UNIVERSITY OF GAZİANTEP
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MODELING OF SHEAR STRENGTH OF REINFORCED CONCRETE BEAMS WITHOUT WEB REINFORCEMENT BY STEPWISE REGRESSION

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Modeling of Shear Strength of Reinforced Concrete Beams without Web Reinforcement by Stepwise Regression

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Pishtewan M.RASHID

ABSTRACT

MODELING OF SHEAR STRENGTH OF REINFORCED CONCRETE BEAMS WITHOUT WEB REINFORCEMENT BY STEPWISE REGRESSION

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M.Sc. in Civil Engineering
Supervisor: Assoc. Prof. Dr. AbdulkadirÇEVIK
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In this thesis, the availability of soft computing (SC) technique (Stepwise Regression), for the prediction and formulation of shear strength of RC beams without web reinforcement was investigated.Literature survey on previous experimental studies has also been carried out regarding shear strength of RC beams without web reinforcement and a wide range of experimental database (398 tests) has been gathered from literature from 46 separate studies.New expressions are proposed based on the database taking into account the observed behavior for the design of normal-strength reinforced concrete beams without web reinforcement. The database is used for studying parameters that affect shear behavior/strength and for evaluating and comparing four national shear design provision (ACI 318-02, CSA A23.3 2004 edition (Collins, 2003), Euro code EC2 Part 1 (1994, 2003) and the German Code (DIN, 2001)) as well as for identifying research needs.

Keywords:RC beams, shear strength, Soft Computing, Stepwise Modeling

ÖZET

ADIMSAL REGRESYON YÖNTEMİYLE KESME DONATISIZ BETONARME KİRİŞLERİN KESME DAYANIMININ MODELLENMESİ

M. Rashid, Pishtewan Latef
İnşaat Mühendisliği Yüksek Lisans
Danışman: Doç. Dr. AbdulkadirÇEVIK
May 2013, 99sayfa

Bu tezde, esnek hesaplama tekniklerinin (Aşamalı regresyon) etriyesiyesiz betonarme kirişlerin kesme dayanımının tahmini ve formule edilmesinde kullanılması incelenmektedir. Betonarme kirişlerin kesme dayanımı konusunda daha once yapılan çalışmalar konusunda literatür taraması yapılmış ve 46 farklı çalışmadan alınan 398 testten oluşan geniş bir deneysel veritabanı oluşturulmuştur.Bu veritabanını esas alarak betonarme kirişlerin kesme dayanımı konusunda yeni formuller önerilmiştir.Ayrıca bu deneysel veritabanı kullanılarak 4 farklı ulusal tasarım kodundaki(ACI 318-02, CSA A23.3 2004 (Collins, 2003), Euro code EC2 Kısım 1 (1994, 2003) ve Alman Kodu (DIN, 2001)) kesme dayanımı yaklaşımları değerlendirilmiştir

Anahtar kelimeler: Betonarme kirişler,Esnek hesaplama, Aşamalı modelleme.

ToMyParents

ACKNOWLEDGEMENT

I express sincere appreciationto my supervisor Assoc. Prof. Dr. Abdulkadir CEVIK for believing in me and for providing me with invaluable advice throughout the duration of this research. I am also thankful to his for his patient and constructive reviews of the thesis.

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My sincere appreciation also extends to my friends and to those gave me the delight of smile.

Finally, and the most sweetly difficult, but the most emotionally important, I say. I have to pronounce the last but the greatest thanks. This is turned to my family for their sacrifices and for the love and encouragements proved to me during my studies. Being part of my family has been a privilege. Without them I would be nothing and no one. I will never say good-bye to you my Father because I know this is not the end for us to see each other. In my triumph you were always proud. I'm very grateful and proud to call you my dad. Unfortunately, I am not able to find a proper form to express what I feel. For this I beg your pardon.

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LIST OF SYMBOLS/ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Official
ACI	American Concrete Institute
ASCE	American Society of Civil Engineer
CEB	Comitè Euro-International du Bèton (Euro-International Committee for Concrete)
CFT	Compression Field Theory
COV	Coefficient of Variation
CSA	Canadian Standard Association
DIN	Deutsches Institut für Normung (German Institute for Standardization)
EC2	Eurocode 2
FIP	Fèdèration International de la Prècontrainte (International Federation for Pre stressing)
LRFD	Load and Resistance Factor Design
MCFT	Modified Compression Field Theory
PL	Point Load
RC	Reinforced Concrete
SDB	Shear Database

SR	Stepwise Regression
TMCF	Truss Model with Crack Friction
a	Shear span of a beam, the distance from a support to a loading point
a/d	Shear span to depth ratio
A_c	Area of concrete cross section
A_{cr}	Area of crack plane
A_s, A_{sl}	Area of longitudinal reinforcement in tension zone
A_{sw}	Cross-sectional area of shear reinforcement within spacing
A_{vmin}	Minimum area of shear reinforcement require (by code)
b_w	Width of web
d	Effective depth of member
$f1$	Principle tensile stress
$f2$	Principal compressive stress, diagonal compressive stress
f_{2max}	Maximum compressive stress
$f'c$	Uniaxial cylinder compressive strength of concrete
f_{cd}	Design value of uniaxial concrete compressive strength, normally may taken as $f_{cd}=f_{ck}/1.5$ in EC2 and new EC2
f_{ck}	Characteristic cylinder compressive strength of concrete ($\approx 0.9\ f'c$) in EC2
f_{cr}	Cracking strength of concrete
f_{ctd}	Design value of tensile strength, normally may taken as $f_{ctd}=f_{ctk,0.05}/1.5$ in new EC2

$f_{ctk,0.05}$	Lower 5% fractile characteristics tensile strength (=0.7 f_{ctm}) in EC2
f_{ctm}	Mean value of the tensile concrete strength (=0.30$(f_{ck})^{2/3}$) in EC2
f_{cwd}	Diagonal compressive stress of web concrete
f_{se}	Effective stress in the pre stressing steel after loses
f_{sx}	Stress of longitudinal reinforcement
f_{sy}, f_v	Stress of shear reinforcement
f_y	Yield strength of non-pre stressing reinforcement
f_{ywd}, f_{wyd}	Design yield strength of shear reinforcement
h	Height of member
I	Moment of inertia about the centroid of the cross section
jd	Flexural lever arm
k	Parameter to account size effect in EC2
l	Span length of member
M	Moment
M_{cr}	Cracking moment
M_{max}	Maximum moment at section considered
M_n	Nominal flexural capacity
M_u	Factored external moment
N_{Ed}	Axial force in the cross section due to loading
N_v	Tensile force in the longitudinal reinforcement caused by shear

S	Spacing of shear reinforcement
S_x, S_y	Spacing of longitudinal and transverse reinforcement
S_{xe}, S_{ze}	Equivalent crack spacing parameter in AASHTO LRFD and CSA 2004
τ, υ	Shear stress
v_{cr}	Shear stress at cracking
v_u, $v_{u,test}$	Ultimate shear stress
V_c, V_{cd}, V_{Rd}	Shear resistance of concrete
V_{ca}	Aggregate interlock
V_{cc}	Shear in compression zone
V_{ci}	Shear force at flexure-shear cracking
V_{code}, V_{pred}	Shear capacity of member obtained by code or an approach
V_{cw}	Web-shear cracking force
V_{cwd}	Diagonal compressive capacity of web concrete
V_d	Shear force at section due to un factored dead load; also dowel action
V_{ccd}	Vertical component of the force in an inclined compression chord
V_{fd}	Vertical component of friction force at crack
V_{max}	Maximum shear force at section considered
V_n	Nominal shear resistance
V_p	Vertical component of prestressing steel

V_{pd}	Vertical component of force in prestressing tendon
V_{Rd1}	Shear resistance of concrete based on the crushing of strut
$V_{Rd,c}$	Shear resistance of members not require shear reinforcement in new EC2
$V_{Rd,ct}$	Shear resistance of members not require shear reinforcement in DIN
V_{Rd}, V_s	Shear resistance provided by shear reinforcement
V_{sd}	Design value of shear resistance in new EC2 and DIN
V_{test}	Ultimate shear capacity of member obtained from experimental test
V_{swd}	Shear force carried by the stirrups across the crack
z	Inner lever arm
α	Angle of inclined stirrups to longitudinal axis
α_c	Reduction factor for the strength of the strut in DIN (=0.75η_1)
α_{cc}	Coefficient taking account of long term effects on the compressive strength and of un favorable effects resulting from the way load is applied
α_{ct}	Coefficient taking account of long term effects on the tensile strength and of un favorable effects resulting from applied loading patterns
β	Factor accounting for shear resistance of cracked concrete; angle of inclination of crack from horizontal axis
βr	Angle of crack
γ_c	Material factor for concrete

γ_{xy}	Shear strain
ε_2	Principal compressive strain
ε'_c	Strain corresponding to $f'c$ in a cylinder test
ε_x, ε_y	Strain in horizontal and vertical direction
η_1	Parameter to account shear resistance of concrete having different weight
θ	Angle of compression strut
θ_{cr}	Angle of (initial) diagonal crack
μ	Mean value
ρ	Reinforcement ratio
ρ_v, ρ_y	Shear reinforcement ratio
ρ_w, ρ_x	Longitudinal reinforcement ratio
σ_{cd}, σ_{cp} , σ_{xd}	Axial stress in the cross section due to loading or pre stressing without considering eccentricity of applied force
τ_{rd}	Basic design shear strength in EC2 (=0.25 $f_{ctk0.05}$)

CHAPTER ONE

INTRODUCTION

1.1 Statement of the problem

Concrete members are routinely used by the profession for building different types of structures. The axial and flexural behavior of these members is well understood. However, shear failure is still an area of research. Shear failure is generally initiated by inclined cracks that are caused not only by shear force, but by shear force in combination with moments and axial loads. Shear failure depends on numerous factors such as the dimensions, geometry, loading and the structural properties of members. Since shear cracks are inclined and shear failure depends on a large number of factors, shear design must consider the response of a finite length of the member, rather than the response of a single section. Due to the complications of shear behavior and the difficulties of shear design, the shear behavior and strength of members have been the major areas of research in reinforced and prestressed concrete structures for decades.

These shortcomings in shear design practice are not due to a lack of experimental research effort. Over the last century, a few thousand beam shear tests have been conducted. Unfortunately, these efforts have not produced the data necessary to develop a sufficiently complete understanding of shear strength and behavior.
The goal of the dissertation is to make specific recommendations that can be used in the codified approaches for improved shear design based on the proposed model Thus, central to the work discussed in this dissertation is modeling of shear behavior of reinforced concrete without shear reinforcement, through a unified approach..

1.2 Objective and Scope of Study

The primary objective of this study is to make specific recommendations that can be used in the codified approaches for improved shear design. To meet this objective ,the shear behavior of reinforced concrete without shear reinforcement is modeled through a unified approach. The primary objective was met with the help of the following four objectives.

- Objective 1: Study the existing codes of practice and their associated theoretical models for prediction of shear strength. Evaluate and compare the shear strength prediction of these codes of practice for reinforced concrete beams without shear reinforcement.
- Objective 2: Develop a rational model for modeling of shear behavior of reinforced concrete beams without shear reinforcement.
- Objective 3: Evaluate the proposed rational model using the large shear database and compare it with the codes of practice.
- Objective 4: To develop a shear test database by conducting an extensive and detailed literature review.

1.3 Outline of the thesis

To meet the primary objective comprehensively, the study consists of two parts representing 5 chapters, an appendix and a list of references.

The first part consists of the first two chapters. Chapter 1 is an introductory chapter. It includes the statement of the problem, objective and scope of research and Outline of the thesis, Chapter 2 contains a comprehensive literature review on the available theoretical models for understanding shear behavior and existing shear design provisions in national codes of practice.

The second part consists of two chapters, Chapters 3 and 4. In Chapter 3, effect of influencing parameters on shear strength. In Chapter 4, numerical application and result, Comparison of current design codes and equations with stepwise model, parametric study. Summary and conclusions of the study are presented in Chapter 5. A comprehensive list of references is also included at the end of this thesis.

CHAPTER 2

LITERACTURE REVIEW

2.1 Introduction to shear:

The shear stress acts parallel or tangential to the section of a material. When a simple beam is subjected to bending, the fibers above the neutral axis are in compression and the fibers located below the neutral axis are in tension. A concrete beam with longitudinal steel when subjected to external loads will develop diagonal tensile stresses which will tend to produce cracks. These cracks are vertical at the center of the span and will become inclined as they reach the support of the beam as shown in Figure 2.1. The stress that causes the inclined cracks in the beam is called diagonal tension stresses (Jose, 2002).

Shear in concrete is a complex problem. In order to establish a basic understanding of the problem, this chapter initially provides an introductory explanation on shear behavior of concrete. This is followed by a discussion of different theoretical models including 45◦ truss model, variable-angle truss model, compression field theory, modified compression field theory, and truss model with crack friction. Then, the shear design provisions of several national codes of practice based on these theoretical models are discussed (Anuja, 2006) .These include ACI 318-02 (2002); ASCE-ACI 445 (2003); CSA A23-3-94 (1994); CSA A23-3 2004 edition; Eurocode EN 1992-1-1 (2003); German Code DIN 1045-1(2001); (Zsutty, 1968); (Collins and Kuchma, 1999)

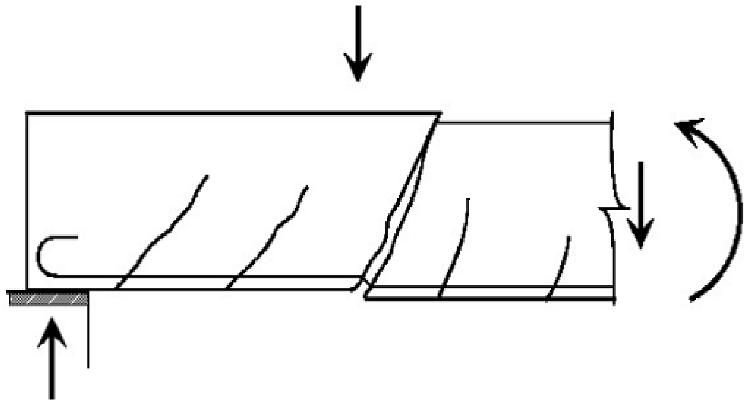


Figure 2.1 Cracks appeared when vertical load is applied at the mid span of a beam(Jose.M.A, 2000)

2.1.1 The problem of the shear transfer

A flexural member supports loads by internal moment and shear forces. Classical beam theory, in which plane sections are assumed to remain plain, provides an accurate, simple, and effective model for designing a member to resist bending in combination with axial forces. The simplicity and rationality of beam theory can be kept even after cracking for several reasons. The first reasons is that flexural cracks from perpendicular to the axis of bending so that the traditional "plane sections remain plane" assumption is valid. The second reason is the weakness of concrete in tension, so that tensile stresses can be effectively neglected at a crack. The third reason is that flexural failure occurs at the maximum moment location such that a consideration of the conditions at the maximum moment section is sufficient for flexural design. Shear failure is initiated by inclined cracks that are caused not only by shear force but by shear force in combination with moment and axial loads. Shear failure depends on numerous factors such as the dimensions, geometry, loading and the structural properties of members (Kang, 2004).

2.1.2 Shear transfer action and mechanism

Shear transfer actions and mechanisms in concrete beams are complex and difficult. The complex stress redistributions that occur after cracking are difficult to model, and are influenced by many factors. Different researchers impose different levels of relative importance to the basic mechanisms of shear transfer. In this section, mechanisms of shear transfer and influencing parameters on shear failure for concrete beams with and without transverse reinforcement are discussed. Figure 2.2

describes the basic mechanisms of shear transfer that are now generally accepted in the research community (Anuja, 2006). In 1973, the ASCE-ACI Committee 426 and again in 1998, its current counterpart the ASCE-ACI Committee 445, reported five important shear transfer actions: shear in the un cracked compression zone of the beam; interface shear transfer due to aggregate interlock or surface roughness of the cracks; dowel action of the longitudinal reinforcement; residual tensile stresses across the crack; shear transfer of the shear reinforcement (in case of beams with transverse reinforcement) and shear transfer of the prestressing reinforcement (in case of prestressed concrete beams) (ASCE-ACI Committee 426, 1973; ASCE-ACI Committee 445, 1998). Each of these actions are depicted in Figures 2.2 and 2.3. They are discussed in more detail next.

Shear in the Un cracked Concrete Zone (V_{cc}): The un cracked compression zone contributes to shear resistance in a cracked concrete member (i.e., a beam or a slab).The relative magnitude of the shear resistance provided by the un cracked compression zone is limited by the depth of the compression zone. Consequently, for example, in a relatively slender beam without axial compression, the shear contribution by the un cracked compression zone becomes relatively small due to the small depth of the compression zone (Anuja, 2006).

Interface shear transfer (Vca): Local roughness in the crack plane provides resistance against slip and thus shear transfer across shear cracks. The contribution of interface shear transfer to shear is a function of the crack width and aggregate size. Thus, its magnitude decreases as the crack opening increases and as the aggregate size decreases. This is why it was also called “aggregate interlock”. It is also called “interface shear transfer” or “friction” since this action still exists even if crack propagation occurs through the aggregate as it does in high strength concrete where the matrix is of a similar strength to the aggregates. Of course, a relatively smooth crack plane in high strength concrete reduces interface shear transfer (Anuja, 2006).

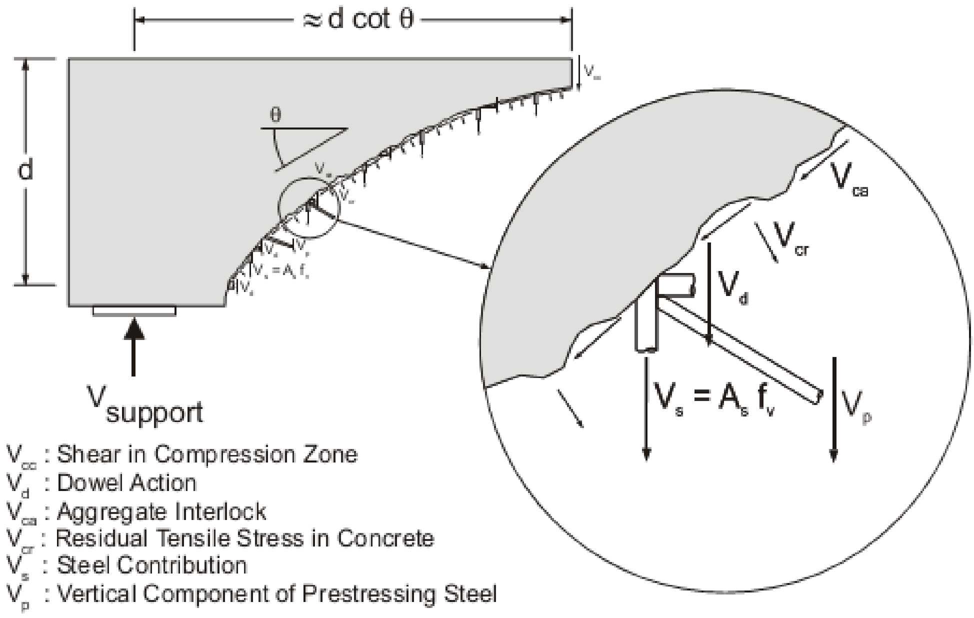


Figure 2.2. Shear transfer/actions contributing to shear resistance

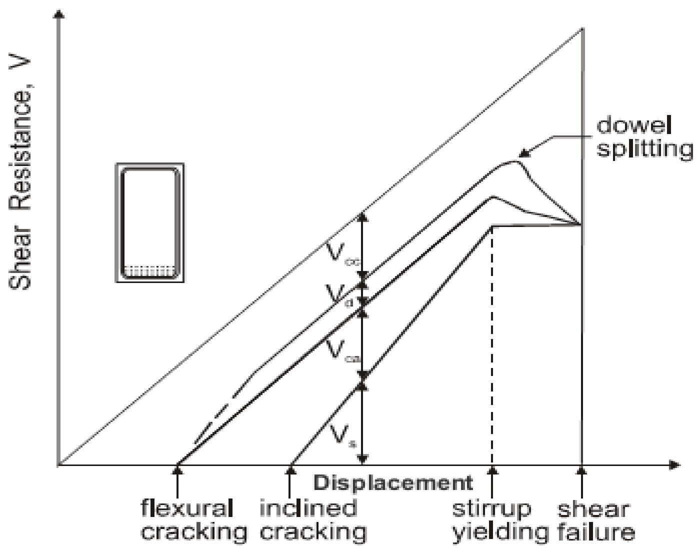


Figure 2.3. Distribution of internal shear resistance (ASCE-ACI Committee 426,1973)

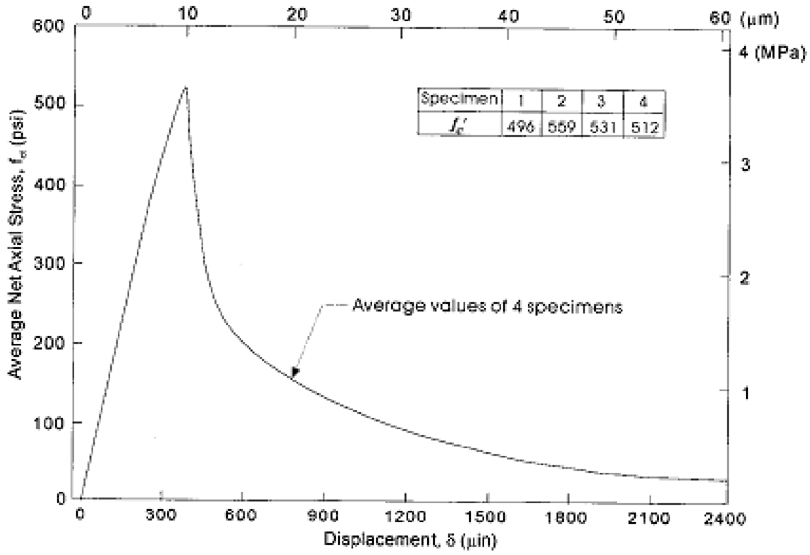


Figure 2.4 Response of Concrete in Uniaxial Tension (Gopalaratnam and Shah, 1985)

Dowel Action (V_d): When a crack forms across longitudinal bars, the dowelling action of the longitudinal bars provides a resisting shear force. The contribution of dowel action to shear resistance is a function of the amount of concrete cover beneath the longitudinal bars and the degree to which vertical displacements of those bars at the inclined crack are restrained by stirrup reinforcement. Typically, little dowel action can be provided by reinforcement that is near the tension face of a member without stirrup reinforcement because that action is then limited by the tensile strength of the reinforcement (Anuja, 2006).

Residual Tensile Stress (V_{cr}): Tensile stresses in concrete are directly transmitted across cracks because small pieces of concrete bridge the crack. Gopalaratnam and Shah (1985) note that even when concrete is cracked and loaded in uniaxial tension figure 2.4,concrete can still transmit tensile stresses up to crack widths of 0.06 mm; other researchers report stresses up to maximum crack width values of 0.16 mm. When a crack opening is small, the resistance provided by residual tensile stresses is significant .However, in a large member, the contribution of residual tensile stresses to shear resistance is less significant due to the large crack widths that often occur before failure in such members (Anuja, 2006).

Shear Reinforcement (Vs): In members with shear reinforcement, a large portion of the shear is carried by the shear reinforcement after diagonal cracking has occurred. The contribution of shear reinforcement to shear resistance is typically modeled either with a 45◦ truss plus a concrete term, or a variable-angle truss with or without a concrete term. Shear reinforcement also provides a certain level of restraint against the growth of inclined cracks and thus helps to ensure a more ductile behavior. Finally, shear reinforcement provides dowelling resistance to shear displacements along the inclined crack. For these reasons, the presence of shear reinforcement changes the relative contributions of the different mechanisms of shear resistances (Anuja, 2006).For beams with transverse reinforcement, the basic model to explain the mechanism for carrying the shear was proposed by Ritter (1899). The load was assumed to flow down the concrete diagonal struts and then lifted to the compression chord by transverse tension ties on its way to support as shown in Figure 2.5 below (Attaullah, 2009).

