UNIVERSITY OF MORATUWA

EVALUATION OF SHEAR DESIGN PROCEDURES ADOPTED IN THE INDUSTRY FOR REINFORCED CONCRETE

By

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DECLARATION

I hereby, declare, that the work included in this thesis in part or whole has not been submitted for any other academic qualification at any institution.

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ABSTRACT

Application of reinforced concrete as a construction material was first found in the middle of the 19^{th} century. Over the last one and half centuries it has become a popular and widely accepted construction material. Its applications span from in small domestic structures to large structures like massive dams, bridges, offshore platforms provides evidence for its potential.

Shear design is an important area of the reinforced concrete designing process. This study reviews the shear designing approaches for reinforced concrete beams. From the beginning the shear behaviour of reinforced concrete beams was mysterious. The first analytical model to explain the shear behaviour of a reinforced concrete beam was postulated in 1899 by a Swiss engineer called Ritter and a German engineer called Mörsch (1902). They independently introduced the Truss Model to use in shear design. Since then various theories have been put forward to explain the shear behaviour of reinforced concrete beams. But, still none of them seems to have resolved the issue by producing results relating theory to experiment to a higher degree of accuracy when compared to flexural design.

This study identifies reasons for those theories to deviate from the experimental results. Some of them are conventional parameters used in design equations whereas others are new for these design methods. Also it identifies when these parameters become critical for deviation of the predicted results from the experiment. Ultimately this study identifies when these theories are justifiable for shear designing of reinforced concrete. Also it evaluates the practices followed in design offices in Sri Lanka for shear design and recommends the best practises to ensure adequate safe guard against a premature failure. Results of this study shows that Canadian Code General method and Australian Code method

give most accurate results and can be recommended to use within the limitations specified in the code. Further this study shows that Japanese Code design method can be recommended for conservative shear designing without any restrictions on parameters. But this method is less accurate than the Canadian Code General method and Australian Code method.

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I appreciate very much invaluable support, encouragement and understanding shown by my parents.

Finally I would like to acknowledge with fraternal love my colleagues and others who have assisted me in various ways to the successful completion of this thesis.

W.K.H.R.E.Wickramage

DEDICATION

To

my parents

and

all those who are interested and committed in advancement of science

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1. Introduction

1.1 Background

Use of cement in structures, in the form of lime mortar began in around 2000 B.C. and still used in some areas. The innovation of the material which is currently known as Portland Cement was achieved in 1845 by a scientist called I.C. Johnson. The use of reinforced concrete for structural applications then started in the middle of 19^{th} century. W.B. Wilkinson of Newcastle first obtained a patent for reinforced concrete floor system. During the period of 1850 to 1900 the science of reinforced concrete developed through a series of patents obtained for various concrete elements. After that knowledge and technology about reinforced concrete started to improve with extensive research work in the concrete technology.

Now reinforced concrete has become a widely used material for constructing various structures. Its applications can be found in buildings of all sorts, underground structures, water tanks, television towers, harbors, offshore structures, dams, etc. Success behind its popularity may be the wide availability of reinforcing bars and the constituent of concrete; gravel, sand and cement, relatively simple skill required in concrete construction and the economy of reinforced concrete compared to other forms of construction.

The first set of building regulations for reinforced concrete was drafted under the leadership of Professor Mörsch of University of Stuttgart and was issued in Prussia in 1904. After that design regulations were issued in Britain, France, Austria and Switzerland between 1907 and 1909. Since then, extensive research has been carried out on various aspects of reinforced concrete behaviour, resulting in the current design procedures.

At the early stage of the design codes, shear design provisions were based on Empirical methods and Truss analogy. During the last 50 years considerable amount of research has been conducted world wide with the aim of developing a rational, general and accurate behavioural model for reinforced and prestressed concrete in shear. As a result of this a large number of theories have been published. Among them, design models such as Modified compression Field theory and Rotating Angle Softened Truss Model have been able to give more rationality to shear design provisions. Also a considerable development could be found on shear friction methods during last two decades. But even in these theories consensus is lacking in several vital areas. Therefore provisions of shear design have been able to resolve some of the issues but lack complete understanding yet, and as such shear designs cannot be achieved to a high degree of accuracy unlike in flexural design. Hence a detailed study is required to improve the situation.

1.2 Historical Development of Shear Design Procedures.

It is important to express the behaviour of beams failing in shear in terms of a mathematical model before designers can make use of this knowledge in design. The first attempt for this appeared at the end of the 19th century. It was a Swiss engineer Ritter (1899) and a German engineer Mörsh (1902) independently published their papers proposing truss model for the shear design, for the first time. One main disadvantage of truss model was that it neglected the shear caring capacity by the concrete. As a result of that, application of truss model in designing of reinforced concrete members without shear reinforcement could lead to unsafe designs. Therefore, from the early stages designers preferred empirical methods for design of beams without shear reinforcement. As a consequence, development of empirical methods also took place in parallel to the truss analogy.

Partial collapse of the Wilkins Air Force warehouse in Shelby, Ohio, USA in 1955 questioned the shear design provisions of ACI building code at that time. Also it emphasised the necessity of safe, rational and accurate shear design procedures to researchers all around the world. Extensive researches were carried out all around the world on this propose.

As a result of that, truss model was remarkably improved by various researchers to give safe and accurate predictions. As mentioned previously, in traditional truss approach the inclination of truss was assumed to be 45^0 to the longitudinal reinforcement and shear carrying capacity was neglected. This traditional approach is called "Standard Truss Model with no Concrete Contribution". One new approach was to add a concrete contribution term to the shear carrying capacity obtained, assuming 45^0 truss (ASCE-ACI Committee 318-95M – 1962). A combination of variable angle truss and a concrete contribution was also proposed (CEB 1978; Ramirez and Breen 1991). This procedure was referred to as " Modified Truss Approach". The concrete contribution of above methods was taken as the combined effect of shear transfer across cracks due to aggregate interlock, dowel action of longitudinal steel and the shear transfer across the uncracked concrete compression zone. Also it was calculated using more refined empirical formulas.

