1-1-2008

Investigating shear capacity of RC beam-column joints using artificial intelligence techniques

Eslam Mohamed Alnaji Hassan Khalifa
University of Nevada, Las Vegas

INVESTIGATING SHEAR CAPACITY OF RC BEAM- COLUMN JOINTS USING ARTIFICIAL INTELLIGENCE TECHNIQUES

By

Eslam Mohamed Alnaji Hassan Khalifa

Bachelor of Civil Engineering
Structural Department
Ain Shams University, Cairo, Egypt
2003

A thesis submitted in partial fulfillment
of the requirements for the

Master of Science Degree in Engineering
Department of Civil and Environmental Engineering
Howard R. Hughes College of Engineering

Graduate College
University of Nevada, Las Vegas
August 2008

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By

Eslam Mohamed Alnaji Hassan Khalifa
2008

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Thesis Approval
The Graduate College
University of Nevada, Las Vegas

August 19, 2008

The Thesis prepared by

Eslam M. Hassan Khalifa

Entitled

Investigating Shear Capacity of RC Beam-Column Joints using Artificial Intelligence Techniques

is approved in partial fulfillment of the requirements for the degree of

Master of Science in Engineering

Examination Committee Chair

Dean of the Graduate College

Examination Committee Member

Examination Committee Member

Examination Committee Member

Graduate College Faculty Representative

1017-53

ABSTRACT

Investigating Shear Capacity of RC Beam-Column Joints Using Artificial Intelligence Techniques

By

Eslam Mohamed Alnaji Hassan Khalifa

Dr. Aly Said, Examination Committee Chair
Assistant Professor of Civil Engineering
University of Nevada, Las Vegas

Beam-column joints are critical zones in reinforced concrete structures. The behavior of joints is very complex and governed by different mechanisms such as flexure, shear, and bond stress between the reinforcement and the concrete. Shear transfer mechanisms through the joint are one of the most important factors affecting the joint performance. Shear failure occurring in the joint can lead to severe damage and may result in the collapse of the structure. This thesis presents an investigation into the shear capacity of reinforced concrete beam-column joints. The performance is influenced by several key parameters. An analysis is carried out to simulate the behavior of the exterior beam-column joints subjected to monotonic loading and of interior joints subjected to reverse cyclic loading. The main parameters considered in this study are: joint shear reinforcement ratio, concrete compressive strength, beam tension longitudinal reinforcement ratio, joint aspect ratio, and column axial stress. The analysis is conducted

using a database collected from different experimental programs in the literature. Based on this database, analytical models are created using two artificial intelligence approaches namely artificial neural networks (ANNs) and genetic algorithms (GAs). Evaluation of the existing formulae is conducted and the effect of each of the investigated parameters is stated and new formulae are proposed for the shear design of a reinforced concrete beam-column joint.

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ACKNOWLEDGEMENT

I express my earnest acknowledgements to Dr. Aly Said for his continuous support and understanding. I genuinely thank him for his guidance in the layout, presentation and completion of my Thesis. I express my deepest gratitude to Dr. Barbra Luke for her support to me during the period of my research. The financial support of the DOE through the "Earthquake Hazards/Seismic Risk in Southern Nevada" project (Grant Number DE-FG52-03NA99204, A004) is greatly appreciated. I also would like to thank few friends for the great support. Thank you very much Gad, Mahmud, Lewis, Safi, Mike. I dedicate my Masters Degree and my Thesis to my Mother and to my Father with all the gratitude and love I have for them in my heart. Thank you mom and dad for everything you taught me in my life. I also want to thank my brothers and sisters for the great support. And to that special person, although you are not here anymore, but thank you very much for everything. You will always be in my heart.

CHAPTER 1

INTRODUCTION

Beam-column joint mechanics is a crucial element that ensures the integrity of reinforced concrete structures. Shear failure in beam-column joints may trigger a total structural collapse. Several studies in the literature (Taylor, 1974; Hoekstra, 1977; Meinheit and Jirsa, 1977; Durrani and Wight, 1982; Bosshard and Menn, 1984; Kordina, 1984; Otani *et al.*, 1984; Sarsam and Phipps, 1985; Park and Ruitoing, 1988; Paulay, 1989; Joh *et al.*, 1991; Pantazopoulou and Bonacci, 1992; Ortiz, 1993; Pantazopoulou and Bonacci, 1993; Scott *et al.*, 1994; Teraoka *et al.*, 1994; Parker and Bullman, 1997; Vollum, 1998; Hamil, 2000; Hwang and Lee, 2000; Zaid, 2001; Bakir and Boduroğlu, 2002a; Bakir and Boduroğlu, 2002b; Bakir, 2003; Hegger *et al.*, 2003) investigated the shear behavior and strength of beam-column joints in many cases such as exterior and interior joints, and monotonically loaded and cyclically loaded joints. These studies used experimental and analytical techniques to examine the key parameters affecting the shear capacity of beam-column joints. They indicated that the following parameters are the main ones governing the shear behavior of reinforced concrete beam-column joints:

1. Joint shear reinforcement ratio.
2. Concrete compressive strength.
3. Beam tension longitudinal reinforcement ratio.
4. Joint aspect ratio.

5. Column axial stress.

Furthermore, the behavior of the beam-column joint is very complex due to the interaction between the various mechanisms that control this behavior such as shear, bond, flexure, and confinement of the joint.

Despite the numerous formulae proposed for calculating the shear capacity of beam-column joints, there is still some uncertainty in calculating the shear capacity of joints. Among the published formulae, the validity of using a specified formula is limited to the range of parameters accounted for in its derivation. This makes it difficult to specify one formula as a design approach for calculating the shear capacity of all beam-column joints. Figure 1.1 shows a typical interior beam-column joint.

The high uncertainty about the joint behavior was a motive for the current study to apply the Artificial Intelligence technique to investigate the shear behavior of beam-column joints. Artificial intelligence can be used to predict the output of a certain system based on the previous system's behavior represented through available input-output data. AI investigates the properties of a specific system by simulating it using a known history of cases that have similar conditions and properties to the investigated system.

In this study, two artificial intelligence techniques were used to investigate the shear behavior of RC beam-column joints. These techniques are the artificial neural networks (ANNs) and the genetic algorithms (GAs). Two critical cases of beam-column joints were investigated which are the exterior monotonically loaded joints and the interior cyclically loaded joints. The study will enable structural engineers to more accurately estimate the strength of existing deficient beam-column joints and to enhance the design of new structures, thus avoiding undesirable modes of failure in joints. Figure 1.2

represents a schematic diagram of a typical exterior beam-column joint. Figure 1.3 shows a typical exterior beam-column joint specimen.

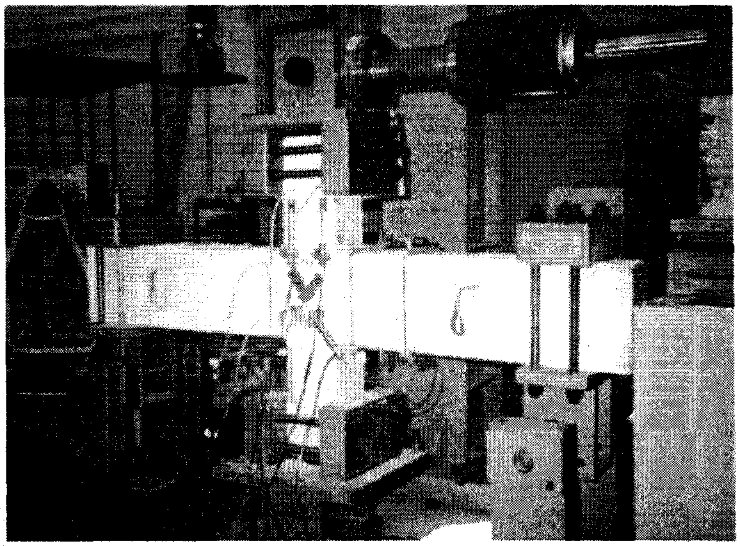


Figure 1.1. Typical interior beam-column joint
University of Auckland in New Zealand
Source: *http://www.cee.auckland.ac.nz*
Accessed on 02/03/2008

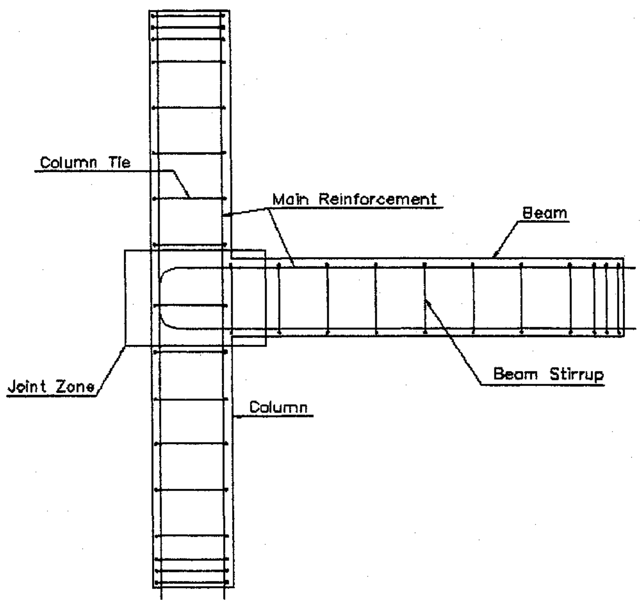


Figure 1.2. Schematic diagram of typical exterior beam-column joint

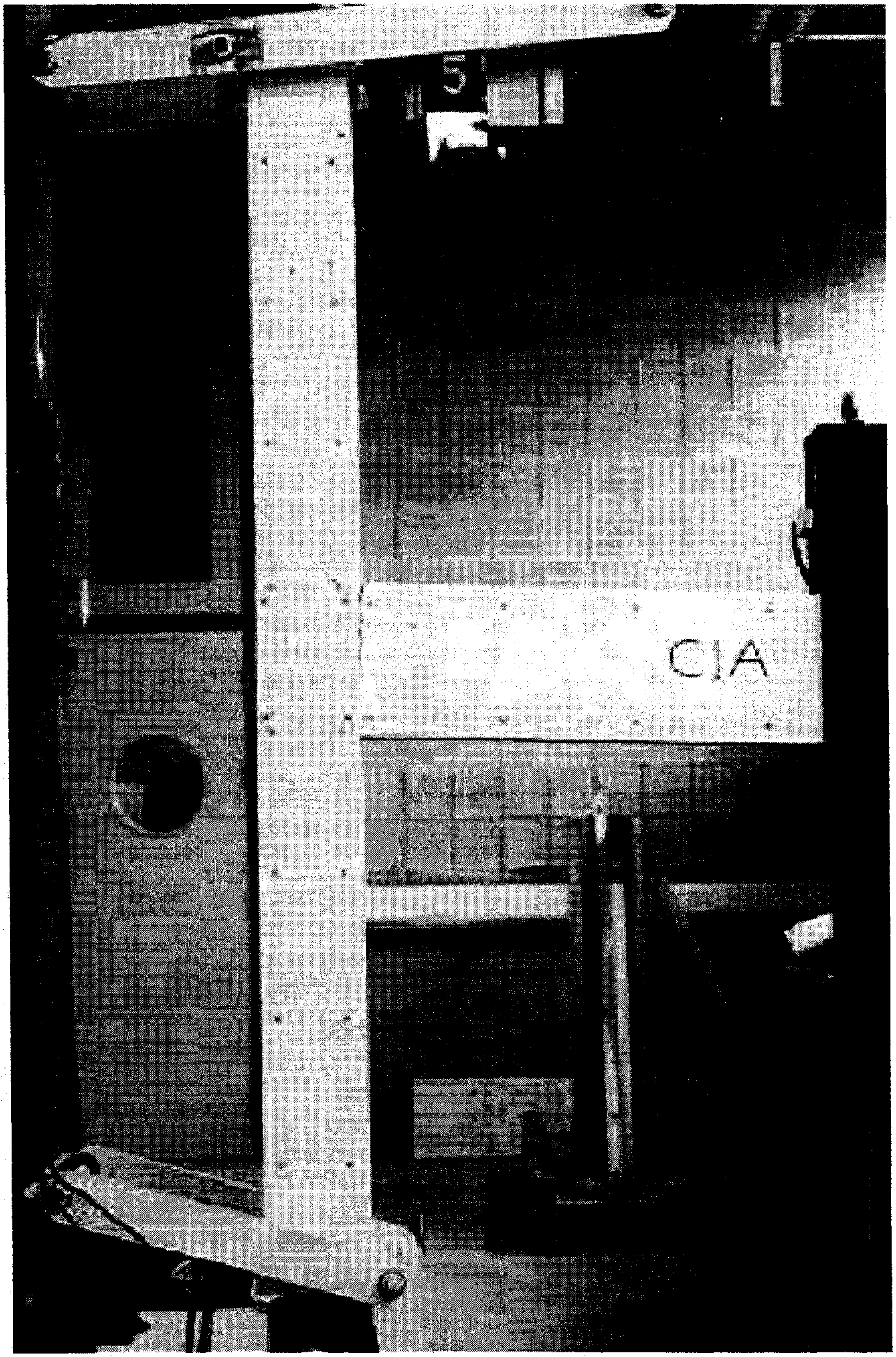


Figure 1.3 Typical exterior beam-column-joint tested by Hamil (2000)

CHAPTER 2

BACKGROUND AND LITERATURE REVIEW

2.1 Introduction

Over the last few decades, several studies were conducted to investigate the shear behavior of reinforced concrete beam-column joints. While most of these studies focused on the performance of cyclically loaded joints, some of them studied joints subjected to monotonic loading. Different techniques were used in these studies including experimental programs and analytical programs. Several formulae were proposed for calculating the shear capacity of beam-column joints.

The monotonically loaded joints and cyclically loaded joints share some key influencing parameters such as the joint shear reinforcement ratio, concrete compressive strength, joint aspect ratio, and column axial stress. The most desired performance in the joint zone is when a flexure failure in the connected beams occurs before the shear failure in the joint. Cyclically loaded joints require more precautions in their design to overcome the displacement demand developed due to the cyclic loading. Furthermore, cyclic loading usually generates higher deformations than those generated by similar monotonic loading due to the strength degradation associated with repeated reversed cyclic loading (Chopra, 2007).

2.2 Monotonically Loaded Exterior Beam-Column Joints

Majority of the studies conducted to investigate the shear capacity of monotonically loaded joints investigated exterior beam-column joints. The equilibrium forces in exterior and interior beam-column joints are shown in Figure 2.1 and Figure 2.2 respectively. The joint shear force V_j is calculated from the following equations:

Exterior monotonically loaded joint: $$V_j = T - V_{col} \tag{2.1}$$

Interior monotonically loaded joint: $$V_j = T_1 - T_2 - V_{col} \tag{2.2}$$

where T is the force in the beam tension reinforcement, C is the compression force on concrete in the beam, and V_{col} is the shear force on the column.

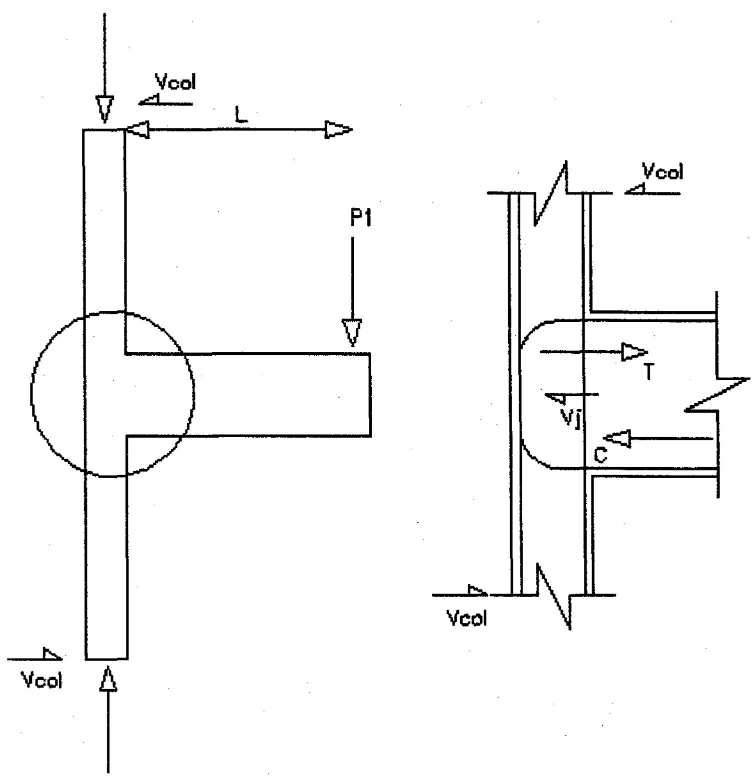


Figure 2.1. Equilibrium forces within an exterior monotonically loaded joint

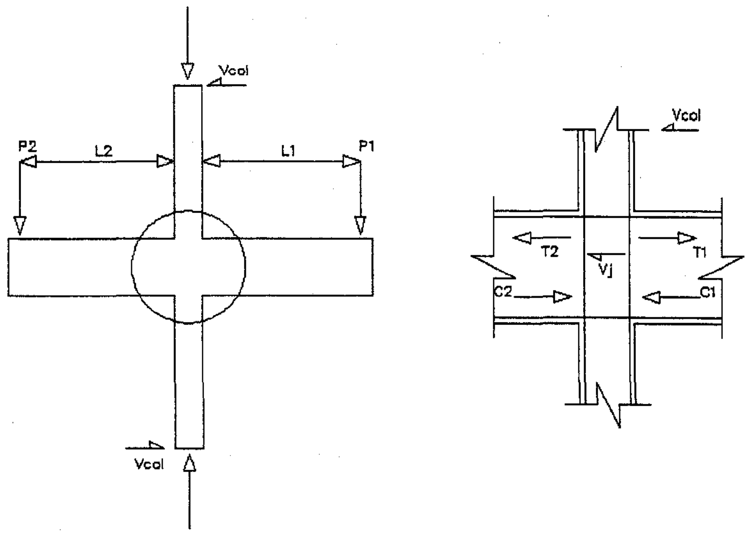
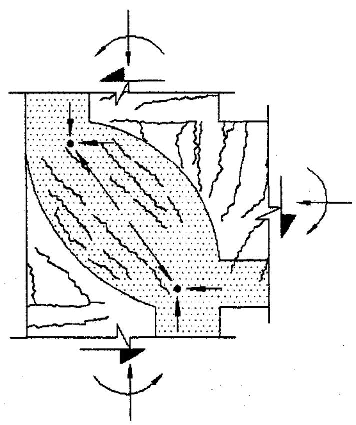


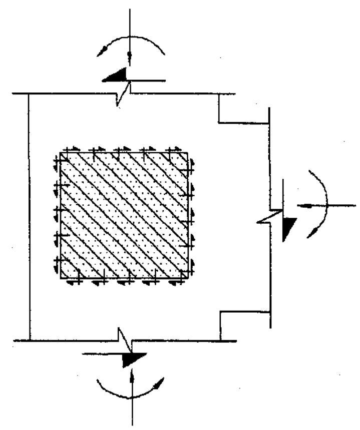
Figure 2.2. Equilibrium forces within an interior monotonically loaded joint

2.2.1 Beam-Column Joint Behavior

According to the model proposed by Paulay (1989), a beam-column joint resists joint shear through two mechanisms. The first mechanism is the strut mechanism that accounts for the contribution of concrete to joint shear strength, and the second mechanism is the truss mechanism, which accounts for the contribution of joint stirrups to joint shear strength as shown in Figure 2.3. In the truss mechanism the horizontal link represents the stirrups that are situated between the top of the beam compressive reinforcement and the beam tensile reinforcement. The vertical tie in the truss mechanism accounts for the intermediate column bars. Paulay (1989) also suggested that this vertical tie equilibrates the vertical shear in the joint. This assumption was disputed by Vollum (1998) and Fuji and Morita (1991) who proved that there is a considerable amount of tensile shift in the forces at the columns from that calculated values. Thus, intermediate column bars are ineffective in resisting vertical shear in the joint. It should be noted that the strut mechanism can develop without any bond stress transfer at the beam and column reinforcement within the joint, while the truss mechanism can exist only when a good bond stress transfer is maintained along the beam and column reinforcement. Thus, the increase of the joint shear strength by the stirrups is related to good bond conditions of the beam reinforcement through the joint. The relative contributions of the strut and truss mechanisms to joint shear strength are argued according to many studies (Sarsam and Phipps, 1985; Pantazopoulou and Bonacci, 1992; Pantazopoulou and Bonacci, 1993; Ortiz, 1993; and Bakir and Boduroğlu, 2002b).



(a) Diagonal strut mechanism



(b) Truss mechanism

Figure 2.3. Shear transfer mechanisms proposed by Paulay (1989) for exterior beam-column joints

2.2.2 Modes of Failure for Monotonically Loaded Joints

A beam-column joint sub assemblage consists of three main elements: beams, columns, and the joint connecting them. The main modes of joint sub assemblage failure are categorized as follows based on the type and location of initial failure:

2.2.2.1 Flexural Bending Failure

If a joint has enough strength to resist the shear forces applied on it and the column has enough strength to withstand the forces on it, the failure will be formed in the beam due to ductile bending failure. This is the most desirable failure mode since it prevents sudden failure in the joint (brittle failure) and the ductility of the beam will provide high amount of energy dissipation before the collapse. Furthermore, repairing the flexural failure in the beam is much easier than repairing the shear failure in the joint. Figure 2.4 shows a diagrammatic representation of flexural bending failure.

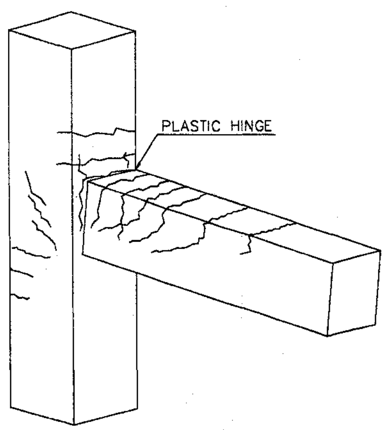


Figure 2.4. Diagrammatic representation of beam flexural failure

2.2.2.2 Joint Shear Failure

If shear failure occurs in the joint before the flexure failure of the beam, then this failure is called a joint shear failure. This type of failure is not a desired mode of failure. Shear failure in the joint can lead to severe damage causing collapse of the structure. Besides, repairing a joint is much harder and more expensive than repairing either the beam or the column. Figure 2.5 shows diagrammatic representation of a joint subjected to joint shear failure.

2.2.2.3 Anchorage Failure

This mode of failure occurs when the tension reinforcement in the beam is not anchored properly within the joint. The tension reinforcement bars are pulled out of the joint at a load below that which causes either beam failure or joint failure.

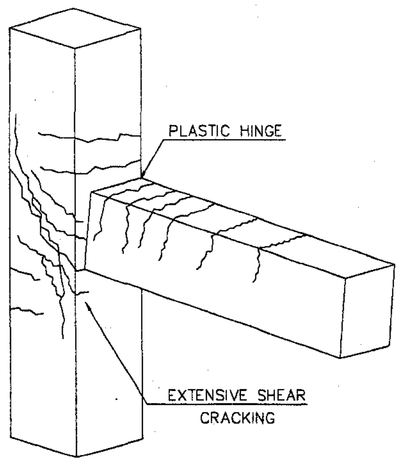


Figure 2.5. Diagrammatic representation of joint shear failure

2.2.3 Previous Studies on Monotonically Loaded Joints

This section summarizes some research studies conducted on monotonically loaded exterior beam-column joints including the experimental programs from which the database for this research was obtained.

2.2.3.1 Research by Taylor (1974)

In the 1970s, Taylor conducted a study to investigate the behavior of RC beam-column joints. This research was one of the earliest attempts to understand this behavior. The study investigated twenty six monotonically loaded exterior beam-column joints. Figure 2.6 shows the dimensions of these specimens.

Taylor grouped his specimens into seven series: P for a preliminary group that was used to develop the method of testing and the other six groups from A to F. His research focused on the effect of the following parameters on joint shear behavior: beam reinforcement ratio, joint reinforcement ratio, beam reinforcement detailing, column axial load, and beam depth. Taylor suggested the following formula to ensure that a joint has enough strength at the ultimate stage to resist applied shear loads:

$$100\rho_b = 100\left(3 + \frac{2d_c}{z_b}\right)\frac{b_c d_c}{b_b d_b}\frac{v_c \beta_1}{0.87 f_y} \tag{2.3}$$

where ρ_b is the limiting steel ratio of the beam, v_c is the nominal shear stress for the column, d_c is the effective depth of the column, b_c is the column width, d_b is the effective depth of the beam, b_b is the beam width, z_b is the lever arm of the beam at the column face, f_y is the characteristic strength of the steel, β_1 is the redistribution factor, equal to (100 - % redistribution)/100.

Based on his study, Taylor made the following conclusions:

1. Designing the columns to carry equal moment value above and below the joint might be unsafe. It would be safer to design for 70% of the beam moment below the joint and 50% above the joint.

2. The detail of bending the beam tension reinforcement up into the column was unsatisfactory. Bending the beam tension reinforcement down into the columns significantly improves the shear capacity of the joint.

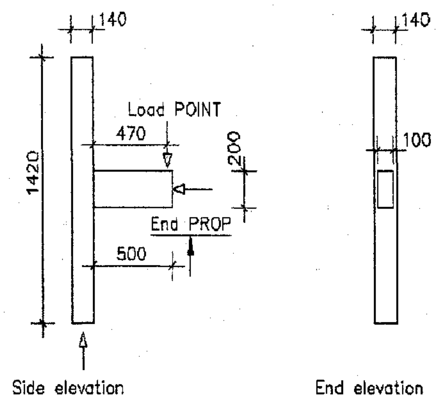
3. Presence of the joint shear reinforcement (ties) did not lead to any significant enhancement in the joint shear capacity.

4. Increasing the column axial load could lead to improvement of the behavior of the joint in the case of U-bar detail (bent back to the beam) only as using a higher column load might anchor the bar in the joint.

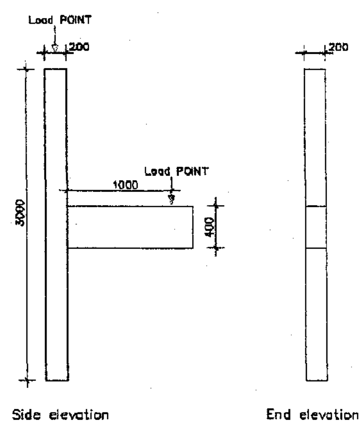
2.2.3.2 Research by Kordina (1984)

Kordina (1984) tested a program consisting of nine reinforced concrete beam-column joint specimens. Dimensions for these specimens are shown in Figure 2.7. In his study, Kordina (1984) focused on studying the following parameters: beam depth, beam reinforcement ratio, column axial load, joint reinforcement ratio, and anchorage method.

It was difficult for Kordina (1984) to draw clear conclusions from his study due to the interaction between so many parameters within small number of specimens and the novel joint strengthening methods tested such as diagonal ties in the joint.



