STATISTICAL MODELLING AND VALIDATION OF SHEAR STRENGTH OF RC EXTERIOR BEAM – COLUMN JOINTS

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ABSTRACT. Beam – column joints are the key components to ensure the structural integrity of reinforced concrete moment resisting frames when subjected to earthquake loading. One of the main contributing factors to the behaviour of beam – column joints is their shear strength capacity. Therefore, a reliable estimation of joint shear strength is essential for both older and modern RC frame buildings. This research proposes separate joint shear strength models for exterior beam – column joints with (reinforced joints) and without (unreinforced joints) transverse reinforcement in the joint region through statistical analysis, conducted on experimental database collected from published literature. For unreinforced joints, most influencing factors were aspect ratio and beam longitudinal reinforcement ratio. Whereas for reinforced joints, apart from these parameters, joint transverse reinforcement ratio and the presence of transverse beams also contributed to the joint shear strength. The developed equations were validated using experimental tests conducted on four beam – column joint sub assemblages subjected to cyclic loading. Also, a comparison study is performed with existing code approaches. Results show that the proposed shear strength models are more accurate when compared to existing relationships. The developed models can therefore be readily implemented for evaluation of earthquake performances of building frames.

Keywords: Beam – column joints, Shear strength, Reinforced joints, Unreinforced joints, Statistical analysis

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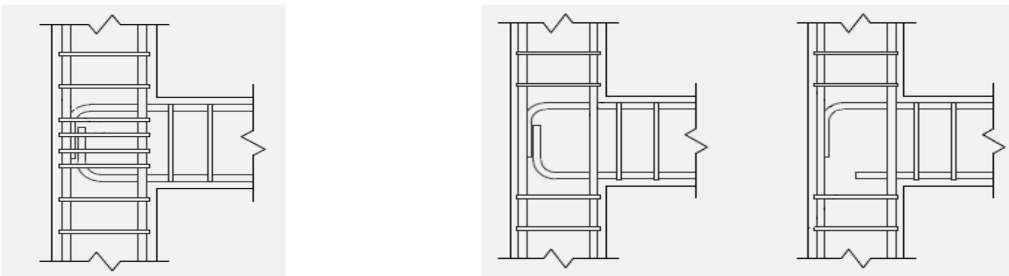
INTRODUCTION

The performance of beam–column joints has been identified as a critical issue since the mid-1960s in the seismic resistance of moment resisting RC frames.Joints are the most crucial zones for effectively transferring forces and moments between columns and beams. Under seismic excitation, high magnitudes of vertical and horizontal shear forces are developed in the beam – column joint region than that experienced by adjoining columns and beams. As a result, beam–column joints are vulnerable to shear mode of failure which is a brittle failure. Such failures must be eliminated through proper design procedures in order to ensure a ductile response of the frame.

The behavior and overall stability of reinforced concrete frames is mainly linked to the shear capacity of beam – column joints. The joint region should behave monolithically while transferring horizontal and vertical shear forces between adjoining members during a seismic event. The total amount of shear force developed in the joint region is bore mainly by the truss mechanism formed by the presence of horizontal stirrups, intermediate bars of column reinforcement and diagonal bars of concrete present between inclined cracks and strut mechanism developed by the concrete part of the joint region [9].

Earthquake reconnaissance reports show that seismic collapses were mainly as a result of joint failure, especially if they are provided with little or no transverse reinforcement or if the longitudinal bars of beams lack sufficient anchorage length into the joint region. Such joints are non-ductile joints (referred to as unreinforced joints) [7]. They respond poorly to seismic action.



(a) Reinforced type b) Unreinforced type

Figure 1 Types of exterior joint based on the presence of transverse reinforcement

Owing to the complexity in behavior of beam – column joints, a widely accepted shear strength model has not yet been developed. Several researchers tried to predict the shear strength of exterior joints, the predictions were not accurate enough, due to its complex behavior. The codal estimates are not predictions of shear capacity, but maximum permissible values of the shear force developing at the joint under the applied load acting at the beam-column joint sub-assemblages. They do not account for many of the factors including the amount of stirrup reinforcement of the joint.

MODELLING THE DATA

Regression analysis is a statistical process for estimating the relationships among dependent and independent variables. Regression analysis helps in understanding how the value of dependent variable changes when any one of the independent variable is varied, keeping the remaining independent variables fixed. The model specification in linear regression is that the dependent variable is expressed as a linear combination of the parameters.

