Exterior RC joints subjected to monotonic and cyclic loading

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Abstract

Purpose – The purpose of this study is to develop two empirical models that predict the shear strength of exterior beam-column joints exposed to monotonic and cyclic loading using Gene expression programming (GEP).

Design/methodology/approach – The GEP model developed for the monotonic loading case is trained and validated using 81 data test points and that for cyclic loading case is trained and validated using 159 data test points that collected from different 9 and 39 experimental programs, respectively. The parameters that are selected to develop the cyclic GEP model are concrete compressive strength, joint aspect ratio, column axial load and joint transverse reinforcement. The monotonic GEP model is developed using concrete compressive strength, column depth, joint width and column axial load.

Findings – GEP models are proposed in this paper to predict the joint shear strength of beam-column joints under cyclic and monotonic loading. The predicted results obtained using the GEP models are compared to those calculated using the ACI-352 code formulations. A sensitivity analysis is also performed to further validate the GEP models.

Originality/value – The proposed GEP models provide an accurate prediction for joint shear strength of beam-column joints under cyclic and monotonic loading that is more fitting to the experimental database than the ACI-352 predictions where the GEP models have higher R^2 value than the code formulations.

Keywords Cyclic loading, Gene expression programming, ACI, Joint shear strength, Monotonic loading

Paper type Research paper

Introduction

Reinforced concrete (RC) buildings are designed according to modern seismic codes that use the capacity design method. According to the capacity design method, beams are designed to be weaker than columns so that plastic hinges are developed in beams rather than columns (Park and Paulay, 1975). However, many existing RC structures are not designed according to the modern seismic codes where beam-column joints have little or no reinforcement. These buildings are vulnerable during earthquake events. Joint shear failure is depicted in these structures and can cause building collapse during recent earthquake events. Pure shear failure occurs in the joint panel without any plastic hinges forming in beams or columns where the reinforcement in beams and columns remains elastic. Joint shear failure is a brittle type of failure that happens under relatively small rotations.

Experimental studies (Murad *et al.*, 2018; Zhou *et al.*, 2018), analytical and numerical models (Murad, 2016), found in the literature, have shown that the key parameters that influence joint shear strength include concrete compressive strength, joint aspect ratio, joint width, column axial load and joint transverse reinforcement. These studies have also shown that joint shear strength increases with increment of the square root of the concrete compressive strength ($\sqrt{f'_c}$) (Vollum and Newman, 1999). Joint aspect ratio is the ratio of beam depth to column depth ($\frac{h_b}{h_c}$). It is also found in the literature that joint shear strength

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decreases as joint aspect ratio increases (Vollum and Newman, 1999). Joint shear strength is significantly reduced by increasing joint aspect ratio due to steeper joint strut inclination (Hassan and Moehle, 2012). Based on the experimental data of joints tested under monotonic loading, it is found that joint shear strength is not significantly dependent on the beam reinforcement (Vollum and Newman, 1999). Park and Mosalam (2013) have shown that joint shear strength of beam-column joints, which experience beam hinge followed by joint shear failure (BJ failure mode), depends on the flexural reinforcement ratio. However, Park and Mosalam (2013) have shown that joint shear strength does not depend on the flexural reinforcement ratio for the cases where joint shear failure occurs prior to beam and column yielding.

The effect of column axial load on joint shear strength depends on the type of loading. Vollum and Newman (1999) have shown that joint shear strength is not affected by column axial load under monotonic loading, whereas in the case of cyclic loading the effect of column axial load on joint shear strength is unclear. Some experimental studies (Clyde *et al.*, 2000; Beres *et al.*, 1996) have shown that joint shear strength increases with increasing axial load, whereas others (Pantazopoulou and Bonacci, 1993) have shown the opposite under cyclic loading. For cyclic loading case, Gan *et al.* (2019) have shown that the bond strength has been increased with an axial load level of 0.4 and a joint tube width-to-thickness ratio of 50. They have shown that the enhancement in the bond strength can change the mode of failure from beam flexural failure to beam flexural failure with bond failure. They have also shown that the bond strength decreases with an axial load level of 0.20 due to the pinching effect. For cyclic loading case, joint shear strains are greatly reduced under high axial loads and this was shown in the previous experimental studies (Hassan and Moehle, 2012). For monotonic loading case, axial load has insignificant effect on the joint strains. Thus, high column axial load can decrease the joint shear strength under seismic loading.

Joint shear strength of unconfined joints is less than that found in confined beam-column joints of the same dimensions. The higher transverse reinforcement in the joint panel, the higher joint shear capacity is. Joint shear strength of confined joints consists of concrete and transverse reinforcement shear capacities in the joint panel (Paulay and Priestley, 1992).

Although code formulations, experimental programs and numerical models propose expressions to predict joint shear strength, there is still lack of simplified formulation that can accurately predict joint shear strength of beam-column joints exposed to either monotonic or cyclic loadings. The experimental behaviour of concrete is generally simulated using empirical modelling based on classical regression techniques. Regression analyses work on the basis of predefined functions that are performed after defining functions. Recently, explicit functions that predict the behaviour of concrete are developed using computer applications, such as gene expression programming (GEP) and artificial neural network (ANN) (Cevik and Sonebi, 2008; Sonebi and Cevik, 2009). GEP is superior to regression techniques and ANNs because it does not require a predefined function to perform the analysis. However, GEP approach works by adding or deleting various combinations of parameters to be considered for the formulation that best fits the experimental results (Cevik and Sonebi, 2008; Sonebi and Cevik, 2009). For the case where analytical expressions are not available, GEP is an efficient tool in determining explicit formulations for the experimental results including multivariate parameters (Cevik and Sonebi, 2008; Sonebi and Cevik, 2009).