Further truss models with crack friction were proposed (Gambarova 1979; Dei Polli et al. 1990; Kupfer et al 1979; Kirmair 1987; Reneick 1990). In this model, it was assumed forces were transferred cross cracks by friction in the failure plane. And this force was added to the 45^0 truss to obtained the final shear capacity. Most advanced procedure to propose based on truss analogy was Rotating Angle Softened Truss Model (Belarbi and Hsu 1991,1995; Hsu 1993; Peng and Hsu 1995). This was a more rational procedure which used equilibrium conditions, compatibility conditions and stress-strain relationships of diagonally cracked concrete to predict load deformation response of a section subjected to shear.

After the warehouse tragedy, other than the truss analogy and the empirical methods, attention was drawn towards developing better models to describe shear behaviour of reinforced concrete beams. As a result of that several new approaches were proposed. One of them was fracture mechanics approach. A number of different fracture mechanics models were proposed. Among them well known ones are the Fictitious Crack Model (Hillerborg et al 1976) and Crack band Model (Bazant and Oh 1983). Fracture mechanics approaches account for the fact that there is a peak tensile stress near the tip of the crack and a reduced tensile stress (softening) near crack zone. Fracture mechanisms can be considered as more rational than empirical.

Another early attempt to develop a rational model was Kani's Tooth Model (1964), in which the secondary diagonal cracks were believed to result from bending of concrete "teeth". The concrete between two adjacent flexural cracks was considered to be analogous to a tooth on a comb. Hamadi and Regan (1980), Reneck (1991) further developed the Tooth model.

Based on the Tension Field theory developed by the German engineer H.A.Winger(1929) to explain the shear carrying mechanism of a thin web steel girder, Collins (1978) developed new method to explain the shear behaviour of cracked concrete beams which was known as Compression Field Theory (CFT). This approach is used equilibrium conditions, compatibility conditions and stress-strain relationships of diagonally cracked concrete to predict load deformation response of a section subjected to shear. This approach is further developed to account for the influence of tensile stresses in the cracked concrete by the same author in 1986. The new approach was called the Modified Compression Field Theory.

Another new approach to the shear design was developed based on the Shear Friction. The shear friction concept was first introduced by Brikeland in 1966. it was originally developed to deal with forces transfer across joints in precast concrete construction. This model is further developed by Paulay et al (1974); Mattock et al (1976); Nielsen et al (1978); Walraven (1981) and Loov and Patnaik (1994). Based on shear friction Loov presented a new shear design approach in 1998. This approach is discussed in this study apart from five major codes.

1.3 Overview of Current Design Procedures

In most of the major design codes, shear design procedure for one way flexural members is based on truss model with concrete contribution. Some of them use a parallel truss model with 45^0 constant inclination diagonals supplemented by an experimentally obtained concrete contribution, where as others use variable angle truss model with empirically obtained concrete contribution. General method of Canadian Code is based on the modified Compression field Theory.

Truss model is widely used for designing of deep beams. But design procedure for deep beams given in the Japanese code still relies on empirical formulae.

1.4 Objectives

The principal objective of this study is to carry out a state of the art review and to evaluate the shear design approaches. This involves finding when such theories are justifiable for reinforced concrete. Also it is aimed at evaluating the practices followed in design offices in Sri Lanka for shear design and recommending the best practises to ensure the adequate safe guards against premature failures.

1.5 Scope

When evaluating several shear designing methods, the scope of this study is limited to reinforced concrete beams. Shear design procedures for both deep and slender beams will be evaluated. The focus of this study will be on beams without axial forces and prestress.

1.6 Organization of Thesis

This thesis consists with five main chapters. Chapter 1 gives an introduction to the subject area and a overview of the current design procedures.

Chapter 2 presents and discusses the available methods found in the literature.

Chapter 3 deals with the methodology, discussion on the preparation of the database, development of the each design method. Flowcharts for computer based calculations are also presented in the same chapter.

A comparison between the results obtained by using various design methods and the test results are presented in Chapter 4. Results and a discussion about the results obtained from the Multinomial Logistic Regression is also presented in this chapter.

Chapter 5 covers the conclusions and recommendations for further research

2. Literature Review

2.1 Introduction

Structural use of reinforced concrete began in the middle part of the 19th century. After that, gradually it became a popular material in construction industry all over the world. Knowledge about mechanisms of reinforced concrete started to spread among practicing engineers in the beginning of 20th century as books, technical articles, and codes presented theory. Since then a large number of papers have been published around the world.

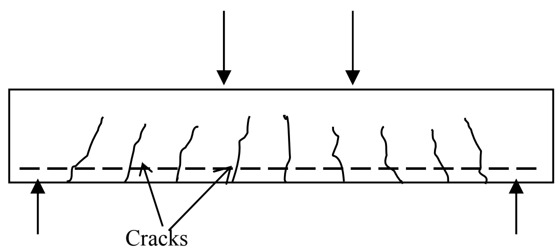
As discussed earlier in the Truss Model was first introduced as a conceptual tool for the analysis and design of reinforced concrete beams. Since then various researchers have proposed more refined truss models. Also a lot of empirical equations have been put forward to estimate the shear strength of a reinforced concrete beam. Interestingly, still the shear design procedures of most of the major codes rely on the truss model as well as the empirical methods. During the last few decades more rational design methods which were equally capable of handling both beams with and without shear reinforcement have been published. Among them Compression Field Theory (1978) and Modified Compression Field Theory (1982, 1986) were quite impressive. In addition to that several methods have been postulated based on shear friction.

This chapter presents a brief discussion on the shear behaviour of the reinforced concrete beams and parameters affecting the shear strength. Also some of shear design models which have been the basis of the some of the major codes, have been presented with a brief discussion of their applications in various codes.