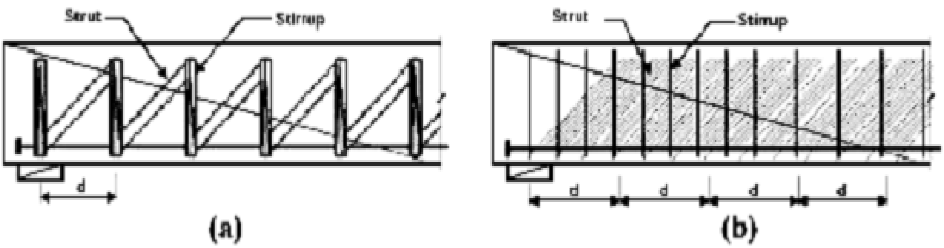


Figure 2.5 Parallel chord truss model. The struts are intercepted by the stirrups at spacing of d (Ritter, 1899)

Prestressing (Vp) : In the early years of prestressed concrete construction, it was be lived that draped prestressing tendons would be effective in all situations as shear reinforcement. This was because longitudinal reinforcement, bent up at 30◦ or more to the longitudinal axis of the beam, extended across the web and anchored on the compression side, was used effectively for many years as shear reinforcement in reinforced concrete beams. However, the University of Illinois tests (Bulletin 493) demonstrated that such reinforcement was effective only in delaying shear cracks formed in the web due to principal tensile stresses. Draping the prestressing tendons did not delay the formation of inclined cracks that developed out of flexural cracks.

Thus, in ACI 318Code (ACI Committee 318, 2002) , the term for the vertical component of the pre-stress force appears only in expressions for the shear strength for web-shear cracking and does not appear in either the expressions for the shear strength for flexure shear cracking or the shear strength contributed by shear reinforcement (Anuja, 2006).

2.1.3 Influencing Parameter for Members without Transverse Reinforcement

Several parameters have been identified as having a significant influence on the contributions of the shear resistance mechanisms and thus the ultimate shear capacity. The influence of the most dominant mechanisms is listed in accordance with the findings of the state-of-the-art reports by ASCE-ACI Committee 426 (1973) and ASCE-ACI Committee 445 (1998).Concrete Strength; Size effect; Shear span to depth ratio; Longitudinal Reinforcement ratio; Axial Force (Anuja, 2006).

Concrete Strength:

It is traditionally considered that the shear strength increases with concrete material strength. Most researchers believe that concrete tensile strength has a greater influence on shear strength than does the compressive strength. The concrete contribution to shear, in ACI 318-02, for example, is regarded as being due to diagonal cracking shear. The concrete compressive strength f'_c , is generally used to estimate the tensile strength as direct tensile test are difficult to conduct, require interpretation of results, and usually have more scatter than do compression test results. In major design codes, the shear strength of a member is usually taken as directly proportional to $(f'_c)^{0.25}$ or $(f'_c)^{0.33}$ or $(f'_c)^{0.5}$. Those power values indicate that the concrete tensile strength is being used as the governing parameter (Kang, 2004).

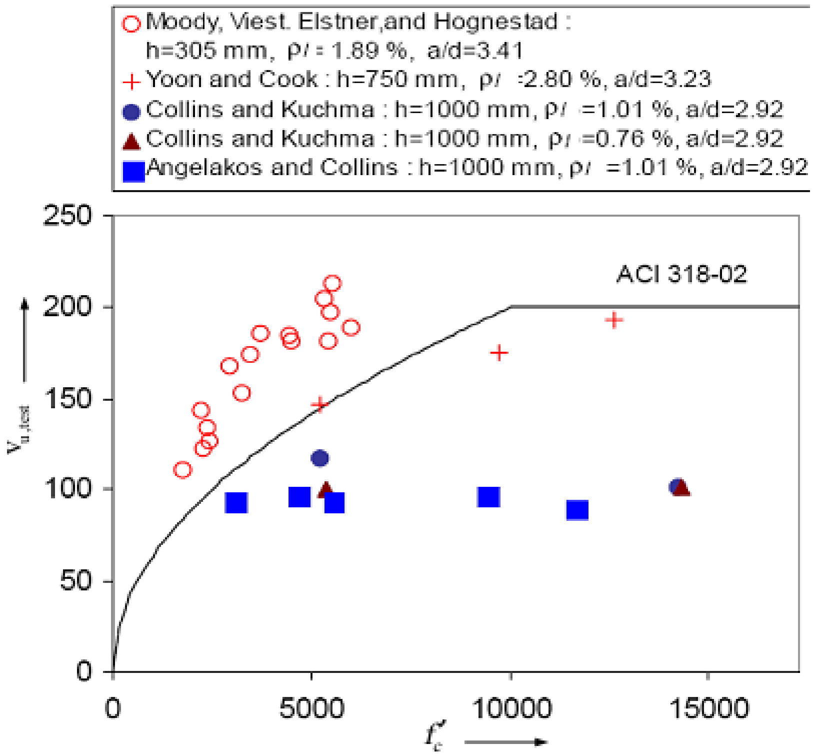


Figure 2.6 . Influence of concrete compressive strength on shear strength (Kuchma and Kim, 2001)

Recent test results have illustrated that the presumed effect of concrete tensile strength on shear capacity is largely influenced by the characteristics of the tests conducted to examine this influence. Results from two beam test series which show very different trends are shown in Figure 2.6. The ACI 318-02 (2002) shear design approach in which the shear strength is proportional to the square root of f'_c is also shown in the same figure. The shear failure stresses of beams tested by Moody et al.(1954) increase as the concrete compressive strengths increases. ACI 318-02 is shown to provide a reasonable estimate of the influence of f'_c for these beams which has less height h, were heavily reinforced, and cast with low-to-medium-strength concrete. Similarly, the ACI provision is only slightly un conservative for the moderately reinforced beams having moderate height that were tested by (Yoon et al. 1996).However, (Angelakos et al.2001) and (Collins and Kuchma, 1999) did not find a similar increase of shear strength for their tests of beams with larger height, light reinforcement, and high strength concrete with small aggregates. The explanation offered by some researchers for why shear stress at failure does not increase as greatly or not at all with increasing concrete compressive strength is that the smoother shear cracks in high-strength concrete members reduce the effectiveness of interface shear transfer (Anuja, 2006).

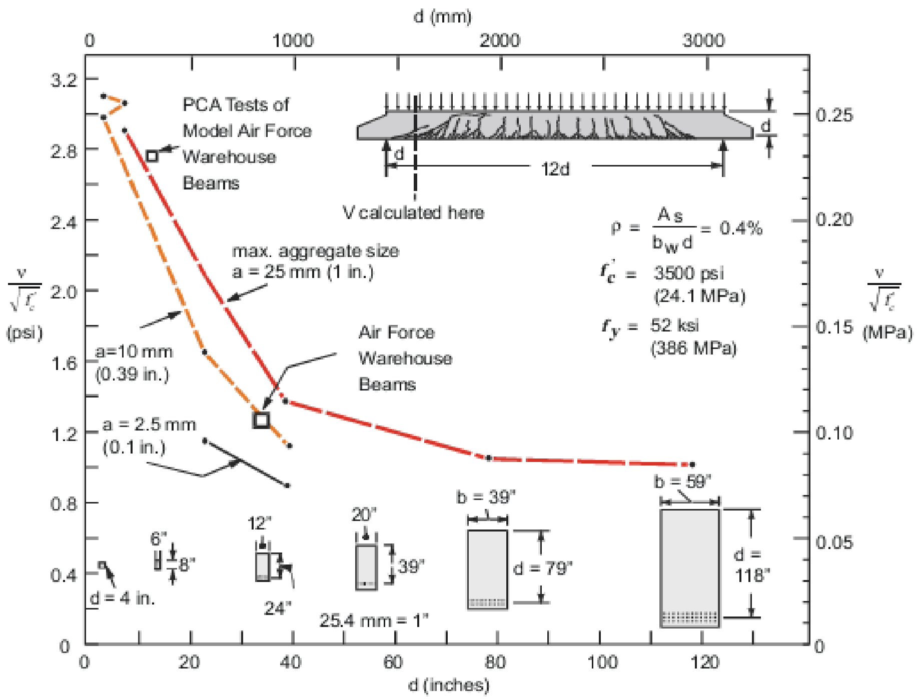


Figure 2.7 . Size effect in Shear (Collins and Kuchma, 1998)

Size Effect :

The shear strength of reinforced and prestressed beams without shear reinforcement decreases as member depth, *d*, increases; this is called the “size effect” in shear. Tests performed by both Kani (1967) and Shioya (1989) effectively demonstrated this effect. Shioya (1989) tested members with depths ranging from 4 ~ 120 inches(102 mm ~ 3.0 m), as shown in Figure 2.7. The ultimate shear stress of the largest member was only about one-third of that of the smallest one, and the ultimate shea rstress of the largest beam was less than one half of the value calculated using ACI318-02. In 1956, beams in the US Air Force warehouse collapsed under a shear forceless than one half of the ACI design value, as shown in Figure 2.7. The depth of these beams was 36 inches (914 mm). Investigators examining these failures conducted experiments with one-third scale models at the Portland Cement Association (PCA).The failure strengths for those model beams are also shown in Figure 2.7. Due to the much higher failure strength of PCA test beams than that of the warehouse beams, the investigators concluded that axial tensile stresses due to shrinkage restraints by columns were the primary cause for those failures. However, it seems more reasonable to explain the results in terms of the size effect in shear. In models used to account for the size effect in shear, some researchers explain the size effect by fracture mechanics and suggest that the large amount of energy that is released in the cracking of large members leads to the faster propagation of inclined cracks and lower shear failure stresses (Bazant and Kazemi, 1991; Bazant and Kim,1984). Other researchers, like Collins and Kuchma (1998) and Reineck (1990; 1991a;1991b), explain the size effect by a reduction of the interface shear transfer due to the larger crack widths that occur in larger members (Anuja, 2006).

Shear Span to Depth Ratio:

ASCE- ACI Committee 326 (1998) has showed the shear capacity as function of shear to moment ratio. The basic equation for the shear strength of RC concrete beams proposed by ACI-318-98, makes the shear span to depth ratio as one of the basic parameters for calculating the shear capacity of RC section (Attaullah, 2009).

When the shear span to depth ratio becomes less than 2.5, the shear capacity of the RC becomes larger than that of slender beams as the shear is directly transferred to supports through compression struts. However the supports condition strongly influences the formation of compression strut. Compressive strut is more likely to form when beam is loaded from upper face and supports to the bottom face (Adebar 1994).

Kotsovos.M.D (1984) studied the effect of web reinforcement for the RC beams having a/d ratio between 1 and 2.5 with the help of nonlinear finite element analysis and observed that placement of web reinforcement in the middle third rather than in the shear span results in improved ductility and load carrying capacity of RC beams. In one of the latest studies by Kotsovos and Pavlovic (2004), they used finite element analysis to study the size effect in beams with smaller shear span to depth ratio less than 2 and compared the results of theoretical model with the actual experiment. They concluded that the shear and flexural capacity of beams with shear span to depth ratio less than 2, is independent of the size of members and the size effect vanishes for such beams.

The shear span to depth ratio a/d has accounted for by most of the building and bridges codes in the world (Attaullah, 2009).

Longitudinal Reinforcement Ratio:

For the same magnitude of loading, as the longitudinal reinforcement ratio decreases, flexural stresses and strains increase. Thus, crack widths increase and the shear strength is lowered. Further, as the longitudinal reinforcement ratio decreases, dowel action also decreases. It has also been reported that members having distributed longitudinal bars over the depth of the member have smaller crack spacing and that improves shear strength significantly (Collinsand Kuchma, 1999).

Axial Force :

As the axial tension in members increases, shear strength decreases. Since axial tension makes the crack angle steeper over almost the full depth of the member, longitudinal reinforcement needs to be provided in both the top and bottom of the member. Once appropriate amounts of longitudinal reinforcement are provided, the failure of such members may occur in a relatively ductile manner. By contrast ,axial compression increases the depth of the un cracked compression zone, decreases the width of the shear cracks, and thus the interface shear transfer is increased. All of these factors lead to an increase in shear capacity with increase in axial compression. However, for members subjected to significant axial compression, brittle failures are common (Attaullah, 2009).

Other Influencing Parameter:

In a simply supported member, high shear and high moment do not coexist. Thus, in high shear region of such members, the effect of moment is relatively small. However in a continuous beam, thus negative moment region is subjected to both high shear and high moment and, thus, the effect of moment can be significant. In a negative moment region, the compression and tension sides are reversed over those in a positive moment region. Consequently, the size of the un cracked compressive zone in negative moment regions in T beams is reduced because the wide slab is no longer in compression. When supports are located on the bottom of the member and loads are applied on the top, applied forces can be transmitted directly to supports through an inclined strut. Thus, such a member may have a higher shear capacity than a simple beam (Kang, 2004).

2.1.4 Shear Failure of Members without Shear Reinforcement

Shear failures of members without transverse reinforcement are initiated by inclined cracks, and these cracks are typically divided into two types, i.e., web-shear cracks and flexure-shear cracks as shown in Figure 2.8. The web-shear cracking occurs in high shear regions when the principal tensile stress reaches the tensile strength of the concrete. Flexure-shear cracking occurs in regions of high moment combined with significant shear and occurs at slightly higher shear forces than those for flexural cracking. As loading increases, flexural cracks form in and near the maximum moment region, as shown in Figure 2.8.(Anuja, 2006).

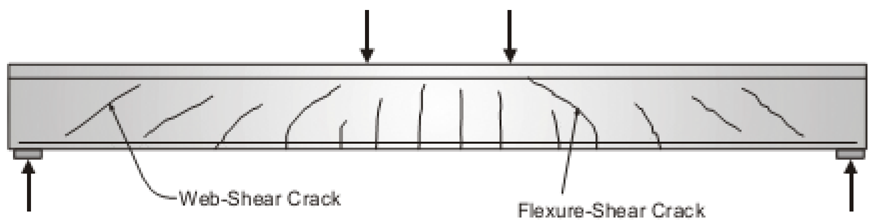
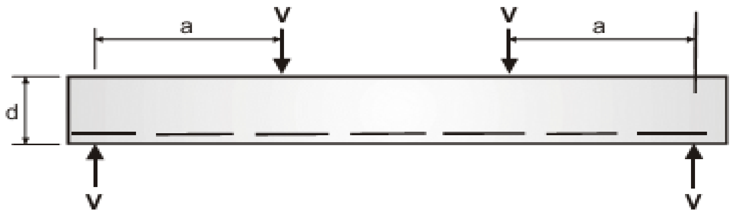
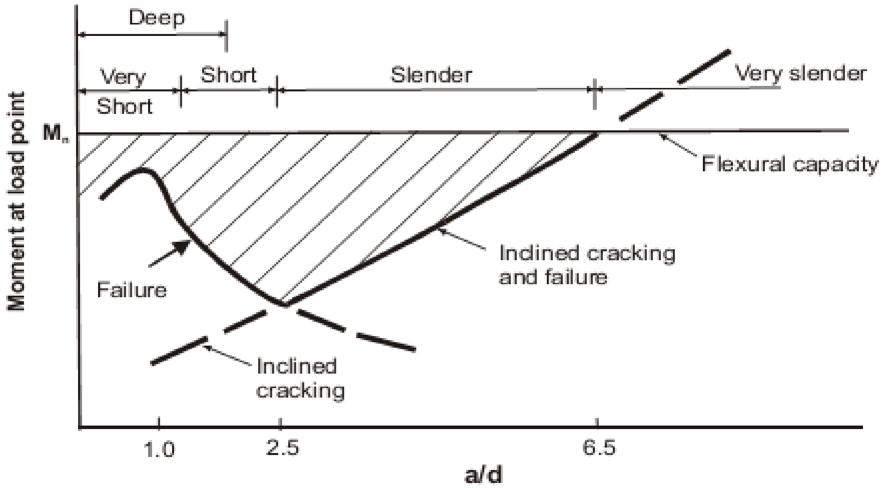


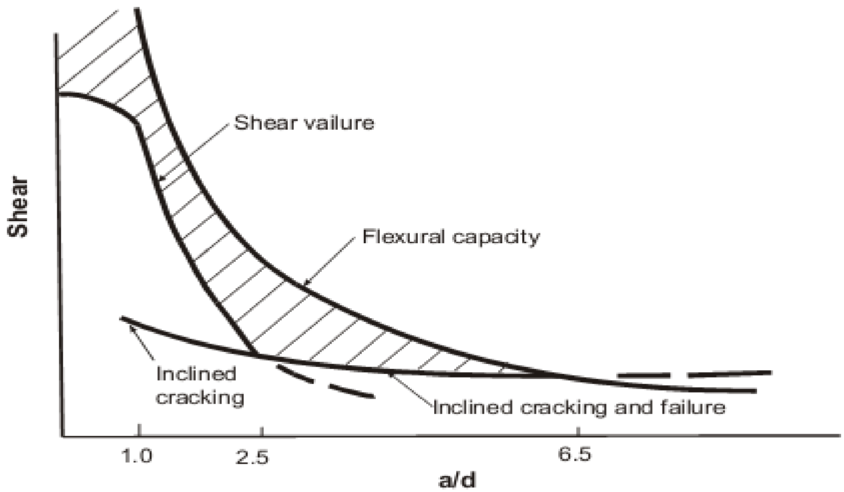
Figure 2.8 . Types of inclined cracks



(a) Beam



(b) Moments at cracking and failure



(c) Shear at cracking and failure

Figure 2.9 . Effect of shear span to depth ratio (a/d) on shear strength of beams without stirrups (MacGregor and Wight, 2004)

“Kani’s Valley of Shear Failures” is presented in Figure 2.9 (MacGregor andWight, 2004). Kani (1964) conducted a very large experimental study on shear and reported relationships between the beam capacity and the shear span to depth ratio a/d (where *a*, the shear span, is the distance between a support and the load closest to that support, and *d* is the depth of the member, as shown in Figure 2.9). Kanitested a large number of rectangular beams without shear reinforcement and having various a/d ratios, while the rest of the beam details remained the same, as shown in Figure 2.9(a). Then, the moment and shear at inclined cracking and failure were observed, as shown in Figure 2.9(b). In the figure, the flexural capacity, M_n, is the horizontal line while the shaded area represents the reduction of strength due to shear. From this figure, beams can be classified into four groups by a/d ratios: very short, short, slender, and very slender beams. Figure 2.9(c) can be obtained by dividing the moment in Figure 2.9(b) by the shear span, a, as the moment is M = V × a for beams with two point loads. Kani also tested beams subjected to uniformly distributed load and used the a/d ratio as a quarter of the span length, i.e., L/4 (Anuja, 2006).

2.1.5 Mode of Shear Failure

The modes of shear failure of beams were also discussed by ASCE-ACI Committee426 (1973) with classification of beams by a/d ratios. The failure modes of simply supported rectangular beams without shear reinforcement were described as follows:

• In very slender beams (a/d > 6), the members will likely fail in flexure even before the formation of inclined cracks.

• In slender beams (2.5 <a/d< 6), some of the flexural cracks grow and may become flexure-shear cracks. The diagonal cracks may continue to propagate toward the top and bottom of the beam and cause yielding of the tension steel .The beam may split into two pieces at failure. This is called as diagonal tension failure, shown in Figure 2.10.

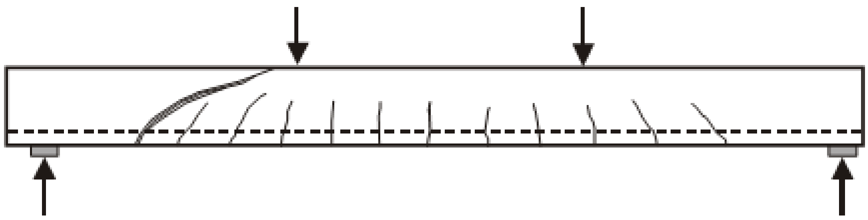
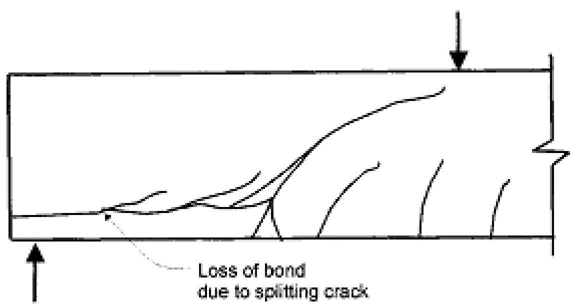


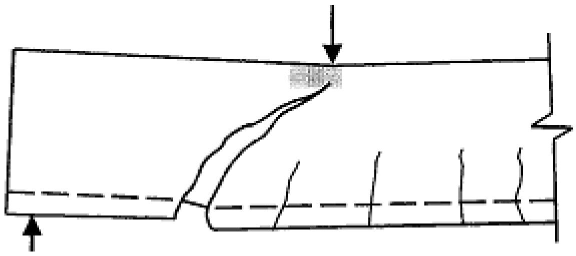
Figure 2.10 . Diagonal tension failure (ASCE-ACI Committee 426, 1973)

• In short beams (1 <a/d< 2.5), a diagonal crack may propagate along the tension steel causing splitting between the concrete and the longitudinal bars This is called a shear-tension failure, shown in Figure 2.11(a). The diagonal crack may propagate toward the top of the beam resulting in crushing of the compression zone resulting in crushing of the compression zone. This is called a shear-compression failure, shown in Figure 2.11(b).

• In very short beams (a/d < 1), inclined cracks occur along the line between load and reaction. Thus, most of the shear force is transferred by arch action with a structural system, as shown in Figure 2.12. The failure modes possible in this type of deep beam are also shown in Figure 2.12. Anchorage failure of tension steel may occur at the end of a tension tie. Bearing failure may occur by the crushing of concrete above a support. Flexural failure is also possible due to the yielding of tension steel or the crushing of the compression zone. Tension failure of “arch-rib” near the top of an edge may occur due to the eccentricity of the thrust of the compressive stresses in the inclined strut. Compression strut failure is also possible by crushing of the web along the crack (Anuja, 2006).



(a) Shear-tension failure



(b)Shear-compression failure

Figure 2.11 . Modes of shear failures in short beams (ASCE-ACI Committee 426,1973)

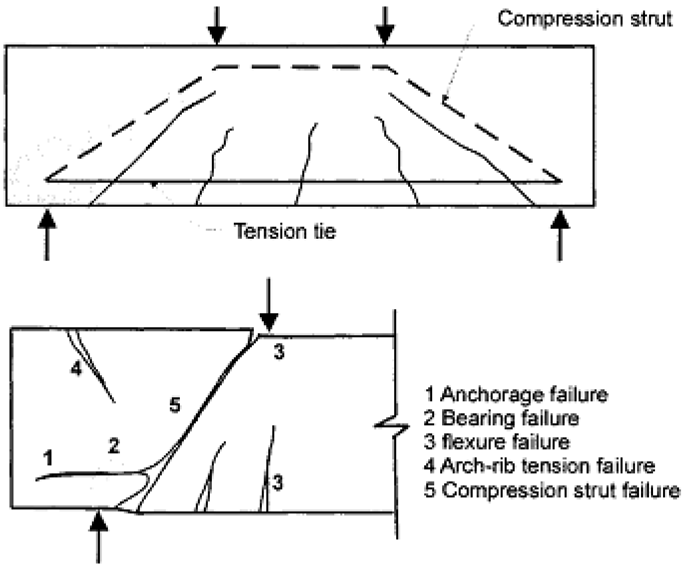


Figure 2.12. Modes of shear failures in deep beams (ASCE-ACI Committee 426,1973)

The failure of I shape beams are somewhat different from those of rectangular beams because the shear stresses in the webs are much higher than in rectangular beams. Web crushing failures are the most common failure mode for I beams, as shown in figure 2.13, although all the failure modes described above for rectangular beams are also possible for I beams as well (Kang, 2004).