All dimensions in mms
Figure 2.6. Dimensions of specimens tested by Taylor (1974)



All dimensions in mms
Figure 2.7 Dimensions of specimens tested by Kordina (1984)

2.2.3.3 Research by Sarsam and Phipps (1985)

Sarsam and Phipps (1985) tested an experimental program consisting of five monotonically loaded exterior reinforced concrete beam-column joint specimens. The specimens were designed in order to investigate the effect of joint shear reinforcement, joint aspect ratio and column axial load on the joint shear capacity. Figure 2.8 represents the dimensions of specimens of the experimental program. They proposed the following formulae to predict the joint shear capacity:

$$V_{ud} = V_{cd} + V_{sd} \tag{2.4}$$

where V_{ud} is the design ultimate shear capacity of the joint (N), V_{cd} is the concrete shear force resistance in joint (N), and V_{sd} is the stirrups shear force resistance (N). The concrete contribution to joint shear capacity can be calculated as follows:

$$V_{cd} = 5.08\,(f_{cu}\,\rho_c)^{0.33}\,(d_c\,/\,d_b)^{1.33}\,(1+\frac{0.29N}{A_g})^{0.5}\,d_c\,b_c \tag{2.5}$$

where f_{cu} is the cube strength of concrete (MPa), ρ_c is the column longitudinal reinforcement ratio, A_g is the gross cross- sectional area of the column at the joint (mm²), N_u is the column axial compression load at ultimate joint strength (N), d_c is the effective depth of the layer of steel furthest away from the maximum compression face in a column (mm), d_b is the effective depth of beam tension reinforcement (mm), and b_c is the width of column section at the joint (mm). The stirrups contribution to joint shear capacity can be calculated as follows

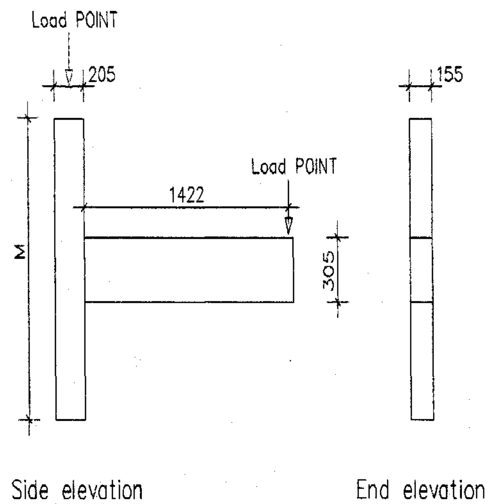
$$V_{sd} = 0.87 A_{js} f_{yv} \tag{2.6}$$

where A_{js} is the total area of horizontal shear links crossing the diagonal plane from corner to corner of the joint between the beam compression and tension reinforcement (mm^2), and f_{yv} is the tensile yield strength of the link reinforcement (MPa).

Based on their study, Sarsam and Phipps reported the following conclusions:

1. The presence of ties in the joint had no effect on the initial joint shear cracking, but had significant effect on the failure shear value for the joint.

2. Increasing the column axial load reduced the initial cracking shear load of the joint, but had no significant effect on the ultimate shear capacity of the joint.



All dimensions in mms
Figure 2.8. Dimensions of specimens tested by Sarsam and Phipps (1985)

2.2.3.4 Research by Ortiz (1993)

In 1993 Ortiz conducted an experimental program consisting of seventeen exterior reinforced concrete beam-column joints. Based on the testing results of those specimens, he proposed a strut and tie model for the behavior of the joint. Figure 2.9 represents the dimensions of specimens of the program tested by Ortiz (1993). The main conclusions of this study are:

1. The initial joint cracking is dependent on the magnitude of the axial column load.

2. Using smaller diameter of the bars in both beam and column would be more effective in transferring the force between them rather than using fewer larger bars.

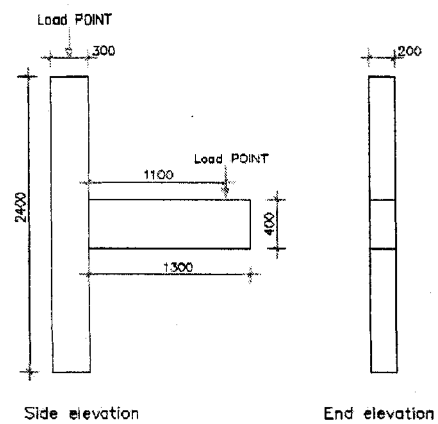
2.2.3.5 Research by Scott *et al.* (1994)

Scott *et al.* (1994) conducted an experimental program consisting of fifteen exterior reinforced concrete beam-column joints. They investigated the effect of the beam tension reinforcement ratio, the beam depth and the detailing of the beam tension reinforcement (bent down into the beam, bent up into the column, and U-bar detail). Figure 2.10 represents the dimensions of specimens tested by Scott *et al.* (1994). They used a high number of electric strain gauges (230 in every specimen) in order to measure the deformation within the main column and beam steel. They also used a prop at the end of the beam to match the effect of a supporting element (continuous beam) and reduce the side sway from happening.

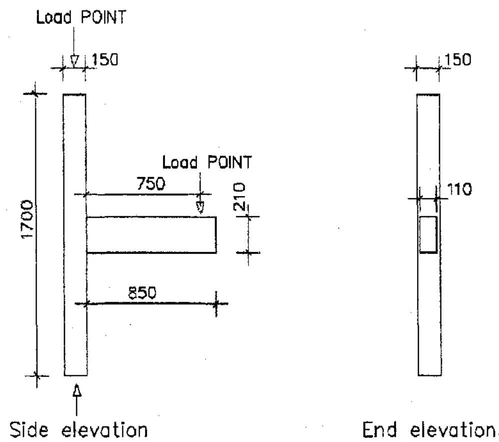
The main conclusions of Scott *et al.* (1994) are:

1. Using the detail of U-bars or bent down bars for the beam tension reinforcement significantly improves the initial joint cracking strength as these two types of details compensate for loss of bond at the bending area by increasing the bond stress over their anchorage length. This improvement in the strength does not occur when using a bent up bar detail.

2. Using the detail of U-bars or bent down bars for the beam tension reinforcement significantly enhances the ultimate shear capacity of the joint. The detail of the bent up bar does not provide this enhancement to the joint shear capacity.



All dimensions in mms
Figure 2.9. Dimensions of specimens tested by Ortiz (1993)



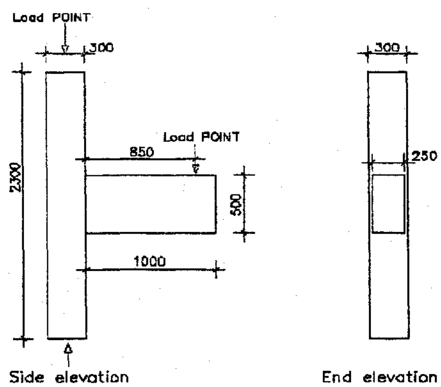
All dimensions in mms
Figure 2.10. Dimensions of specimens tested by Scott *et al.* (1994)

2.2.3.6 Research by Parker and Bullman (1997)

Parker and Bullman (1997) conducted an experimental program with twelve specimens of RC beam-column joints having dimensions shown in Figure 2.11. They grouped their specimens into four categories according to column axial loads and concrete cube compressive strengths. In their study, they investigated the effect of the column longitudinal reinforcement ratio, the joint reinforcement ratio (ties), and the beam longitudinal reinforcement ratio. Based on the results from these tests they proposed a strut and tie model.

Based on their study, Parker and Bullman (1997) drew the following conclusions:

1. Increasing column axial load increases the ultimate joint shear capacity.
2. Increasing joint shear reinforcement increases the ultimate joint shear capacity.
3. Increasing column reinforcement ratio increases the ultimate joint shear capacity.
4. The beam tension reinforcement has no significant effect on the ultimate joint shear capacity.



All dimensions in mms

Figure 2.11. Dimensions of specimens of Parker and Bullman (1997)

2.2.3.7 Research by Vollum (1998)

Vollum (1998) proposed an analytical study of a strut and tie model based on the results of previous studies conducted by Ortiz (1993), Kordina (1984), Taylor (1974), Sarsam (1985), and Scott *et al.* (1994). Vollum proposed two models for the joint depending on the presence or absence of reinforcement in the joint. Figure 2.12 represents his model for beam-column joints with stirrups. Based on this strut and tie model and the calibration he conducted using the database he collected, he proposed the following formulae for design of exterior beam-column joint under monotonic loadings:

$$V_{cd} = 0.642\beta\{1 + 0.555(2 - (h_b/h_c))\}b_{eff}h_c\sqrt{f_c'} \tag{2.7}$$

Where V_{cd} is the concrete shear force resistance in the joint (N), β=1.0 for connection with L- bars tension beam reinforcement bent downward, h_b is the thickness of the beam (mm), h_c is the thickness of the column (mm), b_e is the effective width of the joint (mm), and it is the smaller of $0.50(b_b+b_c)$ and $(b_b+0.50h_c)$ if $b_b<b_c$, and the smaller of $(b_c+0.50h_c)$ and b_b if $b_b > b_c$, and f_c' is the concrete cylindrical compressive strength (MPa).

$$V_{ud} = V_{cd} + \left(A_{sje}f_y - \alpha b_e h_c\sqrt{f_c'}\right) \tag{2.8}$$

where A_{sje} is the cross sectional area of the joint links within the top five eighths of the beam depth below the main beam reinforcement (mm^2), α is a coefficient that depends on different factors including joint aspect ratio, concrete strength, stirrup index, and the column axial load.

The main conclusions proposed by Vollum (1998) are:

1. Increasing joint reinforcement ratio improves the ultimate joint shear strength.

2. Using the detail of bent bars down for the beam longitudinal reinforcement ratio enhance the shear behavior of the joint.

Vollum limited the validity of the previous formulae by the following boundaries:

$$V_j < 0.97b_{eff}h_c\sqrt{f_c'}\left(1 + 0.555(2 - (h_b/h_c))\right) \tag{2.9}$$

$$V_j < 1.33b_{eff}h_c\sqrt{f_c'} \tag{2.10}$$

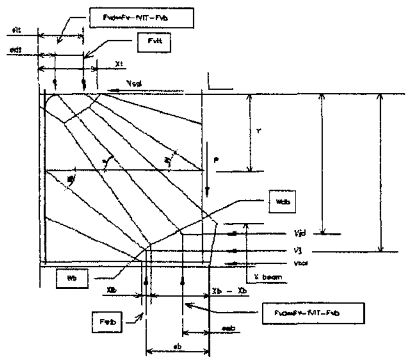


Figure 2.12 Strut and tie model proposed by Vollum (1998)

2.2.3.8 Research by Hamil (2000)

Hamil (2000) conducted his study to investigate forty nine monotonically loaded beam-columns. In his study, he investigated the following parameters: the joint shear reinforcement, the compressive strength of the concrete, the detailing arrangement of the beam tension reinforcement, the joint aspect ratio, the tie anchorage, the beam steel plate anchorage, and the joint shear plates.

Hamil divided his program into categories to provide enough specimens for every investigated parameter. Figure 2.13 shows the layout of Hamil's program. The work of Hamil was essential in the development of this thesis due to the large number of specimens.

Based on his study, Hamil (2000) proposed the following conclusions:

1. The initial joint cracking load is not influenced by the quantity or positioning of joint ties.

2. The initial joint cracking load is not influenced by the use of high strength concrete.

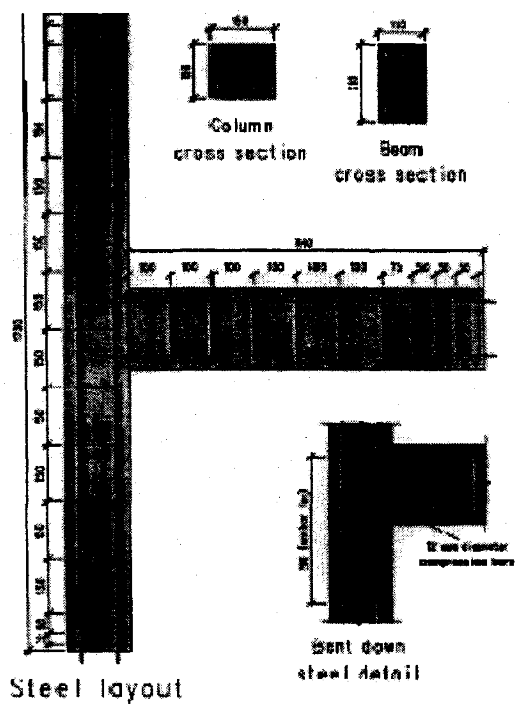
3. The initial joint cracking strength is not influenced by the joint aspect ratio.

4. The use of U-bars detail for beam tension reinforcement reduced the ultimate shear capacity by a value up to 17% due to the transferring of the entire beam's load into the joint region.

5. The use of the detail of bent down bars for the beam tension reinforcement allowed the full capacity of the joint to be reached as the anchor leg transferred a large proportion of the beam's load into the lower column region.

6. The ultimate joint capacity can be significantly improved by the use of joint ties. Also the placement of joint ties around the center of the joint increases the shear capacity of concrete.

7. The placement of joint ties around the level of the beam tension steel reduced the potential of anchorage-induced joint failure by giving confinement to the concrete beneath the top bend of this rebar.



All dimensions in mms

Figure 2.13 Dimensions of specimens tested by Hamil (2000)

2.2.3.9 ACI-ASCE Committee 352 (2002)

Based on the loading conditions for the joint and the anticipated deformations of the connected frame members, ACI-ASCE Committee 352 classifies beam-column joints into two categories: Type 1 and Type 2. Type 1 joints are designed to satisfy ACI 318 (2008) except for seismic provisions (gravity load case), while Type 2 joints are designed to have sustained strength under deformation reversals into the plastic range (seismic loading case).

The ACI-ASCE Committee 352 (2002) proposed the following equation to calculate the shear strength of exterior monotonically loaded beam-column joints:

$$V_n = 0.083\gamma\sqrt{f_c'}b_j h_c \tag{2.11}$$

where V_n is the nominal shear strength of Type 1 joints (N), f_c` is the concrete cylindrical compressive strength (MPa), h_c is the depth of the column in the direction of joint shear being considered (mm), b_j is the effective width of the joint (mm); it is defined as the smaller value of:

$$\frac{b_b + b_c}{2} \tag{2.12a}$$

$$b_b + \sum(mh_c + 2) \tag{2.12b}$$

$$b_c \tag{2.12c}$$

where m = 0.50 for the database case, and γ is 15 for the case of Type 1 joints and planar exterior joints. Thus the formula will be:

$$V_n = 1.245\sqrt{f_c'}b_j h_c \tag{2.13}$$

This formula neglects the steel contribution of both joint and beam reinforcement to the shear capacity of the joint. It also neglects the effect of the column axial stress.

2.2.3.10 Bakir and Boduroğlu (2002a)

Bakir and Boduroğlu (2002a) investigated the effect of several parameters that affect the strength of monotonically loaded reinforced concrete beam-column joints. These parameters include concrete compressive strength, column reinforcement ratio, beam longitudinal reinforcement, beam reinforcement detailing, joint stirrups ratio, joint aspect ratio, column load, and the vertical anchorage length and the radius of bend. Based on their model they proposed the following formula:

$$V_{ud} = \frac{0.71\beta\gamma\left(\frac{100A_{sb}}{b_b d_b}\right)^{0.4289}}{\left(\frac{h_b}{h_c}\right)^{0.61}}\left(\frac{b_c + b_b}{2}\right)h_c\sqrt{f_c'} + \alpha A_j f_y \tag{2.14}$$

where $\beta = 1$ for joints with L- bars bent downward detail for beam tension reinforcement, $\gamma = 1.37$ for inclined bars in the joint and $\gamma = 1.0$ for others, A_{sb} is the steel area of the beam, b_b is the width of the beam, α is a factor depending on the joint stirrup ratio and is equal to 0.664 for joints with low reinforcement ratio (up to 0.003), 0.60 for joints with medium reinforcement ratio (between 0.003 and 0.0055), and 0.37 for joints with high reinforcement ratio (more than 0.0055), A_j is the cross sectional area of the joint links (mm^2).

The main conclusions of their study are:

1. Column axial load significantly affects the failure mode.

2. Increasing the column axial load improves the ultimate joint shear capacity.

3. Joints with medium and high amounts of stirrups are unlikely to exhibit anchorage failure.

4. The use of low reinforcement ratio in the joint increases the risk of exhibiting a shear failure in the joint.

5. For a better behavior of the joint, only L-bars bent down detail for beam tension reinforcement should be used.

2.3 Cyclically Loaded Interior Beam-Column Joints

Interior beam-column joints have a great importance in reinforced concrete structures. The effect of cyclic loading conditions on interior joints is much higher than the effect of monotonic loading. The reasons behind this are:

1. Larger forces can be generated on the joint for the case of cyclic loading depending on the direction of forces (the ground motion) rather than the monotonic loading case.

2. According to Chopra (2007), the amount of lateral displacement of a RC structure when subjected to cyclic loading is almost twice the amount of the displacement generated by the same force value when applied monotonically to the joint.

2.3.1 Behavior of Joints Subjected to Seismic Loading

In any reinforced concrete frame subjected to seismic loading, beams and columns experience flexure and shear forces. These forces are transformed into higher shear values acting on the joint and they might cause a shear failure in the joint. This type of failure has severe damaging results on the structure. Figure 2.14 represents the distribution of these forces within the region.

The strut and truss model proposed by Paulay (1989) can be used for both monotonically loaded exterior beam-column joints and cyclically loaded interior beam-column joints. As shown in Figure 2.15, two mechanisms are used for the transfer of loads through the joint. The first one is the strut mechanism which accounts for the concrete contribution to the shear strength of the joint. In this mechanism, a single concrete compression strut is used to transfer the shear forces through the joint. The second one is the truss mechanism which accounts for the contribution of joint shear reinforcement in transferring the shear forces through the joint. In this mechanism, the load is transferred through a steel tie represented by the joint shear stirrups. To ensure the presence of the tie mechanism, a strong and uniform bond stress distribution along the beam and column reinforcement should exist.

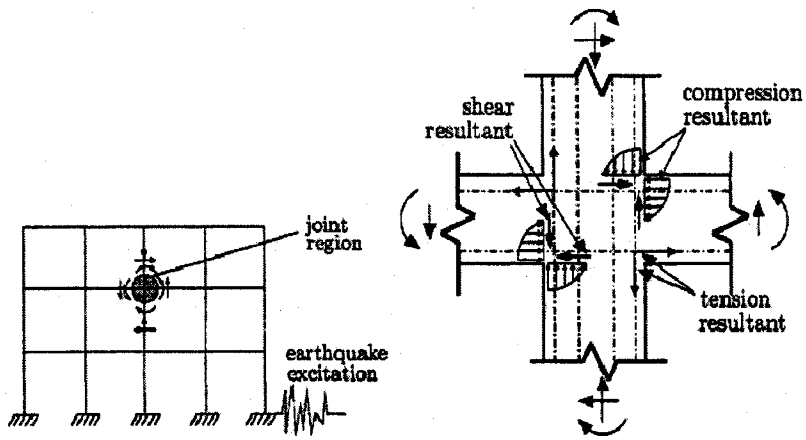
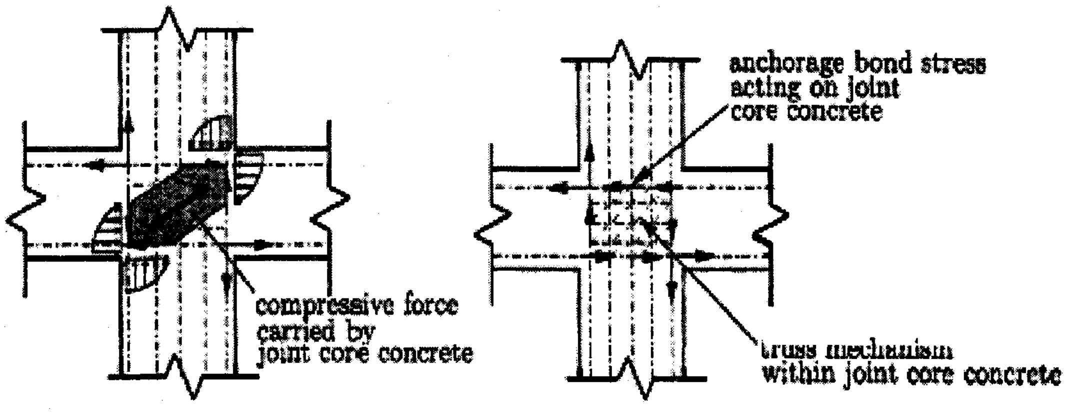


Figure 2.14. Seismic loading in a reinforced concrete beam-column joint region

Several studies were conducted to investigate the lever arm between tension and compression in the joint. While Paulay (1989) assumed that the arm of the tension and compression forces is constant, Shiohara (2001) limited this assumption to a constant bond stress in the beam tension reinforcement. But actually the bond stress can never be constant because the bond stress in the reinforcement changes with the different loading levels.



(a) Compressive force within the joint (b) Anchorage forces within the joint

Figure 2.15 Strut and truss model proposed by Paulay (1989) for interior beam-column joints

2.3.2 Modes of Failure of Cyclically Loaded Joints

Modes of failure of interior cyclically loaded joints are very similar to that of exterior monotonically loaded joints previously discussed in this chapter. The possible modes of failure that could happen in the cyclic loading joint are either joint shear failure or bar slippage of the beam reinforcement or beam bending failure. In the case of cyclically loaded joints, an interaction could happen between the joint shear failure and the beam reinforcement slippage. This combined mode of failure can be divided into two categories; brittle failure (failure occurs before the beam tension reinforcement yield), and ductile failure (failure occurs after the beam tension reinforcement yield).

2.3.3 Previous Studies on Cyclically Loaded Joints

Behavior of interior cyclically loaded beam-column joints is very complicated. Several mechanisms control this behavior including: yielding of reinforcing steel, shearing across concrete crack surfaces, cracking of concrete, crushing of concrete and closing of concrete cracks under load reversal. Understanding these mechanisms and the interaction between them helps produce an accurate modeling of the joint response. Since these mechanisms interact with each others in a complicated way, it is very hard to introduce a perfect model to represent the behavior of the joint. Several studies were introduced to simulate this performance using finite element models including Will *et al.* (1972), Noguchi (1981), Pantazopoulou and Bonacci (1994), Hwang and Lee (2000), Lowes and Altoontash (2003), Elmorsi *et al.* (2000). Several studies also proposed experimental programs. The main experimental studies conducted to investigate interior cyclically loaded joints are summarized as follows:

2.3.3.1 Research by Higashi and Ohwada (1969)

Higashi and Ohwada (1969) conducted an experimental program consisting of seventeen one-third scale interior beam-column joints. Four of these specimens were excluded from the dataset used in this study because they had transverse beams. Six other specimens were excluded because they suffered column reinforcement yielding. The results of this study showed the importance of the joint shear demand in determining the mode of failure especially in determining the type of failure in the joint.

2.3.3.2 Research by Durrani and Wight (1982)

Durrani and Wight (1982) proposed an experimental program consisting of six full-scale interior beam-column joint specimens. Three of these specimens had slabs and were excluded from the dataset used in the artificial intelligence model. The specimens were designed so as to investigate the effect of the joint reinforcement on the shear capacity. The researchers concluded that increasing the joint shear reinforcement ratio and reducing spacing between the stirrups increase the shear capacity of the joint.

2.3.3.3 Research by Otani *et al.* (1984)

They proposed a half-scale experimental program consisting of twelve interior beam-column joints. Six of these specimens had transverse beams and were excluded from the dataset. They investigated the effect of the column longitudinal reinforcement and the joint shear reinforcement ratio on the shear capacity of the joint. The main conclusion was that increasing the joint shear reinforcement ratio increases the shear capacity of the joint. They also concluded that the column interior longitudinal reinforcement does not have a significant effect on the shear capacity of the joint.

2.3.3.4 Research by Kitayama *et al.* (1987)

Kitayama *et al.* (1987) studied the effect of the beam longitudinal reinforcement diameter on the shear capacity of the joint. The program tested four half scale interior beam-column joints. They suggested some limitations on the beam longitudinal reinforcement diameters, and the minimum joint shear reinforcement. They also concluded that the effect of the column axial stress on the joint shear capacity does not appear before an axial stress of 0.50.

2.3.3.5 Research by Endoh *et al.* (1991)

This program consisted of four interior beam-column joints. The main parameter investigated in this study was the concrete compressive strength. The authors concluded that the joint shear strength of light weight concrete is less than that of normal weight concrete. They also concluded that the strength loss in the peak regime of the load deformation response is greater in the light weight concrete as opposed to the normal weight concrete.

2.3.3.6 Research by Joh *et al.* (1991)

They proposed a half-scale experimental program consisting of thirteen interior beam-column joints. Only six specimens were included in the dataset of this thesis. The others were excluded either because they were designed so that the beam yielding occurs away from the beam-column interface, or they were eccentric beam-column joint connections. Based on their program they concluded that using a large number of joint stirrups improves the behavior of the joint by reducing the potential for beam reinforcement slippage. They also concluded that beam stirrups do not significantly improve the slippage of beam longitudinal reinforcement from the joint.

2.3.3.7 Research by Noguchi and Kashiwazaki (1992)

Noguchi and kashiwazaki (1992) tested an experimental program of five interior beam-column joints. Based on the study, they concluded that the concrete compressive strength does not affect the maximum joint shear strength, and that the effect of the joint shear stirrups can only appear at large drift levels. They determined this drift level to be at a drift angle of 1/50 rad.

2.3.3.8 Research by Oka and Shiohara (1992)

Oka and Shiohara (1992) tested an experimental program consisting of eleven 1/4 scale interior beam-column joints. All of these specimens were included in the dataset of this thesis except for two specimens that had slabs attached to them. They concluded that there is proportional nonlinear relationship between the concrete compressive strength and the joint shear strength. They also concluded that increasing the beam longitudinal reinforcement increases the joint shear capacity.

2.3.3.9 Research by Hayashi *et al.* (1994)

They proposed a program of eleven half-scale interior beam-column joints. They used the results from this program to construct a numerical model exploring the relation between the bond strength and the longitudinal beam reinforcement slippage from the joint. The main conclusion of this study was that both beam bar bond and joint shear stress demand play significant roles in joint failure under earthquake loading.

2.3.3.10 Research by Teraoka *et al.* (1994)

This program consisted of seven half-scale interior beam-column joints. All of them were used in the dataset except for one specimen that had steel plates welded to the joint reinforcement to increase the confinement forces on the joint core. Based on the study, the researchers proposed an empirical formula to predict the ultimate shear strength of the joint panel.

2.3.3.11 Research by Walker (2001)

He proposed a half-scale experimental program consisting of twelve specimens. This study investigated the effect of the shear stress and the load history on the joint behavior. Walker concluded that to improve the performance of the joint, the drift demand should be limited to 1.5% and the shear stress should be less than $10\sqrt{f_c'}$ psi where f_c' represents the compressive strength of concrete.

2.3.3.12 Research by Zaid (2001)

Zaid (2001) tested his half-scale experimental program consisting of four interior beam-column joints. One of these four specimens was excluded from the dataset of this research because the beam longitudinal reinforcement was bent down diagonally in the joint. This study confirmed the results obtained from the study conducted by Shiohara (2001); the lever arm distance between the tension and compression forces in the joint is not constant and changes with the change of the bond stress due to loading stages.

2.3.3.13 Research by Attaalla and Agbabian (2004)

Attaalla and Agbabian (2004) conducted their study to investigate the characteristics of shear deformation inside the beam-column joint core. They proposed a model to predict the expansions of beam-column joint core in the horizontal and the vertical directions. The experimental program consisted of four interior reinforced concrete beam-column joints. One of the specimens was excluded because it contained steel fiber instead of steel bars in the joint stirrups. They concluded that assuming a proportional relationship between joint shear capacity and the square root of the concrete compressive strength is not accurate for the case of high strength.

