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Seismic resistance and mechanical behaviour of exterior beam-column joints with crossed inclined bars

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Abstract. Attempts at improving beam-column joint performance has resulted in non-conventional ways of reinforcement such as the use of the crossed inclined bars in the joint area. Despite the wide accumulation of test data, the influence of the crossed inclined bars on the shear strength of the cyclically loaded exterior beam-column joints has not yet been quantified and incorporated into code recommendations. In this study, the investigation of joints has been pursued on two different fronts. In the first approach, the parameters that influence the behaviour of the cyclically loaded beam-column joints are investigated. Several parametric studies are carried out to explore the shear resisting mechanisms of cyclically loaded beam-column joints using an experimental database consisting of a large number of joint tests. In the second approach, the mechanical behaviour of joints is investigated and the equations for the principal tensile strain and the average shear stress are derived from joint mechanics. It is apparent that the predictions of these two approaches agree well with each other. A design equation that predicts the shear strength of the cyclically loaded exterior beam-column joints is proposed. The design equation proposed has three major differences from the previously suggested design equations. First, the influence of the bond conditions on the joint shear strength is considered. Second, the equation takes the influence of the shear transfer mechanisms of the crossed inclined bars into account and, third, the equation is applicable on joints with high concrete cylinder strength. The proposed equation is compared with the predictions of the other design equations. It is apparent that the proposed design equation predicts the joint shear strength accurately and is an improvement on the existing code recommendations.

Key words: mechanical behaviour; deformation; earthquake resistant structures; crossed inclined bars; cyclic loads; joints; shear properties; strut and tie models; anchorage; bond (concrete to reinforcement); beams; columns; reinforced concrete; connections; structural analysis; shear strength.

1. Introduction

The Kocaeli and Duzce earthquakes in Turkey showed that, even when the beams and columns in reinforced concrete multi-storey residences are only slightly damaged after the main shock or aftershocks, the integrity of a building was threatened if the joint, where these members connected failed, as mentioned in previous papers of the author (Bakir 2003a, 2003b, Bakir and Boduroglu 2002a, 2002b, 2002c). The author has investigated the shear resisting mechanisms and the factors influencing the failure modes of monotonically loaded beam-column joints in companion papers (Bakir and Boduroglu 2002d, 2002e). The design of multi-storey structures for gravity loads causes

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no serious problems. Nevertheless, due to the unexpected nature of earthquakes, many aspects of the seismic design of structures still need to be investigated. The aim of this paper is to investigate the shear resisting mechanisms of cyclically loaded exterior beam-column joints and the influence of the crossed inclined bars on the shear strength and the shear resisting mechanisms.

There are still differences in codes regarding the design of beam-column joints. The New Zealand Design Code (1995) is based on the assumption that there are two types of shear resisting mechanisms in beam-column joints as shown in Figs. 1(a) and 1(b) and as first suggested by Paulay (1975). These are the diagonal strut mechanism and the truss mechanism. The strut mechanism transfers shear forces via a diagonal concrete strut which sustains compression only and is assumed to be inclined at an angle close to that of the potential corner-to-corner failure plane. The truss mechanism consists of the contribution of the horizontal reinforcement inside the joint core. The New Zealand Code, which takes into account the influence of both the strut mechanism and the

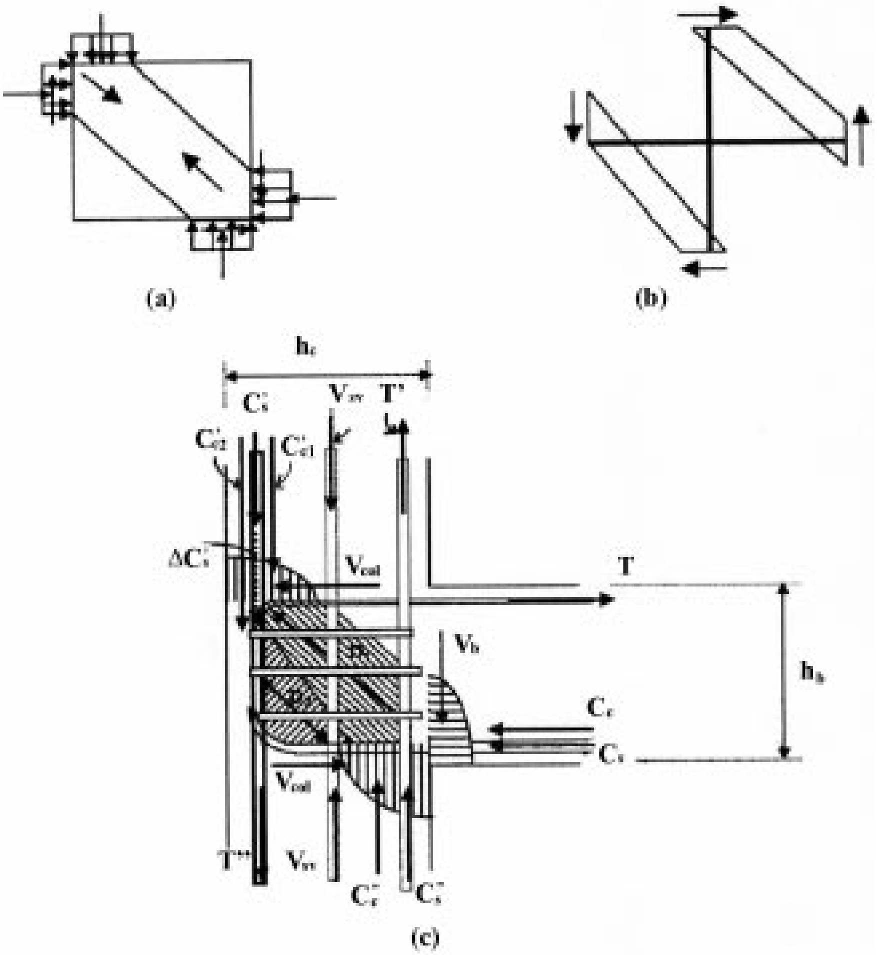


Fig. 1 The diagonal strut mechanism; (a) and truss mechanism, (b) in interior beam-column joints, (c) idealised stress paths in exterior joints

truss mechanism, recommends that the beam bars should be properly anchored within the joint area in order to have a workable truss mechanism. This is because the truss mechanism can only exist when there is good bond transfer in the beam bars. Thus, the bar size is strictly limited in the New Zealand Code relative to the joint dimensions. The New Zealand Code advocates that the bond deterioration of beam bars within a joint is undesirable because pinching in the hysteresis curves increases after bond deterioration, the compressive stresses in the diagonal strut increase and the beam deformations increase due to the loss of bond. In addition to this requirement, the New Zealand Code necessitates large amounts of vertical and horizontal shear reinforcement to be used in the joint area to equilibrate the truss mechanism, as it is based on the assumption that joint shear strength is considerably increased by the provision of vertical and horizontal shear reinforcement. The United States (1995, 1998) and the Japanese Design Codes (1990), on the other hand, are based on the assumption that joint shear is resisted entirely by the direct strut mechanism and the stirrups are only necessary to confine the joint core. Further complicating the problem, all the design recommendations in codes today, are based on tests of joints of normal strength concrete with concrete compressive strengths between 20 and 50 MPa. However, in recent years, high strength concrete is frequently used in the construction industry. Thus, it is more appropriate to alter the design recommendations so as to also cover high strength reinforced concrete structures using the available recent tests on joints with high strength concrete specimens.

2. Previously suggested models and code recommendations for cyclically loaded exterior beam column joints

There is already a large amount of experimental data available, related to the behaviour of joints. Nevertheless, seismic design provisions for beam-column joints are still controversial, despite the great deal of research that has been conducted throughout the years. In the following sections, previously suggested models and design recommendations for cyclically loaded beam-column joints will be reviewed.

