SHEAR STRENGTH EVALUATION OF REINFORCED RECYCLED AGGREGATE CONCRETE BEAMS

BY

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ABSTRACT

A theoretical study is conducted to investigate the shear behaviour of recycled aggregate concrete (RAC) beams with and without shear reinforcements along with the performance evaluation various Code based/other existing equations in predicting shear strength. In addition, three artificial neural network (ANN) models for shear strength prediction of RAC beams with and without shear reinforcements are developed and their performance validated by using 108 beams from available research studies. Most of the Codes and existing methods underestimate the shear capacity of RAC beams with/without shear reinforcement. However, over estimation of shear strength by Codes/existing methods for about 10% RAC beams needs to be addressed when using such Codes/existing methods for shear strength prediction. All three ANN models are found to predict shear strength of RAC beams. Developed ANN models are able to simulate the effect of shear reinforcement on the shear strength of RAC beams.

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NOTATIONS

A_s, A_{sw}	The cross-sectional area of longitudinal reinforcement
A_v, A_{vs}	Area of vertical reinforcement
a, a_v	Shear span
a/d	Shear span to depth ratio
a_g	Maximum aggregate size
b_v, b_w	Width of the web section
d, z	Distance from the extreme compression fiber to the center of gravity of the steel
d_v	Effective shear depth, taken as the greater of 0.9d or 0.72h
b_0	The net minimum width of the member
b_{red}	Reduced web breadth
d_0	Depth to the outermost layer of tensile reinforcement
ε	Degree of agreement
f', f_{1cd}	Concrete compressive strength
f_y	Specified yield strength of non-prestressed reinforcement or anchor steel
f_{ck}	Characteristic, compressive cylinder strength of concrete at 28 days
f_{cd2}	Design value of the concrete tensile strength
f_{cv}	The effective shear strength of the concrete
F_{rcw}	The compression of web concrete
F_{rtw}	Thetension of web steel
f_{sy}, f_{yt}	Yield strength of the stirrups
f_{ywd}	Design value of the web (shear) reinforcement
$f_{y\alpha d}$	The design strength of reinforcement
h	Number of hidden neurons
i	Number of inputs
j_d	Internal lever arm
M_f	Factored moment at the section
M_u	Factored moment at the section
o	Number of outputs
RAC	Recycled aggregate concrete (concrete made by recycled aggregates)

RCA	Recycled concrete aggregate (aggregates produced from recycling concrete)
s	Spacing of the transverse reinforcement
s_z	Equivalent crack spacing parameter
V	Shear force in the section
V_{avg}	Difference between the averages predicted value and the average target value
V_{cv}	Difference between the coefficient of variance of the predicted and target
V_{cz}	Concrete shear force
V_d	Dowel force in equation
V_i	Interface shear force
V_{max}	Difference between the maximum value of predicted data and target data
V_{min}	Difference between the minimum value of the predicted data and target data
V_{range}	Difference between the range of predicted values and the range of target values
$V_{Rd,c}$	Design value for the shear resistance
V_u	Factored shear force on the section
V_{u1}	The ultimate shear force failure due to diagonal compression in the web
V_{u2}	The ultimate shear force failure due to tension in the web
V_{uc}	Shear resisted by concrete
V_{us}	Shear resisted by stirrups
V_{σ}	Difference between the standard deviation of the predicted data and the target data
V_e	Experimentally determined total shear resistance
V_f	Factored shear force
V_p	Predicted total shear resistance
V_r	Factored shear resistance
V_c	Nominal shear strength provided by concrete
V_n	Nominal shear strength
V_s	Nominal shear strength provided by shear reinforcement
w/c	Water-cement ratio
z	The mechanic lever arm
α	The angle of the reinforcement with the member's axis
ε_x	The longitudinal strain in the web
Φ	Strength reduction factor

ϕ_c	Resistance factor for concrete
ϕ_s	Resistance factor for non-prestressed reinforcing bars
ρ_v	Transverse reinforcement ratio
$\rho_w, \rho_l, \rho_s, \rho$	Ratio of longitudinal reinforcement
θ_v, θ	The angle between the axis of the concrete compression strut and the longitudinal axis of the member
γ_c	The partial safety factor for concrete

CHAPTER ONE

1. INTRODUCTION

1.1 GENERAL

The worldwide high volume generation of construction and demolition waste (CDW) requires sustainable solutions to reduce the environmental impacts like landfill extensions and to support the preservation of our limited natural resources. Concrete is one of the most widely used construction material with the largest volume of consumption. According to CSI (The Cement Sustainability Initiative), concrete is the second most consumed material after water with 25 billion globally manufacturing every year. Aggregates are the major components of concrete and have a significant effect on engineering properties of the final product (Manzi et al., 2013). They make about 70-80 % of concrete components while they are the recyclable part of the concrete. The demand of new aggregates in United States is estimated to be two billion tons for each year, while the annual production of construction waste is estimated to be 123 million tons according to the Federal Highway Administration (FHWA, 2004). In Europe, CDW has been identified as a major waste stream according to European Union. Eurostat estimates 970 million tons/year for Europe (Sonigo et al., 2010). Korean domestic construction waste is generated at a rate of more than 100,000 tons every day. This amount will be 90 million tons, considering the current rate by 2021(Thomas et al., 2014).

Many of countries have already started to use recycled concrete aggregate (RCA). Japan was the first country to become interested in the study of RCA concrete and Kasai (1974) first released the criteria for RCA and RCA concrete in 1974 (Thomas et al., 2014). According to Construction Materials Recycling Association (CMRA, 2010), nearly 140 million tons of concrete end up to recycling plants every year.

There are some concerns about using of recycled aggregate concrete in the structural elements. The quality of recycled aggregates affected by several parameters like quality of the parent concrete. Moreover, recycled aggregate particles naturally contain virgin aggregate with part of remaining mortar attached to the surface of aggregate called adhered mortar. According to the research, this residual mortar negatively effects the physical properties of RCA. It increases water absorption and porosity, while decreases density and specific gravity of RCA comparing to natural aggregates (NA). As a matter of fact, the use of recycled aggregate decreases the mechanical properties of hardened concrete. It can increases the drying shrinkage, creep, carbonation rate and water sorptivity, and decreases the compressive strength, modulus of elasticity and freezing and thawing resistance in comparison with concrete made by natural aggregate. These weaknesses are able to be improved with the use of appropriate mix design and mineral admixtures (Kou and Poon, 2015). The other drawback of using RCA lies in its characterization which requires comprehensive studies and is not an easy procedure as the properties of RCA varies by the types, quality and functionality of the sources of aggregates.

1.2 SCOPE AND OBJECTIVES

The potential of using of RCA in construction industry is enormous as it can lead to cost-effective sustainable structures. The objectives of this study are to:

- Conduct a literature review on recycled concrete aggregate concrete,
- Study the shear behavior of reinforced concrete (RC) beams made by recycled concrete aggregates,
- Investigate the applicability of the current Code based and other existing equations to evaluate the shear strength of recycled aggregate concrete (RAC) beams with and without shear reinforcements and
- Develop artificial neural network (ANN) models for predicting the shear strength recycled aggregate concrete beams with and without web/shear reinforcements based on the currently available experimental results.

A database of 96 beams gathered from different experimental studies of previous researches is used to accomplish these objectives.

1.3 REPORT OUTLINE

This project report contains five chapters. **Chapter One** gives an overview about the recycled aggregate concrete and scope/objectives of the study. **Chapter Two** includes a literature review covering the background of recycled aggregate and its application in different types of concrete. It also provides information about the physical properties and mechanical properties of the hardened concrete made with recycled concrete aggregate (RCA). A brief review of the evaluation of shear strength of reinforced concrete beams and truss analogy method is also presented in this chapter. Moreover, the various code provisions currently available and existing design equations from different research studies have also been discussed in this chapter.

Chapter Three includes the analysis of performance of various Code based and existing equations for predicting shear strength of RC beams made with recycled concrete aggregates. Experimental details and results of previous research studies are described to understand the shear behaviour of RCA beams. In total, 96 full-scale RC beam specimens made with RCA are introduced in two groups: beams with and without web reinforcements. Each testing program planned to investigate the effect of two or more influential parameters on shear strength of the beams. The experimental shear strengths of beams are compared with those obtained from current Codes, standards and existing design equations/methods.

In **Chapter Four**, three ANN models for predicting the shear strength of RCA beams with and without web reinforcements are developed. ANN models are trained and their performance is validated using experimental data collected from previous research studies described in **Chapter Three**.

Chapter Five presents conclusions of this study and suggests recommendations for future research.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 GENERAL

This chapter is presented in three parts. The first part comprised a literature review on recycled concrete aggregate. The second part provides a review of different mechanisms of the shear behavior of reinforced concrete beams and the corresponding design approaches. The third part introduces design codes and other exiting design equations/ methods to evaluate the shear capacity of RC beams.

2.2 RECYCLED AGGREGATE CONCRETE

2.2.1 Background

Construction and demolition waste (CDW) is recognized as a major waste stream made by human around the world. Although there is no global statistical data for CDW generation, but it is stated that approximately 900 million tonnes of construction and demolition waste is generated in USA, Europe and Japan (WBCSD, 2012). Recycling of concrete is a relatively new approach to address the two main environmental concerns, to manage the uninterrupted waste generation and to protect the raw material resources. Many of countries have already started to use recycled aggregates. According to Construction Materials Recycling Association (CMRA, 2010), nearly 140 million tons of concrete end up to recycling plants every year.

Japan was the first country to become interested in the study of RCA concrete and Kasai (1974) first released the criteria for RCA and RCA concrete in 1974. The first investigation on the recycled aggregate concrete beams is also published in Japan (Mukai and Kikuchi, 1988). Italy is one of the countries that provides for 30% coarse RCA replacement of virgin aggregate, in order to make structural concrete. This process has been taking place since 2009 (Corinaldesi, 2011). Germany has been set recycling of building and demolition waste at 40% since 1991. Holland has allowed 20% of RCA for course aggregate to produce new concrete, while considering the same properties of fresh and hardened conventional concretes since 1994 (Oikonomou, 2005).

The European Code EN 206 (Annex E) allows for the usage of up to 50% replacement of coarse recycled material in specific conditions (European Committee for Standardization, 2012). In order to ensure the success of reuse of recycled aggregates, it is important to ensure that three main things are achieved (Dosho, 2007):

- assurance of safety and quality of the recycled aggregate
- decrease in environment impact by conserving the natural aggregate
- increase in cost effectiveness of construction

2.2.2 A review of experimental studies

Although the study of the structural behavior of recycled aggregate concrete (RAC) draws attention of researchers since 1974, there is still limited experimental results. It is stated that concrete with RCA has a lower compressive strength and higher absorption at the same water-cement ratio (w/c) and slump than concrete with natural aggregates (NA), while it shows higher freezing-and-thawing durability (Buck, 1977). Etxeberria et al. ((2007) conducted an experimental study of the shear behavior and strength of beams made with RAC. They examined twelve beam specimens with the same compressive strength and different RCA percentage and different transvers reinforcement. They found that the substitute of less than 25% coarse aggregate was not able to have outstanding effects on the shear capacity of RAC beams. Fathifazl et al. (2011) suggested a new mix design method known as Equivalent Mortar Volume (EMV) method. They proved that the shear capacity of recycled aggregate concrete beams will be the same or even higher than the conventional concrete provided that the EMV method applies for the mix design. Knaack and Kurama (2014) stated that the effect of recycled concrete aggregate on shear and flexural behavior of beams is small, while this effect is considerable on the initial stiffness and ultimate flexural deflection.

2.2.3 RCA Production

Recycled concrete aggregates (RCAs) are obtained from demolition of concrete structures after crushing equipment breaks them into smaller particles. Three main types of aggregates produced by construction and demolition waste (CDW) are introduced: crushed concrete, crushed masonry, and mixed demolition debris (Silva et al., 2015).

Production process of RCA is similar to natural coarse aggregates and involves the use same equipment, crushers, screens, removal impurities and transportation facilities (Sharma and Singla, 2014). The Portland Cement Concrete (PCC) recycling procedure consists of demolition, breaking, removing and crushing concrete from an approved source. The steps to produce recycled concrete aggregate based on National Highway Institute (NHI, 1998) and the Environmental Council of Concrete Organization (ECCO, 1999) are as follows:

i. Evaluation of Source Concrete and Removal of Contaminants like steel reinforcing bars, wood and soil,
ii. Breaking of Demolished Concrete and Removal of Embedded Steel and
iii. Crushing and Sizing and blending

As the presence of mortar attached to the aggregate particles is recognised to be the main factor contributing to quality issues relating to RCA, the separation processes to improve the quality of RCA is an important part of the production.

According to Akbarnexhad et al. (2011), there are two types of recycled aggregates; type I and type II, as displayed in Figure 2.1. Type I has a bigger size and may be comprised of more than one natural aggregates, they are both attached to each other, and are surrounded by cement paste. Type II is comprised of only mortar, without the presence of natural aggregate. The amount of adhered mortar depends on several factors like the strength of the original concrete, particle size, crushing procedure and the number of crushing procedures, so it is different for every particle. (Akbarnezhad et al., 2013).

According to results, the average mortar content can be in the range from 20 to 70% by mass, depending on the parameters mentioned above (Li, 2008; Akbarnezhad et al., 2013).

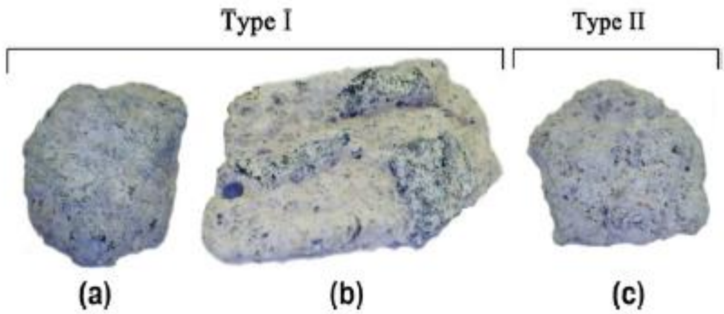


Figure 2.1 Various types of RCA comprising (a) a granite particle surrounded by adhering mortar, (b) three granite particles held together and surrounded by mortar, (c) only mortar (Akbarnezhad et al., 2011)

2.2.4 Recycled concrete aggregates (RCA) properties

The adhered mortar and its relative content in RCA particles play a significant role in physical and mechanical properties of this type of aggregates. Concrete made by RCA contains two types of Interfacial Transition Zone (ITZ), in which one of them is between the original aggregate, and residual mortar, and the other one is between the RCA and fresh paste as shown in Figure 2.2 (Ryu, 2002). As a result, coarse RCA has relatively lower density, higher water absorption and higher abrasion loss than natural aggregate (NA), therefore the use of RCA in higher replacement ratios particularly as structural concrete is not suggested (Dhir et al., 2004; Dos Santos et al., 2004; Tam and Tam, 2008). Some of the physical and mechanical properties of RCA are discussed as follows:

Shape and texture: Previous studies done by researchers like Malhotra (1976) used microscopic scanning to identify the shape of the recycled aggregate particles. RCA demonstrated more angular particles compared to natural aggregates. The angular shape of recycled aggregate is because of adhered mortar (Topco and Mustafa, 1997).

Figure 2.2 Interfacial Transition Zones (ITZs) locations on RCA concrete (Ryu, 2002)

Absorption capacity: The specific gravity and water absorption are directly affected by the presence of adhered mortar (Parekh and Modhera, 2011). The porous nature of the cement paste portion of the recycled aggregates increases its absorption capacity. Absorbing capacity can be controlled by the limited use of recycled fine aggregate.

Specific gravity: Same as absorption capacity, the specific gravity which is a measure of the density of an aggregate of RCA is adversely affected by adhered mortar. According to Hansen and Narud (2003), the specific gravity (saturated surface dry, SSD) value of the RCA is 4.6 to 6.5 % less than natural aggregates. It gains lower specific gravity because of the porosity and entrained air structure of the adhered aggregate.

L.A. Abrasion mass loss: According to studies, adhered mortar in the recycled aggregate causes to form a weak zone between the parents aggregate and the cement mortar paste. As a results RCA has lower mechanical strength compare to NA and (Sharma and Singla, 2014). Based on experimental studies such as the aggregate impact value test, aggregate crushing value test and Los Angeles abrasion or micro-deval degradation tests, RCA are not as strong as the virgin aggregates (Parekh and Modhera, 2011).

Durability: Sulfate soundness test is one of the common tests implemented in USA for the prediction of freezing and thawing durability of aggregates. However, the results of different researchers show large differences about this property of RCA. This is debated by many of researchers that the sulfate soundness test is an improper test because of the fact that chemical attack occurs to the concrete materials by the exposure to environment in a long period of the time (Kou et al., 2002).

2.2.5 Use of RCA in different types of concrete

RCA applies in several types of construction such as Aggregate Base Course, Pipe Bedding, Paving Blocks, Building Blocks, and Landscape Materials. It is reported by FHWA (2004), 38 states within the United Stated are recycling demolished concrete for granular base material. It is also common to use of the RCA in providing the Portland cement concrete in the structural elements. The natural aggregate is replaced by the RCA either partially or entirety considering the type of construction and the quality of the RCA.

2.2.6 Hardened RCA concrete properties

The effects of implementing RCA into concrete are highly dependent on the nature, composition, and gradation of the RCA (American Concrete Pavement Association, 2010). Following are a brief review of properties of the hardened concrete made by recycled aggregates:

Compressive Strength: In structural concrete with specific mix design, it is recommended that the RCA replaced with a portion of natural aggregates. In fact there is a defined limit which RCA can be replaced with NA which the compressive strength is not effected significantly, as the compressive strength of recycled aggregate concrete generally decrease with increasing recycled aggregate contents. In some research papers it is stated that concrete made with recycled aggregates has a compressive strength about 22%-32% less than the strength of the concrete made with natural aggregates (Jitender and Sandeep, 2014).

Tensile and Flexural Strength: The results of the experiments made by many researchers show that the tensile splitting strength of the given concrete mix decreased as the RCA content increased (Kou et al., 2007).

Modulus of Elasticity: According to the previous research the stiffness or modulus of elasticity of RCA concretes is 20 to 40 % lower than that of conventional concrete with the same water-cement ratio (Anderson et al. 2009). This reduction in modulus of elasticity might be due to the presence of adhered mortar which is attached to the aggregate with lower modulus of elasticity.

Drying Shrinkage and Creep: Some researchers believe that concrete mixes with RCA have higher paste content (due to adhered mortar) and higher drying shrinkage will occur to them compared with conventional concrete. According to Parekh and Modhera (2011), increase of shrinkage occurs due to higher water cement ratios, higher paste contents, and lower coarse aggregate contents. The amount of voids existing in fresh cement paste can control the drying shrinkage in hardened concrete.

Bond Strength between Concrete and Reinforcement: Anderson et al. (2009) observe that application of RCA in concrete can cause a remarkable reduction in bond strength between the concrete and the reinforcement used in the concrete. The reduction will be higher by fine aggregates are used.

Durability: The following characteristics can specify the durability of hardened concrete: freeze-thaw resistance, d-cracking susceptibility, permeability and absorption, and alkali silica reaction susceptibility. Buck (1977), stated that when comparing the two types of concretes with virgin and natural, with the same design mix, RCA shows a better frost resistance due to its higher entrained air content and porous nature of adhered mortar.

2.3 SHEAR STRENGTH EVALUATION OF RC BEAMS

2.3.1 Background

Reinforced concrete (RC) beams demonstrate a complex behavior under shear compared to their flexural behavior. The mechanism of internal load transfer is notably different before and after formation of cracks. The shear failure of RC beams has a sudden brittle nature due to lack of ductility and cracking pattern characteristics (Neilson, 2004). In this section an overview of the shear behavior of RC beams and corresponding design approaches are presented. The parameters affecting shear behavior, shear transfer mechanisms and shear design principles are also described. A brief description of the truss analogy method is presented at the end.

2.3.2 Shear behavior of reinforced concrete beams

The behavior of an uncracked beam subjected to flexure and shear effects is the same as homogenous elastic beam. According to the theoretical concepts, the principal stresses are acting on each of the beam small elements perpendicular to the face of the element. The direction of the principal stress varies based on the element location along the cross section of the beam from extreme compression and tension fibers to neutral axis. Stress trajectories as hypothetical curves could be drawn according to the values and directions of the principal stresses. When the concrete tensile stress exceeds its modulus of rupture, the diagonal tension cracks begin to occur along the theoretical stress trajectories. The most common shear failure is diagonal tension failure or shear tension failure which is characteristic of beams subjected to point loads with shear span to depth ratio (a/d) value ranging from 2.5 to 5.5. The flexural cracks begins in the mid-span, by a few fine vertical cracks and eventually grow till the bond between the reinforcing steel and the concrete decreases significantly. At this stage, one of the inclined cracks widens and the crack propagates to the top of the beam and a sudden failure occurs, as in shown in Figure 2.3.

According to ASCE-ACI 426 (1973): " immediately after the inclined cracks develop, the concrete shear force (V_{cz}) accounts for up to 40% of the total shear force, whereas the remaining portion is resisted by the dowel force (V_d) and the interface shear force (V_i)" as shown in Figure 2.4.

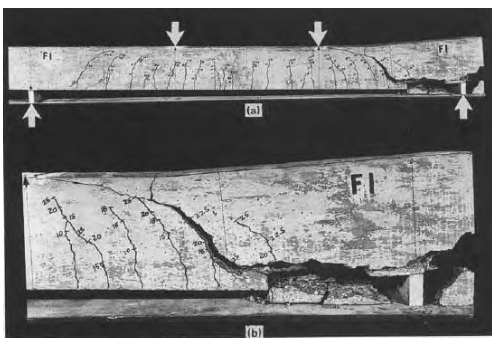


Figure 2.3 Shear failure of RC beams (Neilson, 2004)

The interface transfer shear mechanism becomes less effective by widening the crack and the (V_i) decreases. Finally, the dowel force (V_d) is removed and inclined cracks develop toward the upper face of the beam with an angle of almost 45 degrees.

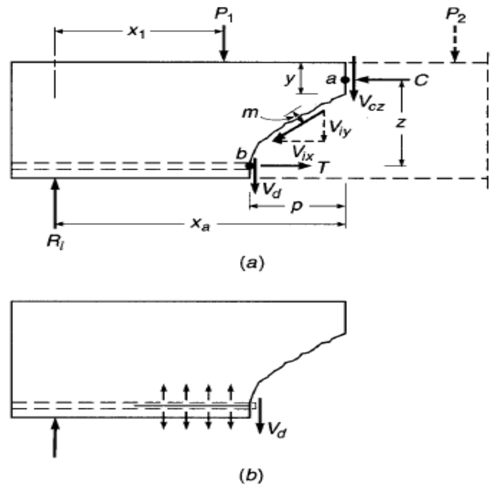


Figure 2.4 Forces at a diagonal crack in a beam without web reinforcement (Neilson, 2004)

2.3.3 Shear strength parameters

The factors and parameters govern the shear strength of a beam are the presence of web reinforcement, longitudinal reinforcement, coarse aggregate size, presence of axial loads, depth of the member, tensile strength of the concrete, and shear span to depth ratio (a/d). Shear design equations contain some but not all the parameters mentioned above.

The effect of web reinforcement: Shear strength of concrete beams increases by using web reinforcement. The brittle nature of the concrete makes it necessary to use web reinforcement to provide more ductility in concrete beams and prevent shear failure. Vertical stirrups are normally used as web reinforcement and depending on shear requirements are spaced along the beam. Shear reinforcement has a small effect before diagonal cracks form. After cracking, major part of shear force is carried by the shear reinforcement. Moreover, it assist preventing crack growth and propagation.

The effect of longitudinal reinforcement: The longitudinal reinforcement ratio (ρ_s) increases the shear capacity of the beam section. It reduces the width and severity of flexural cracks.

The effect of aggregate size: The coarse aggregate type and size affect the shear capacity significantly, especially for beams without stirrups. As an example, the tensile strength of lightweight aggregate is lower than the normal aggregate. Moreover, large diameter aggregate increases the roughness of the crack surfaces, allowing higher shear stresses to be transferred.

The effect of axial forces: The results of research works indicated that axial compression forces increase the shear capacity and axial tension forces decrease considerably the shear strength.

The effect of beam size: The size of the beam influence the shear capacity of beams at failure. By increasing the overall depth of a beam shear the force at failure becomes smaller.

The effect of shear span to depth ratio: The shear strength increases by the decrease of shear span to depth ratio decreases. The effect is clearly observed in deep beams ($a/d \leq 2.5$). In deep beams, the initial diagonal cracking suddenly grows to the entire length of the test region (Wight and MacGregor, 2009).

The effect of concrete strength: The shear strength is also greatly affected by tensile strength but as the compressive strength is pretty easier to be tested than tensile strength, it is common to use compressive strength ($f'c$).

2.3.4 Shear analysis methods

According to the ASCE-ACI Committee 426 (1973), shear is transferred by the following four mechanisms: shear stress in the uncracked concrete, interface shear transfer or aggregate interlock, dowel action, and arch action. Residual tensile stresses is the new shear transfer mechanism introduced by ASCE-ACI Committee 445 in 1998. It states that after first cracks occur as long as the crack width do not exceed 0.00197-0.0059 inches, the tensile forces will still be transferred by the concrete section.

Different shear design methods and guidelines use different approaches in shear modeling such as truss model, Strut and Tie Model (STM), Modified Compression Field Theory (MCFT), and fracture mechanics approach. As it is not the purpose of this study to provide a detailed description of the design principals, a brief explanation of the truss model is given as a typical modeling.

2.3.4.1 The Truss analogy method

A conservative design requires 45° angle of inclination based on the traditional truss model. The truss model theory provides an accurate modeling for shear resistance of concrete beams after cracking and various load conditions could be included reliably by applying the model (ASCE-ACI Committee 445, 1998; ASCE-ACI Committee 426, 1973).The model consists of the several vertical tension members and diagonal compression members parallel to the inclined shear crack. The stirrups crossing a crack are collected into a vertical tension member. A conservative design requires 45° angle of inclination based on the traditional truss model. From the free body diagram of Figure 2-5 and 2.6, the shear force (V) can be obtained by:

$V= A_v f_{sy} j_d /s$ (2-1)

where, A_v is the area of vertical reinforcement, f_{sy} is the yield strength of the stirrups, j_d is an internal lever arm and s is the spacing of stirrup legs. The traditional truss model provides a simple design approach as shown in Figures 2.5 and 2.6. However, it has some drawbacks due to not considering the shear transfer by the aggregate interlock along the crack surface, the dowel action of longitudinal reinforcement and the uncracked compression zone (Hong and Ha, 2001).

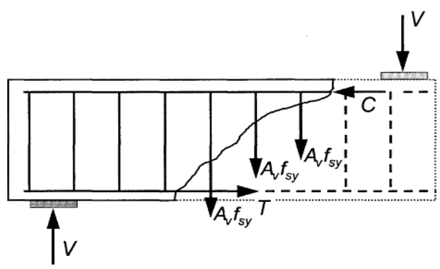


Figure 2.5 Internal forces at an inclined crack (Hong and Ha, 2001)

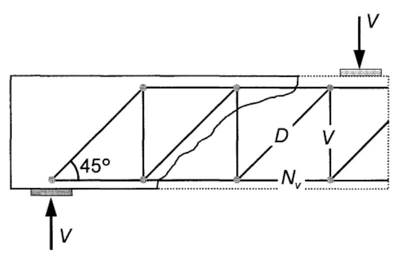


Figure 2.6 Equivalent truss of a beam (Hong and Ha, 2001)

2.4 DESIGN CODE REVIEWES

Various design Code/Standard provisions for shear strength evaluation are applied around the world. The main component of some of these Codes/Standards for shear strength evaluation is based on empirical formulas such as the ACI 318-11 (2011), while others such as the AASHTO LRFD(2010) rely more on concrete models like Modified Compression Field Theory (MCFT) (Bentz et al., 2006).

2.4.1 American Concrete Institute, ACI 318-01 (2011)

An average shear stress distribution across the entire cross section is considered to calculate the shear strength of the beam section. As explained previously, 45 degree truss model is the basis of the formulation. ACI code presents the following basic equations for normal-weight, non-prestressed reinforced concrete, all formulas written based on imperial units:

$$V_u \leq \phi \ V_n = \phi \ (V_c + V_s) \qquad (2\text{-}2)$$

$$V_c = \left(1.9\sqrt{f_c'} + 2500 \ \rho_w \ \frac{V_u}{M_u} \ d\right) b_w d \leq 3.5 \sqrt{f_c'} b_w d \qquad (2\text{-}3)$$

Simplified version:

$$V\text{c} = 2 \sqrt{f_c'} \ b_w \ d \qquad (2\text{-}4)$$

$$A_{v,min} = \left(0.75\sqrt{f_c'} \frac{b_w}{f_{yt}} s\right) \geq 50 \ \frac{b_w s}{f_{yt}} \qquad (2\text{-}5)$$

$$V_s = \frac{A_v f_{yt} d}{s} \qquad (2\text{-}6)$$

where, V_u is the factored shear force on the section, ϕ is the shear strength reduction factor equal to 0.75, V_n is the nominal shear strength, V_c is the nominal shear strength provided by concrete V_s is nominal shear strength provided by shear reinforcement, ρ_w is the ratio of longitudinal reinforcement equals A_s to $b_w.d$, A_s is the area of longitudinal reinforcement, b_w is the width of the web, d is the distance from the extreme compression fiber to the center of gravity of the steel, M_u is the factored moment at the section, $f'c$ is the concrete compressive strength (psi),f_{yt} is the yield strength of the transverse reinforcement (psi), s is the spacing of the transverse reinforcement, and A_v is the area of stirrups and A_v is the area of shear reinforcement.

2.4.2 Canadian Standards Association, CSA A23.3-04 (2004)

The Canadian Standards Association method is based on Modified Compression Field Theory, MCFT and the shear strength of beams is presented in two simplified and general methods:

$$V_r \geq V_f \qquad (2\text{-}7)$$

$$V_r = V_c + V_s \qquad (2\text{-}8)$$

$$V_{r,max} = 0.25\, \phi_c\, f'_c\, b_w\, d_v \qquad (2\text{-}9)$$

$$V_c = \beta \sqrt{f'_c}\, b_w d_v \qquad (2\text{-}10)$$

$$A_{v_{min}} = 0.06 \sqrt{f'_c}\, \frac{b_w\, s}{f_y} \qquad (2\text{-}11)$$

2.4.2.1 Simplified method

a) If the section contains at least the minimum transverse reinforcement,

$A_v < 0.06\sqrt{f'_c}\,\frac{b_w\, s}{f_y}$, β shall be taken as 0.18

b) If the section contains no transverse reinforcement and the specified nominal maximum size of coarse aggregate is not less than 20 mm, β shall be taken as:

$A_v \geq 0.06\sqrt{f'_c}\,\frac{b_w\, s}{f_y}$ and, $ag \geq 20mm$,

$$\beta = \frac{230}{1000 + d_v} \qquad (2\text{-}12)$$

c) If sections containing no transverse reinforcement for all aggregate sizes, replacing the parameter d_v in equation (2-12) by the equivalent crack spacing parameter, s_{ze}, where

$$s_{ze} = \frac{35 s_z}{15 + a_g} \qquad (2\text{-}13)$$

2.4.2.2 General method

The value of β shall be determined from the following equation:

$$\beta = \frac{0.4}{1 + 1500\varepsilon_x} * \frac{1300}{1000 + s_{ze}} \qquad (2\text{-}14)$$

$$\varepsilon_x = \frac{M_{f/d_v} + V_f}{2 E_S\, A_s} \qquad (2\text{-}15)$$

$$V_s = \phi s\, \frac{A_v f_y d_v\, cot\theta}{s} \qquad (2\text{-}16)$$

$$\theta = 29 + 7000\, \varepsilon_x \qquad (2\text{-}17)$$

where, V_f is factored shear stress, V_r is the factored shear resistance, ϕ_c is the resistance factor for concrete equals 0.65, ϕ_s is resistance factor for non-prestressed reinforcing bars, V_c is the nominal shear strength provided by concrete, V_s is nominal shear strength provided by shear reinforcement, A_s is the area of longitudinal reinforcement, b_v is the width of the web, d is the distance from the extreme compression fiber to the center of gravity of the steel, d_v is the effective shear depth, taken as the greater of 0.9d or 0.72h, M_f is the factored moment at the section, f'_c is the specified compressive strength of concrete, f_y is the specified yield strength of non-prestressed reinforcement or anchor steel, s is the spacing of the transverse reinforcement, A_v is the area of shear reinforcement, β is the factor accounting for shear resistance of cracked concrete, θ is the angle of inclination of diagonal compressive stresses to the longitudinal axis of the member, s_z is the equivalent crack spacing parameter, a_g is the maximum aggregate size.

The term a_g should be taken as zero if f'_c exceeds 70 MPa. The crack spacing parameter s_z can be taken as d_v or as the maximum distance between layers of distributed longitudinal reinforcement, whichever is less.

2.4.3 Australian Standards: AS3600 (2009)

There are two alternative methods for the shear design of beams in AS3600. The first one is based on truss analogy and the second one is based on the equations provided in the standard and utilizes the Sample Average Approximate (SAA) method. The truss analogy method takes no account of the shear resisted by the concrete compressive zone, and therefore is over-conservative in this respect (Warner et al, 1998). The ultimate shear strength V_u is given as:

$$V_u = V_{uc} + V_{us} \tag{2-18}$$

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_o \left[\frac{A_{st} f'_c}{b_v d_o} \right]^{1/3} \tag{2-19}$$

$$\beta_1 = 1.1\left[1.6 - \frac{d_o}{1000}\right] \geq 1.1 \quad (2\text{-}20)$$

$\beta_2 = 1$

$\beta_3 = 1$

The shear force is limited to a maximum of:

$$V_{u.\max} = 0.2\, f'_c\, b\, d_o \quad (2\text{-}21)$$

The ultimate shear strength of a beam provided with minimum shear reinforcement shall be taken as:

$$V_{u.\min} = V_{uc} + 0.6 b\, d_o \quad (2\text{-}22)$$

The minimum area of shear reinforcement ($A_{sv.\min}$) provided in a beam shall be given by

$$A_{sv.\min} = 0.35 b_s / f_{sy} \quad (2\text{-}23)$$

The required shear reinforcement per unit spacing, A_v / s, is calculated as follows:

If $V^* \leq \varphi V_{uc}/2$, $A_v/s=0$, except for $d \geq 750$ mm, $A_{sv.\min}$ shall be provided:

If $V_{u.\min} < V^* \leq V_u$:

$$\frac{A_{sv}}{s} = \frac{V^* - \varphi V_{uc}}{f_{sy} d_0 cot\theta_v} \quad (2\text{-}24)$$

$A_{sv.\min} \geq \max\,(0.35\,\mathrm{b}/\mathrm{f}_{sy})$

θ_v = the angle between the axis of the concrete compression strut and the longitudinal axis of the member, which varies linearly from 30 degrees when $V^* = \varphi V_{u.\min}$ to 45 degrees when $V^* = \varphi V_{u,\max}$ = 35.52 degrees. If $V^* > \varphi V_{\max}$, a failure condition is declared.

where V^* is the factored shear force at a section, V_{uc} is the shear force resisted by concrete, V_{us} is the shear force resisted by steel, d_0 is the depth of the outermost layer of tensile reinforcement, A_{st} is the cross-sectional area of longitudinal reinforcement provided in the tension zone and fully anchored at the cross-section under consideration, φ is the strength reduction factor equals 0.8.