Murad *et al.* (2019) have proposed a GEP model to predict the bond strength between the concrete surface and carbon fibre reinforced polymer sheets under direct pull out. Thus, they have collected a large database containing 770 test specimens and they have shown that the GEP model can predict the bond strength with a reasonable accuracy. The authors have compared the results obtained using the GEP model with the results obtained from several existing models and they have found that the predicted bond strength is in agreement with the overall trends of the existing models and experimental results with R^2 values higher than all other models. Murad *et al.* (2019) have also developed predictive models using GEP to estimate the compressive strength of green concrete. Accurate models that estimate the compressive strength of green concrete are still lacking in the literature. They have proposed four GEP models to predict the compressive strength of plain concrete, fly ash concrete, silica fume concrete and concrete with silica fume and fly ash.

Two equations are proposed in this study to predict the shear strength of exterior RC beam to column joints exposed to monotonic and cyclic loading using GEP. The equations are developed based on large experimental database available in the literature. A sensitivity analysis is then performed to check the sensitivity of the proposed models to the input parameters. Furthermore, a comparison is made between the values of joint shear strength obtained using the GEP models and the ACI-352 formulations (*ACI Committee 318*, 2014) to validate the model.

Experimental database

A large experimental database is collected from literature to develop GEP models for exterior RC joints exposed to monotonic and cyclic loading. The failure mode of the collected specimens, shown in the Appendix, is joint shear. The GEP model developed for the monotonic loading case is trained and validated using 81 data test points that collected from different nine experimental tests (Reys De Otiz, 1993; Parker and Bullman, 1997; Kordina, 1984; Scott, 1992; Sarsam and Phipps, 1985; Yap and Li, 2011; Maariappan *et al.*, 2013; El-Nabawy Atta *et al.*, 2003; Hegger *et al.*, 2003), whereas the GEP model created for cyclic loading case is trained and validated using 159 data test points that collected from different 39 experimental programs (Antonopoulos and Triantafillou, 2003; Del Vecchio *et al.*, 2014; Wong and Kuang, 2005; Rajagopal and Prabavathy, 2014; Beydokhty and Shariatmadar, 2016; Ghobarah and Said, 2002; Alva, 2004; Alva, de Cresce El Debs and El Debs, 2007; Bindhu *et al.*, 2009; Calvi *et al.*, 2001; Chalioris *et al.*, 2008; Chun and Kim, 2004; Chun *et al.*, 2007; Chutarat and Aboutaha, 2003; Pantelides *et al.*, 2002; Durrani and Zerbe, 1987; Ehsani and Alameddine, 1991; Ehsani *et al.*, 1987; Ehsani and Wight, 1985a; Ehsani and Wight, 1985b; Mustafa and Ilhan, 2002; Hamil, 2000; Hakuto *et al.*, 2000; Hwang *et al.*, 2004; Karayannis *et al.*, 2008; Karayannis and Sirkelis, 2005; Karayannis and Sirkelis, 2008; Kuang and Wong, 2006; Kusuhara and Shiohara, 2020; Lee and Ko, 2007; Liu, 2006; Pampanin *et al.*, 2002; Pantelides, 2002; Tsonos *et al.*, 1993; Tsonos, 1999, 2007; Wong and Kuang, 2008). A sample of the collected data is illustrated in Table AI for the monotonic loading case, whereas the experimental database for RC joints tested under cyclic loading is shown in Table AII. The training and validation data is randomly selected from the database where the training data is 75 per cent of the total database, whereas the data used for validation is 25 per cent of the total database for the monotonic loading case. For the cyclic loading case, 70 per cent of the total database is used for training, whereas 30 per cent is used for validation. The validation database for the monotonic case is taken 25 per cent, whereas it is taken 30 per cent for the cyclic case because it has larger database.

Experimental studies and analytical models, found in the literature, have shown that joint shear strength is predominantly controlled by specific parameters. These parameters are selected to develop the GEP model and they include concrete compressive strength, joint aspect ratio, joint width, column depth, column axial load and joint transverse reinforcement.

Code formulations for predicting joint shear strength

Various analytical expressions and code formulations are found in the literature that predicts joint shear strength under either monotonic or cyclic loading. However, an accurate expression that can fit large database of the experimental results is still needed. Therefore, GEP is used in this research to develop empirical models for joint shear strength that can fit a large database of the experimental results available in the literature. Joint shear strength is then predicted using the ACI-352R-02 (ACI-ASCE Committee 352, 2002) formulations for monotonic and cyclic loading cases that depicted in equation (1) where the constant γ is 15 and 20 for the cyclic and monotonic exterior joints, respectively, $\boldsymbol{b_j}$ the effective joint width, $\boldsymbol{h_c}$ is the column depth and $\boldsymbol{f'_c}$ is the concrete compressive strength.

$$V_{jh} = 0.083\ \gamma\ \boldsymbol{b_j}\ \boldsymbol{h_c}\ \sqrt{\boldsymbol{f'_c}} \tag{1}$$

Gene expression programming

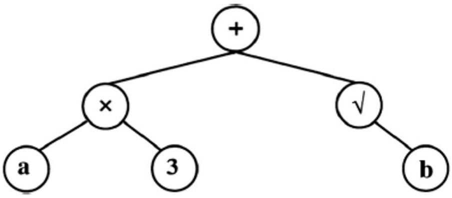
Overview of genetic programming

Genetic programming (GP) was firstly created by Cramer in 1985 and further promoted and developed into a practical tool by Koza (1994). GP is an extension to genetic algorithms.