2.1 Model of Paulay

The shear resisting mechanisms of interior beam-column joints as shown in Figs. 1(a) and 1(b) are different from the shear resisting mechanisms of exterior beam-column joints as shown in Fig. 1(c) according to Paulay (Paulay and Priestley 1992). Because in an exterior joint only one beam frames into a column, the shear load input into a joint will generally be less than that encountered with interior joints. As in the case of interior joints, shear forces, both in horizontal and vertical directions, can be sustained by a diagonal concrete compression field together with horizontal and vertical joint shear reinforcement. A major diagonal strut, sustaining a compression force D_1 can develop at the bend of the top beam bars. The horizontal component of this strut is the tension force T, assumed to be developed at the beginning of the hook, less the column shear force V_{col}. The vertical component of the strut consists of the concrete force C_{c1}', a part of the compression force on the column reinforcement ΔC_s' which is transmitted by bond near the bend of the beam bar, and the compression force originating from the anchorage of the intermediate column bar, acting as vertical shear reinforcement. At the lower and inner abutment of the strut, the horizontal component necessary to support the diagonal force D_1, will consist of part of the beam concrete compression force C_{c1}, reduced by the shear force V_{col}. The remainder of the horizontal force, V_{sh}, must be supplied by horizontal ties. To support the diagonal strut D_1 near the beam, horizontal ties are required. To absorb the tension forces in these ties at the outer face of the joint, another diagonal compression field D_2, needs to be developed. The associated horizontal forces are the bond forces transmitted from the bottom beam bars, C_s, and the remainder of the beam flexural compression force, C_c-C_{c1}. The vertical components at the upper end of the strut D_2, originate from bond forces in the outer column bars, T'' and $C_s' - \Delta C_s'$, and from some column compression force C_{c2}' entering the joint core via the cover concrete (Paulay and Park 1984).

It should be noted however that, due to the interchange of forces between concrete and steel, load transfer within the joint is inseparable from the mechanisms of bond. When a plastic hinge develops adjacent to the joint, with the beam bars entering also the strain hardening range, yield penetration into the joint core and consequent drastic bond deterioration is unavoidable. As a result, after a few cycles of inelastic loading, significant anchorage can be provided only by the hook. Serious bond deterioration in interior joints results in significant loss of stiffness and energy dissipation. Anchorage failure of beam bars in exterior joints, on the other hand, results in complete failure (Paulay and Priestley 1992, Paulay and Park 1984, Paulay *et al*. 1978).

2.2 Model of Tsonos

Tsonos (1997, 1999, 2000, 2001a, 2001b) has carried out extensive experimental and theoretical work on beam-column joints and has suggested a model which is based on the assumption that both the strut and the truss mechanisms depend on the core concrete strength. Thus, the ultimate concrete strength of the joint core under compression/tension also gives the ultimate strength of the connection. From the vertical and horizontal equilibrium, Eqs. (1) and (2) are obtained in the model of Tsonos as shown in Fig. 2.

$$D_{cy} + (T_1 + \dots + T_4 + D_{vy}) = D_{cy} + D_{sy} = V_{jv} \tag{1}$$

$$D_{cx} + (D_{1x} + \dots + D_{vx}) = V_{jh} \tag{2}$$

The vertical normal compressive stress σ and the shear stress τ uniformly distributed over the whole section are given by the Eqs. (3), (4).

$$\sigma = \frac{D_{cy} + D_{sy}}{h_c' \times b_c'} = \frac{V_{jv}}{h_c' \times b_c'} \tag{3}$$

$$\tau = \frac{V_{jh}}{h_c' \times b_c'} \tag{4}$$

The relationship between the average normal compressive stress σ and the average shear stress τ are shown in Eq. (5).

$$\sigma = \frac{V_{jv}}{V_{jh}} \tau \tag{5}$$

where

$$\frac{V_{jv}}{V_{jh}} = \frac{h_b}{h_c} = \alpha \tag{6}$$

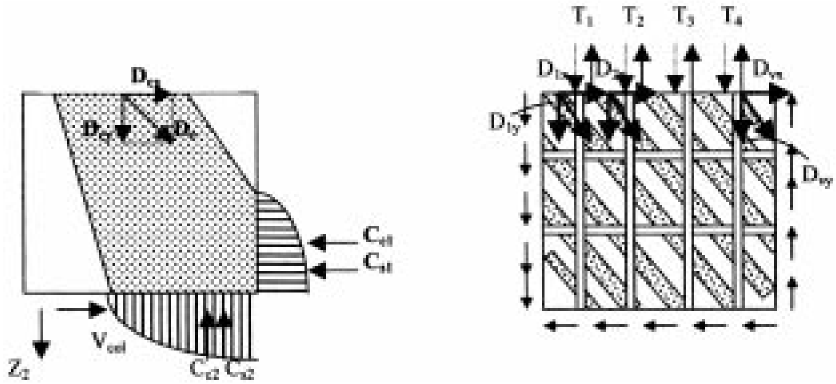


Fig. 2 Forces acting in the joint concrete core from the strut and truss mechanism (Tsonos 1997)

From Mohr's circle,

$$\sigma_1 = -\frac{\sigma}{2} + \frac{\sigma}{2}\sqrt{1 + \frac{4\tau^2}{\sigma^2}} \tag{7a}$$

$$\sigma_2 = \frac{\sigma}{2} + \frac{\sigma}{2}\sqrt{1 + \frac{4\tau^2}{\sigma^2}} \tag{7b}$$

Tegos (1984) has suggested Eq. (8) for representing concrete biaxial strength curve by a parabola of 5^{th} degree.

$$10\frac{\sigma_1}{f_c'} + \left(\frac{\sigma_{II}}{f_c'}\right)^5 = 1 \tag{8}$$

Substituting Eqs. (5), (6) and (7) into Eq. (8) and using $\tau = \gamma\sqrt{f_c'}$, where f_c' is the compressive strength of concrete, the following expression is proposed by Tsonos (1997) for predicting the joint ultimate strength:

$$\left[\frac{\alpha\gamma}{2\sqrt{f_c'}}\left(1 + \sqrt{1 + \frac{4}{\alpha^2}}\right)\right]^5 + \frac{5\alpha\gamma}{\sqrt{f_c'}}\left(\sqrt{1 + \frac{4}{\alpha^2}} - 1\right) = 1 \tag{9}$$

2.3 The design recommendations of the AIJ Guidelines

In the AIJ Guidelines, the joint shear strength is given by

$$V_J = kf_c b_j D_j \tag{10}$$

The effective joint width b_j is taken as

$$b_j = b_b + b_{a1} + b_{a2} \tag{11}$$

The AIJ Code is based on the assumption that the joint shear strength is not significantly influenced by stirrups. In spite of this, the guidelines require some transverse reinforcement to be provided as the joint stirrups are assumed to increase the joint ductility by confining the cracked core concrete and because they increase the bond conditions for the column bars. The AIJ Guidelines require that the minimum amount of stirrup ratio in joints should be 0.002, and the following criterion should be satisfied:

$$\rho_{stir} > 0.003\frac{V_j}{V_{ju}} \tag{12}$$

$$\rho_{stir} = \frac{A_s}{(7db_c/8)} \tag{13}$$

2.4 The ACI-ASCE Committee 352 Recommendations

The recommendations of the ACI-ASCE Committee 352 are based on the assumption that the joints should be stronger than the incoming beams, and failure should occur by hinging in the incoming beams rather than in the joint itself. The cyclically loaded beam-column joints are classified as Type 2 joints in the ACI-ASCE Committee 352 Recommendations, and the design shear force is calculated based on the yield capacity of the beam longitudinal reinforcement as shown in Eq. (14).

$$V_j = A_s f_y - V_{col} \tag{14}$$

The horizontal joint shear force V_{jhor} should not exceed a maximum value, taken as:

$$V_{jhor} = V_u = \phi\gamma\sqrt{f_c}b_jh_c \tag{15}$$

$\sqrt{f_c}$ is in units of psi.
ϕ is taken as 0.85.
γ is 15 for Type 2 exterior beam-column joints.
If f_c is defined in units of MPa, the values of γ should be multiplied by 0.083.

3. Method

In this paper, the parameters that influence the behaviour of the cyclically loaded beam-column joints are explored. An experimental database consisting of a large number of cyclically loaded exterior beam-column joints are used to investigate the shear resisting mechanisms of joints, as shown in Table 1. The tests in Table 1 are all exterior beam-column joint tests by different researchers (Megget and Park 1974, Paulay and Scarpas 1981, Ehsani and Wight 1985, Alameddine 1990, Kaku and Akasuka 1991, Fuji and Morita 1991, Tsonos *et al.* 1992, Tsonos 1997). The specimens are chosen according to the following criteria:

1. Specimens with slabs, transverse beams, beam bars with plate anchorage, beam bottom bars bent downward into the lower column, or specimens that have eccentricity between column and beam axis are omitted.
2. Only specimens failing in a joint or a beam adjacent to a column are considered; specimens with a relocated beam hinge or those that exhibited column or anchorage failures are omitted.