2.3.3.14 Research by ACI-ASCE Committee 352 Formula (2002)

According to the ACI-ASCE Committee 352 (2002), the cyclically loaded joints are categorized as Type 2. Type 2 joints are the ones designed to have sustained strength under deformation reversals into the plastic range (seismic loading case).

The ACI-ASCE Committee 352 (2002) proposes a general formula for the design of beam-column joints and bases on the type of joint the factors of the formula vary. The general formula can is as follows:

$$V_n = 0.083\gamma\sqrt{f_c'}b_j h_c \tag{2.15}$$

where V_n is the nominal shear strength of Type 2 joints, f_c` is the concrete cylinder strength (MPa), h_c is the depth of the column in the direction of joint shear being considered (mm), b_j is the effective width of the joint (mm), it is defined as the smaller value of:

$$\frac{b_b + b_c}{2} \tag{2.16a}$$

$$b_b + \sum(mh_c + 2) \tag{2.16b}$$

$$b_c \tag{2.16c}$$

where $m = 0.50$ for the case of no eccentricity between the beam and column centerlines, $\gamma = 15$ for Type 1 exterior planar joints (database case). Accordingly the formula becomes:

$$V_n = 1.245\sqrt{f_c'}b_j h_c \tag{2.17}$$

2.3.3.15 Research by Architectural Institute of Japan (1998)

Most of the recommendations provided in the Japanese design guidelines for the cyclically loaded beam-column joints are based on studies conducted by Aoyama (1993) on the behavior of cyclically loaded beam-column joints. According to his study, it is stated that there are two earthquake design methods. The first is the strength design, in this method the structure is designed to sustain large lateral load resistance capacity. The second method is the ductility design method, where the structure is designed to have a large inelastic deformation capacity. It is very important for any structure not to suffer brittle failure by dissipating the energy of the earthquake through plastic hinges formed in the beams. This actually represents the strong column weak beam theory. This theory states that the structure should be designed to have a stronger column than the beam to increase the dissipation of energy, and to ensure the simultaneous formation of plastic hinges in the beams. Based on his study, the Architectural Institute of Japan (1998) provides the following formula for calculating of the shear capacity of cyclically loaded beam-column joints.

$$V_u = k * \emptyset * F_j * b_j * D \tag{2.18}$$

where $k = 1$, $\emptyset = 0.85$, $F_j = 0.80*(f_c')^{0.70}$ (MPa), D is the column depth, b_j = effective column width. This leads the formula to be

$$V_u = 0.68 * (f_c')^{0.70} * b_j * D \tag{2.19}$$

CHAPTER 3

ARTIFICIAL INTELLIGENCE MODELING AND METHODOLOGY

3.1 Introduction

Science is built upon facts, as a house is built of stones; but an accumulation of facts is no more a science than a heap of stones in a house (Henri Poincaré, 1905).

As humans we are always looking for a way to understand the behavior of our brains. We try to understand how these tiny cells in our brains can sense, understand, interact, and manage our survival in this complicated world. Artificial intelligence (AI) is one of the newer sciences created by man. Its origin is considered to be in the late forties in the field of molecular biology in order to improve the capability of studying specific properties and was later applied to the study of other sciences.

AI currently encompasses a variety of subfields, ranging from general purpose areas such as learning and understanding to such specific assignments as diagnosing diseases proving mathematical theories, playing chess, and even writing poetry. AI systematizes and mechanizes intellectual tasks and is therefore potentially related to any area of human intellectual activity. In this sense it is truly a worldwide field.

3.2 What is Artificial Intelligence?

The expression "Artificial Intelligence" is very flexible and it can refer to several meanings. Therefore, it is difficult to give a precise definition of AI. Table 1 shows eight definitions of AI previously introduced by several studies (Haugeland, 1985; Bellman, 1978; Charniak and McDermott, 1985; Winston, 1992; Kurzweil, 1990; Rich and Knight, 1991; Poole *et al.*, 1998; and Nilson, 1998). These definitions vary along two main categories, the ones on the top are concerned with thought processes and reasoning, whereas the ones on the bottom describe behavior. The definitions on the left measure success in terms of accuracy of human performance, while the ones on the right measure an ideal concept of intelligence, which we will call rationality. These definitions can be the best way to describe artificial intelligence.

3.3 Artificial Intelligence and Engineering

Many engineering problems can be solved using AI techniques and the technology has been used successfully in several complex applications. The automotive and aerospace industries have extensively used both robotic technology and expert systems in their manufacturing processes. The potential for using artificial intelligence in civil engineering and the construction industry is unlimited. However, its use in such applications is still in the early developing stages.

For many decades, investigating the properties of concrete structures (material/structure) was basically a trial to study a single aspect based on the available notices. However, in reality several aspects and parameters mutually interact. Studying a single parameter without accounting for the overall context of the problem is not very accurate. But with the existence of AI techniques, it became very applicable to build a numerical system that represents the whole context of the investigated problem. This study explores the feasibility of using artificial intelligence in modeling properties of beam-column joints with the aim of a true understanding of the factors governing the behavior of this critical zone in any concrete structure and the share of each factor on this behavior. In the following sections, a brief description about the two artificial intelligence techniques used in this study which are the genetic algorithms (GAs) and the artificial neural networks (ANNs) will be given.

3.3.1 Genetic Algorithms

Genetic algorithms are search procedures that use the mechanics of natural selection and natural genetics. The genetic algorithm, first developed by John H. Holland in the 1960's, allows computers to solve difficult problems. It uses evolutionary techniques, based on functional optimization and artificial intelligence to develop a solution.

The sequences of operation of genetic algorithms are as follows: first a population of solutions to a problem is developed. Then, the better solutions are recombined with each other using some special procedures to form a new set of solutions. Finally the new sets of solutions are used to replace the unqualified original solutions and the process is repeated (El-Chabib, 2006).

A genetic algorithm is used in computing to find true or approximate solutions to optimization and search problems. Genetic algorithms are a particular class of evolutionary algorithms that use techniques inspired by evolutionary biology such as inheritance, mutation, selection, and crossover (Russell and Norvig, 2003).

Genetic algorithms are implemented as a computer simulation in which a population of abstract representations (called chromosomes) of candidate solutions (called individuals) to an optimization problem evolves toward better solutions. Traditionally, solutions are represented in binary as strings of 0s and 1s, but other encodings are also possible. The evolution usually starts from a population of randomly generated individuals and happens in generations. In each generation, the fitness of every individual in the population is evaluated, multiple individuals are stochastically selected from the current population (based on their fitness), and modified (recombined and possibly mutated) to form a new population. The new population is then used in the next generation of the algorithm. Commonly, the algorithm terminates when either a maximum number of generations has been produced, or a satisfactory fitness level has been reached for the population. If the algorithm has terminated due to a maximum number of generations, a satisfactory solution may or may not have been reached (Russell and Norvig, 2003). Figure 3.1 presents the steps of typical genetic algorithm model.

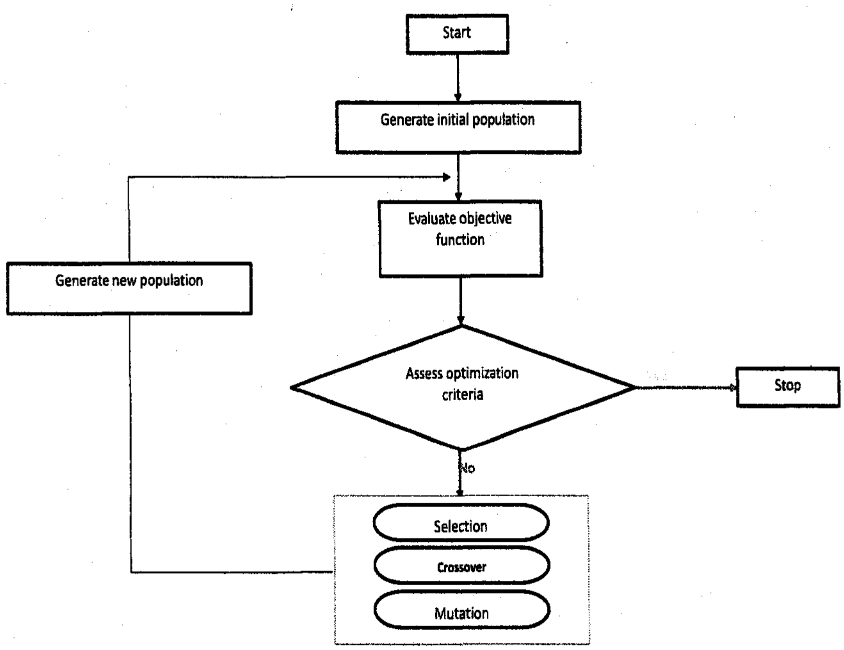


Figure 3.1. Steps of typical genetic algorithms proposed by El-Chabib (2006).

3.3.2 Neural networks approach

An artificial neural network (ANN) is an information processing model that is inspired by the way biological nervous systems, such as the brain, process information. The key element of this model is the narrative structure of the information processing system. It is composed of a large number of highly interconnected processing elements (neurons) working in harmony to solve specific problems. ANNs, like people, learn by example. An ANN is configured for a specific application, such as pattern recognition or data classification, through a learning process. Learning in biological systems involves adjustments to the synaptic connections that exist between the neurons. This is true of ANNs as well.

3.3.2.1 Advantages of Neural Networks

Neural networks, with their remarkable ability to derive meaning from complicated or imprecise data, can be used to extract patterns and detect trends that are too complex to be noticed by either humans or other computer techniques. A trained neural network can be thought of as an "expert" in the category of information it has been given to analyze. This expert can then be used to provide projections given new situations of interest and answer "what if" questions (Russell and Norvig, 2003). Other advantages include:

- Adaptive learning: An ability to learn how to do tasks based on the data given for training or initial experience.
- Self-Organization: An ANN can create its own organization or representation of the information it receives during learning time.
- Real Time Operation: ANN computations may be carried out in parallel, and special hardware devices are being designed and manufactured which take

advantage of this capability (Russell and Norvig, 2003).

The most important part in building an ANN-based model is the training process provided that reliable and comprehensive database is available. The training process consists of providing the network with training patterns each containing input and output vectors, each unit in the first hidden layer compute an output and transmitted to units in the second layer; and So on until the network compute an output. The computed output is compared with the provided one and the difference (error) is calculated. The error is than back propagated to the network to adjust the connection strengths between units; this phenomenon is repeated until the error between predicted and provided outputs reaches a desired assigned value.

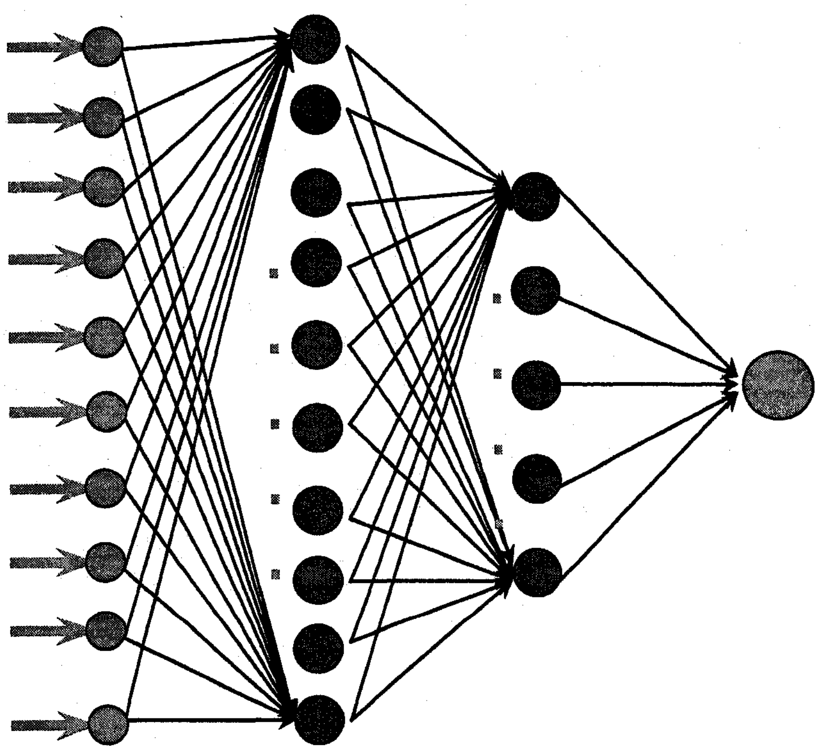


Figure 3.2. Architecture of neural network proposed by El-Chabib (2006)

Table 1. Some definitions of artificial intelligence, Russell and Norvig (2003)

Systems that think like humans	Systems that think rationally
"The exciting new effort to make computers think…machines with minds, in the full and literal sense." (Haugeland, 1985) "{The automation of} activities that we associate with human thinking, activities such as decision-making, problem solving, learning…." (Bellman, 1978)	"The study of mental faculties through the use of computational models." (Charniak and McDermott, 1985). "The study of the computations that make it possible to perceive, reason, and act." (Winston, 1992).
Systems that act like humans	Systems that act rationally
"The art of creating machines that preform functions that require intelligence when performed by people." (Kurzweil, 1990) "The study of how to make computers do things at which, at the moment, people are better." (Rich and Knight, 1991)	"Computational intelligence is the study of the design of intelligence agents." (Poole *et al.*, 1998) "AI…is concerned with intelligent behavior in artefacts." (Nilson, 1998)

CHAPTER 4

EVALUATING SHEAR CAPACITY OF RC EXTERIOR BEAM-COLUMN JOINTS UNDER MONOTONIC LOADING USING ARTIFICIAL NEURAL NETWORKS

4.1 Background

The shear behavior of monotonically loaded exterior beam-column joints is influenced by various key parameters. The effect of each of these parameters has some limit of uncertainty due to the complexity of the joint behavior. Consequently, existing shear design formulae for joints produce varying results depending on the parameters accounted for in each respective formula. This study utilizes artificial neural networks (ANNs) to investigate the effect of some of the basic parameters (joint shear reinforcement, concrete compressive strength, beam longitudinal reinforcement ratio, joint aspect ratio, and column axial stress) on the shear strength of monotonically loaded exterior beam-column joints. For the purpose of this study, the ANN model was developed and trained using an experimental database collected from published literature on monotonically loaded exterior beam-column joints. This database was then used by the ANN model to predict the shear capacity of the joint. To validate the accuracy of the proposed ANN model, a comparison was conducted between the model results and those obtained from other proposed design formulae: ACI-ASCE Committee 352 (2002), Sarsam and Phipps (1985), Vollum (1998), Bakir and Boduroğlu (2002a). Results indicate that the ANN model provides a better prediction of the shear capacity of monotonically loaded exterior beam-column joint than the other previously published formulae. For the sake of evaluation of the existing design formulae and production of ANN model, a database was collected from the literature from different experimental programs.

4.2 Previously Proposed Formulae and Equations

In this chapter, four formulae were investigated and evaluated using the selected database. A detailed preview of these formulae was presented in chapter 2 of this thesis. These formulae are:

1- ACI-ASCE Committee 352 Formula (2002)

2- Design Equation of Sarsam and Phipps (1985)

3- Design Equation of Vollum (1998)

4- Design Equation of Bakir and Boduroğlu (2002a)

4.3 Artificial Neural Network Approach

Artificial neural network is one of the most applicable artificial intelligence techniques used in the optimization of civil engineering problems. Multi-layer perceptron (MLP) networks have been known as the most widely used ANNs in these optimization processes. They are able to map a given input(s) into desired output(s), and they can detect hidden and complex behavioral trends of engineering problems by learning through the database used to train the system.

The structure of MLP networks consists of an input layer which represents the investigated parameters in the network, an output layer which represents the final result of the network or the behavior under investigation, and some hidden layers that the operation of optimization undergoes. Each layer contains a number of processing elements that are fully or partially connected to the elements in successive layers. The strength of the bond between processing elements is a numerical value called the weight of the connection.

The optimization process in ANNs can be expressed as the operation of detecting the optimum weights such that the network can predict an accurate value for the output within the database range.

4.4 Experimental Database

The most important aspect in the success of a neural network is the learning database on which the system is trained. Therefore it is imperative to train a network model on a comprehensive database to capture the actual embedded relationships between the parameters of the input and output layers. The objective of this chapter is to detect the relationships between the different parameters being considered and their effect on the shear capacity of exterior beam-column joints under monotonic loadings.

In this study, shear capacity of this joint type is investigated using a database consisting of 88 concrete beam-column connections collected from published literature (Taylor, 1974; Hoekstra, 1977; Bosshard and Menn, 1984; Sarsam and Phipps, 1985; Ortiz, 1993; Scott *et al.*, 1994; Parker and Bullman, 1997; Kordina, 1984; Hamil, 2000; Hegger *et al.*, 2003).

The quality of the network was improved by imposing several limitations on specimens in the database used by the ANN model. Only specimens failing due to joint shear were used, with no beams in the transverse direction. Specimens with high strength concrete, or reinforcement welding into the joint were omitted. The database was formatted into groups of input vectors, each vector representing one of the investigated parameters in the study. The output vector represents the shear capacity of the joint. Table 2 represents the database range of the parameters investigated in the study.

Table 2. The parameters range for the investigated database for exterior beam-column joints under monotonic loading.

Parameter	*Minimum*	*Maximum*
Volumetric Reinforcement Ratio (%)	0	2.77
Concrete Cylindrical Compressive Strength (MPa)*	20	70
Beam Longitudinal Reinforcement Ratio (%)	0.65	3.50
Column Axial Stress (MPa)	0	16
Joint Aspect Ratio (%)	1	2

* Cylindrical compressive strength (f_c')= 0.80 Cube compressive strength (f_{cu})

4.5 ANN Model

To predict the shear strength of monotonically loaded beam-column joints, an ANN was constructed with the following components: an input layer, an output layer and four hidden layers. The input layer contains five variables representing the common shear design parameters of the reinforced concrete beam-column joint (volumetric reinforcement ratio, concrete compressive strength, beam reinforcement ratio, joint aspect ratio, and column axial stress). The output layer includes one unit representing the shear capacity, V_n and the hidden layers consist of different amounts of processing units. Full bonding connections were used between the processing elements and the elements in other consecutive layers.

The software used in this model is MATLAB (2007). This software is commonly used for the optimization process of engineering problems. This software was coded to divide the given database into training and testing groups to increase the accuracy of the model and give a better understanding of the effect of each parameter in the output layer. Figure 4.1 represents the architecture of the proposed model.

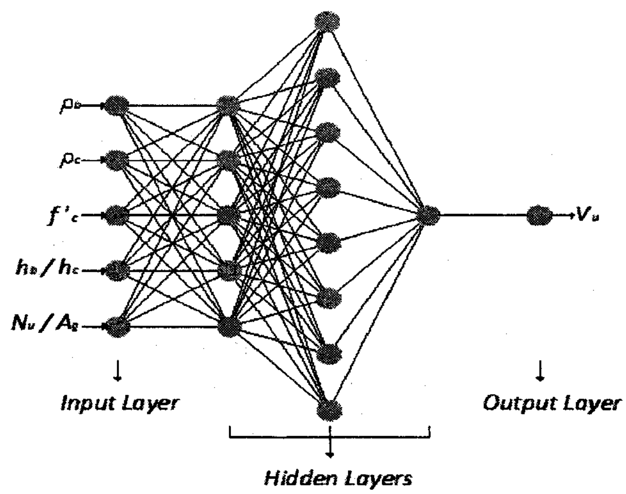


Figure 4.1 Architecture of artificial neural network model

4.6 Results and Discussions

4.6.1 Formulae Verification

To consider an ANN successful, it must be able to accurately predict output values for input values within the range of the database used in the training and the testing process. To evaluate the accuracy of the proposed network, a comparison was held between the network predicted outputs which represent the shear capacity and those calculated using the formulae of ACI-ASCE 352 (2002), Sarsam and Phipps (1985), Vollum (1998), and Bakir and Boduroğlu (2002a). The performance of each model was evaluated based on both the ratio of measured to predicted (or calculated) shear strength (V_m/V_p), and the average absolute error (*AAE*) calculated using the following equation:

$$AAE = \frac{1}{n}\sum\frac{|V_m - V_p|}{V_m} \times 100 \tag{4.10}$$

The average value, the standard deviation (*STDV*), and coefficient of variation (*COV*) for V_m/V_p, and the average absolute error (*AAE*) of the ANN model and other joint shear calculation formulae investigated are listed in Table 3.

Table 3. Performance of different formulae for the calculation of shear strength of RC exterior beam-column joints under monotonic loading.

Method	*AAE (%)*	$V_{measured} / V_{predicted}$		
		Average	*STDV*	*COV*
ACI-ASCE 352 (2002)	*37*	*0.765*	*0.21*	*19.74*
Sarsam and Phipps (1985)	*26*	*1.13*	*0.37*	*32.50*
Vollum (1998)	*28*	*1.43*	*0.25*	*17.50*
Bakir and Boduroğlu (2002a)	*24*	*1.31*	*0.3208*	*15.85*
ANN	*12.25*	*0.975*	*0.167*	*17.152*

In the following sections a detailed discussion of the results of the different investigated formulae is presented:

4.6.1.1 ACI-ASCE Committee 352 Formula (2002)

Figure 4.2 represents a plot of the actual experimental shear strength values against the calculated ones using the ACI-ASCE Committee 352 formula. This formula neglects the influence of the joint aspect ratio, the column axial stress, and the contribution of both joint and beam reinforcements to the shear capacity of the joint. Using the selected data for this study and the actual capacity of the specimens obtained from the experimental programs results, the average absolute error *AAE* for this formula is 37%, which is significantly high, and the *STDV* for V_m/V_p of this formula is 0.21. It is recommended that this formula should not be used to estimate the shear capacity of beam-column joints due to its lack of accuracy and the overestimation of the shear strength. It should rather be used to estimate the minimum shear strength of the joint based on concrete properties and joint dimensions only. As shown in Figure 4.2, the value for V_m/V_p for most of the database lay underneath the unity line which means that the formula over estimates the shear capacity of the joint.

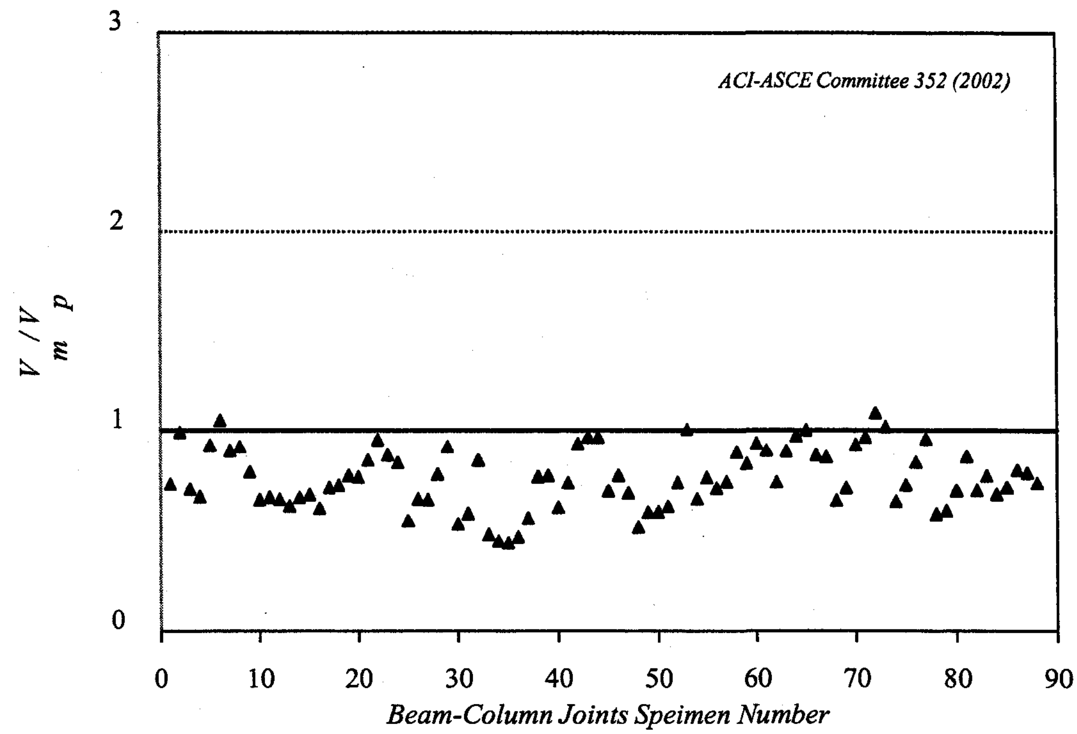


Figure 4.2. Performance of the equation proposed by ACI-ASCE Committee 352 (2002) in calculating the shear capacity of beam-column joints

4.6.1.2 Design Equation of Sarsam and Phipps (1985)

Although this formula accounts for several parameters in calculation of shear capacity of the joint, the *AAE* for this formula is still significantly high at 26%, and the *STDV* for this formula is 0.37. This is mainly due to the limited number of specimens that were initially used to derive the formula. Furthermore, this formula accounts for 87% of the joint stirrups in resisting shear forces in the joint. Results obtained from Ortiz (1993) showed that the effective stirrups are the ones located above the beam compressive chord and below the beam tension reinforcement. This formula also accounts for the column longitudinal reinforcements; the effect of this parameter on shear strength of beam-column joints was neglected by Ortiz (1993) and Bakir and Boduroğlu (2002a). Figure 4.3 represents a plot of the actual experimental shear strength values against the calculated ones using the formula proposed by Sarsam and Phipps (1985). The scatter of the database specimens is random around the unity line leading to the high *AAE* mentioned above.

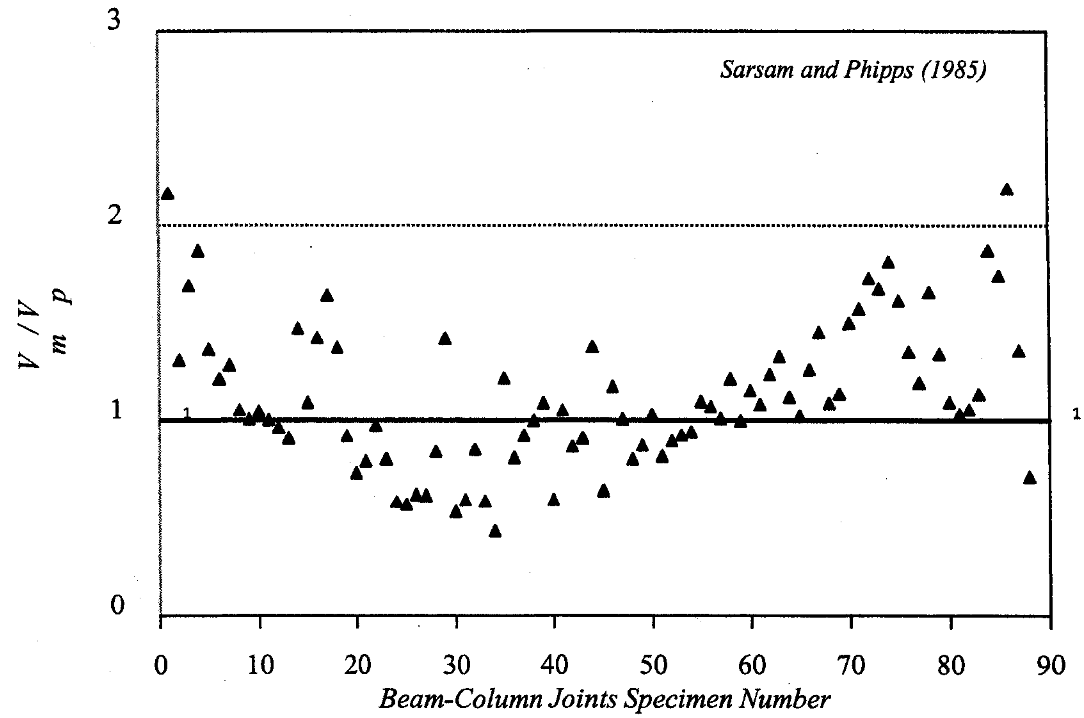


Figure 4.3. Performance of the Equation proposed by Sarsam and Phipps in calculating the shear capacity of beam-column joints

4.6.1.3 Design Equation of Vollum (1998)

In this formula Vollum (1998) accounted for the effect of joint aspect ratio and both concrete and steel contribution to the shear capacity of the joint. Statistical analysis performed on his formula indicated that the *AAE* for the selected data is 28% with a *STDV* of 0.25. It is estimated that the reason behind the inaccurate results obtained from this formula is neglecting the effect of the beam reinforcement ratio. Another reason is that the database used by Vollum (1998) to derive this formula had limited range of parameters which make it unable to predict an accurate shear capacity for wider range of parameters. Figure 4.4 represents a plot of the actual experimental shear strength values against the calculated ones using this formula. Most of the database lay above the unity line which means that in most of the cases the formula underestimated the shear capacity of the joint.