The genetic algorithm is based on natural selection and it involves solving constrained and unconstrained optimisation problems. The solution process involves selecting random values from the population to be parents at each step and these parents are used to produce the children for the next generation. After sequential generations, the population is evolved and an optimal solution is generated. Genetic algorithm can be used for sophisticated problems with discontinuous, non-differentiable, stochastic or highly nonlinear functions.

GP uses nonlinear structure (parse trees) representation to solve the problems of fixed end solutions. It also uses alphabet to create these structure (Ferreira, 2002). GEP is a branch of GP that was developed by Ferreira (Ferreira, 2002), whereas GEP has higher capability of solving relatively complex problems using small population sizes (Ferreira, 2002). The GEP uses chromosomes and the expression trees (ETs) for the developed computer program where the ET is the expression of the genetic information encoded in the chromosomes (Ferreira, 2002; Sarıdemir, 2010; Gandomi *et al.*, 2014; Özcan, 2012; Jafari and Mahini, 2017). Chromosomes may contain one or more genes indicating a mathematical expression. Each gene has a head and a tail where the head consists of both function and terminal symbols (constants, variables, functions and mathematical operators, such as 1,a, b, $\sqrt{}$, cos, *,–,/ (Beheshti Aval *et al.*, 2017), whereas the tail has only terminals (constant and variables), such as 1,a, b, c. Mathematical operators, such as addition, subtraction and division, are used to link between the genes. The ET in Figure 1 can be expressed mathematically as $[(a \times 3) + (\sqrt{b})]$.

Figure 1.
Example of GEP expression tree



Source: Koza (1994)

The development of a new GEP model incorporates selecting fitness function followed by choosing the set of terminals and the set of functions to create the chromosomes. The chromosome architecture is then selected by choosing the length of the head and the number of genes. The linking function and the set of genetic operators that cause variation are finally selected (Ferreira, 2002).

GEP has been used recently to explain concrete behaviour. Various studies have been conducted using GEP that confirm the efficiency of GEP in civil engineering applications (Mousavi *et al.*, 2012; Soleimani *et al.*, 2018; Lim *et al.*, 2016; González-Taboada *et al.*, 2016; Gholampour *et al.*, 2017; Gandomi *et al.*, 2014; Nazari and Pacheco Torgal, 2013). The shear strength of short rectangular RC columns is predicted by Aval *et al.* using GEP (Beheshti Aval *et al.*, 2017). Özcan (2012) used GEP to develop a model for splitting tensile strength of concrete.

Model development

GeneXproTools software (Gepsoft, 2014) is used in the current research to create the GEP model where various GEP models have been developed to choose a GEP model that best fit the experimental database. Several trials have been done by varying the number of genes, chromosomes, head size and linking function to select the best GEP model that can predict the experimental results with a reasonable accuracy. The optimal parameters of the selected GEP models are shown in Table I for monotonic and cyclic joint shear strengths. Increasing the number of chromosomes has resulted in increasing the running time (Gholampour *et al.*, 2017), whereas increasing the number of genes has over-fitted the results but it generates complex function (Gholampour *et al.*, 2017). The number of genes is fixed to 2 in this study and the linking functions are subtraction for the cyclic model, whereas it is division for the monotonic model as shown in Table I. The GEP models are expressed mathematically in equations (2) and (3) for monotonic and cyclic joint shear strength, respectively. Furthermore, the GEP models are also expressed using ET format as shown in Figures 2 and 3 for monotonic and cyclic loading cases, respectively. The parameters d_o, d_1, d_2 and d_3 in the cyclic GEP model's ET are concrete compressive strength $\left(f_c^{'}\right)$, joint aspect ratio (h_b/h_c), joint transverse reinforcement (A_{sj}) and column axial load (P), respectively, and c_O to c_2 are constants. The constants of the cyclic GEP model are $c_1 = -521.72$, $c_0 = -4.37$) for the first gene and are $c_1 = -0.36$, $c_2 = 2.13$) for the second gene. The monotonic GEP model has two constants for the second gene only ($c_0 = 8.27$, $c_2 = 8.93$). The parameters in the ET for

GEP	Monotonic	Cyclic
Function set	+, −, ×,/, x^2, 1/x, ^	+, −, ×,/, x^2, 1/x, $\sqrt{x}$
Genes	2	2
Chromosomes	33	33
Head size	7	7
Linking function	Division	Subtraction
Constant per gene	3	3
Mutation rate	0.05	0.05
Inversion rate	0.1	0.1
Transposition rate	0.1	0.1
One point recombination rate	0.3	0.3
Two point recombination rate	0.3	0.3
Gene recombination rate	0.1	0.1
Gene transportation rate	0.1	0.1