Table 1(a) The experimental database

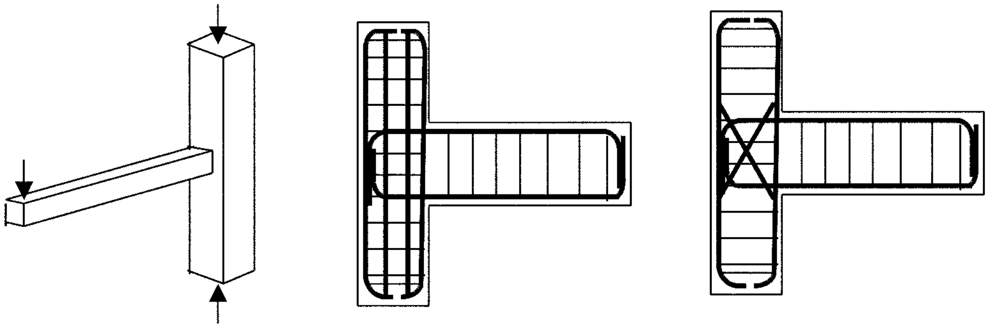
No	Researcher	Specimen	f_c (MPa)	h_b	h_c	b_b	b_c	V_{cal}/V_{exp}	V_{aci}/V_{exp}
1	Kaku&Asakusa (1991)	2	41.7	220	220	160	220	0.74	0.85
2	"	3	41.7	220	220	160	220	0.85	0.97
3	"	5	36.7	220	220	160	220	0.72	0.94
4	"	6	40.4	220	220	160	220	0.82	0.99
5	"	8	41.2	220	220	160	220	0.74	0.85
6	"	9	40.6	220	220	160	220	0.76	0.88
7	"	11	41.9	220	220	160	220	0.76	0.90
8	"	12	35.1	220	220	160	220	0.75	1.00
9	"	13	46.4	220	220	160	220	0.94	0.99
10	"	14	41	220	220	160	220	0.77	0.92
11	"	15	39.7	220	220	160	220	0.73	0.90
12	Fuji&Morita (1991)	B2	30.6	250	220	220	220	0.82	1.74
13	"	B3	30.6	250	220	220	220	0.65	3.72
14	"	B4	30.6	250	220	220	220	0.67	3.54
15	Ehsani (1985)	1B	33.6	480.06	299.72	259.08	300	0.64	1.35
16	"	3B	40.9	480	300	259	300	0.73	1.26
17	"	4B	44.6	439	300	259	300	0.72	1.21
18	"	5B	24.3	480.06	340.36	299.72	340.36	0.66	1.53
19	Scarpas&Paulay (1981)	1	22.6	610	457	356	457	0.81	0.76
20	"	2	22.5	610	457	356	457	0.58	0.85
21	"	3	26.9	610	457	356	457	0.84	0.75
22	Alameddine (1990)	LL8	55.84	508	356	317.5	356	0.83	2.55
23	"	LH8	55.84	508	356	317.5	356	0.92	2.61
24	"	HH8	55.84	508	356	317.5	356	0.78	2.74
25	"	LL11	73.77	508	356	317.5	356	1.12	2.77
26	"	LH11	73.77	508	356	317.5	356	0.99	2.31
27	"	HH11	73.77	508	356	317.5	356	0.90	2.59
28	"	HH14	93.77	508	356	317.5	356	1.05	2.60
29	Tsonos (1992)	S3	18.96	300	200	200	200	0.91	1.19
30	"	X3	26.98	300	200	200	200	1.26	1.05
31	"	S4	20.96	300	200	200	200	0.83	1.90
32	"	X4	16.97	300	200	200	200	0.99	1.78
33	"	S5	24.96	300	200	200	200	0.77	1.70
34	"	X5	22.01	300	200	200	200	0.82	1.45
35	"	S6	32.96	300	200	200	200	0.86	1.63
36	"	X6	26.98	300	200	200	200	0.78	1.27
37	"	S61	28.96	300	200	200	200	0.75	1.52
38	"	X7	18.01	300	200	200	200	0.71	1.46
39	"	X8	18.98	300	200	200	200	0.78	1.67
40	"	P1	16	300	200	200	200	0.71	2.50
41	"	Y1	22.96	300	200	200	200	0.76	2.18
42	"	O1	19.99	300	200	200	200	0.87	2.14
43	"	F2	23.99	300	200	200	200	0.88	1.98

Table 1(a) Continued

No	Researcher	Specimen	f_c (MPa)	h_b	h_c	b_b	b_c	V_{cal}/V_{exp}	V_{aci}/V_{exp}
44	Tsonos (1997)	NC1	18.96	300	200	200	200	0.90	1.49
45	"	NCZ1	21.99	300	200	200	200	0.96	1.46
46	"	N1	20.96	300	200	200	200	0.82	1.20
47	"	NZ1	19.99	300	200	200	200	0.85	1.27
48	"	N2	32.96	300	200	200	200	0.79	1.47
49	"	NZ2	19.99	300	200	200	200	0.87	2.14
50	"	NZO2	15.99	300	200	200	200	0.72	2.49
51	"	NZM2	28.96	300	200	200	200	0.76	1.52
52	"	N3	24.96	300	200	200	200	0.76	1.66
53	"	NZ3	23.99	300	200	200	200	0.78	1.72
54	"	A2	31.03	300	200	200	200	1.10	1.35
55	"	A3	25.99	300	200	200	200	1.01	1.41

Table 1(b) The average and standard deviation values for the proposed equation and the ACI Code Recommendations

	Proposed equation	ACI-ASCE Committee 352 Recommendations
Average $V_{predicted}/V_{actual}$	0.82	1.63
Standard deviation	0.13	0.7



(a) (b) (c)

Fig. 3(a) A typical specimen in the experimental database, (b) Joints without crossed inclined bars, (c) Joints with crossed inclined bars

3. Specimens with flexural over-strength ratios higher than 3 and less than 1 are not included in the experimental database.

The typical specimen type used in the experimental database is shown in Fig. 3(a). The database contains both the joints with and without the crossed inclined bars, as shown in Figs. 3(b) and 3(c).

In analysing the tests, multiple linear regression analysis is used. The possibility of non-linearity is also investigated for each regressor by controlling the residual plots, as will be explained later. For p independent variables, the model for multiple linear regression is:

$$Y = \beta_O + \beta_1 X_1 + \beta_2 X_2 + \ldots + \beta_p X_p + \varepsilon \tag{16}$$

which can be written in matrix notation as;

$$[Y] = [X][\beta] + [\varepsilon] \tag{17}$$

where

$$[Y] = \begin{bmatrix} Y_1 \\ Y_2 \\ Y_3 \\ . \\ . \\ . \\ Y_n \end{bmatrix}, \quad [X] = \begin{bmatrix} 1 & X_{11} & X_{12} & \cdots & X_{1p} \\ 1 & X_{21} & X_{22} & \cdots & X_{2p} \\ . & . & . & \cdots & . \\ . & . & . & \cdots & . \\ . & . & . & \cdots & . \\ 1 & X_{n1} & X_{n2} & \cdots & X_{np} \end{bmatrix}, \quad \beta = \begin{bmatrix} \beta_o \\ \beta_1 \\ \beta_2 \\ . \\ . \\ . \\ \beta_p \end{bmatrix} \tag{18}$$

The use of more than one variable, however, can result in many difficulties and, as stated by Snedecor and Cochran (1980): 'Multiple linear regression is a complex subject'. The complexity arises due to the following facts:

1. It is particularly difficult to select variables and decide which methods to use to obtain the best subset of variables.
2. It is very difficult to interpret the results, more specifically, the regression coefficients.
3. It is difficult to decide whether to use least squares or robust regression methods as there may be outliers or leverage points in the data.