2.4.4 Eurocode 2 (2004)

- **Members not requiring design shear reinforcement**

The design value for the shear resistance $V_{Rd,c}$ is given by:

$$V_{Rd,c} = [C_{Rd,c} k (100 \rho_l f_{ck})^{\frac{1}{3}}] b_w \mathrm{d} \qquad (2\text{-}25)$$

$$k = 1 + \frac{\sqrt{200}}{\mathrm{d}} \qquad (2\text{-}26)$$

$$\rho_l = A_{sl} / b_w d \leq 0.02 \qquad (2\text{-}27)$$

The values of $C_{Rd,c}$,v_{min} for use in a Country may be found in its National Annex.

The recommended values for

$$C_{Rd,c} = 0.18 / \gamma_c \qquad (2\text{-}28)$$

$$V_{min} = 0.035\, k^{2/3} f_{ck}^{\,1/2} \qquad (2\text{-}29)$$

where, A_{sl} is the area of the tensile reinforcement, b_w is the smallest width of the cross-section in the tensile area, f_{ck} is the Characteristic compressive cylinder strength of concrete at 28 days.

The strength classes (C) in this code are denoted by the characteristic cylinder strength f_{ck} determined at 28 days with a minimum value of and a maximum value of C_{min} and the maximum value of C_{max}.The values of C_{min} and C_{max} for use in a Country may be found in its National Annex. The recommended values are C30/37 and C70/85, respectively.

$f_{ck} = f'_c - 1.6$ (MPa) $\approx 0.95 f'_c$, γ_c is the Partial factor for concrete. The recommended values for "persistent & transient" and "accidental" design situation are given in Table 2.1.

Table 2.1 Partial factors for materials for ultimate limit states (Eurocode 2, 2004)

Design situations	γ_C for concrete	γ_S for reinforcing steel	γ_S for prestressing steel
Persistent & Transient	1,5	1,15	1,15
Accidental	1,2	1,0	1,0

- **Members requiring design shear reinforcement**

For members with vertical shear reinforcement (Figure 2.7), the shear resistance, V_{Rd} is the smaller value of:

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot\theta \quad (2\text{-}30)$$

$$V_{Rd,max} = \alpha_{cw} b_w z \nu_1 f_{cd}/(\cot\theta + \tan\theta) \quad (2\text{-}31)$$

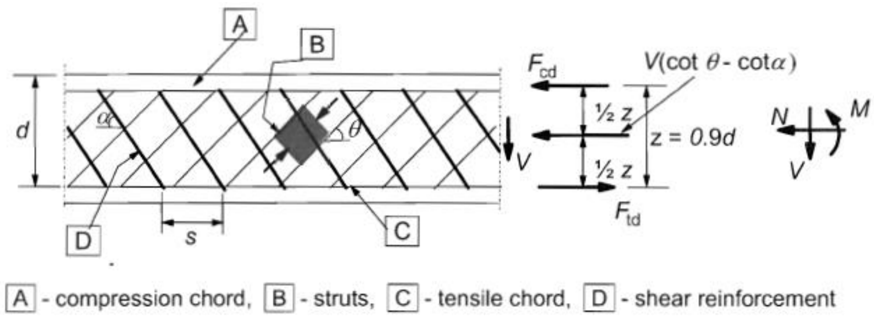


Figure 2.7 Truss model and notation for shear reinforced members

where, z is the inner lever arm, equals 0.9d, A_{sw} is the cross-sectional area of the shear reinforcement, s is the spacing of the stirrups, f_{ywd} is the design yield strength of the shear reinforcement, V_1 is a strength reduction factor for concrete cracked in shear, V_1 = 0,6 in case of $f_{ck} \leq 60$ MPa, $V_1 = 0.9 - f_{ck}\, 1200 > 0.5$ for $f_{ck} \geq 60$ MPa, a_{cw} is a coefficient taking account of the state of the stress in the compression chord, θ is the angle between the concrete compression strut and the beam axis. The limiting values of cot θ for use in a Country may be found in its National Annex. The recommended limits are given as: $1 \leq \cot\theta \leq 2.5$.

2.4.5 CEB-FIP MODEL CODE (1990)

The fundamental “unit-length” of a typical part of the web model of a beam resisting shaer and axial action effects is shown in Figure 2.8.The angle θ between the web compression and the chords may be chosen freely in the range from 45 degree (arc cot 1) to 18.4 degree (arc cot 3).

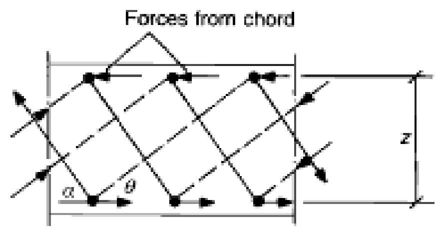


Figure 2.8 Web model (CEB-FIP model code, 1990)

The compression of web concrete, is shown in Figure 2.9 and the tension of web steel is shown in Figure 2.10.

$$F_{Rcw} = f_{cd2} b_w z \cos\theta \tag{2-32}$$

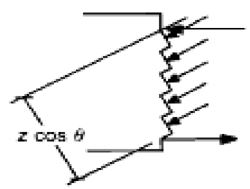


Figure 2.9 Compression of web concrete (CEB-FIP model code, 1990)

The tension of web steel is:

$$F_{Rtw} = \left[\frac{A_{sw} f_{yd}}{s}\right] z\,(\cot\theta + \cot\alpha) \tag{2-33}$$

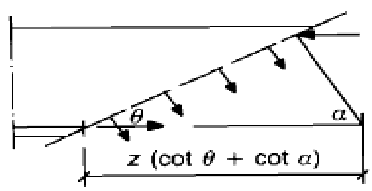


Figure 2.10 Tension of web steel (CEB-FIP model code, 1990)

For the vertical stirrups alone, the minimum amount of θ is obtained by:

$$\cot\theta = \sqrt{\left[\frac{b_w s f_{cd2}}{A_{sw} f_{ywd}} - 1\right]} \leqslant 3 \tag{2-34}$$

For which

$$\frac{V_{Sd}}{zb_w f_{cd2}} = \sqrt{\left[\frac{A_{sw} f_{ywd}}{b_w s f_{cd2}}\right]} \sqrt{\left[1 - \frac{A_{sw} f_{ywd}}{b_w s f_{cd2}}\right]} \leqslant 3 \frac{A_{sw} f_{ywd}}{b_w z f_{cd2}} \qquad (2\text{-}35)$$

In the absence of a more precise calculation the shear force causing shear cracking ,may be estimated as:

$$V_C = 0.15\left(\frac{3d}{a_v}\right)^{1/3} \xi (100 \rho f_{ck})^{1/3} b_{red} d \qquad (2\text{-}36)$$

$$\xi = 1 + \frac{\sqrt{200}}{\mathrm{d}} \qquad (2\text{-}37)$$

where, ρ is the ratio of flexural tensile reinforcement $A_{sw}/b_w z$, a_v is the distance from major load to support, b_{red} is the reduced web breadth, f_{cd2} is the design value of the concrete tensile strength, f_{ywd} is the design value of the web (shear) reinforcement, f_{ck} is the Characteristic, compressive cylinder strength of concrete at 28 days, $A_{s\,w}$ is the area of longitudinal reinforcement, s *is* the space of shear reinforcement and z is the effective depth of the beam.

2.4.6 Spanish Code: EHE-08 (2011)

The general design strut-and-tie method shall be used on all structural members to analyzing bearing capacity of concrete structures with regards to shear strength.

Verification at the limit state of failure due to shear may be carried out based on the effective shear stress V_{rd} form the following expression:

$$V_{rd} = V_d + V_{cd} \qquad (2\text{-}38)$$

where, V_d is the design value of the shear force produced by external actions, V_{cd} is the design value of the component parallel to the section of the resultant normal.

The limit state of failure due to shear will be reached when either the compressive strength of the web or its tensile strength is exhausted. It is necessary to verify that the both the following conditions are simultaneously satisfied:

$$V_{rd} \leq V_{u1}$$

$$V_{rd} \leq V_{u2}$$

Where, V_{rd} is the design value of the effective shear force in Equation (2-38), V_{u1} is the ultimate shear force failure due to diagonal compression in the web and V_{u2} is the ultimate shear force failure due to tension in the web.

2.4.6.1 Obtaining V_{u1}

$$V_{u1} = K\, f_{1cd}\, b_0\, d\, \frac{cotg\ \theta + cotg\ \alpha}{1 + cotg^2\ \theta} \qquad (2\text{-}39)$$

where, f_{cd} is the concrete's compression strength, for $f_{ck} \leq$ 60 N/mm^2 we have $f_{1cd}=0.60\, f_{cd}$, b_0 is the net minimum width of the member, k is the coefficient which depends on the axial force and equals 1in case of non-pre- stressed structures or structures without any axial compression force.

2.4.6.2 Obtaining V_{u2}

2.4.6.2.1 Members without shear reinforcement in regions cracked in flexure

Ultimate shear force failure due to tensile force in the web in conventional and high strength concrete members shall be:

$$V_{u2} = \left[\frac{0{,}18}{\gamma_c}\xi(100\rho_1 f_{cv})^{1/3} + 0{,}15\sigma'_{cd}\right] b_0\, d \qquad (2\text{-}40)$$

With the minimum value of:

$$V_{u2} = \left[\frac{0{,}075}{\gamma_c}\xi^{3/2} f_{cv}^{1/2} + 0{,}15\sigma'_{cd}\right] b_0\, d \qquad (2\text{-}41)$$

where, $\xi = 1 + \frac{\sqrt{200}}{d} \leq 2.0$, $\rho_1 = A_s / b_0 d \leq 0.02$, f_{cv} is the effective shear strength of the concrete in N/mm^2 with the value of $f_{cv}=f_{ck}$, with f_{cv} not more than 15 N/mm^2 in the case of reduced concrete control, with f_{ck} being the concrete's compression strength, which, for the purpose of this paragraph, shall be considered not to exceed 60 N/mm^2 and d is the effective depth of the cross-section.

2.4.6.2.2 Members with shear reinforcement

Ultimate shear force failure due to tensile in the web shall be as the following equation:

$$V_{u2}=V_{cu}+V_{su} \tag{2- 42}$$

$$V_{su} = z \; sen \; \alpha \; (cotg \; \alpha \; + cotg \; \theta \;) \Sigma A_a \, f_{yad} \tag{2- 43}$$

where, A_a is area per unit length of each set of reinforcements forming an angle α with the main axis of the member, f_{yad} is the design strength of reinforcement A_a, θ is the angle between the concrete's compression struts and the axis of the member and shall be satisfy following: $0.5 \leq \cot \theta \leq 2.0$, α is the angle of the reinforcement with the member's axis, z is the mechanic lever arm. In pure bending and in the absence of more accurate calculation, the approximate value of z = 0.9d may be adopted.

$$V_{cu}=\left[\frac{0,15}{\gamma_C} \xi (100 \, \rho_l \, f_{cv})^{1/3} + 0,15 \alpha_l \, \sigma'_{cd} \right] \beta b_0 d \tag{2- 44}$$

$$\beta = \frac{2 \, cotg \; \theta - 1}{2 \, cotg \; \theta_e - 1} \qquad si \; 0,5 \leq cotg \theta < cotg \; \theta_e$$

$$\beta = \frac{cotg \; \theta - 2}{cotg \; \theta_e - 2} \qquad si \; cotg \; \theta_e \leq cotg \; \theta \leq 2,0$$

(2- 45), (2- 46)

$$\Theta_e = 29+7\varepsilon_x \tag{2- 47}$$

$$\varepsilon_x = \frac{\frac{M_d}{z} V_d}{2E_s \, A_s} . 1000 > 0 \tag{2- 48}$$

where, f_{cv} is the effective shear strength of the concrete in N/mm^2, F_{ck} is the compression strength of the concrete in N/mm^2, Θ_e is the reference angle of inclination of cracks, and is calculated by the general method with Equation 2-47 and γ_c is the partial safety factor for concrete obtained by the Table 2.2.

Table 2.2 Partial safety factors for the materials for the Ultimate Limit States (EHE-99, 1999).

Design situation	Concrete γ_c	Steel for passive and active reinforcements γ_s
Persistent or temporary	1.5	1.15
Accidental	1.3	1.0

2.4.7 NZS 3101 (2006)

As per New Zealand Standard, the nominal shear strength for resisted by concrete for normal density concrete, V_c, shall be taken as:

$$V_c = v_c A_{cv} \tag{2- 49}$$

$$V_c = k_d k_a V_b \tag{2- 50}$$

where, A_{cv} is the effective shear area, area used to calculate shear stress, v_c is the shear resisted by concrete, V_b is the smaller of $(0.07+10\,\rho_w)\sqrt{f_c'}$ or $0.2\sqrt{f_c'}$ but need not be taken as less than $0.08\sqrt{f_c'}$, f_c' is the specified compressive strength of concrete and shall not greater than 60 Mpa, k_a is the factor allows for the influence of maximum aggregate size on the shear strength, k_a =1.0, if the maximum aggregate size =20mm, k_a =0.85, if the maximum aggregate size equals 10mm or less, k_d is the influence member depth on shear strength, k_d =1, for members with shear reinforcement equal to or greater than the nominal shear, reinforcement as given in Equation 2-51, k_d =1, for members with an effective depth equal to or smaller than 400 mm.

For members with an effective depth of 200 mm or less, the value of v_c shall be taken as the larger of: $0.17\, k_a \sqrt{f_c'}$ or the value given by Equation (2-50) of the Code. For members with an effective depth between 200 mm and 400 mm, the value of v_c shall be found by linear interpolation.

Nominal shear strength provided by shear reinforcement:

$$V_S = \frac{V^*}{\varphi} - V_C \tag{2-51}$$

Where, V^* is the design shear force at section at the ultimate limit state. Shear force due to transverse reinforcement perpendicular to longitudinal axis of the beams can be derived as:

$$V_S = A_v f_{yt} \frac{d}{s} \qquad (2\text{-}52)$$

where, A_v is the area of shear reinforcement within distance s.

The minimum area of shear reinforcement for non-prestressed members shall be computed by:

$$A_v = \frac{1}{16}\sqrt{f_c'}\,\frac{b_w s}{f_{yt}} \qquad (2\text{-}52)$$

where, f_{yt} is the lower characteristic yield strength of transverse reinforcement.

2.4.7 Zsutty (1971)

The shear strength of concrete beam section without web reinforcement (Vc) can be calculated by Zsutty (1971). If the shear span to depth ratio, a/d, is greater than 2.5:

$$V_c = 2.21(f_c' \rho_s d/a)^{1/3} b_w\, d \qquad (2\text{-}53)$$

If the shear span to depth ratio, a/d, is smaller than 2.5:

$$V_c = [2.21(f_c' \rho_s d/a)^{1/3}](2.5\frac{d}{a}) b_w\, d \qquad (2\text{-}54)$$

Where, V_c is the shear strength provided by concrete, ρ_s is ratio of longitudinal reinforcement equals A_s to $b_w .d$, A_s is the area of longitudinal reinforcement, is the width of the web, d is the distance from the extreme compression fiber to the center of gravity of the steel, $f'c$ is the concrete compressive strength, a is the shear span and $\frac{d}{a}$ is shear span to depth ratio.

2.4.8 Cladera and Marı´ (2004)

- Beams without web reinforcement:

$$V_c = \left[0.225\xi(100\rho_l)^{1/2} f_c^{0.2}\right] b_w d \qquad (2\text{-}55)$$

$$\xi = 1 + \sqrt{\frac{200}{s_x}}\rho_l = \frac{A_l}{b_w d} \leq 0.02\left(1 + \frac{f_c}{100}\right)$$

- Beams with web reinforcement:

$$V_f = V_c + V_S \qquad (2\text{-}56)$$

$V_c = \left[0.17\xi(100\rho_s)^{1/2} f_c^{0.2} \tau^{1/3}\right] . b_w d$ (2-57)

$\xi = 1 + \sqrt{\frac{200}{s_x}} \leq 2.75 \rho_s = \frac{A_l}{b_w d} \leq 0.04$

$\tau = \frac{V_d}{b_w d_v} \leq$ 3MPa

$V_s = d_v \frac{A_w}{s} f_{ywd} cot\theta$ (2-58)

$\theta = 20 + 15\varepsilon_x + 45\frac{\tau}{f_c} \leq 45$

$\varepsilon_x = 0.5 \frac{M_d/d_v + V_d}{E_s A_l} . 1000 \leq 1$

Where, V_c is the shear strength provided by concrete,V_s is the shear strength provided by shear reinforcement, V_f is the failure tension shear strength for members with web reinforcemen, s_x is whichever is smaller: d_v or the vertical distance between longitudinal distributed reinforcement, d_v is the mechanical depth which can be taken as 0.9. d, d is the effective depth in mm, V_d is the designing (factored) shear strength, bw is the smallest width of the crosssection area in mm, A_w is the cross-sectional area of the shear reinforcement, A_l is the cross-sectional area of the shear longitudinal reinforcement, s is the spacing of the stirrups, M_d is the design moment of the section, E_s is the modulus of elasticity of the steel, f_c is the concrete compressive strength, f_{ywd} is the design yielding strength of the shear reinforcement, and θ is the angle of the compression struts, derived as above formula,ε_x is the longitudinal strain in the web expressed in 1/1000, calculated by the above formula.

2.4.9 Gastebled and May (2001)

The shear strength of concrete beam section **without web reinforcement** (V_C)is calculated by Gastebled and May (2001) :

$V_C = 0.15 \frac{37.41}{\sqrt{d}} \left(\frac{3d}{a}\right)^{\frac{1}{3}} (100\rho_s)^{\frac{1}{6}} (1 - \sqrt{\rho_s})^{2/3} (f_c')^{0.35} b_w d$ (2-59)

Where, V_c is the shear strength provided by concrete, ρ_s is ratio of longitudinal reinforcement equals A_s to $b_w . d$, A_s is the area of longitudinal reinforcement, b_w is the width of the web, d is the distance from the extreme compression fiber to the center of gravity of the steel, $f'c$ is the concrete compressive strength, a is the shear span and $\frac{d}{a}$ is shear span to depth ratio.

CHAPTER THREE

3. AN ANALYTICAL STUDY TO EVALUATE THE PERFORMANCE OF CODES/EXISTING PROVISIONS FOR SHEAR STRENGTH PREDICTION OF RCA BEAMS

3.1 GENERAL

Experimental results reveal that mechanical properties and structural behavior of recycled aggregate concrete are varied from conventional concrete, and the variation depends on factors such as the percentage of recycled aggregate replaced with natural aggregate, beam size or shear span to depth ratio (a/d). As a consequence, the performance of existing guidelines and code provisions for evaluation of shear strength of recycled aggregate concrete (RAC) beams is questionable. It becomes difficult for engineers to entirely rely on existing methods and code provisions. For safety and serviceability purposes, specific code provisions needs to be re-examined or modified. This chapter presents a number of experimental studies done by previous researchers. The test results for shear strength of tested RAC beams are used to study the performance of various Codes/existing equations presented in Chapter Two. Then a comparison of test and predicted shear strengths of RAC beams is carried out to study the accuracy/performance of available Codes/existing design guidelines. The test beams were categorized in two groups: beams with and without transverse/shear reinforcements. The experimental studies are described by presenting the effect of following principal parameters on the shear strength of the tested beams: concrete type made with RAC with different proportions of replacement or conventional concrete (CC), the amount of longitudinal reinforcement, the amount of transvers reinforcement if existed, compressive strength of the concrete, beam size and shear span to depth ratio.

3.2 REINFORCED CONCRETE BEAMS WITHOUT WEB REINFORCEMENT

3.2.1 Arezoumandi et al. (2014)

An experimental investigation on the shear strength of reinforced concrete beams composed of recycled concrete aggregate was conducted. Twelve full-scale beams were constructed for tests, in which six of them were composed of 100% recycled aggregate and others with normal aggregate. The test parameters for this study were longitudinal reinforcement ratio and concrete type. The research objectives were:

- To compare the experimental shear strengths of the beams with the shear provisions of both U.S. and international design codes (U.S., Australia, Canada, Europe, and Japan) as well as a shear database of CC specimens.
- To evaluate the shear strengths of RAC beams based on different fracture mechanics approaches and the modified compression field theory (MCFT) method.
- To perform statistical data analyses (both parametric and non-parametric) to evaluate whether or not there was any statistically significant difference between the shear strength of the recycled aggregate concrete (RAC) and CC beams.

For each concrete type six beams without stirrups were built with three different longitudinal reinforcement ratios (1.27%, 2.03%, and 2.71%). The beams were designed to prevent flexural failure and satisfy the minimum and maximum longitudinal reinforcement requirements of ACI 318-11. The beams cross section are rectangular with a width of 300 mm, a height of 460 mm, a length of 4300 mm, and shear span-to-depth ratios of 3.0 or greater. The details of the beams are given in Figure 3.1 and Table 3.1.

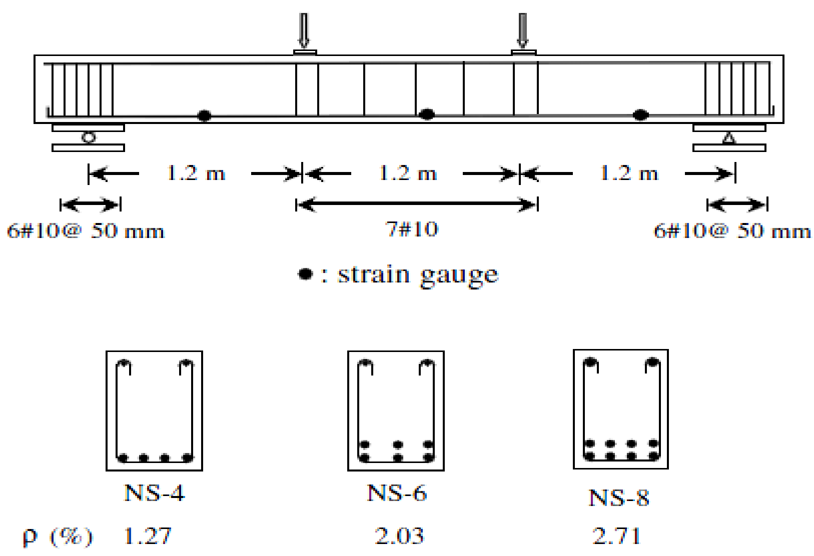


Figure 3.1 Cross sections and reinforcement layout of the test beams (Arezoumandi et al., 2014)

Table 3.1 Details of Reinforced Concrete Beams tested by Mahdi Arezoumandi et al. (2014)

Beam Id	**Section**	**f'_c**	**f_y**	**E_s**	**b_w**	**a/d**	**d**	**a_g**	**ρ_s**	**A_s**	**V_c**
		MPa	MPa	Mpa	mm		mm	mm		mm^2	kN
NS-4	CC1	37.3	449	193140	300	3	400	25	1.27%	1520	121.2
NS-4	CC2	34.2	449	193140	300	3	400	25	1.27%	1520	129.9
NS-6	CC1	37.3	449	193140	300	3	400	25	2.03%	2280	143.2
NS-6	CC2	34.2	449	193140	300	3	400	25	2.03%	2280	167
NS-8	CC1	37.3	449	193140	300	3	400	25	2.71%	3040	173.5
NS-8	CC2	34.2	449	193140	300	3	400	25	2.71%	3040	170.8
NS-4	RAC1	30	449	193140	300	3	400	25	1.27%	1520	114.8
NS-4	RAC2	34.1	449	193140	300	3	400	25	1.27%	1520	113
NS-6	RAC1	30	449	193140	300	3	400	25	2.03%	2280	143.2
NS-6	RAC2	34.1	449	193140	300	3	400	25	2.03%	2280	124.1
NS-8	RAC1	30	449	193140	300	3	400	25	2.71%	3040	131.4
NS-8	RAC2	34.1	449	193140	300	3	400	25	2.71%	3040	140.3

The experimental shear strengths of the beams are compared with the shear provisions of a number of codes and standards. According to the research it is concluded that:

- The behavior of the RAC and CC beams was almost identical regarding to crack progression and load–deflection response.
- Results of the statistical tests show that the beams with 100% RCA have about 12% lower shear strength compared with the CC beams. This result implies that the existing design standards may not be applicable to RAC without modifications to reflect the lower shear capacity.
- The fracture mechanic approaches underestimate the shear strength for both the CC and RAC beams, but appear to be equally applicable to both materials.
- The MCFT method conservatively predicts shear strength for the CC and RAC beams, although in general offers very good agreement with experimental results.
- Statistical data analysis (both parametric and nonparametric) showed that the CC beams had higher shear strength compared with the RAC beams.

3.2.2 Fathifazl et al. (2011)

The shear performance of reinforced concrete (RC) beams made with coarse recycled concrete aggregate (RCA) are examined based on a new method of mixture proportioning named Equivalent Mortar Volume (EMV) method. In this method, the relative amount and properties of adhered mortar of recycled concrete aggregates and the coarse aggregate as two different phases are considered separately. Several beams were designed based on this new mix design method and tested to study the effect of a various parameters to shear behavior of RCA concrete beams without stirrups. The test parameters for this study are the shear span-to depth ratio, beam size and the concrete type. The research objectives were:

- To determine the effect of parameters like shear span to depth ratio and beam size on shear behavior of reinforced concrete beams made by RCA by new mix design method, (EMV).
- To determine the characteristics like load–deflection curve, shear deformations, diagonal cracking load, crack pattern, ultimate shear strength and failure mode for each specimen.

To investigate the effect of shear span-to-depth ratio, a/d values of 1.5, 2, 2.7, and 4 were selected to cover short, intermediate, and long beams. Four beams were tested for each RCA source, i.e. one beam for each a/d ratio. All the beams were rectangular, 200 mm wide and 350–375 mm deep, with effective depth of 305 ± 4 mm. To investigate the size effect, 200 mm wide beams with height of 250, 375, 450, or 550 mm with a constant shear span to depth ratio of 2.7, were tested. Four beams were tested for each RCA source, i.e. one beam for each depth size. In addition, for each RCA source, one companion control beam was made of concrete with 100% coarse natural aggregates of the same type as the original natural aggregate in the corresponding RCA. The latter beams had a nominal a/d ratio of 2.70 and medium height of 375 and medium height of 375 mm. The details of the beams are given in Fig. 3.2 and Table 3.2.

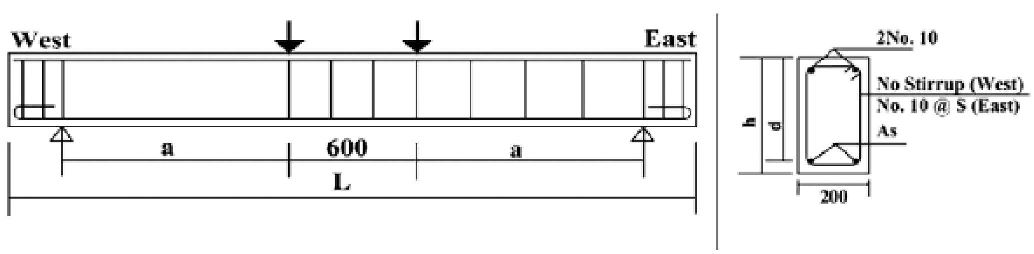


Figure 3.2 Cross sections and reinforcement layout of the test beams (Fathifazl et al., 2009)

The beams were made using concrete mixes involving RCA from two different sources. The RCA from the two sources are termed RCA-M and RCA-V, obtained from aggregate recycling plants in Montreal and Vancouver, Canada, respectively**.** The experimental shear strengths of the beams are compared with the shear provisions of a number of codes and standards.

Table 3.2 Details of Reinforced Concrete Beams tested by Fathifazl et al. (2011)

Beam ID	**f'_c Mpa**	**f_y Mpa**	**E_s Mpa**	**b_w mm**	**a/d**	**d mm**	**a_g mm**	**ρ_s**	**A_s mm2**	**V_c kN**
EM-1.5	41.6	400	200000	200	1.5	300	19	1%	600	186.7
EM-2	41.4	400	200000	200	2	300	19	1.50%	900	169.5
EM-2.7	41.6	400	200000	200	2.59	309	19	1.62%	1001.16	103.9
CL-2.7	37.95	400	200000	200	2.59	309	19	1.62%	1001.16	92.8
EM-4	41.6	400	200000	200	3.93	305	19	2.46%	1500.6	83.2
EV-1.5	49.1	400	200000	200	1.5	300	19	1%	600	195.3
EV-2	49.1	400	200000	200	2	300	19	1.50%	900	179
CG-2.7	34.1	400	200000	200	2.59	309	19	1.62%	1001.16	150
EV-4	49.1	400	200000	200	3.93	305	19	2.46%	1500.6	105.6
EM-L	41.6	400	200000	200	2.69	201	19	1.99%	799.98	89.3
EM-M	41.6	400	200000	200	2.59	309	19	1.62%	1001.16	103.9
CL-M	37.95	400	200000	200	2.59	309	19	1.62%	1001.16	92.8
EM-H	41.6	400	200000	200	2.73	381	19	1.83%	1394.46	99.5
EM-VH	41.6	400	200000	200	2.73	476	19	1.68%	1599.36	104.6
EV-L	49.1	400	200000	200	2.69	201	19	1.99%	799.98	122.6
CG-M	34.1	400	200000	200	2.59	309	19	1.62%	1001.16	150
EV-H	41.6	400	200000	200	2.73	381	19	1.83%	1394.46	111.7
EV-VH	49.1	400	200000	200	2.73	476	19	1.68%	1599.36	88.83

The following conclusions were drawn from the research:

- Applying the EMV method of mix proportioning for RCA-concrete yields concrete mixes with comparable strength as conventional concrete mixes with equivalent amount of mortar and other ingredients.
- For RCA-concrete beams with shear span/depth ratio ranging between 1.5 and 4, regardless of the RCA source, all of the above codes gave conservative estimates of the actual strength of these beams.
- Some other theoretical/empirical expressions proposed by some investigators for determining the concrete's contribution to the shear resistance of beams without stirrups overestimated the strength of some of the current beams.
- For beams up to a total depth of 550 mm, the forgoing codes expressions conservatively estimated the beams' shear strength. Thus within the range of depths tested in the current investigation, despite the size effect, the codes predictions are on the conservative side.

- The shear strength of reinforced RCA-concrete beams had a tendency to increase with decreasing a/d mainly due to the contribution of an arch action mechanism at lower a/d, which conforms to the known behavior of conventional reinforced concrete beams.

3.2.3 Knaack and Kurama (2014)

Knaack and Kurama (2014) investigated the flexural and shear behavior of reinforced concrete beams made by recycled concrete aggregates (RCA) as replacement for coarse natural aggregates (e.g., crushed stone, gravel). The experimental results from 12 twin pairs of normal strength concrete beam specimens are presented and compared with predictions from existing code methods and analytical models for conventional concrete. The test parameters for this study is the concrete type. The research objectives were:

- To provide an experimental and analytical investigation on the behavior of reinforced concrete beams that use recycled concrete aggregates (RCA) as replacement for coarse natural aggregates (e.g., crushed stone, gravel).
- To investigate the inherent variability in the results, specifically focusing on locally available recycled materials with minimal processing and construction methods that are consistent with current U.S. practice.

To investigate the effect of shear span-to-depth ratio, a/d values of 1.5, 2, 2.7, and 4 were selected to cover short, intermediate, and long beams. Four beams were tested for each RCA source, i.e. one beam for each a/d ratio. All the beams were rectangular, 200 mm wide and 350–375 mm deep, with effective depth of 305 ± 4 mm. To investigate the size effect, 200 mm wide beams with height of 250, 375, 450, or 550 mm with a constant shear span to depth ratio of 2.7, were tested. Four beams were tested for each RCA source, i.e. one beam for each depth size. In addition, for each RCA source, one companion control beam was made of concrete with 100% coarse natural aggregates of the same type as the original natural aggregate in the corresponding RCA. The latter beams had a nominal a/d ratio of 2.70 and medium height of 375 and medium height of 375 mm. The details of the beams are given in Figure 3.3 and Table 3.3. The experimental shear strengths of the beams are compared with the shear provisions of ACI-318.

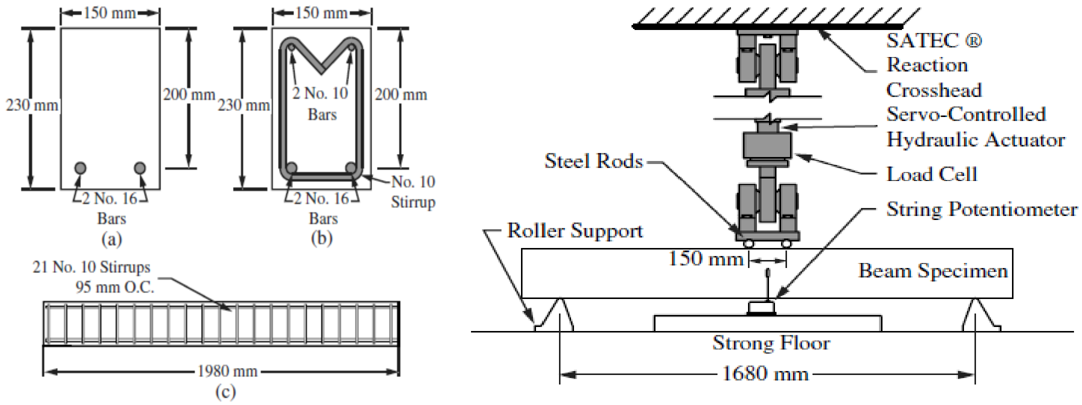


Figure 3.3 Cross sections and reinforcement layout of the test beams (Knaack and Kurama, 2014)

It is concluded that:

- RCA does not cause an observable change in the progression of nonlinear behavior and failure in flexure-critical and shear critical reinforced concrete beams.
- The predicted results are reasonably close to the measured trends, indicating that existing analytical models and code based procedures for conventional reinforced concrete beams can also be applied to RCA concrete beams.

While further research is certainly needed, a preliminary expectation from this paper is that locally available RCA with prequalified absorption and deleterious material content may be suitable for use in reinforced concrete beams constructed in a manner that is consistent with current U.S. practice for conventional concrete applications.