Variables studied include beam and column geometric details, compressive strength of concrete (f_{ck}), yield strength of bars, percentage of longitudinal reinforcement of beam (ρ_b) and column (ρ_c), volumetric joint transverse reinforcement ratio (ρ_j), spacing of joint stirrups (s_j), joint eccentricity (e), column axial load (P), presence of transverse beams (TB) and slab thickness.

Modeling of Uunreinforced Joints

The statistical characteristics such as minimum, maximum, mean and standard deviation of all variables included in the analysis of the data collected were evaluated and are as shown in Table 1.

Table 1 Statistical characteristics of evaluated variables

VARIABLE	NO. OF SAMPLES	MINIMUM	MAXIMUM	MEAN	STANDARD DEVIATION
$\boldsymbol{f_{ck}}$ (MPa)	85	8.30	49.40	30.415	9.443
$\boldsymbol{b_j}$ (mm)	85	120	432	240.812	96.697
h_b/h_c	85	1.00	2.00	1.398	0.206
$\boldsymbol{\rho_b f_{yb}}$(MPa)	85	1.665	12.00	5.869	3.313
V_{exp}(kN)	85	82.957	694.692	264.347	166.747

Multiple linear regression was employed considering all ten parameters on the dataset collected from available literature. The model with the highest R^2 value of 0.968 was selected. Durbin – Watson value of 1.468 indicates the absence of autocorrelation in the residuals. This research uses a 0.05 significance level to determine influencing predictor variables. To examine homoscedasticity, a scatter plot of standardized residuals against standardized predicted values is studied. Fig 2 displays no systematic patterns indicating the absence of heteroscedasticity. Also, Fig 3 shows a histogram which indicates the normal distribution of residuals.

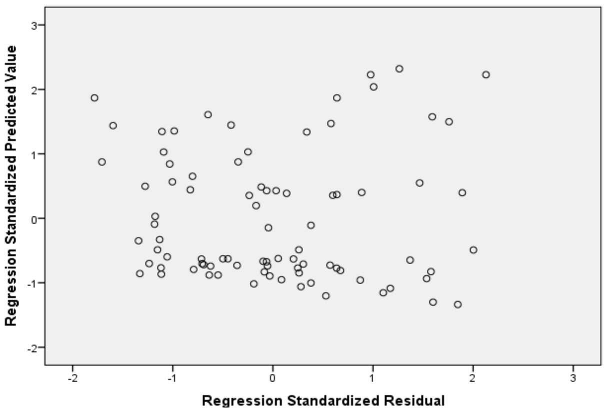


Figure 2 Scatter plot of predicted values versus residuals

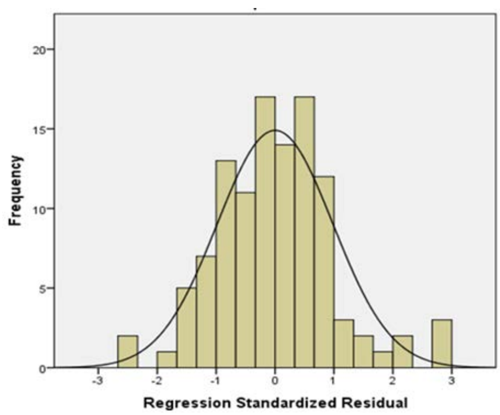


Figure 3 Histogram showing normal distribution of residuals

These examinations provide the appropriateness of the linear regression model. Therefore, the final equation for computing the shear strength of an unreinforced (without transverse reinforcement) beam – column joint is,

$$V_u = 2.833\, f_{ck} + 2.065\, b_j - 49.959\, \frac{h_b}{h_c} + 15.942\, \rho_b f_{yb} - 342.907$$

Where, f_{ck} is the concrete compressive strength

b_j is the effective joint width $b_j = 0.5(b_b + b_c)$

h_b/h_c is the joint aspect ratio

$\rho_b f_{yb}$ is the beam index, $\rho_b = 100A_{st}/b_b d_b$

Modeling of Reinforced Joints

The statistical characteristics such as minimum, maximum, mean and standard deviation of all variables included in the analysis of the data collected were evaluated and are as shown in Table 2.