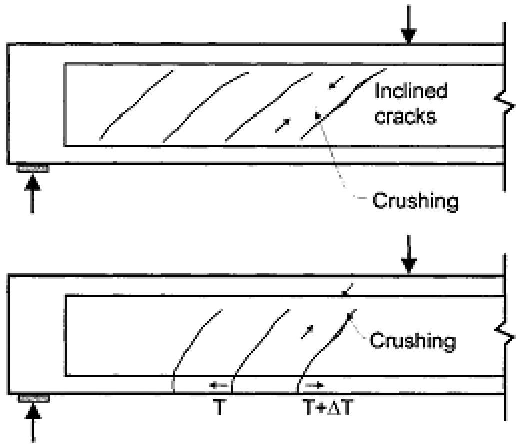


Figure 2.13 Modes of shear failure of I beams (ASCE-ACI Committee 426,1973)

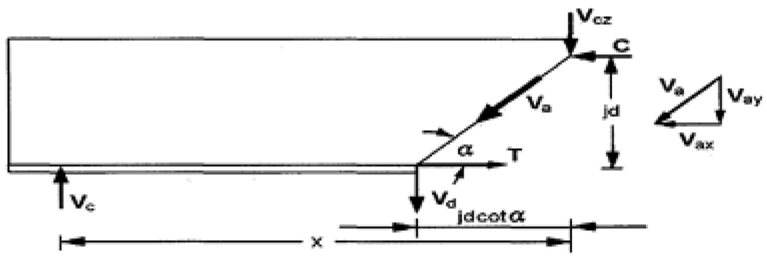
2.1.6 Shear Strength of Normal Strength Reinforced Concrete Beam

The research on shear strength of concrete has shown that reinforced concrete beams without transverse reinforcement can resist the shear and flexure by means of beam and arch actions, also sometimes called concrete mechanisms (Russo et al, 2002). These forces acting on the beam element in its shear span are shown in Figure 2.14. It was assumed that the resultant of the aggregates interlocking at the crack interface can be replaced by Va as shown in the Figure 2.14, whose direction passes through the point of application of the internal compression force C. The shear contribution due to dowel Vd is negligible at the rotation equilibrium. The resultant bending moment is given by

$$M_c = V_c.x = T.jd \qquad (2.1)$$

Where Vc is the shear force due to concrete resisting contribution, T is tensile force in the longitudinal reinforcement and x is the distance between the support and the point where crack has been appeared. The shear force is the derivative of the bending moment $V_c = dM_c/d_x$

$$V_c = jd\ [\frac{d}{dx}\ T]+T.\frac{d}{dx}jd \qquad (2.2)$$



Forces acting in a beam element within the shear span

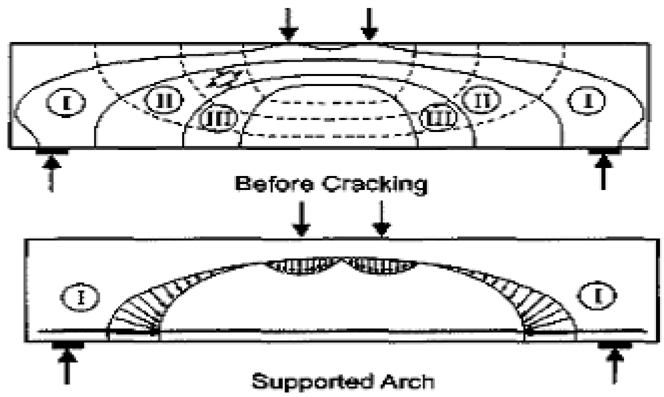


Figure 2.14 Forces acting in a beam element within the shear span and internal arches ina RC beam (Kani, 1964., Russo et al., 2004).

The first term in equation 2.2, is the resistance to shear as contribution of the beam action, whereas the second part is called arch action. In beam action, the lever arm is constant and the tensile force in the steel bars is supposed to vary. The beam action is related to the crack pattern in the shear span, in which the tensile zone is generally divided into blocks or teeth. Beam action describes shear transfer by changes in the magnitude of the compression-zone concrete and flexural reinforcement actions, with a constant lever-arm, requiring load-transfer between the two forces. In a cracked beam, load-transfer from the flexural reinforcement to the compression-zone occurs through the ‘‘teeth’’ of concrete between cracks, requiring bond between the concrete and reinforcement. Bending and failure of this concrete is studied by tooth models.

The second part of the equation shows the shear resisting contribution due to arch action, which is characterized by the internal variation of the lever arm *jd* with the *T* constant. The arch mechanism transfers the vertical loads to the supports through the arch route. Arch action occurs in the un-cracked part of concrete near the end of a beam, where load is carried from the compression-zone to the support by a compressive strut. The vertical component of this strut transfers shear to the support, while the constant horizontal component is reacted by the tensile flexural reinforcement. Both beam action and arch action can act in the same region (Stratford and Burgoyne,2003). Thus shear transfer in the beam can take place by one of the two mechanisms i.e. variation in the magnitude of internal actions and variation in the lever arm between the actions. The details are shown in Figure 2.15. Before cracking of the beams, the shear is resisted by the beam by all the elements of the beams shown in the paths I, II and III (Figure 2.14). However after the cracks, only the un-cracked part of the beams is resisting the shear by transferring it to the supports.

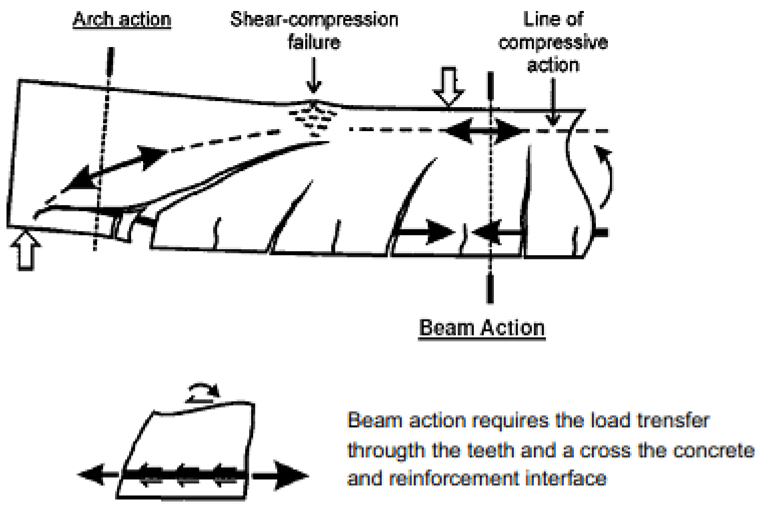


Figure 2.15 Shear in beam with no transverse reinforcement.(Stratford and Burgoyne, 2003)

In one of the earliest research on shear failure, at University of Toronto Canada, Kani (1964) defined the regions of beam action and arch actions for resisting the shear in RC beams, for the first time. It was pointed out by him that initially the shear is resisted by the teeth of cracked concrete, but after destruction of the resistance by teeth of the cracked beam, a quite different mechanism through tied arches in the compression zone occurs. On the basis of actual test results, Kani (1964), reported that in the region of low values of shear span to depth ratio (a/d), the shear capacity of the structure is determined by the strength of remaining arch, whereas in the region with medium value of a/d, the capacity of teeth of cracked concrete determines the shear capacity of the beams. He also proposed an expression for the boundary point separating the two regions. In Figure 2.16, the boundary for shear failure of the beams tested in Toronto has been given, which shows that up to a/d of 2.5, shear failure due to arch action is dominant whereas in the region with a/d more than 2.5 and up to 5.75 or 6 beam action due to concrete teeth (beam action) is dominant and the shear capacity due to arch action is very small (Attaullah, 2009).

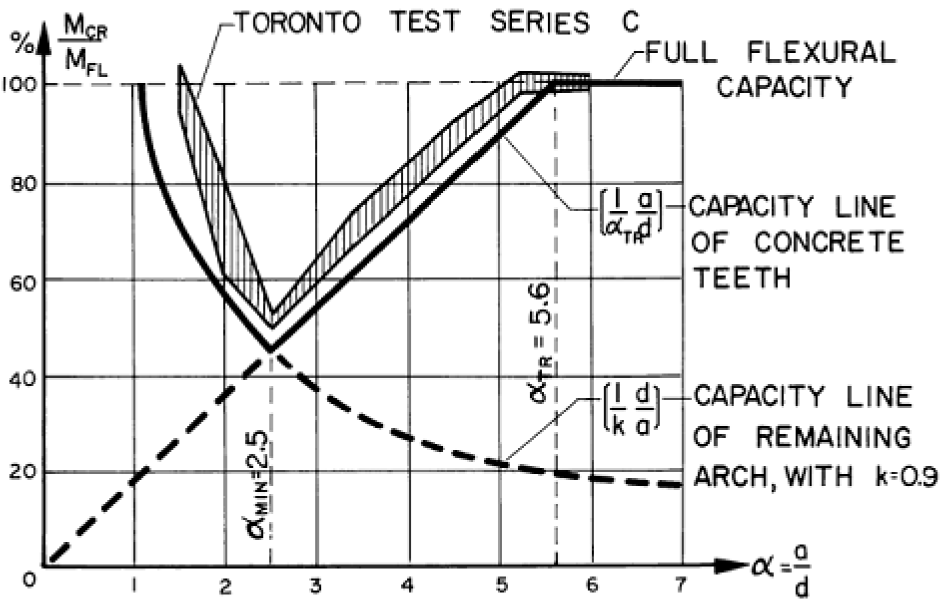


Figure 2.16 Comparison of theoretical and test results of shear failure of beams (Kani.1964)

2.2 Theoretical Models for Shear Behavior in concrete

To understand the design specifications of these codes it is essential to understand the underlying theoretical models. This section presents the history of development of theoretical models followed by some theoretical models for shear behavior in concrete as available in literature. The models include 45◦ truss model, variable-angle truss model, compression field theory, modified compression field theory, and truss model with crack friction (Anuja, 2006).

2.2.1 Historical Development of Shear Design of Reinforced Concrete Beam

Since there is a need for research in the area of shear analysis of concrete beams, research has already been done at various universities all over the world on this topic .Work in this area started as early as the 1900's.

Early researchers did most of the work in reinforced concrete beams with transverse reinforcement. Ritter (1899) suggested that after a reinforced concrete beam cracks due to diagonal tension stresses, it can be idealized as a parallel chord truss with compression diagonals inclined at 45◦ to the longitudinal axis of the beam. Later, Mörsch(1909) introduced the use of truss models for torsion in concrete. Withey (1907,1908) pointed out that these models gave conservative results and pointed out that Mörsch (1909) neglected contribution of concrete in tension. Talbot (1909) confirmed Whithey's findings.

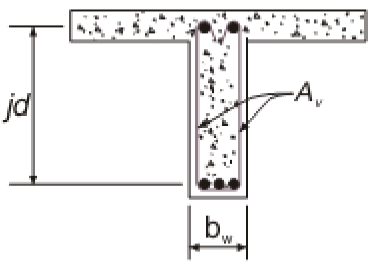
In 1950's and 1960's, a large amount of experimental research was conducted to study the contribution of aggregate interlock and dowel action on shear resistance. Zwoyer and Siess (1954), Bresler and Pister (1958), Guralnick (1959),and Walther (1962) studied the stress conditions in the concrete above flexural cracks of reinforced concrete beams without transverse reinforcement assuming that all shear would be carried in the flexural compression zone. Kani (1964)introduced the "comb" model in which the concrete between the flexural cracks is considered as the teeth of the comb and un cracked concrete as the backbone of the comb. Based on the large amount of available experimental results, the ASCE-ACI Committee 326 (1962)recommended the use of an empirical expression for shear stress at the flexure-shear cracking load which first appeared in the 1963 ACI 318 Code and is still present inACI318-02 (2002) as Equation 11-5.

In the 1950's and 1960's extensive work at the University of Illinois (Bulletin493) and at the Portland Cement Association (PCA) resulted in the development of the shear strength provisions for prestressed concrete beams without transverse reinforcement that continue to be incorporated into ACI 318-02.While the model used for the shear strength provided by the transverse reinforcement was the same as that for reinforced concrete beams, the models used for the shear at inclined cracking differed considerably from those for reinforced concrete beams.

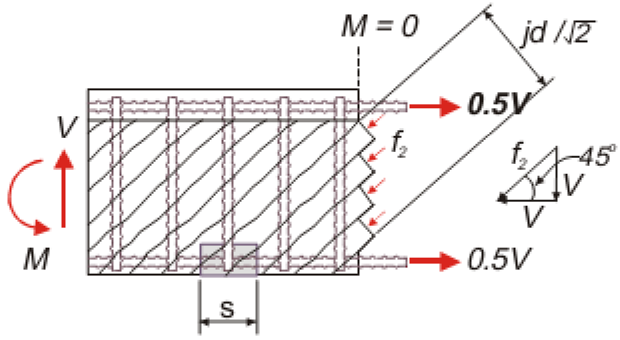
Fenwick and Paulay (1968) suggested that shear resistance carried by the compression zone is only about 25% of the total shear and “aggregate interlock” and dowel forces carried the remainder of the shear. Kupfer (1964) took a step forward for prestressed concrete members with transverse reinforcement in predicting the strut angle using minimum energy principles, and Baumann (1972a,b) further pursued this concept and presented dimensioning diagrams for plate elements with reinforcement in two or even three directions.

Inspired by this work, and also by Wagner’s (1929) tension field theory, Collins and Mitchell (1980, 1981) developed the Compression Field Theory (CFT). Though CFT worked well for members with medium to high percentage of transverse reinforcement, it did not work for other cases like members with no transverse reinforcement. Hence, Vecchio and Collins (1986) developed the Modified Compression Field Theory(MCFT) for widening the scope of applicability. Thürlimann et al. (1983) and Nielsen(1984) introduced plasticity methods for predicting shear strength. Later in the late1990’s, shear friction theory and general friction theory and design with strut-and-tie system were developed (Anuja, 2006).

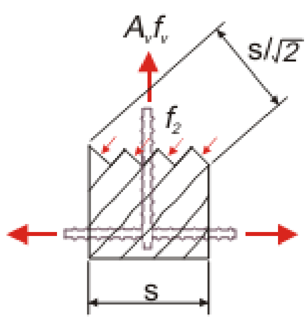
2.2.2 45◦ Truss Model



(a) Cross section



(b) Diagonal stresses and longitudinal equilibrium



(c) Stirrup force

Figure 2.17 . Equilibrium conditions for 45◦ truss model (Collins and Mitchell, 1991)

Truss models were widely used to understand shear behavior of reinforced concrete beams in the early 1900's. Ritter (1899) used a 45∘ truss model for the analysis of the post-cracking behavior of a reinforced concrete beam. In his model, diagonal concrete struts were considered to be the diagonal members of the truss, the stirrups were the vertical members of the truss, the longitudinal reinforcement served as the bottom chord of the truss, and the flexural compression zone served as the top chord of the truss. Mörsch (1909) improved this model by assuming that the diagonal struts extended across more than one stirrup. The tensile stresses in cracked concrete were neglected in this model and diagonal compression stresses were assumed to remain at45∘ after the concrete cracked.

Equilibrium equations for this model, assuming an angle of diagonal compression of θ =45∘ , are shown in Figure 2.17. Assuming a uniform distribution of shear stresses in the effective web area, $b_w jd$, the vertical component of the diagonal compressive force must be balanced by the applied shear (Collins and Mitchell, 1991):

$$f_2 = \frac{2V}{bwjd} \tag{2.3a}$$

where,

f_2= principal compressive stress

V = applied shear

b_w = width of the web of the beam

jd = flexural lever arm (distance between compressive force and tensile force)

The horizontal component of the diagonal compressive force also must be balanced by the tension in longitudinal reinforcement:

$$N_v = V \tag{2.3b}$$

where,

N_v = tension in longitudinal direction

From Figure 2.17, the vertical component of the diagonal compression force must be balanced by the tension in the stirrups over the length jd. cot 45◦ = jd:

$$\frac{Av\, f_v}{s} = \frac{V}{jd} \qquad (2.3c)$$

where,

Av = tension in longitudinal direction

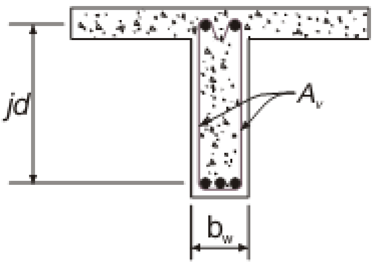
fv = stress in shear reinforcement

s = spacing of shear reinforcement

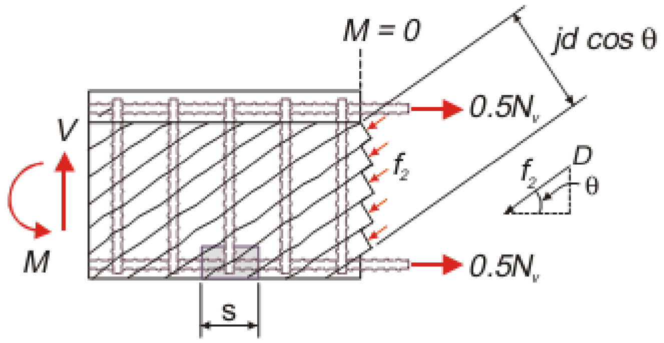
Equation 2.3c is used to design the required amount of stirrups. Equation 2.3a is used to check the compressive stresses in the concrete and this determines the upper limit of the shear force or capacity.

In the middle of the 1960's the 45◦ truss model was reexamined because it gives overly conservative results for predictions of the shear strength of members with shear reinforcement. The model lowers the effectiveness of the stirrups and, consequently, efforts were directed towards predicting the actual strut angle, which may be flatter than the angle of the inclined cracks (Anuja, 2006).

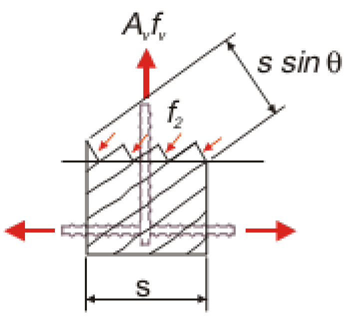
2.2.3 Variable-Angle Truss Model



(a) Cross section



(b) Diagonal stresses and longitudinal equilibrium



(c) Stirrup force

Figure 2.18 . Equilibrium conditions for variable-angle truss model (Collins andMitchell, 1991)

The variable-angle truss model is a version of the 45◦ truss model modified by assuming flatter strut angles, θ ≤ 45◦ (Collins and Mitchell, 1991). In this model, the three equilibrium equations can be derived in the same manner as for the 45◦ truss model. The equilibrium conditions for this model are shown in Figure 2.18. They are:

$$f_2 = \frac{V}{bwjd} \cdot \frac{1}{\sin\theta \ cos\theta} = \frac{V}{bwjd}\left(\tan\theta + \cot\theta\right) \tag{2.4a}$$

where,

f_2= principal compressive stress

V = applied shear

bw = width of the web of the beam

jd = flexural lever arm (distance between compressive force and tensile force)

θ = crack angle or the angle of compression strut

$$N_v = V \, cot \, \theta \tag{2.4b}$$

where,

N_v = tension in longitudinal direction

θ = crack angle or the angle of compression strut

$$\frac{Av\, f_v}{S} = \frac{V}{jd} \tan\theta \tag{2.4c}$$

where,

Av = tension in longitudinal direction

fv = stress in shear reinforcement

s = spacing of shear reinforcement

However, these three equilibrium equations are not sufficient to solve member forces, because there are four unknowns: the principal compressive stress, f_2 ;the tension in the longitudinal direction, N_v; the stresses in the shear reinforcement, f_2;and the strut angle or inclination of the principal compressive stresses, θ.

There have been different approaches to solve for this angle. Kupfer (1964) used minimum energy principles to determine the crack angle θ while assuming linear elastic behavior of both reinforcement and concrete. Baumann (1972a,b) continued this work, which was later taken up by Vecchio and Collins (1982, 1986) in their compression field theory. The traditional truss model assumes that the stirrups yield(i.e., $f_v = f_y$)and θ =45◦ , and uses Equation 2.2c. Plasticity methods assume the yield of the stirrups (i.e., $f_x = f_v = f_y$) and that the maximum compressive stress,$f_2$is attained. However, the lower and the upper limit of angle θ is usually specified in the model (Anuja, 2006).

2.2.4 Compression Field Theory (CFT)

The Compression Field Theory (CFT) uses the same approach for equilibrium conditions as described in the variable-angle truss model. Equations 2.4a, 2.4b, and 2.4ccan be expressed respectively in terms of the stresses as shown below (Mitchell and Collins, 1974). These equilibrium equations can also be derived from Figure 2.19(a) and (b):

$$f_2 = v(\tan\theta + \cot\theta) \qquad (2.5a)$$

where,

f_2 = principal compressive stress

v = shear stress

θ = crack angle or the angle of compression strut

$$\rho_x f_{sx} = v\cot\theta \qquad (2.5b)$$

where,

ρ_x= longitudinal reinforcement ratio

f_{sx}= stress in longitudinal reinforcement

$\rho_y f_{sy} = v \tan\theta$ (2.5c)

where,

ρ_y= shear reinforcement ratio

f_{sy}= stress in shear reinforcement

In determining the crack angle θ in the variable-angle truss model, Wagner (1929) provided an important fundamental in his "tension field theory". In his shear design of thin "stressed-skin" aircraft, he assumed shear would be carried by a diagonal tension field after buckling of the thin metal web. Then, he considered the deformations of the system by assuming that the angle of inclination of the diagonal tensile stresses would coincide with the angle of the inclination of the principal tensile strain.

Similar to the tension field theory, CFT utilizes the deformations for reinforced concrete by assuming that a diagonal compression field would carry shear after cracking. The compatibility condition used in the compression field theory can be derived from Mohr's circle for strains, as shown in Figure 2.19(c and d).

From the Figure 2.19(c) and (d), the relationship between crack angle θ and the strains can be expressed as follows:

$$\tan 2\theta = \frac{\gamma xy}{\varepsilon y - \varepsilon x} \quad (2.6)$$

Where,

γ_{xy} = shear strain

ε_x = strain in horizontal x direction

ε_y = strain in vertical y direction

The relationship between crack angle θ and the strains can also be expressed as:

$$\tan^2\theta = \frac{\varepsilon x - \varepsilon 2}{\varepsilon y - \varepsilon 2} \quad (2.7)$$

Where,

$\varepsilon 2$ = principle compressive strain

Equation 2.7 is Wagner's (1929) and Baumann's (1972a; 1972b) compatibility equation, which can be applied in cracked concrete using average strains. From Equation 2.7, the influence of the crack angle θ on strains can be observed. For steep crack angles, the longitudinal strain becomes high and for flattened crack angles, the transverse strain becomes high.

As shown in Figure 2.19(e), the stress-strain relationships for both longitudinal and transverse reinforcement were assumed as bilinear in this approach. Stress-strain relationships for cracked concrete in compression were proposed by Collins (1979),based on experimental test results, as follows:

$$f_2 = \left(\frac{f'_c}{\varepsilon'_c}\right) . \varepsilon_2 \ \le f_{2\max} = \frac{3.6\, f'_c}{1 + \frac{2(\varepsilon_1 + \varepsilon_2)}{\varepsilon'_c}} \qquad (2.8)$$

Where,

f_2 = principal compressive stress

f_{2max} = maximum compressive stress, a test parameter in softening of compressive strength proposed by Vecchio and Collins (1986)

f'_c= compressive strength of concrete

ε'_c = strain corresponding to f'_c in a cylinder test

ε_1 = principle tensile strain

ε_2 = principle compressive strain

In Equation 2.8, the softening of the compressive strength in cracked concrete is expressed in terms of the principal tensile strain, ε_1 . The principal strain, ε_1 ,can be derived from Figure 2.19(d) and Equation 2.8 as:

$$\varepsilon_1 = \varepsilon_x + \varepsilon_y - \varepsilon_2 = \varepsilon_x + (\varepsilon_x - \varepsilon_2)\cot^2 \theta \qquad (2.9)$$

From Equation 2.9, the longitudinal strain, ε_x, can be expressed as:

$$\varepsilon_x = (\varepsilon_1 \tan^2 \theta + \varepsilon_2)/(1 + \tan^2 \theta) \qquad (2.10)$$

A similar expression for ε_y can be made from Equations 2.9 and 2.10 as:

$$\varepsilon_y = (\varepsilon_1 + \varepsilon_2 \tan^2 \theta)/(1 + \tan^2 \theta) \tag{2.11}$$

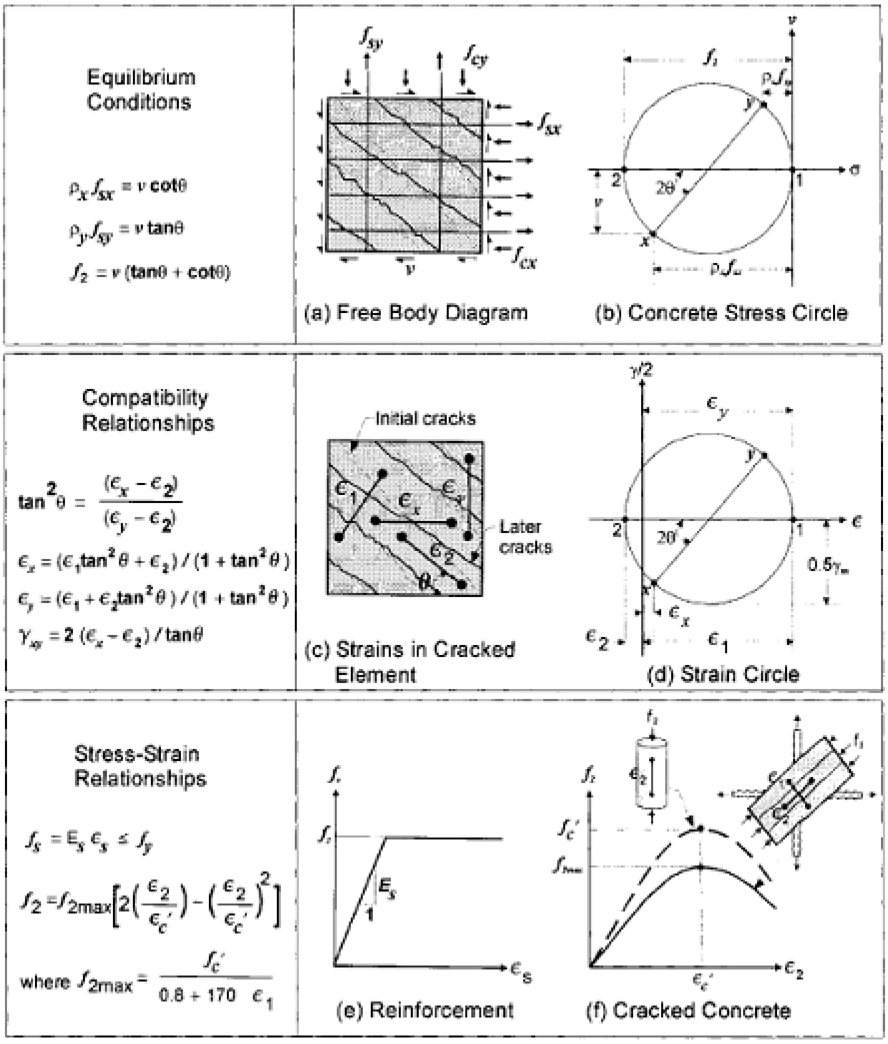


Figure 2.19 . Basic relationships for compression field theory (Mitchell and Collins,1974)

Based on the experimental findings, Equation 2.8 was refined by Vecchio and Collins (1986) to be as follows

$$f_2 = f_{2max}\left[2\left(\frac{\varepsilon 2}{\varepsilon' c}\right) - \left(\frac{\varepsilon 2}{\varepsilon' c}\right)^2\right] \tag{2.12}$$

such that,

$$\frac{f2max}{f'c} = \frac{1}{0.8+170\varepsilon1} \leq 1.0$$

Since the CFT provides the equilibrium conditions, compatibility conditions, and constitutive relationships for reinforcement and cracked concrete, it can predict shear behavior for any load level as well as the shear strength of members. However, since the CFT neglects the tensile stresses in cracked concrete, it gives conservative results on the shear behavior of members, meaning that it underestimates both shear stiffness and shear strength (Anuja, 2006).