Table 3.3 Details of Reinforced Concrete Beams tested by Knaack and Kurama (2014)

Beam ID	f'_c	f_y	E_s	b_w	a/d	d	a_g	ρ_s	A_s	V_c
	MPa	MPa	MPa	mm		mm	mm	%	mm^2	kN
S0-1a	32.6	572	198000	150	3.825	200	19	1.34	402.1	31.1
S0-1b	32.6	572	198000	150	3.825	200	19	1.34	402.1	36.9
S0-2a	50.3	572	198000	150	3.825	200	19	1.34	402.1	40.4
S0-2b	50.3	572	198000	150	3.825	200	19	1.34	402.1	42.3
S50-1a	43.6	572	198000	150	3.825	200	19	1.34	402.1	44
S50-1b	43.6	572	198000	150	3.825	200	19	1.34	402.1	39.1
S50-2a	40.2	572	198000	150	3.825	200	19	1.34	402.1	43.7
S50-2b	40.2	572	198000	150	3.825	200	19	1.34	402.1	41.2
S100-1a	41.4	572	198000	150	3.825	200	19	1.34	402.1	36.4
S100-1b	41.4	572	198000	150	3.825	200	19	1.34	402.1	38
S100-2a	35.7	572	198000	150	3.825	200	19	1.34	402.1	39.9
S100-2b	35.7	572	198000	150	3.825	200	19	1.34	402.1	36.1

3.3 REINFORCED CONCRETE BEAMS WITH WEB REINFORCEMENT

3.3.1 Gonzalez et al. (2007)

The shear behavior of concrete made with recycled concrete aggregates was examined in this study. For the purpose two sets of four test beams constructed with both conventional concrete and concrete comprising 50 % recycled coarse aggregates. The test parameters are amount of transverse reinforcement and concrete type. The research objective was to examine the behavior of the recycled aggregate concrete versus conventional concrete beams in shear failure comprising the maximum deflections, the ultimate loads and crack progression.

All specimens had a rectangular cross section with 350 mm (depth), 200 mm (width) and were tested with a shear span-to-depth of 3.3 as shown in Figure 3.4 and Table 3.4. The beams were tested maintaining two symmetrical spans with different amounts of transverse reinforcement. Failure occurred only in one of the spans, the one with the smallest amount of transverse reinforcement, denominated span S6 (diameter of the vertical stirrups = 6 mm). In the other span, S8 (diameter of the vertical stirrups = 8 mm), it was possible to study the behavior prior to failure.

Based on the test results each specimen without shear reinforcement exhibited an initial flexural crack at the center of the specimen and subsequent flexural cracks away from that section. As the applied load was increased, one of the flexural cracks extended into a diagonal crack near one of the supports, or a diagonal crack formed abruptly at the mid-height of the beam within the shear span. After the formation of the diagonal crack, brittle failure occurred. The specimens with shear reinforcement showed the same crack pattern as the specimens without shear reinforcement until the formation of diagonal cracking, but showed higher load-carrying capacity following it.

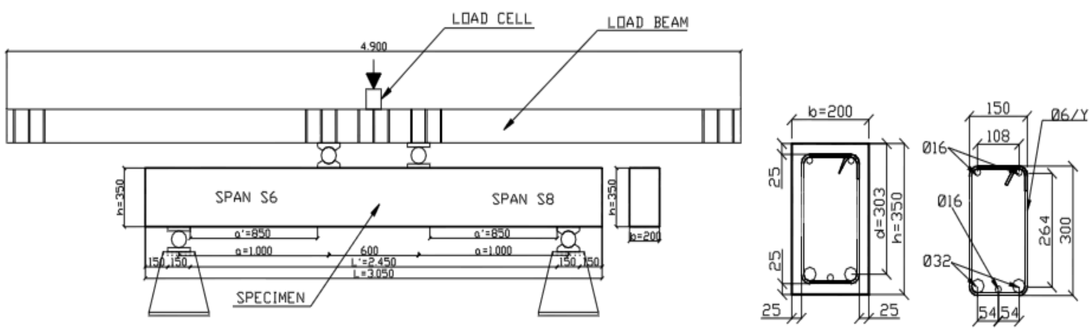


Figure 3.4 Cross sections and reinforcement layout of the test beams (Gonzalez et al., 2007)

Table 3.4 Details of Reinforced Concrete Beams tested by Gonzalez et al. (2007)

Beam ID	f'_c	f_y	E_s	b_w	a/d	d	a_g	ρ_s	A_s	ρ_v	A_v	s	V_c
	MPa	MPa	MPa	mm		mm	mm	%	mm^2	%	mm	mm	kN
V0CC	40.2	500	200000	200	3.3	303	25	2.98	1805.88	0	0	-	88.86
V0RC	39.65	500	200000	200	3.3	303	25	2.98	1805.88	0	0	-	90.64
V24CC	39.16	500	200000	200	3.3	303	25	2.98	1805.88	0.12	57	240	127.98
V24RC	39.23	500	200000	200	3.3	303	25	2.98	1805.88	.012	57	240	164.29
V17CC	39.08	500	200000	200	350	303	25	2.98	1805.88	0.17	57	170	150.83
V17RC	41.49	500	200000	200	3.3	303	25	2.98	1805.88	0.17	57	170	176.99
V13CC	37.66	500	200000	200	3.3	303	25	2.98	1805.88	.022	57	130	190.29
V13RC	40.46	500	200000	200	3.3	303	25	2.98	1805.88	.022	57	130	233.59

The predicted capacities of the beams specimens calculated based on modified compression field theory (MCFT) and using the expressions of different codes: Little differences were observed in the structural behavior of the concrete beams in terms of both deflections and ultimate load. Differences were only evident during the analysis of cracking.

- Premature cracking and notable splitting cracks along the tension reinforcement were observed in recycled concrete beams. Both may be controlled by introducing stricter limits on the minimum stirrups spacing.
- Not all the beams reached shear force at stirrups yield in the spans S8. This shear force was detected in the recycled concrete beams. All the beams, even those with shear reinforcement lower than the minimum demonstrate a notable strength after cracking.

3.3.2 Gonzalez et al. (2009)

The second stage of studies on the behavior of recycled concrete aggregate was conducted by Gonzalez et al. (2009). The results of the first stage (replacement of 50% recycled coarse aggregate with natural aggregate) revealed minor differences between recycled concrete and conventional concrete in ultimate loads and these differences were found to increase when cracking was taken into account (Gonzalez et al., 2007). In this study, the effect of the addition of 8% silica fume to recycle concrete (recycled concrete with silica fume) is examined. The results indicate that the shear behavior of the beams made by new mix improves. Moreover, the use of this material in recycled concrete produced changes in its structural behavior similar to those induced in conventional concrete (conventional concrete with silica fume). The test parameters are amount of transverse reinforcement and concrete type. The research objectives were:

- To determine whether or not the addition of this percentage of silica fume to the recycled concrete would succeed in controlling the premature cracking detected in previous studies, thus avoiding the need to impose stricter limits on minimum stirrup spacing, which would result in reduced costs.
- To determine the possibility of improving the behavior of recycled concrete by the inclusion of a pozzolanic addition, silica fume.

All specimens had a rectangular cross section with 350 mm (depth) 200 mm (width) and were tested with a shear span-to-depth of 3.3 as shown in Figure 3.5 and Table 3.5.The beams were tested maintaining two symmetrical spans with different amounts of transverse reinforcement. Failure occurred only in one of the spans, the one with the smallest amount of transverse reinforcement, denominated span S6 (diameter of the vertical stirrups = 6 mm). In the other span, S8 (diameter of the vertical stirrups = 8 mm), it was possible to study the behavior prior to failure.

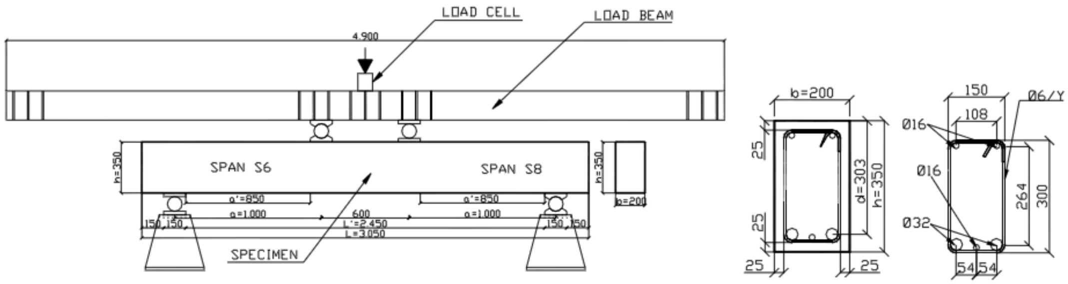


Figure 3.5 Cross sections and reinforcement layout of the test beams (Gonzalez et al.2009)

Table 3.5 Details of Reinforced Concrete Beams tested by Gonzalez et al. (2009)

Beam ID	f'_c	f_y	E_s	b_w	a/d	d	ag	ρ_s	A_s	A_v	s	ρ_v	V_c
	MPa	MPa	MPa	mm		mm	mm	%	mm^2	mm	mm	%	kN
V0CC	46.77	500	200000	200	3.3	303	25	2.98	1805.88	0	-	0	100.53
V0RC	41.45	500	200000	200	3.3	303	25	2.98	1805.88	0	-	0	83.88
V13CC	43.66	500	200000	200	3.3	303	25	2.98	1805.88	57	130	0.12	220.08
V13RC	43.25	500	200000	200	3.3	303	25	2.98	1805.88	57	130	.012	202.36
V17CC	45.16	500	200000	200	3.3	303	25	2.98	1805.88	57	170	0.17	199.79
V17RC	44.49	500	200000	200	3.3	303	25	2.98	1805.88	57	170	0.17	192.92
V24CC	42.75	500	200000	200	3.3	303	25	2.98	1805.88	57	240	.022	150.07
V24RC	41.45	500	200000	200	3.3	303	25	2.98	1805.88	57	240	.022	147.33

The predicted capacities of the beams specimens, computed using the modified compression field theory and using the expressions of different codes are compared. The following conclusions were drawn:

- The notable splitting cracks along the tension reinforcement observed in recycled concrete beams of the first phase were mitigated by the addition of silica fume.
- It was found that the addition of silica fume to recycled concrete causes changes in its behavior similar to the changes that occur when it is added to conventional concrete.
- All the codes studied were conservative and subsequently can be used for the shear design of recycled concrete beams with silica fume.

3.3.3 Ajdukeiwicz and Kliszczewicz (2007)

This was an experimental study to examine the differences in particular properties on the general behavior of reinforced concrete members made of various mixtures of recycled aggregate concrete. Sixteen series of beams and 5 series of columns were tested. Different mixes made by recycled coarse and fine aggregates and by recycled coarse aggregates only were applied for the test beams and columns.

The test specimens constructed with different strength classes of concrete and different types of aggregates. Both cases of replacement with fine and coarse aggregates and coarse aggregates only were applied. The research objective is to determine the differences in behavior of simple structural reinforced concrete members, simply supported beams and axially loaded columns, with different contribution of recycled aggregate (RAC) in concrete mixes in comparison with concretes made by natural coarse aggregates (NAC). Sixteen series of beams were tested. There were three beams in each series prepared with the same reinforcement and from concrete of almost the same composition but mixed with different contribution of natural and recycled aggregate manufactured. The dimension of the beams and the reinforcement layout of beams is illustrated in Figure 3.6 and detailed in Table 3.6.

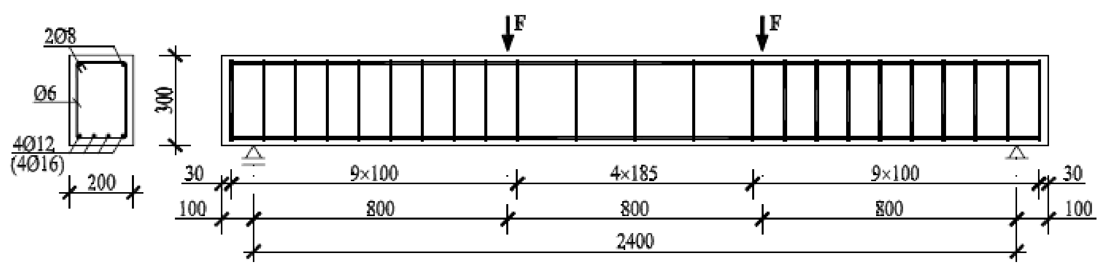


Figure 3.6 Cross sections, reinforcement layout and loading system of the test beams (Ajdukeiwicz and Kliszczewicz 2007)

A general conclusion from tests confirms the full possibility of use of good quality recycled aggregates in structural made of medium or high strength concrete. Differences in behavior of reinforced concrete members made of new, partially recycled and fully recycled aggregate were relatively small within the test series, but quite significant when strains have been analyze.

Table 3.6 Details of reinforced concrete beams tested by Ajdukeiwicz and Kliszczewicz (2007)

Beam Id	**f'_c**	**f_y**	**f_{yw}**	**E_s**	**b_w**	**a/d**	**d**	**a_g**	**ρ_s**	**A_s**	**A_{sw}**	**V_c**
	MPa	MPa	MPa	MPa	mm		mm	mm			mm^2	kN
ORNm-b2	58.3	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	118.5
GNNl-b2	38.7	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	108.5
GRNl-b2	39.3	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	116.5
GRRl-b2	35.8	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	113
GRNm-b2	59.6	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	118.5
GNNh-b2	93.4	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	125
GRNh-b2	89.1	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	121
GRRh-b2	82.2	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	127.5
BNNl-b2	39.6	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	115.5
BRNl-b2	38.8	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	120.5
BRRl-b2	31.4	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	110
BNNh-b2	100.9	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	131
BRNh-b2	107.8	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	130.5
BRRh-b2	100.5	410	210	193140	200	3.2	250	16	1.61	804.3	56.6	128

3.3.4 Etxeberria et al. (2006)

This study investigated the possibility of the use of recycled aggregate concrete as a structural material rather than s being used only as base filler for road construction. Twelve beam specimens were tested with the same compression strength and different percentages of recycled coarse aggregates and transverse reinforcement. The test parameters are amount of transverse reinforcement and percentages of recycled coarse aggregates. The research objective is to study the influence of the percentage of recycled aggregate used on their structural behaviour at service load levels and up to failure. The 12 beams had rectangular cross section of 200 mm width and 350 mm depth, a total length of 3.05 m and were simply-supported with a span length of 2.60 m. The beams were subjected to a symmetric two point load system, with a shear span/depth ratio a/d equal to 3.3, as indicated in Fig. 10. The beam sections with longitudinal and three transverse reinforcement arrangements were considered, as illustrated in Figures 3.7 and 3.8 as well as in Table 3.7.

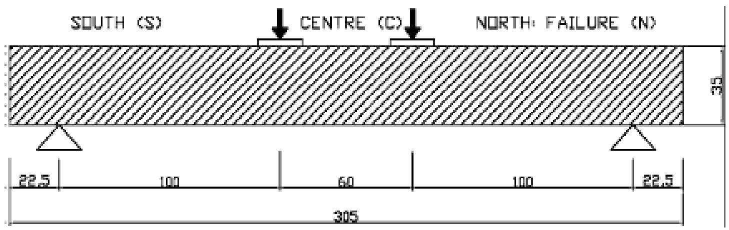


Figure 3.7 Two point loads test set-up (Etxeberria.et al. 2006)

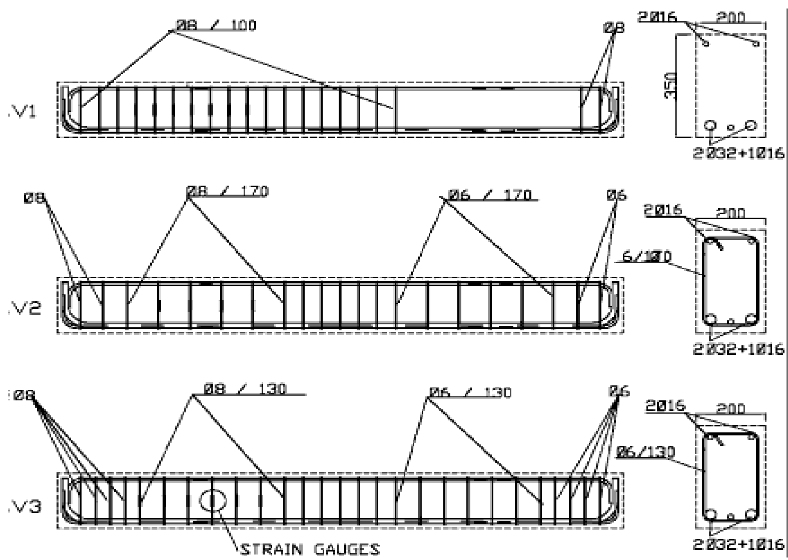


Figure 3.8 Arrangements for the three types of reinforcements (Etxeberria.et al. 2006)

The cracking load for beam specimens with recycled aggregates was lower than that of reference beams (HC), however, the failure load was very similar for all beams of that group. The failure loads are included in Table 3.7.

Table 3.7 Details of Reinforced Concrete Beams tested by Etxeberria.et al. (2006)

Beam ID	**f'_c**	**f_y**	**E_s**	**b_w**	**a/d**	**d**	**ag**	**ρ_s**	**A_s**	**A_v**	**s**	**ρ_v**	**V_c**
	MPa	MPa	MPa	mm		mm	mm	%	mm^2	mm^2	mm	%	kN
HC-1	41.91	500	200000	200	3.3	309	25	2.92	1804.56	0	0	0	100.5
HC-2	41.91	500	200000	200	3.3	309	25	2.92	1804.56	57	130	0.22	213
HC-3	41.91	500	200000	200	3.3	309	25	2.92	1804.56	57	170	0.17	177
HR25-1	42.38	500	200000	200	3.3	309	25	2.92	1804.56	0	0	0	104
HR25-2	42.38	500	200000	200	3.3	309	25	2.92	1804.56	57	130	0.22	186.5
HR25-3	42.38	500	200000	200	3.3	309	25	2.92	1804.56	57	170	0.17	169
HR50-1	41.34	500	200000	200	3.3	309	25	2.92	1804.56	0	0	0	89
HR50-2	41.34	500	200000	200	3.3	309	25	2.92	1804.56	57	130	0.22	220
HR50-3	41.34	500	200000	200	3.3	309	25	2.92	1804.56	57	170	0.17	176
HR100-1	39.75	500	200000	200	3.3	309	25	2.92	1804.56	0	0	0	84
HR100-2	39.75	500	200000	200	3.3	309	25	2.92	1804.56	57	130	0.22	189.5
HR100-3	39.75	500	200000	200	3.3	309	25	2.92	1804.56	57	170	0.17	163

The effect of the use of recycled aggregate on the beams' shear strength depends on the percentage of coarse aggregate substituted, especially for beams without transverse reinforcement. For low percentages of substitution (less than 25%) it can be said that this influence is practically negligible. For beam specimens with web reinforcement, the influence of the amount of RCA in the ultimate shear is very small. Beam specimens with web reinforcement and concrete with 50% and 100% of coarse recycled aggregate achieved approximately the ultimate shear load of conventional concrete.

- In general, Code provisions for shear design overestimate the shear strength of beams made with high amounts (more than 50%) of RCA. However, they offer good correlation for beams made with small amounts (25% or less) of RCA.
- Aggregate interlock and bond between concrete and the reinforcement are mechanisms that look to work correctly in reinforced concrete beams made with RCA, even for high percentages of substitution of coarse aggregate.
- As a general conclusion, it can be said that concrete made with up to 25% of RCA is suitable for structural use, provided that all measures related to dosage, compressive strength and durability aspects have been adopted. In that case, the current code provisions for shear design result adequate for their use to design RC structures with RCA.

3.3.5 Al-Zahraa et al. (2011)

In this research the experimental shear tests on concrete beams with recycled concrete coarse aggregates were conducted. Twelve beams with different percentage of recycled concrete aggregates, shear reinforcement and shear spans were tested under two concentrated loads up to failure. Percentage of recycled concrete aggregates, shear reinforcement and shear spans. The research objectives were:

- To determine modes of failure, deflections, strains and ultimate strength, for the tested beams,
- To compare the experimental results of the present study and other studies. All beams had a rectangular cross section of 100 mm width and 200 mm thickness, a total length of 1,700

mm and were simply supported with an effective span length of 1,500 mm. Table 3.8 gives the reinforcement details of the tested specimens.

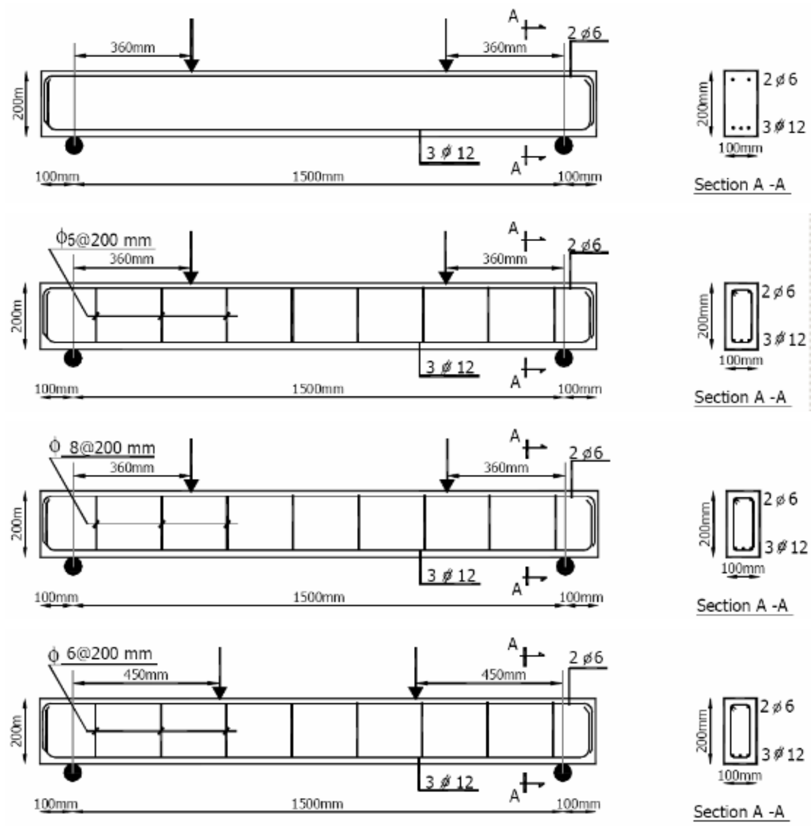


Figure 3.9 (a) Details of group (G1) of tested beam specimens (b) details of group (G2) of tested beam specimens (c) details of group (G3) of tested beam specimens (d) details of group (G4) of tested beam specimens(Al-Zahraa et al., 2011)

Table 3.8 Details of Reinforced Concrete Beams tested by Al-Zahraa et al. (2011)

Beam ID	f'_c	f_y	E_s	b_w	a/d	d	ag	ρ_s	A_s	A_v	s	ρ_v	V_c
	MPa	MPa	MPa	mm		mm	mm	%	mm^2	Mm^2	mm	%	kN
B1	31.67	560	200000	100	2	180	20	1.90	342	0	0	0	31.50
B2	30.42	560	200000	100	2	180	20	1.90	342	0	0	0	43.25
B3	29.58	560	200000	100	2	180	20	1.90	342	0	0	0	35.50
B4	31.67	560	200000	100	2	180	20	1.90	342	60	200	0.3	44.50
B5	36.25	560	200000	100	2	180	20	1.90	342	60	200	0.3	49.75
B6	29.58	560	200000	100	2	180	20	1.90	342	60	200	0.3	4200
B7	37.92	560	200000	100	2	180	20	1.90	342	100	200	0.5	75.00
B8	36.25	560	200000	100	2	180	20	1.90	342	100	200	0.5	6.100
B9	29.58	560	200000	100	2	180	20	1.90	342	100	200	0.5	58.00
B10	37.92	560	200000	100	2.5	180	20	1.90	342	60	200	0.3	37.00
B11	30.42	560	200000	100	2.5	180	20	1.90	342	60	200	0.3	47.25
B12	29.58	560	200000	100	2.5	180	20	1.90	342	60	200	0.3	34.25

The beams were divided into four groups. Each group consists of three beams with three different percentages of RCA (0%, 25%, and 50%). The first group (G1) consisted of three beams B1, B2 and B3. The beams were tested without shear reinforcement and with shear span/depth ratio (a/d = 2), as shown in Figure 3.9(a). The second group (G2) consisted of three beams B4, B5 and B6 reinforced with shear reinforcement of 6 mm-diameter stirrups spaced at 200 mm along the beam length (ρ_v =0.3%).The shear span/depth ratio of the tested beams was (a/d = 2), as illustrated in Figure 3.9(b). The third group (G3) consisted of beams B7, B8 and B9 reinforced with stirrups of 8 mm-diameter bars spaced at 200 mm (ρ_v = 0.5%) as shown in Figure 3.9(c). The shear span/depth ratio of the tested beams was also (a/d = 2). The fourth group (G4) consisted of beams B10, B11 and B12. The shear reinforcement of this group is the same as group (G2) but with shear span/depth ratio equal 2.5, as shown in Figure 3.9(d).

The results showed that for beams that did not have shear reinforcement, using recycled coarse aggregate would increase the cracking load and the ultimate load. This took place despite the compressive concrete strength being identical for all the beams in this group. It might be because of better interlocking of the recycled concrete aggregate developed along the crack. It was concluded that:

- When less than 25% of recycled coarse aggregate was used, the shear strength is almost similar to using conventional concrete. Particularly for low shear reinforcement ratios (less than 0.3%). In addition, when 50% substitution is used, the shear strength decreased by an average of 12%. However, increasing the shear reinforcement ratio could improve the effect of RCA.
- The ACI procedure for shear is validated by comparisons of the predicted failure loads with results of experimental data available in the literature. The results of 38 beams tested in the current study and by other researchers revealed that the ACI Code predictions for shear strength are about 30% more than the experimental values, indicating the possibility of using the same shear design procedure for beams with RCA.

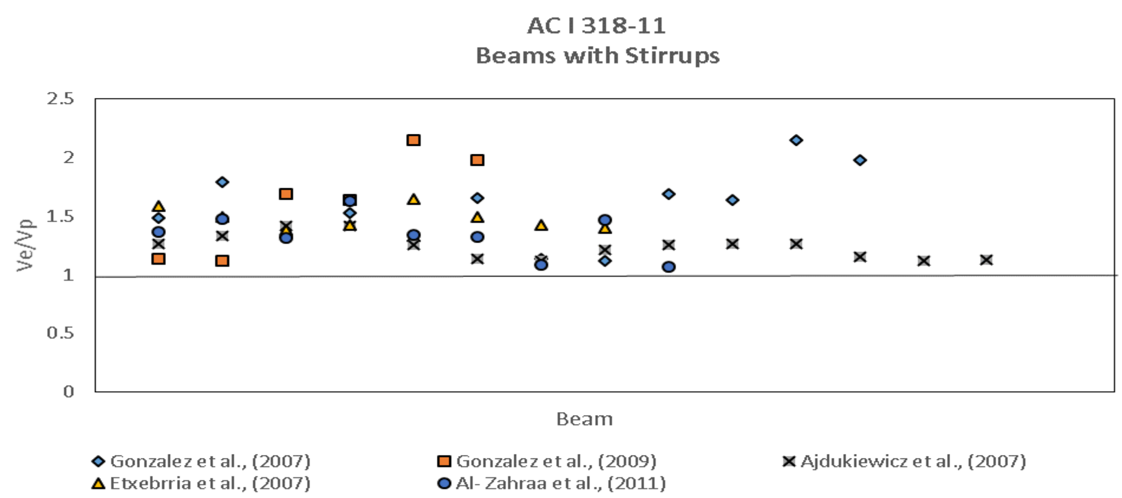
3.4 THERORETICAL ANALYSES

3.4.1 Performance analysis of Codes/existing equation for predicting shear strength of RAC beams

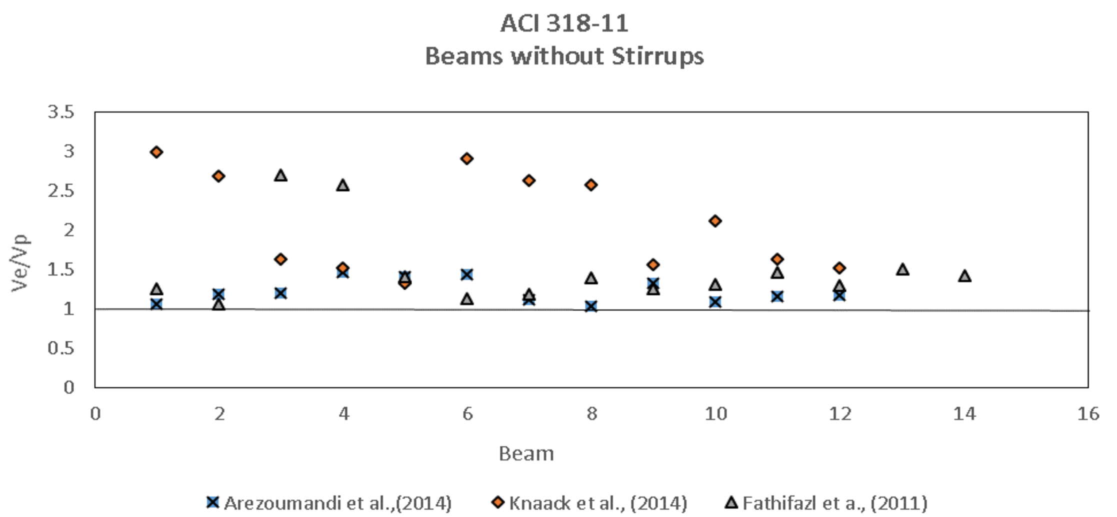
The shear strength of all beam specimens (total 91 beams out of which 48 beams without web reinforcement and 43 with web reinforcement) from experimental research conducted by previous researchers (Fathifzl et al. 2011, Etxeberria et al. 2007, Ajdkiewicz et al. 2007, Knaack et al. 2014, Al-Zahraaet al. 2011, González et al. 2009, González et al. 2007 and Arezoumandi et al. 2014 described in previous section) are calculated based on Codes and existing equations. The performance of Codes/existing equations is studied in terms of the ratio of experimental (V_e) to predicted (V_p) value (V_e/V_p) of shear strength of beams. Various Codes (ACI 318-11, 2011; CSA A23.3-04, 2004; AS3600, 2009, Eurocode 2, 2004; CEB-FIP MODEL CODE, 1990,; Spanish Code: EHE-08, 2011 and NZS 3101, 2006) and other existing equations (proposed by Zsutty 1971, Cladera and Marı 2004 and Gastebled and May 2001) as described in Chapter 2 are used for performance evaluation.

The predicted shear strength values for beams without stirrups and beams with stirrups based on Codes/formulas are presented Tables A-1 to A- 10 and Table A-11 to A-18, respectively in Appendix A.

Figures 3.10 to 3.19 present the V_e/V_p values for beams with and without web reinforcement for performance evaluation of various Code based/other existing equations. The points that fall below the reference line at V_e/V_p equal to .0 indicate that the predicted shear strength by Codes/existing Equations is less than that obtained from test. $V_e/V_p > 1$ means Codes/existing equations are conservative and under-predicting the shear strength of RAC beams.