Table 2 Statistical characteristics of evaluated variables

VARIABLE	NO. OF SAMPLES	MINIMUM	MAXIMUM	MEAN	STANDARD DEVIATION
$\boldsymbol{f_{ck}}$ (MPa)	70	17.70	46.00	30.513	7.512
$\boldsymbol{b_j}$ (mm)	70	130	320	215.814	44.237
h_b/h_c	70	1.00	1.50	1.175	0.178
$\boldsymbol{\rho_b f_{yb}}$ (MPa)	70	2.79	20.50	7.788	3.984
$\boldsymbol{\rho_j f_{yj}}$ (MPa)	70	0.281	6.885	2.279	1.617
TB	70	0	2	0.33	0.675
V_u (kN)	70	111.512	495.10	271.815	85.137

The model with the highest R^2 value of 0.970 was selected. Durbin – Watson value of 1.858 indicates the absence of autocorrelation in the residuals. A 0.05 significance level was used to determine influencing predictor variables. Scatter plot of standardized residuals against standardized predicted values displayed no systematic patterns indicating the absence of heteroscedasticity. Also, histogram indicated the normal distribution of residuals.

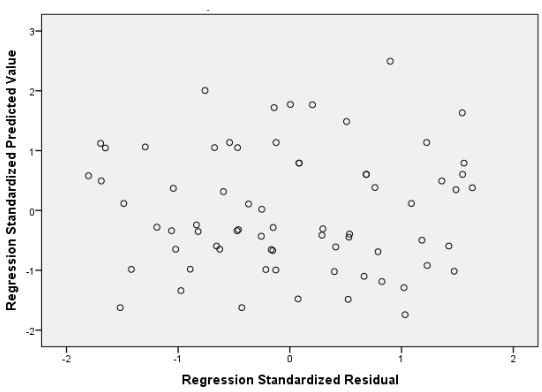


Figure 4 Scatter plot of predicted values versus residuals

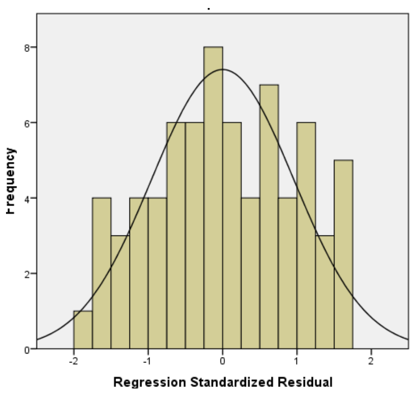


Figure 5 Histogram showing normal distribution of residuals

These examinations provide the appropriateness of the linear regression model. Therefore, the final equation for computing the shear strength of a reinforced (with transverse reinforcement) beam – column joint is,

$$V_u = 3.171\, f_{ck} + 2.165\, b_j - 109.555\, \frac{h_b}{h_c} + 7.41\, \rho_b f_{yb} + 5.912\, \rho_j f_{yj} + 29.456\, TB - 244.362$$

Where, f_{ck} is the concrete compressive strength

b_j is the effective joint width $b_j = 0.5(b_b + b_c)$

h_b/h_c is the joint aspect ratio

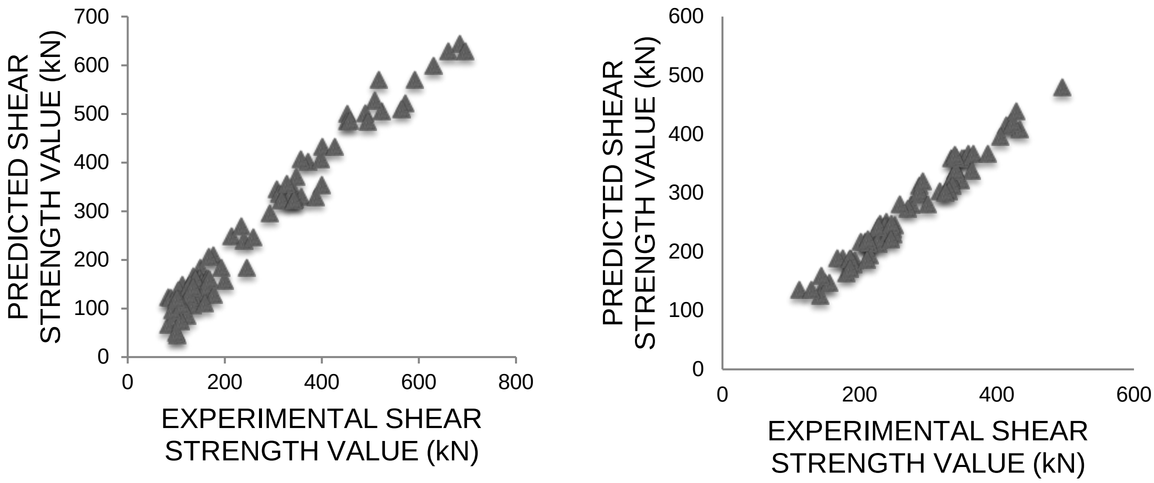
$\rho_b f_{yb}$ is the beam index, $\rho_b = 100A_{st}/b_b d_b$