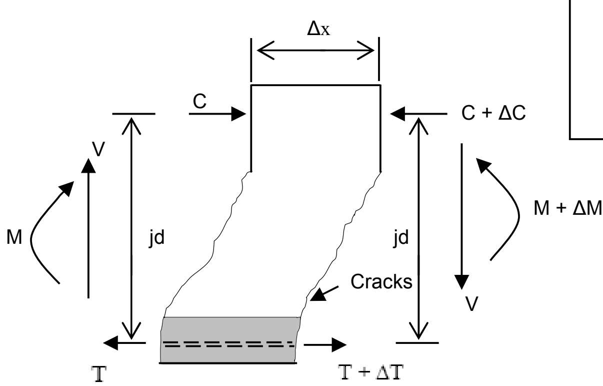
2.2 Behaviour of Beams Failing in Shear

In a reinforced concrete beam, two main actions have been identified to transfer the shear force from load to support. Namely, they are Beam

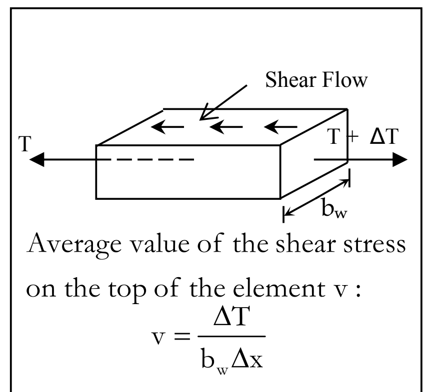
Action and Arch Action. The behaviour of beams failing in shear varies widely on the relative contributions of the beam action and the arch action. Beam Action exists when there is a shear flow between load point and supporting point as shown in Fig 2.1c. Arch action occurs if the shear flow cannot be transmitted due to the steel being unbonded or if the transfer of shear flow is prevented by an inclined crack extending from the load to the reaction. In such a case shear is transferred by *Arch Action* rather than beam action as illustrated in Figure 2.2.



(a) Cracked beam



(b) Portion of beam between two cracks



(c) Highlighted portion at the bottom part of the beam

Figure 2.1
Shear Flow of a Beam

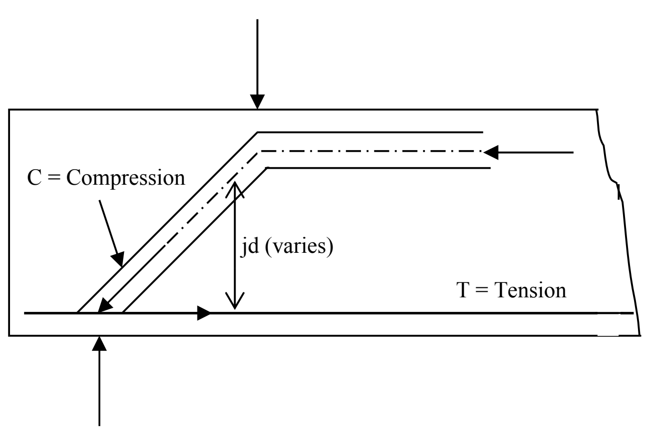


Figure 2.2
Arch action in a beam

2.2.1 Behaviour of Beams without Web Reinforcement

The moments and shear at inclined cracking and failure of rectangular beams without web reinforcement are plotted in Fig. 2.3(b) as a function of the ratio of the shear span to effective depth (a/d).[Ref.2.1]. The beam cross section remains constant as the shear span is varied. The maximum moment and shear that can be developed correspond to the nominal moment capacity, M_n, of the cross section plotted as a horizontal line in Fig. 2.3(b). The shaded area in figure shows the reduction in strength due to shear at different a/d ratios. Web reinforcement is provided to ensure that beam reaches the full flexural capacity.

Fig. 2.3(b) suggests that the shear span can be divided in to four types: Very short, short, slender and very slender. The term “deep beam” is also used to describe beams with very short and short shear spans. Very short shear spans, a/d from 0 to 1, develops inclined cracks joining the load and support. These cracks, in fact, destroy the shear flow from the longitudinal steel to the compression zone and the behaviour changes from beam action to arch action, as shown in Fig 2.2 and Fig. 2.4. Here the reinforcement serves as the tension tie. The most common failure in such a beam is an anchorage failure at the end of the tension tie.

Short shear spans, a/d from 1 to 2.5 develop inclined cracks and, after a redistribution of internal forces, are able to carry additional load, in part by arch action. The final failure of such beams will be caused by a bond failure:, a splitting failure, or a dowel failure along the tension reinforcement as shown in Fig. 2.5(a), or by crushing of the compression zone over the crack, as sown in Fig. 2.5(b). The latter referred to as a “shear compression failure”. Because the inclined cracks generally extend higher in to the beam than a flexural crack, failure occurs at less value than the flexural moment capacity.