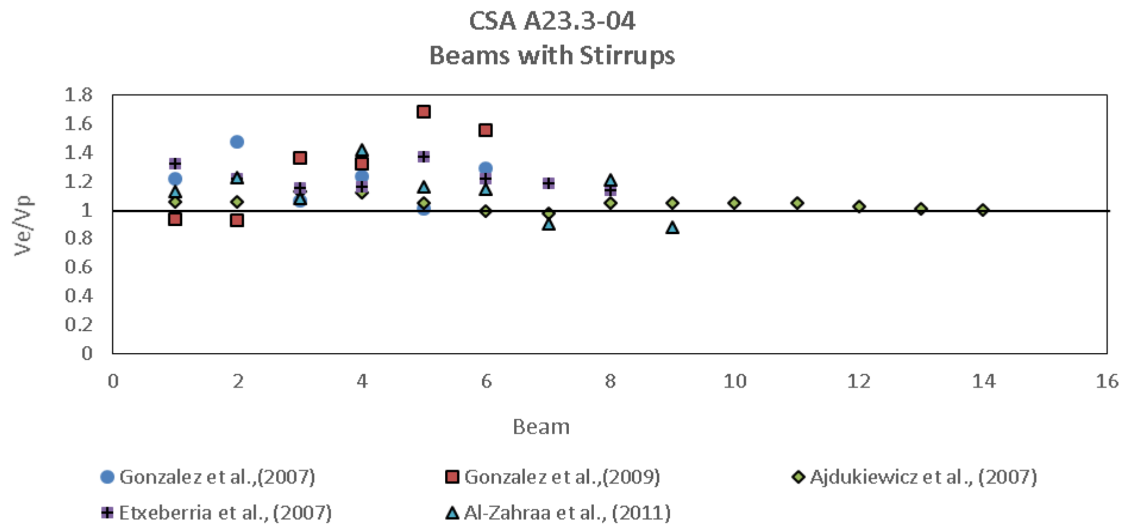


(a)

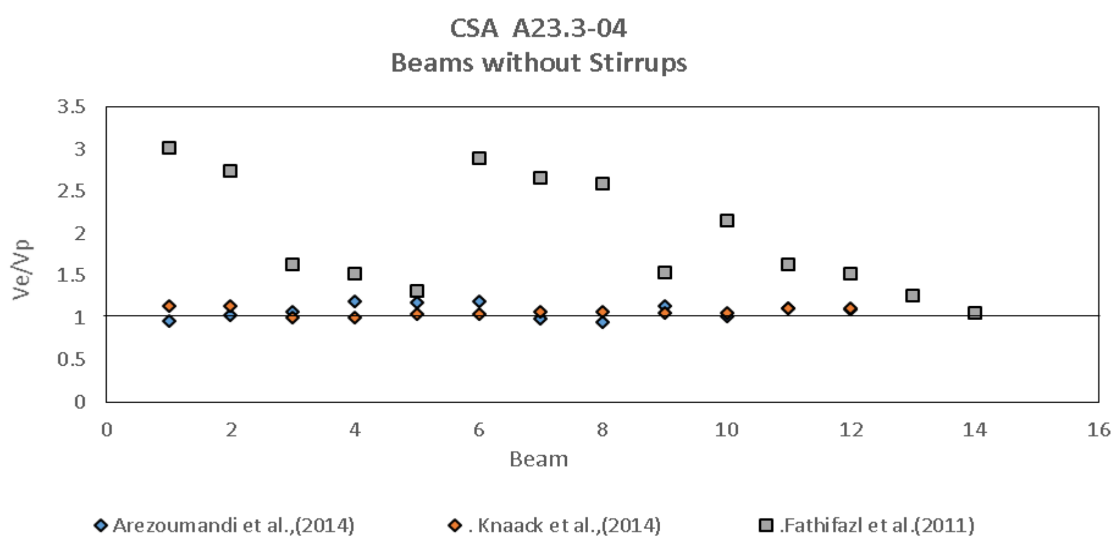


(b)

Figure 3.10 Ratio experiment-to-predicted values using ACI 318-11: (a) with stirrups (b) without stirrups

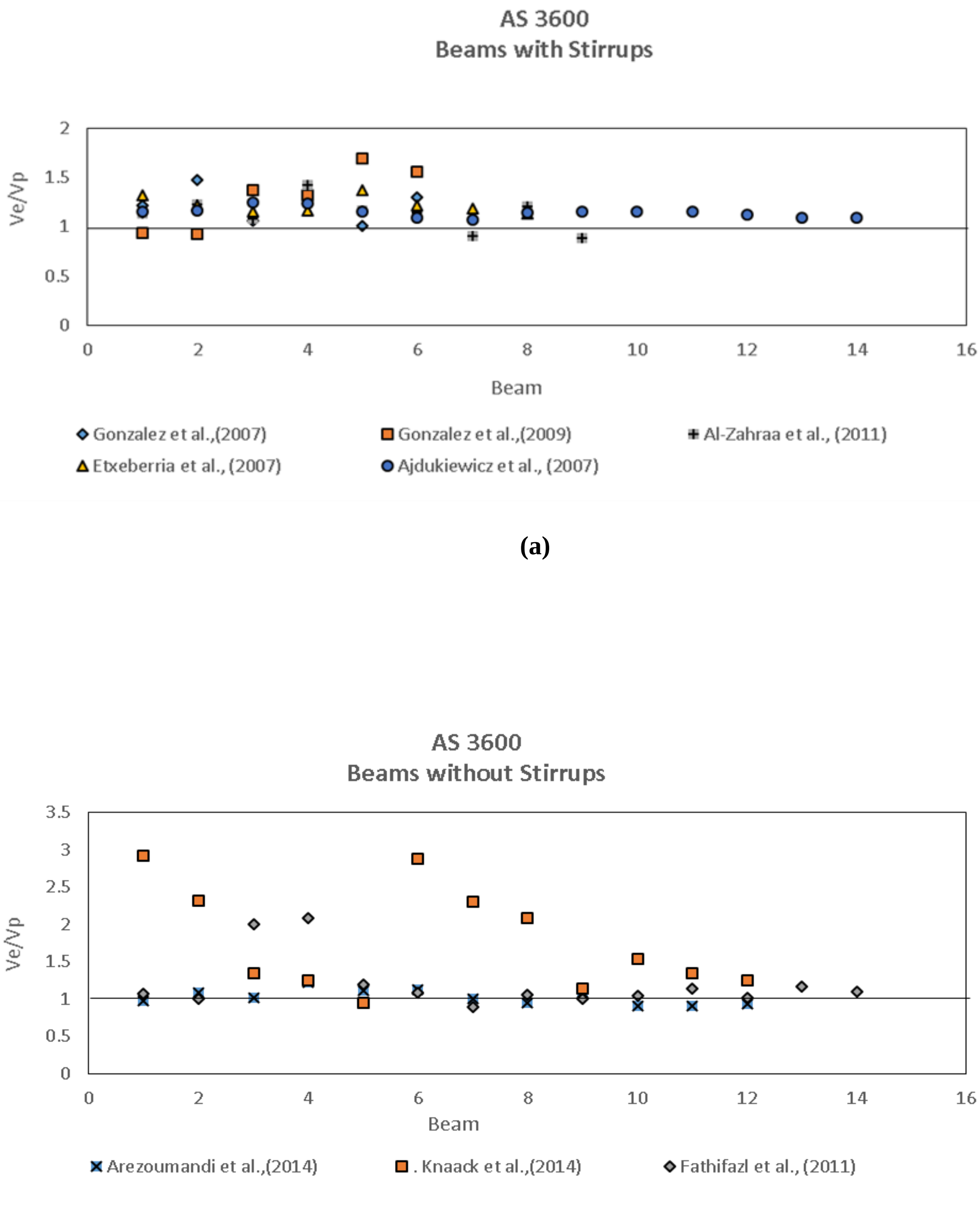


(a)



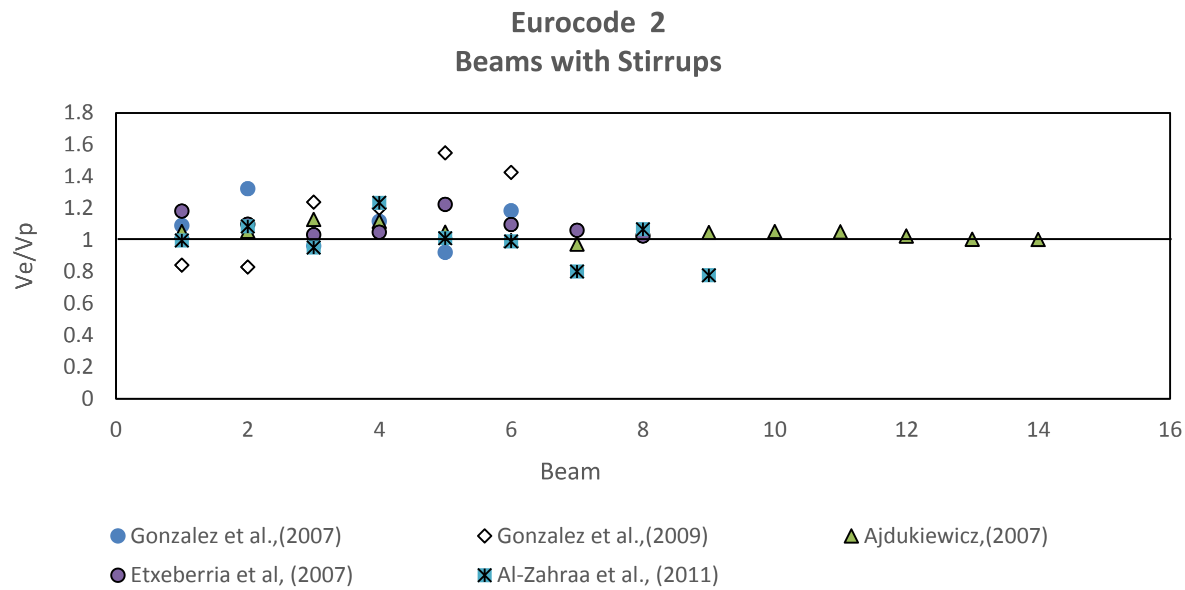
(b)

Figure 3.11 Ratio experiment-to-predicted values using CSA A23.3-04: (a) with stirrups (b) without stirrups

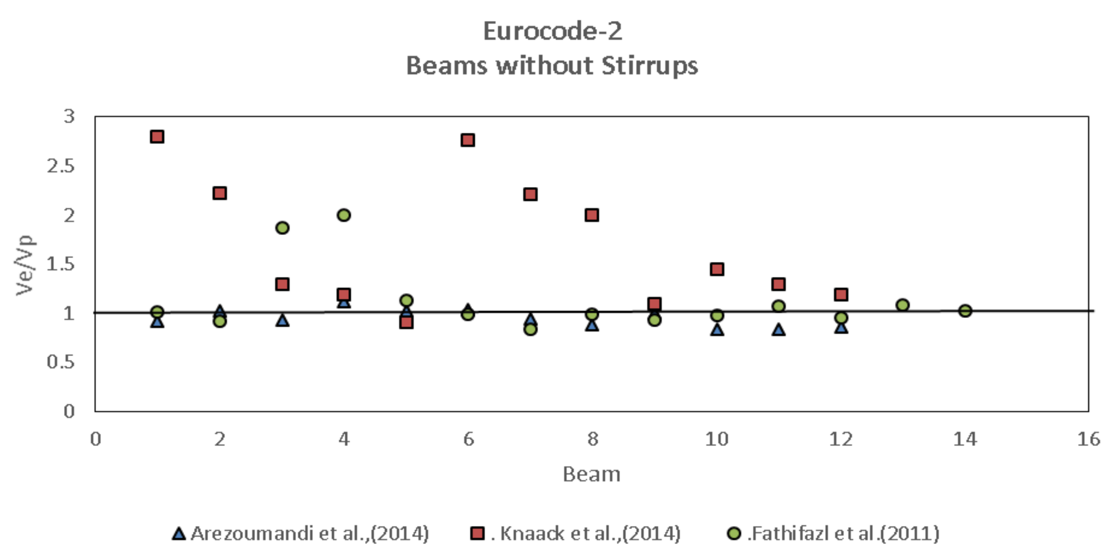


(b)

Figure 3.12 Ratio experiment-to-predicted values using AS 3600: (a) with stirrups (b) without stirrups

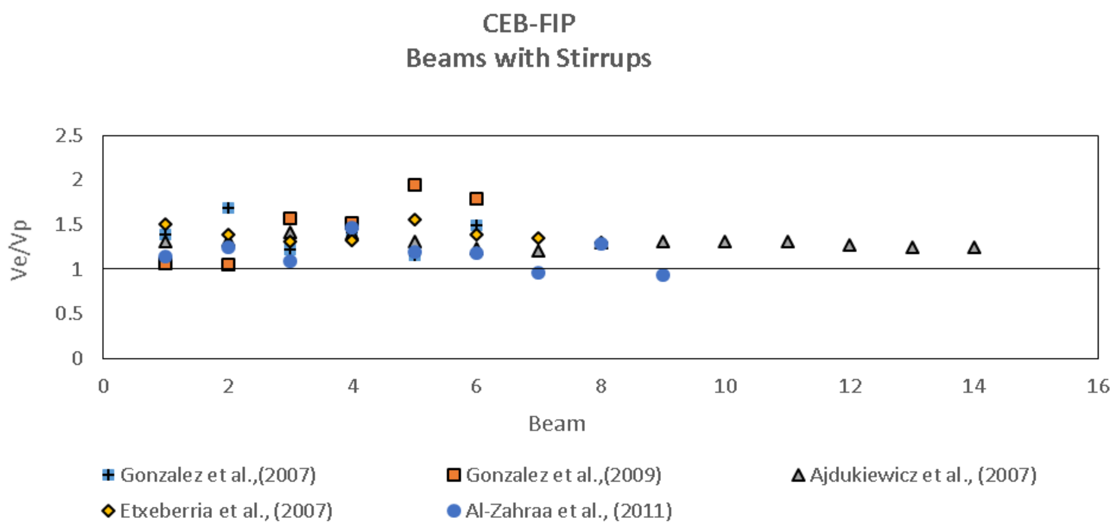


(a)

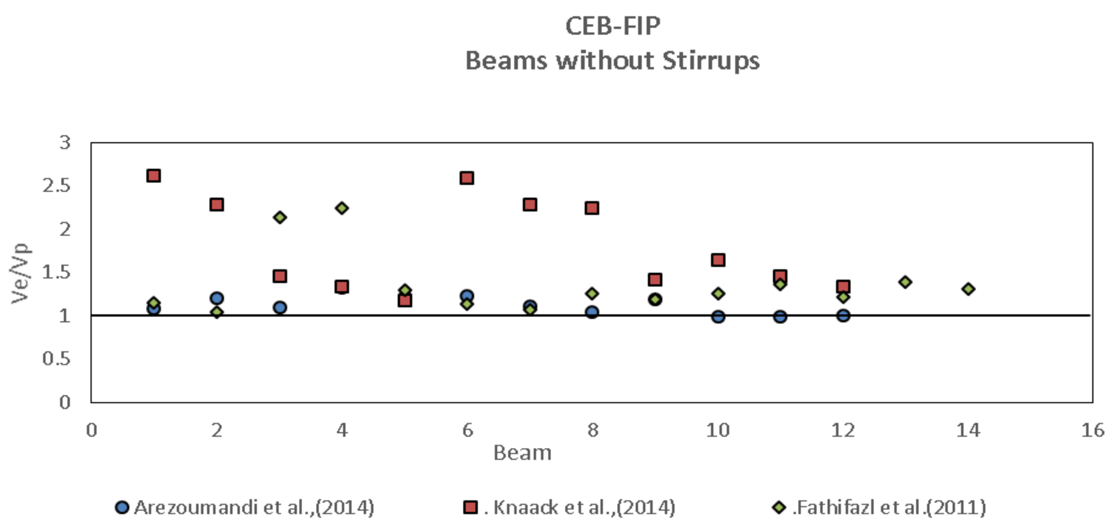


(b)

Figure 3.13 Ratio experiment-to-predicted values using Eurocode 2: (a) with stirrups (b) without stirrups

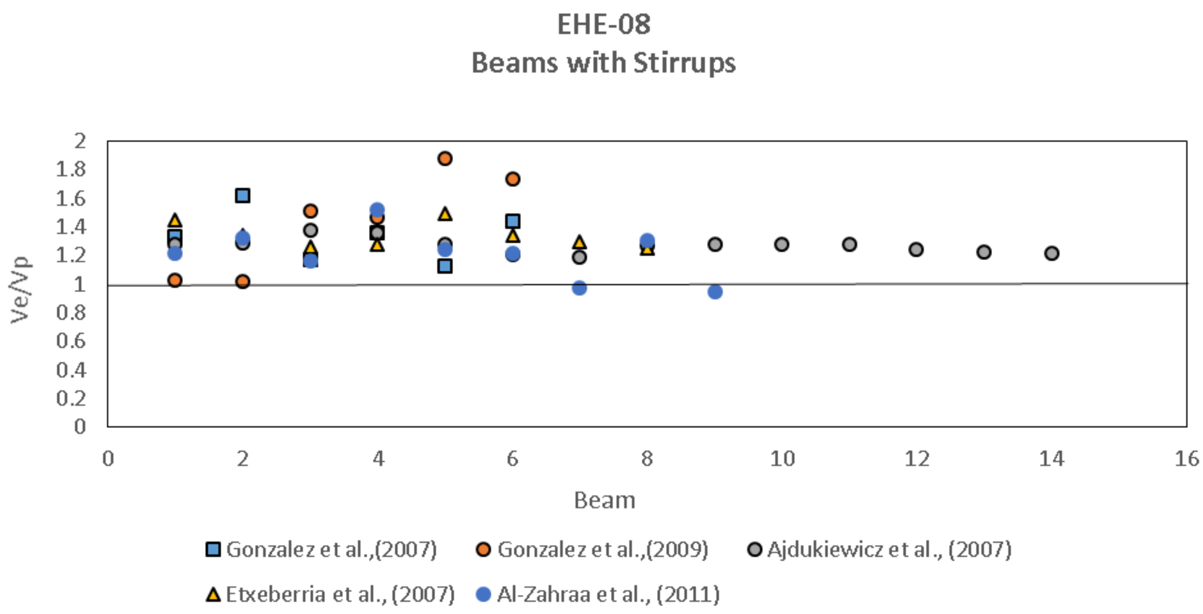


(a)

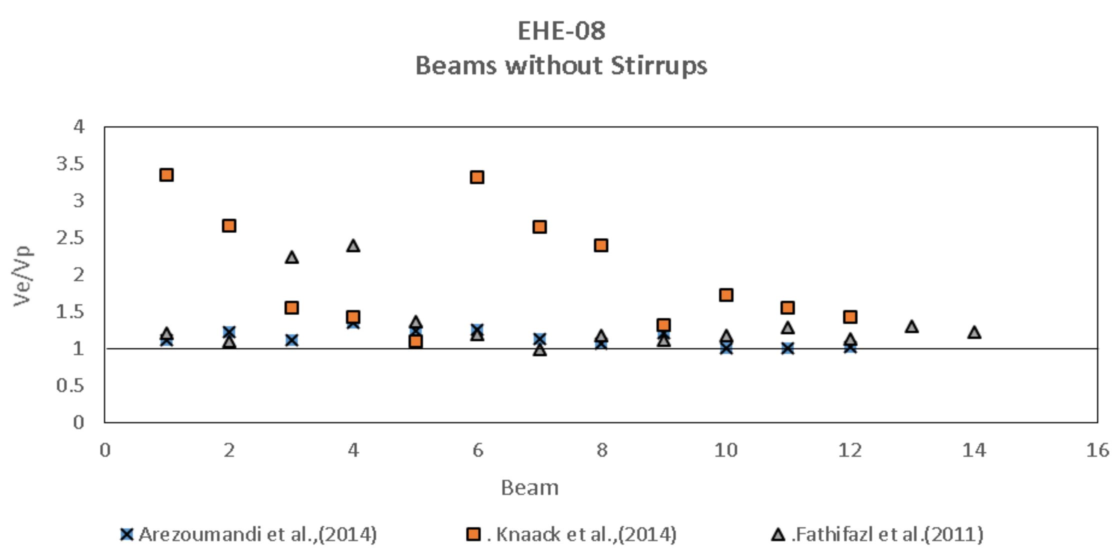


(b)

Figure 3.14 Ratio experiment-to-predicted values using CEB-FIP: (a) with stirrups (b) without stirrups

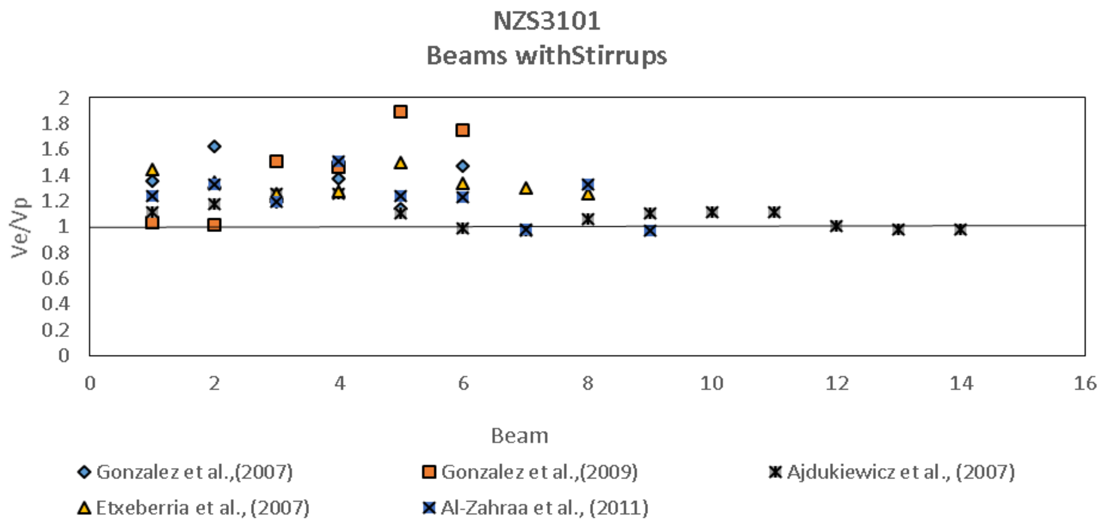


(a)

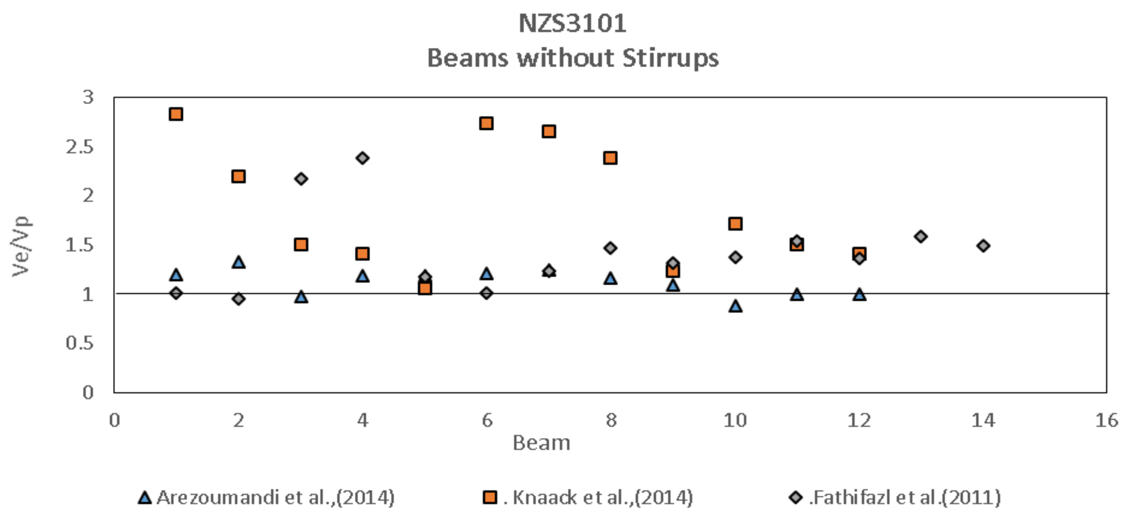


(b)

Figure 3.15 Ratio experiment-to-predicted values using EHE-08: (a) with stirrups (b) without stirrups

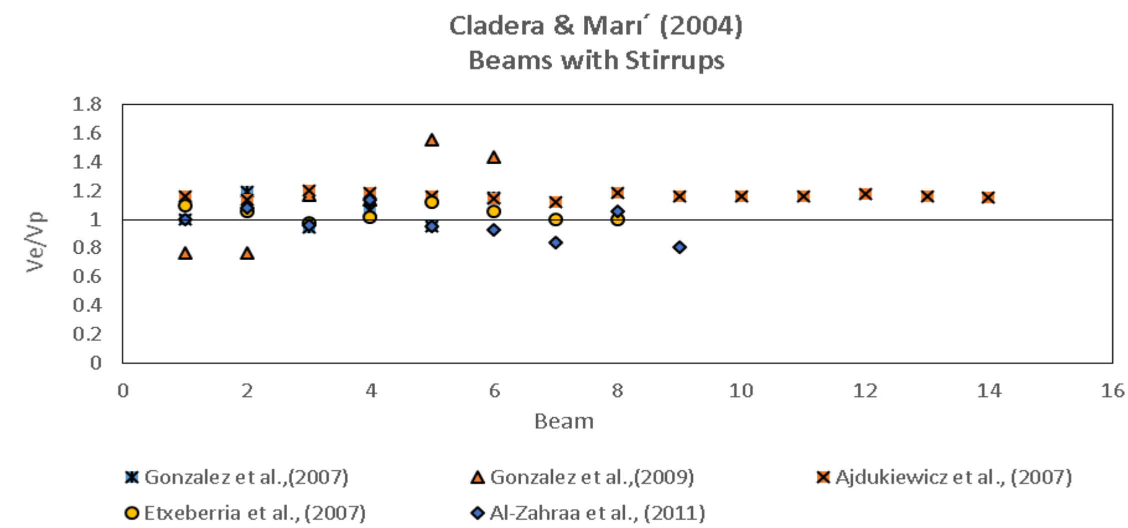


(a)

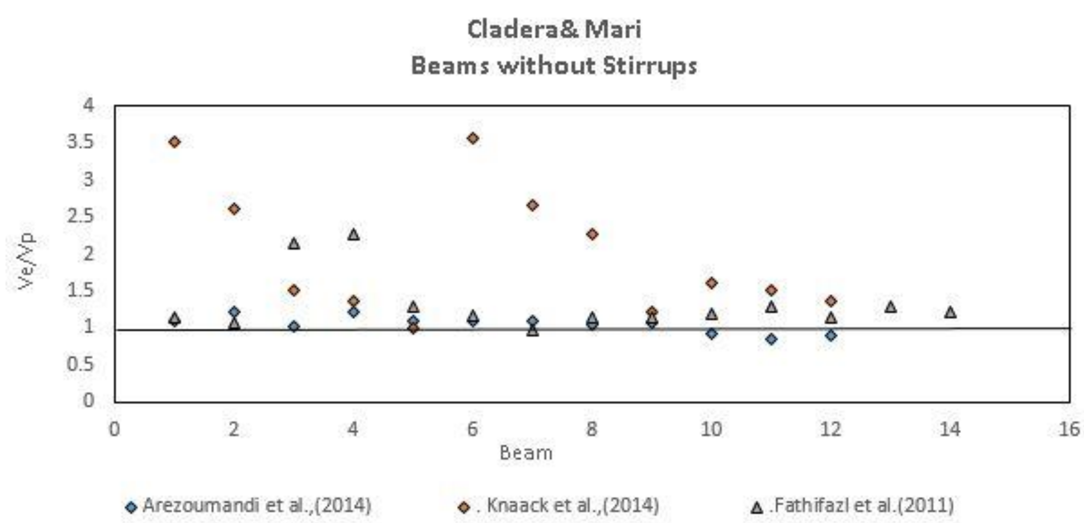


(b)

Figure 3.16 Ratio experiment-to-predicted values using NZS 3101: (a) with stirrups, (b) without stirrups



(a)



(b)

Figure 3.17 Ratio experiment-to-predicted values using Cladera and Mari (2004): (a) with stirrups (b) without stirrups

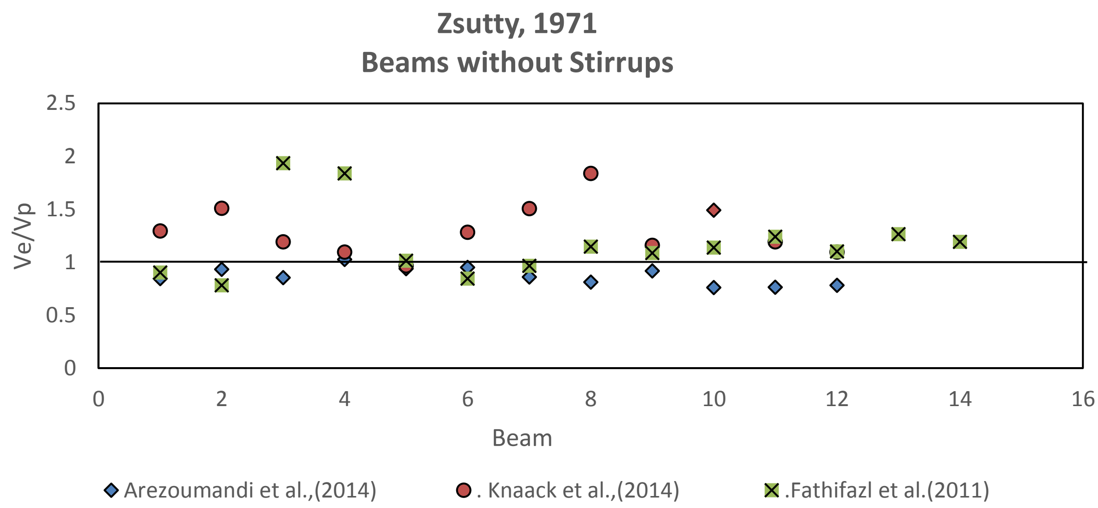


Figure 3.18 Ratio experiment-to-predicted values using Zsutty (1971) without stirrups

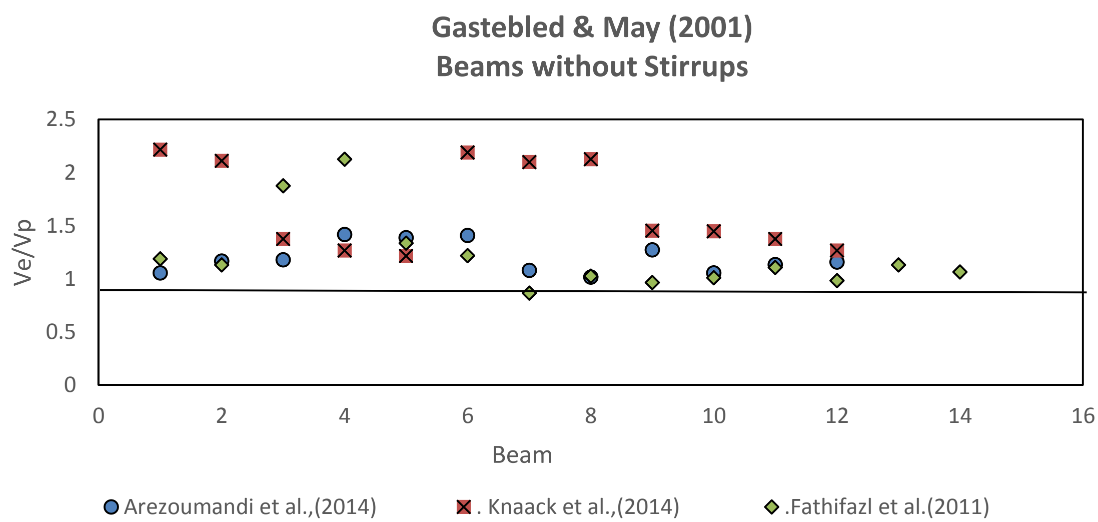


Figure 3.19 Ratio experiment-to-predicted values for Gastebled and May (2001): (a) with stirrups (b) without stirrups

Tables 3.9 and 3.10 present maximum, minimum, average and standard deviation of V_e/V_p values for the analyses of predictive ability of Code based and other existing equations. Based on Figures 3.10 to 3.19 and Tables 3.9 and 3.10, the following information on the predictive ability of Code based and other existing equations are derived:

<u>Beams without web reinforcement:</u> The ratio of experimented to predicted shear strength (V_e/V_p) based on **ACI 318-11** provisions ranges from 0.98 to 2.89 for normal and from 1.04 to 3.00 for simplified version. The shear strength of two beams out of 42 are overestimated by the Code (both general and simplified). For **CSA A23.3-04, Ve/Vp** ranges from 1.04 to 2.96 for simplified method and from 0.95 to 3.01 for general method. The general method overestimates the shear capacity of 3 beams out of 42 and the simplified method has no overestimation. For **AS 3600,** the V_e/V_p ranges from 0.83 to 2.79.This Code overestimates the shear capacity of 10 beams out of 48. For **Eurocode-2,** the V_e/V_p ranges from 0.88 to 3.35 and overestimates the shear capacity of 16 beams over the 48 beams. For **CEB-FIP,** the V_e/V_p ratio ranges from 0.98 to 2.61 and Code overestimates the shear capacity of 2 beams over the 48 beams. For **EHE-08,** the ratio ranges from 0.1.00 to 3.35 and the shear capacity of one beam is overestimated by the Code. The **NZS 3110 Code** has the ratio varying from 0.86 to 2.84 and the shear capacity 4 beams are overestimated. The method offered by **Cladera and Mari** has the ratio varying from 0.86 to 3.56. The method overestimates the shear capacity of 5 beams out of 43. The method of **Zsutty** has the ratio varying from 0.76 to 1.93 with the average of 1.09. The shear capacity of 18 beams are overestimated. The method of **Gastebled & Mari** has the ratio varying from 0.86 to 2.22 with an average value of 1.3 - the shear capacity of 7 beams are overestimated by the method.

<u>Beams with web reinforcement:</u> The V_e/V_p based on **ACI 318-11** provisions ranges from 1.07 to 2.14 (general) and from 1.07 to 2.15 (simplified) which are very close to each other. The simplified equation gives a conservative estimation for shear capacity with a mean value of 1.41. There is no overestimation of shear capacity with both of the equations. For **CSA A23.3-04,** the V_e/V_p ranges from 0.91 to 1.97 for simplified method and from 0.95 to 1.98 for general method. Both the simplified and general methods overestimate the shear capacity of 2 beams out of 43. For AS **3600,** the ratio ranges from 0.88 to 1.69 and the Code overestimates the shear capacity of 4 beams out of 43. For **Eurocode-2,** the ratio ranges from 0.78 to 1.55 with a mean value of 1.07 and the Code overestimates the shear capacity of 11 beams out of 43. For **CEB-FIP,** the ratio ranges from 0.93 to 1.94 and the code overestimates the shear capacity of two beams out of 43. For **EHE-08,** the ratio ranges from 0.95 to 1.88 with an average of 1.30 and the shear capacity of 2 beams with overestimated. The **NZS 3110** has the ratio varying from 0.97 to 1.65 with an average value of 1.90 and the shear capacity of 6 beams are overestimated. The method offered by **Cladera and**

Mari has the ratio of 0.77 to 1.08 with an average of 1.82. This method overestimated the shear capacity of 14 beams.

Table 3.9 Maximum, minimum, average and standard deviation of V_e/V_p for beams without stirrups

Codes and methods		Beams without stirrups			
		$(V_e/V_p)_{max}$	$(V_e/V_p)_{min}$	Average of V_e/V_p	Standard deviation of Ve/Vp
ACI-318	Eq.(11-3)	2.89	0.97	1.51	0.55
	Eq.(11-5)	3	1.04	1.58	0.56
CSA 23.3-04A	Simplified	2.96	1.04	1.54	0.55
	General	3.01	0.95	1.46	0.63
AS3600		2.91	0.89	1.3	0.5
Eurocode-2		2.79	0.83	1.22	0.52
CEB-FIP		2.62	0.98	1.41	0.45
EHE-08		3.35	1	1.5	0.61
NZs3101		2.84	0.86	1.47	0.51
Zsutty		1.93	0.76	1.08	0.28
Cladera and Mari		3.57	0.86	1.42	0.65
Gastebled and May		2.22	0.87	1.32	0.39
Maximum Average and Standard deviation		-	-	1.58 (ACI)	0.65 (Cladera)
Minimum Average and Standard deviation		-	-	1.08 (Zsutty)	0.28 (Zsutty)
Ve: Experimental shear strength; Vp: Predicted shear strength by Codes/existing methods					

Table 3.10 Maximum, minimum, average and standard deviation of V_e/V_p for beams without stirrups

Codes and methods		Beams with stirrups			
		$(V_e/V_p)_{max}$	$(V_e/V_p)_{min}$	Average of V_e/V_p	Standard deviation
ACI-318	Eq.(11-3)	2.14	1.07	1.4	0.24
	Eq.(11-5)	2.15	1.07	1.42	0.22
CSA 23.3-04A	Simplified	1.97	0.91	1.24	0.22
	General	1.98	0.95	1.26	0.21
AS3600		1.69	0.88	1.19	0.16
Eurocode-2		1.55	0.78	1.07	0.15
CEB-FIP		1.94	0.93	1.33	0.2
EHE-08		1.88	0.97	1.25	0.22
NZS3101		1.9	0.97	1.25	0.22
Zsutty		-	-	-	-
Cladera and Mari		1.56	0.77	1.08	0.15
Gastebled and May		-	-	-	-
Maximum Average and Standard deviation		-	-	1.42 (ACI)	0.24 (ACI)
Minimum Average and Standard deviation		-	-	1.07 (Euro code-2)	0.15 (Euro code-2)
Ve: Experimental shear strength; Vp: Predicted shear strength by Codes/existing methods					

In general, most of the Codes and existing methods underestimated the shear capacity of RAC beams without stirrups with ACI 318-11 showing the highest underestimation – hence, they can be used safely for the prediction of shear capacity. However, the method of Zsutty (1971) with an average value of 1.08 for V_e/V_p is found to be more accurate than Codes and other existing methods (Table 3.9).

Most of the Codes and existing methods underestimated the shear capacity of RAC beams with stirrups with ACI 318 showing the highest underestimation – hence, all the Codes/existing methods can be used for the prediction of shear capacity. However, Eurocode -2 and the method of Cladera and Mari with an average value of 1.07 and 1.08, respectively for V_e/V_p are found to be more accurate than other Codes and existing methods (Table 3.10).

However, over estimation by these Codes/existing methods for small percent of RAC beams in this study needs to be carefully addressed when using such Codes or methods.

3.4.2 Comparison of the influence of shear span to depth ratio

The current standards/code provisions or existing methods to evaluate the shear strength of beams are the results of experimental studies conducted using conventional concrete. The shear strength of beams is affected by different parameters like the amount of web reinforcement, the amount of longitudinal reinforcement, coarse aggregate size, beam size, tensile strength of the concrete, and shear span to depth ratio (a/d). The experimental results are used to study of the influencing parameters for shear strength. The shear span to depth ratio is one of the most influential parameters on the shear capacity of concrete beams. In Figure 3.20, the shear strength values of test beams without stirrups are plotted with respect to the shear span to depth ratio. The experimental results are compared with different codes/methods. The trend line is plotted based on the experimental results. To eliminate the effect of the beam size shear value (V) is divided by $b_w.d$. Figure 3.21 shows the $V/b_w.d$ values of beams without stirrups with respect to shear span to depth ratio. As the points in the Figure 3.21 overlaps each other, just three Codes are selected to show the relations. From both experimental and Code predicted values, shear strength is found to decrease with the increase of a/d ratio, as commonly observed for beams with normal aggregate concrete.