Naturally, hand calculations are impractical when p is greater than 2 and the use of computers is necessary. The objective in multiple linear regression analysis is to minimise

$$\sum_{i=1}^{n} \varepsilon_i^2 = \sum (Y_i - \beta_0 - \beta_1 X_{i1} - \beta_2 X_{i2} - \ldots - \beta_p X_{ip})^2 \tag{19}$$

The above equation is differentiated with respect to β_0, β_1, β_2, β_3,..............., β_p to produce $(p + 1)$ equations. The coefficients can be expressed in matrix form as shown below:

$$[\beta] = (X^T X)^{-1} X^T Y \tag{20}$$

The general form of the ANOVA tables are given in Table 2. A small P value indicates that the

Table 2 The explanation of the ANOVA Table

Source	Degrees of freedom	Sum of squares SS	Mean square MS	F
Regression	p	$[\beta]^T [X]^T [Y]$	$(1/p)[\beta]^T [X]^T [Y]$	Msreg/MSres
Residual	$n - p - 1$	$[Y]^T Y - [\beta]^T [X]^T [Y]$	$(1/(n - p - 1))\ [Y]^T Y - [\beta]^T [X]^T [Y]$	
Total	$n - 1$	$[Y]^T [Y]$		

regression equation is of high value in predicting Y. The R^2 value is given as;

$$R^2 = \frac{SS_{reg}}{SS_{total}} \tag{21}$$

The closer the R^2 value to 1, the better the variability in Y is explained by the regression equation. In multiple linear regression however, R^2 will always increase when variables are added to the model. A better measure is R^2_{adj} which is expressed as shown in Eq. (22).

$$R^2_{adj} = 1 - (1 - R^2)\left(\frac{n-1}{n-p}\right) \tag{22}$$

A confidence interval will be constructed for each parameter which has a form as Eq. (23):

$$\hat{\beta}_i \overset{+}{-} t_{\alpha/2,\, n-p-1}\sqrt{MS_{res}}(c_{ii})^{1/2} \tag{23}$$

The next step is to determine the most appropriate subset. Various methods have been used for many years for this purpose. Forward selection is one of them. With this method, a subset is obtained by sequentially adding variables one at a time until the marginal contribution of a regressor is estimated as insignificant by the F test. The opposite of forward selection is backward elimination. With this method, the starting point is the full equation with all of the available variables. A variable is then deleted if the t test indicates that its marginal contribution is not significant. Stepwise regression is basically a combination of these two methods in that it allows for both the addition and deletion of variables. After an F test indicates that a variable should be added to the equation, subsequent F tests are carried out to determine whether any of the other variables in the equation have become unnecessary and should thus be eliminated. Stepwise multiple regression analysis is used in this study to obtain the best subset of regressors. A large number of subsets are tried for the analysis. In choosing the best subset, the following criteria is accepted:

1. R^2 value should be very close to 1.
2. If the above criterion is true, then $R_{adj}{}^2$ should be checked as given by Eq. (22). $R_{adj}{}^2$ is a better measure because, as variables are added to a model, R^2 will certainly increase. However, R_{adj} is not affected by this, because as apparent from Eq. (22), n-p becomes smaller as p increases and thus 1-R^2 must decrease at a faster rate so that $R_{adj}{}^2$ increases.
3. A significance level of 0.05 is used, and it is checked whether the hypothesis that $\beta = 0$ will be rejected. The F value should be considerably greater than the minimum value needed to reject Ho: $\beta = 0$. In this study, it is accepted that the F value should be at least four times the minimum value needed to reject Ho: $\beta = 0$. Any subset of regressors which have a ratio less than 4 are not accepted.
4. If F is greater than the minimum value needed to reject H_o, the t tests of the coefficients are checked. The tests can indicate whether any of the regressors are irrelevant.
5. The confidence intervals are built with the assumption that error terms have a normal distribution and are independent. The assumptions of independent errors and constant error variance are checked by plotting the errors against the particular regressor. The spread of residuals should be reasonably constant over X, and the residuals should not illustrate an obvious pattern. Plots illustrating non-random residuals can imply that regression is inappropriately used for time series data. Plots illustrating a horse shoe shaped, non-constant residual variance, can indicate that a non-linear relationship exists between the regressor and Y.

4. The analysis of the tests

The experimental database is analysed in order to investigate the influence of several different parameters on the joint shear strength. As apparent from Fig. 4(a), for a constant concrete cylinder strength, the joint shear strength is independent of the column axial stress. However, for a constant column axial stress, as the concrete cylinder strength increases, the joint shear strength increases substantially and the highest joint shear strength is obtained when the joints have concrete cylinder strengths higher than 55 MPa. The parametric study in Fig. 4(b) also shows that the joint shear strength is independent of the column axial stress. Other researchers have also suggested that the joint shear strength in exterior beam-column joints is independent of the column axial stress (Vollum 1998, Pantazopoulou 1992, Uzumeri 1975, Paulay 1985, Kitayama 1991, Kurose 1993). Uzumeri (1977) comments that the presence of a large axial compressive force is of help at the early stages of loading and whether a large axial force continues to be of help once joint deterioration starts is debatable. During the latter stages of loading, anchorage of the beam steel is provided at the bend of the beam steel. At this stage, the concrete in the core acts as a series of struts anchored at their ends by the joint steel. Uzumeri concludes that a large axial compressive force applied to these struts may be detrimental rather than helpful. Vollum (1998) states that the joint shear strength is totally independent of the column axial stress. Pantazopoulou (1992) states that the shear strength of a joint depends on the usable compressive strength of concrete, which decreases with increasing principal tensile strain. The principal tensile strain, on the other hand, increases with increasing column axial stress. Consequently, the joint shear strength decreases with the increase in principal tensile strain due to the increase in column axial load. Paulay and Park (1984) state that the beneficial effect of axial compression on the shear strength of exterior beam-column joints depends on the aspect ratio of the joint and is less significant than in the case of interior joints. Paulay further comments that the axial load on the column is not likely to significantly reduce yield penetration and; for this reason, the benefit of axial compression in 'inelastic joints' is likely to be less than in 'elastic joints' (Paulay *et al*. 1978). Kitayama *et al*. (1991) suggests that the column axial load does not seem to influence the joint shear strength even

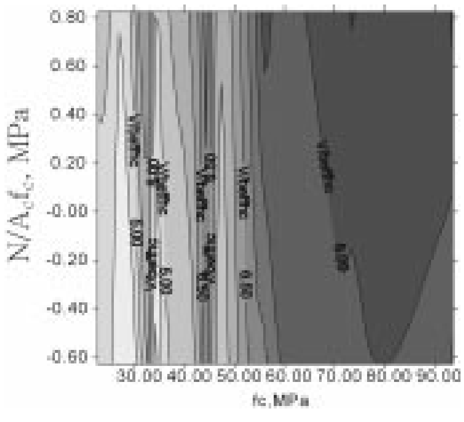


Fig. 4(a) The influence of the $N/A_c f_c$ and the concrete cylinder strength on the joint shear strength

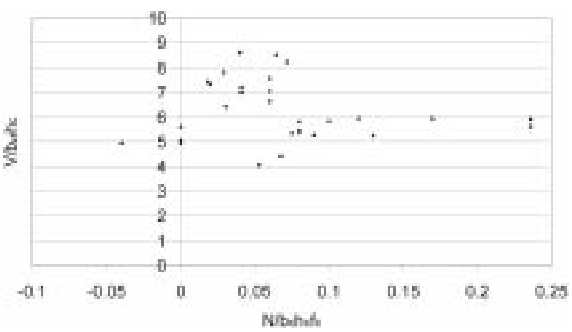


Fig. 4(b) The influence of the column axial stress on the joint shear strength in interior beam-column joints. Kurose (1993) also states that the axial load does not influence the joint shear strength.

In this study, the influence of all possible variables is investigated by using a large number of subsets of regressors as explained in section 3. The tried regressors are the joint aspect ratio h_b/h_c, the stirrup index defined as $A_{sje} * f_{yw} /(b_{eff}\, h_c)$, where b_{eff} is the average width of the beam and the columns, the column longitudinal reinforcement ratio, the beam longitudinal reinforcement ratio, the ratio of the cross-sectional height of the column to the diameter of the beam bars, the ratio of the cross-sectional height of the beam to the diameter of the column bars, the ratio of the crossed inclined bars and the column axial stress. The regression analysis is carried out on 32 specimens with flexural over-strength values between 1 and 1.5 out of the 59 specimens. The best regression statistics are obtained by using the product of the stirrup ratio and the stirrup yield strength, the concrete cylinder strength, the ratio of the height of the column to the diameter of the beam bars and ratio of the crossed inclined bars in the joint area. Thus the equation takes the following form:

$$V=\left(\frac{b_c+b_b}{2}\right)*h_c*\lambda*\left(0.092*f_c+0.34*\frac{A_{inc}*f_{inc}*\cos\theta}{\left(\frac{b_c+b_b}{2}\right)h_c}+0.55*\ln\left(\frac{h_c}{d_b}\right)+0.23*\frac{A_{sje}\,f_{yw}}{\left(\frac{b_b+b_c}{2}\right)h_c}\right) \quad (24a)$$

where λ is a capacity reduction factor of 0.78 and $\cos\theta = h_c/\sqrt{(h_c^2+h_b^2)}$.