Table I.
GEP setting parameter

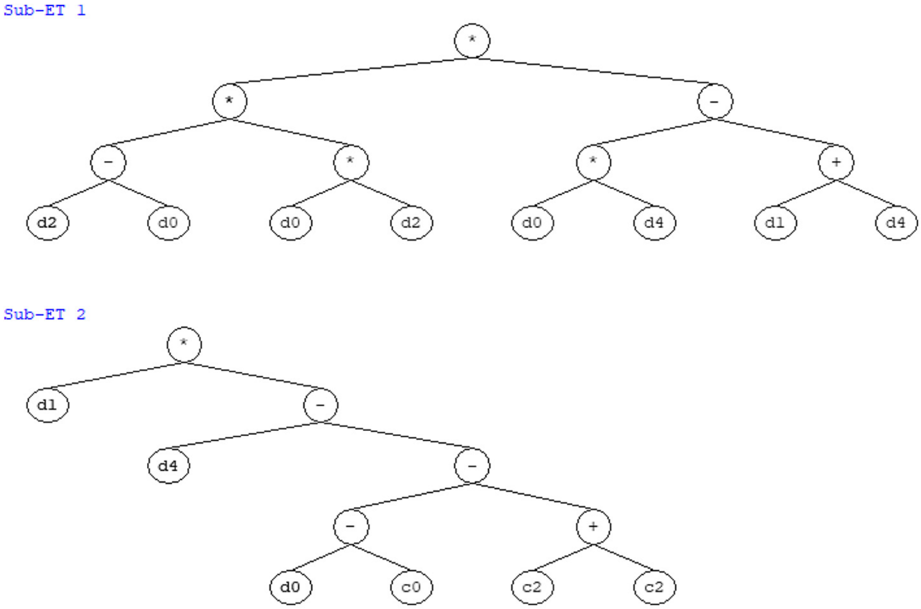


Figure 2.
Expression tree of the developed GEP model for monotonic joint shear strength

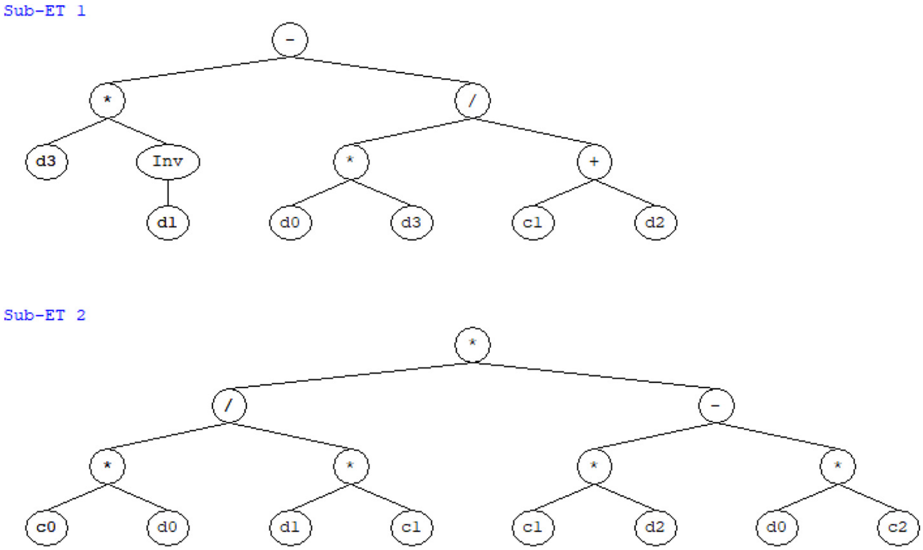


Figure 3.
Expression tree of the developed GEP model for cyclic joint shear strength

the monotonic GEP model d_o, d_1, d_2 and d_3 are concrete compressive strength $\left(f_c'\right)$, column depth (h_c), joint width (b_j) and column axial load (P), respectively. The results have shown that both GEP expressions are able to predict the shear strength of RC joints exposed to cyclic or monotonic loading with a reasonable accuracy.

$$\boldsymbol{V_j} = \left[\frac{f_c^{\prime}\ b_j\ \left(b_j - \ f_c^{\prime}\ \right) \times \ \ \left(P\ f_c^{\prime}\ \ - \ \ h_c - P\right)}{h_c\left(P - \left(\left(f_c^{\prime}\ \ - 8.3\right) - 17.85\right)\right)}\right] \tag{2}$$

$$\boldsymbol{V_j} = \left[\frac{P}{h_b/h_c} - \frac{P\ f_c^{\prime}}{-521.72 + \ A_{sj}}\right] - \left[\frac{-4.37\ f_c^{\prime}\ \left(-0.36\ A_{sj} - 2.13\ f_c^{\prime}\ \right)}{-0.36\ \left(\boldsymbol{h_b}/\boldsymbol{h_c}\right)}\right] \tag{3}$$

The performance of the proposed GEP models is statistically evaluated using the coefficient of determination (R^2) that is expressed in equation (4), the mean absolute error (MAE), the mean and the standard deviation.

$$R^2 = \frac{\left(\sum_{i=1}^{N}\left(X_i - \overline{X}\right)\left(Y_i - \overline{Y}\right)\right)^2}{\sum\limits_{i=1}^{N}\left(X_i - \overline{X}\right)^2 \sum\limits_{i=1}^{N}\left(Y_i - \overline{Y}\right)^2} \tag{4}$$