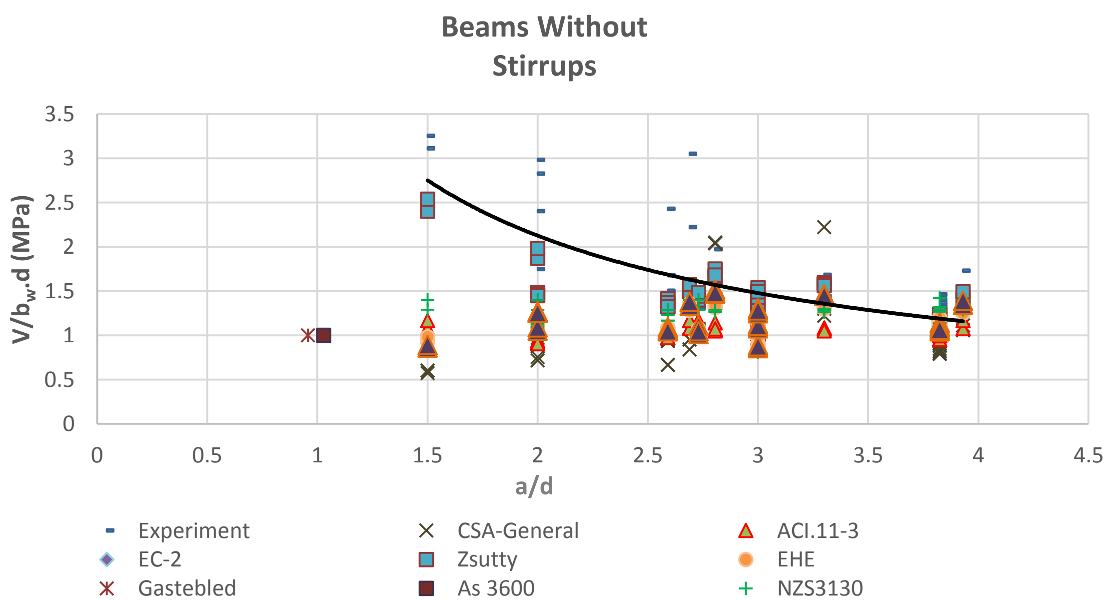


Figure 3.19 Effect of a/d on the shear strength of RAC beams without stirrups

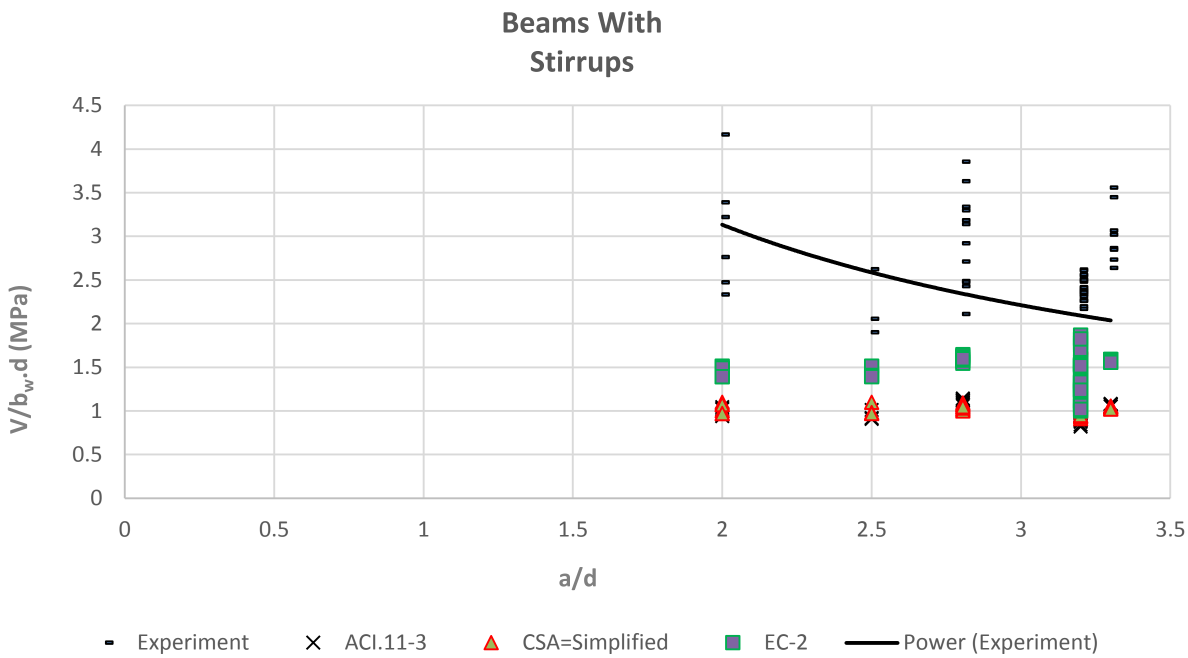


Figure 3.20 Effect of a/d on the shear strength of RAC beams with stirrups

CHAPTER FOUR

4. DEVELOPMENT OF ANN MODEL FOR PREDICTING SHEAR STRENGTH OF RAC BEAMS WITH AND WITHOUT SHEAR REINFORCEMENT

4.1 GENERAL

Artificial neural network (ANN) modelling is a relatively new approach for analysing the behavior of recycled aggregate concrete (RAC) beams in shear. Based on the definition, an artificial neural network is a computational tool that makes a pattern of architecture similar to internal features of human brain and nervous system (Sanad and Saka, 2001). It can be applied for a wide range of system modeling. There are outstanding advantages of ANN model applications such as using various variables without needing to functional relationship amongst them.

According to previous and this research studies, existing code provisions and other existing methods to evaluate the shear strength of conventional concrete beams may need modifications for RAC beams. ANN models can be an alternative solution to predict shear strength of RAC beams. This chapter presents the development of ANN models for predicting the shear strength of RAC beams with/without web/shear reinforcement. The data collected from previous experimental research studies presented in chapter 3 is used to develop and train three ANN models. Model 1 is for beams with and without stirrups, Model 2 is for beams with stirrups and Model 3 for beams without stirrups. The performance three models is then validated by using twelve RAC beams from other research studies.

4.2 Development of Artificial Neural Network (ANN) Model

The neural network modeling in the present study is conducted by using commonly used back-propagation technique (Rumelhart et al., 1986). A typical network structure consists of couple of layers comprising an input layer, an output layer and one or more hidden layers. The influential factors are inserted in the input layer with no computational activities between them while the output layer contains one or more processing units that produce the network outputs. Hidden layers are placed between the input and output layers and various hidden processing units are expected.

The calculation procedure uses input layer and compares the network outputs with known targets, and propagates the error back to the network. Figure 4.1 shows a simple model of a back-propagation network showing input layer, a hidden layer, an output layer, and connections are plotted**.**

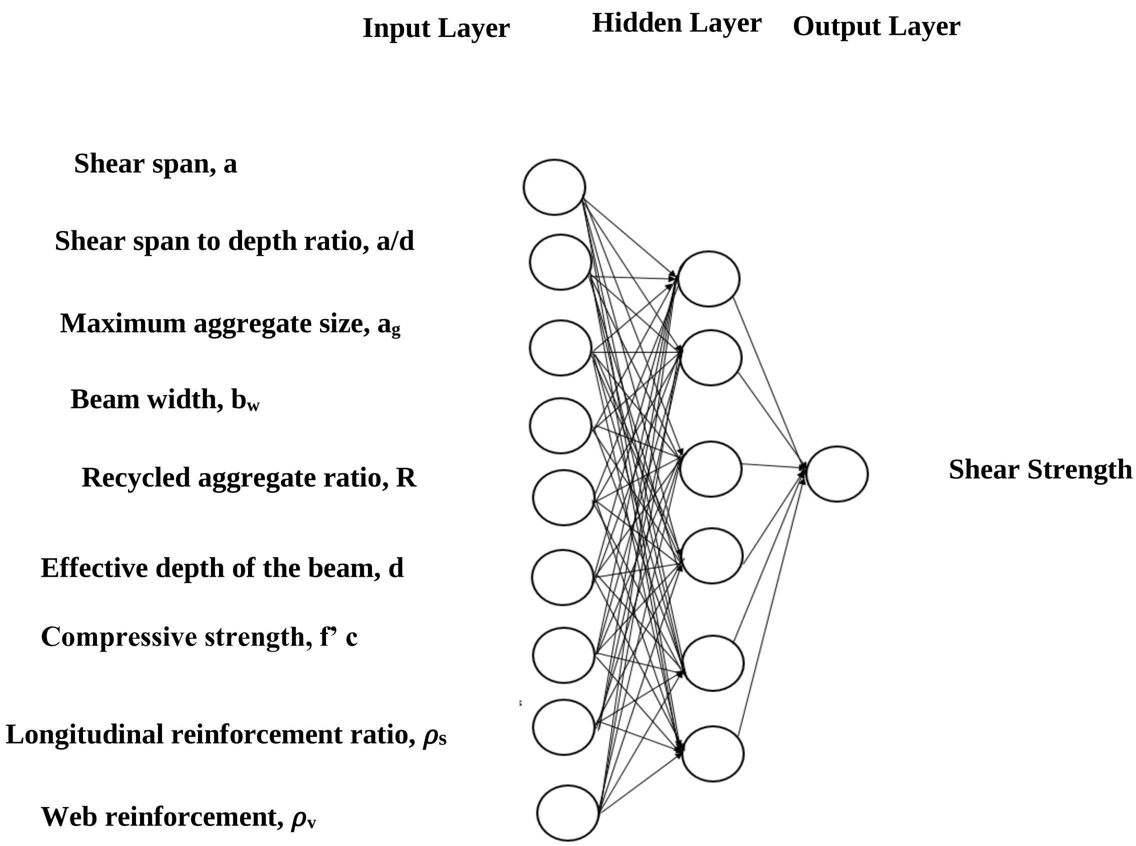


Figure 4.1 ANN Architecture

4.2.1 Experimental database

Data collection is an important part of the ANN model development. In this study, data is collected from previous experimental studies (described in Chapter 3) conducted by Fathifzl et al. 2011, Etxeberria et al. 2007, Ajdkiewicz et al. 2007, Knaack et al. 2014, Al-Zahraaet al. 2011, González et al. 2009, González et al. 2007 and Arezoumandi et al. 2014. A total of 96 RAC beams with and without shear reinforcements are selected to construct the database for training and development of three ANN models. Combined Model 1 developed for RAC beams with and without web reinforcement used all 96 beams, Model 2 developed for beams with web reinforcement used 43 beams and Model 3 developed for beams without web reinforcement used 53 beams. The performance of these three beams are validated by using additional 12 beams tested by Han et al. (2001) and Pellegrino and Faleschini (2013).

4.2.2 Network architecture

Determination of an optimal network architecture is another important step in the development of ANN models. To obtain the optimal network architecture the number of input variables denoted by “i” and the number of hidden neurons denoted by “h” could be selected by series of trials to get output with reasonable accuracy as well as to identify the influential input parameters and to select number of hidden neurons for better model performance. The architecture is of ANN model is denoted by numbers of input, hidden and output neurons such as like “i: h: o”. ANN model architecture shown in Figure 4.1 can be denoted by 9:6:1.

Nine influential input parameters that affect the shear strength are selected based on previous experimental studies and Code based/other existing. The input parameters are: concrete compressive strength (f'_c), longitudinal reinforcement ratio (ρ_s), shear reinforcement (ρ_V), shear span(a), shear span to depth ratio a/d, beam width (b), beam effective depth (d), the recycled aggregate percentage (R%) and maximum aggregate size (a_g) (Figure 4.1). The effect of these parameters on the shear strength of beams is explained in Chapter two as well as in Chapter Three. Comprehensive parametric ANN modeling is conducted to obtain the best network architecture for three ANN models and details are provided in the following sections.

4.2.3 Training of ANN models

The process of developing ANN model for predicting shear strength to determine the most appropriate neural network model consists of validation process which gives evidence for the effectiveness of the network architecture. The ranges of input parameters obtained from 96 experimental RAC beams are provided in Table 4.1.

To find the appropriate number of hidden neurons (h) for predicting shear capacity of RAC beams, the input variables are keep constant and further operations continued until reaching the suitable number of hidden neurons producing output of better accuracy. The next step after finding the most suitable number of hidden neurons, is finding the influence of each of the input parameters by excluding them one by one or two from the input layer and getting the output (shear strength of beams 'V' in this study) with reasonable accuracy.

Nine input neurons are fed into the network with their specific weights and influence the generated weights of output layer. This process is repeated till the output layer has the least error based on regulated weights in each turn until to reach a constant value with respect to the experimental targets (shear Strength) (Yao, 1999). Finally, the training releases the optimized output.

Table 4.1 Experimental input parameter data range

Input	A (mm)	p_s (%)	p_v (%)	B (mm)	D (mm)	$f'c$ (MPa)	a/d	a_g (mm)	R (%)	V (kN)
Min	360.0	1.00	0.00	100	180	29.58	1.50	16	0	0.942
Max	1299.5	2.98	0.50	300	476	107.8	3.93	25	100	4.167
Concrete compressive strength: f'c, longitudinal reinforcement ratio: ρ_s, shear reinforcement ratio: ρ_v, shear span: a, shear span to depth ratio: a/d, beam width: b, beam effective depth: d, the recycled aggregate percentage: R, maximum aggregate size: a_g and shear strength of beam: V										

4.3 Results and performance evaluation

The results of ANN modeling are presented as statistical inferences. The value of degree of agreement denoted by ξ is calculated by the equation 4.1. Perfect agreement between predicted and experimental values will yield a Degree of Agreement of 1, and no agreement between the values will yield a degree of 0.

$$\xi = 1 - \frac{\sum_{m=1}^{n}(V_{pm}-V_{em})^2}{\sum_{m=1}^{n}\{(V_{pm}-V_{e,avg})+(V_{em}-V_{e,avg})\}^2} \qquad (4\text{-}1)$$

Where, V_{pm} is the predicted value for the m^{th} entry, V_{em} is the experimental value of the m^{th} entry, and V_{avg} is the average experimental shear strength.

The Mean Squared Error (MSE) and the Root Mean Squared Error (RMSE) measure the amount of difference between experimental and predicted values and calculating with the equations 4-2) and 4-3):

$$MSE = \frac{1}{N}\sum_{i=1}^{N}(V_{pm} - V_{em})^2 \qquad (4\text{-}2)$$

$$RMSE = \sqrt{\frac{\sum_{i=1}^{N}(V_{pm} - V_{em})^2}{N}} \qquad (4\text{-}3)$$

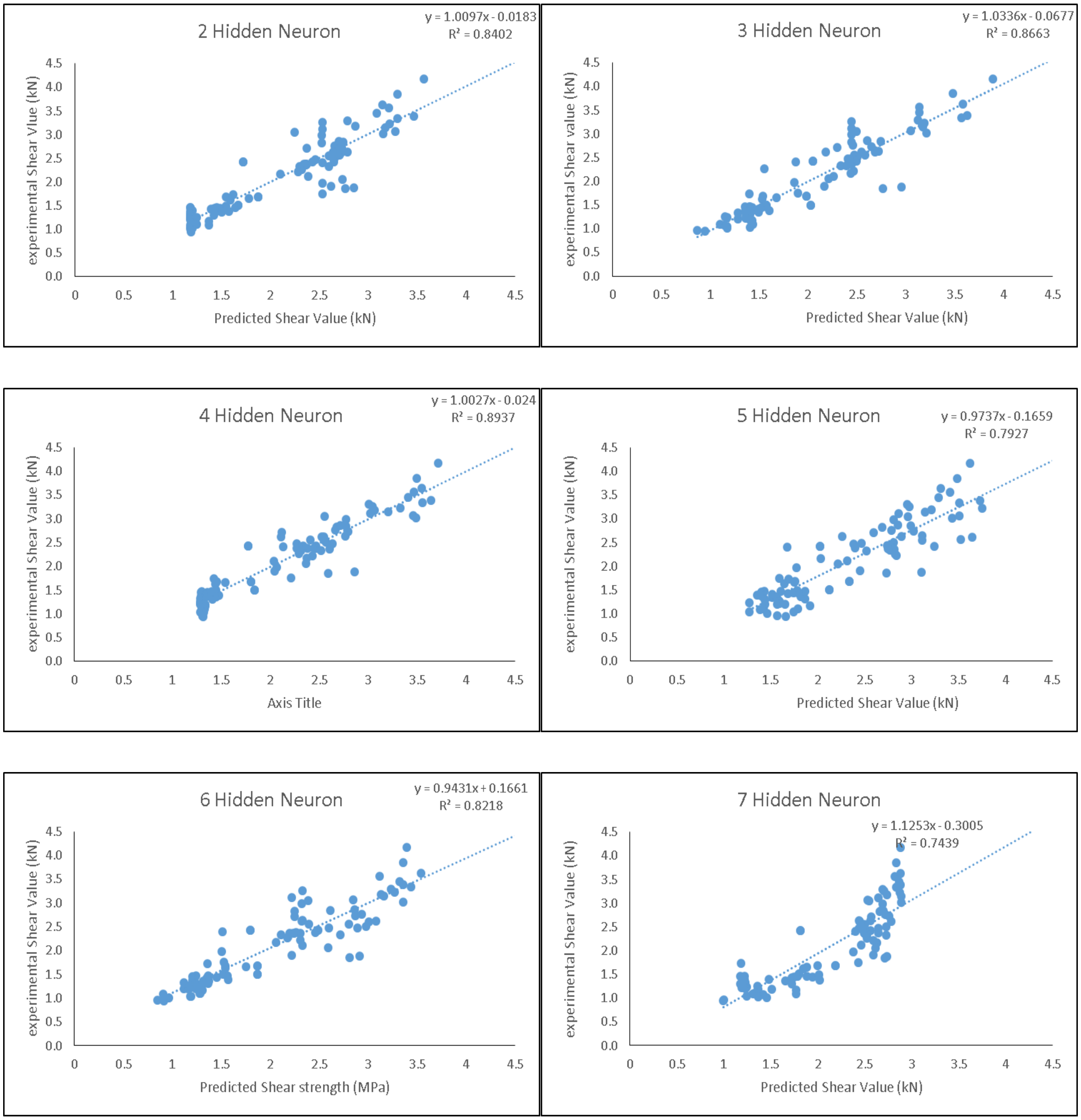
Where, N is the total number of events considered.

4.3.1 Effect of number of hidden neurons

In order to find the optimal number of hidden neurons, different network architecture (i:h:o) was examined by keeping the number of input variables constant and trying for two to nine hidden neurons to obtain the most appropriate shear strength with the higher degree of agreement. Tables 4.2, 4.3 and 4.4 listed the results of applied Artificial Neural Network model for modeling number one, number two and number 3, respectively. The tables contain the Network Architecture, i:h:o, where "i" represents number of inputs, "h" is number of hidden neurons and "o" is number of outputs; V_p/V_e: ratio of shear strength from ANN model to experimental shear strength of RAC beam; ε_ι: degree of agreement; V_{avg}: difference between the average predicted value and the average target value; V_σ: difference between the standard deviation of the predicted data set and the standard deviation of the target data set; V_{cv}: difference between the coefficient of variance of the predicted data set and the coefficient of variance of the target data set; V_{max}: difference between the maximum value in the predicted data set and the maximum value in the target data set; V_{min}: difference between the minimum value of the predicted data set and the minimum value of the target data set; V_{range}: difference between the range of predicted values and the range of target values.

For Model 1 (which contains all beams with and without stirrups), the ANN model with 8 hidden neurons and network architecture of 9:8:1 is found to be in a good agreement (Table 4.2). The agreement factor, ε_ι is 0.9746 which is closer to 1, MSE is 0.04237 which shows that the selected ANN model is having less error, V_p/V_e is 0.9763 which is closer to 1. The V_{avg} of 0.0498 which shows the difference in average of ANN model output shear strength to input shear strength is in a minimum value to keep the developed ANN model in good agreement. V_σ of 0.0328 is the difference in the standard deviation of output shear strength to input shear strength, standard deviation shows the variation from average and the lower the standard deviation the closer data points to expected value. V_{cv} of -0.0068 is the difference in the covariance of the output shear strength and indicates a lesser variant in the ANN output average to experimental average. V_{min} is

0.0429, V_{max} is -0.5347 and V_{range} is 0.5776. Figure 4.2 also shows that the equation of y = 0.9938x + 0.0626 for 8 hidden neurons indicates the coefficient of x value of 0.9938 - which is closer to 1 indicating that the predicted shear strength is closer to experimental shear stress.



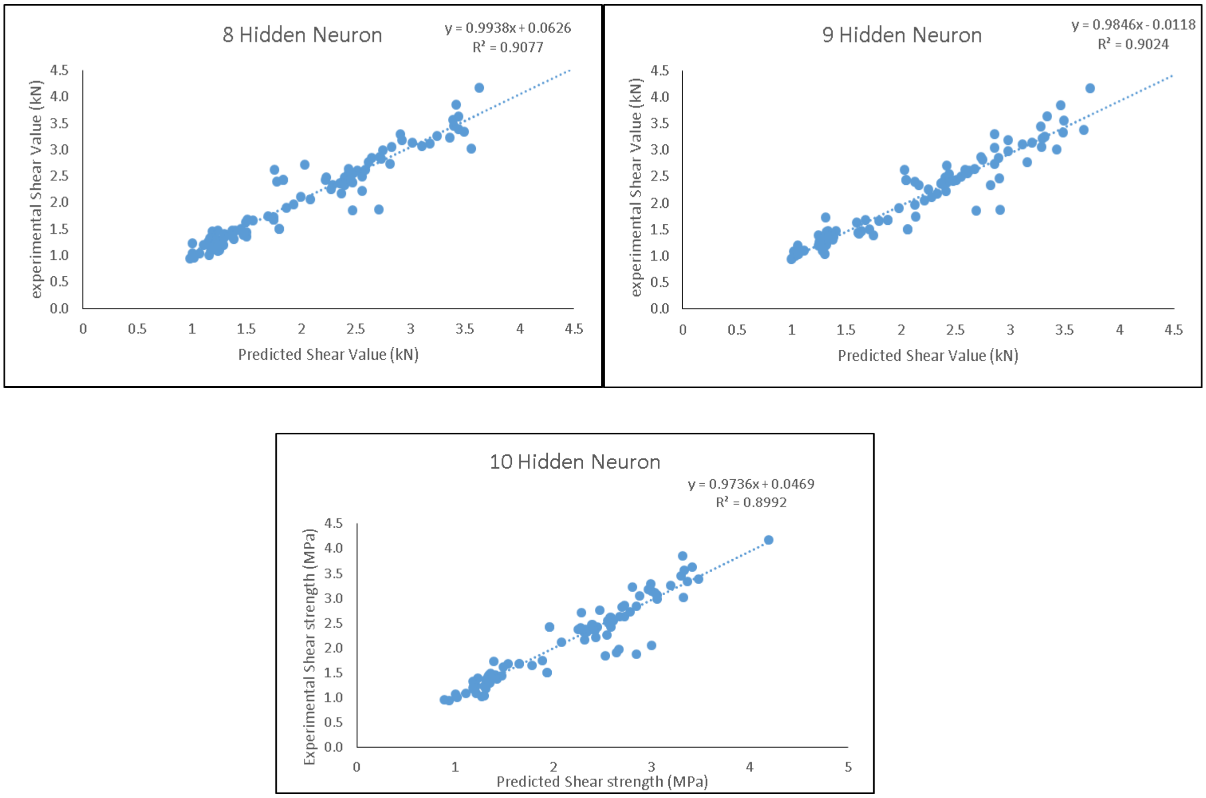


Figure 4.2 Predicted versus experimental shear capacity from various numbers of hidden neurons for Model 1

Table 4.2 Evaluation of Number of Hidden Neurons for Model 1, beams with and without stirrups

Hidden Neurons	i:h:o *	ξ	V_p/V_e	MSE	RMSE	V_{avg}	V_{σ}	V_{range}	V_{min}	V_{max}	V_{cv}
n=2	9:2:1	0.9556	0.9990	0.0703	0.2652	0.0020	0.0732	0.8375	0.2424	-0.5951	-0.0345
n=3	9:3:1	0.9624	0.9987	0.0592	0.2433	0.0028	0.0789	0.2021	-0.0730	-0.2751	-0.0371
n=4	9:4:1	0.9717	1.0087	0.0470	0.2168	0.0183	0.0454	0.7929	0.3415	-0.4514	-0.0661
n=5	9:5:1	0.9235	1.1081	0.1278	0.3575	0.2271	0.0679	0.7389	0.3295	-0.4093	-0.0661
n=6	9:6:1	0.9507	0.9765	0.0814	0.2853	0.0493	0.0307	0.5323	-0.0957	-0.6280	-0.0059
n=7	9:7:11	0.9109	1.0159	0.1175	0.3427	0.0333	0.1853	1.3279	0.0525	-1.2754	-0.0928
n=8	9:8:1	0.9746	0.9763	0.0424	0.2058	0.0498	0.0328	0.5776	0.0429	-0.5347	-0.0068
n=9	9:9:1	0.9738	1.0213	0.0444	0.2108	0.0448	0.0279	0.4861	0.0553	-0.4308	-0.0209
n=10	9:10:1	0.9738	1.0042	0.0447	0.2114	0.0089	0.0206	0.0755	-0.0462	0.0293	-0.0114

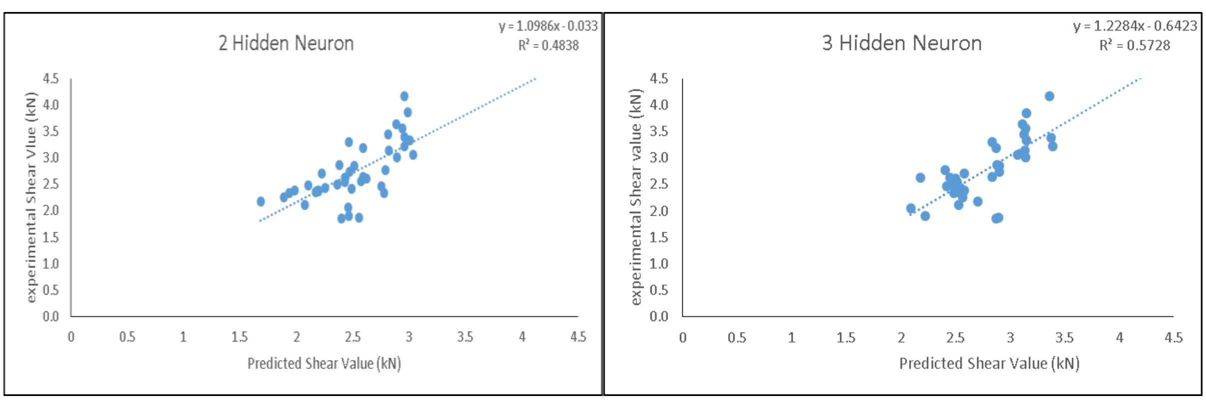
*Network Architecture

For Model 2 (beams with stirrups), the ANN model with 6 hidden neurons and network architecture of 9:6:1 is found to be in a good agreement. The agreement factor, ξ is 0.9605 which is closer to 1, MSE is 0.0239 which shows that the selected ANN model is having less error, V_p/V_e is the ratio of predicted shear strength to experimental shear strength of RAC beam is 1.0288 which is closer to 1. The V_{avg} of 0.0783 which shows the difference in average of ANN model output shear strength to input shear strength is in a minimum value to keep the developed ANN model in a good agreement. V_{σ} of 0.1080 is the difference in the standard deviation of output shear strength to input shear strength, standard deviation shows the variation from average and the lower the standard deviation the closer data points to expected value. V_{cv} of 0.0089 is the difference in the covariance of the output shear strength and indicates a lesser variant in the ANN output average to experimental average. V_{min} is 0.3837 V_{max} is –0.1568 and V_{range} is 0.5405. Figure 4.3 shows that the equation y = 0.9615 x + 0.0295 for 6 hidden neurons indicating the coefficient of x of 0.9615 which is closer to 1 indicating the predicted shear strength is closer to experimental shear strength.

Table 4.3 Evaluation of number of hidden neurons for Model 2, beams with stirrup

Hidden Neurons	i:h:o *	ξ	V_p/V_e	MSE	RMSE	V_{avg}	V_{σ}	V_{range}	V_{min}	V_{max}	V_{cv}
n=2	9:2:1	0.910553	0.921315	0.021426	0.146376	0.213898	0.19852	0.957451	-0.1705	-1.12795	-0.06226
n=3	9:3:1	0.953616	1.006406	0.023405	0.152986	0.017413	0.207751	1.014976	0.237397	-0.77758	-0.0772
n=4	9:4:1	0.95373	1.016062	0.023629	0.153718	0.043664	0.263649	1.367636	0.57827	-0.78937	-0.0986
n=5	9:5:1	0.964729	1.013442	0.023568	0.15352	0.03654	0.06306	0.878637	0.186339	-0.6923	-0.02553
n=6	9:6:1	0.960483	1.028787	0.023925	0.154678	0.078253	0.107984	0.540482	0.383687	-0.15679	0.008888
n=7	9:7:11	0.893864	1.005876	0.023392	0.152946	0.015974	0.141932	0.908868	-0.09535	-1.00422	-0.05307
n=8	9:8:1	0.925696	1.031157	0.02398	0.154856	0.084698	0.231299	1.04705	0.237772	-0.80928	-0.08853
n=9	9:9:1	0.965134	0.998175	0.023213	0.152359	0.00496	0.138155	0.17911	-0.15081	-0.32992	-0.05055
n=10	9:10:1	0.975033	0.997218	0.023191	0.152286	0.007562	0.03462	0.335901	0.16531	-0.17059	-0.01222

*Network Architecture



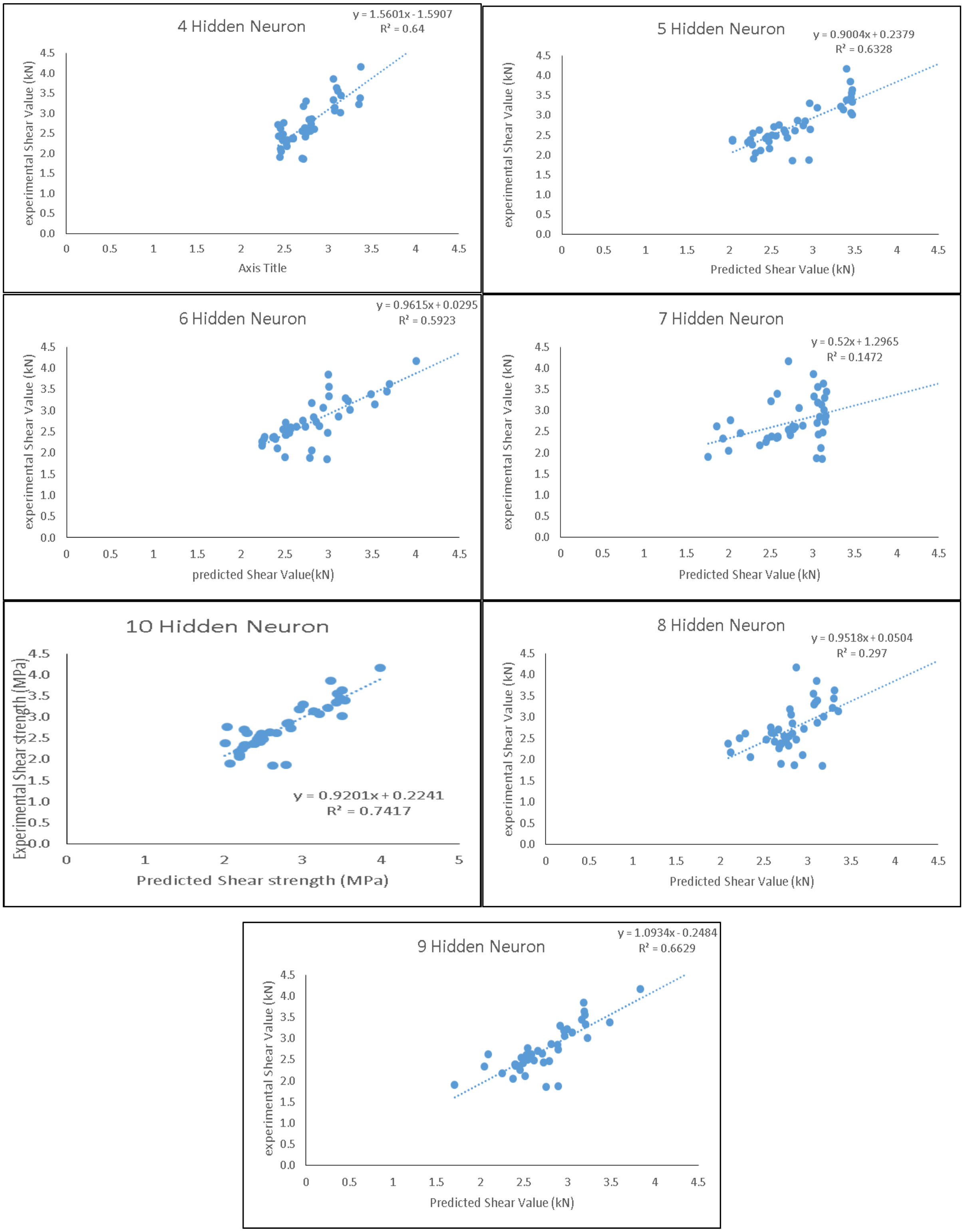
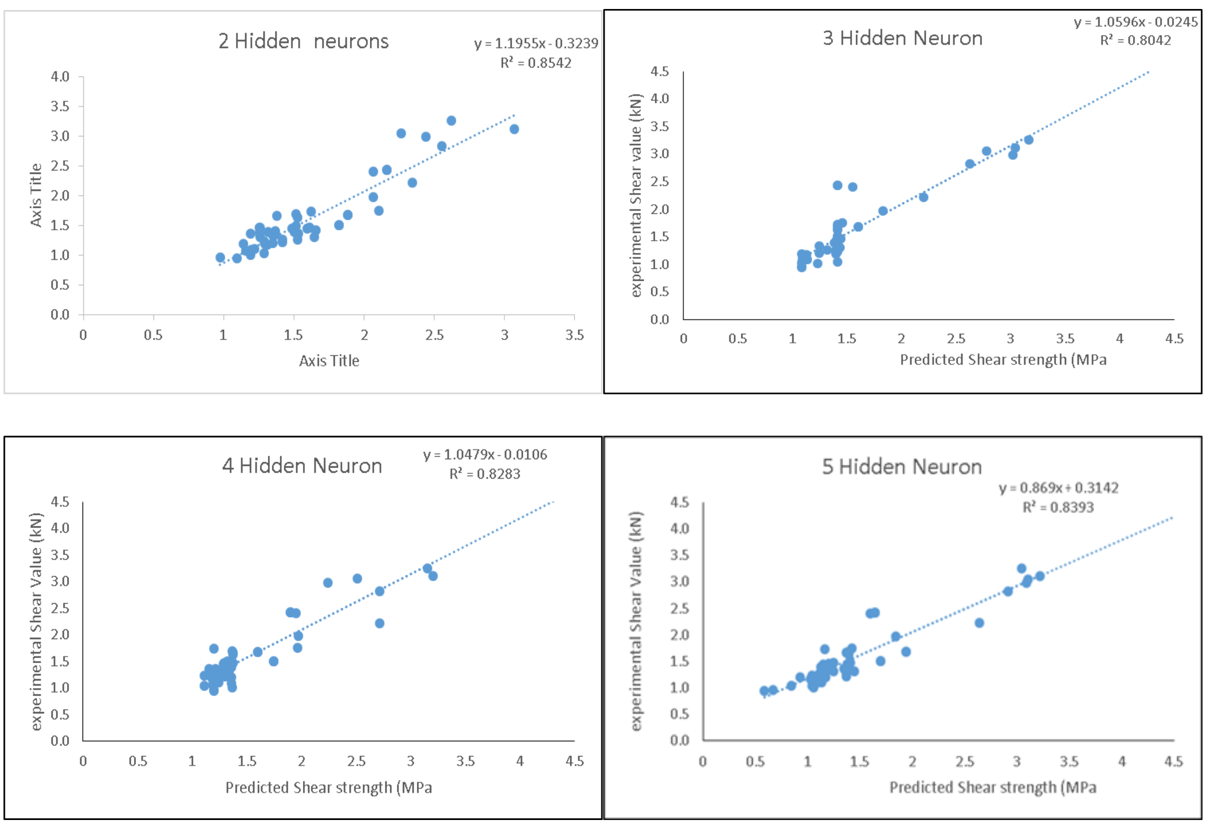


Figure 4.3 Predicted to experimental shear capacity from various numbers of hidden neurons for **Model 2**

For Model 3 (beams without stirrups), the ANN model with 6 hidden neurons and network architecture of 9:6:1 is found to be in a good agreement. The agreement factor, ε is 0.9901 which is closer to 1, MSE is 0.0191 which shows that the selected ANN model is having less error, V_p/V_e is the ratio of predicted shear strength to experimental shear strength of RAC beam and it is 1.0157 which is closer to 1. The V_{avg} of 0.0010 which shows the difference in average of ANN model output shear strength to input shear strength is in a minimum value to keep the developed ANN model in a good agreement. V_σ of 0.0104 is the difference in the standard deviation of output shear strength to input shear strength, standard deviation shows the variation from average and the lower the standard deviation the closer data points to expected value. V_{cv} of 0.0008 is the difference in the covariance of the output shear strength and indicates a lesser variant in the ANN output average to experimental average. V_{min} is 0.0009 V_{max} is -0.0001 and V_{range} is 0.0010. Figure 4.4 shows that the equation y = 0.956x + 0.0463 for 6 hidden neurons has the value of the coefficient of x as 0.9560 closer to 1 indicating the predicted shear strength is closer to experimental.