Table 3 shows the regression statistics, Table 4 shows the ANOVA Table and Table 5 shows the confidence intervals. The proposed equation shows that stirrups increase the joint shear strength; however, the stirrups' contribution to the joint shear strength is much less than their yield capacity. The joint shear strength is considerably increased by increasing the concrete cylinder strength and the ratio of the crossed inclined bars in the joint area. It is apparent that the crossed inclined bars contribute to the joint shear strength by a mechanism explained in Fig. 21 and section 7 as suggested by Tsonos *et al*. Tsonos suggests that the inclined bars contribute to the joint shear strength by their yield capacity. However, in this study, it is found that a capacity reduction factor of β should be used, as shown in Eqs. (24b) and (24c), to account for the fact that the crossed inclined bars' contribution to the joint shear strength is much less than their yield capacity (34%).

$$V_{sx} = \beta*A_{inc}*f_{inc}*\cos\theta \quad (24b)$$

$$\tan\theta = \frac{h_b}{h_c} \quad (24c)$$

The model of Paulay for exterior beam-column joints has shown that load transfer within the joint is inseparable from the mechanisms of bond. Thus, it is very important to prevent the yield penetration from beam bars to the joint area and prevent the formation of plastic hinges at the face of the column so that the joint can remain elastic. To reduce bond stresses, it is necessary to use the smallest bar diameter that is compatible with practicality. The equation proposed is applied on the experimental database. As apparent from Table 1, it gives very accurate predictions of the joint shear strength. Figs. 5 to 12 show the residual and trend-line plots of the step-wise regression analysis.

Table 3 Regression statistics

Regression statistics	
Multiple R	0.82
R Square	0.67
Adjusted R Square	0.59
Standard deviation	0.64
Observation	32

Table 4 ANOVA tables

	degrees of freedom	Sum of squares	Mean Square	F	p
Regression	4	22.86314	5.715785	13.88183	2.84E-06
Residual	28	11.52888	0.411746		
Total	32	34.39202			

Table 5 Confidence intervals, coefficients, standard deviations and t statistics

	Coefficients	Standard deviation	t Statistics	p-value	Low %95	High %95
Intersection	0	-	-	-	-	-
f_c (MPa)	0.092343	0.01296	7.12504	9.41E-08	0.065795	0.118891
$\ln(h_c/d_b)$	0.551015	0.160776	3.427212	0.001904	0.221679	0.880351
$A_{inc} f_{inc} \cos\theta / b_c h_c$	0.343445	0.127032	2.703618	0.011529	0.083232	0.603658
$A_{sje} f_{yw} / b_{eff} h_c$	0.228464	0.060018	3.806616	0.000704	0.105523	0.351405

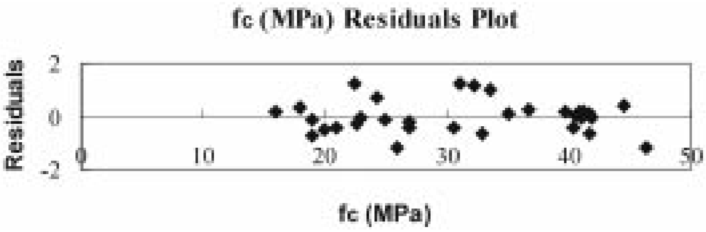


Fig. 5 The residuals plot for the concrete cylinder strength

Fig. 6 The residuals plot for the crossed inclined bar ratio

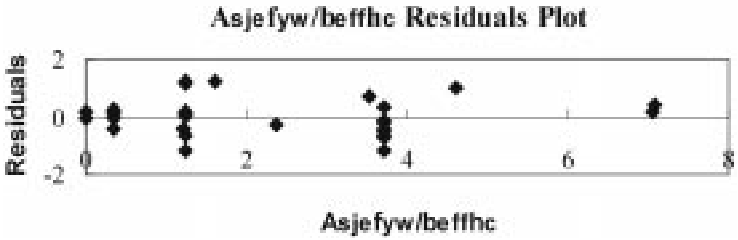


Fig. 7 The residuals plot for the stirrup index

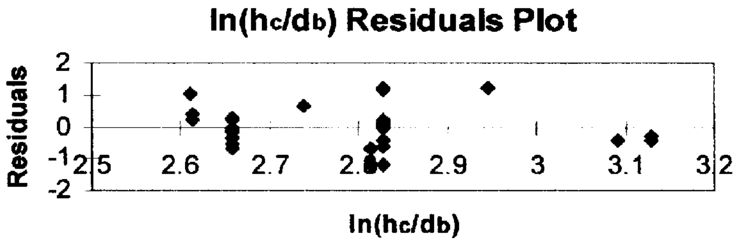


Fig. 8 The residuals plot for the ratio of the cross sectional height of the column to the diameter of the beam longitudinal reinforcement ratio

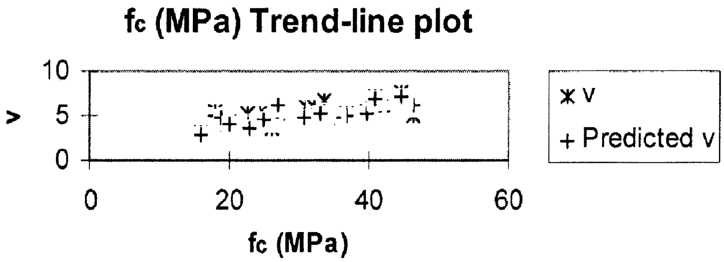


Fig. 9 The trend-line plot for the concrete cylinder strength

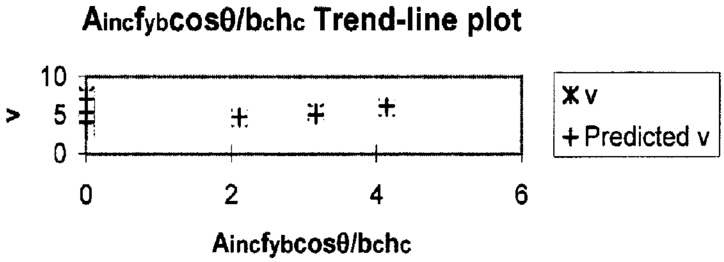


Fig. 10 The trend-line plot for $A_{inc} f_{inc} \cos\theta / b_{eff} h_c$

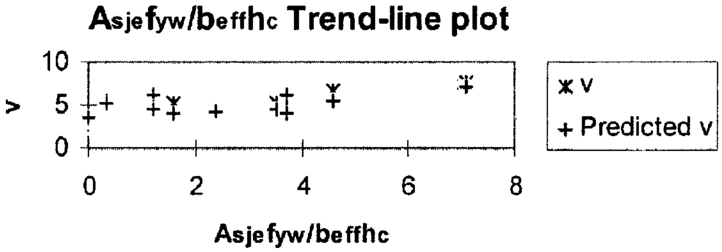


Fig. 11 The trend-line plot for the product of the stirrup ratio and the stirrup yiel strength

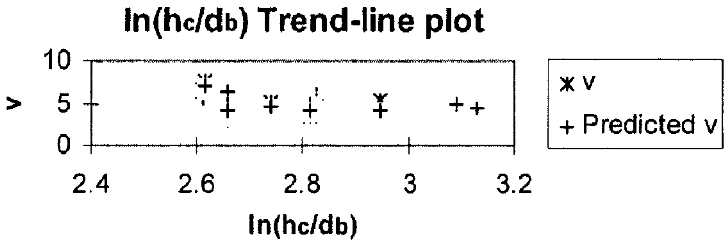


Fig. 12 The trend-line plot for the h_c/d_b

5. Comparison of the parametric studies with the established principles of joint mechanics

In order to investigate the reliability of Eq. (24), the equation is compared with the established equations on the basic mechanics of reinforced concrete beam-column joints. This has been also previously discussed by Paulay (1986) and by Bonacci & Pantazopoulou (1992) in detail for interior joints, who have also taken into account the joint deformations. Bonacci & Pantazopoulou, as well as Paulay, use the average stresses for equilibrium as shown in Fig. 13, and the typical loading system considered in the analysis of the exterior beam-column joints is shown in Fig. 14. This study is complementary to the model of Bonacci and Pantazopoulou in that, the inclined bars are also incorporated into their model.