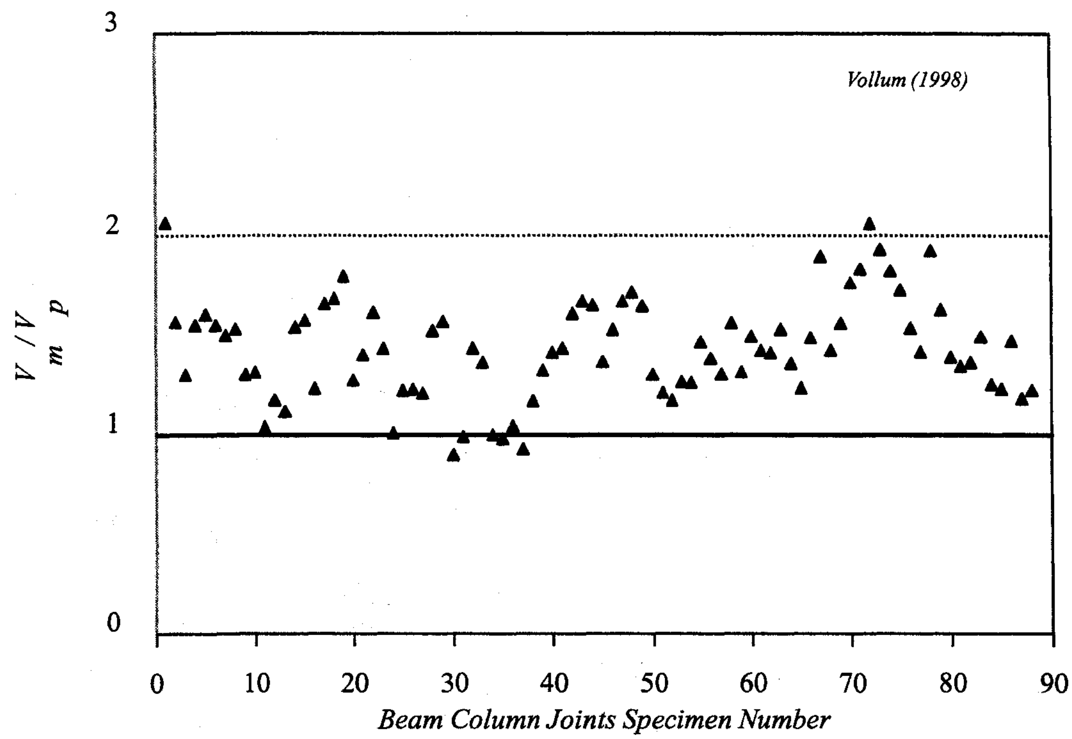


Figure 4.4. Performance of the equation proposed by Vollum (1998) in calculating the shear capacity of beam-column joints

4.6.1.4 Design Equation of Bakir and Boduroğlu (2002a)

Although this formula accounted for several key parameters affecting the performance the joint, the accuracy of formula when used to calculate the capacity of joints of the database is not high. The *AAE* for this formula was 24% which is significantly high and may be attributed to the formula's overestimation of beam longitudinal reinforcement effect on joint shear capacity. Figure 4.5 represents a plot of the actual experimental shear strength values against those calculated using the Bakir and

Boduroğlu (2002a) formula. Most of the database lay above the unity line which means that in most of the cases the formula underestimated the shear capacity of the joint.

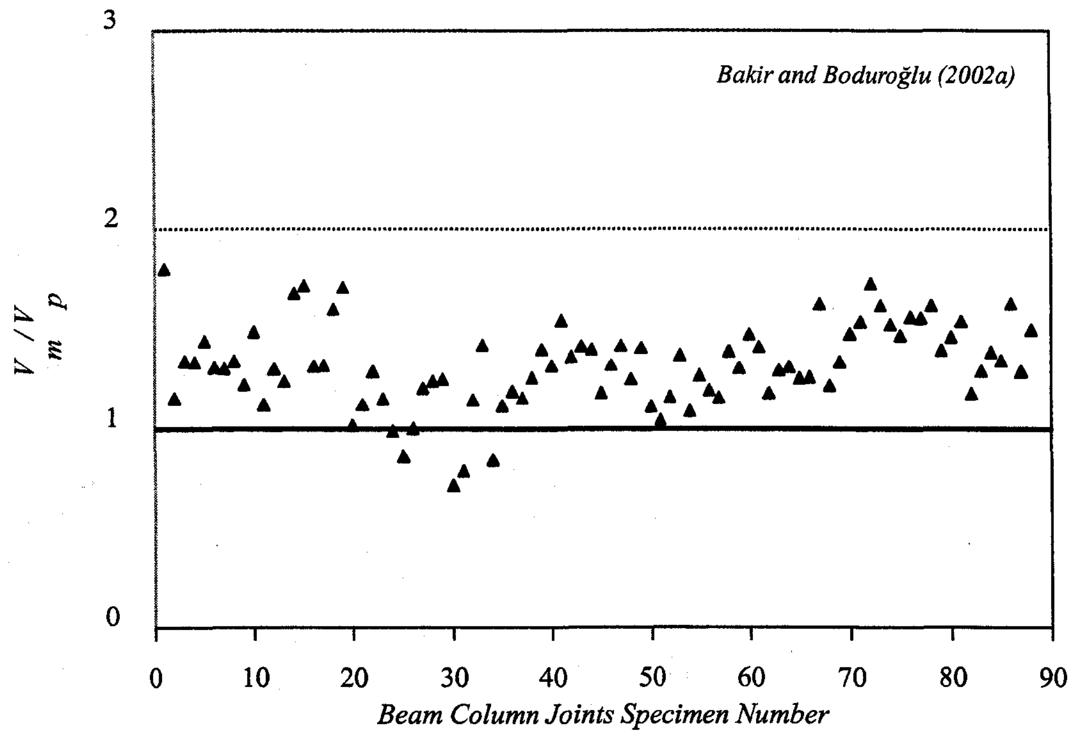


Figure 4.5. Performance of the equation proposed by Bakir and Boduroğlu (2002a) in calculating the shear capacity of beam-column joints

4.6.1.5 Proposed ANN Model

The proposed model from the ANN analysis produced more accurate outputs for predicting the shear capacity of joints than the other investigated formulae. Figure 4.6 shows that this model reduced the *AAE* among the actual and the predicted values to a small value (12%). This is the smallest value among the formulae for calculating shear capacity of beam-column joints. The small value of *AAE* ensures the accuracy of selecting the investigated parameters as the key factors governing the shear behavior of joints. Furthermore, the *STDV* of this formula is about 0.16 which is acceptable scatter for such a case. Evaluation of the effect of each of the investigated parameters is conducted using all proposed formulae in the following section.

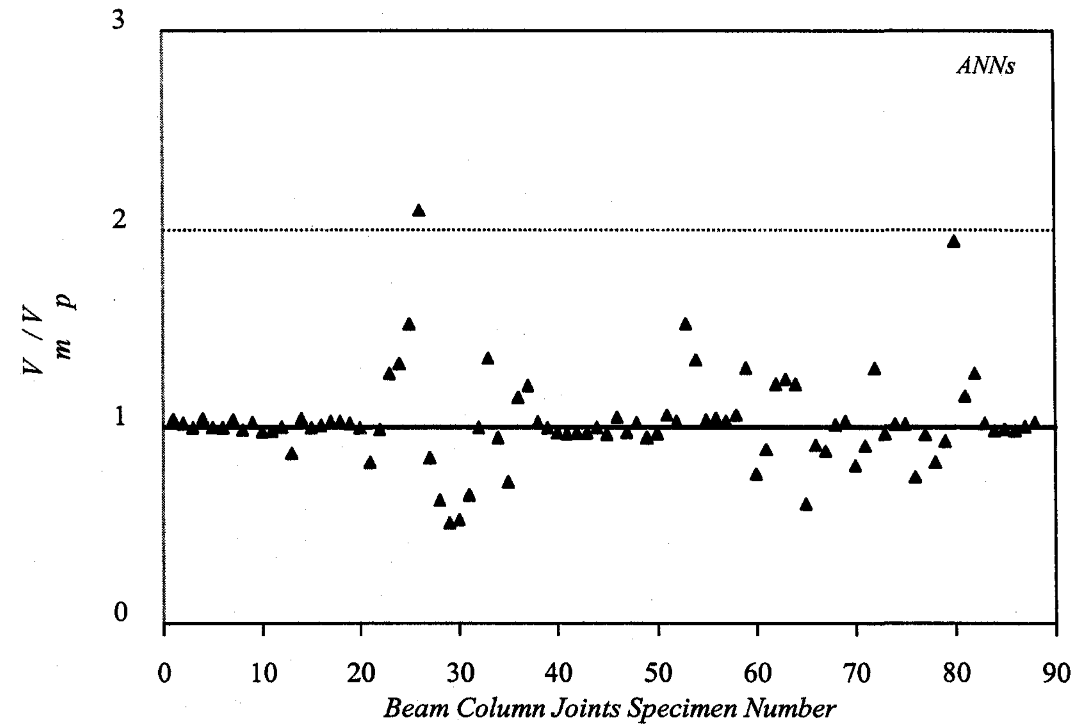


Figure 4.6. Performance of ANNs model in calculating the shear capacity of beam-column joints

4.6.2 Parametric Study on the Effect of Basic Shear Design Parameter

4.6.2.1 Effect of Beam Longitudinal Reinforcement Ratio

An analysis was conducted to study the effect of beam longitudinal reinforcement ratio on the shear strength of beam-column joints using the different proposed formulae and the ANN model. The specimen labeled C_9 tested by Scott *et al.* (1994) was used to evaluate this parameter. Figure 4.7 represents the parametric study of this factor using different proposed formulae and the ANN model. The formulae proposed by ACI-ASCE 352 (2002), Sarsam and Phipps (1985) and Vollum (1998) did not account for the beam longitudinal reinforcement ratio ρ_b as an effective factor on the joint shear capacity. The proposed formula by Bakir and Boduroğlu (2002a) and the ANN model predicted that an increase in the beam longitudinal reinforcement ratio increases the shear capacity. This result is justified because the increase the beam reinforcement ratio increases the confinement of the joint and improves the force transfer between the beam and the column leading to increase in the joint capacity.

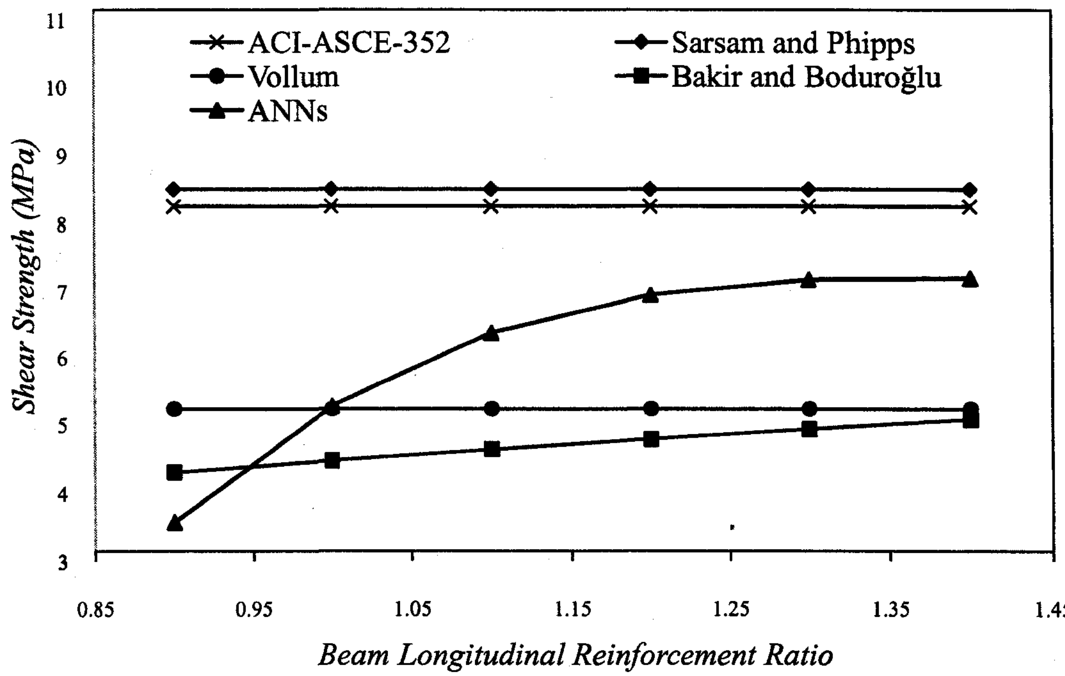


Figure 4.7. Effect of beam longitudinal reinforcement ratio on joint shear capacity

4.6.2.2 Effect of Joint Shear Reinforcement Ratio

The model proposed by the ANN concurs with the formula proposed by Sarsam and

Phipps (1985), Vollum (1998), and Bakir and Boduroğlu (2002a) in the effect of the joint shear reinforcement. According to the ANN model, increasing the joint shear reinforcement ratio increases the shear capacity of the joint. These formulae accounted for the joint stirrups by different values. Sarsam and Phipps (1985) assumed that almost all the stirrups in the joint will yield before the joint fails and therefore they accounted for 87% of the stirrups within the joint. Bakir and Boduroğlu (2002a) specified the contribution of the shear stirrups to the joint shear capacity based on the value of the reinforcement ratio in the joint. Vollum (1998) and the ANN model predicted similar contribution of the stirrups to the joint capacity. Generally, the stirrups that actually resist the shear forces in the joint should be the ones placed between the concrete compression chord and the beam tension reinforcement. Figure 4.8 represents the parametric study of this factor related to different proposed formulae and the ANN model.

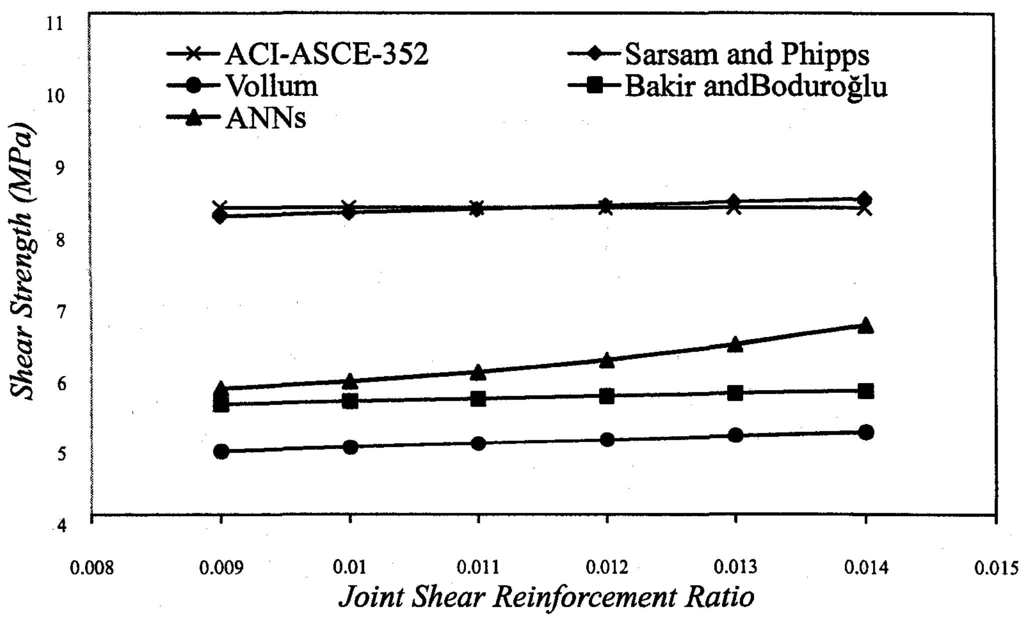


Figure 4.8. Effect of joint shear reinforcement ratio on joint shear capacity

4.6.2.3 Effect of Concrete Compressive Strength

Concrete compressive strength is an important factor in any reinforced concrete element. Increasing concrete strength leads to improvement in properties of all elements of the structure. Investigation of the effect of the concrete compressive strength with the studied formulae is shown in Figure 4.9. For all the formulae and also the ANN model, increasing the concrete compressive strength increases the shear capacity of the joint. The relationship between the concrete strength and the joint shear capacity is almost the same between the different proposed formulae except for that of Sarsam and Phipps (1985) which used the concrete cube strength to express the effect of concrete on shear capacity of the joint. All other formulae including the ANN model assumed a proportional relationship between the square root of the concrete compressive strength and the joint shear capacity.

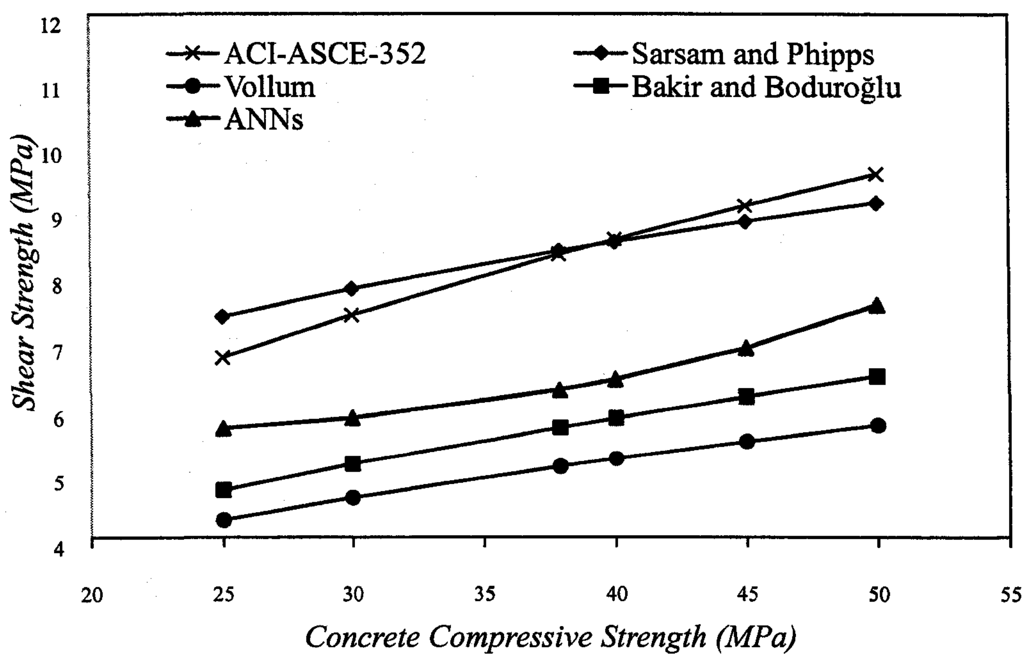


Figure 4.9. Effect of concrete compressive strength on joint shear capacity

4.6.2.4 Effect of Column Axial Stress

The proposed ANNs model concurs with the formulae proposed by ACI-ASCE Committee 352 (2002), Vollum (1998), and Bakir and Boduroğlu (2002a) in the effect of column axial stress on the joint shear capacity. They conclude that the column axial stress has no affect on the shear capacity of the joint as shown in Figure 4.10. Sarsam and Phipps (1985) gave the only formula that accounted for the effect of the column axial stress on the joint shear capacity and according to them increasing the axial stress improves the joint shear capacity.

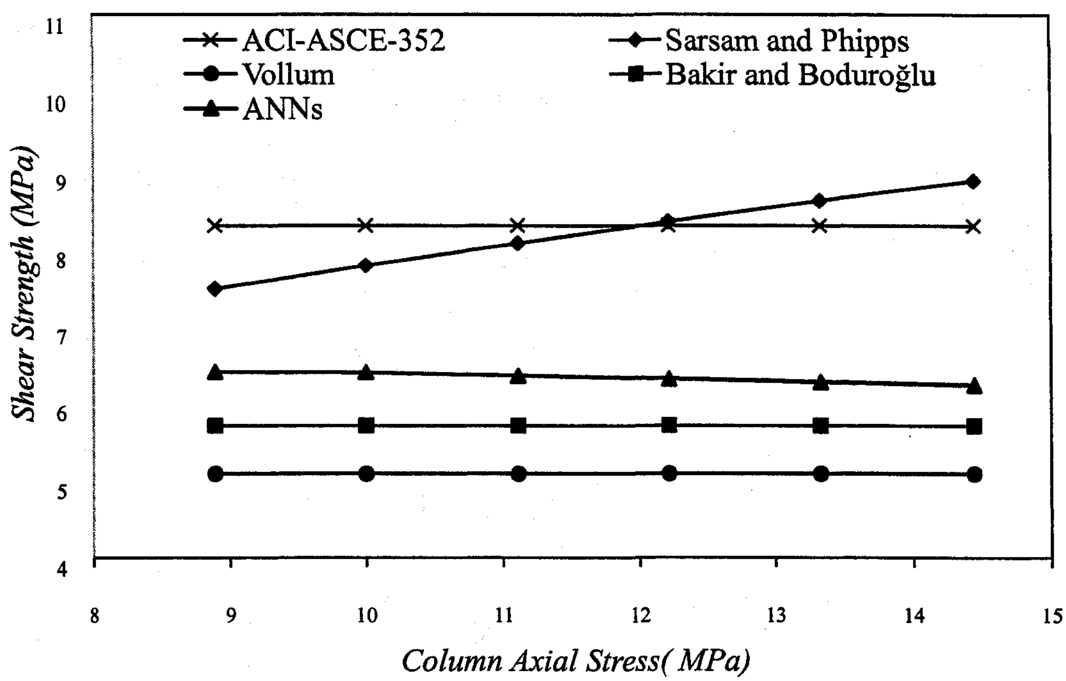


Figure 4.10. Effect of column axial stress on joint shear capacity

4.6.2.5 Effect of Joint Aspect Ratio

Figure 4.11 represents the parametric study of the effect of the joint aspect ratio on joint shear capacity. The investigation showed that according to the ANN model, the joint aspect ratio had no effect on the shear capacity of the joint. ANN and the formulae proposed by Sarsam and Phipps (1985) and by Bakir and Boduroğlu (2002a) were similar in this regard. The formulae proposed by the ACI-ASCE Committee 352 (2002) and by Vollum (1998) indicated that an increase in the joint aspect ratio leads to an increase in the joint shear capacity. It is recommended to further investigate the effect of the joint aspect ratio on the joint shear capacity to eliminate the interaction between this parameter and the other ones.

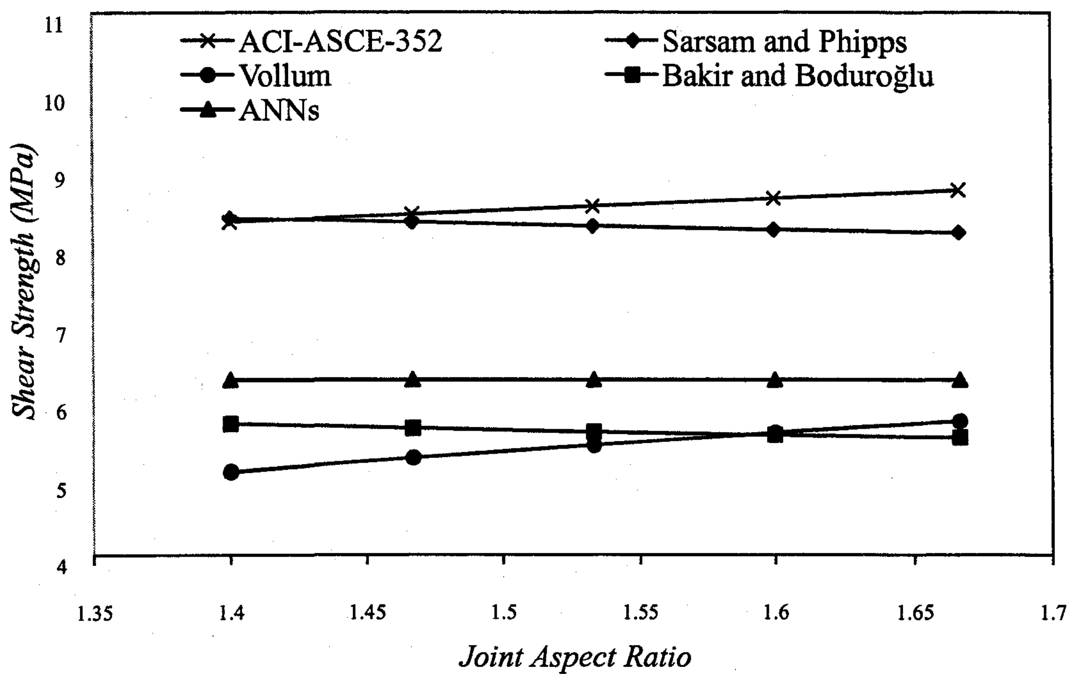


Figure 4.11. Effect of joint aspect ratio on joint shear capacity

CHAPTER 5

EVALUATING SHEAR CAPACITY OF RC EXTERIOR BEAM-COLUMN JOINTS UNDER MONOTONIC LOADING USING GENETIC ALGORITHMS

5.1 Background

Beam-column joint is a very important element for the integrity of frame structures. However, beam-column joints are prone to shear failure as a result of the straining actions transferring between framing beams and columns through the joint. In the last four decades, several studies have been conducted on the shear capacity of monotonically loaded beam-column joints. Different formulae have been proposed to calculate the shear capacity of the monotonically loaded exterior beam-column joints. Several parameters are known to have significant effect on the shear capacity of the joint namely: joint shear reinforcement ratio, concrete compressive strength, beam tension longitudinal reinforcement ratio, joint aspect ratio and column axial stress. The contribution of each of these parameters noticeably varies for each of these formulae. This chapter investigates the accuracy of some of the proposed formulae for calculating the shear capacity of the joint (ACI-ASCE Committee 352, 2002; Sarsam and Phipps, 1985; Vollum, 1998; Bakir and Boduroğlu, 2002a). Genetic algorithms approach is used to optimize the performance of these formulae. An improved shear design equation is also proposed using the same approach.

Results indicate that the current shear design equations are inaccurate in calculating the shear capacity of the exterior beam-column joints subjected to monotonic loading.