$\rho_j f_{yj}$ is the joint index, $\rho_j = \frac{area\ of\ one\ hoop\ layer\ of\ transverse\ reinforcement}{b_c * spacing\ of\ ties}$

f_{yj} is the yield strength of transverse reinforcement

TB is the number of transverse beams

Variation of experimental and predicted values of shear strength



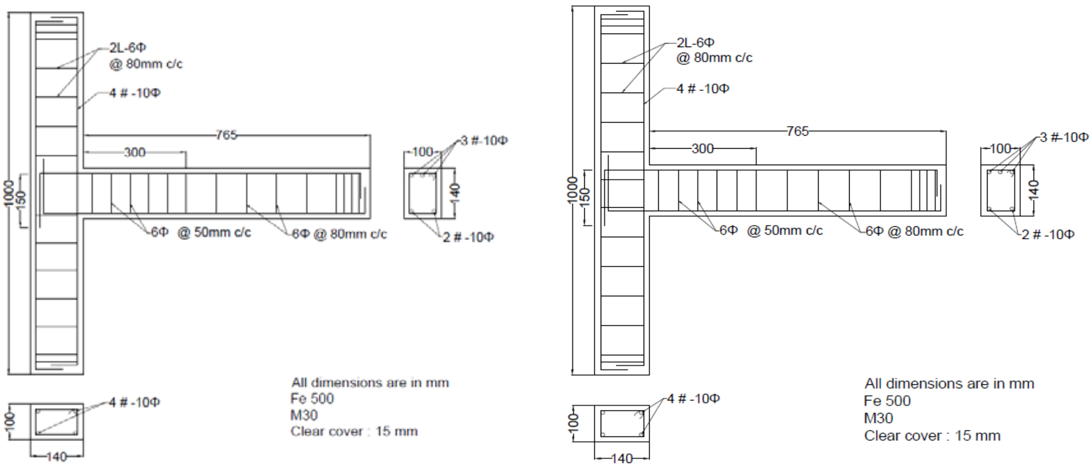
(a) Unreinforced joints (b) Reinforced joints

Figure 6 Variation of experimental and predicted values of shear strength

EXPERIMENTAL PROGRAMME

M30 mix was adopted. Mix design was done as per IS 10262:1982 and the mix proportion was obtained as 1: 1.64: 2.63. Water cement ratio of 0.45 was used. Average 28 day compressive strength of 15cm x 15cm x 15cm concrete cubes was obtained as 38.44 MPa. The beam – column joints were designed as per IS 456:2000. M30 mix was used and HYSD bars of 10mm and 6mm of yield strength 500 N/mm^2 were used as reinforcement. The longitudinal and transverse reinforcement in the beam and transverse reinforcement of column was increased to prevent early degradation of the beam and column, forcing a shear mode of failure in the joint prior to or following the beam yielding.