The equilibrium of forces in the horizontal direction require the average transverse compressive stress in the joint σ_x when inclined bars are used in the joint area defined as:

$$\sigma_x = -\frac{A_{sb}}{d_y d_z} f_s - \frac{A_{sje}}{d_y d_z} f_w - \frac{A_{inc}}{d_y d_z} f_{inc} \cos\theta \tag{25}$$

Consequently, the average normal concrete stress in the y direction σ_y can be expressed as:

$$\sigma_y = -\frac{A_{scol}}{d_x d_z} f_{scol} - \frac{N}{d_x d_z} - \frac{A_{inc}}{d_x d_z} f_{inc} \sin\theta \tag{26}$$

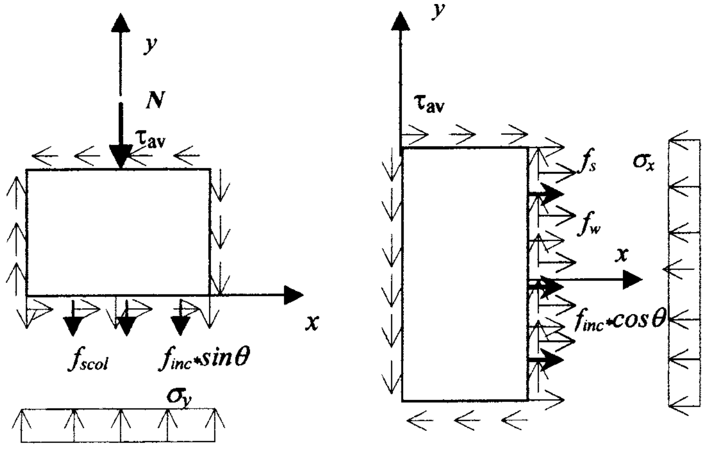


Fig. 13 Stress equilibrium in joints

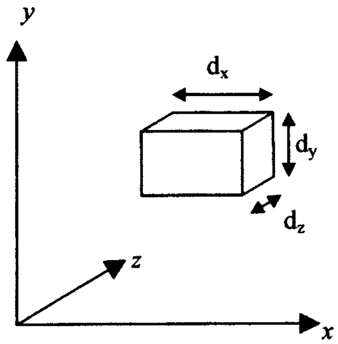


Fig. 14 Joint dimensions

In this study, the last two terms in Eqs. (25) and (26) have been added to account for the shear transfer mechanisms of the crossed inclined bars. Defining the average joint shear stress in the joint as τ_{av}, the maximum principal stress associated with the stress tensor is given as;

$$\sigma^3 - I_1\sigma^2 + I_2\sigma - I_3 = 0 \tag{27}$$

where σ_z is the confining stress provided by stirrups in the z direction.

$$\sigma = \begin{bmatrix} \sigma_x & \tau_{av} & 0 \\ \tau_{av} & \sigma_y & 0 \\ 0 & 0 & \sigma_z \end{bmatrix} \tag{28a}$$

In order to determine the principal stresses, the Eq. (28a) has to be solved;

where $$I_1 = \sigma_x + \sigma_y + \sigma_z \tag{28b}$$

$$I_2 = \sigma_x\,\sigma_y + \sigma_y\,\sigma_z + \sigma_x\,\sigma_z - \tau_{av}^2 \tag{28c}$$

$$I_3 = \sigma_x\,\sigma_y\,\sigma_z - \sigma_z\,\tau_{av}^2 \tag{28d}$$

The tensile stress in the concrete is negligible and therefore $\sigma_1 = 0$, which consequently gives;

$$\sigma_y = \frac{\tau_{av}^2}{\sigma_x} \tag{29}$$

From the Mohr's circle,

$$\tan 2\theta = \frac{2\tau_{av}}{\sigma_x - \sigma_y} \tag{30}$$

If Eq. (29) is substituted into Eq. (30), the following quadratic equation ensues;

$$\tau_{av}^2 + \left(\tan\theta + \frac{1}{\tan\theta}\right)\sigma_x \tau_{av} - \sigma_x^2 = 0 \tag{31}$$

which gives;

$$\sigma_y = -\frac{\tau_{av}}{\tan\theta} \tag{32}$$

Using Eq. (29), we have;

$$\tau_{av} = -\frac{\sigma_x}{\tan\theta} \tag{33}$$

Collins and Mitchell (1991) suggest the following equation for the maximum stress in concrete panels;

$$f_{2\max} = \frac{f_c}{0.8 + 170\varepsilon_1} < f_c \tag{34}$$

The principal compressive stress is given by;

$$\sigma_2 = \left(2\left(\frac{\varepsilon_2}{-0.002}\right) - \left(\frac{\varepsilon_2}{-0.002}\right)^2\right) f_{2\max} \tag{35}$$

σ_2 is also given from Mohr's circle as;

$$\sigma_2 = \sigma_x + \sigma_y = -\tau_{av}\left(\tan\theta + \frac{1}{\tan\theta}\right) \tag{36}$$

Thus, the average joint shear stress can be expressed as;

$$\tau_{av} = -\frac{\sigma_2}{\left(\tan\theta + \frac{1}{\tan\theta}\right)} \tag{37}$$

Eqs. (34) to (36) show very clearly that, as the principal tensile strain increases, the average joint shear stress decreases. Thus, it is necessary to express the principal tensile strain in terms of the strains in the x and y directions in order to investigate the factors that influence the joint shear strength. From Mohr's circle, it is known that;

$$\tan 2\theta = \frac{\gamma}{\varepsilon_x - \varepsilon_y} \tag{38}$$

From Mohr's circle, the principal tensile strain will be;

$$\varepsilon_1 = \frac{(\varepsilon_x + \varepsilon_y)}{2} + \sqrt{\left(\frac{\varepsilon_x - \varepsilon_y}{2}\right)^2 + \left(\frac{\gamma}{2}\right)^2} \tag{39}$$

If Eq. (38) is substituted into Eq. (39) and appropriate trigonometric transformations are carried out, Eq. (40) given by Bonacci and Pantazopoulou is obtained.

$$\varepsilon_1 = \left(\frac{\varepsilon_x - \varepsilon_y \tan^2\theta}{1 - \tan^2\theta}\right) \tag{40}$$

The next step will be to express the strains in the x and y directions in terms of the stresses.

$$\sigma_x = -\tau_{av}\tan\theta = -\frac{A_{sb} f_s}{b_{eff} h_b} - \frac{A_{sje} f_w}{b_{eff} h_b} - \frac{A_{inc} f_{inc}\cos\theta}{b_{eff} h_b}$$

$$= -\left(\frac{A_{sb}}{b_{eff} h_b}\mu + \frac{A_{sje}}{b_{eff} h_b} + \frac{A_{inc}\cos\theta}{b_{eff} h_b}\beta\right) f_w \tag{41}$$

$$\sigma_y = -\frac{\tau_{av}}{\tan\theta} = -\frac{A_{scol} f_{scol}}{d_x d_z} - \frac{N}{d_x d_z} - \frac{A_{inc} f_{inc}\sin\theta}{d_x d_z}$$

$$= f_{scol}\left(-\frac{A_{scol}}{d_x d_z} - \frac{\gamma A_{inc}\sin\theta}{d_x d_z}\right) - \frac{N}{d_x d_z} \tag{42}$$

where $\mu = f_s/f_w$ which is a null value for full bond, $\beta = f_{inc}/f_w$ and $\gamma = f_{inc}/f_{scol}$
The strain in the x direction can therefore be expressed as;

$$\varepsilon_x = \frac{f_w}{E_s} = \frac{\tau_{av}\tan\theta}{E_s\left(\frac{A_{sb}\mu}{d_y d_z} + \frac{A_{sje}}{d_y d_z} + \frac{A_{inc}\beta}{d_y d_z}\cos\beta\right)} \tag{43}$$

The strain in the y direction can similarly be expressed as;

$$\varepsilon_y = \frac{f_{scol}}{E_s} = \frac{1}{E_s}\left(\frac{\left(\frac{\tau_{av}}{\tan\theta} - \frac{N}{d_x d_z}\right)}{\left(\left(\frac{A_{scol}}{d_x d_z} + \frac{\gamma A_{inc}\sin\theta}{d_x d_z}\right)\right)}\right) \tag{44}$$

If Eqs. (43) and (44) are substituted into Eq. (40);

$$\varepsilon_1 = \frac{1}{E_s(1 - \tan^2\theta)}\left(\tau_{av}\tan\theta\left(\frac{1}{\frac{A_{sb}}{d_y d_z}\mu + \frac{A_{sje}}{d_y d_z} + \frac{A_{inc}\beta\cos\theta}{d_y d_z}}\right)\right.$$

$$\left.-d_x d_z \tan^2\theta\left(\frac{\frac{\tau_{av}}{\tan\theta} - \frac{N}{d_x d_z}}{A_{scol} + \gamma A_{inc}\sin\theta}\right)\right) \tag{45}$$

where the angle of inclination can be expressed as equal to the corner-to-corner potential failure plain as shown in Eq. (46).

