal strut in the formulation. Bakir and Boduroğlu (2002) proposed an empirical model based on regression analysis of small experimental database. The model considers the beam longitudinal reinforcement as additional parameter in the formulation, however, neglected the effect of column reinforcement and axial load. Hegger *et al.* (2003) proposed an empirical model considering the effect of concrete strength, aspect ratio, amount of column reinforcement and joint shear reinforcement. Russo and Somma (2004) developed the empirical model considering contribution of tensile strength of concrete and reinforcement, column axial load and joint shear reinforcement based on evaluation of experimental database. Using probabilistic analysis method Kim *et al.* (2009) proposed empirical model for exterior joint using large experimental database. The empirical model proposed by Unal and Burak (2013) predicts the joint shear strength and strain relationship based on collected experimental test results. Recently, Tran *et al.* (2014) proposed an empirical model for both interior and exterior joint considering bond strength between concrete and longitudinal reinforcement of beam as an additional parameter.

Strut and Tie model consist of the mechanism developed by both concrete strut and reinforcement ties inside the joint. The idea behind the strut and tie approach as proposed by Paulay and Priestley in 1992 is, under seismic forces the joint develops two mechanism i.e. diagonal strut mechanism depends on the concrete compressive strength and the truss mechanism based on the bond strength between concrete and reinforcemnt. With this concept, Ortiz in 1993 developed strut and tie model for predicting the joint shear strength for unreinforced and reinforced exterior joints. Later, Parker and Bullman (1997) considered the diagonal concrete strut mechanism for resisting joint shear force and neglected the tie effect. In the same year, Hwang and Lee (1999) proposed iterative algorithm for predicting joint shear strength based on softened strut and tie model considering equilibrium, compatibility and constitutive laws for cracked reinforced concrete sections. Again in 2002 the same authors proposed a simplified strut and tie model. Pantelides *et al.* (2002) developed strut and tie model based on the experimetnal test results on unconfined exterior joints. This model considers the failure of joint either by the crushing of compressive strut or by the failure of compression nodes. Similar to Hwang and Lee (1999), in 2008 Wong *et al.* developed a modified rotating angle strut and tie model based on modified compression field theory (MCFT). Vollum and Parker (2008) proposed an iterative rational strut and tie model consistant with the Eurocode design recommendations. Park and Mosalam (2012) proposed an empirical shear strength model based on single strut mechanism suitable for unreinforced exterior joints. In this model the effect of two paramters i.e. joint aspect ratio and beam longitudinal tension reinforcement have been included and the model developed based on 62 experimental results of unreinforced exterior joints. Recently, Pauletta *et al.* (2015) developed a model for joint shear strength based on the contribution of two diagonal concrete struts along with joint horizontal hoops and column intermediate vertical bars. The model has been calibrated using 61 experimetnal test results.

Theorotical models are based on either the mechanistic or the principle stress approach. Taylor (1974) proposed a model based on principal stress approach considering only concrete tensile strength and column axial load parameters. Scott *et al.* (1994) proposed a theoritical model based on single diagonal concrete strut and neglecting the effect of both horizontal and vertical stirrups. The concrete compressive strength and geometry of beam and column have been considered as governing parameter in this model. Priestley (1997), Hakuto *et al.* (2000) suggested the principle tensile stress approach to calculate the joint shear strength without joint shear reinforceemnt. Attalla (2004) presented a theorotical model considering the compression-softening phenomenon associated with the cracked reinforced concrete in compression. The effect of joint geometry and the presence of transverse beams are also consiered on joint shear strength. A fifth order polynomial equation was proposed by Tsonos (2002, 2007) to find the ultimate joint shear strength. The model considers the combination of softened diagonal strut mechanism similar to Hwang and Lee's (1999) consideration and the truss mechanisms. Tsonos's model satisfies the constitutive laws by complying with Mohr's circle compressive and tensile principal stresses and adopted the fifth degree parabola for concrete biaxial strength curve. Wang *et al.* (2011) proposed a theorotical model for both interior and exterior joints based on Kupfer-Gerstle biaxial tension-compression failure envelope. In this model the reinforced concrete inside the joint panel is considered as homogenous material in a plane stress state and the contributions of joint horizontal stirrups and column intermediate reinforcing bars are also considered. Sharma *et al.* (2012) used the limiting pricipal tensile stress approach in the joint as failure criteria. The model considers the column axial load and concrete compressive and tensile strength as main affecting parameters. Kotsovou and Mouzakis (2012) developed a theoritical model based on equilibrium of diagonal strut mechanism along with experimental study.

Due to brevity only twelve models have been selected in the present study. In the considered models, seven models are based on empirical approach (viz. Sarsam and Phillips 1985, Vollum and Newman 1999, Bakir and Boduroğlu 2002, Hegger *et al.* 2003, Kim *et al.* 2009, Unal and Burak 2013, Tran *et al.* 2014), three Models are based on strut and tie approach (viz. Paulay and Priestley 1992, Hwang and Lee 2002, Pauletta *et al.* 2015) and two models are based on theoretical approach (viz. Wang *et al.* 2011, Sharma *et al.* 2012). These analytical models are then compared with a collected database of 200 experimental results. This paper brings out the limitations of the aforementioned twelve analytical models and identifies the effect of different governing parameters considered in the formulation. The paper also emphasize on the requirement of better analytical model to predict joint shear strength.

2. Review of analytical models

This section present the details of twelve models for exterior beam column joint considered in the study along with their applicability and limitations. The governing parameters of each model have also been identified.

2.1 Sarsam and Phillips (1985)

Sarsam and Phillips (1985) proposed a model based on empirical approach for the design of exterior RC beam-column joint. This model considers mainly the effect of concrete grade (f_c), column reinforcement ratio (ρ_c), depth of column (h_c) and beam (h_b), column axial stress (N_c/A_c), joint shear reinforcement (A_{sj}) and yield strength of shear reinforcement (f_{ysj}) as governing parameters for the joint shear strength.

$$V_{jh} = \left[5.08\, f_c \rho_c^{0.33} \frac{h_c}{h_b} 1.33 b_c h_c \sqrt{\left(1 + 0.29 \frac{N_c}{A_c}\right)}\right] + \left[0.87\, f_{ysj} A_{sj}\right] \tag{1}$$

where, ρ_c is the column longitudinal reinforcement ratio ($A_{sc}/b_c h_c$), in this A_{sc} is the area of the layer of steel farthest from the maximum compression face in the column. Depending on the compressive strength of the concrete ($f_c \leq 70$ MPa), column reinforcement ratio ($\rho_c \leq 0.02$) and column axial stress ($N_c/A_c \leq f_c/3$) the model limits the maximum joint shear strength to,

$$V_{jh} \leq 2.4 (f_c)^{0.33} b_c h_c \tag{2}$$

2.2 Paulay and Priestley (1992)

Paulay and Priestley (1992) proposed a basic theoretical model for evaluating joint shear strength based on sum of two shear resisting mechanisms i.e. strut and truss mechanisms. Strut mechanism is dependent on the concrete compressive strength while the truss mechanism on bond strength between concrete and reinforcement. The formulation consists of parameters like concrete strength (f_c), width of beam (b_b) and column (b_c), depth of column (h_c), column axial stress (N_c/A_c), beam longitudinal reinforcement (A_{sb}), joint shear reinforcement (A_{sj}) and yield strength of beam longitudinal reinforcement (f_{yb}) and joint shear reinforcement (f_{ysj}). The total joint shear strength is based on contributions from concrete strut (V_{ch}) and shear reinforcement (V_{sh}).

$$V_{jh} = V_{ch} + V_{sh} \tag{3}$$

The contribution of concrete strut (V_{ch}) and the column shear force (V_c) is computed as:

$$V_{ch} = \left(1 - \frac{\beta}{\lambda}\right) T + \frac{1.4 c \beta}{\lambda h_c} T - V_c \tag{4}$$

$$V_c = \frac{2TZ_b + V_b h_c}{l_c} \tag{5}$$

in which the coefficients β represents the ratio of beam top to bottom longitudinal reinforcement and c denotes the depth of flexural compression zone in the column [$c = \left(0.25 + 0.85\, N_C / f_c A_c\right) h_c$]. The shear strength provided by horizontal joint shear reinforcement is,

$$V_{sh} = f_{ysjh} A_{sjh} \tag{6}$$

The tensile force (T) from beam reinforcement is calculated using the over strength factor (λ)

(taken in between 1.2 to 1.4) as, $T=\lambda f_{yb} A_{sbtop}$.

In this model the effect of column reinforcement has not been considered. The effect of concrete grade has not been considered explicitly, however, its contribution is considered in the depth of flexural compression zone via column axial load ratio ($N_c/A_c f_c$). To avoid brittle failure of diagonal concrete compression strut mechanism in the joint, the horizontal shear stress should be limited as

$$V_{jh} = \frac{V_{jh\lim}}{A_{jh}} \le 0.25 f_c \le 9MPa \tag{7}$$

2.3 Vollum and Newman (1999)

Vollum and Newman (1999) proposed a semi empirical model based on parametric study for exterior beam column joint subjected to monotonic loading. Several parameters like concrete strength (f_c), depth of beam (h_b) and column (h_c), width of beam (b_b) and column (b_c), joint shear reinforcement (A_{sj}) and yield strength of shear reinforcement (f_{ysj}) have been considered for modeling. The model also considers anchorage type of beam longitudinal reinforcement. However, it neglects the effect of column axial load and longitudinal reinforcement in beam and column.

$$V_{jh} = V_{ch} + \left(A_{sj} f_{ysj} - \alpha b_j h_c \sqrt{f_c}\right) \tag{8}$$

where, the coefficient α is taken to consider the effect of column load, concrete strength, stirrup index, and joint aspect ratio conservatively taken equal to 0.2. The joint shear strength without shear reinforcement (V_{ch}) can be determined by

$$V_{ch} = 0.642\zeta\left[1 + 0.555\left(2 - \frac{h_b}{h_c}\right)\right] b_j h_c \sqrt{f_c} \tag{9}$$

where, the factor, ξ represents the detailing of reinforcement, ξ=1.00 for L bend and ξ=0.90 for U bend bars bent into the joint. Finally the equation for V_{jh} becomes

$$V_{jh} = \left[0.642\xi\left[1 + 0.552\left(2 - {h_b}/{h_c}\right)\right] b_j h_c \sqrt{f_c}\right] + \left[A_{sj} f_{sj}\right] \tag{10}$$

Based on the assumption that the higher joint aspect ratio reduces the joint shear strength and hence the joint shear strength is limited based on the upper limit of aspect ratio as given below

$$V_{jh} = 0.97 b_j h_c \sqrt{f_c}\left[1 + 0.555\left(2 - \frac{h_b}{h_c}\right)\right] \le 1.33 b_j h_c \sqrt{f_c} \tag{11}$$

2.4 Bakir and Boduroğlu (2002)

Bakir and Boduroğlu (2002) proposed an empirical design equation based on regression analysis of 58 experimental test results. This model considers mainly the parameters like, concrete strength (f_c), width of beam (b_b) and column (b_c), depth of beam (h_b) and column (h_c), beam longitudinal reinforcement (A_{sb}), joint shear reinforcement (A_{sjh}) and yield strength of joint shear reinforcement (f_{ysj}). Similar to the previously proposed model by Vollum and Newman (1999), the effect of anchorage of beam bar inside the joint has also been considered. The effect of column axial load and column reinforcement has not been considered.

$$V_{jh} = \left[\frac{0.71\beta\gamma \left({100 A_{sb}} / {b_b h_b} \right)^{0.4289} \left(\frac{b_b + b_c}{2} \right) h_c \sqrt{f_c}}{\left({h_b}/{h_c} \right)^{0.61}} \right] + \left[\alpha A_{sj} f_{ysj} \right] \tag{12}$$

where, β=0.85 for the joints detailed by U type bars and β=1.00 for L type bars. γ=1.37 for inclined bars inside joint and γ=1.00 for other cases. α=0.664 for joints with low amount of stirrups; α=0.60 for joints with medium amount of stirrups; and α=0.37 for joints with low amount of stirrups. In this the joints are consider to have low amount of stirrups when the stirrups ratio is below 0.003, for medium amount of stirrups when ratio ranges between 0.003 to 0.0055 and for high amount of stirrups for ratio exceeds 0.0055.

2.5 Hwang and Lee (2002)

Hwang and Lee (2002) proposed a model based on strut and tie approach. The model is based on contributions from three types of struts in the joint i.e. diagonal, horizontal, and vertical. The model considers the effect of parameters like concrete grade (f_c), width of beam (b_b) and column (b_c), depth of beam (h_b) and column (h_c) and the column axial stress (N_c/A_c). The horizontal shear strength is provided by the component in horizontal axis as

$$V_j = C_d \cos\theta \tag{13}$$

where, the diagonal compressive strength is expressed as $C_d = K\xi f_c A_{str}$. The coefficient of stiffness ξ depend on concrete compressive strength as, $\xi = \left(3.35 / \sqrt{f_c} \right) \le 0.52$.

The area of diagonal strut, $A_{str} = a_s b_j$, in which a_s represents the strut depth and b_j is the effective width of joint.

$$a_s = \left(0.25 + 0.85 N_c / A_c f_c \right) h_c \tag{14}$$

The factor K represents the effect of tie force on joint shear strength and it is the sum of horizontal (K_h) and vertical (K_v) tie forces.

$$K = K_h + K_v \tag{15}$$

$$K_h = 1 + \left(\overline{K_h} - 1 \right) \frac{F_{yh}}{\overline{F_h}} \le \overline{K_h} \tag{16}$$

$$K_v = 1 + \left(\overline{K_v} - 1 \right) \frac{F_{yh}}{\overline{F_h}} \le \overline{K_v} \tag{17}$$

The contribution of horizontal ($\overline{K_h}$) and vertical ($\overline{K_v}$) tie force to the diagonal compressive strength are represented by following expressions

$$\overline{K_h} = \frac{1}{1 - 0.2\left(\gamma_h - \gamma_h^2 \right)} \text{ and } \overline{K_v} = \frac{1}{1 - 0.2\left(\gamma_v - \gamma_v^2 \right)} \tag{18}$$

$$\overline{F_h} = \gamma_h\left(\overline{K_h}\xi f_c A_{str}\right)\cos\theta \text{ and } \overline{F_v} = \gamma_v\left(\overline{K_v}\xi f_c A_{str}\right)\sin\theta \tag{19}$$

The strain parameters γ_h and γ_v of above equations can be evaluated as

$$\gamma_h = \frac{2\tan\theta - 1}{3} \quad 0 \le \gamma_h \le 1.00 \quad \text{and} \quad \gamma_v = \frac{2\cot\theta - 1}{3} \quad 0 \le \gamma_v \le 1.00 \tag{20}$$

The tensile forces in horizontal and vertical stirrups are as follows

$$F_{yh} = A_{sjh} f_{yj} \text{ and } F_{yv} = A_{sjv} f_{yj} \tag{21}$$

2.6 Hegger et al. (2003)

Hegger *et al.* (2003) proposed a complete empirical model based on regression analysis of experimental database. The model considers the parameters i.e., concrete strength (f_c), width of beam (b_b) and column (b_c), depth of beam (h_b) and column (h_c), column reinforcement ratio (ρ_c), joint shear reinforcement (A_{sjh}) and yield strength of joint shear reinforcement (f_{ysj}). From the regression analysis the following expressions are proposed.

$$V_{jh} = \left[2\xi\left(1.2 - 0.3 \,{}^{h_b}\!/_{h_c}\right)\left(1 + \frac{\rho_c - 0.5}{7.5}\right) b_j h_c \sqrt[3]{f_c}\right] + \left[\alpha A_{sj} f_{ysj}\right] \tag{22}$$

where, ξ represents the effect of beam bar anchorage detail; ξ=0.95 and 0.85, for L and U bars respectively. Author proposed different values for the coefficient α based on the anchorage type. For 90° and 180° bent bars the value of α is 0.7 and 0.6 for hairpins and 0.6 and 0.5 for closed stirrups respectively. The upper limit of joint shear strength (V_{max}) according to experimental results is found as

$$V_{max} = \gamma_1\gamma_2\gamma_3 0.25 f_c b_j h_c \le 2V_{ch} \tag{23}$$

in which the coefficients γ_1 is 1.00 for bent bars and 1.20 for headed bars. The other two coefficients γ_2 and γ_3 are based on the column axial stress (N_c/A_c), concrete strength (f_c) and aspect ratio (h_b/h_c) as stated below

$$\gamma_2 = 1.5 - 1.2\frac{(N_c/A_c)}{f_c} \le 1.0 \text{ and } \gamma_3 = 1.9 - 0.6\frac{h_b}{h_c} \le 1.0 \tag{24}$$

The limiting parameters are joint aspect ratio and concrete grade such as; $0.75 \le h_b/h_c \le 2$ and $20 \le f_c \le 100$ MPa respectively.