2.2.5 Modified Compression Field Theory (MCFT)

The tensile stresses in cracked concrete provide significant shear resistance. The Modified Compression Field Theory (MCFT) accounts for the influence of tensile stresses on the post-cracking shear behavior. The equilibrium equations for the MCFT can be derived in a similar manner to those for CFT with a concrete tensile stress term f_1 added (Vecchio and Collins, 1986).

Considering the concrete tensile stress $f_1$in Equations 2.5a, 2.5b, and 2.5c, the equilibrium equations are:

$$f_2 = v(\tan\theta + \cot\theta) - f_1 \qquad (2.13a)$$

where,

f_1 = principal tensile stress

f_2= principal compressive stress

v = shear stress

θ = crack angle or the angle of compression strut

$$\rho_x f_{sx} = v \cot\theta - f_1 \qquad (2.13b)$$

where,

ρ_x= longitudinal reinforcement ratio

f_{sx} = stress in longitudinal reinforcement

$\rho_v f_{sy} = v \tan \theta - f_1$ (2.13c)

where,

ρ_y = shear reinforcement ratio

f_{sy}= stress in shear reinforcement

It should be noted that these conditions are expressed in terms of average stresses. The average principal tensile stress after cracking, f_1 , was suggested by Collins and Mitchell (1991) to be as follows:

$f_1 = \frac{fcr}{1+\sqrt{500\varepsilon 1}}$ (psi) (2.14)

where,

$f_{cr} = 4\sqrt{f'c}$

$f'c$ = compressive strength of concrete

ε_1 = principle tensile strain

It can be seen that the average principal tensile stress f_1 decreases as the average principal tensile strain ε_1 increases.

Both the CFT and MCFT can predict the shear behavior of members with shear reinforcement for all loading histories. However, the CFT predicts no shear strength for those members without shear reinforcement as it neglects the contribution of tensile stress in cracked concrete. The MCFT can predict shear behavior even for those members without shear reinforcement since it accounts for the tensile stress in cracked concrete (Anuja, 2006).

2.2.6 Review of Related Work

Prior to cracking, the maximum shear stress at the web can be calculated by using the traditional theory for homogeneous, elastic and un cracked beams, developed by the 35year-old Russian railway engineer D.J. Jourawski in 1856 (Collins, 2001):

$\tau = \frac{VQ}{Ib}$ (2.15)

Where I is the moment of inertia of the cross section, Q the first moment about the censorial axis of the part of the cross-sectional area lying farther from the censorial axis than the point where the shear stresses are being calculated, and b the width of the member where the stresses are being calculated.

Figure 2.20 shows the principal compressive stress trajectories in an un cracked beam and a photograph of a cracked reinforced concrete beam. Although there is a similarity between the planes of maximum principal tensile stress and the cracking pattern, they are by no means exactly alike. The flexural cracking, which precedes the inclined cracking, disrupts the elastic stress field to such an extent that inclined cracking occurs at a principal tensile stress, based on the un cracked section, of roughly a third of the tensile strength of the concrete (MacGregor and Bartlett 2000).

In 1902 Mörsch derived the shear stress distribution for a reinforced concrete beam containing flexural cracks. Mörsch predicted that shear stress would reach its maximum value at the neutral axis and would then remain constant from the neutral axis down to the flexural steel (Figure 2.21). The value of this maximum shear stress would be:

$$\tau = \frac{V}{bw\, z} \qquad (2.16)$$

where *bw* is the web width and z the flexural lever arm.

Mörsch recognized that this was a simplification, as some of the transverse force could be resisted by an inclination in the main compression, which would cause the ribs of the concrete between flexural cracks to bend, producing dowel forces in the main steel (Antoni, 2002).

Hamadi and Regan (1980), based on extensive experimental work on interface shear, published an analysis of a tooth model. It was assumed that the cracks were vertical and that their spacing was equal to half the effective depth of a particular beam. Reineck (1991) further developed the tooth model, taking all the shear transfer mechanisms into account, carrying out a full nonlinear calculation including compatibility. Reineck (1991), based on his mechanical model, derived an explicit formula for the ultimate shear force, which matched with the results of the test as well as with those of many empirical formulas (Antoni, 2002).

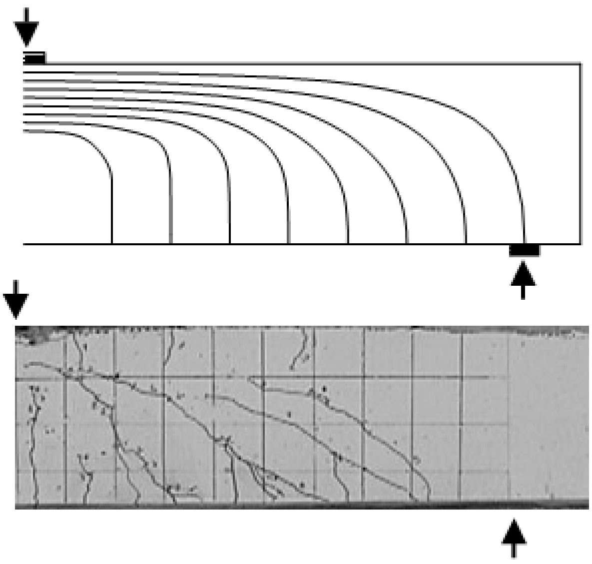


Figure 2.20: Principal compressive stress trajectories in an un cracked beam and photograph of a cracked reinforced concrete beam.

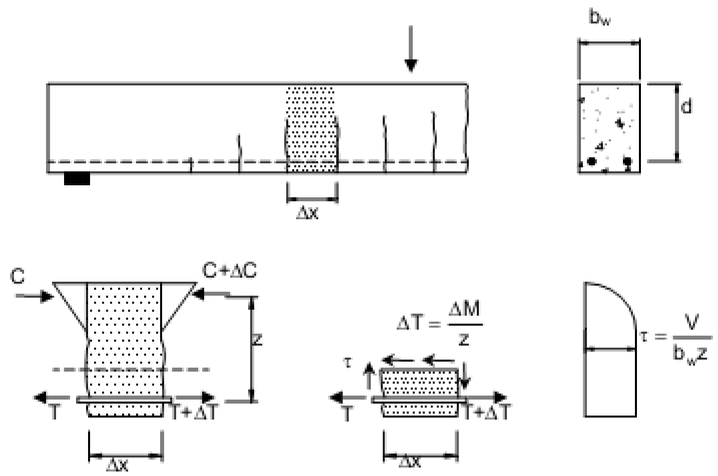


Figure 2.21: Shear stress distribution in a reinforced concrete beam with flexural cracks (adapted from Collins and Mitchell, 1997).

The application of simple strut-and-tie models, which have their theoretical basis in the lower-bound theorem of plasticity, requires a minimum amount of distributed reinforcement in all directions to ensure sufficient ductility in order for internal stresses to be redistributed after cracking. However, it is possible to extend this simple strut-and-tie model to members without web reinforcement by using a clearly different approach. Marti (1980) extended the plasticity approach by using a Coulomb-Mohr yield criterion for concrete that includes tensile stresses. In 1987, Schlaich suggested a refined strut-and-tie approach that includes concrete tension ties. Reineck showed that such truss models comply with the tooth model he had proposed (Antoni, 2002).

Empirically derived equations have been very important in the development of procedures used for designing members without transverse reinforcement. The simplest lower-bound average shear stress at diagonal cracking is given by the equation

$$\frac{Vc}{bd} = \tau = \frac{\sqrt{f'c}}{6} \qquad (2.17)$$

This well-known ACI equation, basis for the Spanish EH-91 shear provisions, is a reasonable lower bound for smaller slender beams that are not subjected to axial load and have at least 1% longitudinal reinforcement (ACI-ASCE Committee 445, 1998).However, it may be un conservative for lowly-reinforced members and high-strength concrete members.

The CEB-FIP Model Code (1990) suggests a more sophisticated empirical formula based on Zsutty's (1968, 1971) equation and adding an extra term to account for the size effect (equation 2.18). It should be noted that the formula implicitly includes the concrete safety factor. To disregard this factor, we should use 0.15 as the constant rather than 0.12.

$$\frac{Vc}{bd} = 0.12\left(1 + \sqrt{\frac{200}{d}}\right)\left(\frac{3d}{as}\right)^{\frac{1}{3}}(100\rho s\; fck)^{\frac{1}{3}} - 0.15\ \sigma_{cd} \qquad (2.18)$$

where σ_{cd} equals N_d/A_c, N_d being the factored axial force that includes the prestress (tensile positive) force and Ac, the cross sectional area of the concrete .

Zsutty's equation took into account the influence of the compression strength of the concrete and the longitudinal reinforcement ratio. When the steel ratio is small, flexural cracks extend higher into the beam and open wider than would be the case with large values of ρ_w. (Antoni, 2002).

The MC-90 equation takes the influence of compression force as a factor. However, members without shear reinforcement subjected to large axial compression and shear may fail in a very brittle manner at the first instance of diagonal cracking (Gupta and

Collins, 1993). As a result, a conservative approach should be used for those members (Antoni, 2002).

Gastebled and May (2001) recently developed a fracture mechanic model for the flexural-shear failure of reinforced concrete beams without stirrups. They assumed that that the ultimate shear load is reached when a splitting crack at the level of the longitudinal reinforcement starts to propagate. If we adopt the format of the CEB-FIP formula, their equation becomes

$$\frac{Vc}{bd} = 0.15\ \frac{37.41}{\sqrt{d}}\left(\frac{3d}{as}\right)^{\frac{1}{3}}(100\rho s)^{\frac{1}{6}}\left(1-\sqrt{\rho s}\right)^{\frac{2}{3}} f'c^{0.35} \qquad (2.19)$$

It is worthy a mention that the analytical and the empirical formulas compare very well (Gastebled et al. 2001). However, Gastebled's equation gives more importance to the size effect than the CEB-FIP formula does (Antoni, 2002).

Other different fracture mechanic models have been proposed to account for the fact that a peak tensile stress is near the tip of a crack and a reduced tensile stress (softening) is located in the crack zone. This approach offers a possible explanation for the size effect in shear. Two well-known models are the fictitious crack model (Hillerborg et al. 1976), and the crack band model (Bazant and Oh, 1983).

ASCE-ACI Committee 445 (1998) emphasized that, although the refined tooth models and the modified compression field theory take different approaches to the problem, the end result of these two methods is very similar for members without transverse reinforcement. Both methods consider that the ability of diagonal cracks to transfer interface shear stress plays an important role in the determination of the shear strength of members without transverse reinforcement (Antoni, 2002).

Shear transfer across cracks by interlocking particles was first looked at, in detail, by Fenwick and Paulay in the late 1960's (Fenwick and Paulay, 1968). This was aggregate interlock action, and shear displacement (or shear slip) parallel to the direction of the crack was a prerequisite of shear transfer by aggregate interlock. The authors examined the principal mechanisms of shear resistance in reinforced concrete beams. Two of the parameters studied were the concrete strength and the crack width. The concrete strength ranged from about 20 MPa to 60 MPa. Based on tests done on concrete shearblocks, the authors found that there was a substantial reduction in shear transmitted by aggregate interlock action when the crack width was increased. Also, as the concrete strength was increased to 60 MPa, the shear transmitted across the cracks increased. However, it is important to note that the crack width for the latter tests was fixed at about 0.2 mm and there was no occurrence of the aggregate interlock action breaking down. The concrete shear blocks exhibited both shear and flexural cracks (Dino, 1999).

Carrasquillo et al (1981) examined the behavior and micro cracking of high strength concrete subjected to short-term loading. The authors concluded that high- strength concrete has much less micro cracking at al1 stress levels than normal strength concrete, but fails more suddenly with fewer planes of failure. The authors looked at the differences in the mechanical properties of 30, 50 and 70 MPa concretes in terms of formation and propagation of micro cracks. They found that under uniaxial compression, normal strength concrete developed highly irregular failure surfaces including numerous instances of bond failure between the coarse aggregates and mortar. Medium strength concrete developed a mechanism similar to the normal strength concretes but at a higher strain. The failure mode of high-strength concretes was typical of that of a nearly homogeneous material. Failure occurred suddenly in a vertical, nearly flat plane passing through the aggregate and the mortar (Dino, 1999).

Walraven performed experiments on concrete push-off specimens of various concrete strengths (Walraven et al 1987 and Walraven 1995). The highest concrete strength examined had a cube strength of 115 MPa. The crack width and normal stress were also varied in the test program to isolate each parameter. Figure 2.22 shows the shear stress and normal stress versus the shear slip for various crack widths. As can be seen in the figure, for a crack width of 1 mm and a shear slip of about 2 mm, the shear stress transmitted across the crack for the 59 MPa and the 115 MPa concretes was 6 MPa and 4 MPa, respectively. In general, Walraven found that the shear friction capacity of cracks in high-strength concrete is significantly reduced due to fracture of the aggregate (Dino, 1999).

It could be expected that the surface of a diagonal tension fracture in a high strength concrete beam would be relatively smooth, as obtained in uniaxial compression, and the smooth surface might be deficient in aggregate interlock which is an important component of shear resistance (Elzanaty et al 1986) .

In the present study, it is expected that the shear stress transmitted across the cracks will decrease as the concrete strength increases, which in turn, May be a prominent factor in the overall shear capacity of the high strength concrete specimens (Dino, 1999).

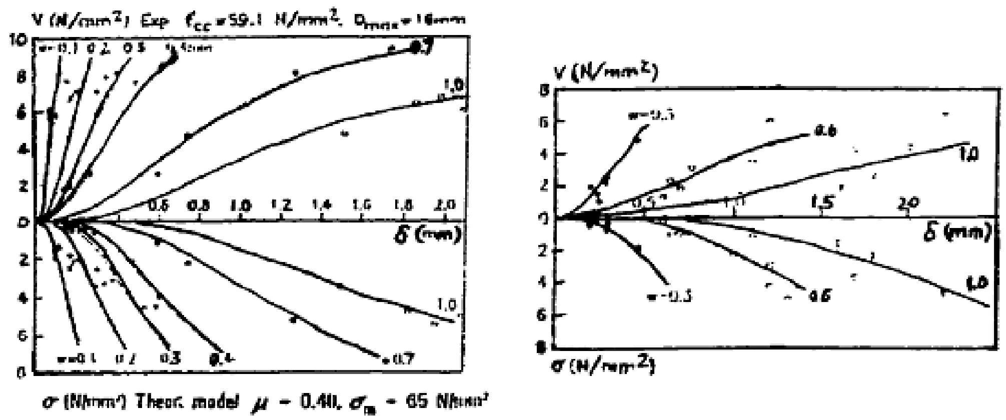


Figure 2.22 Shear stress, *v*, normal stress, σ, and shear displacement *D* at different constant crack widths, *w*, for 59 MPa (left) and 115 MPa concrete (right)

Eizanaty et al (1986) investigated the shear capacity of 18 reinforced concrete beams (3 of which contained stirrups) using high-strength concrete. The variables were the concrete strength, longitudinal reinforcement ratio, and shear span-to-depth ratio. The concrete strength ranged from 21 MPa to 83 MPa. The beams without stirrups were designed to investigate the effects of concrete strength, shear span-to-depth ratio, and percentage of longitudinal reinforcement. Those with stirrups had a constant value for the shear span-to-depth ratio and were designed to study the effects of concrete strength on the shear capacity of the beams (Dino, 1999).

The authors concluded that the shear strength of beams without stirrups increased when the concrete strength increased. However, they found that the crack surfaces were distinctively smoother for the higher strength concretes, indicating that the shear force carried by aggregate interlock decreased with increased concrete strength. The authors also concluded that for all concrete strengths, increasing the percentage of longitudinal reinforcement increased the shear strength of the test beams without stirrups. It was observed that beams with high-strength concrete and small amounts of longitudinal reinforcement had deficient dowel action, and splitting dong the reinforcing bars occurred suddenly (Dino, 1999).

Apart from varying the shear span-to-depth ratio, the present study examines the same parameters as the study by Elzanaty et al (1986). It is expected that the results will be very similar. One significant difference in the two experimental programs, however, is the size of the specimens. The beams tested by Elzanaty et al (1986) measured approximately 300 mm in height and 175 mm in width- The present study examines beams with a height of 1000 mm and a width of 300 mm. This size difference in the specimens will influence the crack widths. Large, lightly reinforced beams, relative to smaller ones at the same stress level, exhibit wider cracks. After cracking, shear is resisted by aggregate interlock, dowel action of the main reinforcing bars, and resistance of the still un cracked concrete at the top of the beam. If the cracks are wider, the aggregate interlock mechanism will not be as effective. Also, as the concrete strength increases and the crack surfaces become smoother and consequently more dowel action is required, the shear capacity of the large lightly reinforced members may not increase for higher concrete strength unless the cracks are contained, either by the addition of stirrups, for example, or increasing the percentage of longitudinal reinforcement (Dino, 1999).

It is expected that the beams in the present study will yield results very similar to those obtained by Stanik (1998). In this study, a wider range of concrete strengths will be examined, relative to that done by Stanik (1998), to try and establish an optimum concrete strength for the shear capacity of lightly reinforced concrete beams with and without stirrups. With the addition of transverse reinforcement it is expected that a sharp increase in strength and ductility will be evident relative to beams without stirrups for al1 concrete strengths. The variation of the longitudinal reinforcement from 0.5 % to 2 % in the present study may also shed more light on the effects of crack widths on the beam's shear capacity (Dino, 1999).

In 1955, the Wilkins Air Force Depot warehouse in Shelby, Ohio, collapsed due to the shear failure of 36 in. (914 mm) deep beams which did not contain any stirrups at the location of failure (Collins and Kuchma, 1997 and Collins and Mitchell, 1997). These beams had a longitudinal steel ratio of only 0.45%. They failed at a shear stress of only about 0.5 MPa whereas the AC1 Building Code of the time (AC1 Committee 3 18, 1951) permitted an allowable working stress of 0.62 MPa for the 20 MPa concrete used in the beams. Experiments conducted at the Portland Cement Association (Elstner and Hognestad, 1957) on 12 in. (305 mm) deep model beams indicated that the beams could resist about 1.0 MPa. However, the application of an axial tension stress of about 1.4 MPa reduced the shear capacity by about 50%. It was thus concluded that tensile stresses caused by thermal and shrinkage movements were the reason for the beam failures (Wassim, 1998).

Kani (1966 and 1967) was amongst the first to investigate the effect of absolute member size on concrete shear strength after the dramatic warehouse shear failures of 1955 (Collins and Kuchma, 1997 and Collins and Mitchell, 1997). His work consisted of beams without web reinforcement with varying member depths, *d*, longitudinal steel percentages, *p*, and shear span-to-depth ratios, a/d. He determined that member depth and steel percentage had a great effect on shear strength and that there is a transition point at a/d=2.5 at which beams are shear critical (i.e. the value of the bending moment at failure was minimum)(see Fig. 2.23).

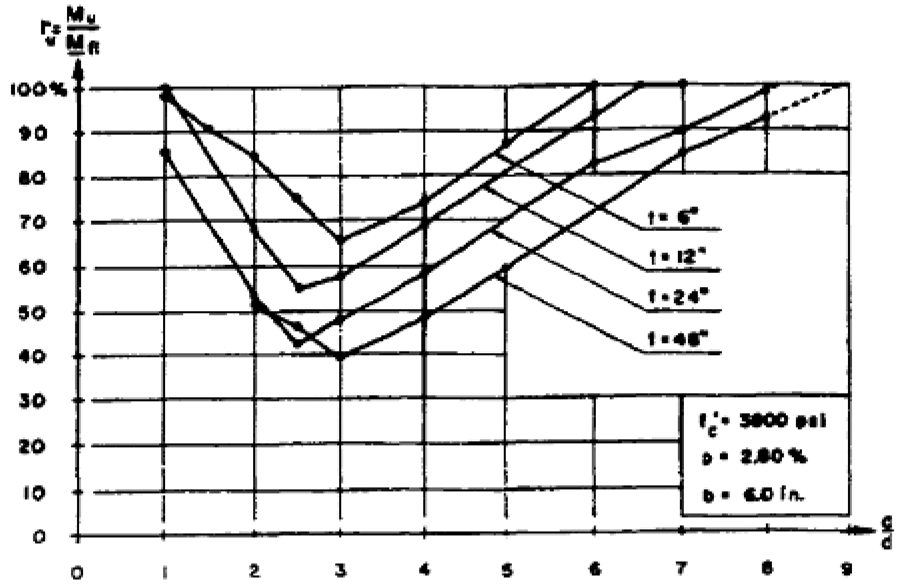


Figure 2.23: Relative strength (ultimate moment/flexural moment) vs. a/d ratio (Kani 1967)

Kani found this value of a/d to be the transition point between failure modes and is the same for different member sizes and steel ratios. Below an “a/d” value of about 2.5 the test beams developed arch action and had a considerable reserve of strength beyond the first cracking point. For “a/d” values greater than 2.5 failure was sudden, brittle and in diagonal tension soon after the first diagonal cracks appeared. This transition point is more emphasized in test beams containing higher reinforcement ratios and almost disappears in specimens with lower reinforcement ratios. In addition. Kani found a clearly defined envelope bounded by limiting values of p and a/d. Inside this envelope diagonal shear failures are predicted to occur and outside of this envelope flexural failures are predicted to occur. These conclusions regarding the influence of both p and a/d were similar for all beam depths tested. Kani also looked at the effect of beam width and found no significant effect on shear strength. Kani's work was summarized in the textbook "Kani on Shear in Reinforced Concrete" (Kani et al. 1979).

More recently, Bazant and Kim (1984) derived a shear strength equation based on the theory of fracture mechanics. This equation accounts for the size effect phenomenon as well as the longitudinal steel ratio and incorporates the effect of aggregate size. This equation was calibrated using 296 previous tests obtained from the literature and was compared with the AC1 Code equations. It was noted afler the comparison that the practice used in the AC1 Code of designing for diagonal shear crack initiation rather than ultimate strength does not yield a uniform safety margin when different beam sizes are considered. It was also found. According to the new equation. that for very large specimen depths the factor of safety in the AC1 Code almost disappears. However, no experimental evidence was available yet to confirm that fact as al1 the tests performed up to that time were on relatively small specimens. This equation was improved by Bazant and Sun (1987) to account for the maximum aggregate size distinctly from the size effect phenomenon and was extended to cover the influence of stirrups. This formula was calibrated using a larger set of test data consisting of 461 test results compiled from the literature (Wassim, 1998).

Later on, Bazant and Kazemi (1991) performed tests on geometrically similar beams with a size range of 1 : 16 and having a constant “a/d” ratio of 3.0 and a constant longitudinal steel ratio, p. Beams tested varied in depth from 1 inch (25 mm) to 1 6 inches (406 mm). The main failure mode of the specimens tested was diagonal shear but the smallest specimen failed in flexure. This study confirmed the size effect phenomenon and helped corroborate the previously published formula. However, the deepest beam tested was relatively small and the authors concluded that for beams larger than 16 inches (406 mm) additional reductions in shear strength due to size effect were likely (Wassim, 1998).