$$\tan\theta = \frac{h_b}{h_c} \tag{46}$$

The above equation shows that the principal tensile strain is increased by increasing the column longitudinal reinforcement ratio and the axial load on the column, whereas it is decreased by increasing the stirrup ratio. The shear stress in the joint is dependent on the principal tensile strain as evident from Eqs. (34), (35) and (36). It is therefore evident from Eqs. (25) and (45) that the joint shear strength increases as the transverse reinforcement ratio increases. Eq. (26) shows that the joint shear strength increases as the column load and the column longitudinal reinforcement increase but Eq. (45) shows that, as the longitudinal column reinforcement and the column load increase, the principal tensile strain increases, which consequently decreases the normalised joint shear strength. Therefore, the increase in the joint shear strength due to Eq. (26) is offset by the increase in the principal tensile strain. As apparent from Eq. (25), crossed inclined bars are only effective in the horizontal direction. Thus, the steeper the angle between the horizontal direction and the crossed inclined bars, the less effective the crossed inclined bars will be in increasing the joint shear strength. Due to geometrical constraints, this angle is dependent on the joint aspect ratio defined as h_b/h_c and the smaller the joint aspect ratio is, the more the crossed inclined bars will contribute to the joint shear strength. The above conclusions are totally in accordance with the predictions of the author's equation. Figs. 15 and 16 are the 3D plots for the exterior beam-column joints. As apparent from Fig. 15, the joint shear strength increases when the h_c/d_b ratio increases and the ratio of the crossed inclined bars is kept constant and vice versa. Fig. 16 shows that for a constant concrete cylinder strength, the joint shear strength increases by increasing the product of the stirrup ratio and the yield strength of stirrups, and likewise, for a constant stirrup ratio, the joint shear strength increases with increasing concrete cylinder strength.

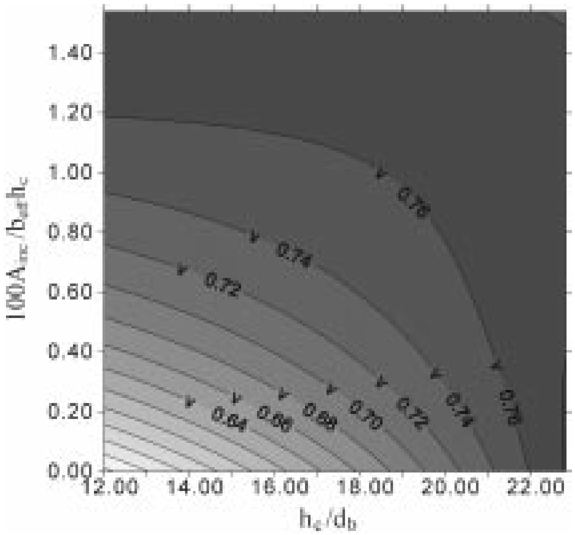


Fig. 15 The influence of the h_c/d_b ratio and the crossed inclined bars on the joint shear strength

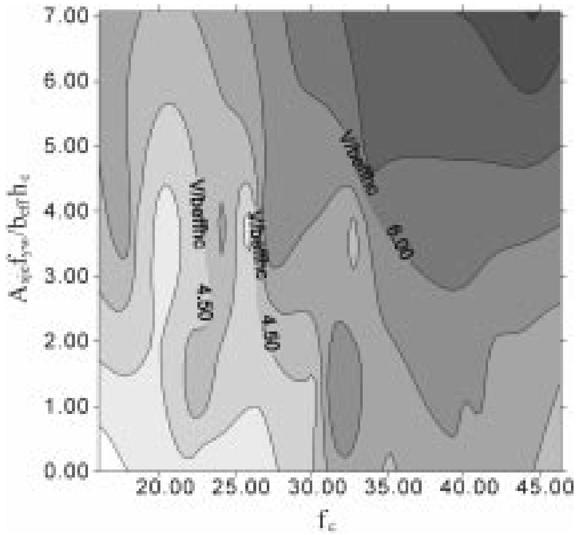


Fig. 16 The influence of the $A_{sje}f_{yw}/b_{eff}h_c$ and the concrete cylinder strength on the joint shear strength

6. Present design guidelines

Both the ACI-ASCE Committee 352, and AIJ Guidelines calculate the joint shear strength on the assumption that the stirrups do not contribute to the joint shear strength. The above methods are considered to be inadequate by the author because they neglect the influence of the stirrups and the influence of the bond conditions on the joint shear strength. The ACI-ASCE Committee 352 Recommendations are compared with the equation of the author in Figs. 17 to 20.

Eq. (24) proposed by the author and the ACI-ASCE Committee 352 Recommendations are applied on the experimental database in Table 1(a). It is apparent from Table 1(b) that the average $V_{predicted}/V_{actual}$ is 0.82 for the proposed equation and 1.63 for the ACI Code Recommendations. The standard deviations for the proposed equation is 0.13 for the proposed equation and 0.70 for the

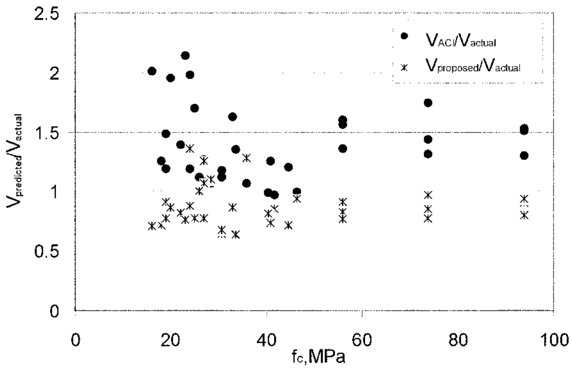


Fig. 17 The influence of the concrete cylinder strength on the predicted joint shear strength of the proposed equation and the equation of ACI

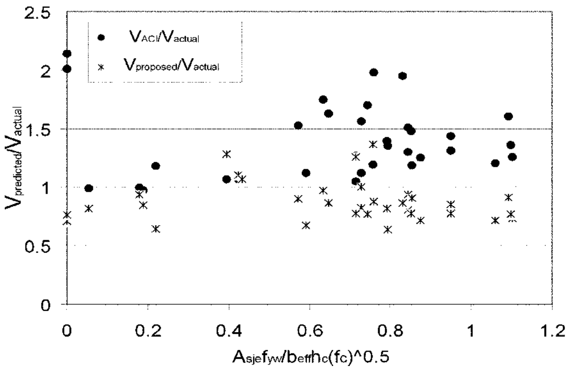


Fig. 18 The influence of the stirrup index on the predicted joint shear strength of the proposed equation and the equation of ACI

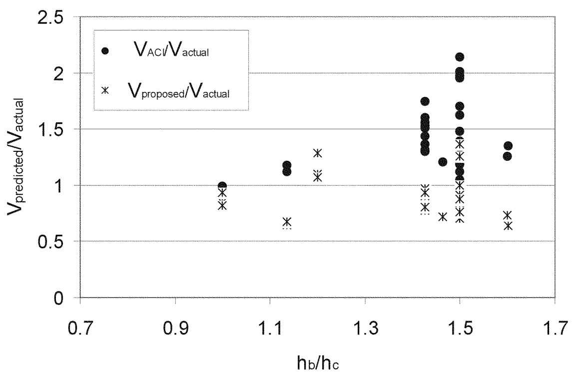


Fig. 19 The influence of the joint aspect ratio on the predicted joint shear strength of the proposed equation and the equation of ACI

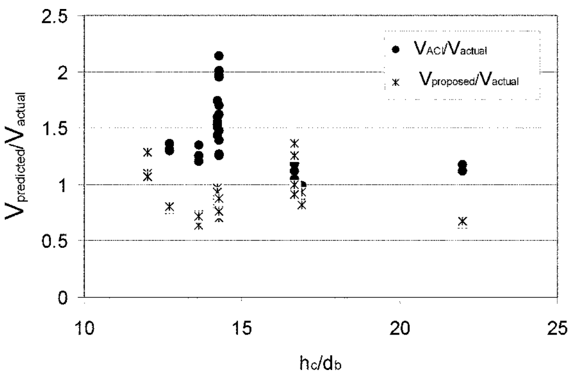


Fig. 20 The influence of the h_c/d_b ratio on the predicted joint shear strength of the proposed equation and the equation of ACI

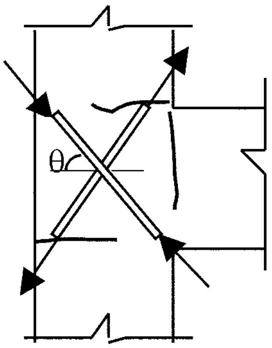


Fig. 21 The shear transfer mechanism of the crossed inclined bars (Tsonos *et al.* 1992)

recommendations of the ACI. It is apparent that the ACI-ASCE Committee 352 Recommendations substantially overestimate the shear strength of the cyclically loaded exterior beam-column joints. The standard deviation of the ACI-ASCE Committee 352 is very high compared to the equation of the author. Thus, the American Code is non-conservative. The proposed equation, on the other hand, gives quite accurate and conservative predictions of the joint shear strength with minimal standard deviation and is an improvement on the existing design codes for joints. It is also evident from Figs. 18 and 19 that the recommendations of ACI overestimates the shear strength at high joint aspect ratio values and high stirrup ratios. It is also evident from Fig. 20 that the ACI code can give very non-conservative results at low h_c/d_b ratios.