2.7 Kim et al. (2009)

Kim *et al.* (2009) developed a complete empirical model based on the Bayesian parameter estimation method for exterior beam column joint. Earlier the model proposed by Kim *et al.* (2007) was found inadequate for evaluating the shear strength in unreinforced beam-to-column joints. Now this model is considered suitable for both interior and exterior joints, both for unreinforced and reinforced types of joints. Author considers the parameters like concrete strength (f_c), width of beam (b_b) and column (b_c), depth of beam (h_b) and column (h_c), beam reinforcement (A_{sb}), eccentricity of beam (e_b) and joint shear reinforcement (A_{sjh}).

$$V_{jh} = 1.3\alpha_t \beta_t \eta_t \left(JI^{0.15}\right)\left(BI^{0.3}\right)\left(f_C^{0.75}\right)A_{sj} \tag{25}$$

where, α_t and β_t are parameters for describing the in-plane and out-of-plane geometry, respectively; η_t is a parameter to account for the influence of beam eccentricity; JI is the joint transverse reinforcement index depending mostly on the volumetric joint shear reinforcement ratio; BI is the beam reinforcement index.

$$\eta_t = \left(1 - \frac{e_b}{b_c}\right)^{0.67}; \; BI = \frac{\rho_b f_{yb}}{f_c} \; \text{ and } \; JI = \frac{\rho_j f_{yj}}{f_c} > 0.139 \tag{26}$$

Beam reinforcement ratio (ρ_b) and joint transverse reinforcement ratio (ρ_j) needed for evaluating the beam and transverse reinforcement index, respectively, can be evaluated as follows

$$\rho_b = \frac{A_{sbtop} + A_{sbbot}}{b_b h_b} \; \text{ and } \; \rho_j = \frac{A_{sj} h_c}{b_c h_c \left(h_b - 2h_b^{'}\right)} \tag{27}$$

2.8 Wang et al. (2011)

Wang *et al.* (2011) proposed a theoretical model based on the assumption that the material is under plane stress state and the effect of tensile straining in the transverse direction on the compressive strength of the idealized material is accounted by using the Kupfer-Gerstle biaxial tension-compression failure envelope. The model considers the effect of tensile reinforcement in the joint but neglects effect parameters like the beam and column reinforcement and column axial load ratio.

$$V_{jhmax} = \beta \frac{1.0 - \left(sin^2 \dfrac{\alpha}{f_{tn}} - 0.8 cos^2 \alpha / f_c\right)\sigma_y}{\left(1/f_{tn} + 0.8/f_c\right) sin 2\alpha} b_j h_c \tag{28}$$

The nominal tensile strength of concrete (f_{tn}) is

$$f_{tn} = f_{ct} + \frac{A_{sh} f_{yh} cos\alpha}{b_j h_c / sin\alpha} + \frac{A_{sv} f_{yv} sin\alpha}{b_j h_c / sin\alpha} \tag{29}$$

The contribution of concrete tensile strength (f_{ct}) is, $f_{ct} = 0.556\sqrt{f_c}$. A_{sh} and A_{sv} is the total area of the horizontal and vertical shear reinforcement of the joint. The angle of inclination α is equal to the ratio of column to beam depth (h_c/h_b) and the reduction factor β according to the confinement action of beams into the joint is taken as 0.8 for exterior joint.

2.9 Sharma et al. (2012)

Sharma *et al.* (2012) considers the principal tensile stress approach as failure criteria for development of new analytical model only for exterior beam column joints. In this model only concrete compressive (f_c) and concrete tensile strength (p_t), width of beam (b_b) and column (b_c), and column axial stress (N_c/A_c) are considered. The model neglects the effect of longitudinal reinforcement from beam and column and the shear reinforcement inside the joint.

$$V_{jh} = \frac{(\sigma - \sigma_a)}{\alpha} b_c h_c \tag{30}$$

The coefficient α is the aspect ratio of joint (h_b/h_c) and σ_a is vertical joint shear stress (N_c/A_c) due to column axial load (N_c). The axial stress (σ) is given by the following equation

$$\sigma = \frac{2\sigma_a + \alpha^2 p_t + \alpha\sqrt{\alpha^2 p_t^2 + 4p_t(\sigma_a + p_t)}}{2} \tag{31}$$

and the principal tensile stress of concrete (p_t) is

$$p_t = \frac{\sigma}{2} - \frac{\sigma}{2}\sqrt{1 + \frac{4(\sigma - \sigma_a)^2}{\alpha^2 \sigma^2}} \tag{32}$$

As the model neglects the effect of reinforcement, predicts the lower joint shear strength as compared to other models. The proposed model requires some assumptions and iterative calculations.

2.10 Unal and Burak (2013)

Unal and Burak (2013) developed a parametric equation based on the correlation of different parameters with the joint shear stress (v_j) considering further emphasis on the geometry of joint. The model considers the effect of concrete strength (f_c), width of beam (b_b) and column (b_c), depth of beam (h_b) and column (h_c), column axial stress (N_c/A_c), joint eccentricity (e) and joint volumetric ratio for one layer of transverse reinforcement (ρ_j). The contributions of beam and column longitudinal reinforcement are neglected here.

$$v_j(MPa) = JT(f_c f_y)^{\frac{1}{6}} \rho_j \sqrt{\frac{1}{1 + e/b_c}} \sqrt{\frac{b_c}{h_c}} \left(1 + \frac{N_c}{A_c f_c}\right) WBCISI \tag{33}$$

Where, *JT* is the joint types taken equal to 1.00 for exterior joint and *CI* is column index depending on the column aspect ratio taken equal to $\sqrt{b_c/h_c}$ when b_c/h_c<1.00 and 1.00 for other cases. SI is slab index taken equal to 1.00 when slab is not present. When wide beam is present in loading direction the effect is considered equal to the expression as, $WB = \left(1 - \frac{h_b}{b_b}\frac{b_j}{b_b}\right)$ otherwise taken equal to 1.00.

2.11 Tran et al. (2014)

Tran *et al.* (2014) considers the contribution of bond strength in the formulation of empirical model. The model is based on the regression analysis based on collected past experimental database. The four parameters are considered such as concrete strength (f_c), width of beam (b_b) and column (b_c), depth of beam (h_b) and column (h_c), column axial stress (N_c/A_c), beam reinforcement (A_{sb}) and joint horizontal and vertical shear reinforcement (A_{sjh} and A_{sjv})

$$V_{jh} = \left(\gamma_1 + \frac{N_c}{b_c h_c f_c} + 1.2\chi_b \right) A_{jh} f_c^{0.5} + \gamma_2 \left(A_{sjh} f_{yjh} + A_{sjv} f_{yjv} \right) \tag{34}$$

where, the coefficients γ_1 =0.34 and γ_2=0.22 for exterior joints. And the additional new parameter of bond strength (χ_b) is defined as

$$\chi_b = \frac{n_b d_b h_c}{b_b h_b} \leq 0.4 \tag{35}$$

In this equation the effect of parameter of column reinforcement is not considered. The beam reinforcement is considered in terms of bond strength (χ_b); n_b is the number of longitudinal beam reinforcement inside the joint core.

2.12 Pauletta et al. (2015)

Pauletta *et al.* (2015) proposed a strut and tie model to determine nominal design shear strength (V_{jh}) of exterior RC beam column joint. The contribution from the horizontal hoops and intermediate column reinforcing bars within the joint region are also considered. The model considers the effect of parameters like concrete grade (f_c), width of beam (b_b) and column (b_c), depths of beam (h_b) and column (h_c), column axial stress (N_c/A_c), horizontal (A_{jh}) and vertical (A_{jv}) joint shear reinforcement as

$$V_{jh} = 0.45 \left[\frac{\delta f_c b_j c \cos\theta}{\alpha} + 0.79 A_{sh} f_{ysh} + 0.52 \frac{A_{sv} f_{ysv}}{\tan\theta} \right] \tag{36}$$

where, A_{sh}, A_{sv} and f_{ysh}, f_{ysv} are areas and yield strengths of horizontal and vertical joint shear reinforcement respectively; θ is the angle of inclination of the strut, $\theta = tan^{-1}\left(h_b''/h_c''\right)$, h_b'' is distance between top and bottom beam longitudinal bars and h_c'' is the distance measured from the centroid of bar extension at the free end of the 90° hooked bar to the centroid of longitudinal column reinforcement in the opposite side; c is the depth of the compression zone in the column. In this δ is the nondimentional interpolating parameter given by the expression

$$\delta = \left[0.74 \left(\frac{f_c}{105} \right)^3 - 1.28 \left(\frac{f_c}{105} \right)^2 + 0.22 \left(\frac{f_c}{105} \right) + 0.87 \right] \tag{37}$$

3. Limitations of analytical models

Eq. (1) proposed by Sarsam and Phillips (1985) considers joint shear reinforcement as to provide additional strength to joint, however, the effect of anchorage and the amount of beam longitudinal reinforcement have been neglected. Eqs. (3)-(7) proposed by Paulay and Priestley (1992) neglects the effect of column longitudinal reinforcement, moreover, the effect of concrete grade has not been considered explicitly, however, its contribution is considered in the form of depth of flexural compression zone in the column (column axial load ratio ($N_c/A_c f_c$)). The effect of beam and column longitudinal reinforcement and the column axial load ratio is neglected in the Eq. (8) proposed by Vollum and Newman (1999). But the intensive focus has been given on the effect of anchorage of beam longitudinal reinforcement inside the joint. Bakir and Boduroğlu (2002) have proposed Eq. (12) for joint shear strength, focused primarily on the amount and anchorage of beam longitudinal bars but neglected the effect of column reinforcement and column axial load ratio. The strut and tie model Eqs. (13)-(21) proposed by Hwang and Lee (2002) predominantly depends on the compressive strength of concrete, aspect ratio of joint and shear reinforcement within the joint. But effect of beam longitudinal reinforcement is neglected. The Eq. (22) of Hegger *et al.* (2003) based on regression analysis have considered the column longitudinal reinforcement but neglected the effect of beam longitudinal reinforcement. Similarly Eq. (25) of Kim *et al.* (2009) ignored the contribution of column longitudinal reinforcement. Moreover, this equation is unable to predict joint shear strength for the joint without shear reinforcement. Wang *et al.* (2011) neglects the influence of beam and column longitudinal reinforcement. The model by Sharma *et al.* (2012) considers mainly the concrete compressive strength and neglects the effect of joint shear reinforcement. The effect of beam and column reinforcement is neglected in Eq. (33) proposed by Unal and Burak (2013). Tran *et al.* (2014) predominantly considers the bond strength of beam reinforcement, but neglected the column reinforcement. The strut and tie model of Pauletta *et al.* (2015) neglects the effect of beam longitudinal reinforcement; however, it considers the effect of vertical column intermediate longitudinal reinforcement.

4. Parametric analysis

In present study, twelve models for computing joint shear strength have been considered. In the previous section it has been observed that the selection of governing parameters for predicting the joint shear strength in the considered models varies. The compressive strength of concrete has been the only governing parameter which is common in all the models.

In this study ten parameters viz. width and depth of beam and column, grade of concrete and steel, longitudinal reinforcement of column and beam, shear reinforcement of joint and, column axial load ratio have been considered. In RC frame buildings these parameters may vary to a great extent, however, in the present study the range of parameters as shown in the Table 1 has been selected on the basis of variation observed in normal low to mid-rise RC frame buildings. Furthermore, the selected range is also justified from the available experimental results (Appendix

Table 1 Database considered for the parametric analysis

Set No.	Beam		Column		Concrete grade	Steel grade	Reinforcement			Column axial load ratio
	Width	Depth	Width	Depth			Column	Beam	Joint	
	b_b	h_b	b_c	h_c	f_c	f_y	ρ_c	ρ_b	ρ_j	$N_c/(A_c f_c)$
	mm	mm	mm	mm	MPa	MPa	%	%	%	MPa
1.	230	300	230	300	20	250	0.80	0.50	0.30	0.00
2.	**300**	**400**	**300**	**400**	**40**	**415**	**1.50**	**1.50**	**0.70**	**0.20**
3.	400	500	400	500	60	500	3.00	2.00	1.00	0.50
4.	500	600	500	600	80	550	4.00	3.00	1.50	0.75
5.	600	700	600	700	100	600	6.00	4.00	2.00	1.00

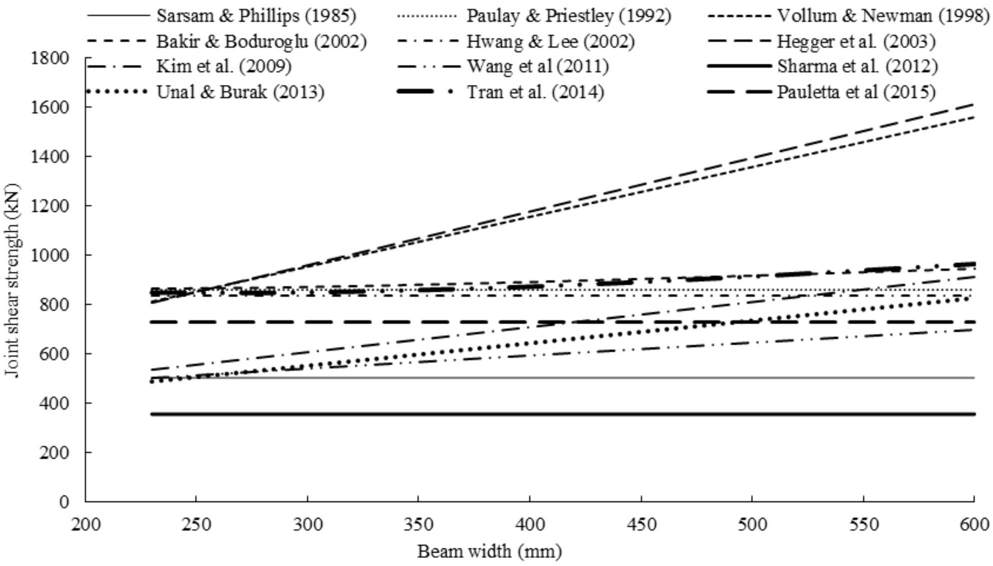


Fig. 1 Inflence of beam width on joint shear strength

1). For the parametric study on twelve joint shear strength models, the set no. 2 (Table 1) has been considered as benchmark. Effect of each parameter has been studied by varying that particular parameter in the set no. 2. Effect of variation of each parameter on the considered twelve models has been shown in following sections.

4.1 Beam width (bb)

Fig. 1 shows the influence of beam width on joint shear strength predicted by twelve different models. Model proposed by Sharma *et al.* (2012) predicts the lowest joint shear strength while model by Hagger *et al.* 2003 predicts the highest. It can be observed from the figure that for the same parameters there is significant variation in predicted joint shear strength i.e. for smaller beam width the variation is about 2.5 times which increases to 4.5 times for larger beam width.

Out of the twelve models considered, the five models viz. Sarsam and Phillips (1985), Paulay and Priestley (1992), Hwang and Lee (2002), Sharma *et al.* (2012), Pauletta *et al.* (2015) are insensitive to beam width. While the other seven models viz. Vollum and Newman (1998), Bakir and Boduroğlu (2002), Hegger *et al.* (2003), Kim *et al.* (2009), Wang *et al.* (2011), Unal and Burak (2013), Tran *et al.* (2014) shows an incremental effect of beam width on joint strength. Out of the seven models the Vollum and Newman (1998), Hegger *et al.* (2003) model shows strong effect of beam width with the linear increamental effect on joint shear strength (approximetely 200 kN per 100 mm increase in width of joint). Similarly Kim *et al.* (2009), Unal and Burak (2013) model indicates a moderate effect of beam width on joint shear strength (linear increase of 100 kN per 100 mm width of joint). While Bakir and Boduroğlu (2002), Tran *et al.* (2014), Wang *et al.* (2011)'s model shows weak effect of beam width on joint shrength. The Wang *et al.* (2011) model is showing increase of only 5 kN per 100 mm increase in the beam width. The remaining Bakir and Boduroğlu (2002), Tran *et al.* (2014) model shows nonlinear increase with beam width, however, the effect is very small.