5.2 Experimental Database

The database used for this study was selected from the available experimental research programs in the literature. A total number of 88 specimens were selected for the study. The selection process was based on special criteria: concrete compressive strength was limited to 70 MPa, only planar specimens with no transverse beams were considered, and specimens with bent up L-bar tension beam reinforcement detail were excluded. The used database is the same one used in chapter 4 of this thesis.

As a powerful optimization tool, the GAs toolbox attached in the computer software MATLAB (2007) was used in the error minimization process. The parameters of the tool box, such as mutation, crossover, victorization, and population input techniques, allow error reduction and improve the accuracy of the investigated design formulae.

5.3 Optimization of Formulae

To consider the optimization process successful, the modified formulae should be able to predict the values of beam-column joint shear capacity more accurately than the original formulae. The performance of the optimization process of each formula was evaluated based on both the ratio of measured to predicted (or calculated) shear strength (V_m/V_p), and the average absolute error (*AAE)* calculated using the following equation:

$$AAE = \frac{1}{n}\sum\frac{|V_m - V_p|}{V_m}x100 \tag{5.1}$$

The standard deviation (*STDV*), and coefficient of variation (*COV*) for V_m/V_p, and the average absolute error (*AAE*) of the GA model and other shear calculation methods investigated are listed in Table 4. In the following sections, a detailed description of the optimization process conducted on each of the previously mentioned formulae is presented.

Table 4. Performance of GA model and shear design methods considered in this study in predicting the shear strength of exterior monotonically loaded beam-column joints

Method	Pre-Optimized				Post-Optimized			
	AAE (%)	$V_{measured} / V_{predicted}$			AAE (%)	$V_{measured} / V_{predicted}$		
		Average	STDV	COV		Average	STDV	COV
ACI-ASCE 352 (2002)	37	0.765	0.21	19.74	16.50	1.06	0.15	19.74
Sarsam and Phipps (1985)	26	1.13	0.37	32.50	15	0.99	0.17	17.70
Vollum (1998)	28	1.43	0.25	17.50	28	1.43	0.337	23.50
Bakir and Boduroğlu (2002)	24	1.31	0.3208	15.85	23	1.219	0.31	20.70
GA	--------	--------	--------	--------	12	1.023	0.165	16.14

5.3.1 Design Equation of ACI-ASCE Committee 352 (2002)

Based on the loading conditions for the joint and the anticipated deformations of the connected frame members, ACI-ASCE Committee 352 classifies beam-column joints into two categories: Type 1 and Type 2. Type 1, joints are designed to satisfy ACI 318

(2008) except for seismic provisions (gravity load case); while Type 2 joints are designed to have sustained strength under deformation reversals into the plastic range (seismic loading case). In this study, only Type 1 joints were investigated.

The ACI-ASCE Committee 352 (2002) proposed the following equation to calculate the shear strength of monotonically loaded exterior beam-column joints:

$$V_n = 0.083\gamma\sqrt{f_c'}b_j h_c \tag{5.2}$$

where V_n is the nominal shear strength of Type 1 joints, f_c` is the concrete cylinder strength (MPa), h_c is the depth of the column in the direction of joint shear being considered (mm), b_j is the effective width of the joint (mm), it is defined as the smaller value of:

$$\frac{b_b + b_c}{2} \tag{5.3a}$$

$$b_b + \sum(mh_c + 2) \tag{5.3b}$$

$$b_c \tag{5.3c}$$

where $m = 0.50$ for the case of no eccentricity between the beam and column centerlines, $\gamma = 15$ for Type 1 exterior planar joints (database case). Accordingly the formula becomes:

$$V_n = 1.245\sqrt{f_c'}b_j h_c \tag{5.4}$$

This formula neglects the influence of the joint aspect ratio, the column axial stress, and the contribution of both joint and beam reinforcements to the shear capacity of the joint. It also neglects the effect of the column's axial stress. Using the selected data for this study and knowing the actual capacity of the specimens obtained from the experimental programs results, the average absolute error *AAE* for this formula is 37%, which is significantly high, and the *STDV* for V_m/V_p of this formula is 0.21. It is recommended that this formula should not be used to estimate the shear capacity of beam-column joints due to its lack of accuracy. It should rather be used to estimate the minimum shear strength of the joint based on concrete properties and joint dimensions.

An optimization process was conducted on this formula using the genetic algorithms approach. The formula was modeled in the following format and then calibrated using the database of the study:

$$V_n = C_1 f_c'^{C_2} b_j h_c \tag{5.5}$$

The results of the optimization process indicated that the best obtained values for C_1 and C_2 are 0.852 and 0.513 respectively. The formula will then become:

$$V_n = 0.852 f_c'^{0.513} b_j h_c \tag{5.6}$$

The *AAE* for this formula is approximately 16.50%. Figure 5.1 represents a plot of calculated values versus actual experimental values for both the original and the optimized formulae with the optimized formula showing less scatter. This is also clear from the smaller value of *STDV* which is 0.151 for the modified formula. Although the optimization process reduced the *AAE* to almost half the value produced by the pre-optimized formula, still the modified formula is not reliable since it does not account for the important parameters.

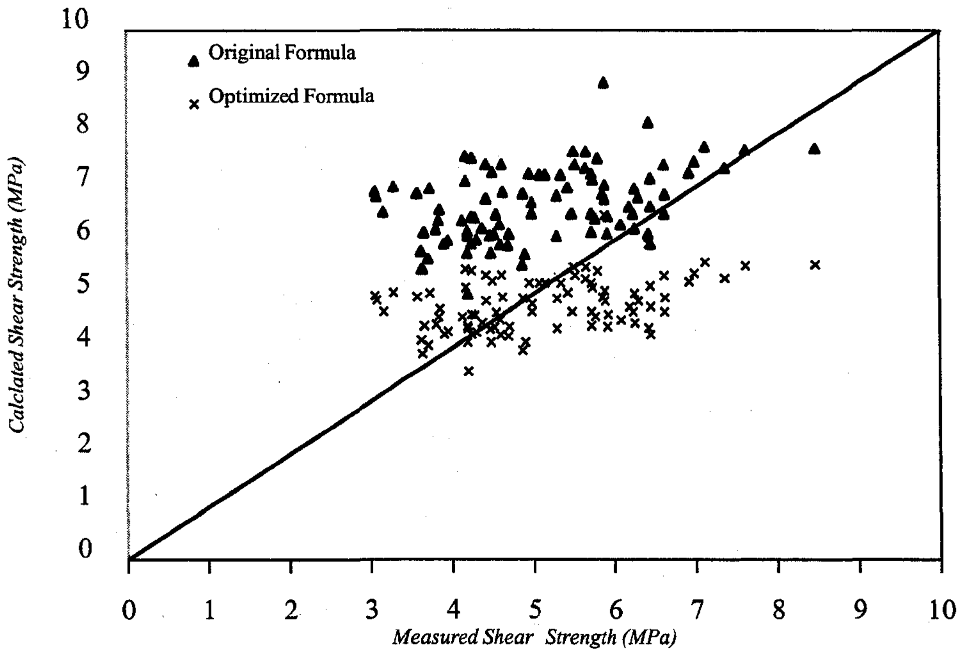


Figure 5.1. Response of original and optimized formulae of ACI-ASCE 352 equations in calculating the shear capacity of the joint

5.3.2 Design Equation of Sarsam and Phipps (1985)

Based on their experimental program, Sarsam and Phipps (1985) proposed the following equations for the shear design of monotonically loaded exterior beam-column joints:

$$V_{ud} = V_{cd} + V_{sd} \tag{5.7}$$

where V_{ud} is the design ultimate shear capacity of the joint (N), V_{cd} is the design shear force resistance of concrete in joint (N), V_{sd} is the design link shear force resistance (N).

$$V_{cd} = 5.08\ (f_{cu}\ \rho_c)^{0.33}\ (d_c\ /\ d_b)^{1.33}\ (1 + \frac{0.29N}{A_g})^{0.5}\ d_c\ b_c \tag{5.8}$$

where f_{cu} is the cube strength of concrete (MPa), A_{so} is the area of the layer of steel furthest away from the maximum compression face in a column (mm²), A_g is the gross cross- sectional area of the column at the joint (mm²), N_u is the column axial compression load at ultimate joint strength (N), d_c is the effective depth of the layer of steel furthest away from the maximum compression face in a column (mm), d_b is the effective depth of beam tension reinforcement (mm), b_c is the width of column section at the joint (mm).

$$V_{sd} = 0.87 A_{js} f_{yv} \tag{5.9}$$

where A_{js} is the total area of horizontal shear links crossing the diagonal plane from corner to corner of the joint between the beam compression and tension reinforcement (mm^2), and f_y is the tensile strength of the link reinforcement (MPa).

Although this formula accounts for several parameters in the calculation of shear capacity of the joint, the *AAE* for this formula is still significantly high at 26%, and the *STDV* for this formula is 0.37. This is mainly due to the limited number of specimens that were initially used to calibrate this formula. Furthermore, this formula suggests that 87% of the amounts of the joint stirrups are the effective ones in resisting shear forces in the joint, which may be higher than the actual value of the effective stirrups in the joint. Results obtained from Ortiz (1993) showed that the effective stirrups are the ones located above the beam compressive chord and below the beam reinforcement. This formula also accounts for the column longitudinal reinforcements; the effect of this parameter on shear strength of beam-column joints was neglected by Ortiz (1993) and Bakir and Boduroğlu (2002a). For the purpose of optimization, the formula could be written as:

$$V_{ud} = C_1 (f_{cu}\, \rho_c)^{C_2} (d_c / d_b)^{C_3} (1 + \frac{C_4 N}{A_g})^{C_5} d_c\, b_c + C_6 A_{js} f_y \tag{5.10}$$

Optimizing this formula led to modify the original coefficients of Sarsam and Phipps (1985) equation into the following expression:

$$V_{ud} = 6.28\, (f_{cu}\, \rho_c)^{0.144} (d_c / d_b)^{0.38} (1 + \frac{0.07 N}{A_g})^{0.10} d_c\, b_c + 0.72\, A_{js}\, f_y \tag{5.11}$$

The formula in its new form is more compatible with the database. An error percentage of 15% is significantly low taking into consideration experimental variation. Figure 5.2 represents a plot of calculated values versus actual experimental values for both the original and the optimized formulae with the optimized formula having less scatter. This is also clear from the smaller value of *STDV* which is 0.17 for the modified formula.

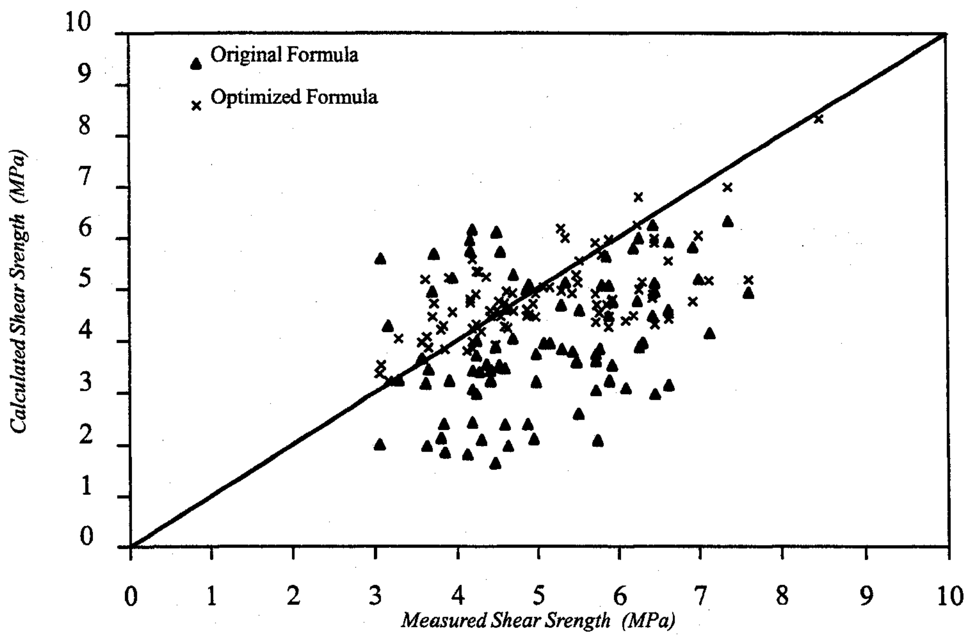


Figure 5.2. Response of original and optimized formulae of Sarsam and Phipps (1985) equations in calculating the shear capacity of the joint

5.3.3 Design Equation of Vollum (1998)

Vollum (1998) conducted a study to investigate the shear behavior of the joint using a strut and tie model. A modified formula for calculating the shear capacity of the joint was developed as follows:

$$ {}_{cd} = 0.642\beta\{1 + 0.555(2 - (h_b/h_c))\}b_{eff}h_c\sqrt{f_c'} \tag{5.12}$$

where β = 1.0 for connection with L- bars tension beam reinforcement bent downward, h_b is the thickness of the beam (mm), h_c is the thickness of the column (mm), b_{eff} is the effective width of the joint (mm), and it is the smaller of $0.50(b_b+b_c)$ and $(b_b+0.50h_c)$ if $b_b < b_c$, and the smaller of $(b_c+0.50h_c)$ and b_b if $b_b > b_c$.

$$V_{ud} = V_{cd} + \left(A_{sje}f_y - \alpha b_{eff}h_c\sqrt{f_c'}\right) \tag{5.13}$$

where A_{sje} is the cross sectional area of the joint links within the top five eighths of the beam depth below the main beam reinforcement (mm²), α is a coefficient that depends on different factors including joint aspect ratio, concrete strength, stirrup index, and the column axial load and is taken 0.20.

In this formula Vollum accounted for the effect of joint aspect ratio and both concrete and steel contribution to the shear capacity of the joint. Statistical analysis performed on his formula indicated that the *AAE* for the selected data is 28% with a *STDV* = 0.25. It is believed that the reason behind the inaccurate results obtained from this formula is neglecting the effect of the beam reinforcement ratio. Another reason is that the database used by Vollum (1998) to derive his formula was limited which makes it unable to predict an accurate shear capacity for a wider range of parameters. The optimized formula was formatted in the following form:

$$V_{ud} = C_1\{1 + C_2(2 - (h_b/h_c))\}b_{eff}h_c f_c'^{C_3} + A_{sje}f_y - C_4 b_{eff}h_c f_c'^{C_3} \tag{5.14}$$

Optimizing this formula resulted into the following formula:

$$V_{ud} = 0.42\{1 + 0.95(2 - (h_b/h_c))\}b_{eff}h_c f_c'^{0.46} + A_{sje}f_y - 1.07 b_{eff}h_c f_c'^{0.46} \tag{5.15}$$

The optimization process conducted on this formula did not lead to noticeable improvement in the *AAE*. This means that the contribution of the parameters accounted for in the formula is reasonably accurate yet not realistic since the *AAE* stands unchanged at 28%. Figure 5.3 represents a plot of calculated values versus actual experimental values for both the original and the optimized formulae with the optimized formula having higher scatter with $STD = 0.337$.

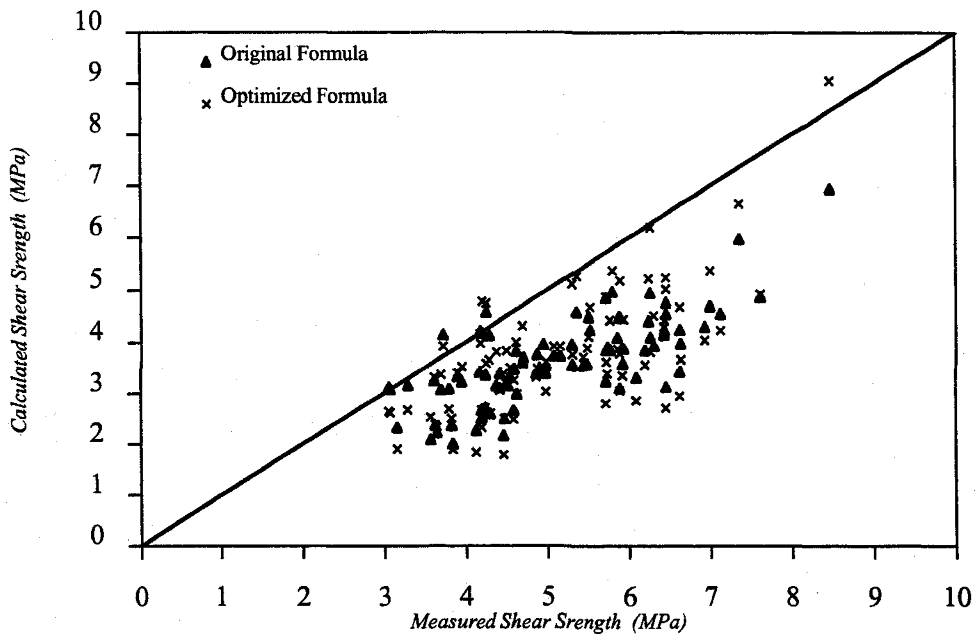


Figure 5.3. Response of original and optimized formulae of Vollum (1998) equations in calculating the shear capacity of the joint

5.3.4 Design Equation of Bakir and Boduroğlu (2002)

Bakir and Boduroğlu (2002a) investigated the effect of several parameters that affect the strength of monotonically loaded exterior beam-column joints. These parameters include concrete cylinder strength, beam longitudinal reinforcement, beam reinforcement detailing, joint stirrups ratio, joint aspect ratio, and joint dimensions. Based on their model they proposed the following formula:

$$ud = \frac{0.71\zeta\lambda\left(\frac{100A_{sb}}{b_b d_b}\right)^{0.4289}}{\left(\frac{h_b}{h_c}\right)^{0.61}}\left(\frac{b_c + b_b}{2}\right) h_c\sqrt{f_c'} + \Lambda A_j f_y \tag{5.16}$$

where $\zeta = 1$ for joints with L- bars beam tension reinforcement detail bent downward, $\lambda =$ 1.0 (for the database case), A_{sb} is the steel area of the beam, b_b is the width of the beam, Λ is a factor depending on the joint stirrup ratio and is taken 0.664 for joints with low reinforcement ratio (up to 0.003), $\Lambda = 0.60$ for joints with medium reinforcement ratio (between 0.003 and 0.0055), $\Lambda = 0.37$ for joints with high reinforcement ratio (more than 0.0055), A_j is the cross sectional area of the joint links (mm^2).

Although this formula accounted for several key parameters affecting the performance the joint, the application of the formula within the database was not accurate. The *AAE* for this formula was 24% which is significantly high. It is believed that this formula overestimates the effect of the beam longitudinal reinforcement effect to the joint shear capacity. Also the factor Λ that determines the contribution of the joint shear strength is not adequate. Larger number of specimens in the database should have been used to specify a different value for Λ based on the joint reinforcement ratio. The formula was put in the following form for the purpose of optimization:

$$V_{ud} = \frac{C_1\left(\frac{100A_{sb}}{b_b d_b}\right)^{C_2}}{\left(\frac{h_b}{h_c}\right)^{C_4}}\left(\frac{b_c + b_b}{2}\right) h_c f_c'^{C_3} + C_5 A_j f_y \tag{5.17}$$

Optimizing this formula resulted in the following equation:

$$V_{ud} = \frac{0.418\left(\frac{100A_{sb}}{b_b d_b}\right)^{0.15}}{\left(\frac{h_c}{h_b}\right)^{0.60}}\left(\frac{b_c + b_b}{2}\right) h_c f_c'^{0.50} + 0.50\, A_j f_y \tag{5.18}$$

The optimized formula managed to reduce the error percentage into about 23%. But the accuracy of the formula is still questionable. Designers can't count on such error values to produce adequate designs. Figure 5.4 represents a plot of calculated values versus actual experimental values for both the original and the optimized formulae with the optimized formula having less scatter.

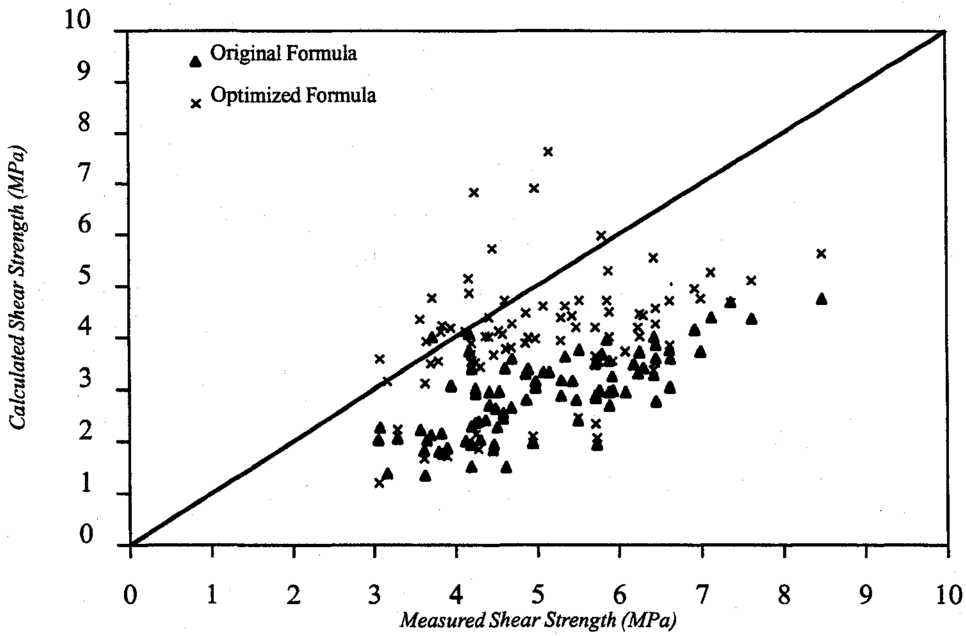


Figure 5.4. Response of original and optimized formulae of Bakir and Boduroğlu (2002a) equations in calculating the shear capacity of the joint

5.3.5 Proposed Formula

Based on the conducted studies using the genetic algorithms approach for optimizing the previously mentioned formulae, the following formula is proposed:

$$V_{ud} = C_1 * h_c b_c * \sqrt{f_c'} * \left\{\frac{100A_{sb}}{b_b d_b}\right\}^{C_2} + C_3 A_{sj} f_y \qquad (5.19)$$

Optimizing this formula lead to the following form

$$ {}_{ud} = 0.75 * h_c b_c * \sqrt{f_c'} * \left\{\frac{100A_{sb}}{b_b d_b}\right\}^{0.02} + 0.60A_{sj}f_y \qquad (5.20) $$

The use of the formula is limited to the monotonically loaded beam-column joints with the following criteria: concrete compressive strength up to 70 MPa, planar specimens with no transverse beams, and specimens with L-bars beam tension beam reinforcement detail. This formula accounted for the beam tension reinforcement, the joint transverse reinforcement, the joint dimensions and the concrete compressive strength. Based on the genetic algorithms model, it was concluded that the effect of the column axial stress is insignificant and can be neglected. This is because most of these columns in the sub assemblages of the database are loaded with significantly small value of the axial stress which prevents any effect of this parameter to appear.

It should be noted that this formula accounted for only 60% of the joint reinforcement. This is in agreement with Vollum (1998) where he accounted for the contribution of 62.50% of the joint stirrups. The contribution of beam reinforcement ratio in the concrete resistance term was found to be limited and lower than what Bakir and Boduroğlu (2002a) included in their formula. It is believed that 60% of the joint stirrups are engaged in resisting shear within the actual lever arm between the compression and tension forces in the joint. This formula managed to reduce the error percentage to 12% which is significantly small. Among all the GA optimization processes, the proposed formula resulted in the lowest *AAE*. The formula also resulted in a scatter (0.165) which is less than other formulae. Accordingly, this formula can be used in the evaluation of shear strength of exterior beam-column joints subjected to monotonic loading. Figure 5.5 represents a plot for the predicted versus the actual shear strength for the proposed formula.

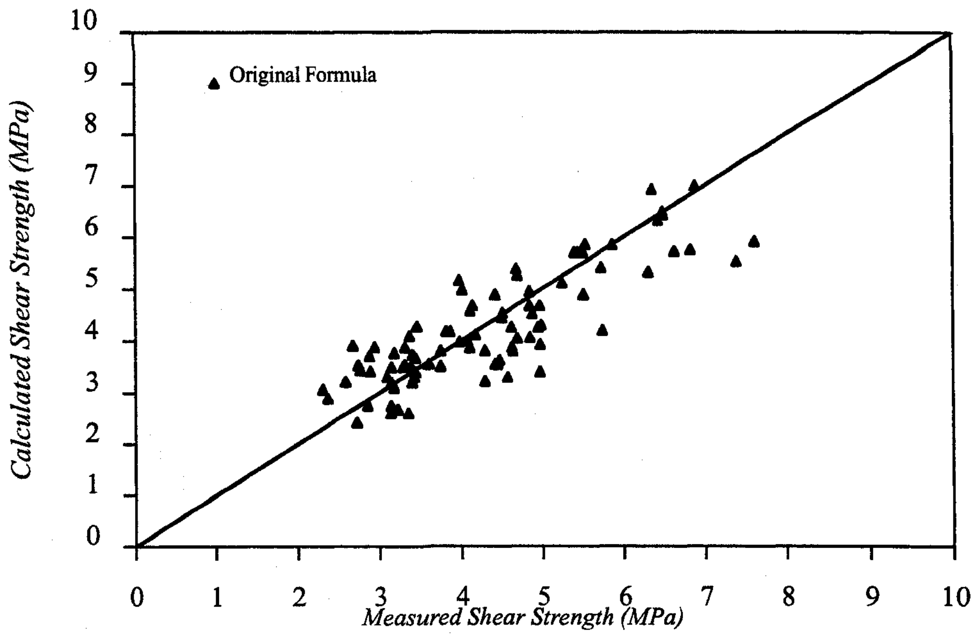


Figure 5.5. Response of the proposed formula in calculating the shear capacity of the joint

5.4 Parametric Study on the Effect of Basic Shear Design Parameter

5.4.1 Effect of Beam Longitudinal Reinforcement Ratio

An analysis was conducted to study the effect of beam longitudinal reinforcement ratio on the shear strength of beam-column joints using the different proposed formulae and the genetic algorithm model. The specimen labeled C_9 proposed by Scott *et al.* (1994) was used to evaluate this parameter. Figure 5.6 represents the parametric study of this factor using different proposed formulae and the GA model. Formulae proposed by ACI-ASCE 352 (2002), Sarsam and Phipps (1985), and Vollum (1998) did not account for the beam longitudinal reinforcement ratio ρ_b as an effective factor on the joint shear capacity. The proposed formula by Bakir and Boduroğlu (2002a) and the GA model predict that the increase in beam longitudinal reinforcement ratio increases the shear capacity; however the contribution of this parameter to joint shear capacity is higher for the formula proposed by Bakir and Boduroğlu (2002a). This result is justified because the increase the beam reinforcement ratio increases the confinement of the joint and improves the bond between the beam and the column leading to increase in the joint capacity.