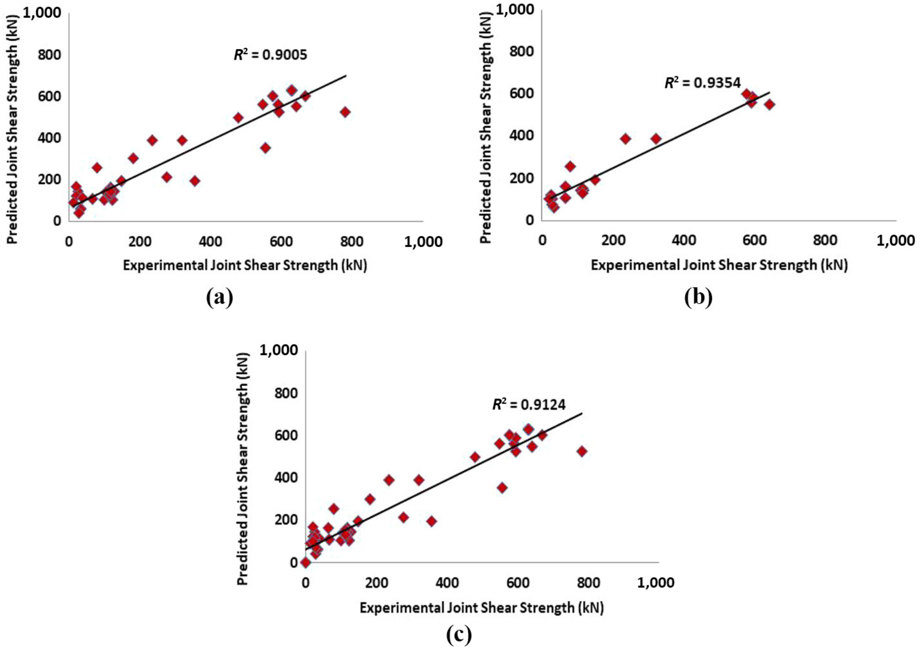
$$MAE = \frac{1}{N}\sum_{i=1}^{N}\left|X_i - Y_i\right| \tag{5}$$

The statistical values of R^2 for the training, validation and all input data of the monotonic model are 90, 93.5 and 91 per cent, respectively, and that for the cyclic model are 92, 95 and 93 per cent, respectively. The mean values, for the predicted and experimental joint shear strengths, are 575 and 590 kN, respectively, for the cyclic loading case, whereas they are 296 and 285 kN for the monotonic loading case, respectively. The standard deviation values, for the predicted and experimental joint shear strengths, are (219, 255) and (473, 594) for the monotonic and cyclic GEP models, respectively. The MAE values are 56.7 and 73 per cent for the monotonic and cyclic GEP models, respectively.

Based on the performance evaluation results, the GEP has shown an excellent correlation between the predicted and measured values where the values of R^2 are high for the validation and testing data. Figure 4 (a)-(c) illustrates a comparison between the predicted and experimental joint shear strength under monotonic loading case for the testing, validation and all data, respectively. Figure 5 (a)-(c) compares between the predicted and experimental joint shear strength under cyclic loading case for the testing, validation and all data, respectively. Both GEP models have an excellent capability in prediction joint shear strength under monotonic and cyclic loading where the distribution of points for both models is close to the ideal fit.

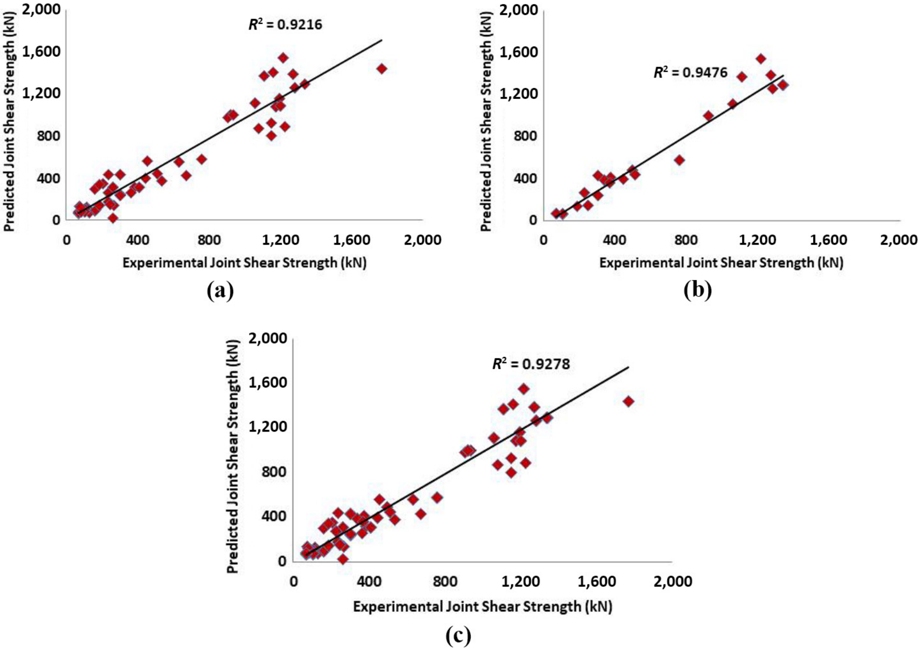
Comparison of the gene expression programming models predictions with ACI-352 expression

Figure 6 compares between the experimental and predicted joint shear strength under monotonic loading case using the GEP model and ACI-352 expression. Figure 7 compares between the experimental and predicted joint shear strength under cyclic loading case using the GEP model and ACI-352 expression. The predicted joint shear strengths for monotonic and cyclic loading case using the GEP models are most fitting the experimental results with high R^2 compared to the code formulations. The R^2 values for the joint shear strength predicted using the code formulation are 31 and 76.6 per cent under monotonic and cyclic



(a) (b) (c)

Figure 4. Comparison between the predicted and experimental values of (a) training data, (b) validating data and (c) all data using monotonic GEP model



(a) (b) (c)

Figure 5. Comparison between the predicted and experimental values of (a) training data, (b) validating data and (c) all data using cyclic GEP model

loading case, respectively, for all data inputs, whereas the R^2 values for the joint shear strength predicted using the GEP models are 91 and 93 per cent under monotonic and cyclic loading case, respectively.