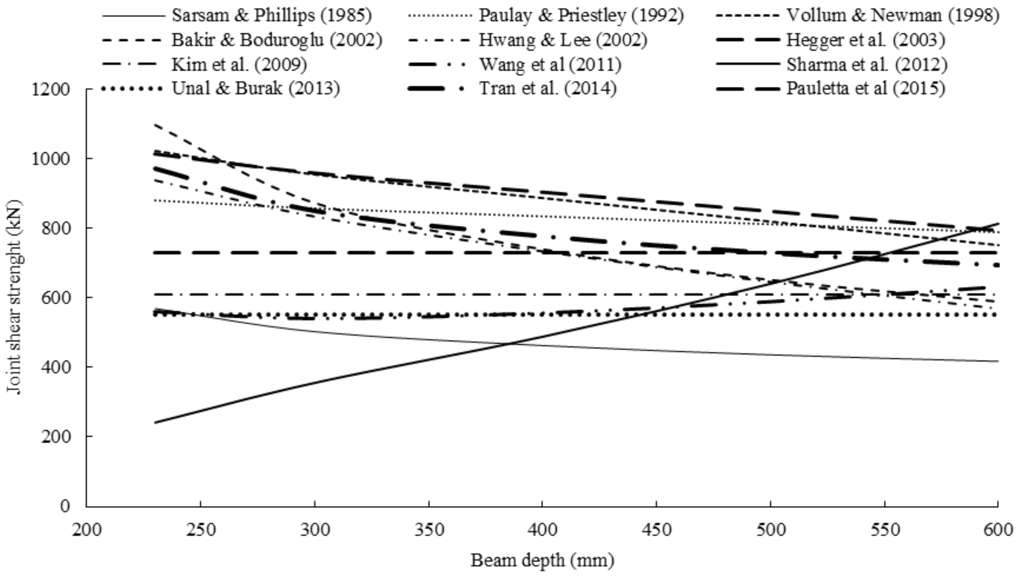


Fig. 2 Inflence of beam depth on joint shear strength

4.2 Beam depth (h_b)

Prediction of joint shear strength by considered models for different beam depth does not show any obvious trend. Fig. 2 shows the significant variation in predicted joint shear strength i.e., for smaller beam depth the variation is about 4.5 times which decreases to 2.0 times for larger beam depth. From the Fig. 2 all the three pattern i.e., increase, no effect and decrease in joint shear strength with increasing beam depth have been observed. Moreover, for smaller beam depth (i.e., 230 mm) the variation in predicted joint shear strength amongst the considered models is significant which reduces with increase in beam depth .

Out of the twelve models, three models (Kim *et al.* 2009, Unal and Burak 2013, Pauletta *et al.* 2015) neglects the effect of beam depth. While the seven models (Sarsam and Phillips 1985, Paulay and Priestley 1992, Vollum and Newman 1998, Hwang and Lee 2002, Bakir and Boduroğlu 2002, Hegger *et al.* 2003, Tran *et al.* 2014) shows the reduction of joint shear strength with increase in beam depth. Amongst seven models, the three models by Paulay and Priestley (1992), Vollum and Newman (1998), Hegger *et al.* (2003) shows linear decrease of joint shear strength of order of 23 kN, 68 kN and 55 kN per 100 mm increase of beam depth respectively. The models of Sarsam and Phillips (1985), Bakir and Boduroğlu (2002), Hwang and Lee (2002), Tran *et al.* (2014) shows nonlinear decrease of joint shear strength in which rate of decrease is high for the lower depth of the beam. In contrast, the two modes i.e., Wang *et al.* (2011), Sharma *et al.* (2012) shows the nonlinear increase in joint shear strength. But Sharma *et al.*(2012) model shows very steep increase in joint shear strength with increase in beam width.

4.3 Column width (b_c)

The considered twelve models does not show any clear pattern on joint shear strength with variation of column width. Fig. 3 shows the indistinguishable variation in predicted joint shear

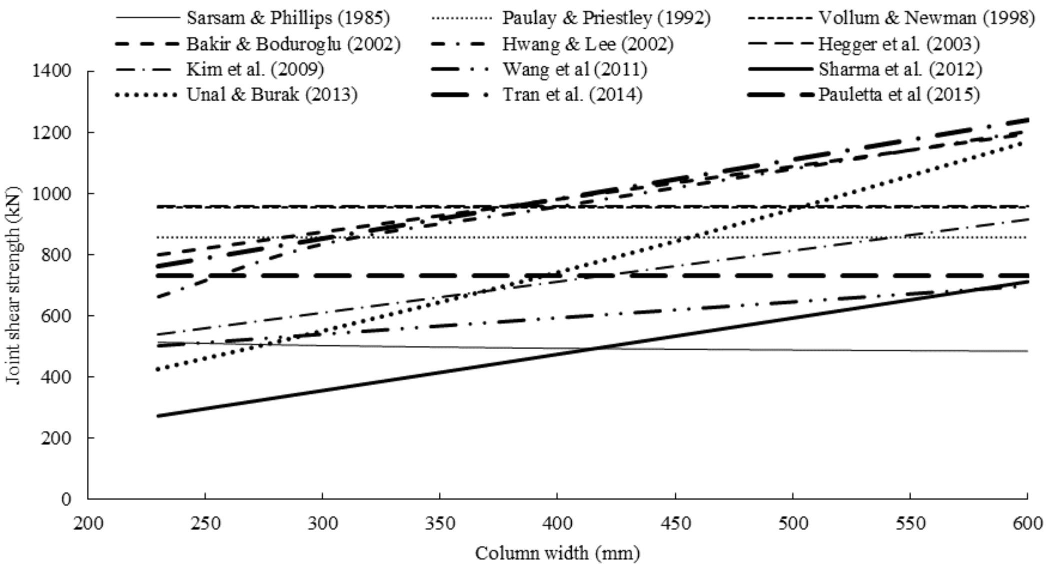


Fig. 3 Inflence of column width on joint shear strength

strength i.e., for smaller column width the variation is about 2.5 times which remains same for greater column width.

Four models i.e., Paulay and Priestley (1992), Vollum and Newman (1998), Hegger *et al.* (2003) and Pauletta *et al.* (2015) are insensitive to column width for predicting the joint shear strength. Model of Sarsam and Phillips (1985) shows decrease in joint shear strength with increase in column width. All remaining models are showing increase in joint shear strength with increase in column width. Significant linear increase in joint shear strength has been observed in Hwang and Lee (2002), Unal and Burak (2013), and Tran *et al.* (2014) model which gives 200 kN per 100 mm increase in width. Whereas the models proposed by Bakir and Boduroğlu (2002), Kim *et al.* (2009) and Sharma *et al.* (2012) gives the moderate rate of increase which is of the order of 100 kN per 100 mm increase in column width. The Wang *et al.* (2011) model gives weak influence of column width on joint shear strength.

4.4 Column depth (h_c)

Except two models (Paulay and Priestley 1992, Sharma *et al*. 2011) other ten models shows increase in joint shear strength with increase in column depth. From Fig. 4, the significant variation in rate of increase in joint shear strength is observed i.e., for smaller column depth the variation is about 2.5 times which goes up to 6.5 times for larger column depth.

Models by Tran *et al.* (2014), Bakir and (2002), Hwang and Lee (2002), Hagger *et al.* (2003), Pauletta *et al.* (2015) shows significant increase while Sarsam and Phillips (1985), Kim *et al.* (2003), Wang *et al.* (2011), Unal and Burak (2013) shows moderate increase in joint shear strength with increase in column depth (Fig. 4). Paulay and Priestley (1992) model does not consider the effect of column depth, hence it is insensetive to this parameter. In case of Sharma *et al.*(2012) model there is mild decrease in joint shear strength with increase in column depth. The models given by Sarsam and Phillips (1985),Vollum and Newman (1998), Hegger *et al.* (2003) and Kim *et*

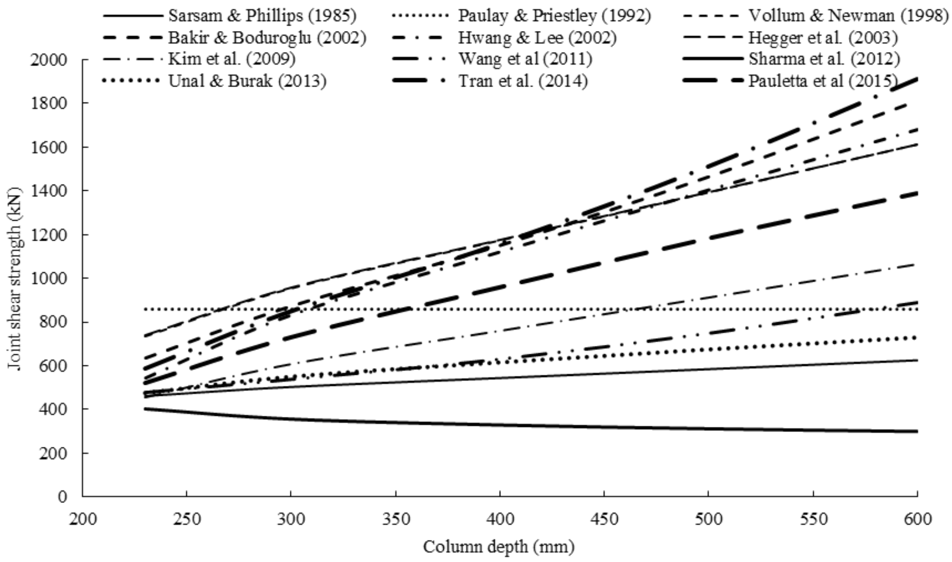


Fig. 4 Inflence of column depth on joint shear strength

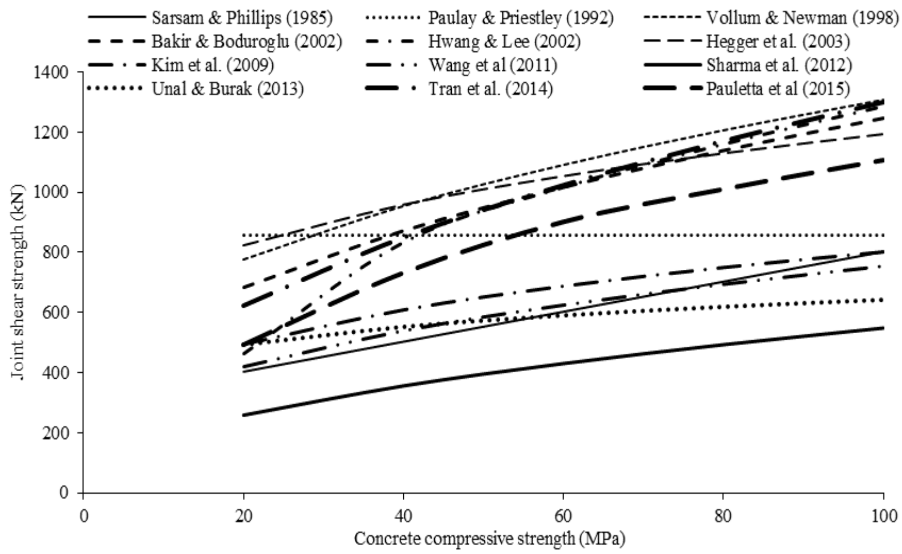


Fig. 5 Inflence of concrete grade on joint shear strength

al. (2009) are showing the linear increment, whereas the Bakir and Boduroğlu (2002), Hwang and Lee (2002), Wang *et al.* (2011), Unal and Burak (2013), Tran *et al.* (2014) and Pauletta *et al.* (2015) model are showing the nonlinear incremental effect of column depth on joint shear strength.

4.5 Concrete grade (f_c)

Being the primary component of shear transfer mechanism in all the considered models, the increase in grade of concrete increases the strength of joint as shown in Fig. 5, except in the model by Paulay and Priestley (1992). The model by Paulay and Priestley (1992), does not considers the direct contribution of concrete strength, however, the effect of concrete strength is considered indirectly in the formulation of depth of flexural compression in column through the column axial load ratio (N_c/A_cf_c).

From Fig. 5 the constant variation of 3% is observed in joint shear strength for lesser and greater concrete compressive strength. The Sarsam and Phillips (1985) model indicates steep linear increase in joint shear strength with increase in concrete strength. Except Sarsam and Phillip's model all other models are showing the nonlinear increase in joint shear strength. The Sharma *et al.* (2012), Unal and Burak (2013) model are showing weak influence of concrete grade on joint shear strength. In general, the rate of increase of joint shear strength in most of the models with the concrete grade is high upto 60 MPa.

4.6 Tensile yield strength of steel reinforcement (f_y)

All the analytical models (excluding model of Sharma *et al.* 2012) consider the effect of yield strength of steel reinforcement into the formulation. It can be observed from the figure that for the same parameters there is significant linear variation in predicted joint shear strength i.e. for yield strength the variation is about 2.5 times which goes up to 5 times for larger yield strength. Similar to concrete grade, the increase in steel yield strength improves the joint shear strength, however, its influence is less than concrete grade (Fig. 6). At initial set of 415 kN the joint shear strength range is in between 356 MPa to 838 MPa while at the last set the range expanded to 356 MPa to 1908 MPa.

Paulay and Priestley (1992) and Bakir and Badiroglu (2002) gives increase of approximately 100 kN and 300 kN per 50 MPa increase in yield strength of steel reinforcement. The three models Hwang and Lee (2002), Unal and Burak (2013), Tran *et al.* (2014) shows increase of

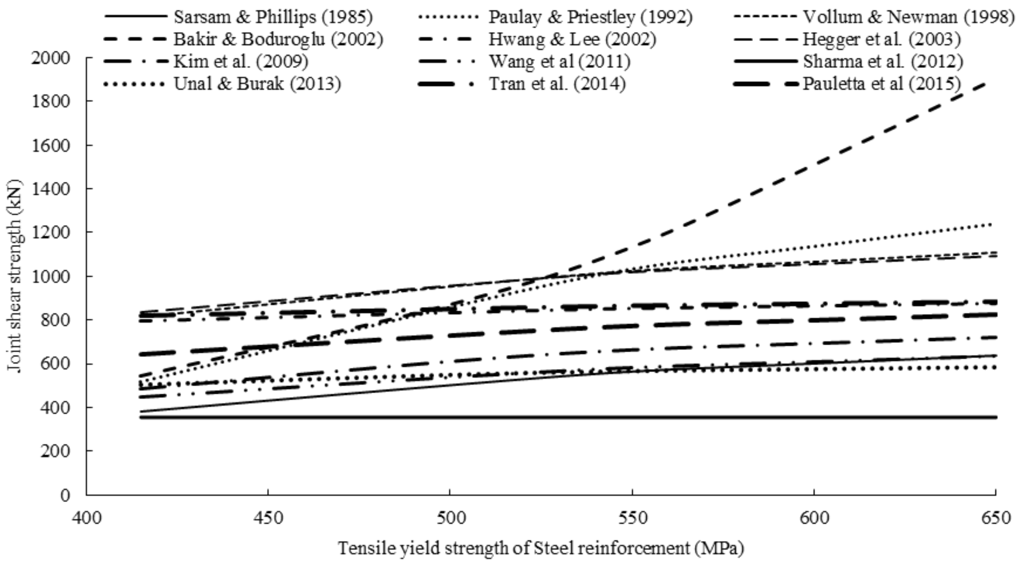


Fig. 6 Inflence of steel reinforcement grade on joint shear strength

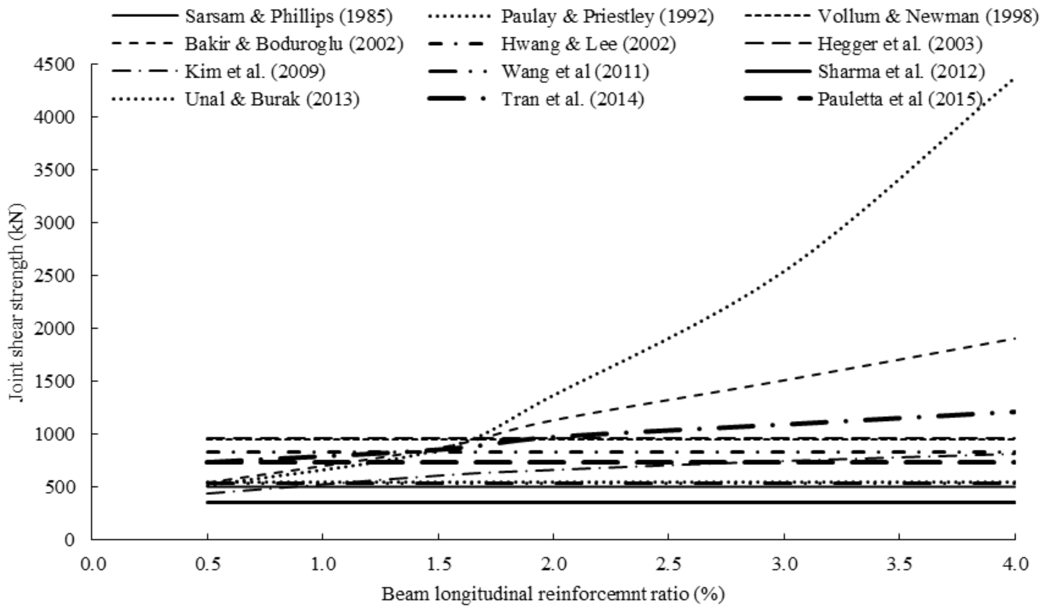


Fig. 7 Inflence of beam longitudinal reinforcment on joint shear strength

approximately 10 kN, whereas other six models Sarsam and Phillips (1985), Vollum and Newman (1998), Hegger *et al.* (2003), Kim *et al.* (2009), Wang *et al.* (2011), Pauletta *et al.* (2015) gives 40 % increase with increase in 50 MPa yield strength of steel reinforcement.