Kim and Park (1994) performed tests on beams with a higher than normal concrete strength (53.7 MPa). Test variables were longitudinal steel ratio, p, shear span-to-depth ratio, a/d, and effective depth. d. Beam heights varied from 170 mm to 1000 mm while the longitudinal steel ratio varied from 0.01 to 0.049 and a/d varied from 1.5 to 6.0. Their findings were similar to Kani's from which it was concluded that the behavior of the higher strength concrete is similar to that of normal-strength concrete. However, since only one concrete strength was investigated no general conclusions could be made with respect to concrete strength and shear capacity (Wassim, 1998).

Shioya (1989) conducted a number of tests on large-scale beams in which the influence of member depth and aggregate size on shear strength was investigated. In this study, lightly reinforced concrete beams containing no transverse reinforcement were tested under a uniformly distributed load. The beam depths in this experimental program ranged from 100 mm to 3000 mm. Shioya found that the shear stress at failure decreased as the member size increased and as the aggregate size decreased. It is interesting to note that the beams tested by Shioya contained about the same amount of longitudinal reinforcement as the roof beams of the Air Force warehouse which collapsed in 1955 (Collins and Kuchma, 1997 and Collins and Mitchell, 1997). The warehouse beams had an effective depth of 850 mm and failed at a shear stress of about $0.1\sqrt{f'c}$ MPa. This shear stress level corresponds with the failure shear stress observed in beams having a depth of 1000 mm in the Shioya tests. It is important to mention that there was a tendency for reduced shear stresses at failure even with tests including 3000 mm deep beams. Figure 2.24 illustrates the results obtained by Shioya (Wassim, 1998).

Mphonde and Frantz (1984) tested concrete beams without shear reinforcement with varying a/d ratios from 0.0 15 to 0.036 and concrete strengths ranging from

21 to 103 MPa. They conclude that the effect of concrete strength becomes more significant with smaller a/d ratios and that failures became more sudden and explosive with greater concrete strength (Wassim, 1998).

Elzanaty et al. (1986) looked at the problem of shear in high-strength concrete and observed a smoother failure plane in the higher strength concrete specimens. Their study was performed on a total of 18 beams with concrete strengths, f'c, ranging from 21 to 83 MPa. Apart from concrete strength, test variables included p and the shear span-to-depth ratio, dd. The conclusions drawn from these tests were that the shear strength increased with increasing f'c but less than that predicted using the AC1 Code equations. These equations predict an increase in shear strength in proportion to $\sqrt{f'_c}$. Elzanaty et al. concluded that an increase in the steel ratio led to an increase in the shear capacity of the specimens regardless of concrete strength.

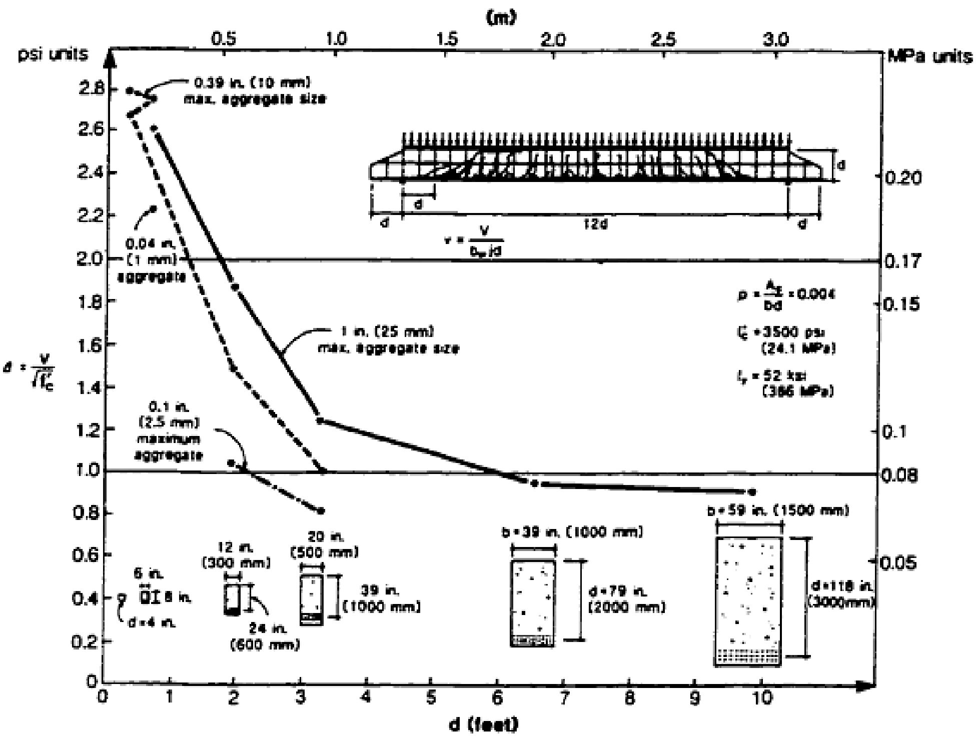


Figure 2.24: Influence of member depth and aggregate size on shear stress at failure for tests carried out by Shioya 1989. taken from Collins and Mitchell, 1997.

It was also found that there is a greater scatter in the results of specimens with small a/d ratios due to the possible variations in the failure modes (Wassim, 1998).

Ahmad et al. (1986) studied the effects of the a/d ratio and longitudinal steel percentage on the shear capacity of beams without web reinforcement. For their tests, the concrete strength was maintained as constant as possible with $f'c$ in the range of 63 to 70 MPa. Findings were similar to previous experiments with a transition in the failure mode at an a/d ratio of approximately 2.5. The envelope involving limits on a/d and p which separates shear failures from flexural failures was found to be similar to the envelope for the normal-strength concrete. However, more longitudinal steel was required to prevent flexural failures. Ahmad et al. found that the shear capacity was proportional to $f'c^{\,0.3}$ (Wassim, 1998).

2.3 Experimental Studies

In this study an extensive literature survey on experimental studies related to shear strength of RC beams without web reinforcement has been carried out and an experimental database has been constructed. A total of 398 tests from 46 separate studies were included in the database with ranges of variables as shown in details in Table 2.1

Table 2.1 Experimental database and Range Of Variables (ACI Structural Journal 100 (2003), No.2, March-April, 240-249; Karl-Heinz Reineck; Daniel A. Kuchma; Kang Su Kim; and Sina Marx)

Researcher	*N**	*b (mm)*	*d (mm)*	*f'c (Mpa)*	*Pl %*	*a/d*
Ahmad, Kahloo (1986)	16	127	184 to 208	59.3 to 65.5	1.77 to 6.62	2.7 to 4
Aster. Koch (1974)	9	1000	250 to 750	18.9 to 29.5	0.42 to 0.91	3.65 to 5.50
Bhal (1968)	8	240	300 to 1200	22.5 to 28.8	0.63 to 1.26	3.0
Bresler, Scordelis (1963)	3	305 to 310	461 to 466	21.4 to 35.9	1.81 to 2.73	3.97 to 6.93
Chana (1981)	3	203	356	31.2 to 37	1.74	3.0
Cossio. Siess (1960)	5	152	254	18.5 to 29.9	0.98 to 3.33	3.0 to 5.0
Kani (1967)	33	150 to 612	133 to 1097	23.5 to 28	2.59 to 2.84	2.41 to 8.04
Krefeld, Thurston (1966)	28	152 to 254	238 to 483	15.9 to 36.5	1.34 to 4.51	2.89 to 6.09
Collins, Kuchma (1997, 1998)	6	300	925	34.2 to 93.1	1.01	2.92
Podgornaik-Stanik (1998)	7	300	110 to 925	35.2 to 94.1	0.50 to 0.91	2.92 to 3.07
Elzanaty, Nilson, Slate (1986)	11	178	267 to 273	19.7 to 75.3	1.0 to 3.30	4.0 to 6.0

Feldman. Siess (1955)	4	152	252	24.5 to 34.9	3.35	3.02 to 6.04
Grimm (1997)	12	300	146 to 746	85.6 to 105.4	0.83 to 4.22	3.53 to 3.90
Hallgren (1994, 1996)	22	150 to 337	191 to 211	29.5 to 87.8	0.57 to 4.10	2.61 to 3.66
Laupa, Siess (1953)	6	152	262 to 269	14 to 30.7	1.90 to 4.11	4.54 to 4.61
Leonhardt, Walther (1962)	27	100 to 502	140 to 600	12.2 to 37.2	0.91 to 2.07	2.46 to 6.0
Mathey, Watstein (1963)	9	203	403	22.3 to 29	0.47 to 2.55	2.84 to 3.78
Morrow, Viest (1957)	11	305 to 308	356 to 375	14 to 43.4	1.28 to 3.92	2.76 to 5.87
Hamadi, Regan (1980)	3	100	370 to 372	20.9 to 28.8	1.08 to 1.70	3.44 to 5.97
Hanson (1958, 1961)	10	152	267	19.9 to 70	1.25 to 4.99	2.48 to 4.95
M phonde, Frantz (1984)	14	152	298	20.1 to 91.7	2.34 to 3.36	2.50 to 3.58
Niwa, Yamada (1987)	3	600	1000 to 2000	23.4 to 25.8	0.14 to 0.28	3.0
Rajagopalan, Fergison(1968)	10	151 to 154	259 to 268	22.5 to 34.7	0.25 to 1.73	3.83 to 4.27
Reinock, Koch, Schlaich (1978)	3	500	225 to 226	23.5 to 24.6	0.8 to 1.37	2.50 to 3.50
Remmel(1991)	4	150	160 to 165	80.3 to 80.8	1.87 to 4.09	3.06 to 4.0

Ruesch, Haugli (1962)	3	90 to 180	111 to 262	22 to 23.1	2.64 to 2.65	3.6 to 3.62
Scholz (1994)	3	200	362 to 372	76.6 to 92	0.81 to 1.94	3.0 to 4.0
Taylor (1968, 1972)	12	200 to 400	370 to 930	20.9 to 40.8	1.03 to 1.54	2.47 to 3.02
Walraven (1978)	2	200	420 to 720	22.9 to 23.2	0.74 to 0.79	3.0
Thorenfeldt, Drangshold (1990)	14	150 to 300	207 to 442	51.3 to 92.8	1.82 to 3.24	3.0 to 4.0
Moody, et al. (1954)	22	152 to 178	262 to 272	14.6 to 39.1	1.6 to 2.37	2.95 to 3.41
Ferguson (1956)	1	101	189	27.8	2.08	3.23
Al-Alusi (1957)	4	330	127	24.2 to 27.2	2.62	3.4 to 4.5
Yoon, Cook, Mitchell (1996)	3	375	655	34.2 to 82.7	2.80	3.28
Ferguson. Thompson (1953)	20	432 to 483	159 to 210	16.6 to 43.1	2.51 to 4.76	3.0 to 4.48
Islam, Pam, Kwan (1998)	17	150	203 to 207	25.3 to 79.1	2.02 to 3.22	2.90 to 3.94
Xie, Ahmad, Yu (1994)	2	127	216	35.8 to 94	2.07	3.0
Angelakos, Bentz, Collins (2000)	7	300	925	20 to 76	0.50 to 2.09	2.92 to 3.02
Adebar, Collins (1996)	6	290 to 360	178 to 278	43.9 to 56	1.1 to 3.04	2.88 to 4.49

Thorenfeldt (1990)	2	150	207	55.1	3.23	3.0 to 4.0
Kung (1985)	5	140	200	18 to 19.1	0.56 to 1.82	2.50
Yoshida, Bentz, Collins (2000)	1	300	1890	34.2	0.49	2.86
Cederwall, Hedman, Losberg (1974)	1	135	234	27.8	1.07	3.42
Kulkarni, Shah (1998)	3	102	152	38.5 to 41.4	1.38	3.50 to 5.0
Lambotte, Taerwa (1990)	2	200	415	32.3 to 35.3	0.97 to 1.45	3.01
Marti, Pralong, Thurlimann (1977)	1	400	162	28.1	1.38	3.95

N*: Number of test specimen

2.4 Code Review

2.4.1 Introduction

The theoretical models for shear behavior in concrete have been discussed in the previous section. In this section, the following national codes of practice based on these theoretical models are discussed: ACI 318-02 (2002); CSA A23.3-94(Canadian Standards: Design of Concrete Structures, 1994); CSA A23.3 2004 edition; Euro code EC2, part 1(1991) and (2003);and the German Code DIN 1045-1 (2001).

2.4.2 Design Codes

2.4.2.1 ACI 318-02

Non-prestressed members: As discussed in Section 2.2.2, when the 45^o truss model was introduced into the American literature, it was observed to be quite conservative. For example, the 45^o truss model predicts zero shear strength for beams without shear reinforcement, clearly underestimating the shear capacity for such beams. To account for the concrete contribution to shear resistance, as documented by the ASCE-ACI Committee 326 (1962), the concept of a concrete contribution Vc, was added to the steel contribution Vs,fromthe45^o truss, as shown in Figure 2.25

In 1963, ACI 318 set the concrete contribution equal to the shear at inclined cracking because beams without shear reinforcement often failed simultaneously with inclined cracking. The concrete contribution term Vc, for slender members has remained unchanged through ACI 318-02 (2002). Except for those members designed with strut and tie method, the nominal shear strength Vn, of non-prestressed members is the sum of the concrete contribution Vc, and shear reinforcement contribution Vs.Thus,

$$V_n = V_c + V_s \qquad (2.20)$$

where,

Vn = nominal shear strength of the member

Vc = concrete contribution towards shear strength

Vs = transverse steel contribution towards shear strength

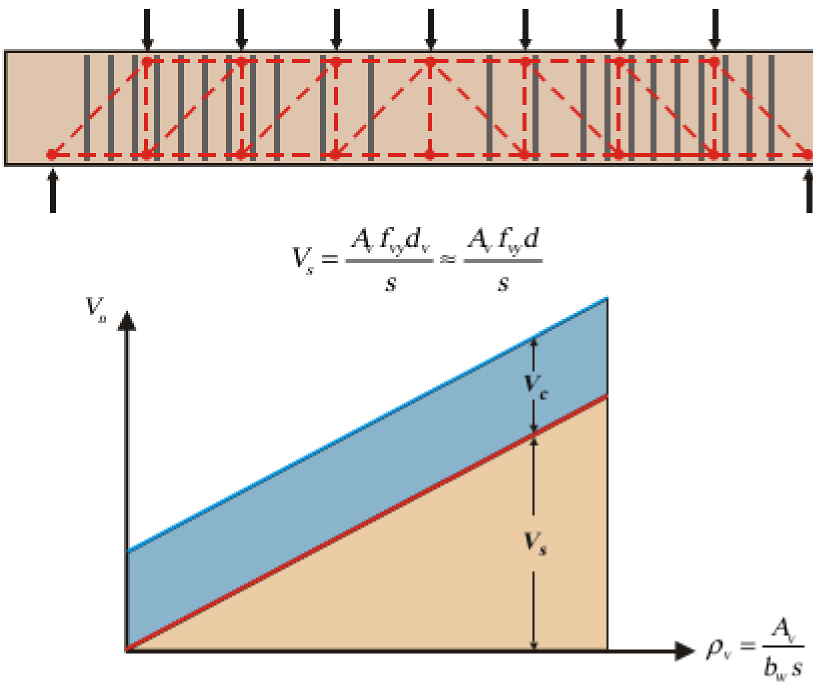


Figure 2.25 . Concrete and steel contributions on shear

The concrete contribution term *Vc*, can be calculated by either of following two equations:

$$Vc = 2\sqrt{f'_c}\, bwd \qquad \text{(Ibs)}$$

$$Vc = \frac{\sqrt{f'c}}{6}\, bwd \qquad \text{(Mpa)} \qquad (2.21)$$

where,

f'_c = compressive strength of concrete

bw = width of the beam

d = depth of the beam

$$Vc = \left(1.9\sqrt{f'_c} + 2500\,\rho l\,\frac{Vud}{Mu}\right) bwd \le 3.5\sqrt{f'_c}\, bwd \qquad \text{(Ibs)}$$

$$Vc = \left(\sqrt{f'_c} + 120\,\rho w\,\frac{Vud}{Mu}\right)\frac{bwd}{7} \le 0.30\sqrt{f'_c}\, bwd \qquad \text{(mm, Mpa)} \qquad (2.22)$$

such that $\frac{V_{ud}}{M_u} \le 1.0$

where,

ρl = longitudinal reinforcement ratio

Vu = factored shear force at section

Mu = factored moment at section

2.4.2.2 CSA A23.3-94 (Canadian Standards: Design Of Concrete Structures, 1994)

The Canadian Standards CSA A23.3-94 provides two methods for predicting the shear strength of reinforced concrete members, the Simplified Method and the General Method. The simplified method is similar to ACI 318 except that there is a consideration to account for the effect of member size. The general method is based on the modified compression field theory (MCFT) and thus has the same background as the AASHTO LRFD method. The Canadian code also recommends the use of the strut-and-tie method for the design of deep beams and other portion of members in which the variation of straining is complex- so called D region. The general method is not presented in this chapter.

The simplified method is based on the 45^0 truss model. The shear resistance is also divided in to two components, Vc and Vs. The concrete contribution, Vc, can be taken by:

$$Vc = 0.2\sqrt{f'_c}\,bwd\ (mm, Mpa) \tag{2.23}$$

$$when\ Av \geq \frac{0.06\sqrt{f'_c}\,bws}{fy}\ or\ d \leq 300\ mm$$

or $$Vc = \frac{260}{1000+d}\sqrt{f'_c}\,bwd \geq 0.1\sqrt{f'_c}\,bwd\ (mm, Mpa) \tag{2.24}$$

$$when\ Av < \frac{0.06\sqrt{f'_c}\,bw\,s}{fy}\ or\ d > 300\ mm$$

The steel contribution, Vs, is calculated as:

$$Vs = \frac{Avfvd}{s}\ where\ Vs \leq 0.8\sqrt{f'_c}\,bwd\ (mm, Mpa) \tag{2.25}$$

2.4.2.3 Euro code EC2 Part1 (1991)

The first version of Euro code EC2, Part 1 (1991) is partly based on the Plasticity Theory developed by Thürlimann (1975, 1983) and also by Nielsen (1984). It provides two methods, the standard method and the variable strut inclination method, The standard method is basically a combination of a concrete contribution term and a steel contribution term based on the 45^0 truss model. The method is applicable for concretes ranging from $12 \leq f'c \leq 50\ Mpa.$

In standard method the total shear resistance consists of the concrete contribution, V_{cd}, and the steel contribution, V_{wd}. Thus, the total shear resistance V_{Rd3} is:

$$V_{Rd3} = V_{cd} + V_{wd} \leq V_{Rd2,max} \tag{2.26}$$

Where

V_{cd} = concrete contribution and is taken as equal to V_{Rd1}

$V_{wd} = \frac{A_{sw} \cdot fy_{wd}(0.9d)}{s}$ is the steel contribution

A_{sw} = area of shear reinforcement within spacing, s

f_{ywd} = yield strength of shear reinforcement

$V_{Rd2,max}$ = upper limit of the shear resistance as a result of prevent web crushing

V_{Rd1} is the shear capacity of members without shear reinforcement based on an empirical formula:

$$V_{Rd1} = \beta\ \tau rd\ k(\ 1.2 + 40\rho l)bwd\ (mm, Mpa) \tag{2.27}$$

Where

$\beta = \frac{2.5\ d}{x}, (1.0\ \leq\ \beta\ \leq 5.0)$ is an enhancement factor that can be applied if the member is loaded by a concentrated load situated at a distance, $x \leq 2.5$ d, from the face of the support. Otherwise, $\beta = 1.0$.

τ_{rd} = basic design shear strength(=0.25 $f_{ctk0.05}$)

$f_{ctk0.05}$ = lower 5% fractile characteristics tensile strength (=0.7 f_{ctm})

f_{ctm} = mean value of the tensile concrete strength(=0.30(f_{ck})$^{2/3}$)

f_{ck} = characteristic cylinder compressive strength of concrete ($\cong 0.9 f'_c$)

k = (1.6 – d/1000) $\geq$ 1.0 (mm0

$\rho_l = \frac{Asl}{bwd} \leq 0.02$

A_{sl} = area of longitudinal reinforcement in tension

b_w = effective web width

d= effective depth

Thus, Eq.(2.27) can be simplified as follows:

$$V_{Rd1} = 0.0525\, k\, \beta\, (f_{ck})^{2/3}\, (1.2+40\rho)\, b_w d \quad (mm, Mpa) \tag{2.28}$$

2.4.2.4 Euro code EN 1992-1-1 (2003)

The Eurocode EC2, part 1 (1991) was revised and the final draft was published in April 2003 for comments by the different nations. The Eurocode EC2 is applicable up to concrete strengths of f_{ck} = 90 MPa, which corresponds to about f'_c = 91.6MPa. The characteristic value fck for the cylinder compressive strength is defined as a 5% fractile, whereas f'_c is a 9% fractile, and the relation is $f_{ck} = f'_c - 1.6$ (MPa).

The format of the new EC2 is such that, for many applications, only recommended rules or values are given and the values used in different countries are subject to a National Annex. In the following the recommended values are given.

The design value for the uniaxial concrete compressive strength, based on f_{ck},is:

$$f_{cd} = \alpha_{cc} f_{ck} / \gamma_c \tag{2.29}$$

where,

f_{cd} = design value of uniaxial concrete compressive strength

f_{ck}= characteristic cylinder compressive strength of concrete

γ_c = partial safety factor for concrete (normally $\gamma c = 1.50$)

α_{cc} = coefficient taking account of long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied

The value of α_{cc} for use in each country should lie between 0.8 and 1.0 and may be found in its National Annex. The recommended value is 1.

The value of the design tensile strength, f_{ctd}, is defined as:

$$f_{ctd} = \alpha_{ct} f_{ctk,0.05} / \gamma_c \tag{2.30}$$

where,

f_{ctd}= design value of tensile strength of concrete

$f_{ctk,0.05}$ = lower 5% fractile characteristic tensile strength of concrete

α_{ct} = coefficient taking account of long term effects on the tensile strength and of unfavorable effects resulting from the way the load is applied

The value of α_{ct} for use in each country may be found in its National Annex. The recommended value is 1.0.

***Members Not Requiring Shear Reinforcement*:** The design value for the shear resistance $V_{Rd,c}$ is given by:

$$V_{Rd,c} = (0.12\ k\ (100\rho_l f_{ck})1/3 - 0.15\ \sigma_{cp})\ b_w d \tag{2.31}$$

where,

$V_{Rd,c}$ = shear resistance of members not requiring shear reinforcement (N)

k = parameter to account for size effect = $1 + \frac{\sqrt{200}}{d} \leq 2.0\ with\ d\ in\ mm$

ρ_l = longitudinal reinforcement ratio = $A_{sl}/b_w\ d \leq 0.02$

A_{sl} = area of the tensile reinforcement, which extends $\leq (l_{bd} + d)$beyondthesection considered (see Figure 2.26).

b_w = smallest width of the cross-section in the tensile area *(mm)*

d = effective depth of member, measured from the extreme compression fiber tothe centroid of longitudinal tension reinforcement

$\sigma_{cp} = N_{Ed}/A_c > -0.2f_{cd}$ *(MPa)*= axial stress in the cross-section due to loading or prestressing without considering eccentricity of applied force

N_{Ed}= axial force in the cross-section due to loading or prestressing (N)(N_{Ed}< 0 for compression).

A_c = area of concrete cross-section (mm^2)

V_{Sd} = design value of shear resistance

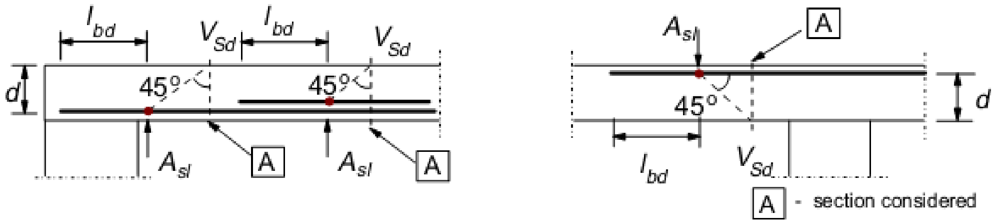


Figure 2.26 . Definition of *Asl* in Equation 2.31

2.4.2.5 German Code DIN 1045-1 (2001)

In 2001 the new German code DIN 1045-1 (DIN 1045-1, 2001) was published and thereby replaces the previous codes DIN 1045 (1988) for reinforced structures and DIN 4227 (1988) for prestressed concrete structures. The work on this code was parallel to that on the new EC 2 Code and many rules are similar. In the case of differences the new EC 2 allows different national application rules under same principles so that the new German code may also be applied when the Eurocode becomes effective.