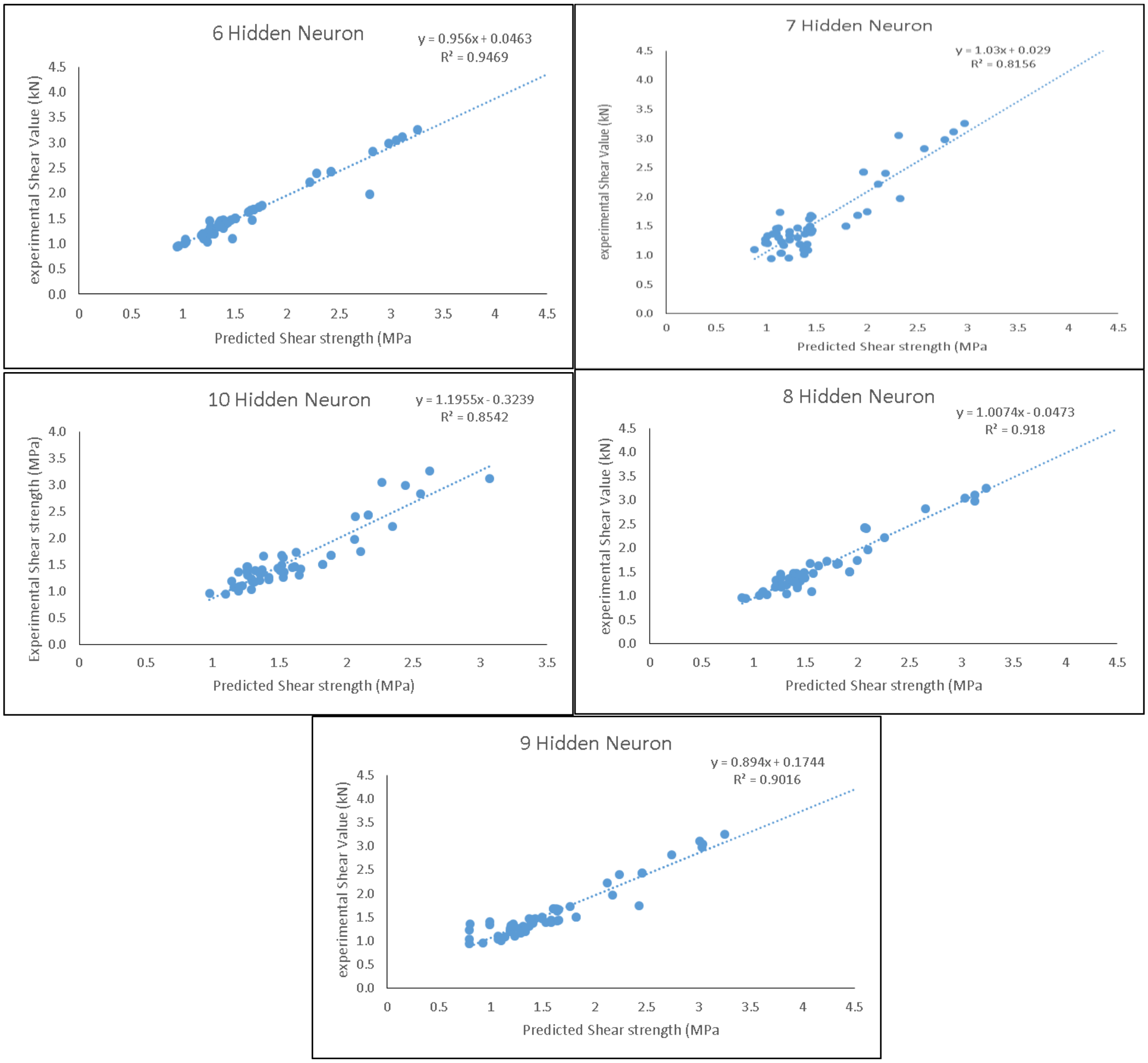


Figure 4.4 Predicted to experimental shear capacity from various numbers of hidden neurons for **Model 3**

Table 4.4 Evaluation of number of hidden neurons for Model 3, beams without stirrups

Hidden Neurons	i:h:o *	ξ	V_p/V_e	MSE	RMSE	V_{avg}	V_σ	V_{range}	V_{min}	V_{max}	V_{cv}
n=2	9:2:1	0.9652	1.0060	0.0565	0.2377	0.2153	0.1325	0.2153	0.0327	-0.1826	-0.0846
n=3	9:3:1	0.9622	0.9582	0.0708	0.2661	0.2320	0.0897	0.2320	0.1452	-0.0868	-0.0427
n=4	9:4:1	0.9668	0.9606	0.0620	0.2489	0.2191	0.0768	0.2191	0.1677	-0.0514	-0.0350
n=5	9:5:1	0.9664	0.9245	0.0747	0.2733	0.3200	0.0317	0.3200	-0.3549	-0.0349	0.0513
n=6	9:6:1	0.9901	1.0157	0.0191	0.1380	0.0010	0.0104	0.0010	0.0009	-0.0001	0.0008
n=7	9:7:11	0.9641	0.9533	0.0675	0.2598	0.2239	0.0719	0.2239	-0.0597	-0.2836	-0.0293
n=8	9:8:1	0.9842	1.0220	0.0287	0.1693	0.0327	0.0286	0.0327	-0.0524	-0.0197	-0.0253
n=9	9:9:1	0.9816	0.9965	0.0372	0.1928	0.1414	0.0362	0.1414	-0.1489	-0.0075	0.0241
n=10	9:10:1	0.9652	1.0060	0.0565	0.2377	0.2153	0.1325	0.2153	0.0327	-0.1826	-0.0846

*Network Architecture

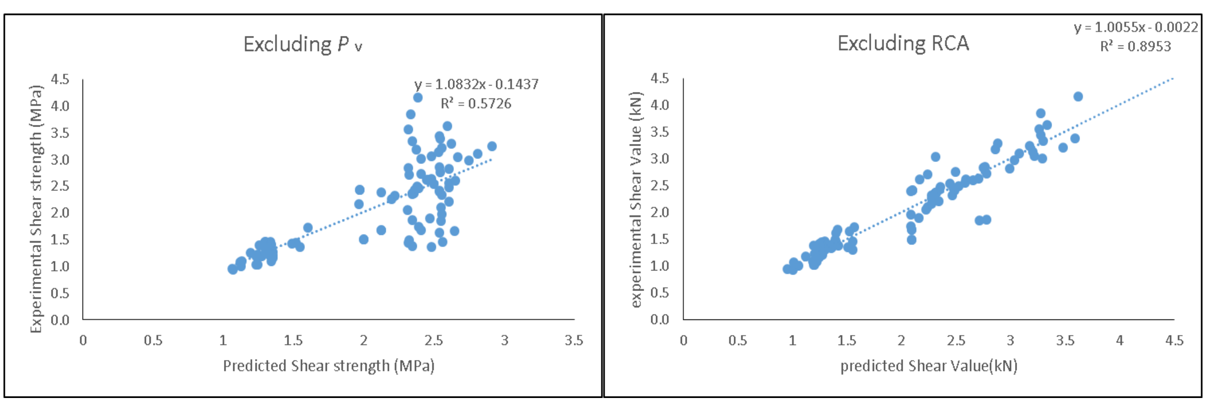
4.3.2 Effect of input parameters

In Model 1, the network architecture of (9:8:1) is selected based on the optimum number of hidden neurons. Parametric studies are conducted excluding one or two input variables to verify the influence of input variables to the model. Table 4.5 shows the results of excluding input variables or their combinations with the constant hidden neurons of eight. The ANN model with all the nine input variables with an architecture network 9:8:1 is in a good agreement. The agreement factor, ξ, is 0.9746 which is closer to 1, MSE is 0.0424 which is less having less error, V_p/V_e is t is 0.9763 which is closer to 1. Figure 4.5 showing the predicted against experimental shear strength from various input combinations also confirmed that the ANN model with all the nine input variables with an architecture network 9:8:1 is good (showing value of slope of the trend line close to 1).

Table 4.5 Evaluation of input combinations with 8 hidden neurons for Model 1

Excluded variables	i:h:o *	ξ	V_p/V_e	MSE	RMSE	V_{avg}	V_σ	V_{range}	V_{min}	V_{max}	V_{cv}
None	9:8:1	0.9746	0.9763	0.0424	0.2058	0.0498	0.0328	0.5776	0.0429	-0.5347	-0.0068
a	8:8:1	0.9756	0.992	0.0073	0.0854	0.0168	0.0324	0.2394	-0.0965	-0.3359	-0.0125
a/d	8:8:1	0.9662	0.9851	0.0072	0.0851	0.0314	0.0504	0.9056	0.0639	-0.8418	-0.0186
a_g	8:8:1	0.9739	0.9987	0.0073	0.0857	0.0028	0.0044	0.2579	0.0068	-0.2512	-0.0016
b_w	8:8:1	0.962	1.0011	0.0074	0.0858	0.0023	0.0837	0.5607	0.08	-0.4807	-0.0402
RCA	8:8:1	0.9721	0.9956	0.0073	0.0856	0.0093	0.0468	0.5602	0.01	-0.5501	-0.0207
d	8:8:1	0.908	1.0932	0.008	0.0897	0.1957	0.0231	0.2295	0.095	-0.1346	-0.0423
f'_c	8:8:1	0.9613	0.9802	0.0072	0.0849	0.0417	0.0841	0.7743	-0.1196	-0.8939	-0.0332
p_s	8:8:1	0.9769	0.9839	0.0072	0.0851	0.0339	0.0392	0.2003	0.0416	-0.1587	-0.0128
p_v	8:8:1	0.8336	0.9864	0.0073	0.0852	0.0286	0.2392	1.372	0.1191	-1.2529	-0.1103

*Network Architecture



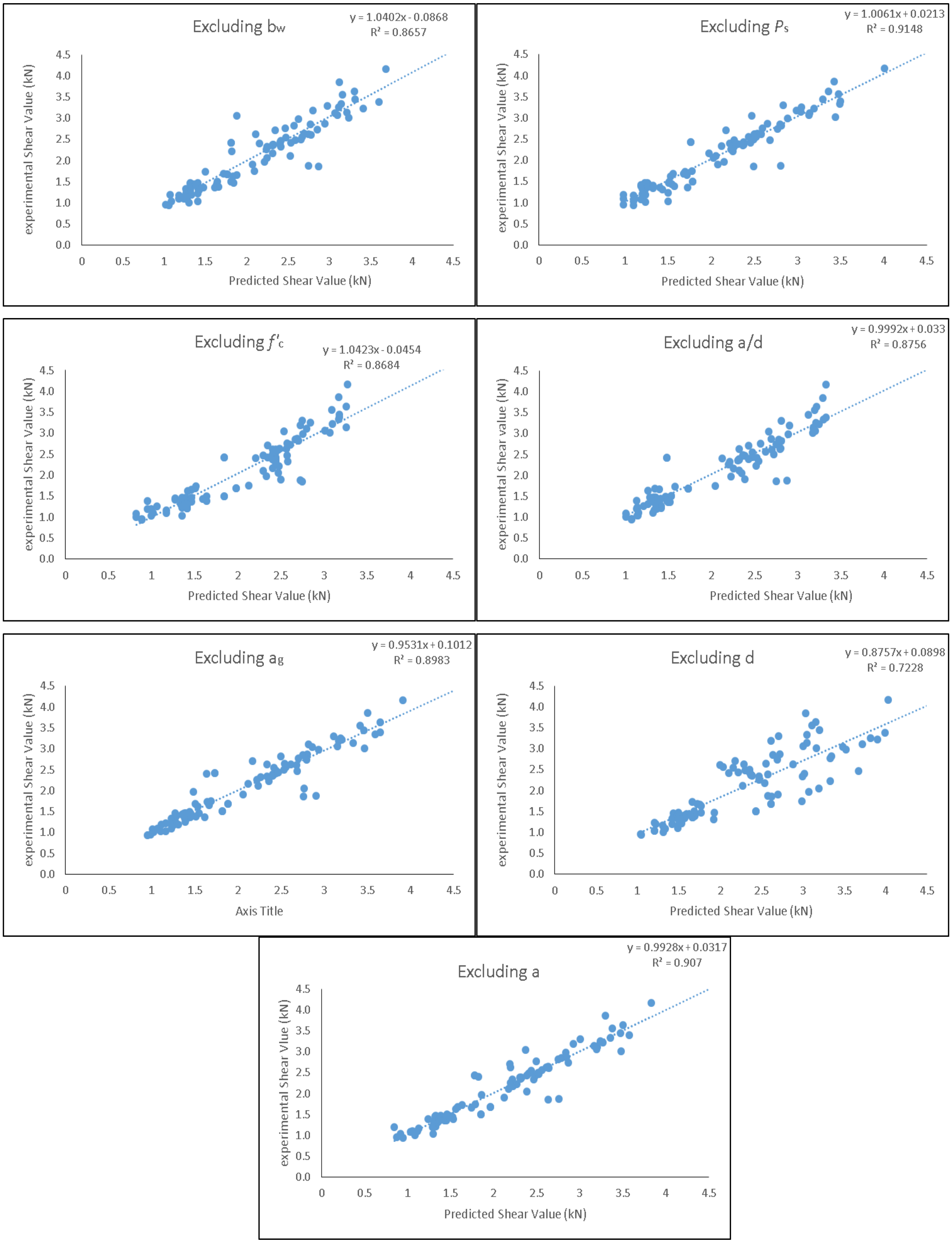
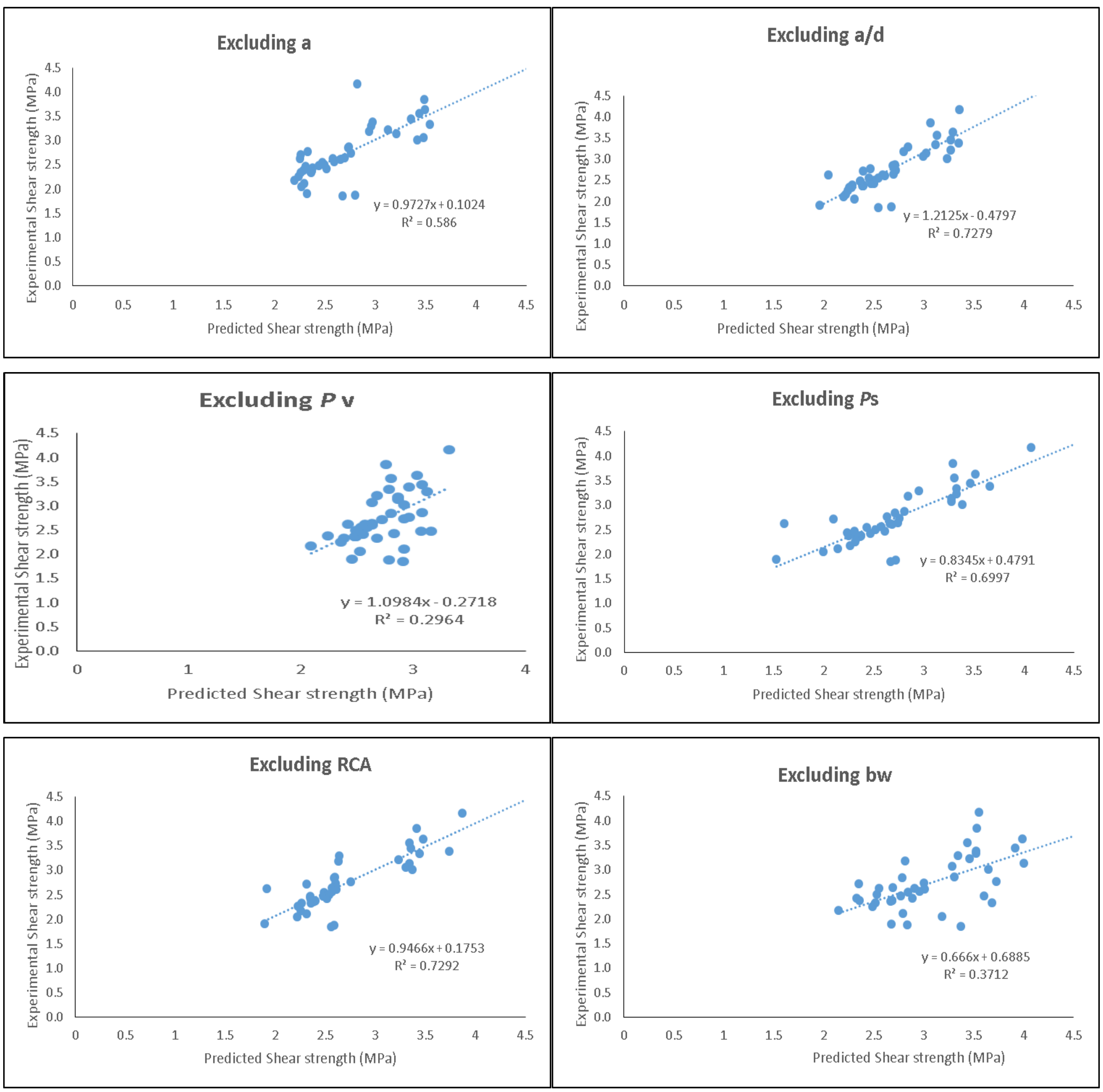


Figure 4.5 Predicted to experimental shear strength from various input combinations for **Model 1**

For Model 2, the network architecture of 9:6:1 is selected based on the optimum number of hidden neurons. Table 4.6 shows the results of excluding input variables or their combinations with the constant hidden neurons of six. The ANN model with all the nine input variables with an architecture network 9:6:1 shows good agreement. The agreement factor, ε, is 0.9605 which is closer to 1, MSE is 0.0239 which is less having less error, V_p/V_e is t is 1.0288 which is closer to 1. Figure 4.6 showing the predicted against experimental shear strength from various input combinations also confirmed that the ANN model with all the nine input variables with an architecture network 9:6:1 is good (showing value of slope of the trend line close to 1).



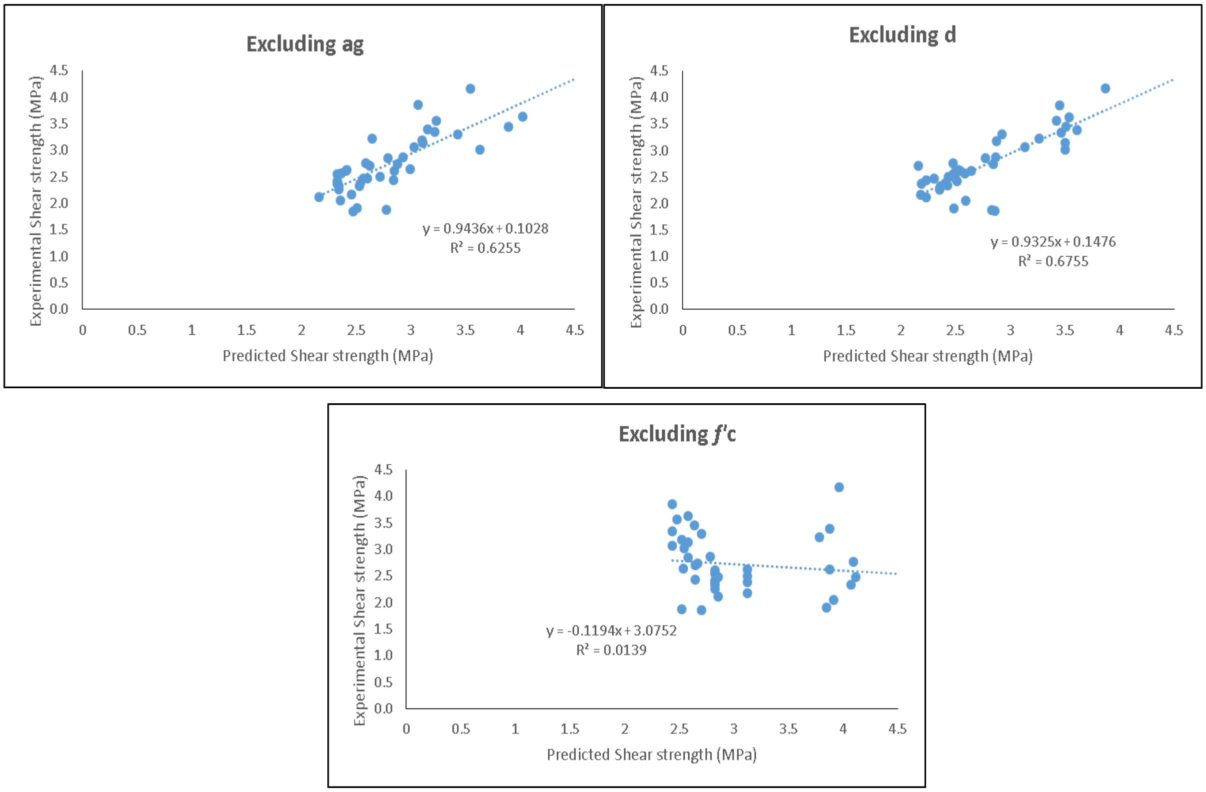


Figure 4.6 Predicted versus experimental shear strength from various input combinations for **Model 2**

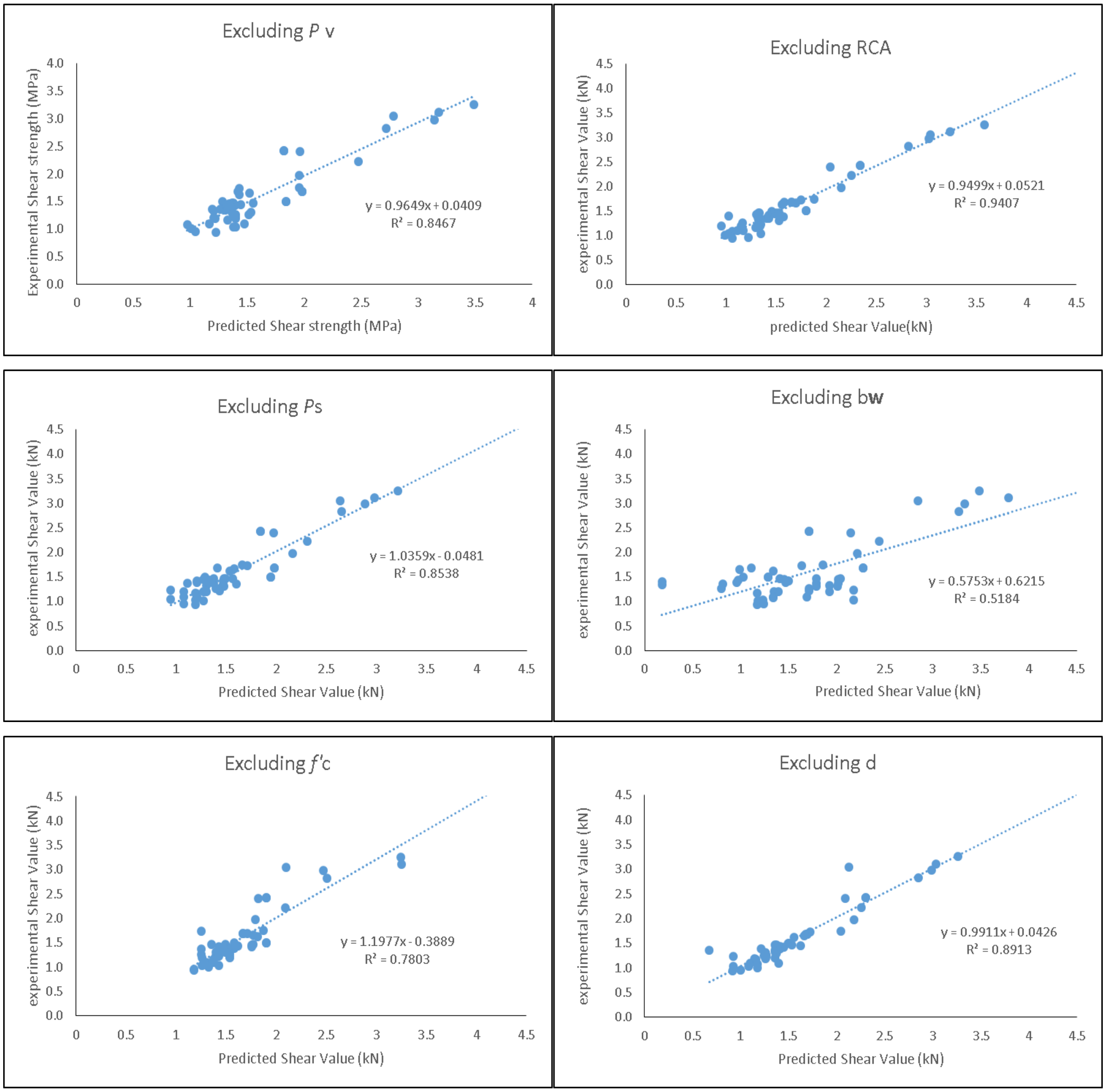
Table 4.6 Evaluation of input combinations with 6 hidden neurons Model 2

Excluded variables	i:h:o *	ξ	V_p/V_e	MSE	RMSE	V_{avg}	V_σ	V_{range}	V_{min}	V_{max}	V_{cv}
None	9:6:1	0.9605	1.0288	0.0239	0.1547	0.0783	0.1080	0.5405	0.3837	-0.1568	0.0089
a	8:6:1	0.9572	0.9893	0.1194	0.3455	0.0290	0.1153	0.9717	0.3456	-0.6261	-0.0407
a/d	8:6:1	0.9653	0.9703	0.0908	0.3013	0.0808	0.1604	0.9135	0.1071	-0.8064	-0.0547
a_g	8:6:1	0.9642	1.0197	0.1106	0.3326	0.0536	0.0876	0.4573	0.3125	-0.1448	-0.0354
b_w	8:6:1	0.9200	1.1212	0.3152	0.5614	0.3296	0.0461	0.4583	0.2958	-0.1626	-0.0367
RCA	8:6:1	0.9730	0.9883	0.0791	0.2813	0.0318	0.0530	0.3330	0.0349	-0.2982	-0.0174
d	8:6:1	0.9691	1.0142	0.0953	0.3088	0.0386	0.0642	0.6055	0.3076	-0.2979	-0.0261
f'_c	8:6:1	0.7895	1.0990	0.7031	0.8385	0.2690	0.0074	0.6335	0.5829	-0.0507	-0.0204
p_s	8:6:1	0.9684	0.9871	0.0950	0.3082	0.0350	0.0013	0.2309	-0.3296	-0.0987	-0.0204
p_v	8:6:1	0.9200	1.0014	0.2019	0.4494	0.0038	0.2729	1.0796	0.2359	-0.8436	-0.1005

*Network Architecture

For Model 3, the network architecture of 9:6:1 is selected based on the optimum number of hidden neurons. Table 4.7 shows the results of excluding input variables or their combinations with the constant hidden neurons of six. The ANN model with all the nine input variables with an architecture network 9:6:1 shows good agreement. The agreement factor, ε, is 0.9901 which is closer to 1, MSE is 0.0191which is less having less error, V_p/V_e is t is 1.0157 which is closer to 1.

Figure 4.7 (showing the predicted against experimental shear strength from various input combinations) also confirmed that the ANN model with all the nine input variables with a network architecture of 9:6:1 is good for prediction (showing value of slope of the trend line close to 1).



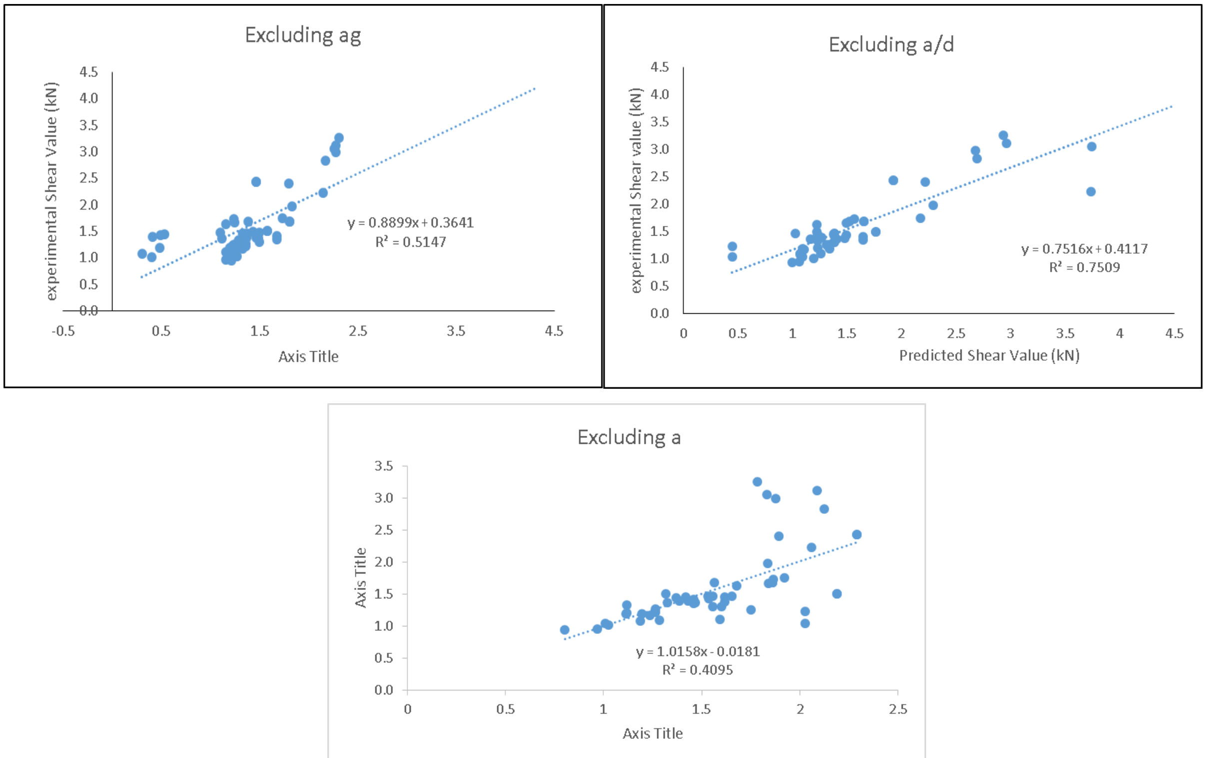


Figure 4.7 Predicted to experimental shear strength from various input combinations for **Model 3**

Table 4.7 Evaluation of input combinations with 6 hidden neurons for Model 3 (Beams without stirrups)

Excluded variables	i:h:o *	ξ	V_p/V_e	MSE	RMSE	V_{avg}	V_{σ}	V_{range}	V_{min}	V_{max}	V_{cv}
None	9:6:1	0.9901	1.0157	0.0191	0.1380	0.0010	0.0104	0.0010	0.0009	-0.0001	0.0008
a	8:6:1	0.8623	0.9956	0.1976	0.4445	0.0071	0.2161	0.8244	-0.1396	-0.9641	0.2312
a/d	8:6:1	0.9475	0.9877	0.1111	0.3333	0.0196	0.0893	0.9762	-0.4910	0.4852	0.4264
a_g	8:6:1	0.8938	0.8677	0.2096	0.4578	0.2114	0.1131	0.3122	-0.6378	-0.9501	0.3394
b_w	8:6:1	0.8667	1.0623	0.2655	0.5153	0.0995	0.1469	1.2935	-0.7601	0.5334	0.4304
RCA	8:6:1	0.9888	1.0185	0.0216	0.1469	0.0295	0.0123	0.3088	0.0143	0.3231	0.3663
d	8:6:1	0.9806	0.9821	0.0372	0.1929	0.0286	0.0277	0.2750	-0.2672	0.0079	0.3543
f'_c	8:6:1	0.9433	1.0381	0.0843	0.2904	0.0609	0.1533	0.2366	0.2366	0.0000	0.2596
p_s	8:6:1	0.9725	0.9944	0.0493	0.2221	0.0090	0.0631	0.0440	0.0038	-0.0402	0.3277
p_v	8:6:1	0.9715	1.0098	0.0519	0.2278	0.0157	0.0271	0.2003	0.0309	0.2313	0.3450

*Network Architecture

In general, all nine input parameters are found influential in affecting the shear strength of RAC beams. The results also show that shear span to depth ratio (a/d), concrete compression strength (f'_c), the web reinforcement ratio and width of the beam are the most influential input parameters.

4.3.3 Validating the performance ANN models

The performance of ANN models 1, 2 and 3 with a network architecture of 9:8:1, 9:6:1 and 9:6:1, respectively is validated by predicting shear strength of additional 12 beams without stirrups from the experimental studies conducted by Han et al. (2001) and Pellegrino and Faleschini (2013).