4.7 Beam longitudinal reinforcement ratio (ρ_b)

Fig. 7 shows that for smaller amount of beam reinforcement (0.5%) the variation is about 1.5 times which increases upto 12 times for higher amount (4%) of beam reinforcement. Only four models (Paulay and Priestley 1992, Bakir and Badiroglu 2002, Kim *et al.* 2003, Tran *et al.* 2014) have considered the effect of beam longitudinal reinforcement on joint shear strength (Fig. 7). The analysis based on four models shows the increase in joint shear strength with increase in beam longitudinal reinforcement. Model by Tran *et al.* (2014) shows linear increase at the rate of 120 kN per 1% increase in beam reinforcement. Also the rate of increase of Paulay and Priestley (1992) and Bakir and Boduroğlu (2002) model is higher than that of Kim *et al.* (2009) model as shown in the Fig. 7.

The amount and detailing of beam longitudinal reinforcement inside the joint affects the shear capacity of joint (Paulay and Priestley 1992). Bending moment in the beam develops tension force at the joint face which is transferred by means of bond with concrete. The embedment length of beam longitudinal bars is effective in preventing the slip of bars under the cyclic loadings. In addition to bond strength, the type of anchorage of beam longitudinal reinforcement also affects the force transfer mechanism (Vollum and Newman 1999, Bakir and Boduroğlu 2002, Hegger *et al.* 2003, Tran *et al.* 2014). In older construction practice it is observed that the beam bottom longitudinal bars are kept straight inside the joint (without bend) which leads to ‘bond-slip failure’. This insufficient embedment length leads to premature failure of joint. The crack at the joint interface develops due to premature failure of joint attributed to bond slip.

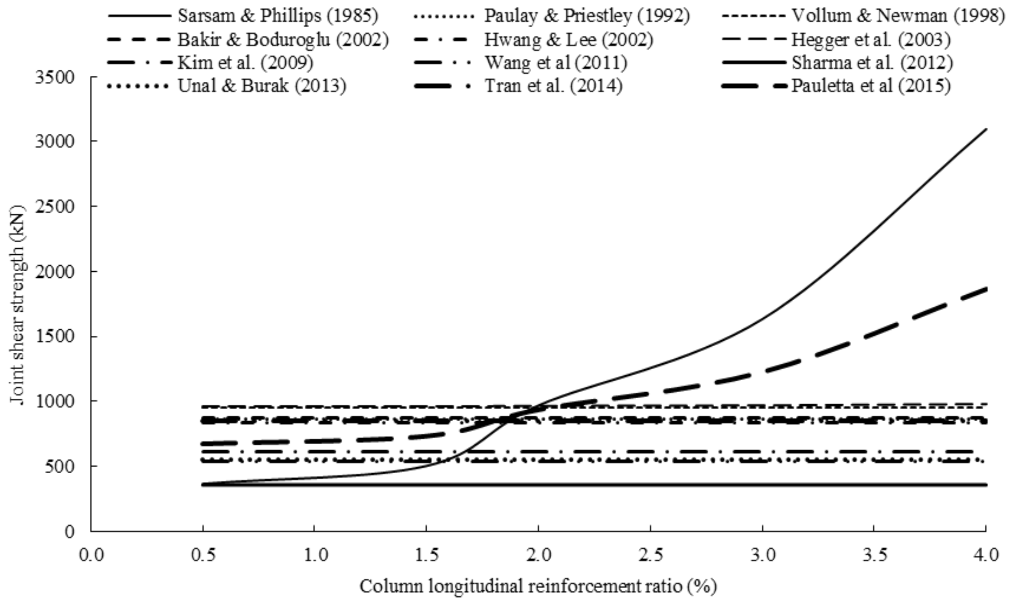


Fig. 8 Inflence of column longitudinal reinforcemnt on joint shear strength

4.8 Column longitudinal reinforcement ratio (ρ_c)

The effect of variation of column longitudinal reinforcement on joint shear strength is shown in Fig. 8. The variation in joint shear strength is 2.0 times for lesser column reinforcement (0.5%) which increases upto 10 times for higher column reinforcement (4.0%). Except the three models by Sarsam and Phillips (1985), Hegger *et al.* (2003) and Pauletta *et al.* (2015), all other models have not considered the effect of column longitudinal reinforcement. All the nine models (viz. Paulay and Priestley 1992, Vollum and Newman 1999, Bakir and Boduroğlu 2002, Hwang and Lee 2002, Kim *et al.* 2009, Wang *et al.* 2011, Sharma *et al.* 2012, Unal and Burak 2013, Tran *et al.* 2014) shows constant trend along with the increase in column reinforcement.

Fig. 8 shows the significant increase in joint shear strength after increasing the amount of column reinforcement from 1.5-3.0%. The joint shear strength is found impulsive increase of almost 50% than that observed upto 3%. Hegger *et al.* (2003) model shows slight increase in joint shear strength of approximately 5% with increase of 1% in column reinforcement.

4.9 Joint shear reinforcment (ρ_j)

Observations from Fig. 9 shows that the variation in joint shear strength is 2 times for smaller joint shear renforcement which increases significantly upto 14 times with the increase in joint shear reinforcement from 0.5-4.0%.

The five models viz., Sarsam and Phillips (1985), Paulay and Priestley (1992), Vollum and Newman (1998), Bakir and Boduroğlu (2002), Hegger *et al.* (2003) gives constant 40% increase while the remaining three models Wang *et al.* (2011), Unal and Burak (2013), Pauletta *et al.* (2015) gives lesser rate of increase of 20 % with increase of each 1% shear reinforcement. The three models by Kim *et al.* (2009), Hwang and Lee (2002), Tran *et al.* (2014) shows moderate rate

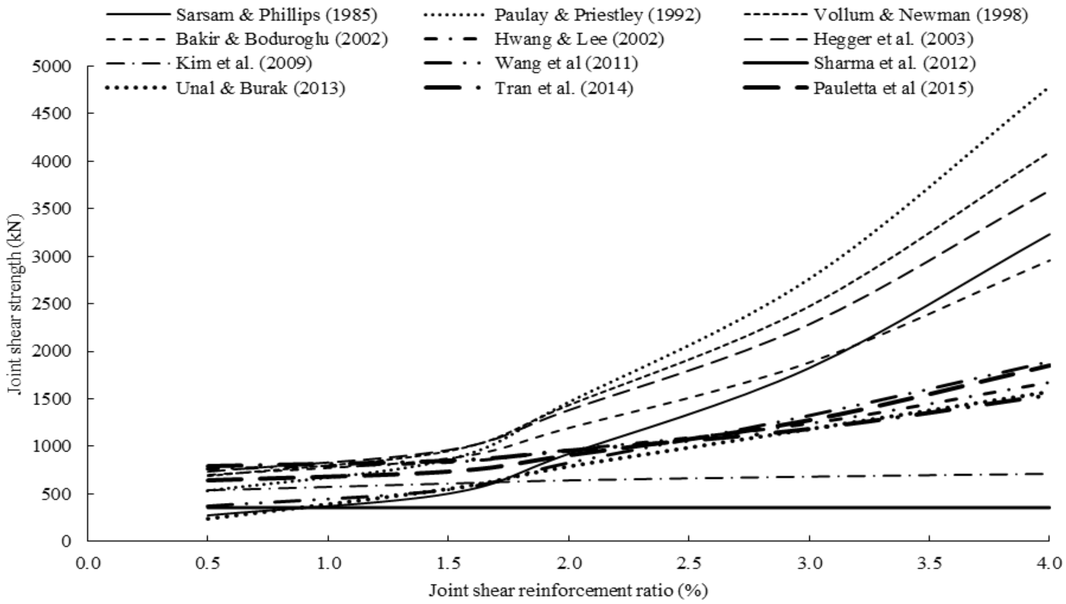


Fig. 9 Inflence of joint shear reinforcment on joint shear strength

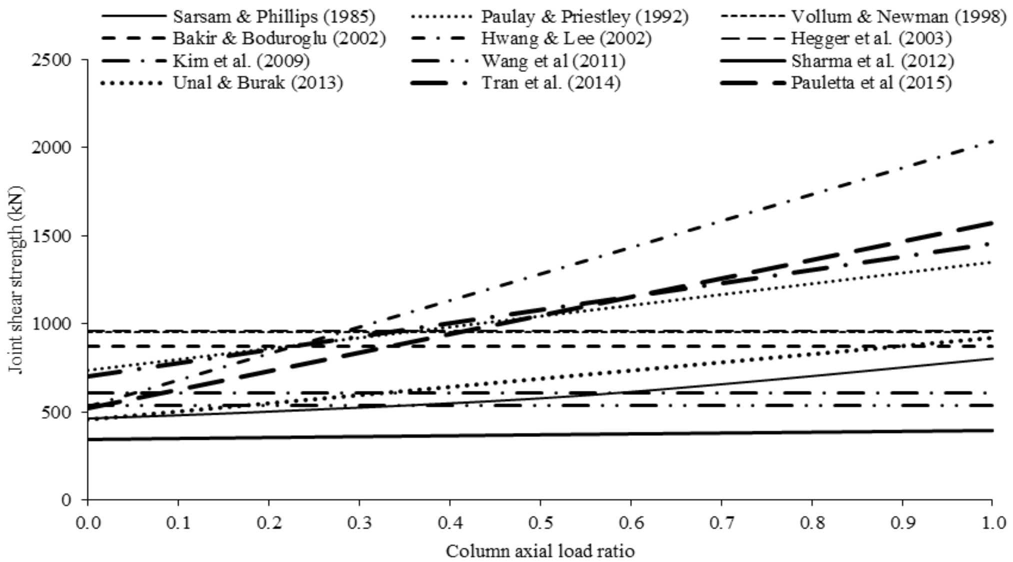


Fig. 10 Inflence of column axial load ratio on joint shear strength

of increase in joint shear strength of 6%, 17% and 15% per 1% increase of joint shear reinforcement as compared with previous mentioned models. The model by Sharma *et al.* (2012) model does not considers the effect of joint reinforcement.

The general pattern of graph shows that for joint shear reinforcemnt upto 1.5% the rate of increase is very less, in between 1.5%-2.5% the joint shear strength increases 2-3 times, while, from 3%-4% the increse rate becomes 3-5 times higher.

4.10 Column axial load ratio (N_c/A_cf_c)

The rate of variation of all the models for column load ratio from 0 to 1.0 is shown in the Fig. 10. The variation in joint shear strength is approximately 3 times for smaller column axial load ratio which increases to 6 times for higher column axial load ratio.

Most of the models are considering positive effect of column axial load on joint shear strength. The models proposed by Vollum and Newman (1999), Bakir and Boduroğlu (2002), Hegger *et al.* (2003), Kim *et al.* (2009), Wang *et al.* (2011) does not consider the column axial load ratio. While the models by Sarsam and Phillip (1985), Paulay and Priestley (1992), Sharma *et al.* (2012), Unal and Burak (2013) show very lower rate of increase in joint shear strength. The Hwang and Lee (2002), Tran *et al.* (2014), Pauletta *et al.* (2015) model are showing constant rate of increase in joint strength.

5. Assessment of analytical models using experimental database

The selected joint shear strength models have also been evaluated with a database of 200 experimental results of exterior RC beam column joints compiled from available literature. All the experimental results considered in this paper were tested under either monotonic or cyclic loadings. Test specimens having wide beam, eccentric joints and the presence of slab/transverse beam were not included in the database. The geometric and material properties of the test specimens along with externally applied forces and the corresponding joint shear strength have been shown in Appendix I. Furthermore, it has also been tried to categorize the experimental database on similar lines of analytical study i.e., experimental results with identical parameters except the parameter which is varying, but this kind of categorization was not possible due significant variability of parameters. However, it has been observed that many research groups have conducted experiments on specimens with similar joint details and it is possible to identify the pairs of specimens having one parameter variable and others constant. These pairs were further analyzed to observe the effect of variation of a single parameter.

Based on the experimental database two types of study have been conducted i.e. effect of variation of a parameter on joint shear strength and statistical compatibility of the considered twelve models with experimental results.

5.1 Parametric analysis of experimental database

Out of the 200 experimental database (Appendix I), the pairs of specimens having single variable parameter has been identified. It has been observed that these pairs can be grouped for five parameters only, viz. concrete grade, beam longitudinal reinforcement, column longitudinal reinforcement, joint shear reinforcement, and column axial load ratio as shown in Appendix II. The effect of variation of each of the aforementioned parameter on joint shear strength is described below;

a) Concrete grade

Eighteen pairs of specimens with the minimum variation of 4 MPa in concrete grade have been identified and tabulated in Table 1 (a) and (b) of Appendix II. Further, these eighteen pairs have been subcategorized into two groups of concrete grades i.e. less than 60 MPa (Table 1(a) of

Appendix II) and more than 60 MPa (Table 1(b) of Appendix II) keeping in view the brittle behavior of high strength concrete under cyclic loading (Paulay and Priestley 1992). In the first group (concrete grade less than 60 MPa) consisting of thirteen pairs, the minimum difference of concrete grade in a pair is 8% and maximum is 43%. It can be observed from the Table 1(a) of Appendix-II that with increase in concrete grades, seven pairs out of the thirteen pairs are showing increase in joint shear strength (F2-X6 of Tsonos *et al.* 1992, #6-#2 of Clyde *et al.* 2000, C6LN5-C4ALN5 of Hamil 2000, Q1-R1 of Murty *et al.* 2003, LVP4-LVP2 of Alva *et al.* 2007, and BSLV2-BSLH2 of Wong and Kuang 2008), while, the remaining six pairs are showing either decreasing effect or insensitive to joint shear strength (C4AL-C6L of Scott 1996, C4ALN0-C6LN0, C4ALN1-C6LN1 and C4ALN3-C6LN3 of Hamil 2000, O7-O6 of Hakuto *et al.* 2000, and P2-S2 of Murty *et al.* 2003). Moreover, it is also important to note that the test results of specimens F2-X6 by Tsonos *et al.* (1992) shows 47% increase in joint shear strength with the increase of 12% in concrete grade (29 MPa-33 MPa), while in case of specimens P2-S2 of Murty *et al.* (2003) for 13% increase in concrete grade (26 MPa-30 MPa), the joint shear strength decreases by 2%. Similarly, from the Table 1(b) (concrete grade more than 60 MPa), the joint shear strength variation is not consistent with concrete grade. For both, low and high grade of concrete, it is difficult to obtain any conclusive effect of increase in grade of concrete on joint shear strength.

Moreover, out of the twelve analytical joint shear strength models considered in the previous section, eleven models predict increase in joint shear strength with increase in concrete grade (Fig. 5), whereas, the model of Paulay and Priestley's (1992) shows no direct effect of concrete grade, however, this model considers the effect of concrete grade in axial load ratio.

b) Beam longitudinal reinforcement

Three pairs of specimens with variation in beam longitudinal reinforcement have been identified and shown in Table 2 of Appendix-II. All the three pairs (i.e., M1-M2 of Tsonos 1999, JC1-JC2 of Chun and Kim 2004, and BSL600-BSL450 of Wong and Kuang 2008) shows increase in joint shear strength with increase in beam reinforcement percentage. The joint shear strength of specimens BSL600-BSL450 of Wong and Kuang (2008) increased by 10% with increase in beam reinforcement from 1.21% to 1.61%, whereas specimens JC1-JC2 of Chun and Kim (2004) in which the beam reinforcement is increased from 1.52% to 3.04%, the joint shear strength increased by 53%.