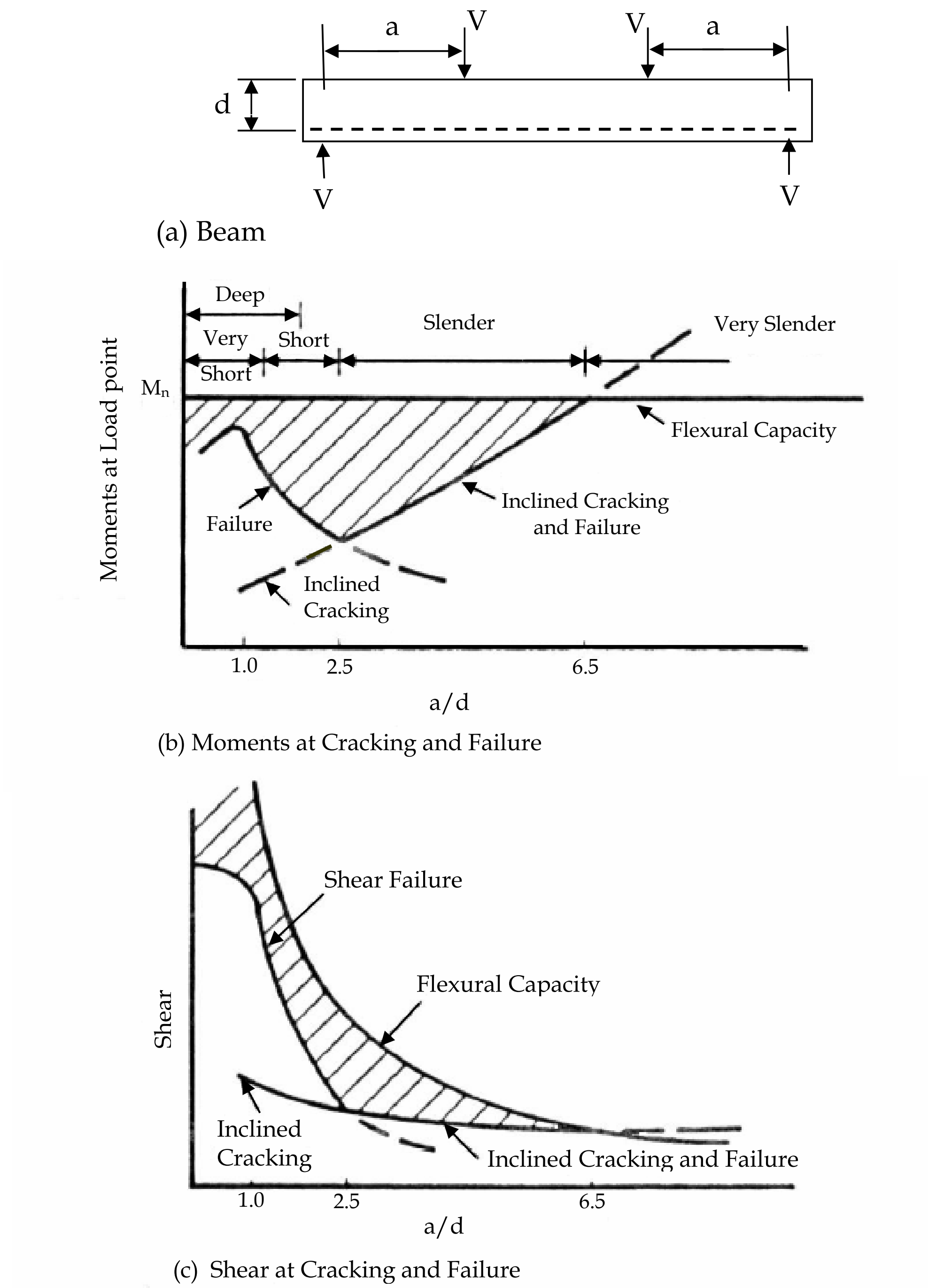
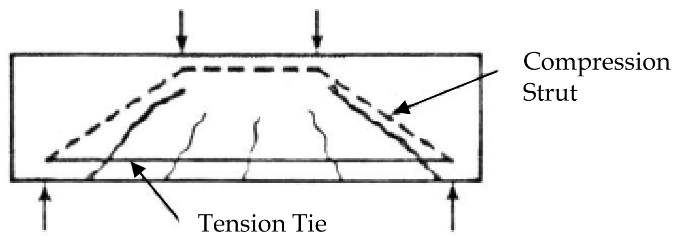
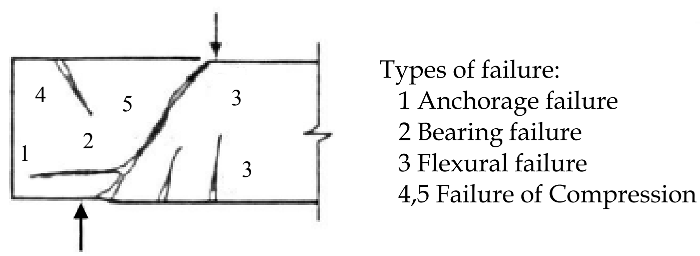


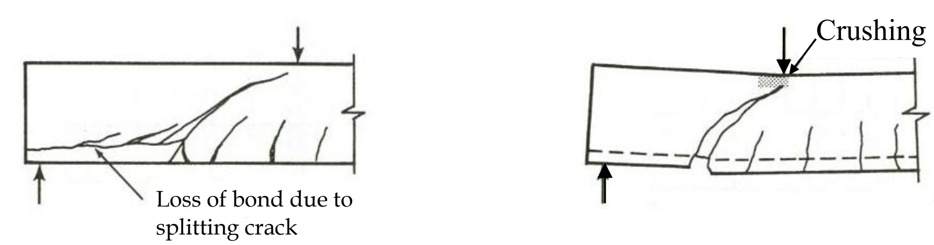
Figure 2.3
Effect of a/d Ratio on the shear strength of beams without stirrups. (Adapted from Ref. 2.6)

In slender shear spans, a/d from above 2.5 to about 6, the inclined cracks disrupt equilibrium to such an extent that the beams fail at the inclined cracking load as shown in Fig.2.3b. Very slender beams with a/d grater than about 6 will fail in flexure prior to the formation of inclined cracks.

Figure 2.4
Modes of failure of Beams,
a/d = 0.5 to 2.0
(Adapted from Ref 2.1)







(a) Shear Tension Failure

(b) Shear Compression Failure

Figure 2.5.
Modes of failure of beams with
Short Shear Spans, a/d = 1.5 to 2.5
(Adapted from Ref 2.1)

Fig. 2.4 and 2.5, present an excellent description of the behaviour of beams falling in shear. It is important to note that for short and very short beams, a major portion of the load capacity after inclined cracking is due to load transfer by the compression struts shown in Fig. 2.4. If the beam is not loaded on the top and supported on the bottom in the manner shown in Fig. 2.4, these compression struts are not effective and failure occurs at, or close to, the inclined cracking load.

Because the moment at the point where the load is applied is M=Va for a beam loaded with concentrated loads, as sown in Fig2.3a and 2.3b, can be re-plotted in terms of shear capacity, as shown in Fig.2.3c. The shear corresponding to a flexural failure is the upper curved line. If stirrups are not provided, the beam will fail at the shear given by the “Shear Failure” line. This is roughly constant for a/d grater than about 2. Again the shaded area indicates the loss in capacity due to shear. Note that the inclined cracking loads of the short shear spans and very slender shear spans are roughly a constant. Inclined cracking causes immediate failure if no web reinforcement is provided. For very slender beams, the shear required to form an inclined crack exceeds the shear corresponding to flexural failure and the beam will fail in flexure before inclined cracking occurs.

2.2.2 Behaviour of Beams with Web Reinforcement

Due to inclined cracking, the strength of beams drops below the flexural capacity as shown in Fig. 2.3(b) and (c). The purpose of web reinforcement is to ensure that the full flexural capacity can be developed.

Prior to inclined cracking, the strain in the stirrups is equal to the corresponding strain of the concrete at the same level. Since concrete cracks at very small strains, prior to inclined cracking the stress in the stirrups are very low. Thus stirrups do not prevent inclined cracks forming: they come to play only after cracks have formed.

2.3 Factors Affecting Shear Strength of Beams without Shear Reinforcement.

The shear capacity of a beam without web reinforcement is taken equal to the inclined cracking shear. This is because, beams without web reinforcement generally fail when inclined cracks occurs or shortly afterwards. The inclined cracking load of a beam is affected by five principal variables.