All three models are found to predict shear strength of RAC beams with good agreement based on V_p/V_e, degree of agreement (ξ), MSE and RMSE values as presented in Tables 4.8, 4.9 and 4.10. V_p/V_e values presented in Figures 4.8 to 4.10 also exhibits similar information. However, Model 2 shows slightly better agreement as it is particularly derived for RAC beams without stirrups with comparatively worse agreement is found for Model 3 which is applicable to only beams with shear reinforcement.

It is interesting to note that developed ANN models clearly simulating the effect of web/shear reinforcement in the prediction of shear strength of RAC beams. Model 1 is more robust than the other two as it is applicable to RAC beams with and without web reinforcement.

Table 4.8 Validation of Model 1 using data from Han et al. (2001) and (Pellegrino and Faleschini 2013)

Trial#	V_p/V_e	ξ	MSE	RMSE
1	1.05043	0.98583	0.08754	0.29586
2	0.99047	0.98124	0.08254	0.2873
3	1.07522	0.97736	0.0896	0.29934
4	0.99423	0.96699	0.08285	0.28784

Table 4.9 Validation of Model 2 using data from Han et al. (2001) and (Pellegrino and Faleschini 2013)

Trial#	V_p/V_e	ξ	MSE	RMSE
1	1.0024	0.9968	0.02439	0.1561
2	1.1232	0.8385	1.22259	1.1057
3	1.0376	0.9896	0.07719	0.2778
4	1.0942	0.9737	0.26014	0.5100

Table 4.10 Validation of Model 3 using data from Han et al. (2001) and (Pellegrino and Faleschini 2013)

Trial#	V_p/V_e	ξ	MSE	RMSE
1	0.97991	0.9635	0.16331	0.40412
2	1.03377	0.99189	0.17229	0.41508
3	1.00056	0.99965	0.16676	0.40836
4	1.05890	0.93359	0.17648	0.42010

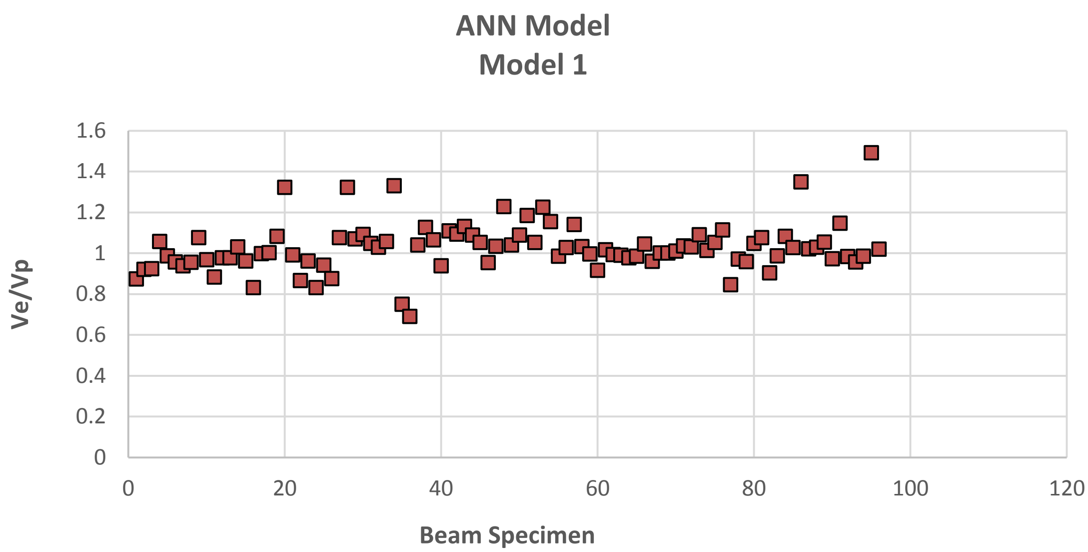


Figure 4.7 Experiment-to-predicted shear strength ratio for ANN **Model 1**: beams with and without stirrups

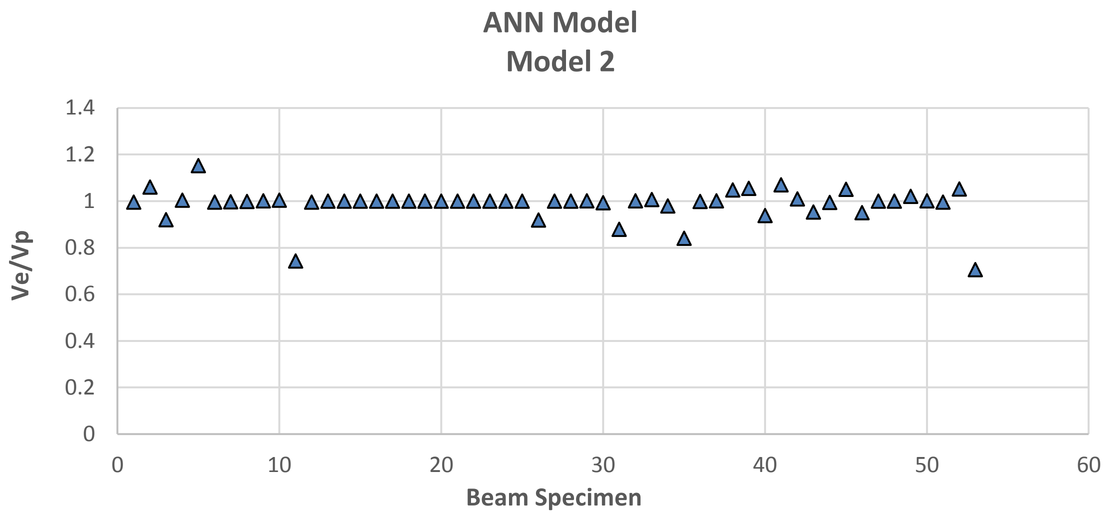


Figure 4.8 Experiment-to-predicted shear strength ratio for ANN **Model 2**: beams without stirrups

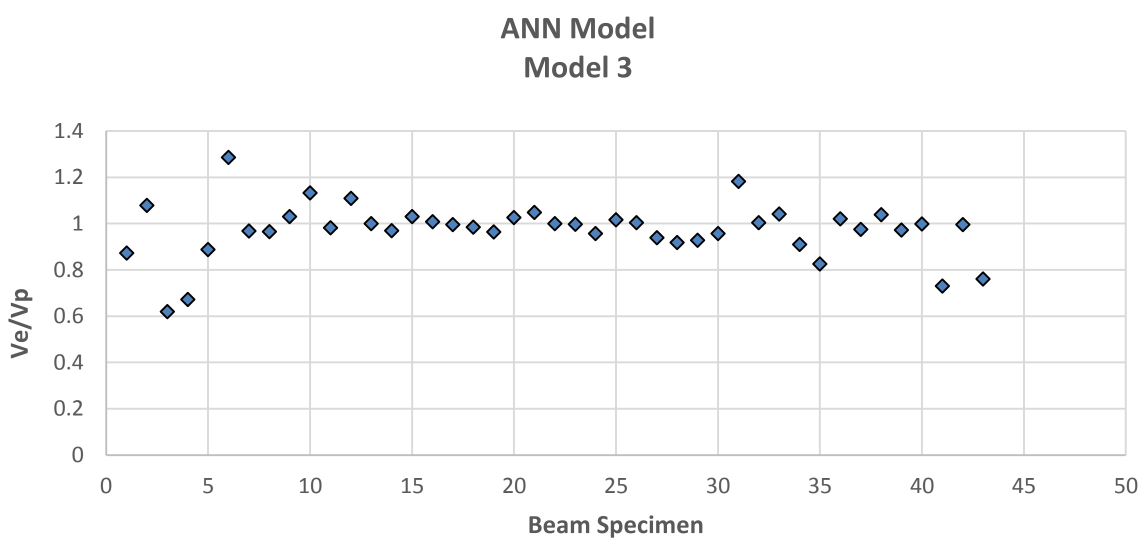


Figure 4.9 Experiment-to-predicted shear strength ratio for ANN **Model 3**: beams with stirrups

CHAPTER FIVE

CONCLUSIONS AND RECOMMANDATIONS

5.1 GENERAL

This research was intended to conduct a literature review on the use of recycled concrete aggregate (RCA) in concrete, to study the shear behavior of recycled aggregate concrete (RAC) beams with and without shear reinforcements and evaluate the performance of various Code based/other existing equations in predicting shear strength of RAC beams. In addition, three artificial neural network (ANN) models for predicting the shear strength RAC beams with and without shear reinforcements are also developed and their performance validated by using 108 experimental beams from available research studies.

5.2. Conclusions

The following conclusions are drawn from the study:

- Literature review suggests great potential for RCA to be use in concrete for sustainable construction. There are lack of research studies on the long term durability and structural performance of RAC. Especially, this study reveals lack of available research data on the shear resistance of RAC beams with and without stirrups. The performance of Code based/other existing equations for the prediction of RAC beams is also not fully understood.
- Most of the Codes and existing methods (ACI Code: ACI318-11, Canadian: CSA A23.3-04, Australian: AS 3600, European: Eurocode-2/CEB-FIP, Spanish: EHE-08 and New Zealand: NZS 3110 as well as equations proposed by Cladera and Mari, Zsutty, Gastebled and May) underestimate the shear capacity of RAC beams (with and with without stirrups) with ACI 318 showing the highest underestimation. The method proposed by Zsutty with an average value of 1.08 for experimental to predicted shear strength (V_e/V_p) is found to be more accurate than Codes/other existing methods for RAC beams without stirrup. For RAC beams with stirrups, Eurocode -2 and the method proposed by Cladera and Mari with an average value of V_e/V_p of 1.07 are found to be more accurate.

- However, over estimation by these Codes/existing methods for small percent of RAC beams needs to be carefully addressed when using such Codes or methods for shear strength prediction.
- All three ANN models are found to predict shear strength of RAC beams showing good agreement with experimental values. Developed ANN models are able to simulate the effect of web/shear reinforcement on the shear strength of RAC beams. Model 1 (developed for RAC beams with and without web reinforcement) is more robust than Model 2 and Model 3 developed for beams with and without web reinforcement, respectively. The developed ANN models can be used for the prediction of shear strength of RAC beams falling within the range of input parameters.

5.3 Recommendations for future studies

- More experimental investigations should be conducted on the long term durability and structural performance of RAC structural elements.
- The performance of Codes and other existing methods should be investigated with more experimental data having wide range of geometric and material parameters. New equations or modified Code based/existing equations should be derived based on wide range of experimental data.
- Developed ANN models should be updated with more experimental data.

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APPENDIX A

Spreadsheets of evaluation of shear capacity based on code provisions and compare them by experimental shear values for tested beams without stirrups

Table A.1 Shear strength comparison results for beams without stirrups based on CSA A23.3-04

Part I

Beam		Vc- CSA A23.3-04		Ve (KN)	CSA A23.3-04	
Paper	ID	simplified	General		simplified-Ve/Vp	General-Ve/Vp
Arezoumandi et al.,(2014)	CC-NS-4-1	111.5493641	48.24624583	121.2	1.086514486	0.96172747
	CC-NS-4-2	106.8133992	48.45158763	129.9	1.216139557	1.027688131
	CC-NS-6-1	111.5493641	57.0038152	143.2	1.283736587	1.067982545
	CC-NS-6-2	106.8133992	62.28956993	167	1.563474257	1.189756206
	CC-NS-8-1	111.5493641	69.06537666	173.5	1.555365208	1.176067874
	CC-NS-8-1	106.8133992	63.70693739	170.8	1.599050318	1.197880795
	RAC-NS-4-1	100.0399142	38.8116504	114.8	1.147541968	0.995601428
	RAC-NS-4-2	106.6571249	42.05556863	113	1.059469773	0.949659366
	RAC-NS-6-1	100.0399142	48.41313883	143.2	1.431428657	1.14009339
	RAC-NS-6-2	106.6571249	46.18669086	124.1	1.163541583	1.021921774
	RAC-NS-8-1	100.0399142	44.423788	131.4	1.313475737	1.106234722
	RAC-NS-8-2	106.6571249	52.21589627	140.3	1.31543017	1.094889141
. Knaack et al.,(2014)	S0-1a	30.04819527	27.4405345	31.1	1.035003923	1.143381421
	S0-1b	30.04819527	27.4405345	36.9	1.228027163	1.143381421
	S0-2a	37.32446449	34.08535009	40.4	1.08239999	1.00388974
	S0-2b	37.32446449	34.08535009	42.3	1.13330494	1.00388974
	S50-1a	34.74984226	31.73416029	44	1.266192798	1.047877377
	S50-1b	34.74984226	31.73416029	39.1	1.125184964	1.047877377
	S50-2a	33.36741935	30.47170765	43.7	1.309660767	1.073713949
	S50-2b	33.36741935	30.47170765	41.2	1.234737382	1.073713949
	S100-1a	33.86177841	30.92316495	36.4	1.074958307	1.064281
	S100-1b	33.86177841	30.92316495	38	1.122209222	1.064281
	S100-2a	31.44442814	28.71559864	39.9	1.268905251	1.112643273
	S100-2b	31.44442814	28.71559864	36.1	1.148057132	1.112643273

Part II

Beam		Vc- CSA A23.3-04		Ve (KN)	CSA A23.3-04	
Paper	ID	simplified	General		simplified-Ve/Vp	General-Ve/Vp
.Fathifazl et al.(2011)	EM-1.5	63.07605747	61.99553718	186.7	2.959918668	3.011507094
	EM-2	62.92424965	61.8463299	169.5	2.693715077	2.74066384
	EM-2.7	64.55660025	63.8554033	103.9	1.609440392	1.627113676
	CL-2.7	61.65948255	60.98975334	92.8	1.505040201	1.521567065
	EM-4	63.90090457	63.02879613	83.2	1.302015997	1.320031559
	EV-1.5	68.52651108	67.35262215	195.3	2.849991878	2.89966439
	EV-2	68.52651108	67.35262215	179	2.612127733	2.65765451
	CG-2.7	58.44820461	57.81335546	150	2.56637481	2.594556203
	EV-4	69.42263389	68.47516585	105.6	1.521117741	1.542164939
	EM-L	45.44958701	41.53700991	89.3	1.964814333	2.149889946
	EM-M	64.55660025	63.8554033	103.9	1.609440392	1.627113676
	CL-M	61.65948255	60.98975334	92.8	1.505040201	1.521567065
	EM-H	75.75796641	78.73433222	99.5	1.313393227	1.263743493
	EM-VH	88.9824005	98.36625233	104.6	1.175513353	1.063372829
	EV-L	49.37692292	45.12625684	122.6	2.482941276	2.716821837
	CG-M	58.44820461	57.81335546	150	2.56637481	2.594556203
	EV-H	75.75796641	78.73433222	111.7	1.474432397	1.418694956
	EV-VH	96.67144235	106.8661605	119.6	1.237180258	1.119156892

Table A.2 Shear strength comparison results for beams without stirrups based on ACI 318-11

Beam		Vc- ACI 318-11		Ve (KN)	ACI 318-11	
Paper	ID	Equation (11-3)	Equation (11-5)		Ve/Vp (11-3)	Ve/Vp (11-5)
Arezoumandi et al.,(2014)	CC-NS-4-1	122.14745	113.4064	121.2	0.9922434	1.0687229
	CC-NS-4-2	116.96153	108.9613	129.9	1.1106216	1.1921662
	CC-NS-6-1	122.14745	118.6178	143.2	1.1723536	1.2072385
	CC-NS-6-2	116.96153	114.1727	167	1.4278199	1.4626959
	CC-NS-8-1	122.14745	123.2807	173.5	1.4204144	1.4073577
	CC-NS-8-1	116.96153	118.8356	170.8	1.4603092	1.4372798
	RAC-NS-4-1	109.54451	102.6039	114.8	1.0479758	1.1188662
	RAC-NS-4-2	116.79041	108.8146	113	0.9675452	1.0384632
	RAC-NS-6-1	109.54451	107.8153	143.2	1.3072312	1.3281974
	RAC-NS-6-2	116.79041	114.0261	124.1	1.0625872	1.0883476
	RAC-NS-8-1	109.54451	112.4782	131.4	1.1995124	1.1682269
	RAC-NS-8-2	116.79041	118.6889	140.3	1.2012973	1.1820817
. Knaack et al.,(2014)	S0-1a	64.498062	62.141196	31.1	2.8946606	3.0044481
	S0-1b	64.342832	62.865284	36.9	2.6343261	2.6962417
	S0-2a	66.433004	63.569116	40.4	1.5639817	1.6344415
	S0-2b	63.451678	61.013695	42.3	1.4625303	1.52097
	S50-1a	65.57303	62.751146	44	1.2688143	1.3258722
	S50-1b	70.071392	66.918336	39.1	2.7871574	2.9184826
	S50-2a	70.071392	67.775479	43.7	2.5545375	2.6410732
	S50-2b	60.147061	58.181166	41.2	2.4938874	2.5781539
	S100-1a	71.239249	67.607906	36.4	1.4823289	1.5619475
	S100-1b	43.213702	42.138436	38	2.0664742	2.1192054
	S100-2a	66.433004	63.569116	39.9	1.5639817	1.6344415
	S100-2b	63.451678	61.013695	36.1	1.4625303	1.52097
.Fathifazl et al.(2011)	EM-1.5	81.912539	78.967168	186.7	1.2147102	1.2600173
	EM-2	102.33693	97.760441	169.5	1.022114	1.0699624
	EM-2.7	46.947833	45.33912	103.9	2.6114091	2.7040666
	CL-2.7	60.147061	58.181166	92.8	2.4938874	2.5781539
	EM-4	81.912539	78.967168	83.2	1.3636496	1.4145119
	EV-1.5	111.17994	105.34017	195.3	1.0757336	1.1353693
	EV-2	28.548205	26.272019	179	1.0893855	1.1837689
	CG-2.7	28.548205	26.272019	150	1.2925506	1.4045361
	EV-4	35.461246	32.197483	105.6	1.1392719	1.2547565
	EM-L	35.461246	32.197483	89.3	1.1928515	1.3137673
	EM-M	33.015148	30.100827	103.9	1.3327216	1.4617538
	CL-M	33.015148	30.100827	92.8	1.1843049	1.2989676
	EM-H	31.701735	28.975045	99.5	1.3784735	1.5081944
	EM-VH	31.701735	28.975045	104.6	1.2996134	1.4219133
	EV-L	32.171416	29.377628	122.6	1.1314392	1.2390381
	CG-M	32.171416	29.377628	150	1.1811728	1.2935013
	EV-H	29.874738	27.409048	111.7	1.3355765	1.4557237
	EV-VH	29.874738	27.409048	119.6	1.2083788	1.3170833

Table A.3 Shear strength comparison results for beams without stirrups based on AS 3600

Beam		As3600	Ve (KN)	As3600
Paper	ID			Ve/Vp
Arezoumandi et al.,(2014)	CC-NS-4-1	123.37076	121.2	0.9824046
	CC-NS-4-2	119.85367	129.9	1.0838217
	CC-NS-6-1	141.22426	143.2	1.0139901
	CC-NS-6-2	137.1982	167	1.2172171
	CC-NS-8-1	155.43742	173.5	1.1162049
	CC-NS-8-1	151.00616	170.8	1.1310797
	RAC-NS-4-1	114.7316	114.8	1.0005962
	RAC-NS-4-2	119.73674	113	0.9437371
	RAC-NS-6-1	131.33489	143.2	1.0903424
	RAC-NS-6-2	137.06435	124.1	0.9054142
	RAC-NS-8-1	144.55276	131.4	0.9090107
	RAC-NS-8-2	150.85883	140.3	0.9300085
. Knaack et al.,(2014)	S0-1a	64.049891	186.7	2.9149152
	S0-1b	73.201136	169.5	2.3155378
	S0-2a	76.94437	103.9	1.3503262
	S0-2b	74.624769	92.8	1.2435549
	S50-1a	87.565917	83.2	0.9501414
	S50-1b	67.688432	195.3	2.8852788
	S50-2a	77.483912	179	2.310157
	S50-2b	72.010728	150	2.083023
	S100-1a	92.540354	105.6	1.1411238
	S100-1b	58.087844	89.3	1.5373268
	S100-2a	76.94437	103.9	1.3503262
	S100-2b	74.624769	92.8	1.2435549
.Fathifazl et al.(2011)	EM-1.5	93.296676	99.5	1.0664903
	EM-2	104.45522	104.6	1.001386
	EM-2.7	61.387692	122.6	1.997143
	CL-2.7	72.010728	150	2.083023
	EM-4	93.296676	111.7	1.197256
	EV-1.5	110.3891	119.6	1.0834403
	EV-2	35.057864	31.1	0.8871048
	CG-2.7	35.057864	36.9	1.0525456
	EV-4	40.510622	40.4	0.9972693
	EM-L	40.510622	42.3	1.0441706
	EM-M	38.625586	44	1.1391413
	CL-M	38.625586	39.1	1.0122824
	EM-H	37.594265	43.7	1.1624113
	EM-VH	37.594265	41.2	1.0959118
	EV-L	37.964676	36.4	0.958786
	CG-M	37.964676	38	1.0009304
	EV-H	36.135632	39.9	1.1041733
	EV-VH	36.135632	36.1	0.9990139

Table A.4 Shear strength comparison results for beams without stirrups based on CEB-FIP

Beam		CEB-FIP	Ve (KN)	CEB-FIP
Paper	ID			Ve/Vp
Arezoumandi et al.,(2014)	CC-NS-4-1	111.18274	121.2	1.0900972
	CC-NS-4-2	108.01311	129.9	1.2026318
	CC-NS-6-1	129.99744	143.2	1.1015601
	CC-NS-6-2	126.29144	167	1.3223383
	CC-NS-8-1	143.13941	173.5	1.212105
	CC-NS-8-1	139.05875	170.8	1.2282578
	RAC-NS-4-1	103.39706	114.8	1.110283
	RAC-NS-4-2	107.90773	113	1.047191
	RAC-NS-6-1	120.89424	143.2	1.1845064
	RAC-NS-6-2	126.16823	124.1	0.9836074
	RAC-NS-8-1	133.11593	131.4	0.9871095
	RAC-NS-8-2	138.92308	140.3	1.0099114
. Knaack et al.,(2014)	S0-1a	71.370412	186.7	2.61593
	S0-1b	74.109073	169.5	2.2871693
	S0-2a	71.490921	103.9	1.4533314
	S0-2b	69.335722	92.8	1.3384154
	S50-1a	70.788881	83.2	1.1753258
	S50-1b	75.424816	195.3	2.5893335
	S50-2a	78.444969	179	2.2818544
	S50-2b	66.906952	150	2.2419195
	S100-1a	74.810249	105.6	1.4115713
	S100-1b	54.43914	89.3	1.6403639
	S100-2a	71.490921	103.9	1.4533314
	S100-2b	69.335722	92.8	1.3384154
.Fathifazl et al.(2011)	EM-1.5	86.208424	99.5	1.1541796
	EM-2	100.04441	104.6	1.0455357
	EM-2.7	57.531714	122.6	2.1309986
	CL-2.7	66.906952	150	2.2419195
	EM-4	86.208424	111.7	1.295697
	EV-1.5	105.72773	119.6	1.1312075
	EV-2	29.233619	31.1	1.0638436
	CG-2.7	29.233619	36.9	1.2622453
	EV-4	33.780498	40.4	1.1959563
	EM-L	33.780498	42.3	1.2522018
	EM-M	32.208627	44	1.3660936
	CL-M	32.208627	39.1	1.2139605
	EM-H	31.348642	43.7	1.3939998
	EM-VH	31.348642	41.2	1.3142515
	EV-L	31.657516	36.4	1.1498059
	CG-M	31.657516	38	1.2003469
	EV-H	30.132335	39.9	1.3241589
	EV-VH	30.132335	36.1	1.1980485

Table A.5 Shear strength comparison results for beams without stirrups based on Cladera & Mari

Beam		Cladera& Mari	Ve (KN)	Cladera& Mari Ve/Vp
Paper	**ID**			
Arezoumandi et al.,(2014)	CC-NS-4-1	109.51949	121.2	1.1066523
	CC-NS-4-2	107.63534	129.9	1.2068527
	CC-NS-6-1	138.46422	143.2	1.0342022
	CC-NS-6-2	136.08211	167	1.2272003
	CC-NS-8-1	159.98313	173.5	1.0844893
	CC-NS-8-1	157.23081	170.8	1.0863011
	RAC-NS-4-1	104.85132	114.8	1.0948837
	RAC-NS-4-2	107.57232	113	1.0504561
	RAC-NS-6-1	132.56231	143.2	1.0802467
	RAC-NS-6-2	136.00243	124.1	0.9124837
	RAC-NS-8-1	153.16399	131.4	0.857904
	RAC-NS-8-2	157.13875	140.3	0.8928415
. Knaack et al.,(2014)	S0-1a	52.944414	186.7	3.5263399
	S0-1b	64.78093	169.5	2.6165107
	S0-2a	68.937843	103.9	1.5071548
	S0-2b	67.683278	92.8	1.371092
	S50-1a	84.102658	83.2	0.9892672
	S50-1b	54.729034	195.3	3.5684898
	S50-2a	67.029103	179	2.6704818
	S50-2b	66.250613	150	2.26413
	S100-1a	86.937541	105.6	1.2146651
	S100-1b	55.171951	89.3	1.6185761
	S100-2a	68.937843	103.9	1.5071548
	S100-2b	67.683278	92.8	1.371092
.Fathifazl et al.(2011)	EM-1.5	86.220409	99.5	1.1540191
	EM-2	98.50215	104.6	1.0619058
	EM-2.7	57.031655	122.6	2.1496833
	CL-2.7	66.250613	150	2.26413
	EM-4	86.220409	111.7	1.2955169
	EV-1.5	101.8224	119.6	1.1745942
	EV-2	32.223574	31.1	0.9651319
	CG-2.7	32.223574	36.9	1.1451244
	EV-4	35.143401	40.4	1.1495757
	EM-L	35.143401	42.3	1.2036399
	EM-M	34.152892	44	1.2883243
	CL-M	34.152892	39.1	1.1448518
	EM-H	33.602795	43.7	1.3004871
	EM-VH	33.602795	41.2	1.2260885
	EV-L	33.801055	36.4	1.0768895
	CG-M	33.801055	38	1.1242253
	EV-H	32.814352	39.9	1.2159314
	EV-VH	32.814352	36.1	1.1001284

Table A.6 Shear strength comparison results for beams without stirrups based on Eurocode-2

Beam		Eurocode-2	Ve (KN)	Eurocode-2
Paper	ID			Ve/Vp
Arezoumandi et al.,(2014)	CC-NS-4-1	131.15751	121.2	0.9240798
	CC-NS-4-2	127.41843	129.9	1.0194758
	CC-NS-6-1	153.35241	143.2	0.9337969
	CC-NS-6-2	148.98059	167	1.1209514
	CC-NS-8-1	168.85542	173.5	1.0275062
	CC-NS-8-1	164.04164	170.8	1.041199
	RAC-NS-4-1	121.97307	114.8	0.9411913
	RAC-NS-4-2	127.29412	113	0.8877079
	RAC-NS-6-1	142.61375	143.2	1.0041107
	RAC-NS-6-2	148.83524	124.1	0.8338079
	RAC-NS-8-1	157.03115	131.4	0.8367766
	RAC-NS-8-2	163.8816	140.3	0.8561059
. Knaack et al.,(2014)	S0-1a	66.823721	186.7	2.7939181
	S0-1b	76.371281	169.5	2.2194207
	S0-2a	80.303199	103.9	1.2938463
	S0-2b	77.882341	92.8	1.191541
	S50-1a	91.371606	83.2	0.9105673
	S50-1b	70.619836	195.3	2.7655119
	S50-2a	80.839532	179	2.2142632
	S50-2b	75.154189	150	1.9958967
	S100-1a	96.562236	105.6	1.0935952
	S100-1b	61.926621	89.3	1.4420293
	S100-2a	80.303199	103.9	1.2938463
	S100-2b	77.882341	92.8	1.191541
.Fathifazl et al.(2011)	EM-1.5	98.549089	99.5	1.0096491
	EM-2	114.36569	104.6	0.91461
	EM-2.7	65.444542	122.6	1.8733419
	CL-2.7	75.154189	150	1.9958967
	EM-4	98.549089	111.7	1.1334453
	EV-1.5	120.86256	119.6	0.9895537
	EV-2	37.39456	31.1	0.8316718
	CG-2.7	37.39456	36.9	0.9867745
	EV-4	43.210759	40.4	0.9349523
	EM-L	43.210759	42.3	0.9789229
	EM-M	41.20008	44	1.0679591
	CL-M	41.20008	39.1	0.9490273
	EM-H	40.100019	43.7	1.089775
	EM-VH	40.100019	41.2	1.0274309
	EV-L	40.495119	36.4	0.8988738
	CG-M	40.495119	38	0.9383847
	EV-H	38.544164	39.9	1.0351762
	EV-VH	38.544164	36.1	0.936588

Table A.7 Shear strength comparison results for beams without stirrups based on EHE-08

Beam		EHE-08	Ve (KN)	EHE-08
Paper	ID			Ve/Vp
Arezoumandi et al.,(2014)	CC-NS-4-1	109.29792	121.2	1.1088957
	CC-NS-4-2	106.18202	129.9	1.2233709
	CC-NS-6-1	127.79367	143.2	1.1205563
	CC-NS-6-2	124.15049	167	1.3451417
	CC-NS-8-1	140.71285	173.5	1.2330075
	CC-NS-8-1	136.70137	170.8	1.2494389
	RAC-NS-4-1	101.64423	114.8	1.1294296
	RAC-NS-4-2	106.07843	113	1.0652495
	RAC-NS-6-1	118.84479	143.2	1.2049329
	RAC-NS-6-2	124.02937	124.1	1.0005695
	RAC-NS-8-1	130.8593	131.4	1.004132
	RAC-NS-8-2	136.568	140.3	1.027327
. Knaack et al.,(2014)	S0-1a	55.686434	186.7	3.3527017
	S0-1b	63.642734	169.5	2.6633048
	S0-2a	66.919332	103.9	1.5526156
	S0-2b	64.901951	92.8	1.4298492
	S50-1a	76.143005	83.2	1.0926808
	S50-1b	58.849864	195.3	3.3186143
	S50-2a	67.366277	179	2.6571158
	S50-2b	62.628491	150	2.3950761
	S100-1a	80.46853	105.6	1.3123143
	S100-1b	51.605517	89.3	1.7304351
	S100-2a	66.919332	103.9	1.5526156
	S100-2b	64.901951	92.8	1.4298492
.Fathifazl et al.(2011)	EM-1.5	82.124241	99.5	1.2115789
	EM-2	95.304741	104.6	1.097532
	EM-2.7	54.537118	122.6	2.2480102
	CL-2.7	62.628491	150	2.3950761
	EM-4	82.124241	111.7	1.3601343
	EV-1.5	100.7188	119.6	1.1874645
	EV-2	31.162134	31.1	0.9980061
	CG-2.7	31.162134	36.9	1.1841294
	EV-4	36.008966	40.4	1.1219428
	EM-L	36.008966	42.3	1.1747074
	EM-M	34.3334	44	1.2815509
	CL-M	34.3334	39.1	1.1388327
	EM-H	33.416683	43.7	1.30773
	EM-VH	33.416683	41.2	1.2329171
	EV-L	33.745932	36.4	1.0786485
	CG-M	33.745932	38	1.1260616
	EV-H	32.120137	39.9	1.2422114
	EV-VH	32.120137	36.1	1.1239055

Table A.8 Shear strength comparison results for beams without stirrups based on Gastebled & May

Beam		Gastebled & May	Ve (KN)	Gastebled &May Ve/Vp
Paper	ID			
Arezoumandi et al.,(2014)	CC-NS-4-1	114.8183	121.2	1.0555808
	CC-NS-4-2	111.38383	129.9	1.1662375
	CC-NS-6-1	121.35969	143.2	1.1799634
	CC-NS-6-2	117.72956	167	1.4185053
	CC-NS-8-1	125.14465	173.5	1.3863956
	CC-NS-8-1	121.4013	170.8	1.4069042
	RAC-NS-4-1	106.39114	114.8	1.0790372
	RAC-NS-4-2	111.26974	113	1.0155502
	RAC-NS-6-1	112.45242	143.2	1.2734274
	RAC-NS-6-2	117.60896	124.1	1.0551917
	RAC-NS-8-1	115.95958	131.4	1.1331534
	RAC-NS-8-2	121.27694	140.3	1.1568563
. Knaack et al.,(2014)	S0-1a	84.176448	186.7	2.2179601
	S0-1b	80.322739	169.5	2.1102368
	S0-2a	75.604459	103.9	1.3742576
	S0-2b	73.213112	92.8	1.2675325
	S50-1a	68.487656	83.2	1.2148175
	S50-1b	89.204436	195.3	2.189353
	S50-2a	85.264237	179	2.0993561
	S50-2b	70.522675	150	2.1269755
	S100-1a	72.578528	105.6	1.4549758
	S100-1b	61.653949	89.3	1.4484068
	S100-2a	75.604459	103.9	1.3742576
	S100-2b	73.213112	92.8	1.2675325
.Fathifazl et al.(2011)	EM-1.5	83.669404	99.5	1.1892041
	EM-2	92.599189	104.6	1.1295995
	EM-2.7	65.336634	122.6	1.8764358
	CL-2.7	70.522675	150	2.1269755
	EM-4	83.669404	111.7	1.3350161
	EV-1.5	98.13028	119.6	1.2187879
	EV-2	35.951696	31.1	0.8650496
	CG-2.7	35.951696	36.9	1.0263772
	EV-4	41.84485	40.4	0.9654713
	EM-L	41.84485	42.3	1.0108771
	EM-M	39.802788	44	1.1054502
	CL-M	39.802788	39.1	0.9823432
	EM-H	38.687649	43.7	1.1295595
	EM-VH	38.687649	41.2	1.0649393
	EV-L	39.08799	36.4	0.9312323
	CG-M	39.08799	38	0.9721656
	EV-H	37.113089	39.9	1.0750924
	EV-VH	37.113089	36.1	0.9727027

Table A.9 Shear strength comparison results for beams without stirrups based on NZS3101