As indicated in section 4.7, only four models (i.e., Paulay and Priestley 1992, Bakir and Badiroglu 2002, Kim *et al.* 2003, Tran *et al.* 2014) considered the effect of beam longitudinal reinforcement on joint shear strength (Fig. 7). Out of these four models, two models (Paulay and Priestley 1992, Bakir and Badiroglu 2002) predicts steep increase in joint shear strength with increase in beam longitudinal reinfeocement beyond 1.5%, which is also evident from experimental results of specimens M1-M2 of Tsonos (1999) and JC1-JC2 of Chun and Kim (2004). However, based on these few test results, apparently these two analytical models capture the effect of beam reinforcement properly.

c) Column longitudinal reinforcement

Only three pairs of experimental specimens have been found to consider the effect of column reinforcement as tabulated in Table 3 of Appendix II. The test pairs X6-S'6 of Tsonos (1992) shows no effect on joint shear strength even with 50% increase in column reinforcement, while, in case of test pair 4C-4F of Parker and Bullman (1997), 7% increase of joint shear strength is observed with 75% increase in column reinforcement. On contrary, the test pairs 1T55-3T3 of Hwang *et al*. (2005) in which the column reinforcement is increased by 13% shows reduction of 17% in joint shear strength.

As discussed in section 4.8 only three models (Sarsam and Phillips 1985, Hegger *et al.* 2003, Pauletta *et al.* 2015) shows increase in joint shear strength with increase in column reinforcement. Moreover, from the comparison of test pairs and analytical models no conclusive observations have been obtained.

d) Joint shear reinforcement

The total 31 pairs of specimens have been identified which considers the effect of variation in joint shear reinforcement as shown in Table 4 of Appendix II. The variation of joint shear reinforcement ranges from zero to 2.86%. Out of 31 pairs, seven pairs show notable increase (around 20%) in joint shear strength (C4ALN0-C4ALN1, C6LH3-C6LH5 and C6LN3-C4ALN5 of Hamil 2000, Q1-P2 of Murty *et al.* 2003, BSL450-BSLH1 of Wong and Kuang 2008, BS450-H1T10 and BS600-H2T8 of Kaung and Wong 2011). Seventeen pairs show marginal increase (upto 13%), while, seven pairs show slight reduction (upto 8%) in joint shear strength. From most of the observations it is apparent that increase in joint reinforcement increases the joint shear strength. However, from the identified pairs of specimens, it is difficult to comment on rate of increase in joint shear strength with percentage increase in joint reinforcement. For example the pair X1-S1 and X2-S2 of Tsonos *et al.* (1992) shows only 5% increase in joint shear strength for 74% increase in joint shear reinforcement (0.76% to 2.5%). The two pairs of Chun and Shin (2014) i.e., H1.0U-H1.0S and H0.7U-M0.7U with joint reinforcement variation from 0.31% to 0.46%, shows contradicting effect on joint shear strength i.e., for one pair an increase of 10% and fort the other pair a decrease of 2%. Joint shear strength for specimens of Kaung and Wong (2011) increases by 20% with only 10% increase in joint shear reinforcement, while, the two pairs of specimens of Karayannis *et al.* (2008) with difference of 30% and 56% in joint shear reinforcement demonstrates no effect on joint shear strength. Moreover, it is also difficult to clearly bring out the precise effect of joint reinforcement in comparison to the unreinforced joints.

From the section 4.9 it can be observed that most of the analytical models predict an increase in joint shear strength with increase in joint shear reinforcement which is also evident from most of the identified experimental specimens. However, as observed form analytical models that the joint shear strength significantly increases with increase in joint shear reinforcement beyond 2%, could not be verified due to lack of sufficient numbers of experimental pairs in that range.

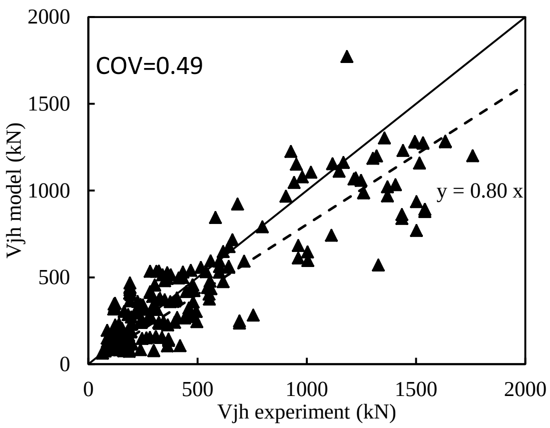
e) Column axial load ratio

To observe the effect of column axial load ratio on joint shear strength, eight pairs with variation in column axial load ratio have been obtained and shown in Table 5 of Appendix II. For the column axial load ratio up to 0.20, five pairs are showing the increase in joint shear strength with the increase in column axial load ratio. For the remaining three pairs no effect on joint shear strength has been observed even with variation in column axial load ratio from 0.15 to 0.30. Test specimens 5-4 of Pantelides *et al.* (2002) shows an 83% increase in joint shear strength with 60 % increase in column axial load ratio (0.08-0.21), while, specimens T10 and T9 of Masi *et al.* (2008) shows 6% decrease in joint shear strength for 50% increase in column axial load ratio (0.15-0.3).

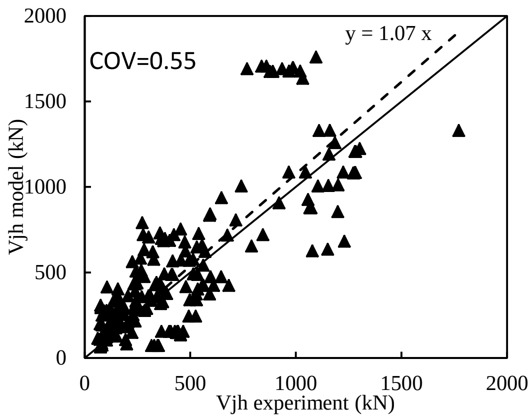
From the section 4.10 and Fig. 10 it can be observed that for seven analytical models, there is a mild increase in the joint shear strength up to axial load ratio of 0.4 and beyond this four models predict steep increase. However, all the pairs considered here are subjected to an axial load ratio less than 0.4, and therefore, it is difficult to comment on the experimental validation of analytical models with high load ratio.

5.2 Compatibility of models with experiments

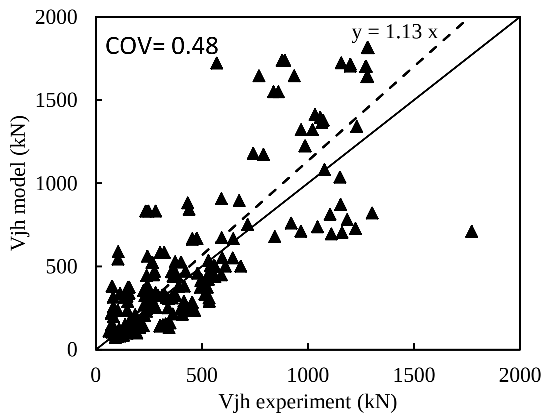
To evaluate the analytical models, the ratio of experimentally obtained joint shear strength to analytically predicted joint shear strength ($V_{jhexpt}/V_{jhmodel}$) has been plotted in Fig. 11. For a good analytical model the ratio ($V_{jhexpt}/V_{jhmodel}$) should be near to unity and the coefficient of variation (COV) has to be minimum. From the statistical evaluation of the ratio ($V_{jhexpt}/V_{jhmodel}$) it has been observed that the analytical models are either predicting lower or higher value, hovering around the desired diagonal as shown in Figs. 11 (a to l). To identify the behavior of analytical model a linear trend line pattern equation has also been plotted in the figures. Majority of the predictions by the Sarsam and Phillips (1985) model are less than the experimental results. On an average this model predicts 20% less strength; also, the COV is 0.49 which shows significant scatter from the experimental results.



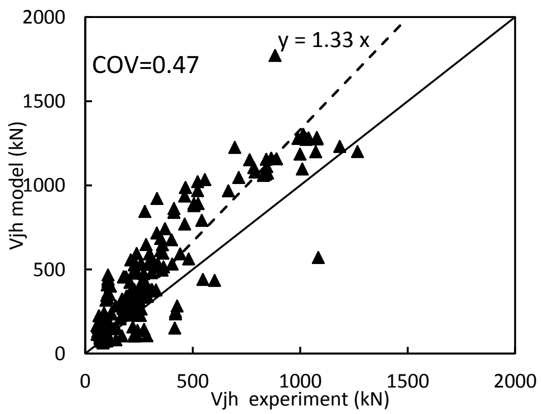
(a) Sarsam and Phillips (1985)



(b) Paulay and Priestley (1992)

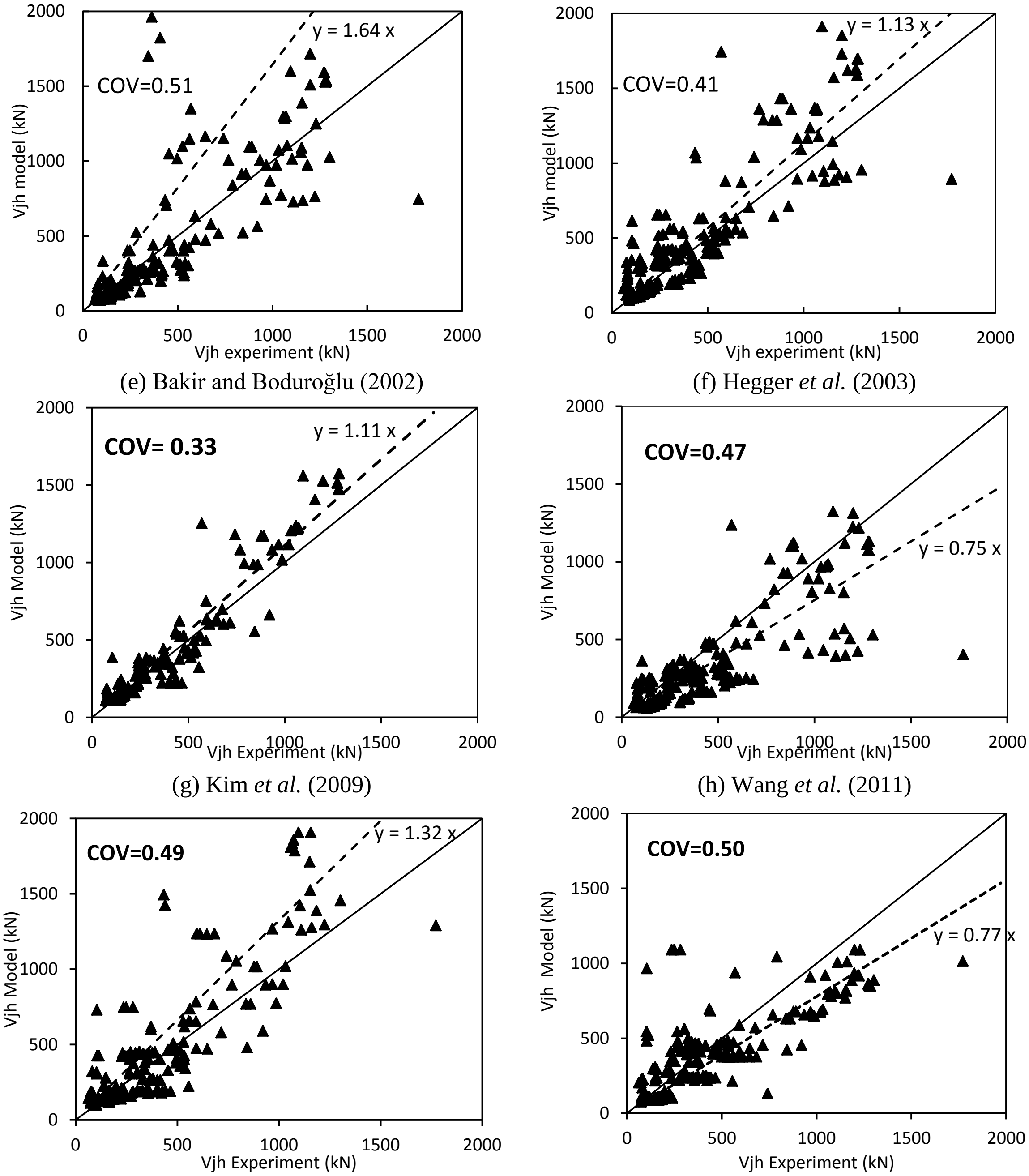


(c) Vollum and Newman (1999)



(d) Hwang and Lee (2002)

Fig. 11 Comparison of experimental results with analytical models



(e) Bakir and Boduroğlu (2002)

(f) Hegger *et al.* (2003)

(g) Kim *et al.* (2009)

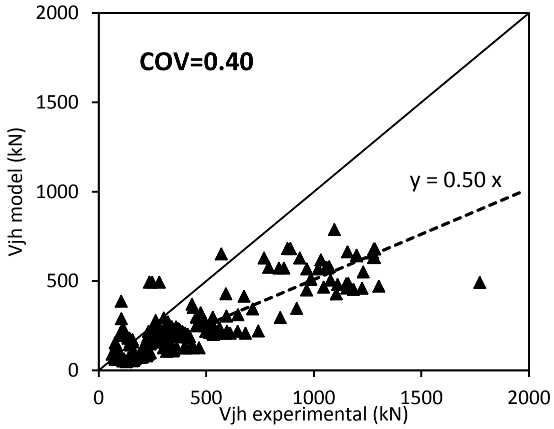
(h) Wang *et al.* (2011)

(i) Sharma *et al.* (2012)

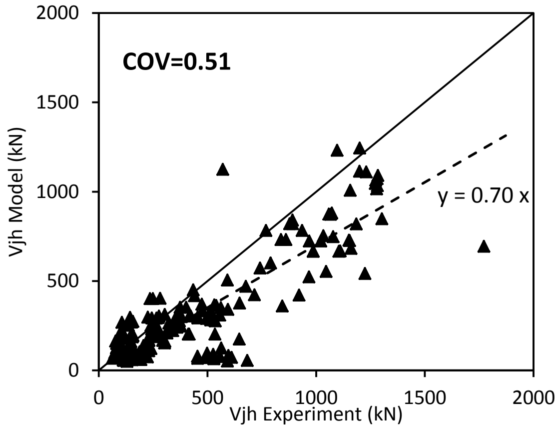
(j) Unal and Burak (2013)

Fig. 11 Continued

Out of the seven empirical models, three models viz., Sarsam and Phillips (1985), Unal and Burak (2013), Tran *et al.* (2014) model predicts 20%, 33% and 50% lesser strength than experimental results. While remaining Vollum and Newman (1999), Bakir and Boduroğlu (2002),



(k) Tran *et al.* (2014)



(l) Pauletta *et al.* (2015)

Fig. 11 Continued

Hegger *et al.* (2003), Kim *et al.* (2009) models are predicting 13%, 64%, 13% and 11% more than experimental results. Kim *et al.* (2009) model is found suitable for shear reinforced joints while for unreinforced joint the model yields zero values which are not shown in the Fig. 11(g). Strut and tie models of Paulay and Priestley (1992), Hwang and Lee (2002) are predicting 7% and 33% more, while the model by Pauletta *et al.* (2015) shows lesser results by 30% than experimental test results. Both theoretical models predicting contrast predictions i.e., Wang *et al.* (2011) is showing 25% lesser and Sharma *et al.* (2012) higher by 32% than experimental results.

6. Conclusions

Detailed review of twelve analytical models for predicting the joint shear strength has been presented in this paper. From the review of analytical models, ten important governing parameters affecting the joint shear strength have been identified. The effect of each governing parameter on joint shear strength predicted by analytical models has been studied in detail by the parametric analysis. The study distinctly points out large variation in predicting the joint shear strength by different models. Moreover, consensus on the relative effect of seven governing parameters viz. beam width and depth, column width and depth, beam and column longitudinal reinforcement, and column axial load ratio has not been attained. Further, to evaluate the accuracy of the twelve analytical models, comparison of joint shear strength with 200 experimental results on exterior RC beam column joint have been considered and studied by two approaches. In the first approach it has been tried to categorize the experimental results on the similar lines of analytical study, but the same was not possible due to significant variability of governing parameters, therefore, various pairs of experimental specimens with only one variable parameter has been identified and grouped for possible five governing parameters. However, from this study also no conclusive findings on effect of governing parameters could be drawn. In the second approach, statistical method is used to correlate the analytical models with experimental results. It has been observed that none of the considered model is capable of predicting the joint shear strength with sufficient accuracy. The minimum COV of 0.33 has been obtained by Kim *et al*. (2009) model, however the comparison is made for only 145 experimental results since the model is incapable in predicting the joint shear strength without joint shear reinforcement. Further, the models by Tran *et al*. (2014) and Hegger *et al*. (2003) are having COV of 0.40 based on 200 experimental results and for other models the COV is higher. It can be concluded that a new improved analytical model for predicting the exterior beam column joint shear strength considering the effect of each governing parameter is required.