The DIN is applicable up to concrete strengths of f_{ck} = 100 *MPa*, which corresponds to about $f'c$ = 101.6 *MPa*. As in the Eurocode, the characteristic value f_{ck} for the cylinder strength is defined as the 5% fractile, whereas $f'c$ is the 9% fractile, and the relation to $f'c$ is about $f_{ck} = f'c - 1.6$ (*MPa*). The design value for the uniaxial concrete compressive strength based on f_{ck} is:

$$f_{cd} = \alpha . f_{ck} / \gamma_c \qquad (2.32)$$

where,

f_{cd}= design value of uniaxial concrete compressive strength

f_{ck}= characteristic cylinder compressive strength of concrete

γ_c = partial safety factor for concrete (normally γ_c = 1.50)

α = coefficient taking account of long term effects on the compressive strength and the difference between cylinder strength and uniaxial compressive strength (prism strength). Higher values $\alpha < 1$ may be used, if justified, for short time loading (typically α =0.85)

Members Not Requiring Shear Reinforcement :

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

$$V_{Rd,ct} = [0.10\eta_1 \, k(100\rho_l f_{ck})^{1/3} - 0.12\sigma_{cd}] b_w d \qquad (2.33)$$

where,

$V_{Rd,ct}$ = shear resistance of members not requiring shear reinforcement

η_1 = parameter to account shear resistance of concrete having different weight

η_1 = 1 for normal concrete; and

η_1 =0.4+0.6ρ/2200 for lightweight concrete with ρ in kg/m^3

k = parameter to account for size effect = $1 + \frac{\sqrt{200}}{d} \leq 2.0$with d in mm

ρ_l = longitudinal reinforcement ratio = $A_{sl}/b_w\, d \leq 0.02$

A_{sl}= area of the tensile reinforcement, which extends d beyond the section considered and is anchored there effectively (see Figure 2.27).

b_w= smallest width of the cross-section in the tensile area (mm)

d = effective depth of member, measured from the extreme compression fiber to the centroid of longitudinal tension reinforcement

$\sigma_{cd} = N_{Ed}/A_c$ (MPa)= axial stress in the cross-section due to loading or prestressing without considering eccentricity of applied force (N_{Ed}< 0 for compression)

N_{Ed} = axial force in the cross-section due to loading or prestressing (N)(N_{Ed}< 0 for compression).

A_c = area of concrete cross-section (mm^2)

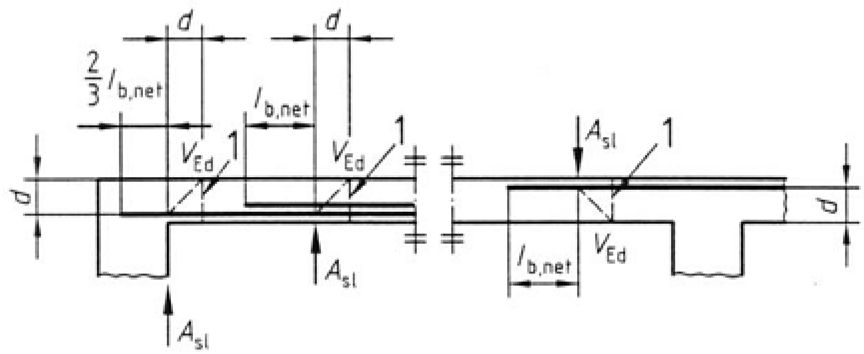


Figure 2.27 . Definition of A_{sl} for determining ρ_l in Equation 2.33 (1: section considered)

2.5 Future of research on shear design of RC members

Shear is one of the most researched properties of RC members in last 6 decades. Regan (1993), classified research on shear into three broad groups;

i- The first of kind of research relates to shear sensitive areas like shear in fire, shear connections between members, shear in high strength concrete and punching shear. This group of research aims at filling the knowledge gap in the above areas.

ii- The second group relates to understand the behavior of basic material at fundamental level. In this group of research, topics like " role of aggregate interlocking in shear" , " Size effect on shear" and other basic concepts of fracture mechanics related to shear are investigated. This group of research is related to more basic and fundamental topics in shear strength of RC members.

iii- The third group is engaged in translating the research results into a more meaningful tool for the building codes in the form of methods and rules for the shear analysis and design of RC members.

There is a general feeling in the minds of many researchers, that enough research has been carried out on this topic and there seems no more room for further research in this field. Regan (1993) tried to answer this basic question, where research on shear is waste of time or service to humanity? After reviewing the research of last 4-5 decades, Regan (1993), highlighted the significance of the research on the shear of RC beams in the following ways;

i- The research on shear for 40 years has enabled the structural engineers to design the RC members without web reinforcement, pre-stressed beams and flat slab buildings more accurately.

ii- The research on shear has been focused on making the design provisions of building codes more rational and comprehensive. Considerable achievements have been made in this direction. In these endeavors many misconceptions and doubts were also created, which were clarified in later works.

iii- Most of the proposed models developed in the meanwhile were based on the existing data but these models could poorly predict the behavior of actual beams, mainly due to the fact that important variables were not considered in the models at times.

iv- More experimental tests and researches are required for significant improvement in the shear design concept for its further rationalization involving parametric studies.

Despite of the fact that research on shear strength of RC beams, has been conducted for more than six decades, but even then the riddle of shear failure initiated by Kani(1964) is still unexplained. The exact behavior of RC concrete in shear is still an active area in contemporary research.

Mitchell et al. (2008) in a long term project, reviewed the results of 1849 tests on the shear strength of RC beams to judge the adequacy and safety provided by the shear equations used in North America. The findings of the research provide the latest state of research on the shear strength the some of the important findings and conclusions of the research of Mitchell et al (2008) are given as follows:

i- The traditional approach to design the shear reinforcing for the region where the external shear is exceeding the concrete shear capacity $V_c = 2\sqrt{f'_c}bwd$ may lead to un-conservative results and the chances of brittle failure may enhance. Hence there is a need to revised and rationalize the shear design equation of ACI and particularly the simplified shear design equation.

ii- The new load factors introduced in ACI-318-02, have led to increased flexural stresses in flexural reinforcement at service loads, which have further reduced the safety against shear failure.

iii- The design engineers must understand that the shear strength of RC beams is also affected by member depth, crack roughness and strain in longitudinal reinforcement, in addition to concrete strength.

iv- The recent research data shows that for RC members without web reinforcement, the influence of strain in longitudinal steel is more pronounced. High strength in the longitudinal steel and wider crack widths may decrease the shear strengths of RC members.

v- In high strength concrete with small aggregates sizes, the cracks surfaces are relatively smoother and can lead to reduction in the shear capacity of RC members. The equations based on the Modified Compression Field Theory (MCFT) accounts for the strain effect, size effect, and concrete strength in a reliable way, hence it can considered a suitable substitute of the traditional ACI equation. However the complexity in application of MCFT for the design of RC members would need further simplification

vi- An attempt to use the Simplified Modified Compression Field theory based equations, would reduce the complexity to some extent and it seems more advisable that the modified MCFT is used instead of traditional ACI equation, which would ensure ductile failure of RC structures and at the same time would also satisfy the basic ACI equation.

To sum up the literature review on the shear design of normal strength RC beams, we can infer that research on shear design of RC members will continue to be an area of interest for many young researchers to come and the riddle of shear failure will continue to be the focus of future research (Attaullah, 2009).

CHAPTER 3

EFFECT OF PARAMETERS ON SHEAR STRENGTH OF CONCRETE BEAMS

3.1 Introduction

As discussed in detail in Chapter 2, in spite of the development of many advanced theoretical models, realistic shear behavior of concrete is not captured. Since shear behavior is very complex, to improve our understanding, numerous studies were conducted throughout the world. If the database is compiled properly, they can provide additional insight in the problem. Hence, the development of a comprehensive database of shear test results was initiated. The resulting database is the largest database thus far created and is therefore being utilized to gain new insight into the factors that affect shear strength as well as to evaluate and compare models for shear behavior (Anuja, 2006).

The development of a comprehensive database of shear test results was initiated in order to provide the community with a resource for identifying research needs and for developing improved design code provisions. The resulting database is the largest database thus far created and therefore is being used to develop new insight in to the factors that affect shear strength and for the evaluation/comparison of models for shear behavior and of relationships for shear strength (Kang, 2004).

Existing empirical design code provisions do not provide uniform levels of safety against failure. One reason for this is that only a small portion of existing test results is typically used to evaluate or develop code provisions. Because the types of members in the test database do not well represent the types of structures that will be designed using these provisions, a more comprehensive database is needed.

One of the complications in developing this shear database was that only a brief summary of experimental test results is typically published in technical journals. Consequently, there is often insufficient information on geometric details and material characteristics. For this reason, it takes a considerable amount of time for researchers to do literature surveys, and consequently researchers often review only a limited number of test results before engaging in an experimental research program. Thus, researchers often repeat previous experiments and focus on studying a relatively limited number and range of influencing factors (Anuja, 2006).

3.2 Effect of Parameters

The database was used to investigate the influence of dominant parameters on shear strength. In this section, the shear strength, $v_u = V_u/(b_w d)$ or normalized shear strength, $v_u/f'_c = V_u/(b_w d f'_c)$, is plotted for each primary parameter, namely compressive strength f'_c, effective depth d, longitudinal reinforcement ratio ρ_l, shear span to depth ratio "*a/d*", and shear reinforcement strength $\rho_v f_{vy}$. Then, observations are noted from the analysis performed. This section presents those observations for RC members without shear reinforcement in this section, 398 RC members without shear reinforcement are studied.

3.2.1 Concrete Strength :

Figure 3.1 shows the relationship between the shear strength $v_{u,SR}$ and the concrete compressive strength f'_c for 398 RC members without shear reinforcement in the evaluation database. Most of the beams have ultimate shear stresses ranging from 0.31 to 3.69 Mpa. It is observed in most cases that the shear strength increases as the concrete strength increases. In most codes of practice, shear strength is proportional to the concrete strength to an exponent between 0.25 to 0.5.

3.2.2 Effective Depth :

Figure 3.2 shows the relationship between the shear strength vu,SR and the effective depth d for 398 RC members without shear reinforcement in the evaluation database. Although only a limited number of the tested beams had large depth, the shear strength clearly decreases as the effective depth of the member increases. It can be also seen that the shear strength of the members having effective depths less than 300 mm is very high. Many of the major codes and empirical equations account for a size effect in shear. In their expressions, the shear strength decreases as a function of the member depth. However, the expressions vary, ranging from "1/d,1/d1/2 ,1/d1/3 ,and1/d1/4" .The CSA considers the size effect relationship for members without shear reinforcement in a somewhat different manner by considering a crack spacing parameter, as described in Section 2.4.2.1 ACI 318-02 (2002) still does not include any consideration of the size effect.

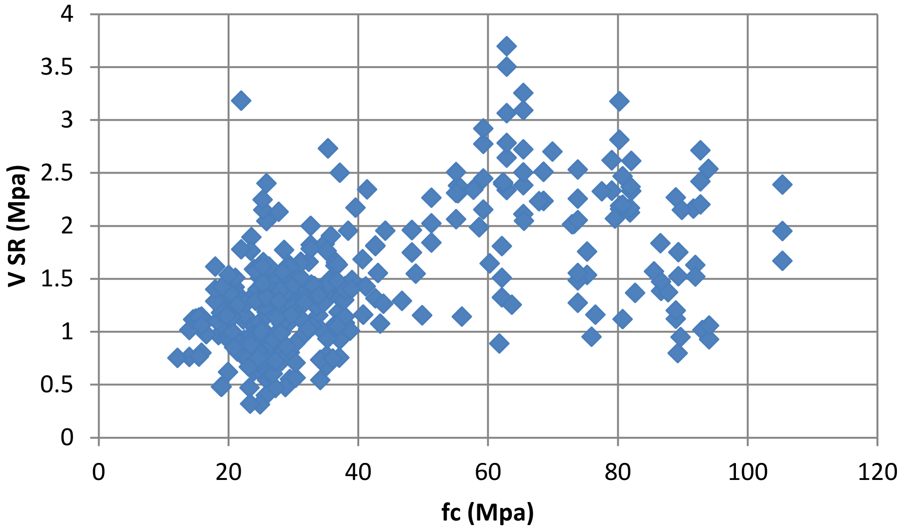


Figure 3.1 . Ultimate shear stress $v_{u,SR}$ at failure versus concrete compressive strength f'_c for RC members without shear reinforcement (sample size: 398 members)

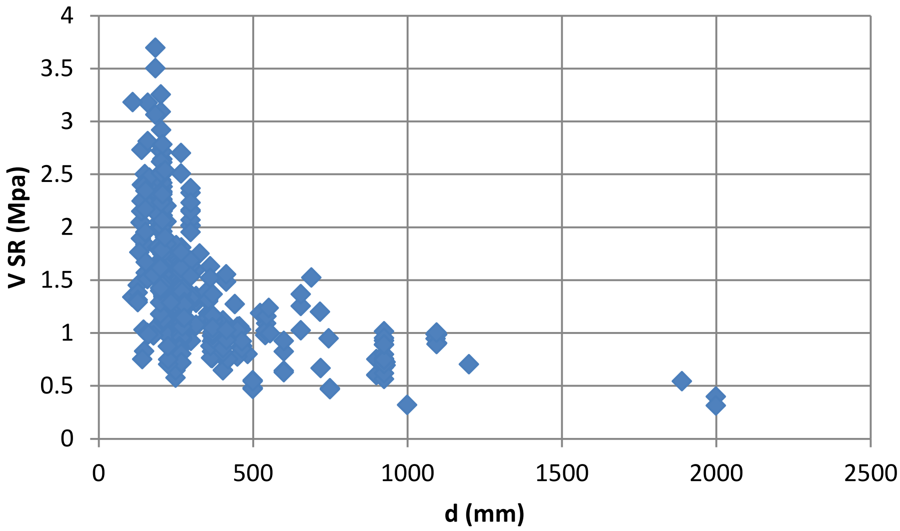


Figure 3.2 . Ultimate shear stress $v_{u,SR}$ at failure versus effective depth d for RC members without shear reinforcement (sample size: 398 members)

3.2.3 Longitudinal Reinforcement Ratio :

Figure 3.3 shows the relationship between the shear strength $v_{u,SR}$ and the longitudinal reinforcement ratio ρ_l for 398 RC members without shear reinforcement in the evaluation database. As can be seen in Figure 3.3, the shear strength clearly increases as the longitudinal reinforcement increases.

Most building codes or empirical formulae account for the influence of longitudinal reinforcement ratio directly or indirectly. For example, CSA considers the influence of longitudinal reinforcement by using the longitudinal strain, ε_x, which is a function of the longitudinal reinforcement amount as well as other sectional forces and sectional properties.

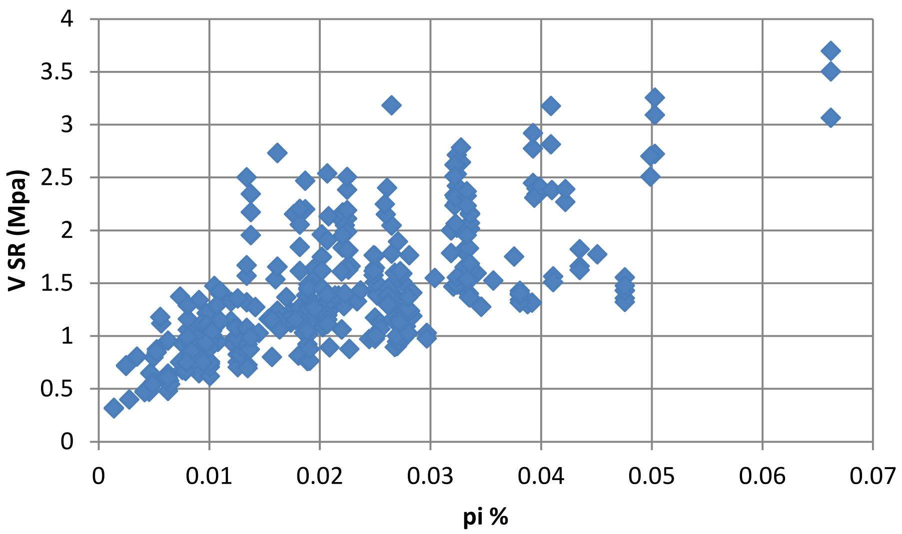


Figure 3.3 . Ultimate shear stress $v_{u,SR}$ at failure versus longitudinal reinforcement ratio ρl for RC members without shear reinforcement (sample size: 398 members)

3.2.4 Shear Span to Depth Ratio (Moment-shear ratio) :

Figure 3.4 shows the relationship between the shear strength $v_{u,SR}$ and the shear span to depth ratio (or moment-shear ratio) "*a/d*" for 398 RC members without shear reinforcement in the evaluation database. Many of the tested beams had an "*a/d*" ratio around 2.5. The shear strengths of members with an "*a/d*" ratio of 2.5 are clearly higher than that of the members with an "*a/d*" ratio around 3.0. This result is due to the beneficial effect of direct load transfer by arch action. From the Figure 3.4, it can be seen that most members having high shear strengths are heavily reinforced. Many code provisions and empirical equations consider a variable related to shear span to depth ratio, or moment to shear ratio, in their prediction of shear capacity. It should be again noted that the "*a/d*"ratio cannot be clearly defined for members subjected to uniformly distributed load, and thus it is more appropriate to use the moment to shear ratio in the design code expression than an" *a/d*"ratio.

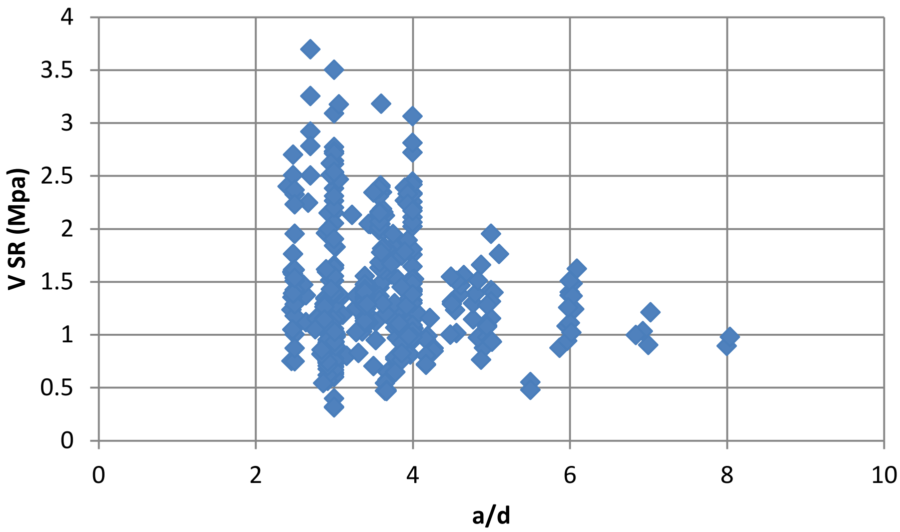


Figure 3.4. Ultimate shear stress $v_{u,SR}$ at failure versus shear span to depth ratio *a/d* for RC members without shear reinforcement (sample size: 398 members)

CHAPTER 4

NUMERICAL APPLICATION AND RESULTS

In this chapter, it is mentioned about calculation or modeling of shear strength of reinforced concrete beams without web reinforcement, current program used for finding the equation of shear strength (using stepwise regression), comparison of current design codes with the equation found by stepwise regression and finally, discussion about parametric study affected on shear strength.

4.1 Soft computing (stepwise regression)

While dealing with large number of independent variables, it is of significance importance to determine best combination of these variables to predict dependent variable. Stepwise regression serves as a robust tool for the selection of best subset models i.e. the best combination of independent variables that best fits the dependent variable with considerably less computing than is required for all possible regressions (Campbell, 2001).

The determination of subset models is based on adding or deleting the variable/variables with the greatest impact on the residual sum of squares. The selection of variables may be either forward, backward or a combination of them. In forward selection, the subset models are chosen by adding one variable at a time to the previously chosen subset. At each successive step, the variable in the subset of variables not already in the model that causes the largest decrease in the residual sum of squares is added to the subset. Without a termination rule, forward selection

Continues until all variables are in the model. On the other hand, backward stepwise selection of variables chooses the subset models by starting with the full model and then eliminating at each step the one variable whose deletion will cause the residual sum of squares to increase the least and continues until the subset model contains only one variable (Rawlings, 1998, Cevik, 2007). Regarding forward and backward procedures, it should be noted that the effect of adding or deleting a variable on the contributions of other variables to the model is not being considered. Thus stepwise regression is actually a forward selection process that rechecks at each step the importance of all previously included variables. If the partial sums of squares for any previously included variables do not meet a minimum criterion to stay in the model, the selection procedure changes to backward elimination and variables are dropped one at a time until all remaining variables meet the minimum criterion. Stepwise selection of variables requires more computing than forward or backward selection but has an advantage in terms of the number of potential subset models checked before the model for each subset size is decided. It is reasonable to expect stepwise selection to have a greater chance of choosing the best subsets in the sample data, but selection of the best subset for each subset size is not guaranteed. The stopping rule for stepwise selection of variables uses both the forward and backward elimination criteria. The variable selection process terminates when all variables in the model meet the criterion to stay and no variables outside the model meet the criterion to enter (Rawlings, 1998).

4.2 Comparison of Current Design Codes and Equations with stepwise Model

Using the experimental database given in table 2.1 stepwise regression (SR) models were constructed the results of the proposed SR model are also compared with results of current design codes and existing equations summarized in Table 4.1. The overall comparison of COV (coefficient of variation) of the proposed Stepwise model, current design codes and existing equations of the experimental database used in the study are given in Table 4.1. As seen from the results, the overall accuracy of the proposed Stepwise model is satisfactory compared to design codes and existing equations.

Table 4.1 Comparison of Current Design Codes and Equations with stepwise Model

No.	Name	Current Design Codes and Existing Equation	MEAN	R2	MSE	COV %
1	ACI-318-02 (2002) Eq.(11-3)	$V_c = \sqrt{f'_c}\,\frac{b.d}{6}$	1.37	0.684	5078.6	39.59
2	ACI-318-02 (2002) Eq.(11-5)	$V_c = \min\{(\sqrt{f'_c} + 120.\rho.\frac{d}{a})\frac{b.d}{7}\ ;\ 0.30\sqrt{f'_c}\ b.d\}$	1.41	0.706	3532.2	36.50
3	ASCE-ACI 445 (2003)	$V_c = 6.85(\frac{f'_c.\rho}{d})^{1/3} b.d$	1.46	0.760	1958.6	29.16
4	CSA (1994)	$V_c = 0.20\sqrt{f'_c}\ b.d$ $d \leq 300$ $V_c = \frac{260}{1000+d}\sqrt[2]{f'_c}\ b.d$ $d > 300$	1.19	0.738	2118.1	36.10
5	CSA (2005)	$V_c = 0.20\,\lambda\,\theta c\sqrt{f'_c}\ b.d$ $d \leq 300$ $V_c = \max\{\ \frac{260}{1000+d}\lambda\,\theta c\sqrt{f'_c}\ b.d$ $d > 300$ Or $0.10\,\lambda\,\theta c\ \sqrt{f'_c}\ b.d\ \}$ $\{\lambda = 1.0\ ;\ \theta c = 0.6\}$	1.99	0.737	2706.7	36.18

6	Euro Code (1992)	$V_{u,cd}=\tau RD\ K\ \beta\ (1.2+40\rho)\ b.d$ $V_{u,cd}=0.0525\ K\ (f_{ck})^{2/3}\ (1.2+40\rho)\ b.d$ Where $K=1.6-d$ (d in m) $fck \cong 0.9f'c$	1.01	0.628	2245.9	36.85
7	Euro Code EN 1992-1-1 (2003)	$V_c=[\ 0.12\ K\ (100.\rho.f_{ck})^{1/3}-0.15\sigma_{cp}]\ b.d$ $f_{ck} \cong 0.9f'c$ $k=1+\sqrt{\frac{200}{d}} \leq 2.0$ $\rho=\frac{Asl}{b.d} \leq 0.02$	1.61	0.808	1854.5	32.38
8	German Code DIN 1045-1 (2001)	$V_c=[\ 0.10\ \eta1\ K\ (100.\rho.f_{ck})^{1/3}-0.12\sigma_{cd}]\ b.d$ $f_{ck} \cong 0.9f'c$ $\eta1=1.0$ for normal concrete $k=1+\sqrt{\frac{200}{d}} \leq 2.0$ $\rho=\frac{Asl}{b.d} \leq 0.02$	1.94	0.808	2808	32.38

9	Zsutty (1968)	V_c=2.20 $(f'_c.\rho.\frac{d}{a})^{1/3}$. b.d $\frac{a}{d} \geq 2.5$ V_c=2.20 $(f'_c.\rho.\frac{d}{a})^{1/3}$. b.d .(2.5 $\frac{d}{a}$) $\frac{a}{d} < 2.5$	1.04	0.789	2764.4	31.03
10	Collins, Kuchma (1999)	$V_c=\frac{245}{1275+SE}\sqrt{f'_c}$ b.d $SE=\frac{35.Sx}{a+16}$ Sx=0.9 d a=10 mm	1.53	0.731	1870.3	35.40
11	Stepwise Regression	$V_c=6.405+76.3\sqrt[3]{f'_c.\rho}\,(\frac{b.d}{83206}+\frac{d}{a})$	0.99	0.82	862.17	27.31

4.3 Parametric Study

A wide range of parametric studies has been performed by using the SR model to investigate the interacting influence of each parameter on shear strength. Influence of *b*on the effects of *d*, f'_c, *%Reinforcement and a/d*on shear strength values are shown in figures 4.1 – 4.4. Influence of *d* on the effects of f'_c, *%Reinforcement* and a/d on shear strength values are shown in figures 4.5-4.7.Influence of f'_c on the effects of *%Reinforcement and a/d*on shear strength value sare shown in figures 4.8-4.9. Influence of *%Reinforcement* on the effect *a/d*on shear strength value is shown in figures 4.10.Also to investigate influence of each parameter on shear strength of RC beam without web reinforcement calculated by stepwise regression is shown in figure (4.11).