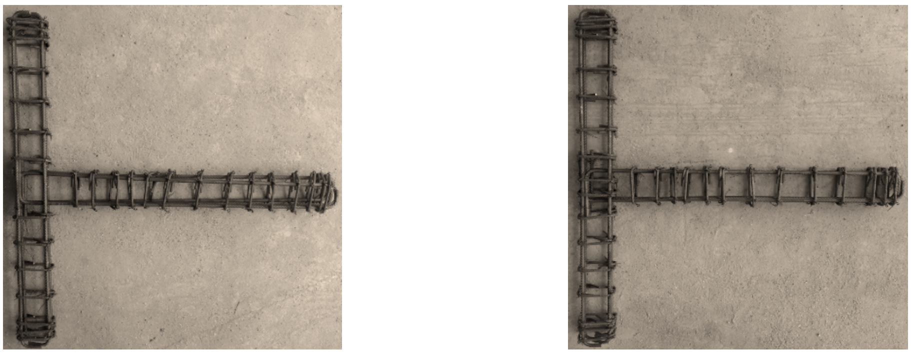
The columns were of size 100x140 mm and of length 1000 mm. Beams were of cross – section 100x140 mm and of length 765 mm. For reinforced type specimens, the column transverse steel was provided continuously throughout the joint region as shown in Figure 7(b). Whereas, for the unreinforced types, transverse steel was absent in the joint region as shown in Figure 7(a). However, the design still followed the weak beam strong column concept. The spacing of transverse reinforcement for both beam and column ends was reduced to give adequate strength where forces are applied during the test programme. The 1:3 scale down reinforcement details of beam – column joint designed as per IS 456:2000 are shown below.



(a) Unreinforced type (SP1 & SP2) (b) Reinforced type (SP3 & SP4)

Figure 7 Reinforcement details of beam – column joints

Total of four beam – column joint assemblages were cast, consisting two joints without transverse reinforcement and two with transverse reinforcement in the joint region. The reinforcement cages prepared for casting is shown in Figure 8. Specimens were remoulded after 24 hours and subjected to water curing for 28 days.



(a) Unreinforced type (SP1 & SP2) (b) Reinforced type (SP3 & SP4)

Figure 8 Reinforcement cage of specimen

Prepared specimens were subjected to quasi static reverse cyclic loading, loading being applied at the beam end. The test was load controlled with a load increment of 1kN/cycle. Load values were recorded using load cells with a least count of 1kN. The experimental laboratory setup is shown in Figure 9.

Figure 9 Laboratory test setup

Test Results and Discussion

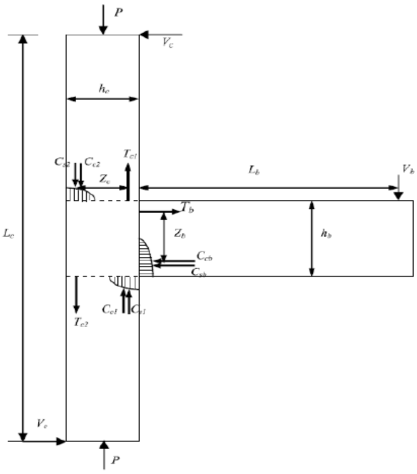
The joint types which had transverse reinforcement in the joint region (reinforced joints) had greater load carrying capacities when compared to poorly detailed ones (unreinforced joints). SP1 and SP2 failed at an ultimate load of 10 kN whereas SP3 and SP4 failed at an ultimate load of 11 kN. First crack developed at the seventh cycle of loading at a load of 7kNin the positive loading direction for SP1 and SP2. Diagonal cracks developed in the joint region in the initial stages and flexural cracks formed in the beam near to the beam – column interface at a later stage. These specimen types clearly exhibited joint shear failure with beam yielding carrying a maximum load up to 10 kN. For SP3 and SP4 cracks developed during the eighth loading cycle. Hairline flexural cracks developed at the joint interface depicting the beginning of yielding of beam longitudinal reinforcement. As load increased diagonal shear cracks developed and propagated at the joint region up to a load of 11 kN. SP3 and SP4 exhibited joint shear failure after yielding of beam reinforcement. Crack patterns of all specimens are shown in Figure 10.

(a) SP1 (b) SP2 (c) SP3 (d) SP4

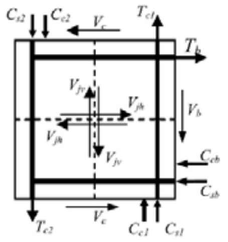
Figure 10 Crack patterns of joint specimens

Validation

Shear strength is computed and compared with the predicted results. Also, the predicted values are compared with various codal approaches. Figure 11 shows the mechanics of an exterior joint when subjected to seismic forces. Considering joint equilibrium, the tested joint shear strength for exterior joints equals the tensile strength of beam longitudinal tension reinforcement at joint(T_b) deducted by the column shear force (V_c).