Gene expression programming models sensitivity

A sensitivity analysis is performed in this section for the proposed GEP models to check the sensitivity of the input parameters to the predicted joint shear strength. Therefore, each input parameter is varied while keeping the other parameters constant to check the effect of each input parameter on the predicted joint shear strength. A comparison is then made between the trends obtained from the GEP models and the previous experimental results to further validate the GEP models. The reference input data for the monotonic GEP model is concrete compressive strength $\left(f_c'\right) = 35\,\text{MPa}$, column depth ($h_c$) = 400 mm, joint width (b_j) = 200 mm and column axial load (P) = 300 kN, whereas that for the cyclic GEP model is concrete compressive strength $\left(f_c'\right) = 35\,\text{MPa}$, joint aspect ratio ($h_b/h_c$) = 1.2, joint transverse reinforcement (A_{sj}) = 800 mm^2 and column axial load (P) = 300 kN. The variations of the input parameters with the monotonic and cyclic GEP models are shown in Figures 8 and 9, respectively. Figure 8 (a)-(d) shows that the predicted joint shear strength, for beam-column joints exposed to monotonic loading, increases by increasing the concrete compressive strength and joint width, whereas it decreases by increasing the column depth. The monotonic joint shear strength almost remains constant by the variation of column axial load. It is shown in Figure 9 (a)-(d) that the predicted joint shear strength, for beam-column joints exposed to cyclic loading, increases by increasing the concrete compressive strength, joint reinforcement area and column axial load, whereas it decreases by increasing

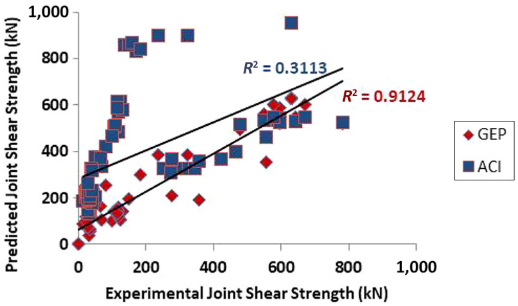


Figure 6.
Comparison between the ACI and GEP models for monotonic loading case

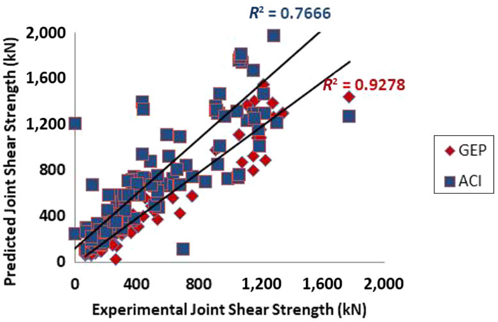
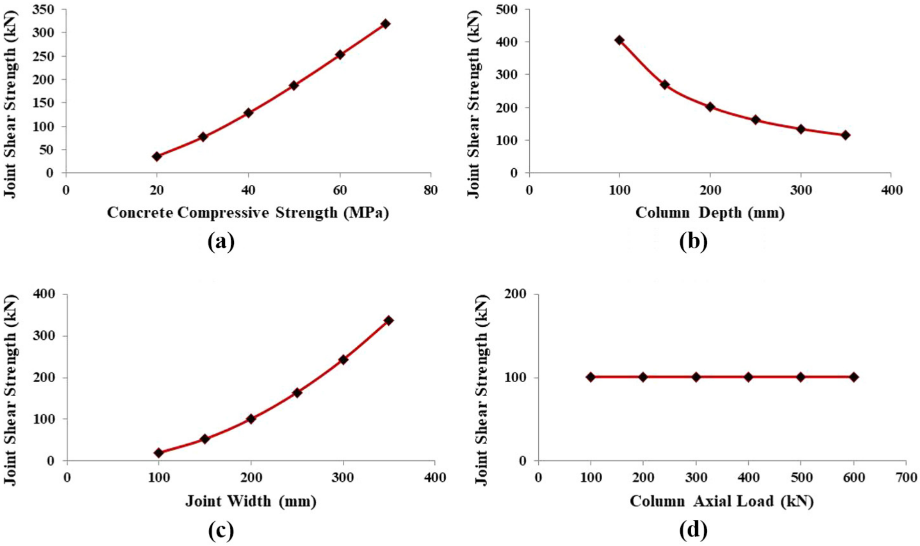


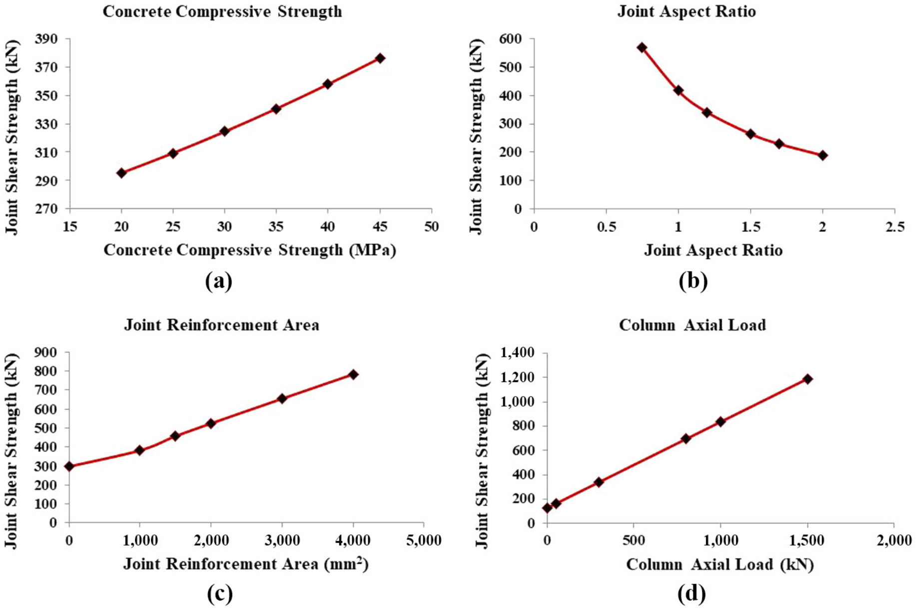
Figure 7.
Comparison between the ACI and GEP models for cyclic loading case