1. **Tensile Strength of Concrete**

   The inclined cracking load is a function of the tensile strength of the concrete. The flexural cracking which precedes the inclined cracking disrupts the elastic stress field to such an extent that inclined cracking occurs at a principle tensile stress of roughly a one third of splitting tensile strength (f_{ct}).

2. **Longitudinal Reinforcement Ratio, ρ**

   The shear capacity is a function of the longitudinal reinforcement area ratio $\rho = A_s / b_w d$. When the steel ratio is small flexural cracks extend higher into the beam and open wider than would be the case of large values of ρ. As a result, inclined cracking occurs earlier.

3. **Shear Span to Depth Ratio, a/d**

   The shear span to depth ratio a/d has a significant effect on the inclined cracking shear and ultimate shear particularly for beams with low shear spans to depth ratios. With the increase of the a/d ratio its effect gets reduced.

4. **Depth of the Beam**

As overall depth of the beam increases the shear capacity of the beam tends to decrease. As the depth of the beam increases the crack widths at points above the main reinforcement tend to increase. This leads to reduction in aggregate interlock across the crack, resulting in earlier inclined cracking.

5. **Axial Force**

Axial tensile forces tend to decrease the inclined cracking load, while axial compression forces tend to increase it. As the axial compressive force is increased, the onset of flexural cracking is delayed and the flexural cracks do not penetrate as far in to the beam. As a result, a larger shear is required to cause inclined cracks.

2.4 Shear Design Methods

2.4.1 Empirical Methods

The simplest approach, and the first to be proposed by Mörsch (1909) was to relate shear strength to tensile strength of the concrete. Many other empirical equations have since been proposed. Some of them have been presented below. These equations typically contains the following parameters: the concrete tensile strength, usually expressed as a function of compressive strength of concrete f_c; the longitudinal reinforcement area $\rho = A_s/b_w d$; the shear span to depth ratio a/d or M/Vd; the axial force or amount of prestress; and the depth of the member d.

A simple lower bound average shear stress at diagonal cracking is given by the following equation

$$\frac{V_c}{bd} = v_c = \frac{\sqrt{f_c^{'}}}{6} \tag{2.1}$$

This well-known equation is the basis for the ACI Code Equation 11.3 and the Simplified method of the Canadian Code which is based on the ASCE -ACI committee 326 report presented in 1962. Zsutty presented the following equation in 1971.

$$v_c = 59\left(f_c^{'}\rho \frac{d}{a} \right)^{\frac{1}{3}} \tag{2.2}$$

Considering all the main parameters Okamura and Higai in 1980 presented the following empirical equation.

$$v_c = 0.20\frac{\rho^{1/3}}{d^{1/4}}\left(f_c^{'} \right)^{1/3}\left(0.75 + \frac{1.40}{a/d} \right) \tag{2.3}$$

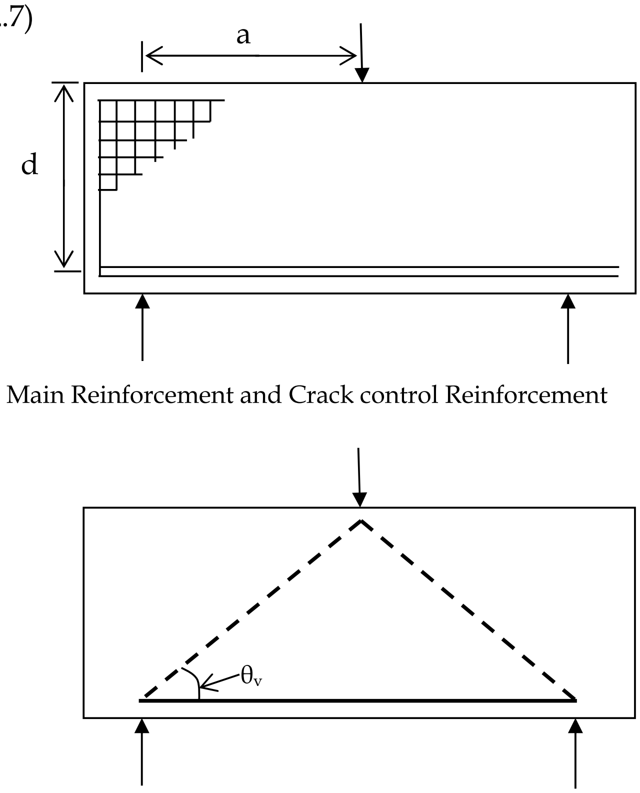
This equation may be considered as one of the most reliable empirical formulae.

With respect to the various empirical formulae, considerable difference exist as a result of following factors: the uncertainty in assessing the influence of complex parameters in a simple formula; the scatter of the selected test results due to inappropriate tests being considered (for example, bending failures or anchorage failures) and the poor representation of some parameters (for example, very few specimens with low reinforcement or high concrete strength). These issues limit the validity of empirical formulas and increase the necessity of rational models and theoretically justified relationships.

2.4.2 Strut and Tie Approach

Since the beginning of the 20th century the truss model has been used for designing of concrete members (Ritter 1899 and Mörsch 1902). One of the main advantages of using truss members to represent key resisting elements of a concrete member is that the flow of forces can be easily visualized by the designer. The flow of compressive stresses is idealized as compression members called struts and tension is taken by tension tie. Another advantage of using truss model to idealize flow of forces is that the influence of both shear and moment are accounted for simultaneously and directly in the design.