7. The shear resisting mechanisms of the cyclically loaded exterior beam-column joints with crossed inclined bars

As mentioned before, cyclically loaded beam-column joints resist the joint shear via two mechanisms. These are the strut mechanism and the truss mechanism, as shown by the model of Paulay in Figs. 1(b) and 1(c). The strut mechanism represents the contribution of concrete to the joint shear strength, whereas the truss mechanism is used to account for the contribution of stirrups. The presence of the inclined bars introduces an additional new mechanism of shear transfer, as shown in Fig. 21, as first suggested by Tsonos *et al.* (1992). This is the truss mechanism of the inclined bars. However, in this study, it is shown that the contribution of the inclined bars to the joint shear strength is much less than their yield capacity (34%).

8. Conclusions

This study aims at understanding the influence of different parameters on the shear strength of cyclically loaded exterior beam-column joints. Particular emphasis is given to codify the influence of the crossed inclined bars on the joint shear strength. A step-wise multiple regression analysis is carried out and the predictions of this analysis are compared with the joint mechanics; the stress equilibrium and the strain compatibility as well as the 3D interaction plots. The conclusions of the present study are as follows:

1. Crossed inclined bars are a feasible solution for increasing the shear strength of the cyclically loaded beam-column joints. But due to the geometrical constraints, the increase in the joint shear strength due to the crossed inclined bars is dependent on the joint aspect ratio. The greater the joint aspect ratio (h_b/h_c) is, the less the contribution of the crossed inclined bars will be to the joint shear strength.
2. The contribution of the crossed inclined bars is much less than their yield capacity (34%).
3. The stepwise regression analysis shows that the joint shear strength increases as the concrete cylinder strength and the h_c/d_b ratio increases, and the joint shear strength is independent of the column axial stress or the column longitudinal reinforcement ratio. These findings are also confirmed by the joint mechanics as apparent from Eq. (45), which shows that the principal tensile strain is increased by the column longitudinal reinforcement ratio and the axial load on the column, whereas it is decreased by increasing the stirrup ratio. The shear stress in the joint is dependent on the principal tensile strain, as evident from Eqs. (34), (35) and (36). It is clear from Eqs. (25) and (45) that the joint shear strength increases as the transverse reinforcement ratio increases. Eq. (26) shows that the joint shear strength increases as the column load and the column longitudinal reinforcement increase, but Eq. (45) shows that as the longitudinal column reinforcement and the column load increases, the principal tensile stresses increase, which consequently decrease the normalised joint shear strength. Therefore, the increase in the joint shear strength due to Eq. (26) is offset by the increase in the principal tensile strain.
4. The 3D interaction plots also confirm the findings of the joint mechanics and the proposed equation.
5. A large number of subsets of regressors are tried for the step-wise multiple regression analysis and any subset of regressors which have a ratio less than 4 are not accepted. Throughout the step-wise regression analysis, a significance level of 0.05 is used, and it is checked whether the hypothesis that $\beta = 0$ will be rejected. The F value should be considerably greater than the minimum value needed to reject Ho: $\beta = 0$. In the present analysis the F value is five times the minimum value needed to reject Ho: $\beta = 0$. The residual plots are checked for the possibility of non-linearity, however, the spread of the residuals are fairly constant over x and there are no plots illustrating a horse-shoe shaped non-constant residual variance.
6. The existing code recommendations are thought to be inadequate by the author because they do not take into account the beneficial influence of the crossed inclined bars and the ratio of the height of the column to the diameter of the beam longitudinal reinforcement. In addition to this, both the ACI-ASCE Recommendations and the Japanese Code are based on the assumption that the joint shear strength is not significantly influenced by stirrups. The present study has shown, however, that the joint shear strength is increased by increasing the ratio of stirrups, but this increase is much less than their yield capacity (23% of their yield capacity) as opposed to that recommended by the New Zealand design philosophy for joints. The proposed equation has been compared with the equation of ACI in Figs. 17 to 20. It is apparent that the proposed equation is an improvement on the equation of ACI. The equation of ACI is non-conservative and substantially overestimates the shear strength of cyclically loaded exterior beam-column joints. The standard deviation of the equation of the ACI-ASCE Code is also substantially high. The proposed equation, on the other hand, predicts the shear strength of the cyclically loaded beam-column joints quite accurately and with minimal standard deviation.
7. The proposed equation takes into account the bond conditions of the beam bars. It is apparent from this study that the shear strength of beam-column joints is closely related to the bond

conditions of the beam bars. The equation requires that the diameter of the beam bars relative to the column cross-sectional height should be kept as small as possible to increase the joint shear strength.

8. The equation is also applicable on joints with high strength concrete because it is derived from a database consisting of a large number of high strength concrete specimens.

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Notation

A_{inc}	: cross-sectional area of the crossed inclined bars
A_s	: cross-sectional area of the beam longitudinal top reinforcement
A_{scol}	: cross-sectional area of the total column reinforcement
A_{sje}	: area of joint stirrups
b_{a1}, b_{a2}	: smaller of one quarter of the column depth and one half the distance between the beam and column faces
b_b	: beam width
b_c	: width of the column
b_j	: effective joint width which is taken as $b_j = (b_b + b_c)/2$ but not greater than the beam width plus one half the column depth on each side of the beam
b_{eff}	: average of the beam and the column width
c_{ii}	: ith diagonal element of $(X^TX)^{-1}$
d	: effective depth of the beam

d' : cover
d_b : diameter of the beam longitudinal reinforcement
C_c : concrete compressive force in the flexural compression zone
C_s : compression force in the compression reinforcement
D_1 : diagonal concrete compressive force originating at the beam bar anchorage hook and subjected to a clamping force from the intermediate column bars
D_2 : diagonal concrete compressive force maintaining equilibrium of the joint ties
Dj : taken as equal to column depth for interior joints and as equal to the horizontal projected length of the beam reinforcement in exterior beam-column joints
E_s : modulus of elasticity of steel
f_2 : principal compressive stress in concrete
$f_{2\max}$: maximum stress in concrete panels
f_c : concrete cylinder strength
f_{inc} : average stress in the crossed inclined bars
f_s : average stress in the beam reinforcement
f_{scol} : average stress in the column reinforcement
f_y : yield strength of the beam longitudinal reinforcement
f_{yw} : yield strength of the stirrups
f_w : average stress in the transverse reinforcement
H : total height of the column
h_c : column depth in the direction of joint shear
h_b : beam depth
k : factor that is dependent on the type of beam-column joint which is equal to 0.3 for interior beam-column joints and 0.18 for exterior beam-column joints
N : column axial load
p : number of independent variables
SS : sum of squares
SS_{reg} and the SS_{total} : sum of squares for the regression and total analysis as explained in Table 2
$T_1, ..., T_4$: forces acting in the longitudinal column bars between the corner bars in side faces of the column
V_{col} : shear force in the upper column
V_{cv} : ideal vertical joint shear strength provided by concrete shear resisting mechanism
V_j : applied joint shear
V_{jh} : total horizontal shear force across a joint
V_{jv} : total vertical shear force across a joint
V_{ju} : joint shear strength
V_{sv} : ideal vertical joint shear strength provided by vertical joint shear reinforcement
α : joint aspect ratio defined as h_b/h_c
ΔT : bond force
ε_1 : principal tensile strain
ε_2 : compressive strain
ε_c : compresive strain at failure (–0.002)
ε_x : tensile strain in the x direction
ε_y : tensile strain in the y direction
θ : angle between the direction of the principal compressive stress and the transverse tensile strain ε_t
γ : joint shear stress expressed as a multiple of $\sqrt{f_c}$
ρ_{stir} : lateral reinforcement ratio
σ_x : average normal concrete stress in the x direction
σ_y : average normal concrete stress in the y direction
σ_z : confining stress provided by stirrups in the z direction