(a) (b) (c) (d)

Figure 8.
Influence of the input parameters on the predicted joint shear strength of the monotonic GEP model

Notes: (a) Concrete compressive strength; (b) column depth; (c) joint width; (d) column axial load



(a) (b) (c) (d)

Figure 9.
The influence of the input parameters on the predicted joint shear strength of the cyclic GEP model

Notes: (a) Concrete compressive strength; (b) joint aspect ratio; (c) joint reinforcement area; (d) column axial load joint aspect ratio. The trends of the proposed GEP models conform to the trends of the existing experimental results available in the literature that illustrated previously in the introduction. This confirms the accuracy of the GEP models and its sensitivity to the input parameters.

Conclusion

Two empirical models that predict the shear strength of exterior beam-column joints exposed to cyclic and monotonic loading is developed in this research using GEP. The GEP model developed for monotonic loading case joints is trained and validated using 81 data test points collected from different nine experimental tests, whereas the GEP model created for cyclic loading case joints is trained and validated using 159 data test points collected from different 39 experimental programs. Experimental studies and analytical models, found in the literature, have shown that joint shear strength is predominantly controlled by specific parameters. These parameters are selected to develop the cyclic GEP model including concrete compressive strength, joint aspect ratio, column axial load and joint transverse reinforcement. The monotonic GEP model is developed using concrete compressive strength, column depth, joint width and column axial load. The models are validated using the experimental results and statistical assessments are used to evaluate the performance of the proposed GEP models. The predicted results obtained using the GEP models are compared to those calculated using ACI-352 code formulations. Both models provide an accurate prediction for joint shear strength of beam-column joints exposed to cyclic and monotonic loading that is more fitting to the experimental database than that predicted using the ACI-352 formulations. The GEP models have higher R^2 value than the code formulations. The proposed GEP model is considered a very useful tool to evaluate the shear strength of beam-column joints exposed to cyclic or monotonic loading for design and analysis purposes.

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Appendix

Authors and year	Specimen name	f'_c (MPa)	h_c (mm)	b_j (mm)	P (*N*)	Joint shear strength (*N*)
Reys De Otiz (1993)	BCJ 1	34	300	200	0	11,8000
	BCJ 2	38	300	200	0	12,5000
	BCJ 3	33	300	200	0	11,8000
	BCJ 4	34	300	200	0	13,,0000
	BCJ 5	38	300	200	30,0000	11,5000
	BCJ 6	35	300	200	30,0000	11,5000
Parker and Bullman (1997)	4b	44.48	150	130	27,5000	27,300
	4c	35.92	150	130	27,5000	27,900
	4d	51.2	150	130	50,000	23,900
	4e	51.2	150	130	10,0000	24,600
	4f	48.8	150	130	50,000	28,700
	5b	36.8	150	130	50,000	33,900
	5f	100.8	150	130	10,0000	36,100
Yap and LI (2011)	NS01	101.6	150	130	10,0000	37,100
	NS02	96.8	150	130	10,0000	41,200
	NS03	45.6	150	130	50,000	33,500
El-Nabawy Atta *et al.* (2003)	G1-A	41.6	150	130	50,000	35,200
	G1-B	50.4	150	130	50,000	39,500
	G1-C	39.2	300	275	30,0000	13,8000
	G2-B	36.8	300	275	57,0000	17,0000
	G2-C	39.2	300	275	0	15,0000
	G3-B	40	300	275	30,0000	16,0000
	G3-C	37.6	300	275	60,0000	18,3000
	G3-D	43.2	300	275	30,0000	23,6000
	G3-E	43.2	300	275	60,0000	32,2000
	G3-F	30	350	300	0	62,9800
Hegger *et al.* (2003)	RK4	30	350	300	0	62,9800
	RK5	24	200	200	90,000	27,0230
	RK6	24	200	200	18,0000	29,7366
	RK7	24	200	200	27,0000	25,2139
	RK8	24	200	200	90,000	31,0934
Maariappan *et al.* (2013)	BCJ 1	24	200	200	18,0000	34,4854
	BCJ 2	24	200	200	27,0000	29,0017
	BCJ 3	67	200	200	40,0000	59,5382
	BCJ 4	36	200	200	40,0000	46,5142
	BCJ 5	65	200	200	40,0000	54,8868
	BCJ 6	60	200	200	40,0000	47,9097
Scott (1992)	C1AL	65	200	200	40,0000	59,0731
	C2	62	200	200	40,0000	59,5382
	C3L	68	200	200	40,0000	57,6777
	C4	64	200	200	40,0000	64,1897
	C4A	68	200	200	40,0000	66,9805
	C4AL	62	200	200	40,0000	78,1439
	C5	27.64	200	150	5,000	28,197
	C6	51.7	200	150	50,0000	35,7000
	C6L	54.9	200	150	50,0000	42,3000
	C7	86.5	200	150	50,0000	55,6000
	C8	54.7	200	150	50,0000	27,7000
	C9	38.6	200	150	50,0000	27,3000