The strut and tie model is equally applicable for deep members with very short shear spans: $0 < a/d < 1.0$ or short shear spans: $1.0 < a/d < 2.5$ (Figure 2.6) and for slender members with shear spans: $a/d > 2.5$. (Figure 2.7)



a) Main Reinforcement and Crack control Reinforcement

b) Truss model

Figure 2.6
Truss Model for a Deep Beam (a/d <2.5)

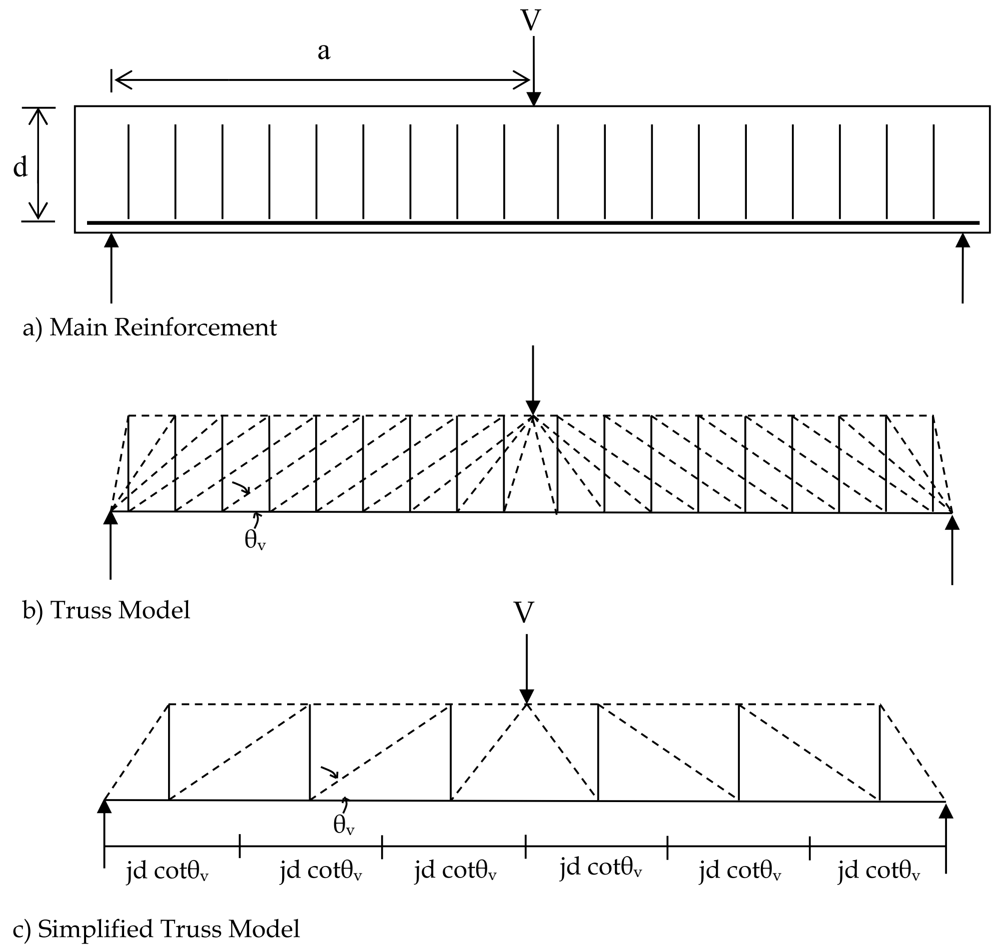


Figure 2.7
Truss Model for Slender Beams.

The strut and tie model is particularly useful in designing of disturbed regions where the normal assumption of plane strain and uniform shear stress distribution is inappropriate. Therefore most of the major codes uses strut and tie model for the designing of deep members.

A strut and tie model for slender members have been illustrated in Figure 2.7. The truss shown in Figure2.7b is statically indeterminate but can be solved if it is assumed that the force in each stirrup causes it to just reach yield. Then the truss in Figure 2.7b is referred to as the Plastic Truss Model since we are depending on yielding of the stirrups to make it statically determinate.

Figure 2.7c illustrates another manner in which a truss model is developed for the design of a slender member, according to the ASCE-ACI Committee 445 report. Further it gives the design steps as:

1) A reasonable angle, θ_v, for compression struts is chosen. Typical angles vary from 18^0 to 65^0.
2) The truss model used for design, having a depth equal to the flexural lever arm, jd is simplified to a statically determinate truss as shown in Figure 2.7c. each vertical member represents a group of stirrups within a length jd $\cot\theta_v$. Each diagonal member of the simplified truss represent a zone of diagonal compression which is equal to $jdb_w\cos\theta_v$, where b_w is the width of the web. Once the diagonal force, D, is found, the diagonal compressive stress in the concrete can be found from:

$$f_2 = \frac{D}{b_w jd \cos\theta_v} = \frac{V}{b_w jd \sin\theta_v \cos\theta_v} \qquad (2.4)$$

3) Solve the forces in the truss members and design the transverse and longitudinal reinforcement. Check the compressive stress in the concrete diagonals. Check the anchorage of reinforcement at critical points.

The truss model ignores the shear contribution from concrete and the also from the dowel action of the longitudinal reinforcement and the shear reinforcement is design to resist the entire shear.

2.4.2.1 Truss Approach with Concrete Contribution

The traditional truss model assumes that the compression struts are parallel to the direction of cracking and no stress is transferred across the cracks. This approach has shown to yield conservative results when compared to test evidence. More recent theories have recognized the importance of shear carrying capacity of cracked concrete. As a results of this shear capacity of a non-pre-stressed beam (V_r) has been modified as the addition of:

$$V_r = V_c + V_s \tag{2.5}$$

Where V_s is the shear carrying capacity of shear reinforcement which is found from the truss model and V_c is taken as the shear contribution of concrete. Most of the times value of V_c is found from empirical formulae. In such case, the value of V_c stand for the net effect of shear transfer across the cracks by interlocking of aggregate particles, shear transfer across the uncracked compression zone and the dowel action of the longitudinal reinforcement. Most of the major codes use this approach for the calculation of shear capacity of slender beams.

2.4.3 Compression Field Approaches

The web of a concrete beam cracks due to principle tensile stresses and subsequently resist shear by means of compression struts between the cracks and tensions in stirrups, as described in the truss analogy. Away from the load and the reactions this load carrying mechanism is referred to as compression field. Collins and Mitchell (1974) developed the *Compression Field Theory* to explain the strength and behaviour of such web.

Small transverse tensile stresses can develop in concrete struts between the cracks due to the bond between the stirrups and the strut. These tend to increase the shear carrying capacity of the beam. Collins and Mitchell (1986) extended the compression field theory to include the effect of these tensions and it was called the *Modified Compression Field Theory*.

2.4.3.1 Compression Field Theory

Figure 2.8c shows the average shear distribution over a cracked web of a beam. In the Compression Field Theory (CFT) it is assumed that this shear stress distribution can be represented as a uniformly distributed shear stress over the depth d_v as shown in Figure 2.8e. where d_v is the distance between resultant compression and tension force due to flexure. A further major assumption is that web of a crack beam as shown in Figure 2.8 can be approximated with the uniformly strained reinforced concrete panels having an ε_x and ε_y in longitudinal x and y directions.