Beam		NZS3101	Ve (KN)	NZS3101
Paper	ID			Ve/Vp
Arezoumandi et al.,(2014)	CC-NS-4-1	101.23206	121.2	1.1972491
	CC-NS-4-2	97.290764	129.9	1.335173
	CC-NS-6-1	146.57694	143.2	0.9769613
	CC-NS-6-2	140.35384	167	1.1898499
	CC-NS-8-1	146.57694	173.5	1.1836787
	CC-NS-8-1	140.35384	170.8	1.2169243
	RAC-NS-4-1	91.653829	114.8	1.2525391
	RAC-NS-4-2	97.160712	113	1.1630215
	RAC-NS-6-1	131.45341	143.2	1.0893593
	RAC-NS-6-2	140.14849	124.1	0.8854894
	RAC-NS-8-1	131.45341	131.4	0.9995937
	RAC-NS-8-2	140.14849	140.3	1.001081
. Knaack et al.,(2014)	S0-1a	65.788023	186.7	2.8379026
	S0-1b	77.211398	169.5	2.1952717
	S0-2a	68.89888	103.9	1.5080071
	S0-2b	66.001031	92.8	1.4060386
	S50-1a	78.687636	83.2	1.0573453
	S50-1b	71.47282	195.3	2.7325073
	S50-2a	67.264253	179	2.661146
	S50-2b	62.788944	150	2.3889556
	S100-1a	85.487098	105.6	1.2352741
	S100-1b	51.856442	89.3	1.7220618
	S100-2a	68.89888	103.9	1.5080071
	S100-2b	66.001031	92.8	1.4060386
.Fathifazl et al.(2011)	EM-1.5	98.295046	99.5	1.0122585
	EM-2	109.81962	104.6	0.952471
	EM-2.7	56.337399	122.6	2.1761743
	CL-2.7	62.788944	150	2.3889556
	EM-4	95.273968	111.7	1.1724084
	EV-1.5	118.73338	119.6	1.0072989
	EV-2	25.058466	31.1	1.2410975
	CG-2.7	25.058466	36.9	1.4725562
	EV-4	30.617934	40.4	1.3194881
	EM-L	30.617934	42.3	1.3815432
	EM-M	28.650782	44	1.5357347
	CL-M	28.650782	39.1	1.3647097
	EM-H	27.594535	43.7	1.5836469
	EM-VH	27.594535	41.2	1.4930492
	EV-L	27.972253	36.4	1.3012895
	CG-M	27.972253	38	1.3584891
	EV-H	26.125265	39.9	1.5272573
	EV-VH	26.125265	36.1	1.3818042

Table A.10 Shear strength comparison results for beams without stirrups based on Zsutty

Beam		Vc-Zsutty		Ve (KN)	Zsutty
Paper	ID	(a/d>2.5)	(a/d<2.5)		
Arezoumandi et al.,(2014)	CC-NS-4-1	143.34106		121.2	0.8455358
	CC-NS-4-2	139.25465		129.9	0.9328234
	CC-NS-6-1	167.5977		143.2	0.854427
	CC-NS-6-2	162.81977		167	1.0256739
	CC-NS-8-1	184.54083		173.5	0.9401713
	CC-NS-8-1	179.27989		170.8	0.9527003
	RAC-NS-4-1	133.30346		114.8	0.8611929
	RAC-NS-4-2	139.11879		113	0.8122555
	RAC-NS-6-1	155.86151		143.2	0.9187644
	RAC-NS-6-2	162.66093		124.1	0.7629368
	RAC-NS-8-1	171.61818		131.4	0.7656532
	RAC-NS-8-2	179.10498		140.3	0.7833395
. Knaack et al.,(2014)	S0-1a		144.12068	31.1	1.2954421
	S0-1b		112.23821	36.9	1.5101809
	S0-2a	87.193399		40.4	1.191604
	S0-2b	84.564826		42.3	1.097383
	S50-1a	86.086306		44	0.966472
	S50-1b		152.30787	39.1	1.2822712
	S50-2a		118.80493	43.7	1.5066715
	S50-2b	81.602593		41.2	1.8381769
	S100-1a	90.976689		36.4	1.1607369
	S100-1b	59.981368		38	1.4887956
	S100-2a	87.193399		39.9	1.191604
	S100-2b	84.564826		36.1	1.097383
.Fathifazl et al.(2011)	EM-1.5	110.02073		186.7	0.9043751
	EM-2	133.59059		169.5	0.7829893
	EM-2.7	63.388784		103.9	1.9340961
	CL-2.7	81.602593		92.8	1.8381769
	EM-4	110.02073		83.2	1.0152633
	EV-1.5	141.17959		195.3	0.8471479
	EV-2	32.169664		179	0.9667493
	CG-2.7	32.169664		150	1.1470434
	EV-4	37.173203		105.6	1.0868044
	EM-L	37.173203		89.3	1.1379165
	EM-M	35.443463		103.9	1.2414137
	CL-M	35.443463		92.8	1.1031653
	EM-H	34.497107		99.5	1.2667729
	EM-VH	34.497107		104.6	1.1943031
	EV-L	34.837002		122.6	1.044866
	CG-M	34.837002		150	1.0907942
	EV-H	33.158641		111.7	1.2033062
	EV-VH	33.158641		119.6	1.0887056

Table A.11 Shear strength comparison results for beams with stirrups based on CSA A23.3-04

Beam		Vc- CSA A23.3-04		Vs	Vu Simplified	Vu General	Ve(KN)	CSA A23.3-04	
Paper	ID	Simplified	General					Ve/Vp Simplified	Ve/Vp General
Gonzalez et al.,(2007)	V13CC	60.48631	60.24594	85.49145	145.9778	145.7374	190.29	1.303555	1.305705
	V13RC	62.69456	62.44542	85.49145	148.186	147.9369	233.59	1.57633	1.578984
	V17CC	61.6161	61.37124	65.37581	126.9919	126.7471	150.83	1.187713	1.190008
	V17RC	63.48756	63.23527	65.37581	128.8634	128.6111	176.99	1.37347	1.376164
	V24CC	61.67913	61.43403	46.30787	107.987	107.7419	127.97	1.18505	1.187746
	V24RC	61.73424	61.48891	46.30787	108.0421	107.7968	164.29	1.520611	1.524072
Gonzalez et al.,(2009)	V13CC	64.44437	64.18827	85.49145	149.9358	149.6797	150.07	1.000895	1.002607
	V13RC	63.45695	63.20478	85.49145	148.9484	148.6962	147.33	0.989134	0.990812
	V17CC	66.23597	65.97275	65.37581	131.6118	131.3486	199.79	1.518025	1.521067
	V17RC	65.74279	65.48153	65.37581	131.1186	130.8573	192.92	1.47134	1.474277
	V24CC	65.12666	64.86785	46.30787	111.4345	111.1757	220.08	1.974971	1.979569
	V24RC	64.82014	64.56255	46.30787	111.128	110.8704	202.36	1.820963	1.825194
Ajdukiewicz, (2007)	ORNm-b2	64.51171	61.84709	38.51348	103.0252	100.3606	118.5	1.150204	1.180743
	GNN/- b2	52.56053	50.38955	38.51348	91.07401	88.90303	108.5	1.191339	1.220431
	GRN/- b2	52.96641	50.77867	38.51348	91.47988	89.29214	116.5	1.273504	1.304706
	GRR/- b2	50.55287	48.46481	38.51348	89.06634	86.97829	113	1.268717	1.299175
	GRNm-b2	65.227	62.53284	38.51348	103.7405	101.0463	118.5	1.142274	1.17273
	GNNh-b2	81.65404	78.28138	38.51348	120.1675	116.7949	125	1.040215	1.070253
	GRNh-b2	79.75228	76.45817	38.51348	118.2658	114.9716	121	1.023119	1.052433
	GRRh-b2	76.60201	73.43801	38.51348	115.1155	111.9515	127.5	1.107583	1.138886
	BNNm-b2	65.88037	63.15923	38.51348	104.3938	101.6727	119	1.139914	1.170422
	BRNm-b2	65.227	62.53284	38.51348	103.7405	101.0463	119	1.147093	1.177678
	BRRm-b2	64.12325	61.47468	38.51348	102.6367	99.98815	118	1.149686	1.18014
	BNNh-b2	84.86915	81.36368	38.51348	123.3826	119.8772	131	1.061738	1.092785
	BRNh-b2	87.72303	84.09969	38.51348	126.2365	122.6132	130.5	1.033774	1.064323
	BRRh-b2	84.70076	81.20225	38.51348	123.2142	119.7157	128	1.038841	1.0692
Etxeberria,(2007)	HC-2	64.79669	64.81303	87.18435	151.981	151.9974	213	1.401491	1.40134
	HC-3	64.79669	64.81303	66.67039	131.4671	131.4834	177	1.346345	1.346177
	HR25-2	65.15901	65.17544	87.18435	152.3434	152.3598	186.5	1.224208	1.224076
	HR25-3	65.15901	65.17544	66.67039	131.8294	131.8458	169	1.28196	1.2818
	HR50-2	64.35454	64.37077	87.18435	151.5389	151.5551	220	1.451773	1.451617
	HR50-3	64.35454	64.37077	66.67039	131.0249	131.0412	176	1.343256	1.343089
	HR100-2	63.10482	63.12074	87.18435	150.2892	150.3051	189.5	1.260903	1.260769
	HR100-3	63.10482	63.12074	66.67039	129.7752	129.7911	163	1.256018	1.255864
Al-Zahraa et al., (2011)	B4	18.04516	16.41011	20.13705	38.18221	36.54716	44.5	1.165464	1.217605
	B5	19.30593	17.55664	20.13705	39.44298	37.69369	49.75	1.261315	1.31985
	B6	17.43957	15.85939	20.13705	37.57662	35.99644	42	1.117716	1.166782
	B7	19.74563	17.9565	35.50479	55.25042	53.46129	75	1.357456	1.402884
	B8	19.30593	17.55664	35.50479	54.81072	53.06144	61	1.112921	1.149611
	B9	17.43957	15.85939	35.50479	52.94436	51.36419	58	1.09549	1.129191
	B10	19.74563	17.9565	20.13705	39.88267	38.09355	37	0.927721	0.971293
	B11	17.68546	16.083	20.13705	37.8225	36.22005	47.25	1.249256	1.304526
	B12	17.43957	15.85939	20.13705	37.57662	35.99644	34.25	0.911471	0.951483

Table A.12 Shear strength comparison results for beams with stirrups based on NZS 3101

Beam		NZS3101			V_e	NZS3101
Paper	ID	V_c	V_s	V_u		V_e/V_p
Gonzalez et al.,(2007)	V13CC	74.37771	66.42692	140.8046	190.29	1.351447
	V13RC	77.09311	66.42692	143.52	233.59	1.627578
	V17CC	75.76697	50.79706	126.564	150.83	1.191729
	V17RC	78.06823	50.79706	128.8653	176.99	1.37345
	V24CC	75.84448	35.98125	111.8257	127.97	1.14437
	V24RC	75.91223	35.98125	111.8935	164.29	1.468271
Gonzalez et al.,(2009)	V13CC	79.24478	66.42692	145.6717	150.07	1.030193
	V13RC	78.03059	66.42692	144.4575	147.33	1.019885
	V17CC	81.44784	50.79706	132.2449	199.79	1.510758
	V17RC	80.8414	50.79706	131.6385	192.92	1.465529
	V24CC	80.08377	35.98125	116.065	220.08	1.896179
	V24RC	79.70686	35.98125	115.6881	202.36	1.749186
Ajdukiewicz, (2007)	ORNm-b2	76.35444	29.925	106.2794	118.5	1.114985
	GNN/-b2	62.20932	29.925	92.13432	108.5	1.177628
	GRN/-b2	62.68971	29.925	92.61471	116.5	1.257899
	GRR/-b2	59.8331	29.925	89.7581	113	1.258939
	GRNm-b2	77.20104	29.925	107.126	118.5	1.106174
	GNNh-b2	96.64368	29.925	126.5687	125	0.987606
	GRNh-b2	94.3928	29.925	124.3178	121	0.973312
	GRRh-b2	90.66422	29.925	120.5892	127.5	1.057308
	BNNm-b2	77.97435	29.925	107.8994	119	1.10288
	BRNm-b2	77.20104	29.925	107.126	119	1.110841
	BRRm-b2	75.89466	29.925	105.8197	118	1.115105
	BNNh-b2	100.449	29.925	130.374	131	1.004802
	BRNh-b2	103.8268	29.925	133.7518	130.5	0.975688
	BRRh-b2	100.2497	29.925	130.1747	128	0.983294
Etxeberria,(2007)	HC-2	80.01609	67.74231	147.7584	213	1.441542
	HC-3	80.01609	51.80294	131.819	177	1.34275
	HR25-2	80.46351	67.74231	148.2058	186.5	1.258385
	HR25-3	80.46351	51.80294	132.2664	169	1.277724
	HR50-2	79.47009	67.74231	147.2124	220	1.494439
	HR50-3	79.47009	51.80294	131.273	176	1.340717
	HR100-2	77.92683	67.74231	145.6691	189.5	1.300893
	HR100-3	77.92683	51.80294	129.7298	163	1.256458
Al-Zahraa et al., (2011)	B4	20.2594	15.6465	35.9059	44.5	1.239351
	B5	21.67487	15.6465	37.32137	49.75	1.333016
	B6	19.5795	15.6465	35.226	42	1.192301
	B7	22.16852	27.58725	49.75577	75	1.507363
	B8	21.67487	27.58725	49.26212	61	1.238274
	B9	19.5795	27.58725	47.16675	58	1.22968
	B10	22.16852	15.6465	37.81502	37	0.978447
	B11	19.85556	15.6465	35.50206	47.25	1.330909
	B12	19.5795	15.6465	35.226	34.25	0.972293

Table A.13 Shear strength comparison results for beams with stirrups based on ACI 318-11

Beam		Vc- ACI 318-11			Vu			ACI 318-11	
Paper	ID	Equ. (11-3)	Equ. (11-5)	Vs	Simplified	Vu General	Ve(KN)	Ve/Vp Equ. (11-3)	Ve/Vp Equ. (11-5)
Gonzalez et al.,(2007)	V13CC	61.98142	62.50813	66.42692	128.4083	128.9351	190.29	1.481913	1.475859
	V13RC	64.24426	64.4477	66.42692	130.6712	130.8746	233.59	1.787617	1.784838
	V17CC	63.13914	63.50046	50.79706	113.9362	114.2975	150.83	1.323811	1.319626
	V17RC	65.05686	65.14422	50.79706	115.8539	115.9413	176.99	1.5277	1.526549
	V24CC	63.20373	63.55582	35.98125	99.18498	99.53707	127.97	1.290216	1.285652
	V24RC	63.2602	63.60422	35.98125	99.24145	99.58547	164.29	1.655458	1.649739
Gonzalez et al.,(2009)	V13CC	66.03732	65.98461	66.42692	132.4642	132.4115	150.07	1.13291	1.13336
	V13RC	65.02549	65.11733	66.42692	131.4524	131.5443	147.33	1.120786	1.120003
	V17CC	67.8732	67.55823	50.79706	118.6703	118.3553	199.79	1.683573	1.688053
	V17RC	67.36783	67.12505	50.79706	118.1649	117.9221	192.92	1.632634	1.635995
	V24CC	66.73647	66.58388	35.98125	102.7177	102.5651	220.08	2.142571	2.145758
	V24RC	66.42238	66.31466	35.98125	102.4036	102.2959	202.36	1.976102	1.978183
Ajdukiewicz, (2007)	ORNm-b2	63.6287	58.84763	29.925	93.5537	88.77263	118.5	1.266652	1.334871
	GNN/- b2	51.8411	48.74398	29.925	81.7661	78.66898	108.5	1.326956	1.379197
	GRN/- b2	52.24143	49.08712	29.925	82.16643	79.01212	116.5	1.417854	1.474457
	GRR/- b2	49.86092	47.04668	29.925	79.78592	76.97168	113	1.41629	1.468072
	GRNm-b2	64.3342	59.45235	29.925	94.2592	89.37735	118.5	1.257172	1.325839
	GNNh-b2	80.5364	73.33995	29.925	110.4614	103.2649	125	1.131617	1.210479
	GRNh-b2	78.66066	71.73218	29.925	108.5857	101.6572	121	1.114328	1.190275
	GRRh-b2	75.55351	69.0689	29.925	105.4785	98.9939	127.5	1.208777	1.287958
	BNNm-b2	64.97863	60.00472	29.925	94.90363	89.92972	119	1.253904	1.323256
	BRNm-b2	64.3342	59.45235	29.925	94.2592	89.37735	119	1.262476	1.331434
	BRRm-b2	63.24555	58.51922	29.925	93.17055	88.44422	118	1.266495	1.334174
	BNNh-b2	83.70749	76.05803	29.925	113.6325	105.983	131	1.152839	1.236047
	BRNh-b2	86.52232	78.47073	29.925	116.4473	108.3957	130.5	1.120678	1.203922
	BRRh-b2	83.54141	75.91567	29.925	113.4664	105.8407	128	1.128087	1.209365
Etxeberria,(2007)	HC-2	66.68007	66.52868	67.74231	134.4224	134.271	213	1.584558	1.586344
	HC-3	66.68007	66.52868	51.80294	118.483	118.3316	177	1.493885	1.495796
	HR25-2	67.05292	66.84827	67.74231	134.7952	134.5906	186.5	1.38358	1.385684
	HR25-3	67.05292	66.84827	51.80294	118.8559	118.6512	169	1.42189	1.424343
	HR50-2	66.22508	66.13869	67.74231	133.9674	133.881	220	1.642191	1.64325
	HR50-3	66.22508	66.13869	51.80294	118.028	117.9416	176	1.491171	1.492264
	HR100-2	64.93903	65.03636	67.74231	132.6813	132.7787	189.5	1.428234	1.427187
	HR100-3	64.93903	65.03636	51.80294	116.742	116.8393	163	1.396242	1.395078
Al-Zahraa et al., (2011)	B4	16.88283	17.40243	15.6465	32.52933	33.04893	44.5	1.367996	1.346488
	B5	18.06239	18.41348	15.6465	33.70889	34.05998	49.75	1.475872	1.460659
	B6	16.31625	16.91679	15.6465	31.96275	32.56329	42	1.31403	1.289796
	B7	18.47377	18.76608	27.58725	46.06102	46.35333	75	1.628275	1.618007
	B8	18.06239	18.41348	27.58725	45.64964	46.00073	61	1.336265	1.326066
	B9	16.31625	16.91679	27.58725	43.9035	44.50404	58	1.321079	1.303253
	B10	18.47377	18.1798	15.6465	34.12027	33.8263	37	1.0844	1.093823
	B11	16.5463	16.52768	15.6465	32.1928	32.17418	47.25	1.46772	1.468569
	B12	16.31625	16.3305	15.6465	31.96275	31.977	34.25	1.07156	1.071082

Table A.14 Shear strength comparison results for beams with stirrups based on Cladera & Mari

Beam		Cladera & Mari			V_e	Cladera & Mari
Paper	ID	V_c	V_s	V_u		V_e/V_p
Gonzalez et al.,(2007)	V13CC	99.8902568	89.6763462	189.5666	190.29	1.003816
	V13RC	105.931902	89.6763462	195.6082	233.59	1.194173
	V17CC	90.9264702	68.5760294	159.5025	150.83	0.945628
	V17RC	97.0603324	68.5760294	165.6364	176.99	1.068546
	V24CC	86.1141999	48.5746875	134.6889	127.97	0.950116
	V24RC	93.6262327	48.5746875	142.2009	164.29	1.155337
Gonzalez et al.,(2009)	V13CC	104.998782	89.6763462	194.6751	150.07	0.770874
	V13RC	101.472877	89.6763462	191.1492	147.33	0.770759
	V17CC	102.788741	68.5760294	171.3648	199.79	1.165876
	V17RC	101.293539	68.5760294	169.8696	192.92	1.135695
	V24CC	92.8078869	48.5746875	141.3826	220.08	1.556627
	V24RC	92.0656865	48.5746875	140.6404	202.36	1.438847
Ajdukiewicz, (2007)	ORNm-b2	61.4003296	40.39875	101.7991	118.5	1.164058
	GNN/- b2	54.9308493	40.39875	95.3296	108.5	1.138156
	GRN/- b2	56.4223746	40.39875	96.82112	116.5	1.20325
	GRR/- b2	54.819318	40.39875	95.21807	113	1.18675
	GRNm-b2	61.671746	40.39875	102.0705	118.5	1.160962
	GNNh-b2	68.6810492	40.39875	109.0798	125	1.14595
	GRNh-b2	67.3030697	40.39875	107.7018	121	1.123472
	GRRh-b2	67.3920447	40.39875	107.7908	127.5	1.182847
	BNNm-b2	62.0050761	40.39875	102.4038	119	1.162066
	BRNm-b2	61.7583637	40.39875	102.1571	119	1.164872
	BRRm-b2	61.1659005	40.39875	101.5647	118	1.161822
	BNNh-b2	70.8488611	40.39875	111.2476	131	1.177553
	BRNh-b2	71.7009315	40.39875	112.0997	130.5	1.164142
	BRRh-b2	70.2480183	40.39875	110.6468	128	1.156835
Etxeberria,(2007)	HC-2	103.296105	91.4521154	194.7482	213	1.09372
	HC-3	97.113998	69.9339706	167.048	177	1.059576
	HR25-2	99.0419118	91.4521154	190.494	186.5	0.979033
	HR25-3	95.8418021	69.9339706	165.7758	169	1.019449
	HR50-2	104.12992	91.4521154	195.582	220	1.124848
	HR50-3	96.6656546	69.9339706	166.5996	176	1.056425
	HR100-2	98.302458	91.4521154	189.7546	189.5	0.998658
	HR100-3	93.4882809	69.9339706	163.4223	163	0.997416
Al-Zahraa et al., (2011)	B4	23.3817502	21.122775	44.50453	44.5	0.999898
	B5	24.9317845	21.122775	46.05456	49.75	1.08024
	B6	22.6243858	21.122775	43.74716	42	0.960062
	B7	28.8461697	37.2427875	66.08896	75	1.134834
	B8	26.6849021	37.2427875	63.92769	61	0.954203
	B9	25.1943427	37.2427875	62.43713	58	0.928934
	B10	22.7930079	21.122775	43.91578	37	0.842522
	B11	23.6624575	21.122775	44.78523	47.25	1.055035
	B12	21.1371888	21.122775	42.25996	34.25	0.81046

Table A.15 Shear strength comparison results for beams with stirrups based on Eurocode-2

Beam		Eurocode-2			V_e	Eurocode-2
Paper	ID	V_c	V_s	V_u		V_e/V_p
Gonzalez et al.,(2007)	V13CC	93.74539	80.70871	174.4541	190.29	1.090774
	V13RC	96.01338	80.70871	176.7221	233.59	1.321793
	V17CC	94.90913	61.71843	156.6276	150.83	0.962985
	V17RC	96.82131	61.71843	158.5397	176.99	1.116376
	V24CC	94.97385	43.71722	138.6911	127.97	0.922698
	V24RC	95.0304	43.71722	138.7476	164.29	1.184092
Gonzalez et al.,(2009)	V13CC	97.79166	80.70871	178.5004	150.07	0.840727
	V13RC	96.79018	80.70871	177.4989	147.33	0.830033
	V17CC	99.59582	61.71843	161.3142	199.79	1.238514
	V17RC	99.10082	61.71843	160.8192	192.92	1.199608
	V24CC	98.48068	43.71722	142.1979	220.08	1.547702
	V24RC	98.17144	43.71722	141.8887	202.36	1.426189
Ajdukiewicz, (2007)	ORNm-b2	76.14952	36.35888	112.5084	118.5	1.053255
	GNN/- b2	66.42751	36.35888	102.7864	108.5	1.055587
	GRN/- b2	66.76905	36.35888	103.1279	116.5	1.129665
	GRR/- b2	64.72499	36.35888	101.0839	113	1.117884
	GRNm-b2	76.71137	36.35888	113.0702	118.5	1.048021
	GNNh-b2	89.10319	36.35888	125.4621	125	0.996317
	GRNh-b2	87.71426	36.35888	124.0731	121	0.975231
	GRRh-b2	85.38893	36.35888	121.7478	127.5	1.047247
	BNNm-b2	77.22279	36.35888	113.5817	119	1.047704
	BRNm-b2	76.71137	36.35888	113.0702	119	1.052443
	BRRm-b2	75.84352	36.35888	112.2024	118	1.051671
	BNNh-b2	91.42705	36.35888	127.7859	131	1.025152
	BRNh-b2	93.46534	36.35888	129.8242	130.5	1.005205
	BRRh-b2	91.30607	36.35888	127.6649	128	1.002624
Etxeberria,(2007)	HC-2	97.97086	82.3069	180.2778	213	1.18151
	HC-3	97.97086	62.94057	160.9114	177	1.099984
	HR25-2	98.33573	82.3069	180.6426	186.5	1.032425
	HR25-3	98.33573	62.94057	161.2763	169	1.047891
	HR50-2	97.52467	82.3069	179.8316	220	1.223367
	HR50-3	97.52467	62.94057	160.4652	176	1.096811
	HR100-2	96.25798	82.3069	178.5649	189.5	1.061239
	HR100-3	96.25798	62.94057	159.1985	163	1.023879
Al-Zahraa et al., (2011)	B4	25.63737	19.0105	44.64787	44.5	0.996688
	B5	26.81802	19.0105	45.82852	49.75	1.085569
	B6	25.06052	19.0105	44.07102	42	0.953007
	B7	27.22368	33.51851	60.74219	75	1.234727
	B8	26.81802	33.51851	60.33653	61	1.010996
	B9	25.06052	33.51851	58.57903	58	0.990115
	B10	27.22368	19.0105	46.23418	37	0.800274
	B11	25.29553	19.0105	44.30603	47.25	1.066446
	B12	25.06052	19.0105	44.07102	34.25	0.777155

Table A.16 Shear strength comparison results for beams without stirrups based on AS 3600

Beam		As 3600			V_e	As 3600
Paper	ID	V_c	V_s	V_u		V_e/V_p
Gonzalez et al.,(2007)	V13CC	89.84712	66.42692	156.274	190.29	1.217669
	V13RC	92.0208	66.42692	158.4477	233.59	1.47424
	V17CC	90.96247	50.79706	141.7595	150.83	1.063985
	V17RC	92.79513	50.79706	143.5922	176.99	1.232588
	V24CC	91.02449	35.98125	127.0057	127.97	1.007592
	V24RC	91.0787	35.98125	127.0599	164.29	1.293012
Gonzalez et al.,(2009)	V13CC	93.72514	66.42692	160.1521	150.07	0.937047
	V13RC	92.7653	66.42692	159.1922	147.33	0.925485
	V17CC	95.45427	50.79706	146.2513	199.79	1.366073
	V17RC	94.97986	50.79706	145.7769	192.92	1.323392
	V24CC	94.3855	35.98125	130.3668	220.08	1.688161
	V24RC	94.08912	35.98125	130.0704	202.36	1.555773
Ajdukiewicz, (2007)	ORNm-b2	72.67784	29.925	102.6028	118.5	1.154939
	GNN/- b2	63.39906	29.925	93.32406	108.5	1.162616
	GRN/- b2	63.72502	29.925	93.65002	116.5	1.243993
	GRR/- b2	61.77416	29.925	91.69916	113	1.232291
	GRNm-b2	73.21407	29.925	103.1391	118.5	1.148934
	GNNh-b2	85.04095	29.925	114.966	125	1.087278
	GRNh-b2	83.71534	29.925	113.6403	121	1.064763
	GRRh-b2	81.49602	29.925	111.421	127.5	1.144308
	BNNm-b2	73.70218	29.925	103.6272	119	1.148347
	BRNm-b2	73.21407	29.925	103.1391	119	1.153782
	BRRm-b2	72.38579	29.925	102.3108	118	1.153349
	BNNh-b2	87.25886	29.925	117.1839	131	1.117901
	BRNh-b2	89.20422	29.925	119.1292	130.5	1.095449
	BRRh-b2	87.1434	29.925	117.0684	128	1.093378
Etxeberria,(2007)	HC-2	93.44523	67.74231	161.1875	213	1.321442
	HC-3	93.44523	51.80294	145.2482	177	1.218604
	HR25-2	93.79325	67.74231	161.5356	186.5	1.154545
	HR25-3	93.79325	51.80294	145.5962	169	1.160745
	HR50-2	93.01966	67.74231	160.762	220	1.368483
	HR50-3	93.01966	51.80294	144.8226	176	1.21528
	HR100-2	91.81147	67.74231	159.5538	189.5	1.187687
	HR100-3	91.81147	51.80294	143.6144	163	1.134984
Al-Zahraa et al., (2011)	B4	23.73672	15.6465	39.38322	44.5	1.129923
	B5	24.82985	15.6465	40.47635	49.75	1.229113
	B6	23.20264	15.6465	38.84914	42	1.081105
	B7	25.20543	27.58725	52.79268	75	1.420651
	B8	24.82985	27.58725	52.4171	61	1.163742
	B9	23.20264	27.58725	50.78989	58	1.141959
	B10	25.20543	15.6465	40.85193	37	0.90571
	B11	23.42023	15.6465	39.06673	47.25	1.209469
	B12	23.20264	15.6465	38.84914	34.25	0.881615

Table A.17 Shear strength comparison results for beams without stirrups based on EHE-08

Beam		EHE-08			V_e	EHE-08
Paper	ID	V_c	V_s	V_u		V_e/V_p
Gonzalez et al.,(2007)	V13CC	78.12115	64.56697	142.6881	190.29	1.333608
	V13RC	80.01115	64.56697	144.5781	233.59	1.615666
	V17CC	79.09094	49.37474	128.4657	150.83	1.174088
	V17RC	80.68442	49.37474	130.0592	176.99	1.360842
	V24CC	79.14487	34.97378	114.1186	127.97	1.121377
	V24RC	79.192	34.97378	114.1658	164.29	1.439048
Gonzalez et al.,(2009)	V13CC	81.49305	64.56697	146.06	150.07	1.027454
	V13RC	80.65848	64.56697	145.2255	147.33	1.014492
	V17CC	82.99652	49.37474	132.3713	199.79	1.509316
	V17RC	82.58402	49.37474	131.9588	192.92	1.461972
	V24CC	82.06723	34.97378	117.041	220.08	1.880367
	V24RC	81.80953	34.97378	116.7833	202.36	1.732782
Ajdukiewicz, (2007)	ORNm-b2	63.45793	29.0871	92.54503	118.5	1.280458
	GNN/- b2	55.35626	29.0871	84.44336	108.5	1.284885
	GRN/- b2	55.64087	29.0871	84.72797	116.5	1.374989
	GRR/- b2	53.93749	29.0871	83.02459	113	1.361043
	GRNm-b2	63.92614	29.0871	93.01324	118.5	1.274012
	GNNh-b2	74.25266	29.0871	103.3398	125	1.209602
	GRNh-b2	73.09522	29.0871	102.1823	121	1.184158
	GRRh-b2	71.15744	29.0871	100.2445	127.5	1.27189
	BNNm-b2	64.35233	29.0871	93.43943	119	1.273552
	BRNm-b2	63.92614	29.0871	93.01324	119	1.279388
	BRRm-b2	63.20293	29.0871	92.29003	118	1.278578
	BNNh-b2	76.18921	29.0871	105.2763	131	1.244345
	BRNh-b2	77.88778	29.0871	106.9749	130.5	1.219913
	BRRh-b2	76.0884	29.0871	105.1755	128	1.217014
Etxeberria,(2007)	HC-2	81.64238	65.84552	147.4879	213	1.444186
	HC-3	81.64238	50.35246	131.9948	177	1.340962
	HR25-2	81.94644	65.84552	147.792	186.5	1.261909
	HR25-3	81.94644	50.35246	132.2989	169	1.27741
	HR50-2	81.27056	65.84552	147.1161	220	1.495418
	HR50-3	81.27056	50.35246	131.623	176	1.337152
	HR100-2	80.21498	65.84552	146.0605	189.5	1.297408
	HR100-3	80.21498	50.35246	130.5674	163	1.248397
Al-Zahraa et al., (2011)	B4	21.36447	15.2084	36.57287	44.5	1.216749
	B5	22.34835	15.2084	37.55675	49.75	1.324662
	B6	20.88377	15.2084	36.09217	42	1.163687
	B7	22.6864	26.81481	49.50121	75	1.515115
	B8	22.34835	26.81481	49.16316	61	1.240766
	B9	20.88377	26.81481	47.69858	58	1.215969
	B10	22.6864	15.2084	37.8948	37	0.976387
	B11	21.07961	15.2084	36.28801	47.25	1.302083
	B12	20.88377	15.2084	36.09217	34.25	0.948959

Table A.18 Shear strength comparison results for beams without stirrups based on CEB-FIP

Beam		CEB-FIP			V_e	CEB-FIP
Paper	ID	V_c	V_s	V_u		V_e/ V_p
Gonzalez et al.,(2007)	V13CC	76.98331	59.78423	136.7675	190.29	1.391339
	V13RC	78.84577	59.78423	138.63	233.59	1.684989
	V17CC	77.93897	45.71735	123.6563	150.83	1.219752
	V17RC	79.50924	45.71735	125.2266	176.99	1.413358
	V24CC	77.99211	32.38313	110.3752	127.97	1.159409
	V24RC	78.03856	32.38313	110.4217	164.29	1.487842
Gonzalez et al.,(2009)	V13CC	80.30609	59.78423	140.0903	150.07	1.071237
	V13RC	79.48368	59.78423	139.2679	147.33	1.057889
	V17CC	81.78766	45.71735	127.505	199.79	1.566919
	V17RC	81.38117	45.71735	127.0985	192.92	1.517878
	V24CC	80.87191	32.38313	113.255	220.08	1.943225
	V24RC	80.61796	32.38313	113.0011	202.36	1.790779
Ajdukiewicz, (2007)	ORNm-b2	63.17838	26.9325	90.11088	118.5	1.315047
	GNN/- b2	55.11239	26.9325	82.04489	108.5	1.322447
	GRN/- b2	55.39575	26.9325	82.32825	116.5	1.415067
	GRR/- b2	53.69988	26.9325	80.63238	113	1.401422
	GRNm-b2	63.64452	26.9325	90.57702	118.5	1.308279
	GNNh-b2	73.92555	26.9325	100.8581	125	1.239366
	GRNh-b2	72.77321	26.9325	99.70571	121	1.213571
	GRRh-b2	70.84397	26.9325	97.77647	127.5	1.303995
	BNNm-b2	64.06883	26.9325	91.00133	119	1.307673
	BRNm-b2	63.64452	26.9325	90.57702	119	1.313799
	BRRm-b2	62.9245	26.9325	89.857	118	1.313198
	BNNh-b2	75.85357	26.9325	102.7861	131	1.274492
	BRNh-b2	77.54466	26.9325	104.4772	130.5	1.249077
	BRRh-b2	75.7532	26.9325	102.6857	128	1.246522
Etxeberria,(2007)	HC-2	80.45324	60.96808	141.4213	213	1.506138
	HC-3	80.45324	46.62265	127.0759	177	1.392868
	HR25-2	80.75287	60.96808	141.721	186.5	1.315966
	HR25-3	80.75287	46.62265	127.3755	169	1.326786
	HR50-2	80.08684	60.96808	141.0549	220	1.559676
	HR50-3	80.08684	46.62265	126.7095	176	1.389004
	HR100-2	79.04663	60.96808	140.0147	189.5	1.353429
	HR100-3	79.04663	46.62265	125.6693	163	1.297055
Al-Zahraa et al., (2011)	B4	24.87796	14.08185	38.95981	44.5	1.142203
	B5	26.02364	14.08185	40.10549	49.75	1.240479
	B6	24.3182	14.08185	38.40005	42	1.093749
	B7	26.41728	24.82853	51.24581	75	1.463534
	B8	26.02364	24.82853	50.85217	61	1.199556
	B9	24.3182	24.82853	49.14673	58	1.18014
	B10	24.52363	14.08185	38.60548	37	0.958413
	B11	22.78672	14.08185	36.86857	47.25	1.281579
	B12	22.57502	14.08185	36.65687	34.25	0.934341