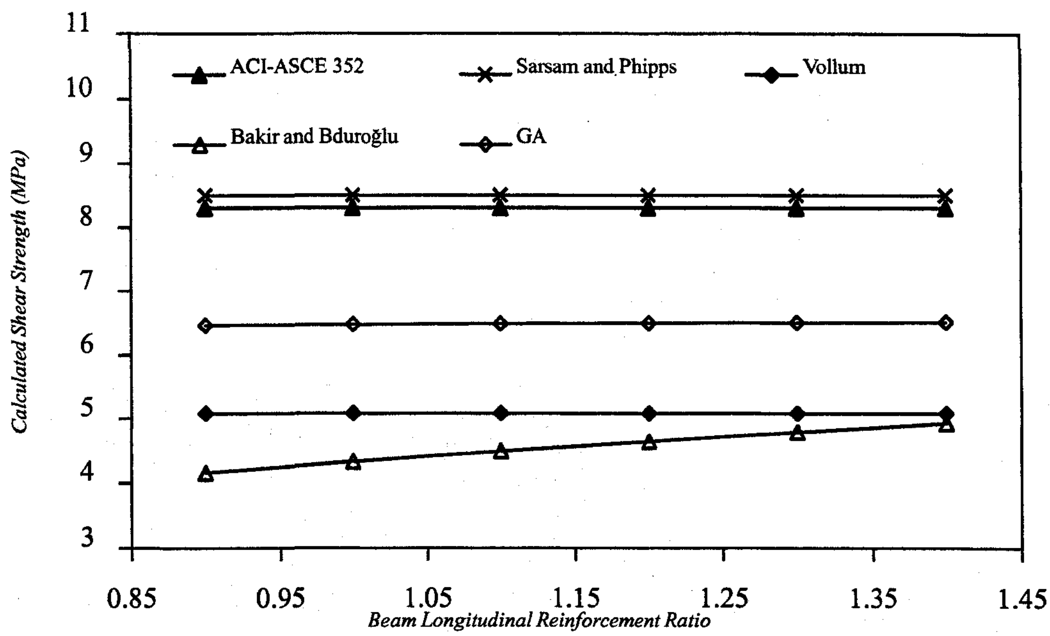


Figure 5.6. Effect of beam longitudinal reinforcement ratio on joint shear capacity

5.4.2 Effect of Joint Shear Reinforcement Ratio

While the ACI-ASCE Committee 352 (2002) formula neglects the effect of shear stirrups on joint shear capacity, the model proposed by the GA concurs with the formula proposed by Sarsam and Phipps (1985), Vollum (1998), Bakir and Boduroğlu (2002a) in the effect of the joint shear reinforcement. According to the GA model, increasing the joint stirrups ratio increases the shear capacity of the joint. These formulae accounted for the joint stirrups by different values. Sarsam and Phipps (1985) assumed that all the stirrups in the joint will yield before the joint failure and therefore they counted for all the stirrups within the joint. Bakir and Boduroğlu (2002a) specified the contribution of the shear stirrups to the joint shear capacity based on the value of the reinforcement ratio in the joint. Both Vollum (1998) and the proposed model predicted very similar values for the contribution of the stirrups to the joint capacity. Generally, the effective stirrups that actually resist the shear forces in the joint should be the ones placed between the concrete compression chord and the beam tension reinforcement. Paulay (1989) proved in his strut and truss model that the shear stirrups resist the majority of the shear forces in the joint after the cracking stage starts. Special precautions should be given to the joint reinforcement ratio and detailing. The contribution of this factor to the investigated formulae is shown in Figure 5.7.

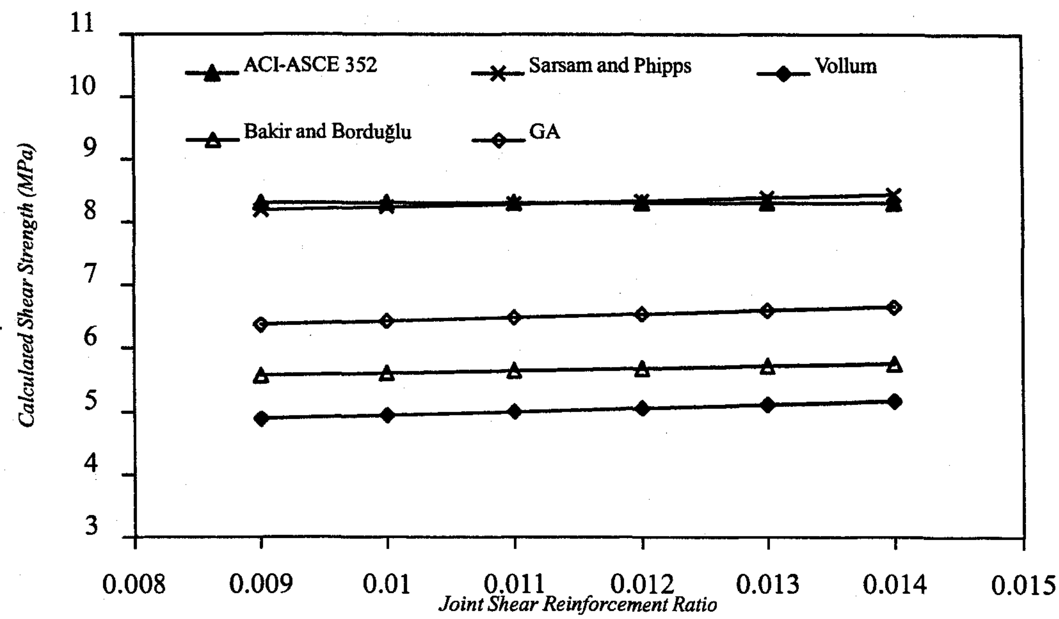


Figure 5.7. Effect of joint shear reinforcement ratio on joint shear capacity

5.4.3 Effect of Concrete Compressive Strength

Concrete compressive strength is an important factor in any reinforced concrete element. Increasing concrete strength leads to improvement in properties of all elements of the structure. Investigation of the effect of the concrete compressive strength with the studied formulae is shown in Figure 5.8. For all the formulae and also the GA derived equation, increasing the concrete compressive strength increases the shear capacity of the joint. The relationship between the concrete strength and the joint shear capacity varies between the different proposed formulae. Except for Sarsam and Phipps (1985) that used the concrete cube strength to express the effect of concrete on shear capacity of the joint, all other formula including the genetic algorithm model assumed a proportional relationship between square root of the concrete compressive strength and the joint shear capacity. This research was limited to specimens up to 70MPa due to lack of high strength concrete specimens. However the effect of high strength concrete compressive strength on the joint shear capacity is expected to be similar to the results of this research.

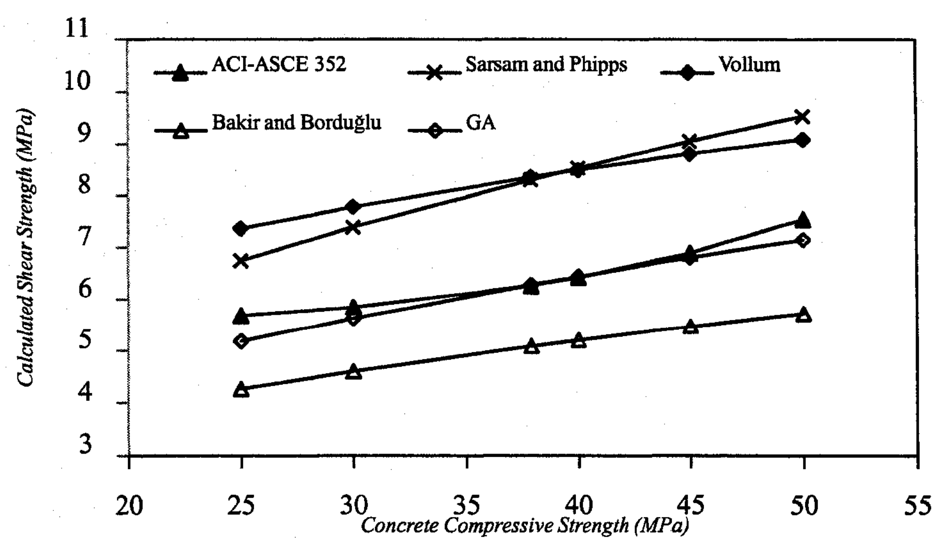


Figure 5.8. Effect of concrete compressive strength on joint shear capacity

5.4.4 Effect of Column Axial Stress

The proposed GA model concurs with the formulae proposed by ACI-ASCE Committee 352 (2002), Vollum (1998), and Bakir and Boduroğlu (2002a) in the effect of column axial stress on the joint shear capacity. They conclude that the column axial stress has no affect on the shear capacity of the joint as shown in Figure 5.9. The formula of Sarsam and Phipps (1985) was the only one that accounted for the effect of the column axial stress on the joint shear capacity and according to them increasing the axial stress improves the joint shear capacity.

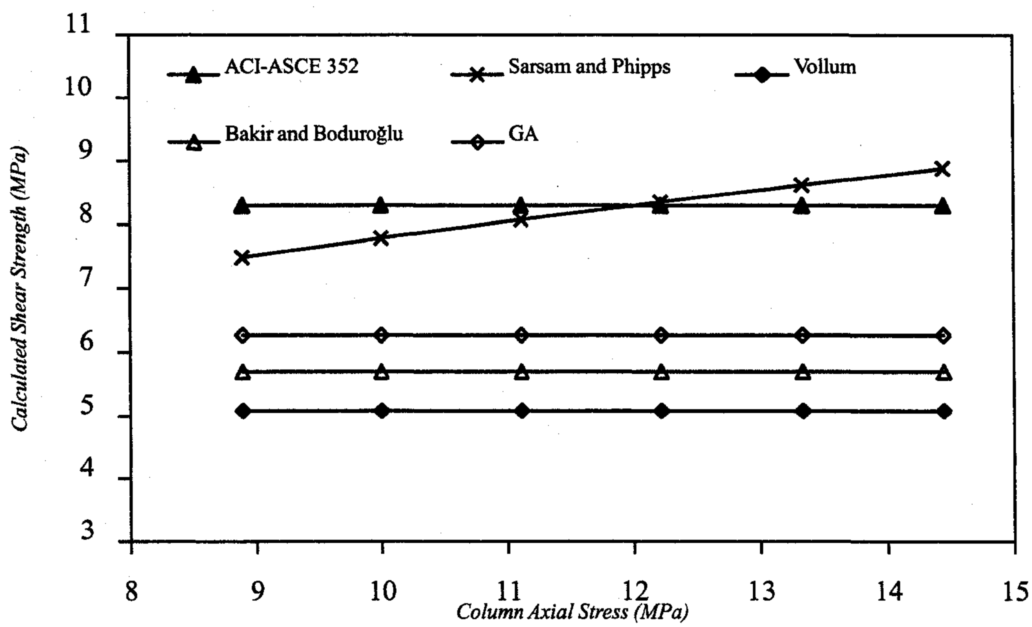


Figure 5.9. Effect of column axial stress on joint shear capacity

5.4.5 Effect of Joint Aspect Ratio

In order to eliminate any interaction between the different parameters when studying the joint aspect ratio (h_b / h_c), this parameter was studied for two cases. The first involved an increase in the joint aspect ratio by increasing the height of the beam with fixed column height, and the other case was by changing the column height with the same beam height. The values for all other parameters were fixed by changing the values of the area of steel in columns or beams or the axial force on the column. In both cases there was no effect on the joint capacity with a change in the joint aspect ratio. This is shown in Figure 5.10.

On the other hand, application of the parametric study of this parameter with other proposed formulae is not clear due to the interaction in these formulae as a result of the modification in the values of beam and joint and axial stress on the column to maintain the genetic algorithms application correct.

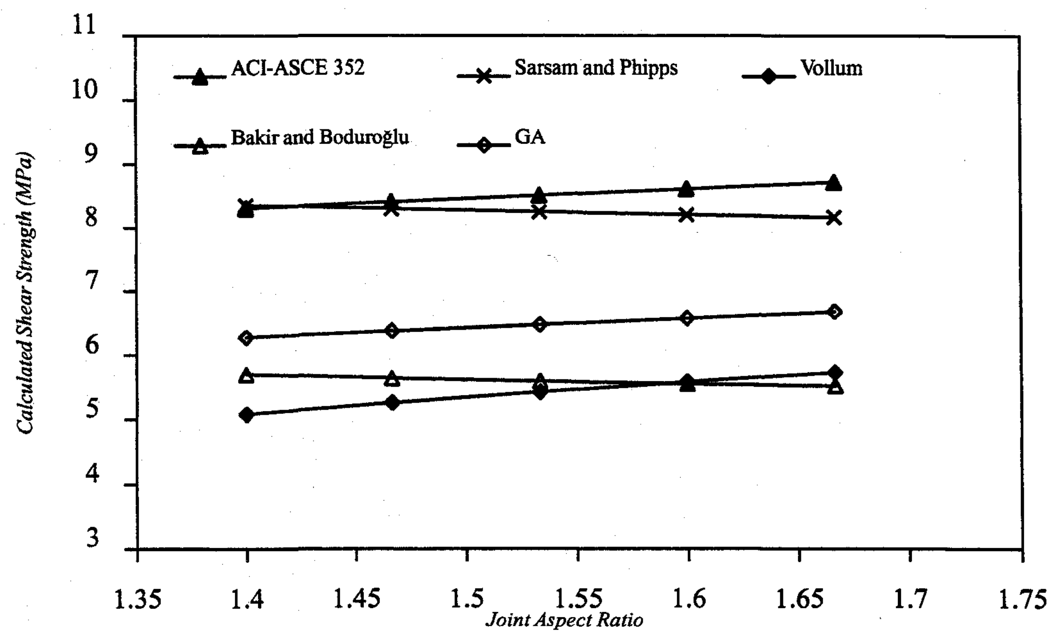


Figure 5.10. Effect of joint aspect ratio on joint shear capacity

CHAPTER 6

EVALUATING SHEAR CAPACITY OF RC INTERIOR BEAM-COLUMN JOINTS UNDER CYCLIC LOADING USING ARTIFICIAL NEURAL NETWORKS

6.1 Background

For the last few decades, many RC structures collapsed during earthquakes. Several studies have investigated the reasons behind this failure (Moehle and Mahin, 1991; Park *et al.*, 1995; EERI, 1999a; EERI, 1999b; EERI, 1999c; Uang *et al.*, 1999; Sezen *et al.*, 2000). Observation of damages indicated the reason of collapse for most of the investigated cases was the lack of the shear capacity of beam-column joints due to inadequate design approach and inappropriate detailing of the joint reinforcement. Figure 6.1 shows a damaged joint in RC structure in the earthquake in Tehuacan-Mexico (1999) and it is noticeable that the joint did not have any shear stirrups. In the earthquake in Izmit, Turkey (1999) many RC structures experienced severe collapse in beam-column joints for the same reason (lack of transverse reinforcement) as shown in Figure 6.2. For these two cases, failure happened in the beam-column joint due to in adequate detailing and insufficient shear capacity of the joint.

This study investigates the shear behavior of interior beam-column joints subjected to cyclic loading using artificial neural networks (ANNs) and experimental testing results collected from the literature. The study aims to clarify the effect of some of the key parameters affecting the shear capacity of the cyclically loaded interior joint including joint shear reinforcement, concrete compressive strength, column axial stress, and joint aspect ratio. The study also evaluates the accuracy of current shear design formulae of the ACI-ASCE Committee352 (2002) and Architectural Institute of Japan (1998) using the experimental testing results.

Figure 6.1. Inadequate detailing of joint in the Tehuacan, Mexico, earthquake,1999 (EERI 1999a)

Figure 6.2. Inadequate detailing of joint in the Izmit, Turkey, earthquake, 1999 (Sezen *et al.*, 2000)

6.2 Previously Proposed Formulae and Equations

In this chapter, two formulae were investigated and evaluated using the selected database, detailed preview of these formulae was proposed in the chapter 2 of this thesis, these formulae are:

1-ACI-ASCE Committee 352 Formula (2002)

2-Architectural Institute of Japan (1998)

6.3 Artificial Neural Network Approach

The most common and applicable type of networks used in engineering program is the Multi-layer perceptron networks (MLP). This type of network is capable of solving complicated regression cases which is the case of most of the engineering areas. You can easily use the network to predict the output using the input data or parameters given to the network. The network train itself to capture the complex behavior using the data set, by dividing the data set into training, selection, and testing sets. And calibrate the accuracy of the result.

The main structure of the multiple layers is an input layer containing the input parameters or data (an input layer, an output layer and one or more hidden layers). Each layer contains a number of processing elements (units) partially or fully connected to units in the consecutive layer. Connections between processing units are initially assigned random numerical values (weights) representing their strength. The main objective in building an artificial neural network-based model is to train specific network architecture to search for an optimum set of weights, for which the trained ANN can predict accurate values of outputs for a given set of inputs from within the range of the training data.

6.4 Experimental Database

The most important aspect in the success of a neural network is the learning database on which the system is trained. Therefore it is imperative to train a network model on a comprehensive database to capture the actual embedded relationships between the parameters of the input and output layers. In this study our aim is to detect the relationships between the different parameters being considered and their effect on the shear capacity of interior beam-column joints under cyclic loadings.

In this study, shear capacity of this joint type is investigated using a database consisting of 58 concrete beam-column connections collected from published literature (Otani *et al.*, 1984; Meinheit and Jirsa, 1977; Walker, 2001; Alire, 2002 Park and Ruitoing, 1998; Kitayama *et al.*, 1987; Higashi and Ohwada, 1969; Attaalla and Agababian, 2004; Hayashi *et al.*, 1994; Zaid, 2001; Fujii and Morita, 1991; Goto and Shibata, 1991; Teraoka *et al.*, 1997).

The quality of the network was improved by imposing several limitations on specimens in the database used by the ANN model. Specimens failing due to joint shear were strictly used, with no beams in the transverse direction. Specimens with high strength concrete, and reinforcement welding into the joint were omitted. The database was formatted into groups of input vectors, each vector representing one of the investigated parameters in the study and the output vector represents the shear capacity of the joint. Table 5 represents the database range of the parameters investigated in the study.

Table 5. The parameters range for the investigated database for interior beam-column joints under cyclic loading.

Parameter	*Minimum*	*Maximum*
Joint Aspect Ratio	1	1.3
Concrete Compressive Strength MPa	21.2	70
Volumetric Reinforcement Ratio (%)	0	3.15
Column Axial Stress (MPa)	0	17.8

6.5 ANN Model

To predict the shear strength of cyclically loaded beam-column joints, an ANN was constructed with the following components: an input layer, an output layer and two hidden layers. The input layer contains four variables representing the common shear design parameters of reinforced concrete beam-column joint (volumetric reinforcement ratio, concrete compressive strength, joint aspect ratio, and column axial stress). The output layer includes one unit representing the shear capacity, V_n and the hidden layers consisted of eight and four processing units consecutively. Full bonding connections were used between the processing elements and the elements in other consecutive layers.

The software used in this model is MATLAB (2007). This software is commonly used for the simulation process of engineering problems. This software divides the given database into training and testing groups to increase the accuracy of the model and give a better understanding of the effect of each parameter in the output layer. Figure 6.3 represents the architecture of the proposed model.

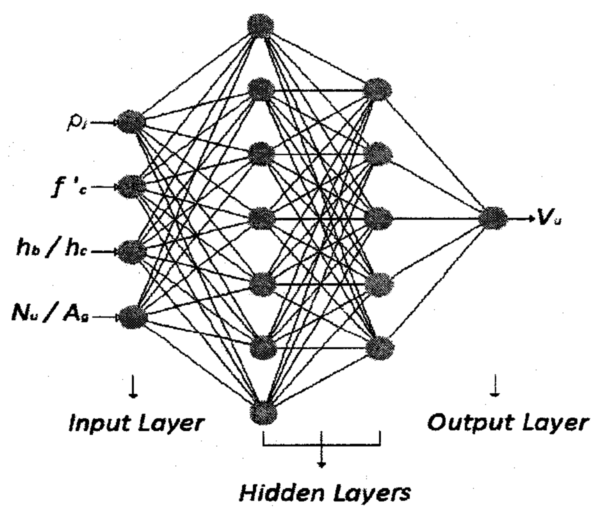


Figure 6.3. Architecture of artificial neural network model

6.6 Results and Discussions

6.6.1 Formulae Evaluation

To consider an ANN successful, it must be able to accurately predict output values for input values within the range of the database used in the training and the testing process. To evaluate the accuracy of the proposed network, a comparison was held between the network predicted outputs which represent the shear capacity and those calculated using the formulae by ACI-ASCE 352 (2002) Architectural Institute of Japan (1998) The performance of each model was evaluated based on both the ratio of measured to predicted (or calculated) shear strength (V_m/V_p), and the average absolute error (*AAE*) calculated using the following equation:

$$AAE = \frac{1}{n}\sum \frac{|V_m - V_p|}{V_m} \times 100 \tag{6.6}$$

The average value, the standard deviation (*STDV*), and coefficient of variation (*COV*) for V_m/V_p, and the average absolute error (*AAE*) of the ANN model and ACI-ASCE 352 (2002) are listed in Table 6.

Table 6. Performance of different formulae for the calculation of shear strength of RC interior beam-column joints under cyclic loading.

Method	*AAE (%)*	$V_{measured} / V_{predicted}$		
		Average	*STDV*	*COV*
ACI-ASCE Committee 352 (2002)	63	0.77	0.29	38.7
Architectural Institute of Japan (1998)	90	0.651	0.297	48.00
ANN	8.15	0.99	0.0988	10

In the following sections a detailed discussion of the results of the different investigated formulae is presented:

6.6.1.1 ACI-ASCE Committee 352 Formula (2002)

Figure 6.4 represents a plot of the actual experimental shear strength values versus the calculated ones using the ACI-ASCE Committee 352 formula. This formula neglects the influence of the joint aspect ratio, the column axial stress, and the contribution of both joint and beam reinforcements to the shear capacity of the joint. It also neglects the effect of the column axial stress. Using the selected data for this study and knowing the actual capacity of the specimens obtained from the experimental programs results, the average absolute error *AAE* for this formula is 63%, which is significantly high, and the *STDV* for V_m/V_p of this formula is 0.29. It is recommended that this formula should not be used to estimate the shear capacity of beam-column joints due to its lack of accuracy and the over estimation of the shear strength. It should rather be used to estimate the minimum shear strength of the joint based on concrete properties and joint dimensions.

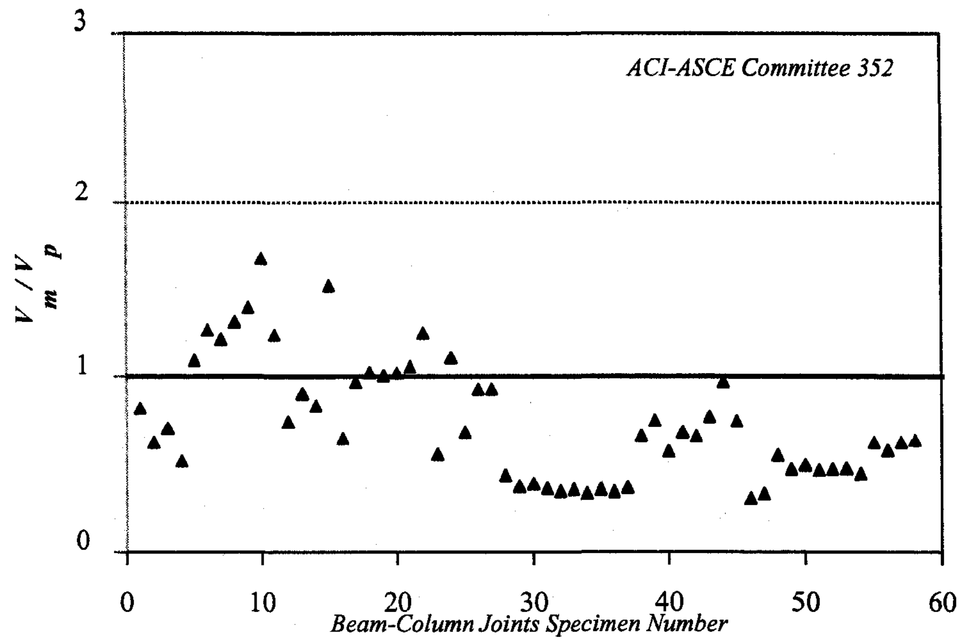


Figure 6.4. Performance of the equation proposed by ACI-ASCE 352 (2002) in calculating the shear capacity of beam-column joints

6.6.1.2 Architectural Institute of Japan (1998)

Figure 6.5 represents a plot of the actual experimental shear strength values versus the calculated ones using the formula proposed by the Architectural Institute of Japan (1998). This formula neglects the influence of the joint aspect ratio, the column axial stress, and the contribution of both joint and beam reinforcements to the shear capacity of the joint. Using the selected data for this study and knowing the actual capacity of the specimens obtained from the experimental programs results, the average absolute error *AAE* for this formula is 90%, which is extremely high, and the *STDV* for V_m/V_p of this formula is 0.297. It is recommended that this formula should not be used to estimate the shear capacity of beam-column joints due to its lack of accuracy and the over estimation of the shear strength. Neglecting several major factors governing the behavior of the joint refute the accuracy and the validity of this formula.

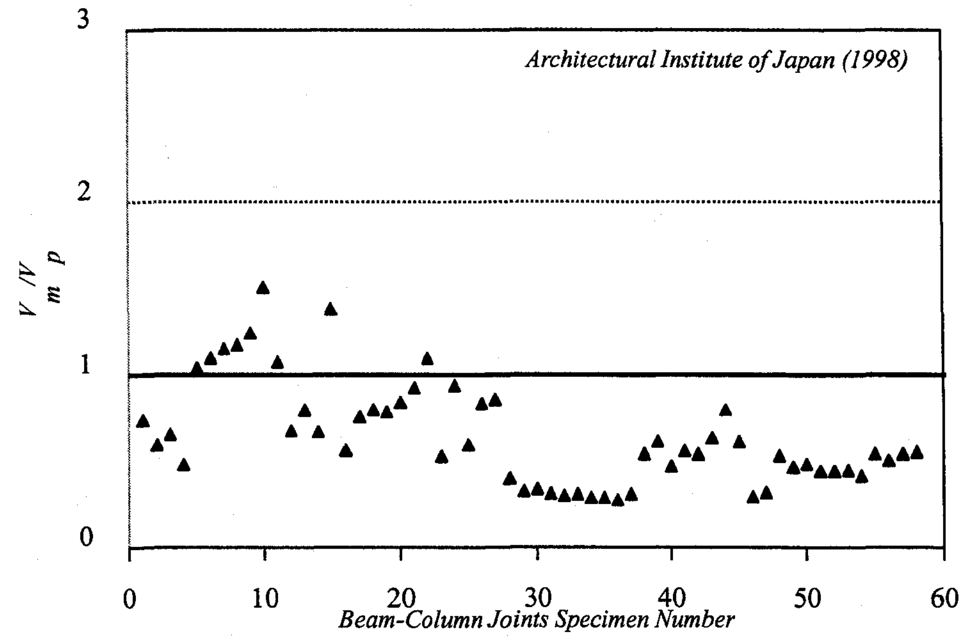


Figure 6.5. Performance of the equation proposed by Architectural Institute of Japan (1998) in calculating the shear capacity of beam-column joints

6.6.1.3 Proposed ANN

The proposed model for the ANNs produced much more accurate outputs for predicting the shear capacity of joints than the formula proposed by ACI-ASCE 352. Figure 6.6 shows that this model reduced the *AAE* between the actual and the predicted values to a very small value (8.15 %). The model also resulted in a smaller scatter for the data with *STDV* of 0.0988. The small value of *AAE* ensures the accuracy of selecting the investigated parameters as the key factors governing the shear behavior of joints.