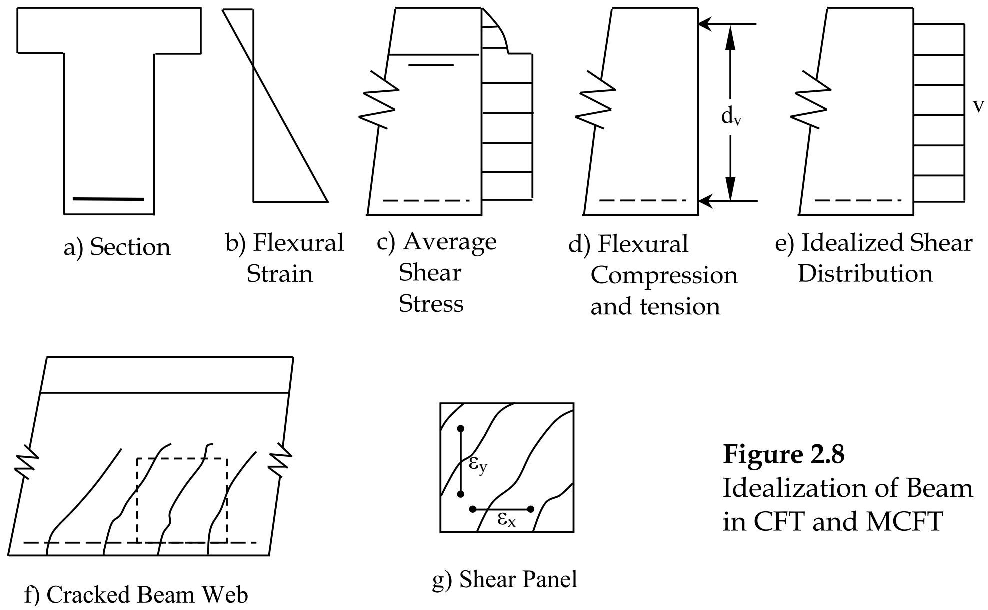


Figure 2.8
Idealization of Beam in CFT and MCFT

For the derivation of Compression Field Theory consider the web of a cracked beam.

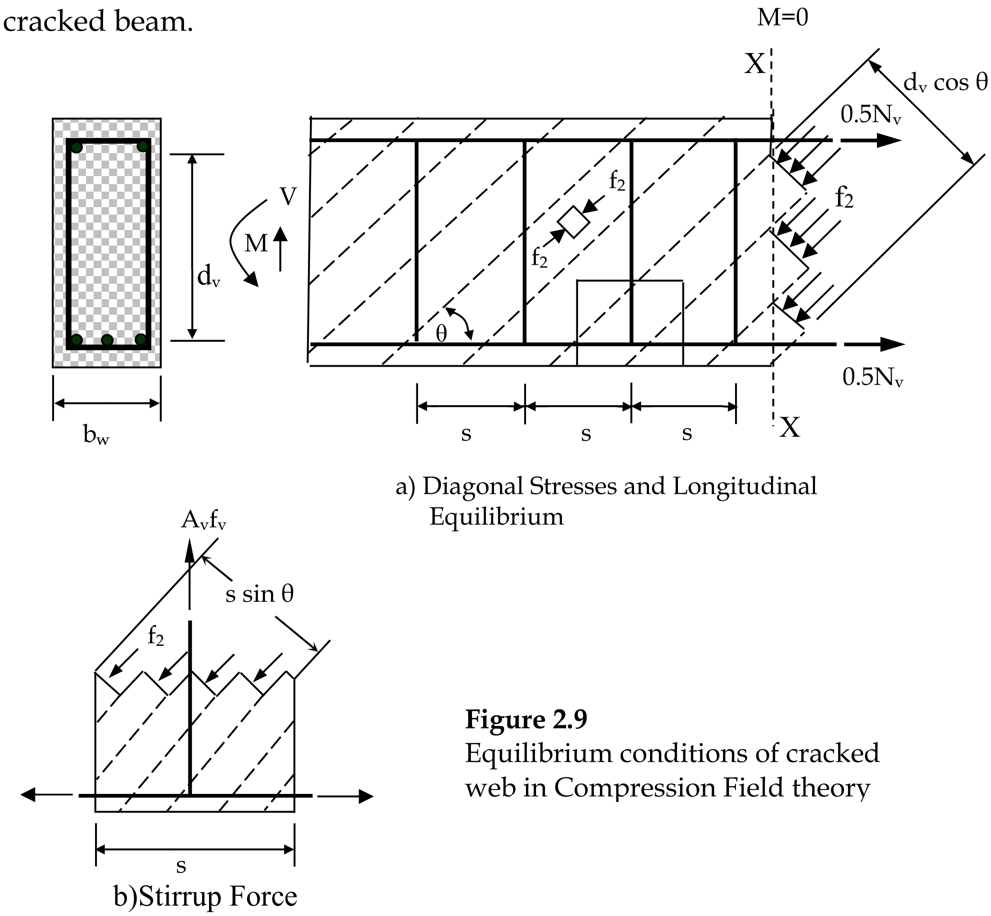


Figure 2.9
Equilibrium conditions of cracked web in Compression Field theory

Equilibrium - Consider the equilibrium at the section X-X of the reinforced concrete beam where the bending moment is zero, as shown in Fig. 2.9.

Compressive stress in concrete: The vertical Component of the diagonal compressive force in the concrete, which inclined at θ to the longitudinal axis, must be equal to the shear force.

i.e. $V = (f_2 b_w d_v \cos\theta)\sin\theta$ (2.6)

Where f_2 is the principle compressive stress.

$$f_2 = \frac{V}{b_w d_v}(\tan\theta + \cot\theta) = v(\tan\theta + \cot\theta)$$
$$f_2 = v(\tan\theta + \cot\theta) \qquad (2.7)$$

It should be mentioned that it above equations were derived on the assumption that concrete does not carry tension after cracking.

Tensile Stress in Stirrups: Consider the equilibrium of part of the beam shown in figure 2.9b. the diagonal compression in the concrete transfer vertical force to the stirrups.

i.e. $A_v f_v = (f_2 b_w s \sin\theta)\sin\theta$

$$\left(\frac{A_v}{sb_w}\right) f_v = \left(\frac{V}{d_v b_w}\right)\tan\theta \Longrightarrow \rho_v f_v = v\tan\theta \qquad (2.8)$$

Tensile Stresses in Longitudinal Reinforcement : the longitudinal component of the diagonal compression in the concrete is equilibrated by the Longitudinal Reinforcement.