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CC

Notations

A_{jh}	Cross sectional area of joint on horizontal plane (mm^2)
A_{sbtop}	Area of top beam longitudinal reinforcement (mm^2)
A_{sbbot}	Area of bottom beam longitudinal reinforcement (mm^2)
A_{sb}	Total area of beam longitudinal reinforcement (mm^2)
A_{sc}	Total area of column longitudinal reinforcement (mm^2)
A_{sj}	Total area of shear reinforcement inside the joint (mm^2)
A_{sjh}	Area of horizontal shear reinforcement inside the joint (mm^2)
A_{sjv}	Area of vertical shear reinforcement inside the joint (mm^2)
b_b	Width of beam (mm)
b_c	Width of column (mm)
b_j	Width of joint (mm)
c	Depth of compression zone in column (mm)
d_b	Diameter of beam longitudinal bar (mm)
f_c	Compressive strength of concrete (N/mm^2)
f_y	Tensile strength of reinforcement (N/mm^2)
f_{yb}	Yield strength of beam longitudinal reinforcement (N/mm^2)
f_{yc}	Yield strength of column longitudinal reinforcement (N/mm^2)
f_{ysj}	Yield strength of joint shear reinforcement (N/mm^2)
h_b	Depth of beam (mm)
h_c	Depth of column (mm)
l_b	Total length of beam of joint test specimen (mm)
l_c	Total length of column of joint test specimen (mm)
N_c	Column axial compressive load (kN)
T	Tensile force in the beam longitudinal reinforcement, ($T=\lambda f_{yb}A_{sbtop}$)
v_{jh}	Horizontal shear stress in MPa
V_{ch}	Horizontal shear strength from concrete strut mechanism (kN)
V_{jh}	Total horizontal joint shear strength (kN)
V_{jhlim}	Limiting horizontal joint shear strength (kN)
V_{sh}	Horizontal shear strength from truss mechanism (kN)
β	Ratio of beam bottom to top reinforcement, ($\beta=A_{sb,bot}/A_{sb,top}$)
θ	Angle of inclination of concrete strut inside the joint panel
ρ_c	Column longitudinal reinforcement ratio (%) (A_{sc}/b_ch_c)
ρ_b	Beam longitudinal reinforcement ratio (%) (A_{sb}/b_bh_b)

Appendix-I

Table 1 Experimental database of exterior RC beam column joints

Sr. No.	*Researchers*	*Speci men Details*	*Column Properties*					*Beam Properties*							*Joint Properties*							
			L	*bc*	*hc*	*Asc*	*fyc*	*L*	*bb*	*hb*	*fyb*	*Asb bot*	*Asb top*	*Asb total*	*Asjh*	*Asjv*	*fyj*	*fc* MPa	*Axial load* (KN)	*Axial Load Ratio*	*Vjh expt* (KN)	*Failure mode*
			mm	mm	mm	mm^2	MPa	mm	mm	mm	MPa	mm^2	mm^2		mm^2	mm^2	MPa					
1	Ehsani *et al.* (1987)	1	1727	340	340	1061	428	1745	300	480	428	1169	1169	2338	760	573	428	64.6	133	0.02	676	BFJF
2	Ehsani *et al.* (1987)	2	1727	340	340	1061	428	1745	300	480	428	1433	1433	2866	760	573	428	67.2	338	0.04	592	BFJF
3	Ehsani *et al.* (1987)	3	1727	300	300	1061	428	1725	259	439	428	1257	1257	2514	760	573	428	64.6	383	0.07	716	BFJF
4	Ehsani *et al.* (1987)	4	1727	300	300	1400	428	1725	259	439	428	1558	1558	3116	760	573	428	67.2	325	0.05	921	JF
5	Ehsani *et al.* (1987)	5	1067	300	300	1146	428	1674	259	439	276	2021	2021	4042	760	573	428	44.2	222	0.06	844	BFJF
6	Ehsani and Alameddine (1991)	LL8	1791	356	356	3482	458	1778	318	508	458	1962	1962	3924	2280	774	427	55	278	0.04	861	BFJF
7	Ehsani and Alameddine (1991)	LH8	1791	356	356	3482	458	1778	318	508	458	1962	1962	3924	2280	774	427	55	278	0.04	838	BFJF
8	Ehsani and Alameddine (1991)	HL8	1791	356	356	3925	458	1778	318	508	458	2640	2640	5280	1520	1013	427	55	487	0.07	987	JF
9	Ehsani and Alameddine (1991)	HH8	1791	356	356	3925	458	1778	318	508	458	2640	2640	5280	1520	1013	427	55	487	0.07	986	BFJF
10	Ehsani and Alameddine (1991)	LL11	1791	356	356	3482	458	1778	318	508	458	1962	1962	3924	2280	774	427	75	285	0.03	769	BFJF
11	Ehsani and Alameddine (1991)	LH11	1791	356	356	3482	458	1778	318	508	458	1962	1962	3924	2280	774	427	75	285	0.03	935	BFJF
12	Ehsani and Alameddine (1991)	HL11	1791	356	356	3925	458	1778	318	508	458	2640	2640	5280	1520	1013	427	75	570	0.06	968	JF
13	Ehsani and Alameddine (1991)	HH11	1791	356	356	3925	458	1778	318	508	458	2640	2640	5280	1520	1013	427	75	570	0.06	1021	BFJF
14	Ehsani and Alameddine (1991)	LL14	1791	356	356	3482	458	1778	318	508	458	1962	1962	3924	2280	774	427	97	246	0.02	878	BFJF
15	Ehsani and Alameddine (1991)	LH14	1791	356	356	3482	458	1778	318	508	458	1962	1962	3924	2280	774	427	97	246	0.02	891	BFJF
16	Ehsani and Alameddine (1991)	HL14	1791	356	356	3482	458	1778	318	508	458	1962	1962	3924	2281	1013	427	97	246	0.02	891	BFJF
17	Ehsani and Alameddine (1991)	HH14	1791	356	356	3925	458	1778	318	508	458	2640	2640	5280	1520	1013	427	97	492	0.04	1033	BFJF
18	Tsonos *et al*. (1992)	S1	650	200	200	462	485	1150	200	300	485	308	308	616	302	616	495	44.6	713.6	0.40	143.4	BFJF
19	Tsonos *et al*. (1992)	X1	650	200	200	462	484	1150	200	300	485	308	308	616	302	0	495	44.6	713.6	0.40	136.1	BFJF
20	Tsonos *et al*. (1992)	S2	650	200	200	157	465	1150	200	300	507	305	305	610	402	616	495	31.3	580	0.46	157.2	JF
21	Tsonos *et al*. (1992)	X2	650	200	200	157	465	1150	200	300	496	305	305	610	302	0	495	31.3	580	0.46	150.7	BFJF
22	Tsonos *et al*. (1992)	S6	650	200	200	308	485	1150	200	300	485	616	616	1232	402	616	495	39.8	636.8	0.40	225.6	JF
23	Tsonos *et al*. (1992)	X6	650	200	200	308	485	1150	200	300	485	616	616	1232	0	0	495	32.5	520	0.40	302.5	JF

Table 1 Continued

Sr. No.	*Researchers*	*Speci men Details*	*Column Properties*					*Beam Properties*							*Joint Properties*							
			L	*bc*	*hc*	*Asc*	*fyc*	*L*	*bb*	*hb*	*fyb*	*Asb bot*	*Asb top*	*Asb total*	*Asjh*	*Asjv*	*fyj*	*fc* MPa	*Axial load* (KN)	*Axial Load Ratio*	*Vjh expt* (KN)	*Failure mode*
			mm	mm	mm	mm^2	MPa	mm	mm	mm	MPa	mm^2	mm^2		mm^2	mm^2	MPa					
24	Tsonos *et al*. (1992)	S'6	650	200	200	616	485	1150	200	300	485	616	616	1232	0	0	495	34.9	558	0.40	303.8	JF
25	Tsonos *et al*. (1992)	F2	650	200	200	308	485	1150	200	300	485	616	616	1232	0	0	495	28.9	462.2	0.40	205.6	JF
26	Scott (1996)	C1	850	150	150	402	525	925	110	210	575	226	226	452	57	0	414	49.90	275	0.24	148.80	BFJF
27	Scott (1996)	C1A	850	150	150	402	525	925	110	210	575	226	226	452	57	0	414	60.00	275	0.20	148.00	BFJF
28	Scott (1996)	CIAL	850	150	150	402	525	925	110	210	575	226	226	452	57	0	414	41.70	50	0.05	114.35	JF
29	Scott (1996)	C2	850	150	150	402	525	925	110	210	575	226	226	452	57	0	414	61.70	275	0.20	110.36	JF
30	Scott (1996)	C2L	850	150	150	402	525	925	110	210	575	226	226	452	57	0	414	45.40	275	0.27	148.48	BFJF
31	Scott (1996)	C3L	850	150	150	402	525	925	110	210	575	226	226	452	57	0	414	44.40	50	0.05	112.31	JF
32	Scott (1996)	C4	850	150	150	402	525	925	110	210	525	402	402	804	57	0	414	51.80	275	0.24	159.61	JF
33	Scott (1996)	C4A	850	150	150	402	525	925	110	210	525	402	402	804	57	0	414	55.40	275	0.22	169.61	JF
34	Scott (1996)	C4AL	850	150	150	402	525	925	110	210	525	402	402	804	57	0	414	44.70	50	0.05	154.26	JF
35	Scott (1996)	C5	850	150	150	402	525	925	110	210	525	402	402	804	57	0	414	41.3	275	0.30	75.61	JF
36	Scott (1996)	C6	850	150	150	402	525	925	110	210	525	402	402	804	57	0	414	49.80	275	0.25	118.66	JF
37	Scott (1996)	C6L	850	150	150	402	525	925	110	210	525	402	402	804	57	0	414	57.30	50	0.04	140.76	JF
38	Scott (1996)	C7	850	150	150	402	525	925	110	300	525	402	402	804	57	0	414	44.00	275	0.28	103.48	JF
39	Scott (1996)	C8	850	150	150	402	525	925	110	300	525	402	402	804	57	0	414	55.60	275	0.22	89.04	JF
40	Scott (1996)	C9	850	150	150	402	525	925	110	300	525	402	402	804	57	0	414	44.90	275	0.27	92.21	JF
41	Parker & Bullman (1997)	4a	1000	300	300	402	550	1000	250	500	570	982	982	1964	0	0	560	49.00	0	0.00	231.3	JF
42	Parker & Bullman (1997)	4b	1000	300	300	402	550	1000	250	500	570	982	982	1964	0	0	560	49.00	300	0.07	270.5	JF
43	Parker & Bullman (1997)	4c	1000	300	300	402	550	1000	250	500	570	982	982	1964	0	0	560	46.00	570	0.14	333.2	JF
44	Parker & Bullman (1997)	4d	1000	300	300	1608	580	1000	250	500	570	982	982	1964	0	0	560	49.00	0	0.00	294	JF
45	Parker & Bullman (1997)	4e	1000	300	300	1608	580	1000	250	500	570	982	982	1964	0	0	560	50.00	300	0.07	313.6	JF
46	Parker & Bullman (1997)	4f	1000	300	300	1608	580	1000	250	500	570	982	982	1964	0	0	560	47.00	600	0.14	358.9	JF
47	Parker & Bullman (1997)	5a	1000	300	300	982	485	1000	250	500	485	982	982	1964	679	0	480	53.00	0	0.00	455.9	JF
48	Parker & Bullman (1997)	5b	1000	300	300	982	485	1000	250	500	485	982	982	1964	679	0	480	54.00	300	0.06	477.2	JF
49	Parker & Bullman (1997)	5c	1000	300	300	982	485	1000	250	500	485	982	982	1964	679	0	480	54.00	600	0.12	474.2	JF
50	Parker & Bullman (1997)	5d	1000	300	300	982	485	1000	250	500	515	1608	1608	3216	679	0	480	54.00	0	0.00	454.6	JF
51	Parker & Bullman (1997)	5e	1000	300	300	982	485	1000	250	500	515	1608	1608	3216	679	0	480	56.00	300	0.00	593.9	JF
52	Parker & Bullman (1997)	5f	1000	300	300	982	485	1000	250	500	515	1608	1608	3216	679	0	480	54.00	600	0.06	647.7	JF
53	Wallace *et al.* (1998)	BCEJ1	1524	457	457	1927	455	3276	457	610	483	1520	2027	3547	1520	1285	462	35.8	0	0.00	741.9	BFJF
54	Wallace *et al.* (1998)	BCEJ2	1524	457	457	1927	455	3276	457	610	483	1520	2027	3547	760	1285	462	33.6	0	0.00	790.5	CFJF
55	Tsonos (1999)	M1	650	200	200	157	465	1100	200	300	465.1	383	383	766	402	0	494.6	34	300	0.22	153.7	CFJF
56	Tsonos (1999)	M2	650	200	200	157	465	1100	200	300	484.1	616	616	1232	402	0	494.6	33.5	300	0.22	282.2	CFJF

Table 1 Continued

Sr. No.	*Researchers*	*Speci men Details*	*Column Properties*					*Beam Properties*								*Joint Properties*			*fc* MPa	*Axial load* (KN)	*Axial Load Ratio*	*Vjh expt* (KN)	*Failure mode*
			L	*bc*	*hc*	*Asc*	*fyc*	*L*	*bb*	*hb*	*fyb*	*Asb bot*	*Asb top*	*Asb total*	*Asjh*	*Asjv*	*fyj*					
			mm	mm	mm	mm²	MPa	mm	mm	mm	MPa	mm²	mm²		mm²	mm²	MPa					
57	Clyde *et al.* (2000)	#2	1283	305	457	1611	470	1499	305	406	454.4	2588	2588	5176	0	774	454.4	55.7	689	0.09	1154	JF
58	Clyde *et al.* (2000)	#4	1283	305	457	1611	470	1499	305	406	454.4	2588	2588	5176	0	774	454.4	49.4	1380	0.20	1303	JF
59	Clyde *et al.* (2000)	#5	1283	305	457	1611	470	1499	305	406	454.4	2588	2588	5176	0	774	454.4	44.6	1357	0.22	1185	JF
60	Clyde *et al.* (2000)	#6	1283	305	457	1611	470	1499	305	406	454.4	2588	2588	5176	0	774	454.4	48.3	587	0.09	1104	JF
61	Hamil (2000)	C6LN0	850	150	150	402	525	825	110	210	525	402	402	804	0	0	414	53.1	50	0.04	113.9	JF
62	Hamil (2000)	C6LN1	850	150	150	402	525	825	110	210	525	402	402	804	57	0	414	53.1	50	0.04	118.7	JF
63	Hamil (2000)	C6LN3	850	150	150	402	525	825	110	210	525	402	402	804	170	0	414	50.6	50	0.04	137.9	JF
64	Hamil (2000)	C6LN5	850	150	150	402	525	825	110	210	525	402	402	804	283	0	414	38.1	50	0.06	164.9	JF
65	Hamil (2000)	C6LH0	850	150	150	402	525	825	110	210	525	402	402	804	0	0	414	105	100	0.04	163.9	JF
66	Hamil (2000)	C6LH1	850	150	150	402	525	825	110	210	525	402	402	804	57	0	414	105	100	0.04	169.1	JF
67	Hamil (2000)	C6LH3	850	150	150	402	525	825	110	210	525	402	402	804	170	0	414	100	100	0.04	187.7	JF
68	Hamil (2000)	C6LH5	850	150	150	402	525	825	110	210	525	402	402	804	283	0	414	104	100	0.04	239.1	BFJF
69	Hamil (2000)	C4ALN0	850	150	150	402	525	825	110	210	525	402	402	804	0	0	414	44	50	0.05	129.6	JF
70	Hamil (2000)	C4ALN1	850	150	150	402	525	825	110	210	525	402	402	804	57	0	414	47.3	50	0.05	162.3	JF
71	Hamil (2000)	C4ALN3	850	150	150	402	525	825	110	210	525	402	402	804	170	0	414	43.2	50	0.05	168.3	JF
72	Hamil (2000)	C4ALN5	850	150	150	402	525	825	110	210	525	402	402	804	283	0	414	52.3	50	0.04	185.2	JF
73	Hamil (2000)	C4ALH0	850	150	150	402	525	825	110	210	525	402	402	804	0	0	414	108	100	0.04	201	JF
74	Hamil (2000)	C4ALH1	850	150	150	402	525	825	110	210	525	402	402	804	57	0	414	98.8	100	0.04	204.9	BFJF
75	Hamil (2000)	C4ALH3	850	150	150	402	525	825	110	210	525	402	402	804	170	0	414	110	100	0.04	229.8	BFJF
76	Hamil (2000)	C4ALH5	850	150	150	402	525	825	110	210	525	402	402	804	283	0	414	102	100	0.04	240.1	BFJF
77	Hakuto *et al.* (2000)	O6	1600	460	460	905	308	1905	300	500	308	905	1357	2262	57	0	398	41	0	0.00	434	BFJF
78	Hakuto *et al.* (2000)	O7	1600	460	460	905	308	1905	300	500	308	905	1357	2262	57	0	398	37.3	0	0.00	440	BFJF
79	Calvi *et al.* (2001)	T1	1165	200	200	151	386	1700	200	330	366	327	327	654	0	0	366	23.9	120	0.13	62.29	BFJF
80	Gencoglu & Eren (2002)	#1	1500	250	400	603	500	1200	250	600	500	462	462	924	503	0	500	29.5	150	0.05	105.8	BFJF
81	Gencoglu & Eren (2002)	#2	1500	250	400	603	500	1200	250	600	500	462	462	924	0	0	500	29.5	150	0.05	114.4	JF
82	Pampanin *et al.* (2002)	T2	1245	200	200	151	386	1700	200	330	366	327	327	654	0	0	366	29.1	100	0.09	72.9	JF
83	Pantelides *et al.* (2002)	1	1600	406	406	2027	470	1880	406	406	458.9	2588	2588	5176	0	0	458.9	39.9	546.7	0.08	1225	JF
84	Pantelides *et al.* (2002)	2	1600	406	406	2027	470	1880	406	406	458.9	2588	2588	5176	0	0	458.9	36.4	1247	0.21	1110	JF
85	Pantelides *et al.* (2002)	3	1600	406	406	2027	470	1880	406	406	458.9	2588	2588	5176	0	0	458.9	41	561.5	0.08	1046	JF
86	Pantelides *et al.* (2002)	4	1600	406	406	2027	470	1880	406	406	458.9	2588	2588	5176	0	0	458.9	38.1	1305	0.21	1772	CFJF
87	Pantelides *et al.* (2002)	5	1600	406	406	2027	470	1880	406	406	458.9	2588	2588	5176	0	0	458.9	38.2	523.6	0.08	966.9	BFJF
88	Pantelides *et al.* (2002)	6	1600	406	406	2027	470	1880	406	406	458.9	2588	2588	5176	0	0	458.9	37.3	1280	0.21	1161	CFJF
89	Hegger *et al.* (2003)	RK1	830	150	240	402	530	970	150	300	530	628	628	1256	628	402	530	57.9	500	0.24	374	BFJF