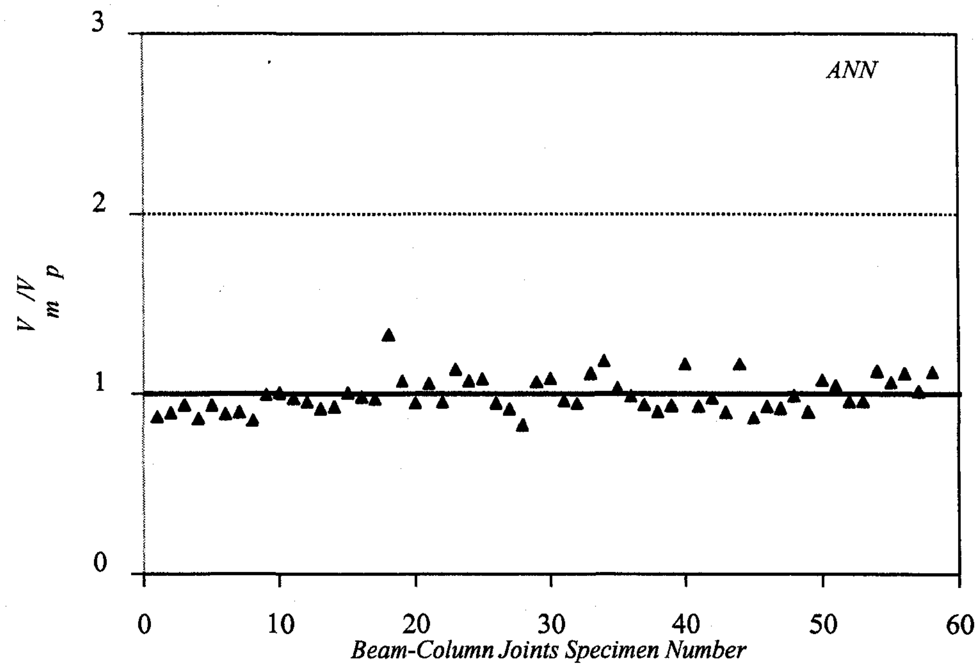


Figure 6.6. Performance of ANN model in calculating the shear capacity of beam-column joints

6.6.2 Parametric Study on Effect of Basic Shear Design Parameter

6.6.2.1 Effect of Joint Shear Reinforcement Ratio

An analysis was conducted to study the effect of joint shear reinforcement ratio on the shear strength of cyclically loaded beam-column joint using the proposed ANN model and the investigated formulae. The specimen labeled J_4 proposed by Noguchi and Kashiwazaki (1992) was used to evaluate the effect of this parameter. Figure 6.7 represents the parametric study of this parameter. The ACI-ASCE Committee 352 (2002) formula and the Architectural Institute of Japan (1998) formula neglect the effect of shear stirrups on joint shear capacity. The model proposed by the ANNs as shown in the figure account for this parameter, increasing the joint shear reinforcement increases the joint shear capacity. The current study suggests that this parameter is one of the major parameter governing the shear capacity of the joint especially at advanced loading levels when the cracks begin spreading in concrete and the effect of concrete compressive strength to joint shear capacity reduces.

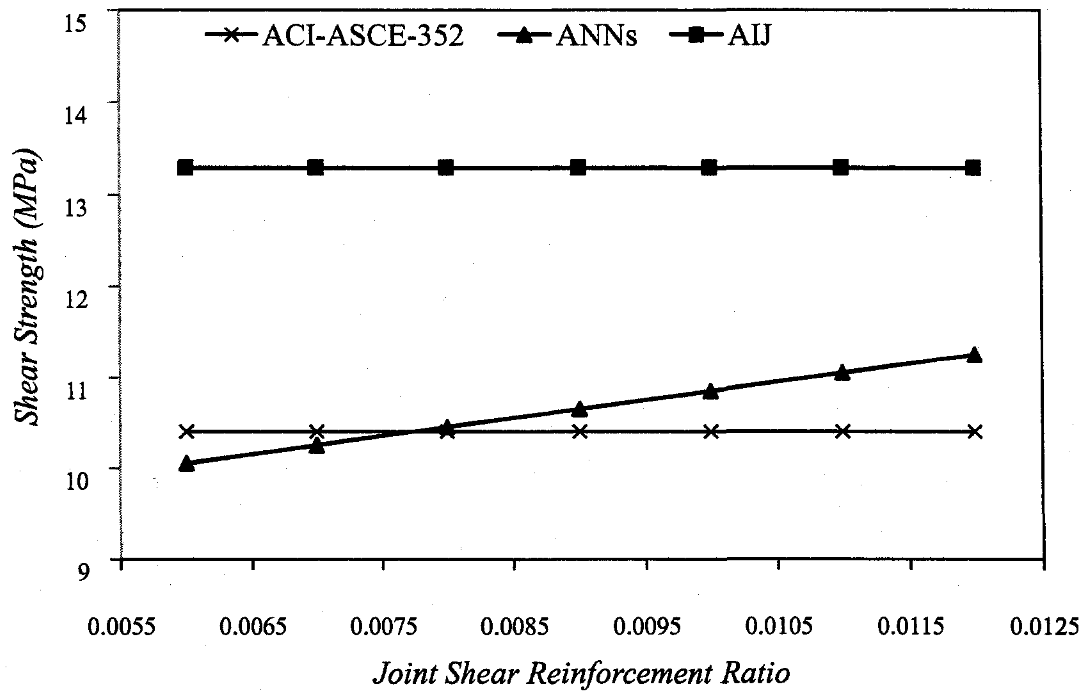


Figure 6.7. Effect of joint reinforcement ratio on joint shear capacity

6.6.2.2 Effect of Concrete Compressive Strength

Concrete compressive strength is an important factor in any reinforced concrete element. Increasing concrete strength leads to improvement in properties of all elements of the structure. Investigation of the effect of the concrete compressive strength with the studied formulae is shown in Figure 6.8. The ANN resulted in similar trend for the effect of concrete compressive strength to the results of the formulae proposed by ACI-ASCE Committee 352 (2002) and the Architectural Institute of Japan (1998), increasing the concrete compressive strength increases the shear capacity of the joint. The formula proposed by the Architectural Institute of Japan gives higher contribution of the concrete compressive strength to joint shear capacity than the other two methods.

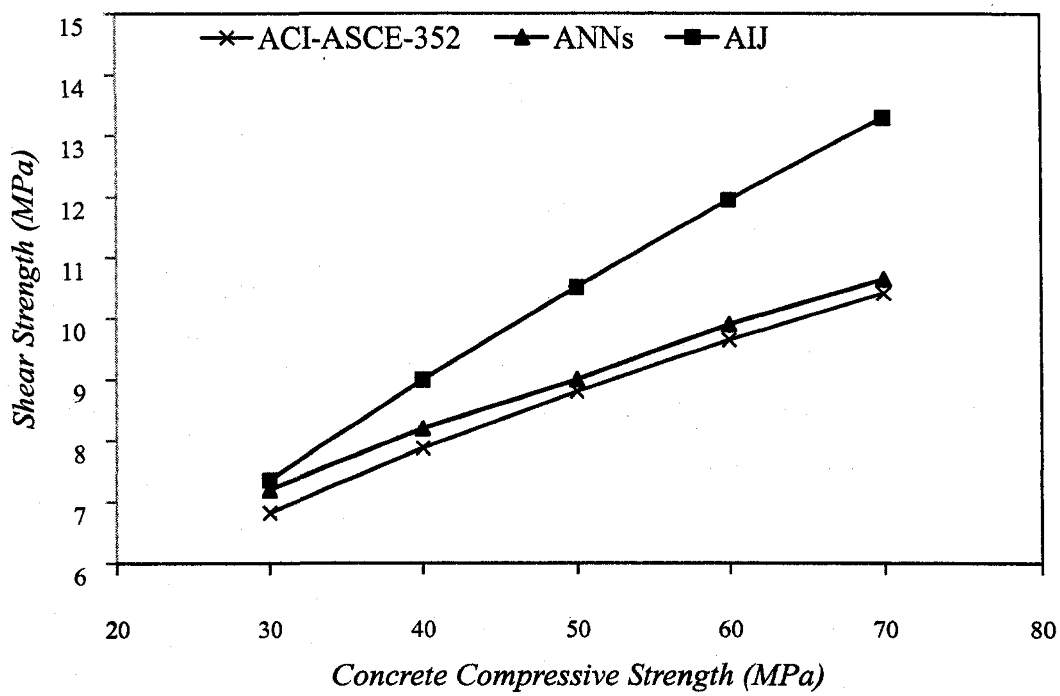


Figure 6.8. Effect of concrete compressive strength on joint shear capacity

6.6.2.3 Effect of Column Axial Stress

The proposed ANNs model concurs with the formulae proposed by ACI-ASCE Committee 352 (2002) and the Architectural Institute of Japan (1998) in the effect of column axial stress on the joint shear capacity. All of these formulae conclude that the column axial stress has no affect on the shear capacity of the joint as shown in Figure 6.9. This result also concurs with the art of the study on the shear capacity of monotonically loaded beam-column joints.

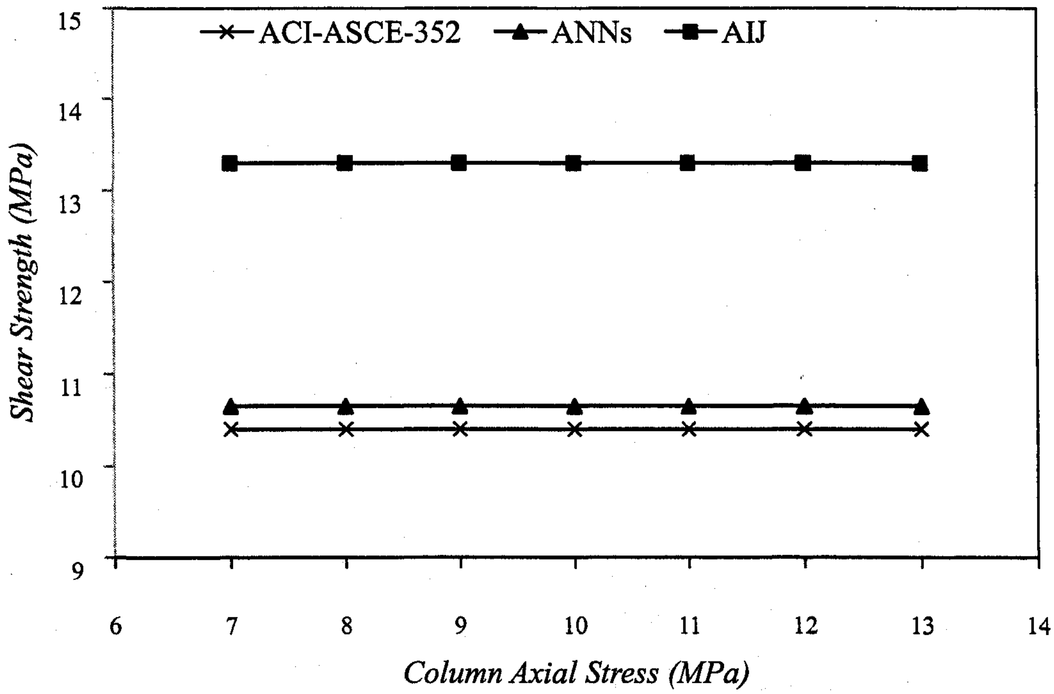


Figure 6.9. Effect of column axial stress on joint shear capacity

6.6.2.4 Effect of Joint Aspect Ratio

Figure 6.10 represents the parametric study for the effect of the joint aspect ratio on the joint shear capacity. Both the ANN model and the Formula proposed by the Architectural Institute of Japan assume no affect for the joint aspect ratio on the capacity of the joint. The ACI-ASCE 352 (2002) assumes a proportional relationship between the aspect ratio and the capacity of the joint. Investigation of this parameter is not very clear due to the changing of the parameters of the aspect ratio (which are the beam height and the column height) on other major parameters like the axial stress and the joint reinforcement ratio.

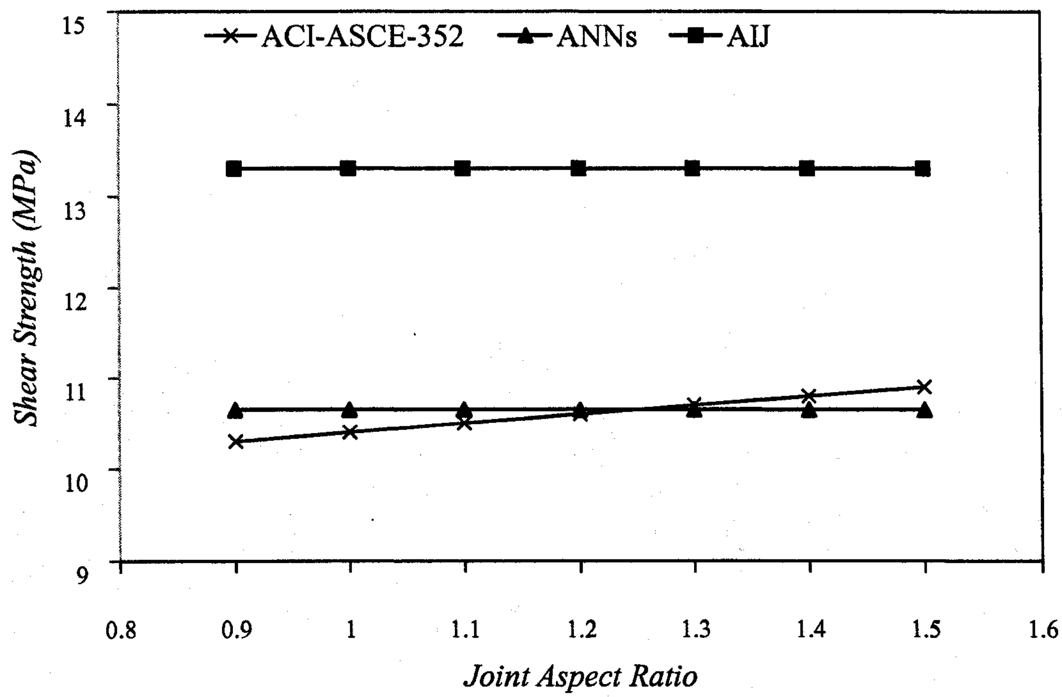


Figure 6.10. Effect of joint aspect ratio on joint shear capacity

CHAPTER 7

EVALUATING SHEAR CAPACITY OF RC INTERIOR BEAM-COLUMN JOINTS UNDER CYCLIC LOADING USING GENETIC ALGORITHMS

7.1 Background

One of the major problems that face designers of RC structures is the design of the beam-column joint especially the cyclic loading condition. The reason is because there is no clear formula that they can rely on during the design phase. Behavior of the cyclically loaded beam-column joints is very complicated and several mechanisms control it. This study aims to evaluate some of the existing shear design formulae of cyclically loaded beam-column joints namely: ACI-ASCE Committee 352 (2002) and Architectural Institute of Japan (1998), and to optimize these formulae using the genetic algorithms technique (GAs). The study also is proposing a new design formula for calculating the shear capacity of RC cyclically loaded beam-column joints. For the sake of the optimization process, a database was collected from the literature from different experimental programs.

7.2 Experimental Database

The database used for this study was selected from the available experimental research programs in the literature. A total number of 58 specimens were selected for the study. The selection process was based on special criteria: concrete compressive strength was limited to 70 MPa, planar specimens with no transverse beams were only considered, and specimens with bent up L-bar tension beam reinforcement detail were excluded.

In the optimization process of the formulae, the genetic algorithms tool box attached in the computer software MATLAB (2007) was used.

7.3 Optimization of Formulae

To consider the optimization process successful, the modified formulae should be able to predict the values of beam-column joint shear capacity more accurately than the original formulae. The performance of the optimization process of each formula was evaluated based on both the ratio of measured to predicted (or calculated) shear strength (V_m/V_p), and the average absolute error (*AAE)* calculated using the following equation:

$$AAE = \frac{1}{n} \sum \frac{|V_m - V_p|}{V_m} x100 \qquad (7.1)$$

The standard deviation (*STDV*), and coefficient of variation (*COV*) for V_m/V_p, and the average absolute error (*AAE*) of the GA model and other shear calculation methods investigated are listed in Table 7. In the following sections, a detailed description of the optimization process conducted on each of the previously mentioned formulae is presented.

Table 7. Performance of GA model and shear design methods considered in this study in predicting the shear strength of interior cyclically loaded beam-column joints

Method	*Pre-Optimized*				*Post-Optimized*			
	AAE (%)	*$V_{measured}$ / $V_{predicted}$*			*AAE (%)*	*$V_{measured}$ / $V_{predicted}$*		
		Average	*STDV*	*COV*		*Average*	*STDV*	*COV*
ACI-ASCE 352 (2002)	*63*	*0.77*	*0.29*	*38.80*	*36*	*1.223*	*0.474*	*38.78*
Architectural Institute of Japan (1998)	*90*	*0.651*	*0.297*	*48.00*	*36*	*1.223*	*0.474*	*38.78*
GA	--------	--------	--------	--------	*21*	*1.07585*	*0.307*	*28.609*

7.3.1 Design Equation of ACI-ASCE Committee 352 (2002)

Based on the loading conditions for the joint and the anticipated deformations of the connected frame members, ACI-ASCE Committee 352 classifies beam-column joints into two categories: Type 1 and Type 2. Type 1, joints are designed to satisfy ACI 318 (2008) except for seismic provisions (gravity load case); while Type 2 joints are designed to have sustained strength under deformation reversals into the plastic range (seismic loading case). In this study, only Type 2 joints were investigated.

The ACI-ASCE Committee 352 (2002) proposed the following equation to calculate the shear strength of monotonically loaded exterior beam-column joints:

$$V_n = 0.083\gamma\sqrt{f_c'}b_j h_c \qquad (7.2)$$

where V_n is the nominal shear strength of Type 1 joints, f_c\` is the concrete cylinder strength (MPa), h_c is the depth of the column in the direction of joint shear being considered (mm), b_j is the effective width of the joint (mm), it is defined as the smaller value of

$$\frac{b_b + b_c}{2} \tag{7.3a}$$

$$b_b + \sum (mh_c + 2) \tag{7.3b}$$

$$b_c \tag{7.3c}$$

where m = 0.50 for the case of no eccentricity between the beam and column centerlines, γ = 15 for Type 1 exterior planar joints (database case). Accordingly the formula becomes:

$$V_n = 1.245\sqrt{f_c'}b_j h_c \tag{7.4}$$

This formula neglects the influence of the joint aspect ratio, the column axial stress, and the contribution of both joint and beam reinforcements to the shear capacity of the joint. Using the selected data for this study and knowing the actual capacity of the specimens obtained from the experimental programs results, the average absolute error *AAE* for this formula is 63%, which is significantly high, and the *STDV* for V_m/V_p of this formula is 0.29. It is recommended that with such a high percentage error and big scatter this formula should not be used to estimate the shear capacity of beam-column joints due to its lack of accuracy.

An optimization process was conducted on this formula using genetic algorithms approach. The formula was modeled in the following format and then calibrated using the database of the study:

$$V_n = C_1 f_c'^{C_2} b_j h_c \tag{7.5}$$

The results of the optimization process indicated that the best obtained values for C_1 and C_2 are 0.78 and 0.50 respectively. The formula will then become:

$$V_n = 0.78 f_c'^{0.50} b_j h_c \tag{7.6}$$

The *AAE* for this formula is approximately 36%. Although the optimization process significantly reduced the *AAE* value produced by the pre-optimized formula, still the modified formula is not reliable since it does not account for important parameters. Figure 7.1 represents a plot of calculated values versus actual experimental values for both the original and the optimized formulae with the optimized formula showing less scatter. This is also clear from the smaller value of *STDV* which is 0.47 for the modified formula.

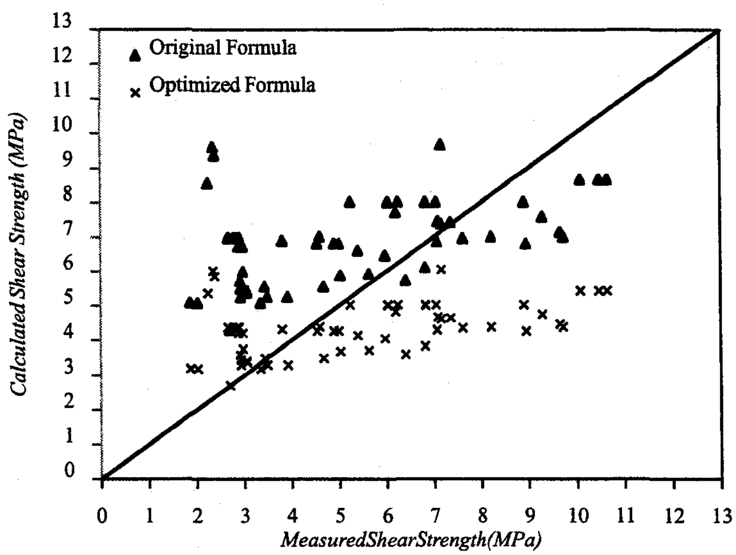


Figure 7.1. Response of original and optimized formulae of ACI-ASCE 352 equations in calculating the shear capacity of the joint

7.3.2 Design Equation of Architectural Institute of Japan (1998)

The architectural Institute of Japan proposed the following formula for calculating the shear capacity of cyclically loaded RC beam-column joints:

$$V_u = k * \emptyset * F_j * b_j * D \tag{7.7}$$

where k = 1, φ =0.85, F_j = 0.80*(f_c') $^{0.70}$ (MPa), D is the column depth, b_j = effective column width. This leads the formula to be

$$V_u = 0.68 * (f_c')^{0.70} * b_j * D \tag{7.8}$$

Results obtained from application of this formula to the experimental database used in this study indicated that this formula is extremely inadequate. The *AAE* of this formula is about 90%. It is highly recommended not to use this formula in design of beam-column joints, furthermore, the *STDV* of this formula is significantly high (0.29). It is believed that the reason behind this is that the formula does not represent the actual parameters governing the capacity of beam-column joints. The formula neglects the effect of the joint shear reinforcement and the joint aspect ratio.

An optimization process was conducted on this formula using genetic algorithms approach. The formula was modeled in the following format and then calibrated using the database of the study:

$$V_u = C_1 \, f_c'^{C_2} b_j D \tag{7.9}$$

The results of the optimization process indicated that the best obtained values for C_1 and C_2 are 0.81 and 0.50 respectively, the resulted formula is almost the same one obtained from optimizing the fo9rmula proposed by the ACI-ASCE Committee 352 (2002) as the parameters are the same in both formula. The formula will then become:

$$V_u = 0.801 \, f_c'^{0.50} b_j D \tag{7.10}$$

The *AAE* for this formula is approximately 36%. Although the optimization process significantly reduced the *AAE* value produced by the pre-optimized formula, still the modified formula is not reliable since it does not account for important parameters. Figure 7.2 represents a plot of calculated values versus actual experimental values for both the original and the optimized formulae with the optimized formula showing less scatter. This is also clear from the smaller value of *STDV* which is 0.47 for the modified formula.

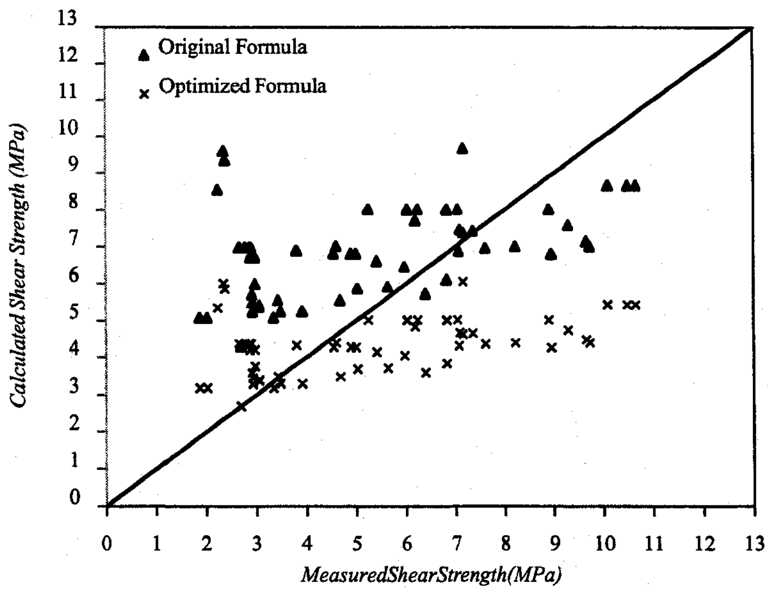


Figure 7.2. Response of original and optimized formulae of Architectural Institute of Japan (1998) equation in calculating the shear capacity of the joint

7.3.3 Proposed Formula

Based on the conducted studies using the genetic algorithms approach for optimizing the previously mentioned formulae, the following formula is proposed:

$$V_{ud} = C_1 * h_c b_j * \sqrt{f_c'} + C_2 A_{sj} f_y \quad (7.11)$$

Optimizing this formula lead to the following form

$$V_{ud} = 0.615 * h_c b_j * \sqrt{f_c'} + 0.65 A_{sj} f_y \quad (7.12)$$

The formula use is limited to the cyclically loaded interior beam-column joints with the following criteria: concrete compressive strength up to 70 MPa, planar specimens with no transverse beams, and specimens with L-bars beam tension beam reinforcement detail. This formula accounted for the joint transverse reinforcement, the joint dimensions, and the concrete compressive strength. Based on the genetic algorithms model, it is concluded that the effect of the axial stress of the column is insignificant and can be neglected. The reason behind this is because the value of column axial stress in most of the specimens is small which makes the contribution of this parameter to joint shear strength significantly small.

As noticed from this formula, the formula accounted for only 70% of the joint reinforcement. This result is justified because the actual lever arm between the compression and tension forces in the joint can never be the hall depth of the beam. This formula managed to reduce the error percentage to 18% which is significantly small. Among all the GAs optimization processes, the proposed formula resulted in the lowest *AAE*. The formula also resulted in a small scatter (0.165) which is less than other formulae. This formula can be used in the evaluation of shear strength of exterior beam-column joints subjected to monotonic loading. Figure 5.5 represents a plot for the predicted versus the actual shear strength for the proposed formula.

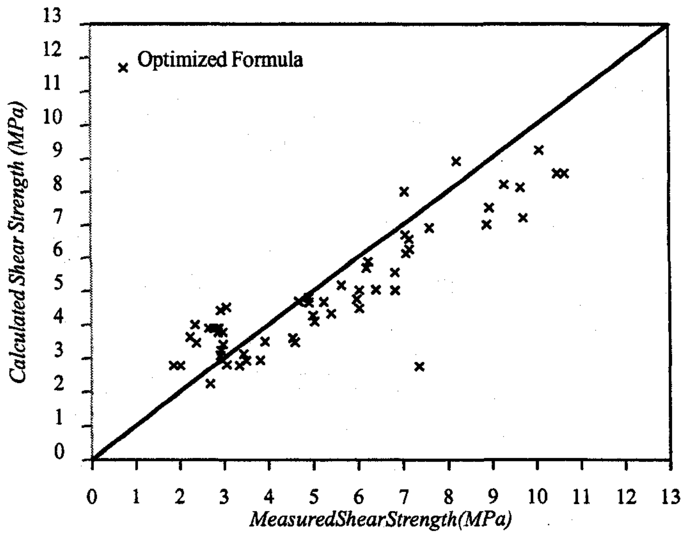


Figure 7.3. Response of the proposed GA equation in calculating the shear capacity of the joint

CHAPTER 8

CONCLUSIONS

The aim of this thesis was to investigate the shear behavior of beam-column joints including the basic parameters controlling this behavior and the existing design formulae for the shear capacity. New formulae were also proposed for the sake of appropriate design of beam-column joints in two major cases namely; exterior monotonically loaded joints and interior cyclically loaded joints. Based on this study the following conclusions are provided:

1- Increasing joint shear reinforcement ratio improves the shear capacity of a beam-column joint, and the amount of effective joint stirrups to shear capacity is between 60% and 70% of the total amount of stirrups in the joint.