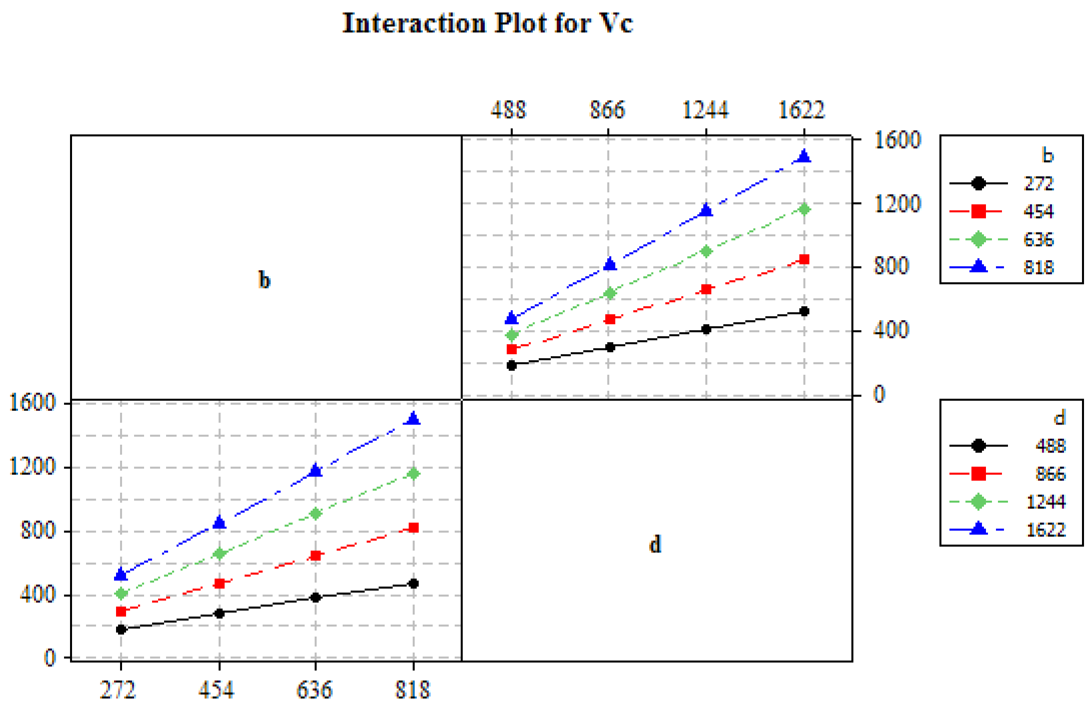


Figure 4.1. Response of Influence of (b) on the effect of (d) on the shear resistance force (V_c)

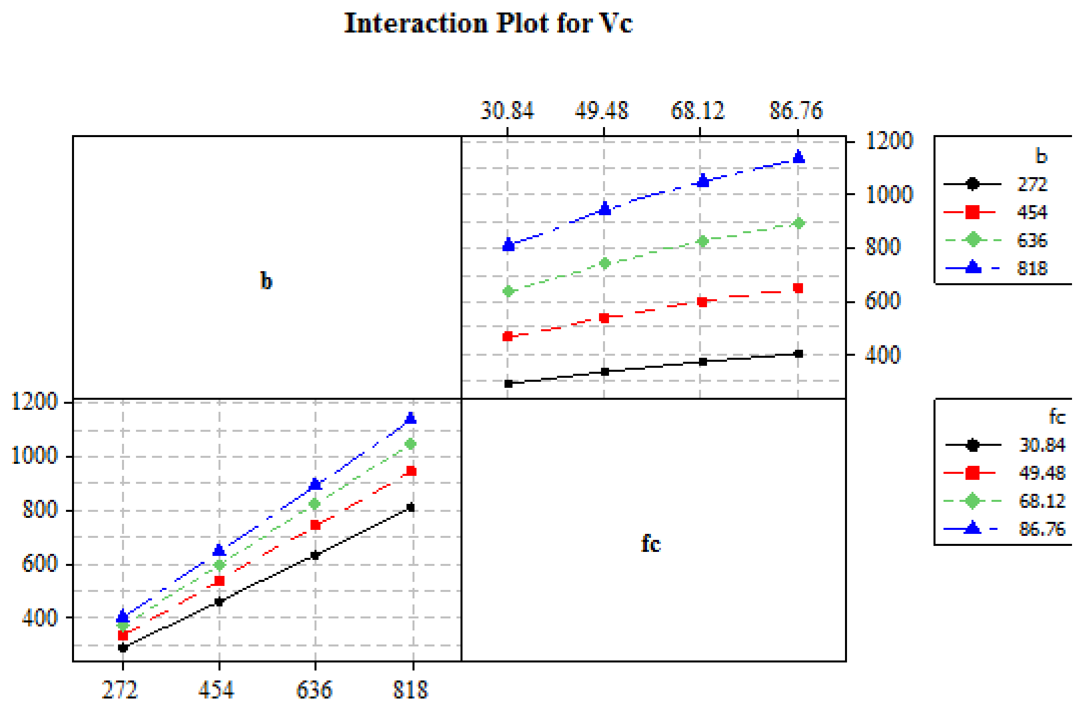


Figure 4.2. Response of Influence of (b) on the effect of (f_c) on the shear resistance force (V_c)

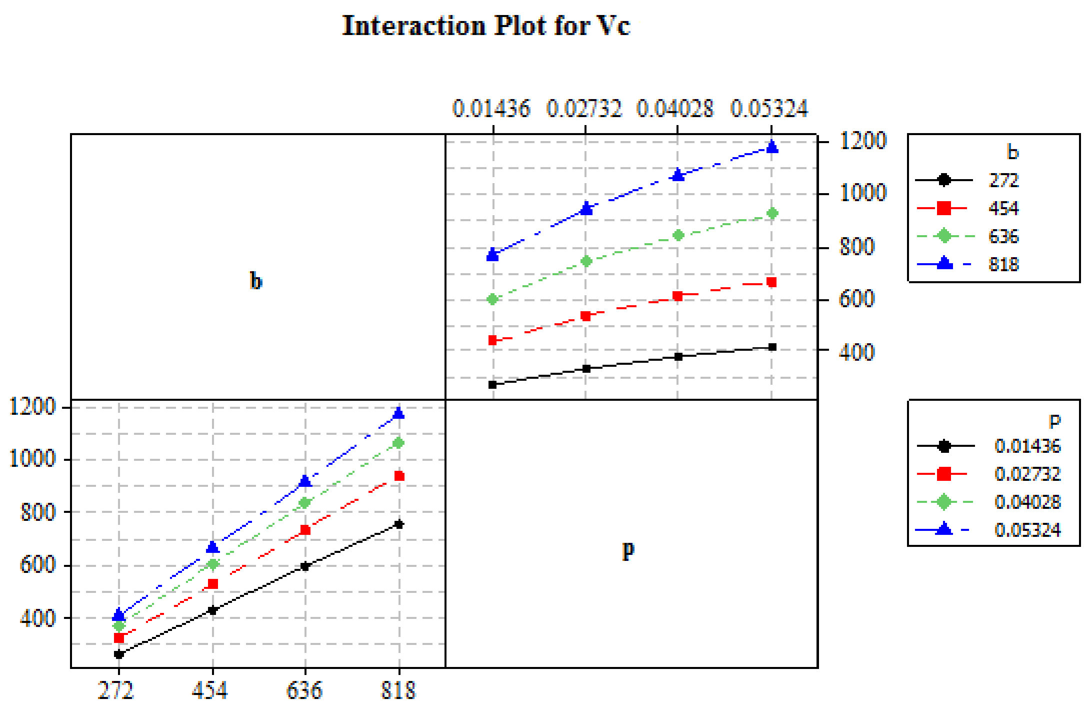


Figure 4.3. Response of Influence of (b) on the effect of (p) on the shear resistance force (V_c)

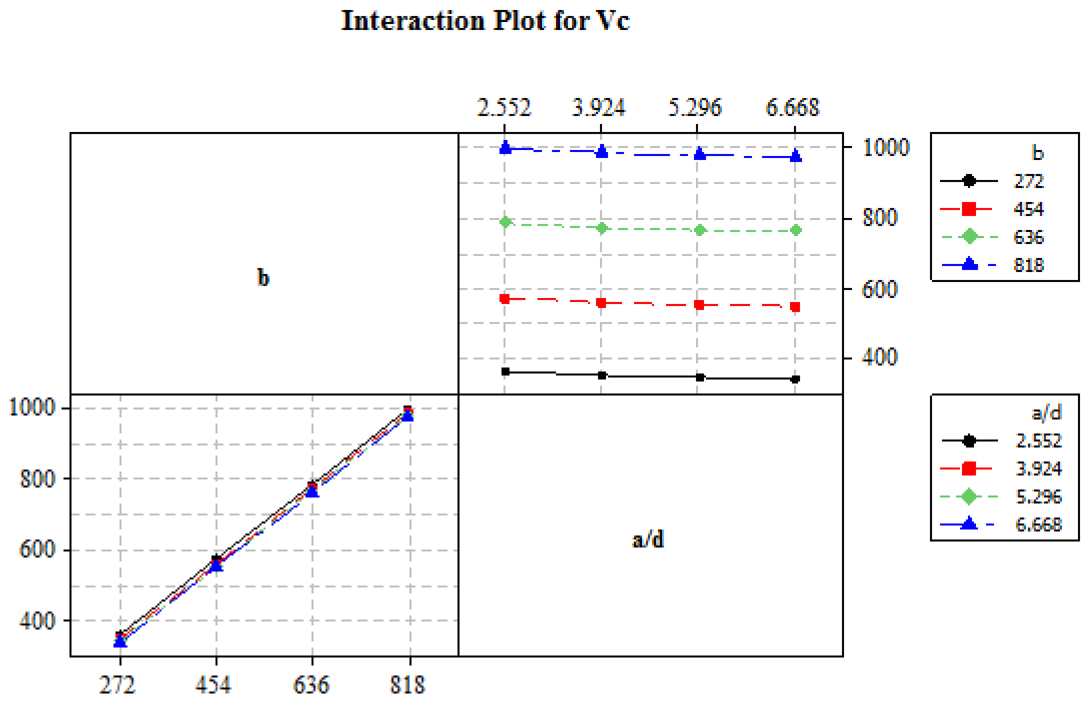


Figure4.4. Response of Influence of (b) on the effect of (a/d) on the shear resistance force (V_c)

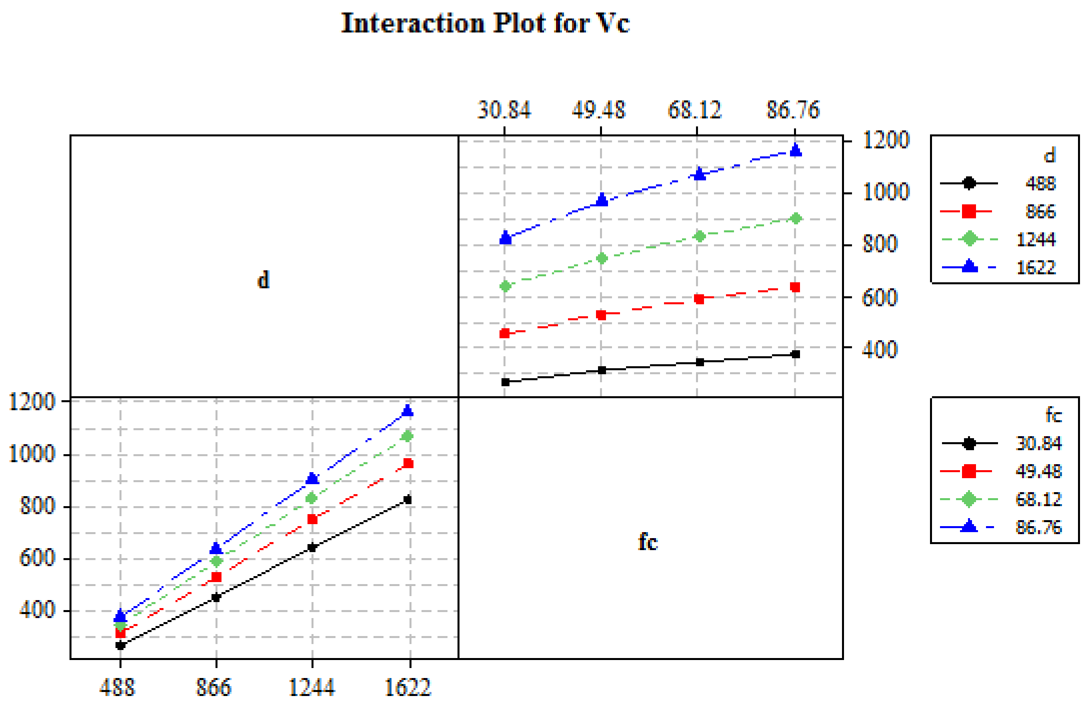


Figure 4.5. Response of Influence of (d) on the effect of (f_c) on the shear resistance force (V_c)

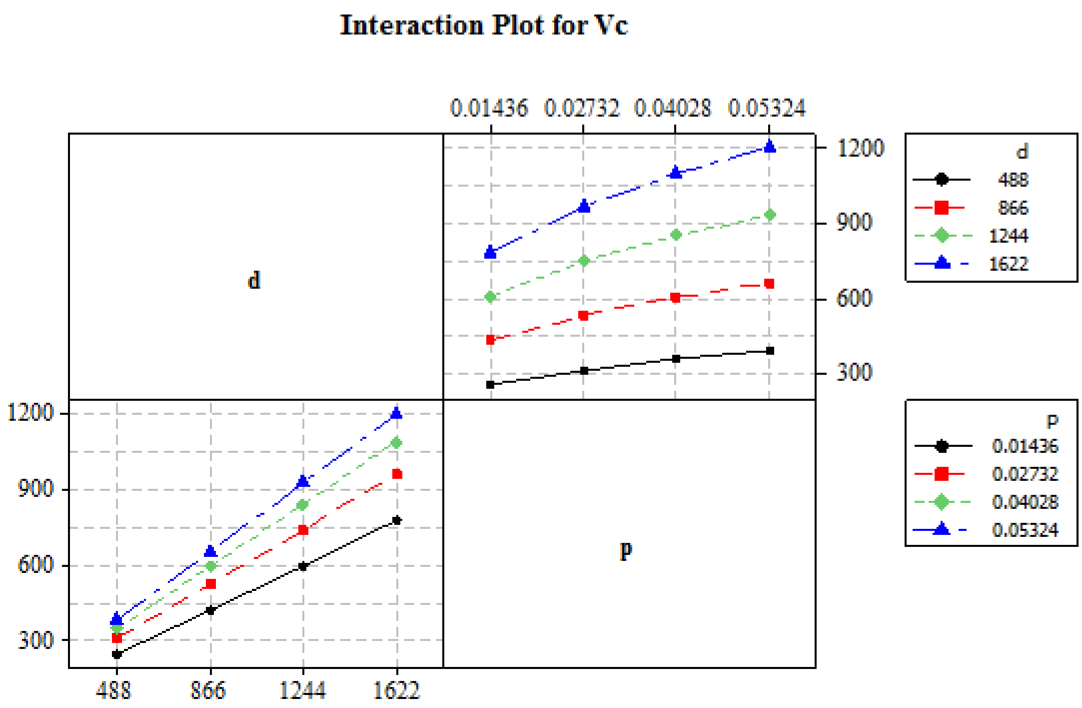


Figure 4.6. Response of Influence of (d) on the effect of (p) on the shear resistance force (V_c)

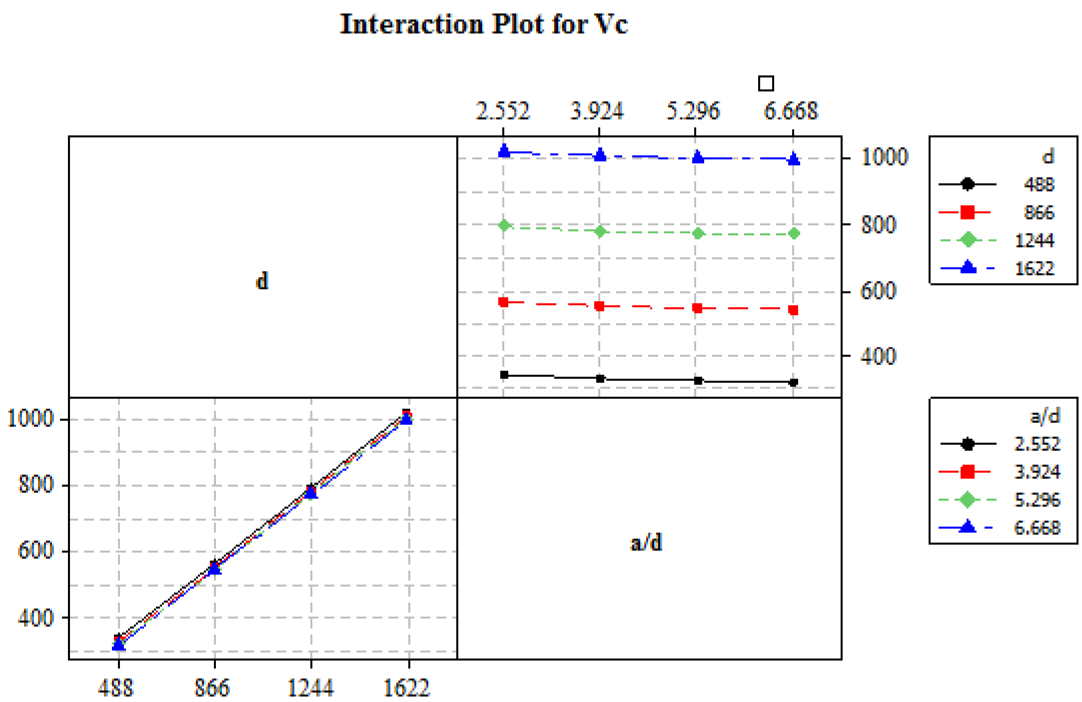


Figure 4.7.Response of Influence of (d) on the effect of (a/d) on the shear resistance force (V_c)

]

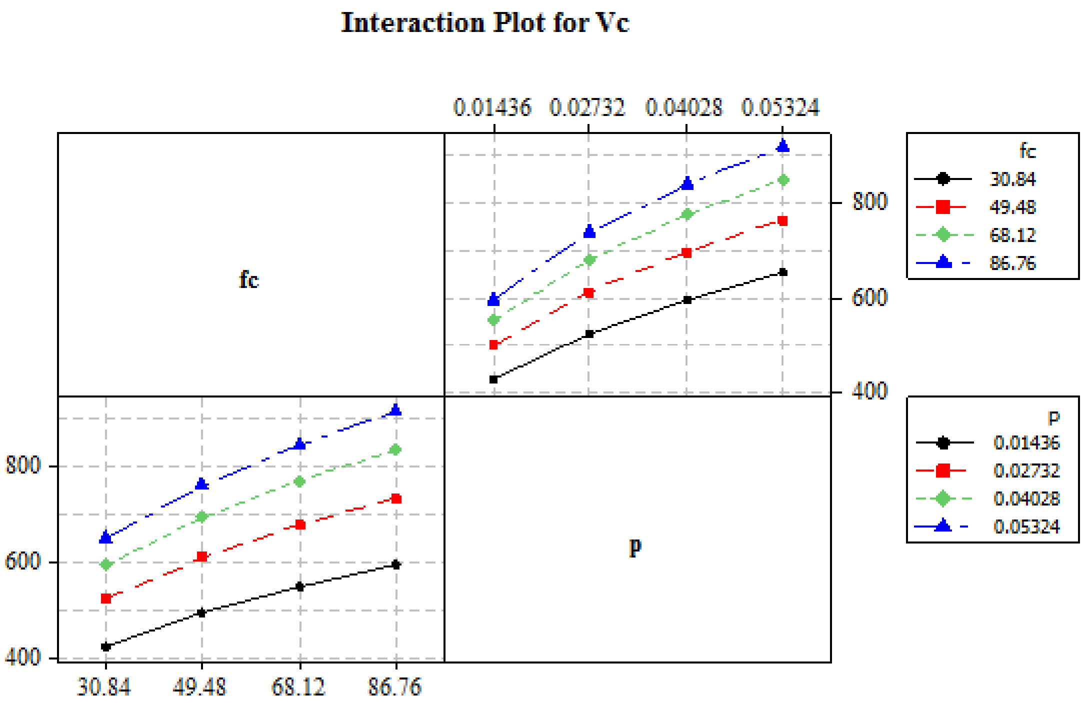


Figure 4.8. Response of Influence of (f_c) on the effect of (p) on the shear resistance force (V_c)

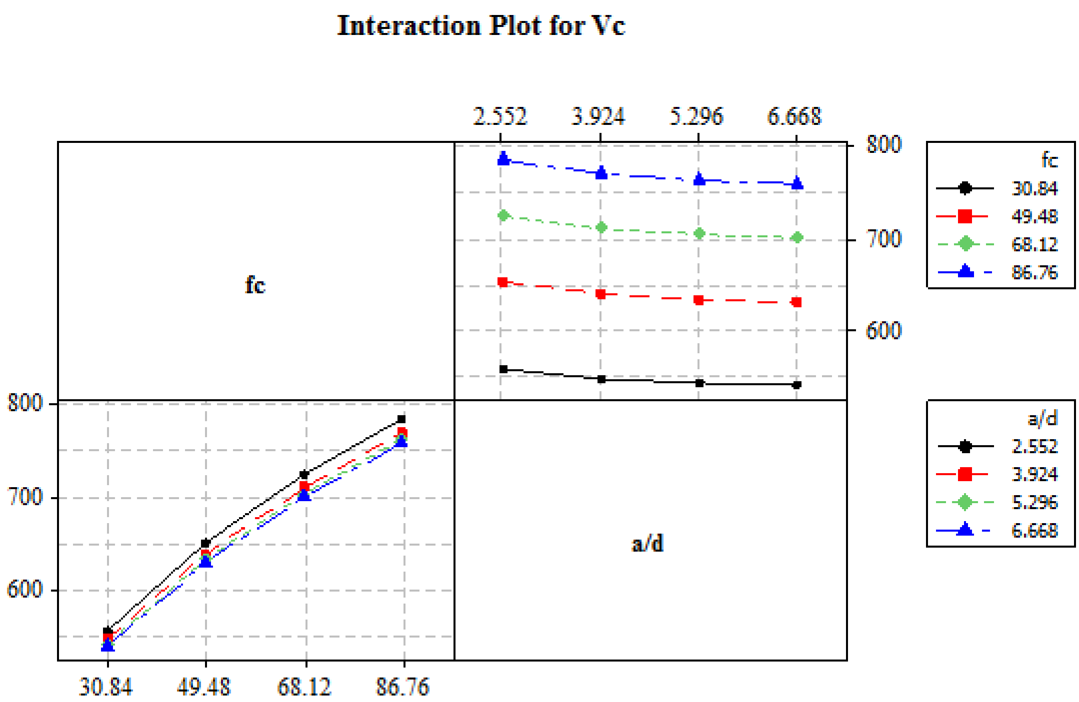


Figure 4.9. Response of Influence of (f_c) on the effect of (a/d) on the shear resistance force (V_c)