Table 1 Continued

Sr. No.	*Researchers*	*Speci men Details*	*Column Properties* *L* mm	*bc* mm	*hc* mm	*Asc* mm^2	*fyc* MPa	*Beam Properties* *L* mm	*bb* mm	*hb* mm	*fyb* MPa	*Asb bot* mm^2	*Asb top* mm^2	*Asb total*	*Joint Properties* *Asjh* mm^2	*Asjv* mm^2	*fyj* MPa	*fc* MPa	*Axial load* (KN)	*Axial Load Ratio*	*Vjh expt* (KN)	*Failure mode*
90	Hegger *et al*. (2003)	RK2	830	150	240	402	530	970	150	300	530	628	628	1256	402	353	530	57.4	500	0.24	417	BFJF
91	Hegger *et al*. (2003)	RK3	830	150	240	402	530	970	150	300	530	628	628	1256	628	402	530	57.2	500	0.24	402	BFJF
92	Hegger *et al*. (2003)	RK4	830	150	200	402	530	970	150	300	530	628	628	1256	628	402	530	51.7	500	0.32	357	JF
93	Hegger *et al*. (2003)	RK5	830	150	200	402	530	970	150	300	530	628	628	1256	628	402	530	54.9	500	0.30	423	JF
94	Hegger *et al*. (2003)	RK6	830	150	200	402	530	970	150	300	530	628	628	1256	628	402	530	86.5	500	0.19	556	JF
95	Hegger *et al*. (2003)	RK7	830	150	200	402	530	970	150	300	530	628	628	1256	628	402	530	54.7	500	0.30	277	JF
96	Hegger *et al*. (2003)	RK8	830	150	200	402	530	970	150	300	530	628	628	1256	628	402	530	38.6	500	0.43	273	JF
97	Murthy *et al*. (2003)	P1	550	200	250	1200	415	1150	200	400	415	630	630	1260	0	0	0	27	0	0.00	346	JF
98	Murthy *et al*. (2003)	P2	550	200	250	1200	415	1150	200	400	415	630	630	1260	200	0	415	26	0	0.00	408	BFJF
99	Murthy *et al*. (2003)	P3	550	200	250	1200	415	1150	200	400	415	630	630	1260	200	0	415	27	0	0.00	364	BFJF
100	Murthy *et al*. (2003)	Q1	550	200	250	1200	415	1150	200	400	415	630	630	1260	0	0	0	26	0	0.00	317	JF
101	Murthy *et al*. (2003)	Q2	550	200	250	1200	415	1150	200	400	415	630	630	1260	200	0	415	27	0	0.00	443	BFJF
102	Murthy *et al*. (2003)	Q3	550	200	250	1200	415	1150	200	400	415	630	630	1260	200	0	415	27	0	0.00	428	BFJF
103	Murthy *et al*. (2003)	R1	550	200	250	1200	415	1150	200	400	415	630	630	1260	0	0	0	30	0	0.00	350	JF
104	Murthy *et al*. (2003)	R2	550	200	250	1200	415	1150	200	400	415	630	630	1260	200	0	415	27	0	0.00	467	BFJF
105	Murthy *et al*. (2003)	R3	550	200	250	1200	415	1150	200	400	415	630	630	1260	200	0	415	27	0	0.00	440	BFJF
106	Murthy *et al*. (2003)	S1	550	200	250	1200	415	1150	200	400	415	630	630	1260	0	0	0	28	0	0.00	330	JF
107	Murthy *et al*. (2003)	S2	550	200	250	1200	415	1150	200	400	415	630	630	1260	200	0	415	27	0	0.00	440	BFJF
108	Murthy *et al*. (2003)	S3	550	200	250	1200	415	1150	200	400	415	630	630	1260	200	0	415	30	0	0.00	400	BFJF
109	Hwang *et al*. (2004)	70-3T44	1350	420	420	2458	421	2110	320	450	430	2027	2027	4054	2280	1639	498	92.5	196	0.01	1096	BFJF
110	Hwang *et al*. (2004)	70-3T4	1350	450	450	2458	458	2125	320	450	491	2027	2027	4054	1140	1639	436	90.6	196	0.01	1284	BFJF
111	Hwang *et al*. (2004)	70-2T5	1350	450	450	2458	458	2125	320	450	491	2027	2027	4054	792	1639	469	92.3	196	0.01	1275	BFJF
112	Hwang *et al*. (2004)	70-1T55	1350	450	450	2458	458	2125	320	450	491	2027	2027	4054	792	1639	469	84	196	0.01	1282	BFJF
113	Hwang *et al*. (2004)	28-3T4	1350	550	550	3278	458	2175	380	500	491	2027	2027	4054	760	3278	436	42.4	196	0.02	1200	BFJF
114	Hwang *et al*. (2004)	28-0T0	1350	550	550	3278	458	2175	380	500	491	2027	2027	4054	0	3278	458	39.8	196	0.02	1230	JF
115	Chun & Kim (2004)	JC1	1500	500	500	1901	403	2400	350	500	402.9	1140	1521	2661	1013	3041	383.9	61.7	0	0.00	570	BFJF
116	Chun & Kim (2004)	JC2	1500	500	500	1901	403	2400	350	500	402.9	2281	3041	5322	1013	3041	383.9	60.1	0	0.00	1199	JF
117	Hwang *et al*. (2005)	0T0	1350	420	420	2458	421	2110	320	450	430	2027	2027	4054	0	1639	430	81.1	196	0.01	1078	BFJF
118	Hwang *et al*. (2005)	3T44	1350	420	420	2458	421	2110	320	450	430	2027	2027	4054	1140	1639	498	92.5	196	0.01	1157	JF
119	Hwang *et al*. (2005)	1B8	1350	420	420	2458	430	2110	320	450	435	2027	2027	4054	0	1639	435	74.5	196	0.01	1151	BFJF
120	Hwang *et al*. (2005)	3T3	1350	420	420	2458	421	2110	320	450	430	2027	2027	4054	638	1639	471	83.1	196	0.01	1058	BFJF
121	Hwang *et al*. (2005)	2T4	1350	420	420	2458	421	2110	320	450	430	2027	2027	4054	507	1639	498	85.5	196	0.01	1066	BFJF
122	Hwang *et al*. (2005)	1T44	1350	420	420	2458	421	2110	320	450	430	2027	2027	4054	507	1639	498	87.7	196	0.01	1072	BFJF

Table 1 Continued

Sr. No.	*Researchers*	*Speci men Details*	*Column Properties*					*Beam Properties*							*Joint Properties*			*fc* MPa	*Axial load* (KN)	*Axial Load Ratio*	*Vjh expt* (KN)	*Failure mode*
			L	*bc*	*hc*	*Asc*	*fyc*	*L*	*bb*	*hb*	*fyb*	*Asb bot*	*Asb top*	*Asb total*	*Asjh*	*Asjv*	*fyj*					
			mm	mm	mm	mm^2	MPa	mm	mm	mm	MPa	mm^2	mm^2		mm^2	mm^2	MPa					
123	Hwang *et al*. (2005)	3T4	1350	450	450	2458	458	2125	320	450	491	2027	2027	4054	1140	1639	436	90.7	196	0.01	1280	BFJF
124	Hwang *et al*. (2005)	2T5	1350	450	450	2458	458	2125	320	450	491	2027	2027	4054	792	1639	469	92.3	196	0.01	1272	BFJF
125	Hwang *et al.* (2005)	1T55	1350	450	450	2458	458	2125	320	450	491	2027	2027	4054	792	1639	469	84	196	0.01	1278	BFJF
126	Kuang & Wong (2006)	BS-OL	1550	300	300	982	520	1650	260	450	520	942	942	1884	0	0	520	30.9	403	0.14	264.2	JF
127	Kuang & Wong (2006)	BS-LL	1550	300	300	982	520	1650	260	450	520	942	942	1884	0	0	520	30.9	403	0.14	534.6	JF
128	Kuang & Wong (2006)	BSU	1550	300	300	982	520	1650	260	450	520	942	942	1884	0	0	520	30.9	403	0.14	411.2	JF
129	Kuang & Wong (2006)	BSLLS	1550	300	300	982	520	1650	260	450	520	942	942	1884	0	0	520	30.9	403	0.14	415.7	JF
130	Liu (2006)	RC-1	1000	230	230	236	324	1525	200	330	323.8	471	471	942	0	0	383.7	18	75	0.08	148.7	JF
131	Liu (2006)	RC-6	1000	250	250	452	307	1525	250	330	306.7	452	452	904	57	226	383.7	25	100	0.06	148.9	BFJF
132	Liu (2006)	NZ-7	1000	250	250	452	307	1525	250	330	306.7	452	452	904	283	226	383.7	25	100	0.06	147.4	BFJF
133	Alva *et al*. (2007)	LVP2	1250	200	300	603	594	1700	200	400	594	804	804	1608	201	804	602	44.2	397	0.15	514	JF
134	Alva *et al*. (2007)	LVP3	1250	200	300	603	594	1700	200	400	594	804	804	1608	402	804	602	23.9	215	0.15	364	JF
135	Alva *et al.* (2007)	LVP4	1250	200	300	603	594	1700	200	400	594	804	804	1608	201	804	602	24.6	221	0.15	327	JF
136	Alva *et al*. (2007)	LVP5	1250	200	300	603	594	1700	200	400	594	804	804	1608	402	804	602	25.9	233	0.15	380	JF
137	Genesan *et al*. (2007)	Hpr	500	150	200	156	428	650	150	200	428	226	226	452	57	0	428	76.2	15.7	0.01	81.12	BFJF
138	Idayani (2007)	S1	865	180	180	402	460	1000	150	300	460	402	628	1030	0	0	250	36.1	90	0.08	194	JF
139	Idayani (2007)	S2	865	180	180	402	460	1000	150	300	460	402	628	1030	0	0	250	94	90	0.03	199	JF
140	Idayani (2007)	S3	865	180	180	402	460	1000	150	300	460	402	628	1030	170	0	250	36.1	90	0.08	224	JF
141	Tsonos (2007)	A1	700	200	200	236	500	1000	200	300	500	314	314	628	424	157	540	35	200	0.14	157.3	BFJF
142	Tsonos (2007)	E2	700	200	200	462	495	1000	200	300	495	308	308	616	424	308	540	35	200	0.14	152.5	BFJF
143	Tsonos (2007)	E1	700	200	200	462	495	1000	200	300	495	462	462	924	424	308	540	26.5	200	0.19	234	JF
144	Tsonos (2007)	G1	700	200	200	462	495	1000	200	300	495	462	462	924	452	308	500	26.5	200	0.19	239.3	JF
145	Karayannis *et al*. (2008)	A0	750	200	200	157	580	1100	200	300	580	157	157	314	0	0	580	31.6	152.3	0.12	82.56	JF
146	Karayannis *et al*. (2008)	A1	750	200	200	157	580	1100	200	300	580	157	157	314	101	0	580	31.6	126.4	0.10	82.93	BFJF
147	Karayannis *et al*. (2008)	A2	750	200	200	157	580	1100	200	300	580	157	157	314	201	0	580	31.6	152.3	0.12	82.93	BFJF
148	Karayannis *et al.* (2008)	A3	750	200	200	157	580	1100	200	300	580	157	157	314	302	0	580	31.6	152.3	0.12	82.2	BFJF
149	Karayannis *et al*. (2008)	B0	750	200	300	157	580	1150	200	300	580	471	471	942	0	0	580	31.6	228.4	0.12	227.8	JF
150	Karayannis *et al*. (2008)	B1	750	200	300	157	580	1150	200	300	580	471	471	942	101	0	580	31.6	228.4	0.12	253.3	JF
151	Karayannis *et al*. (2008)	C0	750	200	300	308	580	1150	200	300	580	452	452	904	0	157	580	31.6	228.4	0.12	239.7	JF
152	Karayannis *et al*. (2008)	C2	750	200	300	308	580	1150	200	300	580	452	452	904	201	157	580	31.6	228.4	0.12	242.8	BFJF
153	Karayannis *et al*. (2008)	C3	750	200	300	308	580	1150	200	300	580	452	452	904	302	157	580	31.6	228.4	0.12	239.7	BFJF
154	Karayannis *et al*. (2008)	C5	750	200	300	308	580	1150	200	300	580	452	452	904	503	157	580	31.6	228.4	0.12	243.5	BFJF
155	Karayannis & sirkelis (2008)	A1	750	200	200	157	574	1100	200	300	574	157	157	314	0	0	574	36.4	70	0.05	74.95	JF