(a) External actions and forces in beams and columns



(b) horizontal and vertical joint shear

Figure 11 Mechanics of exterior joint under seismic actions [8]

The shear strength of joint can be computed as,

$$\boldsymbol{V_{jh} = V_b\left(\frac{L_b}{z_b} - \frac{(L_b + 0.5\,h_c)}{L_c}\right)}$$

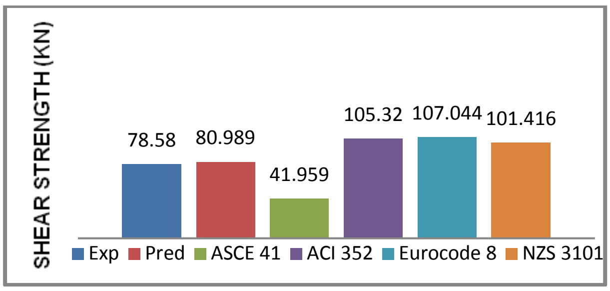
Where, z_b is the lever arm, approximated as $d_b - d'_b$.

Table 3 compares the experimental values of shear strengths with those predicted by SPSS software.

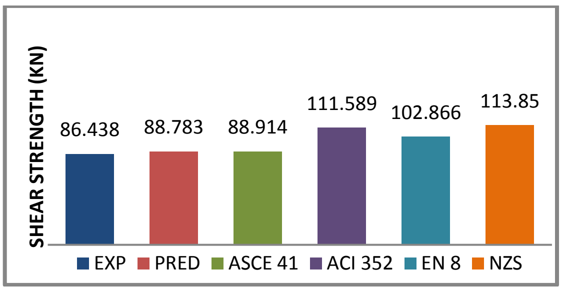
Table 3 Comparison of shear strength of specimens

SPECIMEN	ULTIMATE LOAD (kN)	EXPERIMENTAL SHEAR STRENGTH, V_{exp} (kN)	PREDICTED SHEAR STRENGTH, V_{pred} (kN)	$\frac{\mathbf{V_{pred}}}{\mathbf{V_{exp}}}$
SP1	10	78.58	80.989	1.02
SP2	10	78.58	80.989	1.02
SP3	11	86.438	88.783	1.027
SP4	11	86.438	88.783	1.027

Figure 12(a) presents a comparison of shear strength values of beam – column joint specimens without transverse reinforcement (unreinforced joints). It can be seen that the SPSS model predicted a very close value as the R squared value of the model was significantly high. ASCE 41 has highly underestimated the shear strength by about 50 %. The ACI 352R-02 and NZS 3101-06 equations predicted a fairly better value. Whereas the Euro Code 8 has greatly overestimated the joint shear strength.



(a) SP1 & SP2



(b) SP3 and SP4

Figure 12 Comparison of shear strength values

For the reinforced specimens, i.e., beam – column joint specimens with transverse reinforcement in the joint region, the SPSS model predicted a very close value. It can be noted that ASCE 41 for these type of joints gave a shear strength value close enough. Both the ACI 352 and NZS 3101-06 equations showed a similar trend as for unreinforced joints. Euro Code 8 has still estimated a fairly high value. The same has been represented in Figure 12 (b).

CONCLUDING REMARKS

Shear strength models were developed for exterior beam – column joints with and without transverse reinforcement in the joint region. Multi linear regression was performed on the collected dataset using the SPSS software. The developed models were validated through experimental tests on four beam – column joint sub assemblages and also compared with various codal approaches.

Joint shear strength model for beam – column joints without transverse reinforcement in the joint region (referred to as unreinforced joints) had an R square value of 0.968 and was able to predict the experimental value of shear strength with a percentage error of 3 %. The joint shear strength model for beam – column joints with transverse reinforcement in the joint region (referred to as reinforced joints) had an R square value of 0.97 and was able to predict the experimental value of shear strength with a lower percentage error of 2.7 %.

Reinforced specimens SP3 and SP4 showed an improved load carrying capacity when compared to unreinforced specimens, SP1 and SP2 due to the presence of transverse reinforcement in the joint region, which increased the shear capacity of joint. All the specimens failed in shear accompanied by yielding of longitudinal beam reinforcement.

ACI 352-02, Euro Code 8 and NZS 3101-06 overestimated the joint shear strength values. ASCE 41 underestimated shear strength of unreinforced specimens and gave a fairly better value for reinforced specimens.

Based on the above inferences, it can be concluded that the both the developed models can be successfully used to predict the shear strength of exterior RC beam – column joints. However, it is to be noted that the results of SPSS modelling would be dependable only if large experimental dataset tested under similar laboratory procedures is available.

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