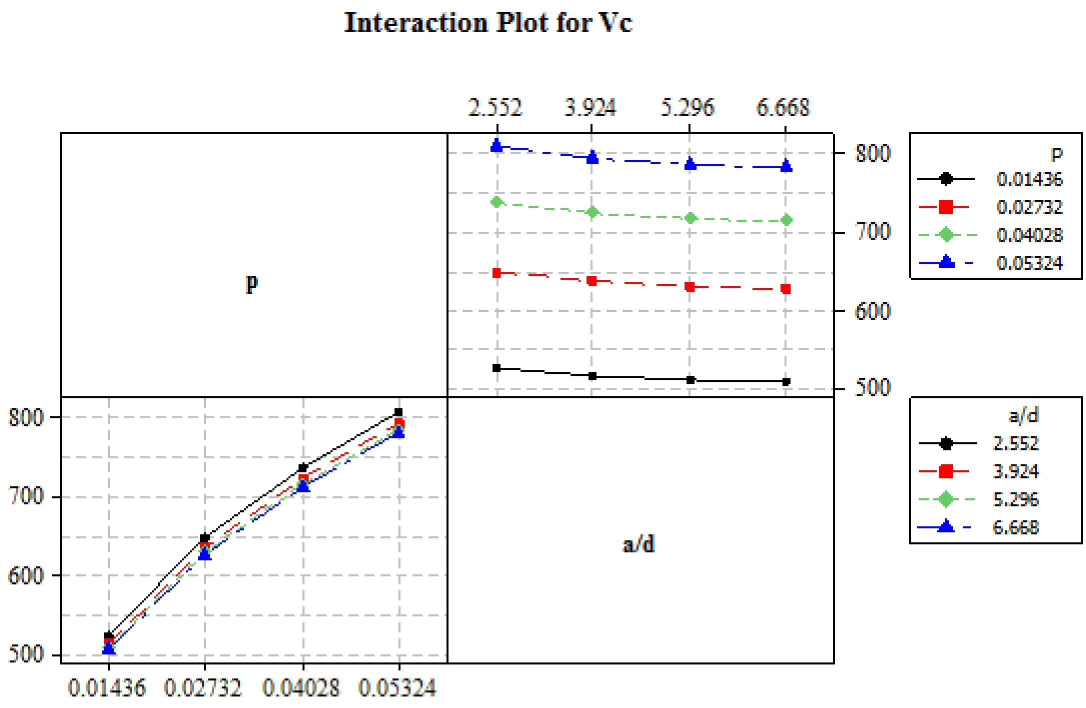


Figure 4.10.Response of Influence of (p) on the effect of (a/d) on the shear resistance force (V_c)

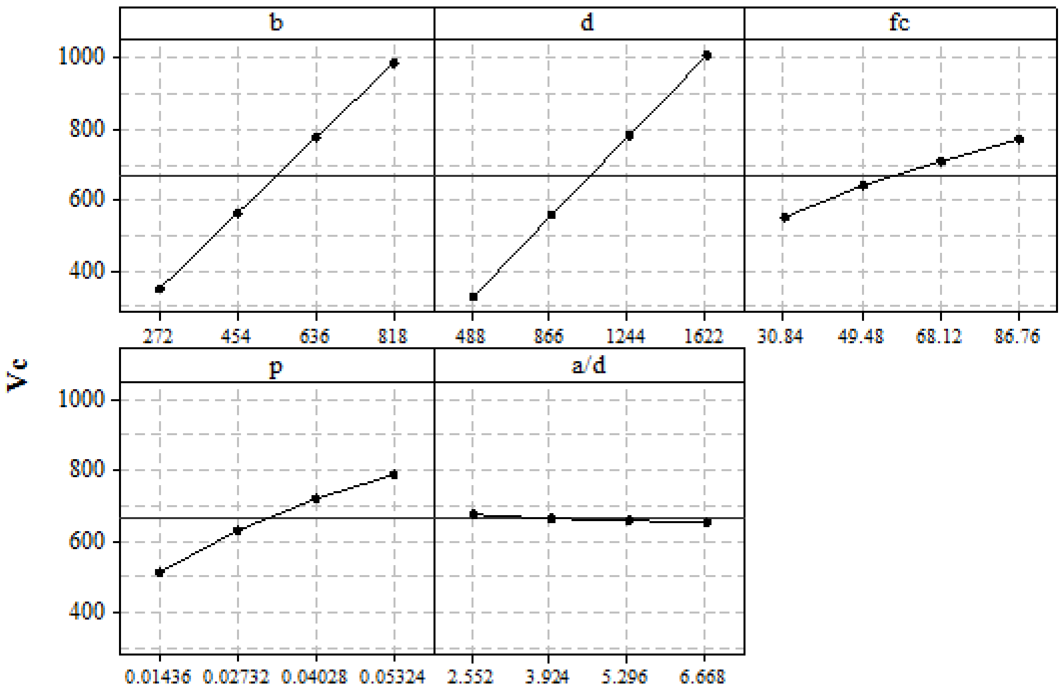


Figure 4.11.Main effect of each parameter on shear strength of RC beam without web reinforcement calculated by stepwise regression

4.4 Graph of R^2 in Each Codes and SR Model

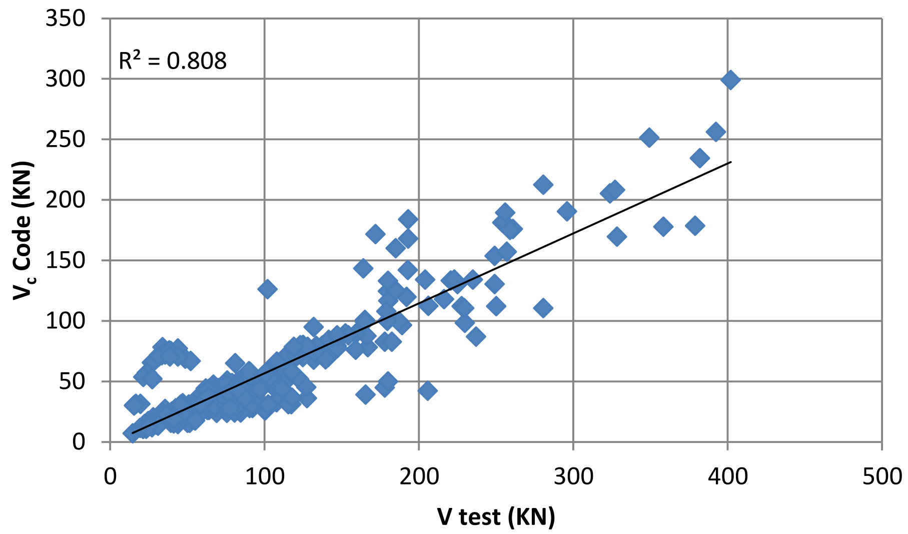


Figure 4.12. German code DIN 1045-1 (2001)

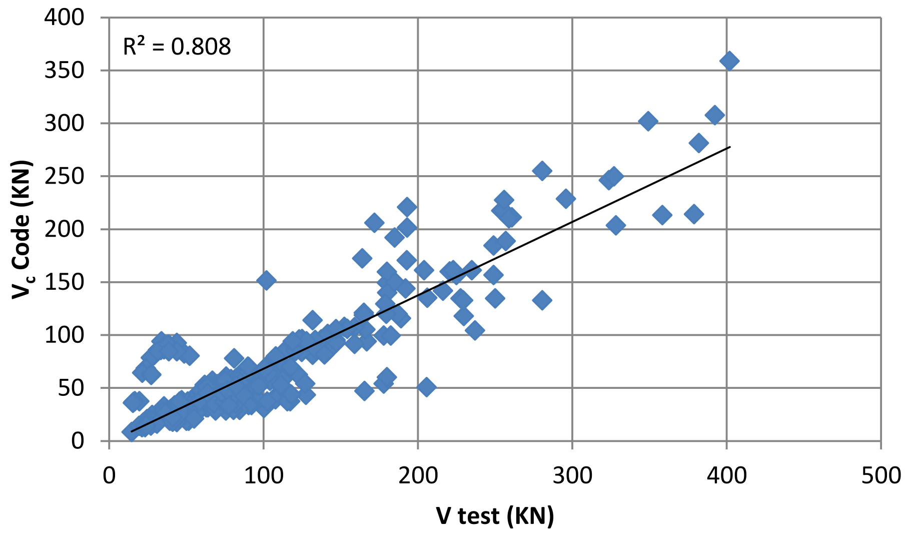


Figure 4.13. Eurocode EN 1992-1-1 (2003)

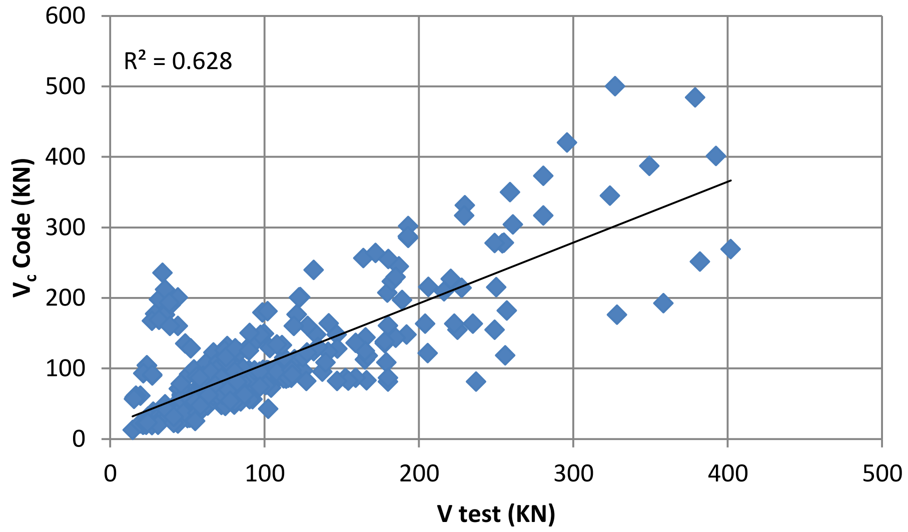


Figure 4.14. Eurocode (1992)

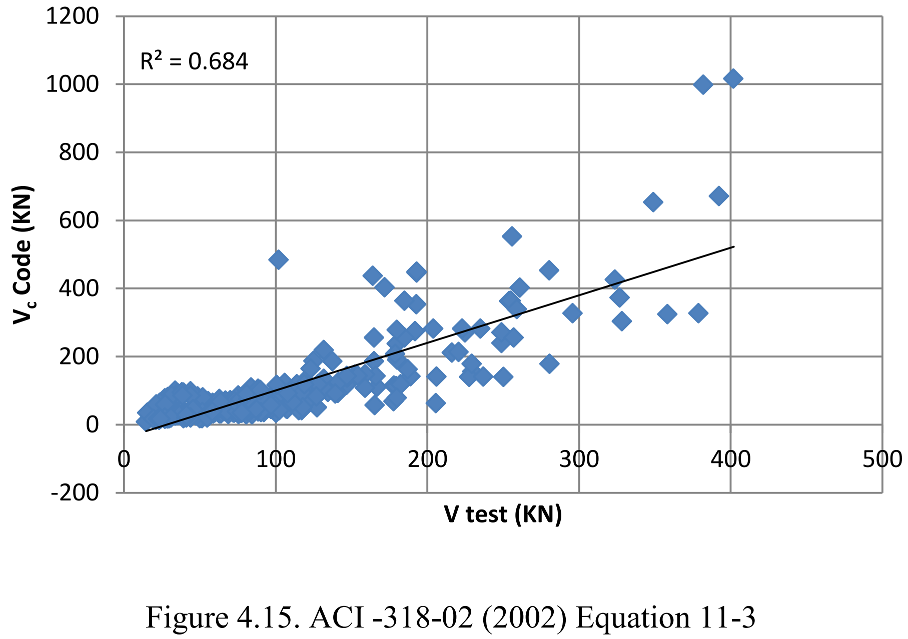


Figure 4.15. ACI -318-02 (2002) Equation 11-3

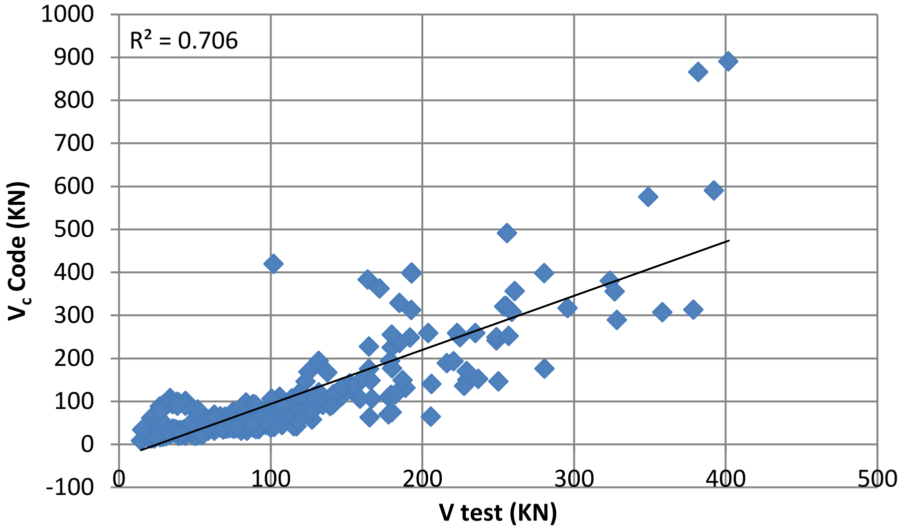


Figure 4.16. ACI -318-02 (2002) Equation 11-5

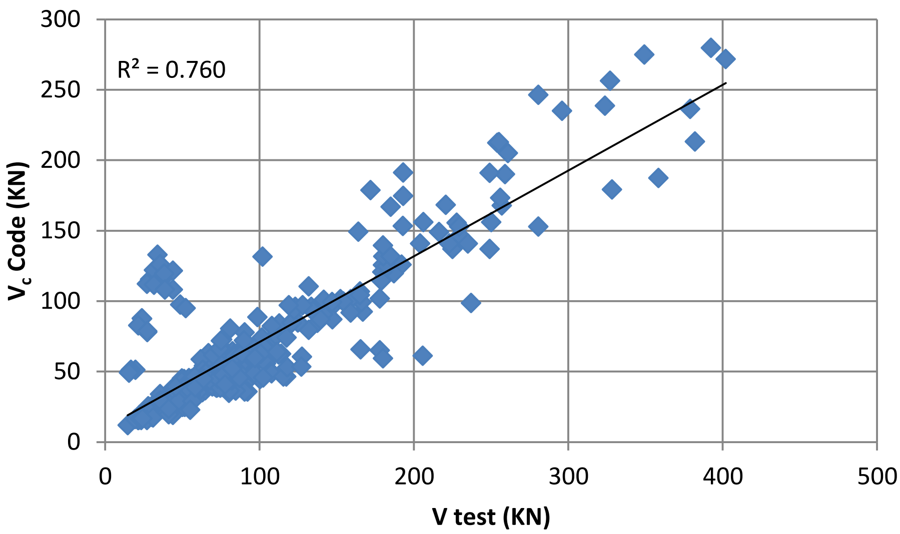


Figure 4.17. ASCE-ACI 445 (2003)

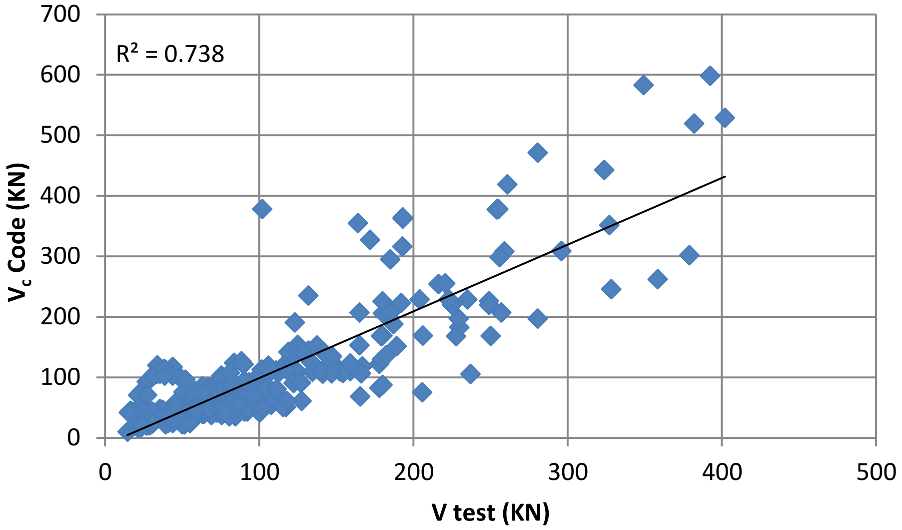


Figure 4.18. CSA (1994)

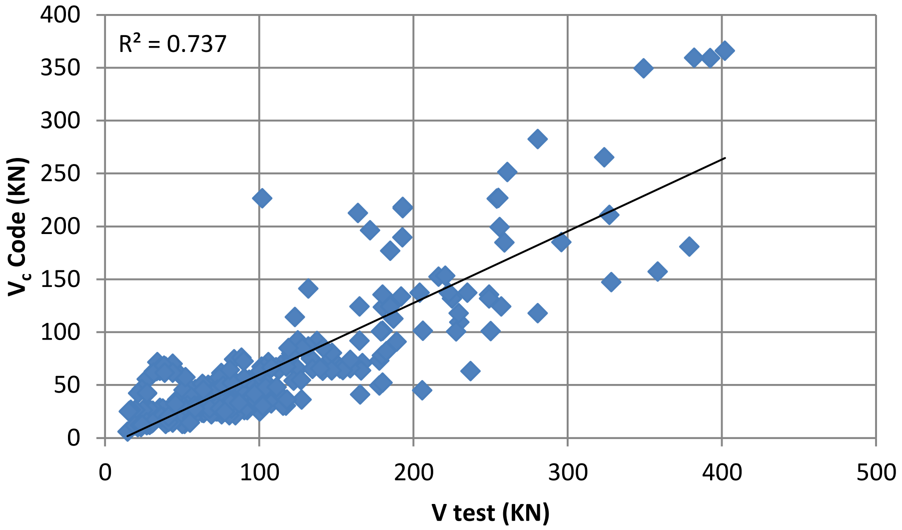


Figure 4.19. CSA (2005)

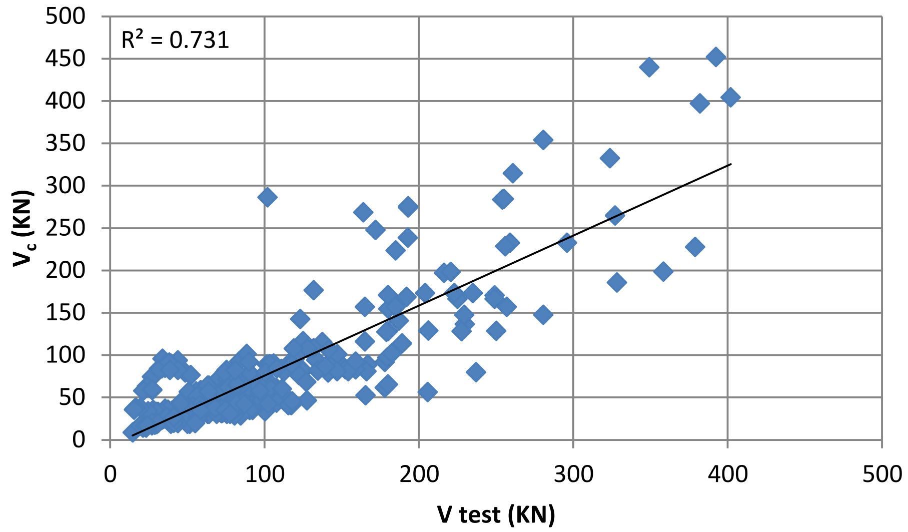


Figure 4.20. Collins, Kuchma (1999)

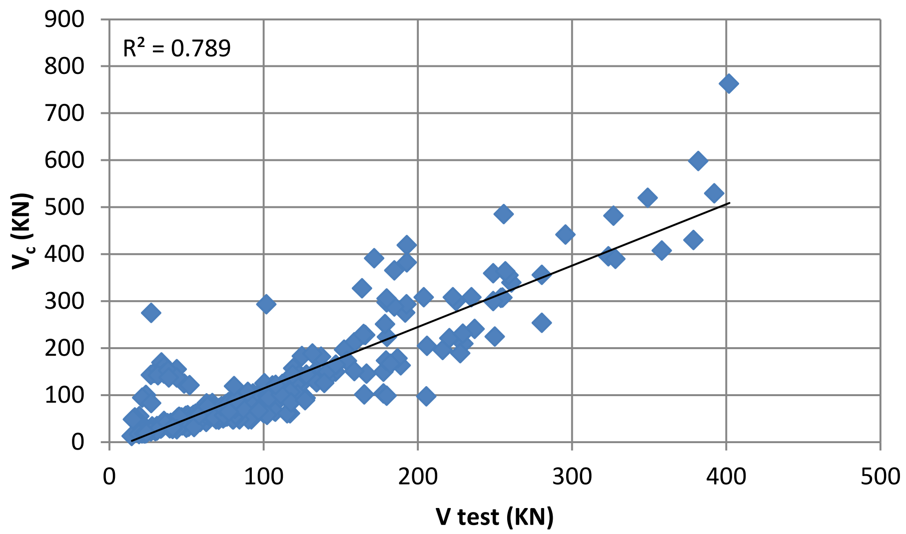


Figure 4.21. Zsutty (1968)

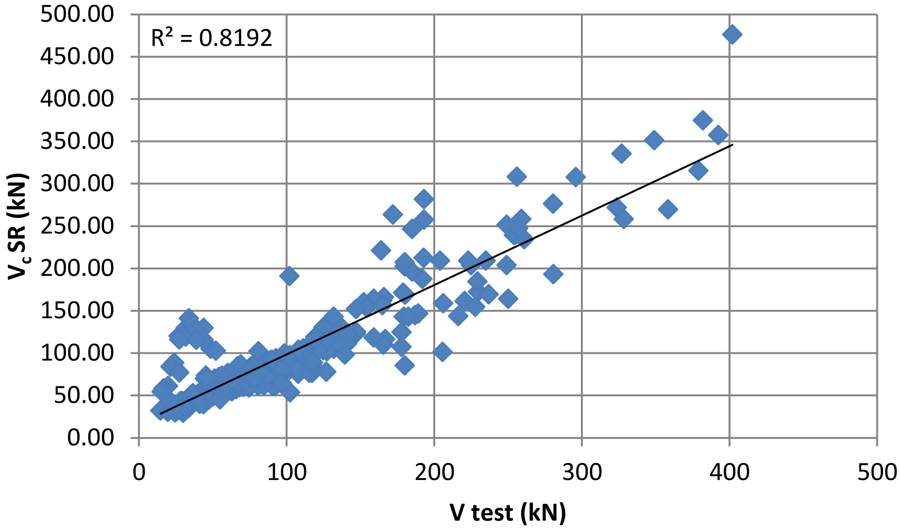


Figure 4.22. Stepwise Regression

CHAPTER FIVE

CONCLUSION

5.1 Introduction

This pioneer thesis presents stepwise regression as an alternative tool for the empirical modeling for shear strength of RC beams without web reinforcement. The proposed SR model in this paper is actually a realistic empirical model based on a wide range of experimental results collected from literature consisting of 398 test results belonging to 46 separate studies. The results are also compared with current design codes existing equations and are found to be more accurate. The main purpose of this research was to improve the understanding of the shear behavior of RC members. This research began with an extensive literature review of previous experimental research on shear, failure theories, complete models for describing shear behavior, and of shear design provisions in national codes of practice.

5.2 Conclusions

Based on this study, several important conclusions can be made. They are:

- Among widely used soft computing techniques stepwise regression is applied to shear strength of RC beams without stirrups.
- The accuracy of proposed SR model is also compared with existing design codes and equations available in the literature. The SR model was seen to be more accurate.
- The shear strength increases as the concrete strength increases. In most codes of practice, shear strength is proportional to the concrete strength to an exponent between 0.25 to 0.5.
- The shear strength clearly decreases as the effective depth of the member increases. Many of the major codes and empirical equations account for a size effect in shear .In their expressions, the shear strength decreases as a function

of the member depth. However, the expressions vary, ranging from $1/d, 1/d^{1/2}$ $, 1/d^{1/3}$,and$1/^{d1/4}$.

- The shear strength clearly increases as the longitudinal reinforcement increases.
- Many of the tested beams had an *a/d* ratio a round 2.5. The shear strengths of members with an *a/d* ratio of 2.5 are clearly higher than that of the members with an *a/d* ratio around 3.0.

5.3 Recommendations for Future Work

In this thesis, SR was used to model shear strength of RC beams without stirrups. On the other hand SR can also be applied to shear strength modeling of RC beams with stirrups, prestressed RC beams or torsion strength modeling of RC beams in further studies. Based on the results of the present study, the following future work is recommended :The model is verified using experimental results conducted using point loads. Thus ,it needs to be verified by other load conditions of practical significance, e.g., underuniformly loading conditions.

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