Table 1 Continued

Sr. No.	*Researchers*	*Speci men Details*	*Column Properties*					*Beam Properties*							*Joint Properties*							
			L	*bc*	*hc*	*Asc*	*fyc*	*L*	*bb*	*hb*	*fyb*	*Asb bot*	*Asb top*	*Asb total*	*Asjh*	*Asjv*	*fyj*	*fc* MPa	*Axial load* (KN)	*Axial Load Ratio*	*Vjh expt* (KN)	*Failure mode*
			mm	mm	mm	mm^2	MPa	mm	mm	mm	MPa	mm^2	mm^2		mm^2	mm^2	MPa					
156	Karayannis & sirkelis (2008)	A2	750	200	200	157	574	1100	200	300	574	157	157	314	0	0	574	36.4	70	0.05	76.06	JF
157	Karayannis & sirkelis (2008)	B1	750	200	200	339	574	1100	200	300	574	157	157	314	402	0	574	36.4	70	0.05	77.19	BFJF
158	Karayannis & sirkelis (2008)	B2	750	200	200	339	574	1100	200	300	574	157	157	314	402	0	574	36.4	70	0.05	77.19	BFJF
159	Bindu & Jaya (2008)	A1	500	100	150	129	432	625	100	150	432	157	157	314	339	57	432	36.7	15.92	0.03	74.71	BFJF
160	Bindu & Jaya (2008)	A2	500	100	150	129	432	625	100	150	432	157	157	314	339	57	432	36.7	15.92	0.03	73.18	BFJF
161	Kusuhara & Shiohara (2008)	E1	735	300	300	265	375	1350	300	300	379	1206	1206	2412	170	265	366	30.4	216	0.08	535.6	JF
162	Kusuhara & Shiohara (2008)	E2	735	300	300	265	375	1350	300	300	379	1206	1206	2412	170	265	366	30.4	216	0.08	371.1	JF
163	Kusuhara & Shiohara (2008)	B2	735	300	300	398	357	1350	300	300	456	796	796	1592	170	265	326	28.6	216	0.08	370	JF
164	Wong & Kuang (2008)	BSL300	1550	300	300	982	520	1650	260	300	520	942	942	1884	0	0	500	42.6	575.1	0.15	561.8	JF
165	Wong & Kuang (2008)	BSL450	1550	300	300	982	520	1650	260	450	520	942	942	1884	0	0	500	38.6	521.1	0.15	377.4	JF
166	Wong & Kuang (2008)	BSL600	1550	300	300	982	520	1650	260	600	520	942	942	1884	0	0	500	45.5	614.3	0.15	340.1	JF
167	Wong & Kuang (2008)	BSLV2	1550	300	300	982	520	1650	260	450	520	942	942	1884	0	314	500	40.7	549.5	0.15	514.2	JF
168	Wong & Kuang (2008)	BSLV4	1550	300	300	982	520	1650	260	450	520	942	942	1884	0	628	500	35.5	477.9	0.15	533.8	JF
169	Wong & Kuang (2008)	BSLH1	1550	300	300	982	520	1650	260	450	520	942	942	1884	157	0	500	41.6	561.6	0.15	495.5	JF
170	Wong & Kuang (2008)	BSLH2	1550	300	300	982	520	1650	260	450	520	942	942	1884	314	0	500	52.6	710.1	0.15	531.1	JF
171	Masi *et al.* (2008)	T1	1600	300	300	308	478	2150	300	500	478	226	226	452	0	0	478	21.5	290.3	0.15	103.8	BFJF
172	Masi *et al.* (2008)	T2	1600	300	300	462	478	2150	300	500	478	512	603	1115	603	0	478	21.5	580.5	0.30	264.8	BFJF
173	Masi *et al.* (2008)	T3	1600	300	300	462	478	2150	600	240	478	512	603	1115	603	0	478	21.5	580.5	0.30	282	BFJF
174	Masi *et al.* (2008)	T4	1600	300	300	308	478	2150	600	240	478	226	226	452	603	0	478	21.5	580.5	0.30	235.6	BFJF
175	Masi *et al.* (2008)	T5	1600	300	300	462	478	2150	300	500	478	512	603	1115	603	0	478	21.5	290.3	0.15	267.9	BFJF
176	Masi *et al.* (2008)	T6	1600	300	300	308	478	2150	300	500	478	226	226	452	0	0	478	21.5	580.5	0.30	103.3	BFJF
177	Masi *et al.* (2008)	T7	1600	300	300	308	478	2150	600	240	478	226	226	452	0	0	478	21.5	290.3	0.15	104.6	BFJF
178	Masi *et al.* (2008)	T8	1600	300	300	308	478	2150	600	240	478	226	226	452	603	0	478	21.5	580.5	0.30	248.5	BFJF
179	Masi *et al.* (2008)	T9	1600	300	300	462	580	2150	300	500	580	512	603	1115	603	0	580	21.5	580.5	0.30	303.2	BFJF
180	Masi *et al.* (2008)	T10	1600	300	300	462	580	2150	300	500	580	512	603	1115	603	0	580	21.5	290.3	0.15	322.5	BFJF
181	Kaung and Wong (2011)	BS450	1325	300	300	1965	520	1500	260	450	520	942	942	1884	0	0	0	39	0	0.00	315	JF
182	Kaung and Wong (2011)	BS450 H1T10	1325	300	300	1965	520	1500	260	450	520	942	942	1884	80	0	500	42	0	0.00	389	JF
183	Kaung and Wong (2011)	BS450 H2T10	1325	300	300	1965	520	1500	260	450	520	942	942	1884	160	0	500	53	0	0.00	480	JF
184	Kaung and Wong (2011)	BS600	1325	300	300	1965	520	1500	260	600	520	942	942	1884	0	0	0	46	0	0.00	284	JF

Table 1 Continued

Sr. No.	Researchers	Speci men Details	*Column Properties* *L* mm	*bc* mm	*hc* mm	*Asc* mm^2	*fyc* MPa	*Beam Properties* *L* mm	*bb* mm	*hb* mm	*fyb* MPa	*Asb bot* mm^2	*Asb top* mm^2	*Asb total*	*Joint Properties* *Asjh* mm^2	*Asjv* mm^2	*fyj* MPa	*fc* MPa	*Axial load* (KN)	*Axial Load Ratio*	*Vjh expt* (KN)	*Failure mode*
185	Kaung and Wong (2011)	BS600 H2T8	1325	300	300	1965	520	1500	260	600	520	942	942	1884	80	0	500	53	0	0.00	360	JF
186	Kaung and Wong (2011)	BS600 H4T8	1325	300	300	1965	520	1500	260	600	520	942	942	1884	160	0	500	37	0	0.00	342	JF
187	Chun & Shin (2014)	H0.7S	1300	300	300	3482	461	1200	250	200	488	850	1134	1984	345	0	460	35	0	0.00	646	BF
188	Chun & Shin (2014)	H1.0S	1250	300	300	3482	461	1200	250	300	488	850	1134	1984	345	0	460	35	0	0.00	563	BF
189	Chun & Shin (2014)	H1.5S	1175	300	300	3482	461	1200	250	450	488	850	1134	1984	345	0	460	35	0	0.00	498	BFJF
190	Chun & Shin (2014)	H2.0S	1100	300	300	3482	461	2400	250	600	488	850	1134	1984	345	0	460	48	0	0.00	526	BFJF
191	Chun & Shin (2014)	H2.5S	1025	300	300	3482	461	2400	250	750	488	850	1134	1984	345	0	460	48	0	0.00	454	BFJF
192	Chun & Shin (2014)	H0.7U	1300	300	300	3482	461	1200	250	200	488	850	1134	1984	235	0	460	35	0	0.00	611	BF
193	Chun & Shin (2014)	H1.0U	1250	300	300	3482	461	1200	250	300	488	850	1134	1984	235	0	460	35	0	0.00	529	BF
194	Chun & Shin (2014)	M0.7S	1300	300	300	3482	461	1200	250	200	488	850	1134	1984	345	0	460	35	0	0.00	596	BF
195	Chun & Shin (2014)	M1.0S	1250	300	300	3482	461	1200	250	300	488	850	1134	1984	345	0	460	35	0	0.00	557	BF
196	Chun & Shin (2014)	M1.5S	1175	300	300	3482	461	1200	250	450	488	850	1134	1984	345	0	460	35	0	0.00	530	BFJF
197	Chun & Shin (2014)	M2.0S	1100	300	300	3482	461	2400	250	600	488	850	1134	1984	345	0	460	48	0	0.00	493	BFJF
198	Chun & Shin (2014)	M2.5S	1025	300	300	3482	461	2400	250	750	488	850	1134	1984	345	0	460	48	0	0.00	455	NA
199	Chun & Shin (2014)	M0.7U	1300	300	300	3482	461	1200	250	200	488	850	1134	1984	235	0	460	35	0	0.00	683	BF
200	Chun & Shin (2014)	M1.0U	1250	300	300	3482	461	1200	250	300	488	850	1134	1984	235	0	460	35	0	0.00	592	BF

Appendix-II

Table 1 Effect of variation in concrete grade on joint shear strength
(a) For concrete grade less than 60 MPa

Sr. No	Research groups	Specimen compared	Concrete grade Range (Round off)	Concrete grade % Diff.	V_{jh} Expt. Range (Round off)	V_{jh} Expt. % Diff.	Remarks
1.	Tsonos *et al. (*1992)	F2 X6	29 33	12%	205 302	+47%	Increasing effect
2.	Scott (1996)	C4AL C6L	45 57	21%	154 140	- 9%	Decreasing effect
3.	Clyde *et al.* (2000)	#6 #2	48 56	14%	1104 1154	+5%	Increasing effect
4.	Clyde *et al.* (2000)	#5 #4	45 49	8%	1185 1302	+10%	Increasing effect
5.	Hamil (2000)	C4ALN0 C6LN0	44 53	17%	130 114	-12%	Decreasing effect
6.	Hamil (2000)	C4ALN1 C6LN1	47 53	11%	162 118	-37%	Decreasing effect
7.	Hamil (2000)	C4ALN3 C6LN3	43 51	16%	198 138	-22%	Decreasing effect
8.	Hamil (2000)	C6LN5 C4ALN5	38 52	27%	165 185	+11%	Increasing effect
9.	Hakuto *et al.* (2000)	O7 O6	37 41	9%	440 434	-1%	Insignificant
10.	Murty *et al.* (2003)	Q1 R1	26 30	13%	316 350	+10%	Increasing effect
11.	Murty *et al.* (2003)	P2 S2	26 30	13%	408 400	-2%	Insignificant
12.	Alva *et al.* (2007)	LVP4 LVP2	25 44	43%	327 514	+57%	Increasing effect
13.	Wong and Kuang (2008)	BS-L-V2 BS-L-H2	41 53	23%	514 531	+3%	Insignificant

(b) For concrete grade more than 60 MPa

Sr. No	Research groups	Specimen compared	Concrete grade Range (Round off)	Concrete grade % Diff.	V_{jh} Expt. Range (Round off)	V_{jh} Expt. % Diff.	Remarks
1.	Hamil (2000)	C4ALH1 C6LH1	99 105	6%	205 169	-17%	Decreasing effect
2.	Hamil (2000)	C6LH3 C4ALH3	100 110	9%	188 230	+18%	Increasing effect
3.	Hwang *et al.* (2004)	70-1T55 70-2T5	84 92	9%	1282 1275	-1%	Insignificant
4.	Hwang *et al.* (2005)	1B8 0T0	75 81	7%	1151 1078	-6%	Decreasing effect
5.	Hwang *et al.* (2005)	1T55 2T5	92 84	9%	1278 1272	-0.4%	Insignificant

Table 2 Effect of variation in beam longitudinal reinforcement on joint shear strength

Sr. No.	Research groups	Specimen compared	Beam Reinf.		V_{jh} Expt.		Remarks
			Range (Round off)	% Diff.	Range (Round off)	% Diff.	
1.	Tsonos (1999)	M1 M2	1.28 2.05	+38%	154 282	+45%	Increasing effect
2.	Chun and Kim (2004)	JC1 JC2	1.52 3.04	+50%	570 1199	+53%	Increasing effect
3.	Wong and Kuang (2008)	BS-L-600 BS-L-450	1.21 1.61	+25%	340 377	+10%	Increasing effect

Table 3 Effect of variation in column longitudinal reinforcement on joint shear strength

Sr. No.	Research groups	Specimen compared	Column Reinf.		V_{jh} Expt.		Remarks
			Range (Round off)	% Diff.	Range (Round off)	% Diff.	
1.	Tsonos (1992)	X6 S'6	0.77 1.54	50%	302 303	0%	Insignificant
2.	Parker and Bullman (1997)	4C 4F	0.45 1.79	75%	333 358	+7%	Increasing effect
3.	Hwang *et al.* (2005)	1T55 3T3	1.21 1.39	13%	1278 1058	-17%	Decreasing effect

Table 4 Effect of variation in joint shear reinforcement on joint shear strength

Sr. No	Research groups	Specimen compared	Joint Shear Reinf.		V_{jh} Expt.		Remarks
			Range (Round off)	% Diff.	Range (Round off)	% Diff.	
1.	Tsonos *et al.* (1992)	X1 S1	0.76 2.30	74%	136 143	+5%	Increasing effect
2.	Tsonos *et al.* (1992)	X2 S2	0.76 2.55	70%	151 157	+4%	Increasing effect
3.	Wallace *et al.* (1998)	BCEJ2 BCEJ1	0.98 1.34	27%	790 742	-6%	Decreasing effect
4.	Hamil (2000)	C4ALN0 C4ALN1	0.00 0.35	35%	130 162	+19%	Increasing effect
5.	Hamil (2000)	C6LN0 C6LN1	0.00 0.35	35%	113 118	+4%	Increasing effect
6.	Hamil (2000)	C6LN3 C4ALN5	1.03 1.72	40%	137 185	+26%	Increasing effect
7.	Hamil (2000)	C6LH0 C6LH1	0.00 0.35	35%	164 169	+3%	Increasing effect
8.	Hamil (2000)	C6LH3 C6LH5	1.03 1.72	40%	187 239	+22%	Increasing effect
9.	Hamil (2000)	C4ALH0 C4ALH1	0.00 0.35	35%	201 205	+2%	Increasing effect
10.	Hamil (2000)	C4ALH3 C4ALH5	1.03 1.72	40%	230 240	+4%	Increasing effect
11.	Gencoglu and Eren (2002)	#2 #1	0.00 0.50	50%	114 106	-8%	Decreasing effect

Table 4 Continued

12.	Hegger *et al.* (2003)	RK2 RK3	2.10 2.86	36%	417 402	-4%	Decreasing effect
13.	Murty *et al.* (2003)	Q1 P2	0.00 0.40	40%	316 408	+22%	Increasing effect
14.	Murty *et al.* (2003)	P1 P3	0.00 0.40	40%	345 363	+5%	Increasing effect
15.	Murty *et al.* (2003)	R1 S3	0.00 0.40	40%	350 400	+13%	Increasing effect
16.	Hwang *et al.* (2004)	28-0T0 28-3T4	1.57 1.93	19%	1230 1200	-5%	Decreasing effect
17.	Hwang *et al.* (2004)	70-2T5 70-3T4	1.69 1.93	13%	1274 1284	+1%	Insignificant
18.	Hwang *et al.* (2005)	0T0 3T3	1.22 1.69	28%	1078 1058	-2%	Decreasing effect
19.	Hwang *et al.* (2005)	2T5 3T4	1.69 1.93	13%	1272 1280	+1%	Insignificant
20.	Liu (2006)	RC-6 NZ-7	0.45 0.81	44%	148 147	-1%	Insignificant
21	Idayani (2007)	S1 S3	0.00 0.63	63%	194 224	+13%	Increasing effect
22.	Tsonos (2007)	E1 G1	1.83 1.90	4%	233 240	+2%	Increasing effect
23.	Karayannis *et al.* (2008)	C0 C1	0.26 0.60	56%	239 242	1%	Insignificant
24.	Karayannis *et al.* (2008)	C2 C3	0.77 1.10	30%	239 243	1%	Insignificant
25.	Karayannis *et al.* (2008)	B0 B1	0.00 0.17	17%	227 253	+10%	Increasing effect
26.	Wong and Kuang (2008)	BS-L-450 BS-L-H1	0.00 0.21	21%	377 495	+24%	Increasing effect
27.	Wong and Kuang (2008)	BS-L-V2 BS-L-V4	0.40 0.81	50%	514 533	+4%	Increasing effect
28.	Kaung and Wong (2011)	BS-450 H1T10	0.00 0.10	10%	315 389	+19%	Increasing effect
29.	Kaung and Wong (2011)	BS-600 BS-600H2T8	0.00 0.10	10%	284 360	+21%	Increasing effect
30.	Chun and Shin (2014)	H1.0U H1.0S	0.31 0.46	32%	529 563	+10%	Increasing effect
31.	Chun and Shin (2014)	H0.7U M0.7U	0.31 0.46	32%	611 596	-2%	Insignificant

Table 5 Effect of variation in column axial load ratio on joint shear strength

Sr. No.	Research groups	Specimen compared	Column axial load ratio ($N/A_c f_c$)		V_{jh} Expt.		Remarks
			Range (Round off)	% Diff.	Range (Round off)	% Diff.	
1.	Parker and Bullman (1997)	4a 4b 4c	0.0 0.07 0.14	7% 50%	231 270 333	+14% +19%	Increasing effect

Table 5 Continued

2.	Parker and Bullman (1997)	4d 4e 4f	0.0 0.07 0.14	7% 50%	293 313 358	+6% +12%	Increasing effect
3.	Parker and Bullman (1997)	5a 5b 5c	0.0 0.06 0.12	6% 50%	455 477 474	+4% 0%	Increasing effect
4.	Parker and Bullman (1997)	5e 5f	0.0 0.06	6%	593 648	+8%	Increasing effect
5.	Pantelides *et al.* (2002)	5 4	0.08 0.21	60%	966 1772	+83%	Increasing effect
6.	Masi *et al.* (2008)	T5 T2	0.15 0.30	50%	268 265	-1%	Insignificant
7.	Masi *et al.* (2008)	T1 T6	0.15 0.30	50%	103 103	0%	Insignificant
8.	Masi *et al.* (2008)	T10 T9	0.15 0.30	50%	322 303	-6%	Decreasing effect