Table AI. Sample of the experimental database for monotonic RC joints

Table AII. Sample of the experimental data base for cyclic RC joints

Authors and year	Specimen name	f'_c (MPa)	(hb/hc)	Asj (mm^2)	P (*N*)	Joint shear strength (*N*)
Alva (2004)	LVP1	40.4	1.3	1,206	36,0000	53,9500
Alva *et al.* (2007)	LVP2	44.2	1.3	1,005	39,7620	51,4100
	LVP3	23.9	1.3	1,206	21,5010	36,4400
	LVP4	24.6	1.3	1,005	22,1580	32,7200
	LVP5	25.9	1.3	1,206	23,3190	38,0400
Bindhu *et al.* (2009)	A1	36.7	1.0	396	15,92000	74,710
Calvi *et al.* (2001)	T1	23.9	1.7	0	12,0000	62.29
Chalioris *et al.* (2008)	JA-0	34	1.0	157	10,2000	24,1230
	JA-s5	34	1.0	660	10,2000	24,2760
	JA-X12	34	1.0	383	10,2000	24,1230
	JA-X14	34	1.0	465	10,2000	24,0460
	JB-0	31.6	1.0	0	94,800	23,0970
	JB-s1	31.6	1.0	101	94,800	25,2530
	JB-X10	31.6	1.0	157	94,800	24,7160
	JB-X12	31	1.0	226	94,800	24,5630
	JCa-0	20.6	1.0	0	41,200	69,7400
	JCa-X10	20.6	1.0	157	41,200	70,470
	JCa-s1	20.6	1.0	101	41,200	69,370
	JCa-s1-X10	20.6	1.0	258	41,200	70,470
	JCa-s2	20.6	1.0	201	41,200	69,010
	JCa-s2-X10	20.6	1.0	358	41,200	70,110
	JCb-0	23	1.0	0	46,000	10,5770
	JCb-X10	23	1.0	157	46,000	10,3860
	JCb-s1	23	1.0	101	46,000	10,4980
	JCb-s1-X10	23	1.0	258	46,000	10,5340
	JCb-s2	23	1.0	201	46,000	10,6080
	JCb-s2-X10	23	1.0	358	46,000	10,6440
Chun and Kim (2004)	JC-2	60.1	1.0	2,752	49,0000	13,43890
	JM-2	60.1	1.0	2,752	49,0000	13,42220
Chun *et al.* (2007)	JC-2	60.1	1.0	4,054	0	11,99900
	JM-2	60.1	1.0	4,054	0	11,97600
	JC-No. 11-1	32.8	1.0	3,103	0	11,25630
	JM-No. 11-1a	32.8	1.0	3,103	0	11,13890
	JM-No. 11-1 b	32.8	1.0	3,103	0	10,80400
Chutarat and Aboutaha (2003)	I-Group 1	27.6	1.1	3,575	0	93,2470
	II-Group 1	27.6	1.1	3,575	0	11,88600
Clyde *et al.* (2000)	Test 2	55.7	0.9	774	68,9000	11,54340
	Test 4	49.4	0.9	774	13,80000	13,02610
	Test 5	44.6	0.9	774	13,57000	11,84770
	Test 6	48.3	0.9	774	58,7000	1,104.49
Durrani and Zerbe (1987)	J1	47.4	1.2	2,280	17,5000	45,7790
	J2	47	1.2	2,280	17,5000	63,3500
	J5	46.6	1.2	2,280	17,5000	98,5980
	J7	49	1.2	2,280	17,5000	76,0570
Ehsani and Alameddine (1991)	LL11	75.8	1.4	2,294	28,5000	92,9150
	LH11	75.8	1.4	3,054	27,6000	92,5090
	HL11	75.8	1.4	2,533	58,7000	11,77460
	HH14	96.5	1.4	3,293	60,5000	12,17610
	LL14	96.5	1.4	2,294	23,6000	93,6500
	LH14	96.5	1.4	3,054	22,2000	93,3530

(*continued*)

Table AII.

Authors and year	Specimen name	f'_c (MPa)	(hb/hc)	Asj (mm^2)	P (*N*)	Joint shear strength (*N*)
Ehsani *et al.* (1987)	1	64.6	1.4	1,333	13,3000	67,6160
	2	67.2	1.4	1,333	33,8000	59,2680
	3	64.6	1.5	1,333	38,3000	71,6270
	4	67.2	1.5	1,534	32,5000	92,0970
Del Vecchio *et al.* (2014)	T_C1	12.6	1.7	0	19,200	25,1778
	T_C2	16.4	1.7	0	19,200	20,9309
	T_C3	16.3	1.7	0	19,200	32,3065
Rajagopal and Prabavathy (2014)	A1	22.64	1	0	10,000	26,8760
	B1	22.64	1	0	10,000	25,0352
	C1	22.64	1	0	10,000	26,4342
Wong and Kuang (2005)	BS-L	30.88	1.5	0	10,00000	31,5500
	BS-OL	30.88	1.5	0	10,00000	21,9200
	BS-LL	42.08	1.5	0	10,00000	39,8700
	BS-U	31.04	1.5	0	10,00000	34,1200
Beydokhty and Shariatmadar (2016)	NS5	38.5	0.75	0	70,000	60,3102

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