2- Concrete compressive is an important factor to the shear capacity of beam-column joints.

3-No significant effect was noticed for the column axial stress on the shear capacity of the joint. It is suggested that since all the specimens used in the database were designed to test the shear capacity of the joint, the axial loading level on the column was relatively small. The effect of higher column axial loading level could be more significant.

4- Two artificial neural networks were proposed for the two investigated cases. The models succeeded to realistically simulate the behavior of beam-column joint and capture the hidden relationships between the shear capacity and the investigated parameters.

5- A new formula is proposed using the genetic algorithm technique and the selected database for calculating shear capacity of exterior monotonically loaded beam-column joints. The formula is as follows:

$$V_{ud} = 0.75 * h_c b_c * \sqrt{f_c'} * \left\{\frac{100A_{sb}}{b_b d_b}\right\}^{0.02} + 0.60A_{sj}f_y \tag{8.1}$$

The formula gave significantly small error and less scatter than other existing formulae. The *AAE* of the new formula is 12% and the *STDV* is 0.165.

6- A new formula is proposed using the genetic algorithm technique and the selected database for calculating shear capacity of interior cyclically loaded beam-column joints. The formula is as follows:

$$V_{ud} = 0.615 * h_c b_j * \sqrt{f_c'} + 0.65A_{sj}f_y \quad (8.2$$

The *AAE* of the new formula is 21%. This percentage is significantly smaller than the ones obtained by different design equations.

7-Increasing the beam longitudinal reinforcement ratio improves the shear capacity of beam-column joints due to its confinement effect on the concrete core of the joint. Enough embedment should be given to the beam tension longitudinal bars into the column to ensure the confinement action for the case of exterior joints.

REFERENCES

ACI Committee 318 (2008), 2008, "Building Code Requirement for Structural Concrete (ACI 318R-2008)", American Concrete Institute, Farmington Hills, Mich, pp.369.

Alire, D. A., 2002, "Seismic Evaluation of Existing Unconfined Reinforced Concrete Beam-Column Joints", MSCE Thesis, University of Washington, Seattle, 250 p.

Architectural Institute of Japan, 1998, "Recommendations of RC Structural Design after Hanshin-Awaji Earthquake Disaster-Cause of Particularly Noticed Damages and Corresponding RC Structural Details", Architectural Institute of Japan.

Attaalla, S, A., and Agbabian, M, S., 2004, "Performance of Interior Beam-Column Joints Cast from High Strength Concrete under Seismic Loads", Advances in Structural Engineering, V. 7, No. 2, pp. 147-157.

Bakir, P. G., and Boduroğlu, H. M., 2002a, "A New Design Equation for Predicting the Joint Shear Strength of Monotonically Loaded Exterior Beam-Column Joints", Engineering Structures, V. 24, No. 8, pp. 1105-1117.

Bakir, P. G., and Boduroğlu, H. M., 2002b, "Predicting the Failure Modes of Monotonically Loaded Reinforced Concrete Exterior Beam-Column Joints", Structural Engineering and Mechanics, V. 14, No. 3, pp. 307-330.

Bellman, R, E., 1978, "An Introduction to Artificial Intelligence: *Can Computers Think?*", Boyd and Fraser Publishing Company, San Francisco.

Bosshard, M., and Menn, C., 1984, "Versuche über den Einfluss der Bewehrungsanordnung auf das Tragverhalten von Rahmenecken aus Stahlbeton", ETH Zürich, Switzerland, 34 p.

Charniak, E., and McDermott, D., 1985, "Introduction to Artificial Intelligence", Addison-Wesley, Reading, Massachusetts.

Chopra, A.K., 2007, "Dynamics of Structures", Prentice Hall, Englewood Cliffs, New Jersey, 876p.

Durrani, A, J., and Wight, J, K., 1982, "Experimental and Analytical Study of Beam to Column Connections Subjected to Reserve Cyclic Loading", Technical Report UMEE82 R3, Department of Civil Engineering, University of Michigan, pp. 295.

Earthquake Engineering Research Institute, 1999, "December 1999. The Chi-Chi, Taiwan Earthquake of September 21, 1999", EERI Special Earthquake Report, 1999c, http://www.eeri.org/earthquakes/earthquakes.html

Earthquake Engineering Research Institute, 1999, "November 1999. The Athens, Greece Earthquake of September 7, 1999", EERI Special Earthquake Report, 1999b, http://www.eeri.org/earthquakes/earthquakes.html

Earthquake Engineering Research Institute, 1999, "September 1999. The Tehuacan, Mexico, Earthquake of June 15, 1999", EERI Special Earthquake Report, 1999a, http://www.eeri.org/earthquakes/earthquakes.html

El-Chabib, H. H., 2006, "Modeling Properties of Special Concretes using artificial intelligence", PhD Dissertation, The University of Western Ontario, Canada, 284p.

Elmorsi, M., Kianoush, M, R., and Tso, W, K., 2000, "Modeling Bond-Slip Deformations in Reinforced Concrete Beam-Column", Canadian Journal of Civil Engineering, V. 27, pp. 490-505.

Endoh, Y., Kamura, T., Otani, S., and Aoyama, H., 1991, "Behavior of RC Beam-Column Connections Using Light-Weight Concrete", Transactions of Japan Concrete Institute, pp. 319-326.

Fuji, S., and Morita, S., 1991, "Comparison between Interior and Exterior Reinforced Concrete Beam-Column Joint Behavior", ACI SP-123, pp. 145-165.

Hamil, S. J., 2000, "Reinforced Concrete Beam-Column Connection Behavior", PhD. Dissertation, School of Engineering, University of Durham, England, 395 p.

Haugeland, J., 1985, "Artificial Intelligence: *The Very Idea*", MIT Press, Cambridge, Massachusetts.

Hayashi, K., Teraoka, M., Mollick, A. A., and Kanoh, Y., 1994, "Bond Properties of Main Reinforcing Bars and Restoring Force Characteristics in RC Interior Beam-Column Assemblages Using High Strength Materials", In Proceedings, Second US-Japan-New Zealand- Canada Multilateral Meeting on Structural Performance of High Strength Concrete In Seismic Regions, Honolulu, Hawaii, pp. 15-27.

Hegger, J., Sherif, A., and Roeser, W., 2003, "Nonsiesmic Design of Beam-Column Joints", ACI Structural Journal, V. 100, No. 5, pp. 654-664.

Higashi, Y., and Ohwada, Y., 1969, "Failing Behavior of Reinforced Concrete Beam-Column Connections Subjected to Lateral Load", Memories of Faculty of Technology Tokyo Metropolitan University, Tokyo, Japan, pp.91-101.

Hoekstra, A. S., 1977, "De Invloed van de Wapeningsdetaillering ophet Gedrag van Doorgaande-Kolom-Balkverbinding," TH Delft, The Netherlands, 102 p.

Hwang, S, J., and Lee, H, J., 2000, "Analytical Model for Predicting Shear Strength of Interior Reinforced Concrete Beam-Column Joints for Seismic Resistance", ACI Structural Journal, V. 97, No. 1, pp. 35-44.

Joh, O., Goto, Y., and Shibata, T., 1991, "Influence of Transverse Joint, Beam Reinforcement and Relocation of Plastic Hinge Region on Beam-Column Joint Stiffness Determination", In ACI Special Publications SP 123-12: Design of Beam-Column Joints for Seismic Resistance, Farmington Hills, Michigan, pp. 187-223.

Joint ACI-ASCE Committee 352, 2002, "Recommendation for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures", American Concrete Institute, Farmington Hills, Mich, 40 p.

Kitayama, K., Otani, S., and Aoyama, H., 1987, "Earthquake Resistant Design Criteria for Reinforced Concrete Interior Beam-Column Joints", In Pacific Conference on Earthquake Engineering, Wairakei, New Zealand, pp. 315-326.

Kordina, K., 1984, "Bewehrungsfuhrung in Ecken und Rahmenendknoten", Deutcher Ausschuss für Stahlbeton, Heft 354, Berlin, Germany, 152 p.

Kurzweil, R., 1990, "The Age of Intelligent Machines", MIT Press, Cambridge, Massachusetts.

Lowes, L, M., and Altoontash, A., 2003, "Modeling Reinforced-Concrete Beam-Column Joints Subjected to Cyclic Loading", Journal of Structural Engineering, V. 129, No. 12, pp. 1686-1697.

Meinheit, D. F., and Jirsa, J. O., 1977, "The Shear Strength of Reinforced Concrete Beam-Column Joints", University of Texas, Austin, CESRL Report No. 77-1.

Moehle, J. P., and Mahin, S. A., 1991, "Observations on the Behavior of Reinforced Concrete Buildings during Earthquakes", American Concrete Institute, Farmington Hills, Mich, SP-12, pp.67-90.

Nilson, N, J., 1998, "Artificial Intelligence: *A New Synthesis*", Morgan Kaufmann, San Mateo, California.

Noguchi, H., 1981, "Nonlinear Finite Element Studies on Shear Performance of RC Interior Column-Beam Joints", In IABSE Colloquium, Delft, The Netherlands, pp.639-653.

Noguchi, H., and Kashiwazaki, T., 1992, "Experimental Studies on Shear Performance of RC Interior Column-Beam Joints", In Tenth World Conference on Earthquake Engineering, Madrid, Spain, pp. 3163-3168.

Oka, K., and Shiohara, H., 1992, "Test on High -Strength Concrete Interior Beam-Column Sub-Assemblages", In Tenth World Conference on Earthquake Engineering, Madrid, Spain, pp. 3211-3217.

Ortiz, R., 1993, "Strut and Tie Modeling of Reinforced Concrete Short Beams and Beam-Column Joints", PhD. Dissertation, University of Westminster, 208 p.

Otani, S., Kobayashi, Y., and Aoyama, H., 1984, "Reinforced Concrete Interior Beam-Column Joints under Simulated Earthquake Loadings", In US-New Zealand- Japan Seminar on Design of Reinforced Concrete Beam-Column Joints, Monterey, CA.

Pantazopoulou, S., and Bonacci, J., 1992, "Consideration of Questions about Beam-Column Joints", ACI Structural journal, V. 89, No. 1, pp. 27-36.

Pantazopoulou, S., and Bonacci, J., 1993, "Parametric Investigation of Joint Mechanics", ACI Structural journal, V. 90, No. 1, pp. 61-71.

Pantazopoulou, S., and Bonacci, J., 1994, "On Earthquake Resistant Reinforced Concrete Frame Connections", Canadian Journal of Civil Engineering, V. 21, pp. 307-328.

Park, R., and Ruitong, D., 1988, " A Comparison of the Behavior of Reinforced Concrete Beam-column Joints Designed for Ductility and Limited Ductility", Bulletin of the New Zealand National Society of Earthquake Engineering, V. 21, No. 4, pp. 255-278.

Park, R., Billings, I, J., Clifton, G, C., Cousins, J., Filiatrault, A., Jennings, D, N., Jones, L, C, P., Perrin, N, D., Rooney, S, L., Sinclair, J., Spurr, D, D., Tanaka, H., Walker, G 1995, "The Hyogo-Ken Nanbu Earthquake (the great Hanshin earthquake) of 17 January 1995", report of the NZNSEE Reconnaissance Team, Bulletin of the New Zealand National Society for Earthquake Engineering, pp. 1-98.

Parker, D. E., and Bullman, P. J., 1997, "Shear Strength within Reinforced Concrete Beam-Column Joints", the Structural Engineer, V. 75, No. 4, pp. 53-57.

Paulay, T., 1989, "Equilibrium Criteria for Reinforced Concrete Beam-Column Joints", ACI Structural Journal, V. 86, No. 6, pp. 635-643.

Poincaré H., 1905, "Science and Hypothesis", London, Walter Scott Publishing.

Poole, D., Mackworth, A, K., and Goebel, R., 1998, "Computational Intelligence; A Logical Approach", Oxford University Press, Oxford, UK.

Rich, E., and Knight, K., 1991, "Artificial Intelligence", Second Edition, McGraw-Hill, New York.

Russell, S. J., and Norvig, P., 2003, "Artificial Intelligence: A modern Approach", Second Edition, Prentice Hall Series in Artificial Intelligence, New Jersey.

Sarsam, K. F., and Phipps, M. E., 1985, "The Shear Design of in Situ Reinforced Concrete Beam-Column Joints Subjected to Monotonic loading", Magazine of Concrete Research, V. 37, No. 130, pp. 16-28.

Scott, R. H., Feltham, I., and Whittle, R. T., 1994, "Reinforced Concrete Beam-Column Connections and BS8110," The Structural Engineer, V. 72, No. 4, pp. 55-60.

Sezen, H., Elwood, K. J., Whittaker, A. S., Mosalam, K. M., Wallace, J. W., and Stanton, J.F., 2000, "Structural Engineering Reconnaissance of the August 17, 1999 Earthquake: Kocaeli (Izmit), Turkey" ,Pacific Earthquake Engineering Research Center, University of California, Berkeley, PEER-2000/09.

Shiohara, H., 2001, "New Model for Shear Failure of RC Interior Beam-Column Connections", Journal of Structural Engineering, ASCE, V.127, No. 2, pp.152-160

Taylor, H. P. J., 1974, "The Behavior of in Situ Concrete Beam-Column Joints", Technical Report 42.492, Cement and Concrete Association, London, 32 p.

Teraoka, M., Kanoh, Y., Hayashi, K., and Sasaki, S., 1997, "Behavior of Interior Beam-and-Column Sub Assemblages in RC Frame", First International Conference on High Strength Concrete, Kona, Hawaii, pp. 93-108.

Teraoka, M., Kanoh, Y., Tanaka, K., and Hayashi, K., 1994, "Strength and Deformation Behavior of RC Interior Beam-Column Joint Using High Strength Concrete", In Proceedings, Second US-Japan-New Zealand- Canada Multilateral Meeting on Structural Performance of High Strength Concrete In Seismic Regions, Honolulu, Hawaii, pp. 1-14.

The Math Works., 2007, "MATLAB (2007)", Orchard Hill, Michigan, United States.

Uang, C. M., Elgamal, A., Li, W. S., and Chou, C. C., 1999, "Ji-Ji Taiwan Earthquake of September 21, 1999", A Brief Reconnaissance Report, Department of Structural Engineering, University of California, San Diego, http://www.structures.ucsd.edu/Taiwaneq

Vollum, R. L., 1998, "Design and Analysis of Exterior Beam-Column Connections", PhD. Dissertation, Department of Civil Engineering, Imperial College, London, 603p.

Walker, S. G., 2001, "Seismic Performance of Existing Reinforced Concrete Beam Column Joints", MSCE Thesis, University of Washington, Seattle. 308 p.

Will, G. T., Uzumeri, S. M., and Sinha, S. K., 1972, "Application of Finite Element Method to Analysis of Reinforced Concrete Beam-Column Joints", In Proceeding of Specialty Conference on Finite Element Method in Civil Engineering, CSCE, EIC, Canada, pp. 745-766.

Winston, P, H., 1992, "Artificial Intelligence", Third Edition, Addison-Wesley, Reading, Massachusetts.

Zaid, S., 2001, "Behavior of Reinforced Concrete Beam-Column Connections under Earthquake Loading", PhD Dissertation, Department of Architecture, University of Tokyo, Japan.

APPENDIX A

Beam-Column Joints Database

Table 8. Database for monotonically loaded exterior beam-column joints

Nr	Author	Label	b_b (cm)	h_b (cm)	b_c (cm)	h_c (cm)	f_c' (MPa)	V_j (KN)
1	Kordina (1984)	RE4	20	30	20	20	30.6	195.136478
2	Sarsam and Phipps (1985)	EX2	15.2	30.5	15.7	20.4	53.9	188.63268
3	Bosshard and Menn (1984)	R2	24	20	30	20	33	257.280952
4		R3	24	20	30	20	28.2	234.519048
5		R4	24	20	30	20	26.8	217.515873
6	Hoekstra (1977)	1402	15	30	15	20	19.3	109.137359
7		1404	15	30	15	20	24.6	125.966184
8		1408	15	30	15	20	24.9	114.176006
9		1410	15	30	15	20	23.4	129.291948
10		1615	15	30	15	20	21.5	125.890151
11		1616	15	30	15	20	21.5	134.465161
12	Nilsson (1972)	T1	20	20	20	20	24.3	83.4510313
13		TI5	20	20	20	20	30.1	55.902682
14		TI4	20	20	20	20	25.2	79.7840358
15	Taylor (1974)	P1/41/24	10	20	14	14	33	92.2895425
16		P2/41/24	10	20	14	14	29	96.0895425
17		P2/41/24A	10	20	14	14	46.5	135.89085
18		A3/41/24	10	20	14	14	27	95.4359477
19		B3/41/24	10	20	14	14	22	82.4104575
20		C3/41/24BY	10	20	14	14	31.7	77.5352941

21		C3/41/13Y	10	20	14	14	28.2	72.6825947
22		C3/41/24Y	10	20	14	14	59.5	126.208497
23		D3/41/24	10	20	14	14	53	139.794771
24		E3/41/24A	10	20	14	14	42.8	73.0993464
25		E3/41/24B	10	20	14	14	44.5	81.8640523
26		E3/41/24C	10	20	14	14	41.6	114.929412
27	Parker and Bullman (1997)	4b	25	50	30	30	39.5	223.883895
28		4c	25	50	30	30	36.9	275.799001
29		4d	25	50	30	30	39.5	243.35206
30		4e	25	50	30	30	40.3	259.575531
31		4f	25	50	30	30	37.8	297.014357
32		5b	25	50	30	30	43.8	382.873908
33		5f	25	50	30	30	43.8	522.395755
34	Scott *et al* (1994)	C1AL	11	21	15	15	30.2	103.432099
35		C3L	11	21	15	15	32.2	101.780247
36		C4	11	21	15	15	37.9	139.355556
37		C4A	11	21	15	15	40.6	149.366667
38		C4AL	11	21	15	15	32.4	133.365432
39		C6	11	21	15	15	36.3	102.297531
40		C6L	11	21	15	15	42.1	122.319753
41		C7	11	30	15	15	31.9	94.5341564
42		C9	11	30	15	15	32.6	82.3308642
43	Hamil (2000)	C6LN0	11	21	15	15	47.4	99.5111111
44		C6LN1	11	21	15	15	47.4	103.949383
45		C6LN3	11	21	15	15	45	120.467901
46		C6LN5	11	21	15	15	33.4	140.907407
47		C6LN1B	11	21	15	15	35.7	95.5901235
48		C6LN1T	11	21	15	15	36.5	112.208642
49		C6LN1TA	11	21	15	15	45	116.02963
50		C6LN2A	11	21	15	15	47.4	124.288889
51		C6LN2B	11	21	15	15	47.4	149.166667

52		C6LN3A	11	21	15	15	42.7	132.648148
53		C6LN3B	11	21	15	15	48.2	157.525926
54		C6LN3C	11	21	15	15	44.2	145.345679
55		C4ALN0	11	21	15	15	38.8	112.208642
56		C4ALN1	11	21	15	15	41.9	140.907407
57		C4ALN3	11	21	15	15	38	145.345679
58		C4ALN5	11	21	15	15	46.6	165.785185
59		C4ALN1T	11	21	15	15	36.5	128.82716
60		C6L04SF	11	21	15	15	39.6	99.5111111
61		C6L04LF	11	21	15	15	30.4	95.5901235
62		C4AL04SF	11	21	15	15	32.7	128.82716
63		C4AL15SF	11	21	15	15	34.2	137.08642
64		C4AL04LF	11	21	15	15	30.4	145.345679
65		C4AL15LF	11	21	15	15	36.5	149.166667
66		C6LH0	11	21	15	15	99.1	149.166667
67		C6LH1	11	21	15	15	100	153.704938
68		C6LH3	11	21	15	15	94.7	170.223457
69		C7LNO	11	30	15	15	35	92.9880658
70		C7LN1	11	30	15	15	34.2	103.333745
71		C7LN3	11	30	15	15	36.5	123.402058
72		C7LN5	11	30	15	15	36.5	140.466667
73		C9LN0	11	30	15	15	37.3	86.6806584
74		C9LN1	11	30	15	15	35	86.3691358
75		C9LN3	11	30	15	15	33.4	98.472428
76		C9LN5	11	30	15	15	31.9	119.263786
77		C6LN1R	11	21	15	15	45	114.377778
78		C6LN1E	11	21	15	15	40.3	119.333333
79	Ortiz (1993)	BCJ1	20	40	20	30	34	297292.69
80		BCJ2	20	40	20	30	38	330396.73
81		BCJ3	20	40	20	30	33	344500.65
82		BCJ4	20	40	20	30	34	343536.6

83		BCJ5	20	40	20	30	38	339516.99
84		BCJ6	20	40	20	30	35	339116.99
85	Hegger *et al* (2003)	RK4	15	30	15	20	51.7	346686.17
86		RK7	15	40	15	20	54.7	384729.42
87		RK8	15	30	15	20	38.6	422472.88
88		RK9	15	30	15	20	42.8	254241.19

Table 9. Database for cyclically loaded interior beam-column joints

Nr	Author	Label	b_b (cm)	h_b (cm)	b_c (cm)	h_c (cm)	f_c' (MPa)	V_j (KN)
1	Durrani and Wight (1982)	X1	42	36.2	36.2	36.2	34.34	783.0134
2	Otani *et al* (1984)	J4	30	30	30	30	25.7	353.2385
3		J5	30	30	30	30	28.74	421.4073
4		J6	30	30	30	30	28.74	309.6821
5	Meinheit and Jirsa (1977)	U1	45.8	33.1	33.1	33.1	26.21	762.6102
6		U2	45.8	33.1	33.1	33.1	41.79	1115.585
7		U3	45.8	33.1	33.1	33.1	26.62	854.1221
8		U5	45.8	33.1	33.1	33.1	35.86	1072.012
9		U6	45.8	33.1	33.1	33.1	36.76	1154.809
10		U12	45.8	33.1	33.1	33.1	35.17	1357.441
11		U13	45.8	33.1	33.1	33.1	41.31	1085.084
12	Walker (2001)	PEER14	50.9	40.7	40.7	40.7	31.77	858.2053
13		PEER22	50.9	40.7	40.7	40.7	38.41	1154.882
14		PEER0995	50.9	40.7	40.7	40.7	60.46	1335.351
15		PEER4150	50.9	40.7	40.7	40.7	32.99	1800.7
16	Park and Ruitiong (1998)	U4	45.7	30.5	30.5	30.5	40.1	472.5013
17	Nouguchi and	J1	30	30	30	30	70	907.928

18	Kashiwazaki (1992)	J4	30	30	30	30	70	958.7247
19		J5	30	30	30	30	70	943.3283
20		J6	30	30	30	30	53.5	836.0909
21	Oka and Shiohara (1992)	J10	30	30	30	30	39.2	739.683
22		J11	30	30	30	30	39.2	875.5431
23	Kitayama *et al* (1987)	J6	30	30	30	30	25.69	314.8395
24	Park and Milburn (1983)	U2	45.7	30.5	30.5	30.5	46.9	877.9709
25	Endoh *et al* (1991)	HLC	30	30	30	30	40.6	486.4731
26		LA1	30	30	30	30	34.81	613.7374
27		A1	30	30	30	30	30.6	576.0872
28	Higashi and Ohwada (1969)	SD35Aa-4	30	20	20	20	30.3	118.986
29		SD35Aa-7	30	20	20	20	38.05	114.87
30		SD35Aa-8	30	20	20	20	38.05	118.986
31		LSD35Aa-1	30	20	20	20	41.09	115.7847
32		LSD35Aa-2	30	20	20	20	41.09	110.7493
33		LSD35Ab-1	30	20	20	20	41.09	114.184
34		LSD35Ab-2	30	20	20	20	41.09	106.4
35	Atalla and Agababian (2004)	SHC1	20.3	12.7	12.7	12.7	56.54	53.8608
36		SHC2	20.3	12.7	12.7	12.7	59.55	53.22714
37		SOC3	20.3	12.7	12.7	12.7	47.2	50.69252
38	Teraoka *et al* (1997)	HJ4	40	40	40	40	53.98	965.1939
39		HJ6	40	40	40	40	53.98	1092.387
40	Hayashi *et al* (1994)	NO44	40	40	40	40	54.27	838.9312
41		NO45	40	40	40	40	54.27	998.8635
42		NO47	40	40	40	40	54.27	965.1939
43		NO48	40	40	40	40	54.27	1127.93
44		NO49	40	40	40	40	54.27	1425.341
45		NO50	40	40	40	40	54.27	1091.453
46	Zaid (2001)	S1	30	30	30	30	24.02	168.1613

47		S2	30	30	30	30	24.02	181.9828
48		S3	30	30	30	30	24.02	301.7689
49	Joh Goto and Shibata (1991)	B1	35	30	30	30	21.2	242.3053
50		B2	35	30	30	30	22.54	262.1649
51		B8HH	35	30	30	30	25.61	263.2828
52		B8HL	35	30	30	30	27.41	275.4343
53		B8LH	35	30	30	30	26.9	275.4343
54		B 8MHY	35	30	30	30	28.11	263.2828
55	Fujii and Morita (1991)	A1	25	22	22	22	40.22	237.1404
56		A2	25	22	22	22	40.22	220.2043
57		A3	25	22	22	22	40.22	237.1404
58		A4	25	22	22	22	40.22	241.98

VITA

Graduate College
University of Nevada, Las Vegas

Eslam Hassan Khalifa

Local Address:
4441 Escondido Street, # 2203
Las Vegas, Nevada 89119

Home Address:
11A- Adly Kafafy Street, #10
Heliopolis, Cairo
Egypt

Degrees:
Bachelor of Civil Engineering, Structural Department, 2003
School of Engineering, Ain Shams University
Cairo, Egypt

Conference Papers:

[1] Khalifa, E., and Said A., 2008, "Evaluating Shear Capacity of Monotonically Loaded Beam-Column Joints Using Genetic Algorithms". Proceedings of the 5th International Engineering and Construction Conference, Irvine, CA.

[2] Khalifa, E., Said, A. and El-Chabib, H., 2008, "Predicting Shear Strength of RC Beam-Column Joints Subjected to Monotonic Loading Using Artificial Neural Networks". Proceedings of the 5th International Engineering and Construction Conference, Irvine, CA.

Ongoing Publication:

[1] Khalifa, E., and Said, A., "Investigating Shear Capacity of Cyclically Loaded Beam-Column Joints using Neural Networks".

[2] Khalifa, E., and Said, A., "Investigating Shear Capacity of Cyclically Loaded Beam-Column Joints using Genetic Algorithms".

[3] Said, A., and Khalifa, E., "Evaluating Existing Equations for Calculating Punching

Shear Capacity of RC Slabs With Shear Reinforcement".

[4] Said, A., and Khalifa, E., "Evaluating Existing Equations for Calculating Punching Shear Capacity of RC Slabs With Shear Reinforcement".

Thesis Title: Investigating Shear Capacity of RC Beam-Column Joints using Artificial Intelligence Techniques.

Thesis Examination Committee:

Chairperson, Dr. Aly Said, Ph.D.
Committee Member, Dr. Samaan G. Ladkany, Ph.D
Committee Member, Dr. Barbra Luke, Ph.D
Committee Member, Dr. Ying Tian, Ph.D
Graduate Faculty Representative, Dr. Brendan J. O'tool, Ph.D.