$N_v = A_x f_x = V\cot\theta$

$$\left(\frac{A_x}{d_v b_w}\right) f_x = \frac{V}{d_v b_w}\cot\theta \Longrightarrow \rho_x f_x = v\cot\theta \qquad (2.9)$$

Strain Compatibility – Consider the average strains of a element taken from the cracked web.

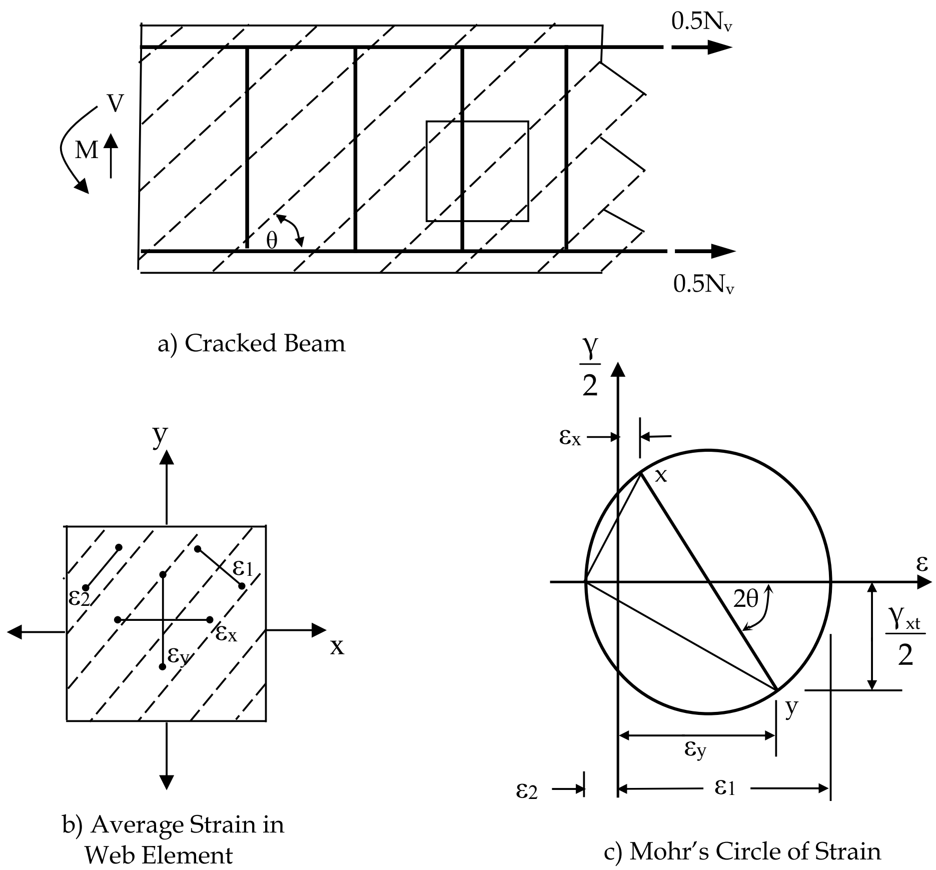


Figure 2.10
Strain Compatibility for cracked web

Using the Mohr's circle of strain in Fig. 2.10c, following useful relationships can be derived:

$$\gamma_{xt} = \frac{2(\varepsilon_x - \varepsilon_2)}{\tan\theta} \tag{2.10}$$

$$\varepsilon_x + \varepsilon_y = \varepsilon_1 + \varepsilon_2 \tag{2.11}$$

$$\tan^2\theta = \frac{\varepsilon_x - \varepsilon_2}{\varepsilon_y - \varepsilon_2} = \frac{\varepsilon_1 - \varepsilon_y}{\varepsilon_1 - \varepsilon_x} = \frac{\varepsilon_1 - \varepsilon_y}{\varepsilon_y - \varepsilon_2} = \frac{\varepsilon_x - \varepsilon_2}{\varepsilon_1 - \varepsilon_x} \tag{2.12}$$

It should be noticed that these compatibility equations are expressed in terms of Average Strains. i.e. strain measured over a based lengths long enough to include several cracks.

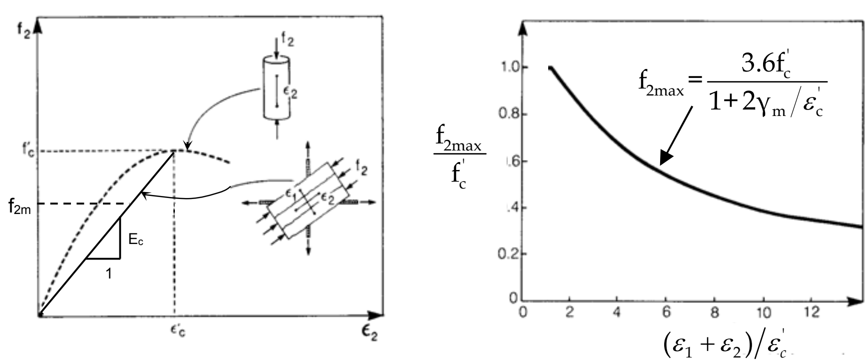
Also it should be mentioned that for the derivation of these relationships it was assumed that the inclination of the diagonal compressive stress coincides with the inclination of the principle compressive strain.

<u>Stress-strain Relationships for Cracked Concrete:</u> Based on the results of a series of intensively instrumented beams, Collins (1978) suggested that the relationship between principal compressive stress, f_2 and principal compressive strain, ε_2 for diagonally cracked concrete would differ from the usual compressive stress strain curve derived from a cylinder test (Figure 2.11). The relationships proposed were

$$f_{2max} = \frac{3.6f_c^{'}}{1+2\gamma_m/\varepsilon_c^{'}} \qquad (2.13)$$

Where γ_m = diameter of the strain circle (i.e $\varepsilon_1+\varepsilon_2$); and ε'_c = strain at which the concrete in cylinder test reaches the peak stress f'_c. for values of f_2 less than f_{2max}

$$\varepsilon_2 = \frac{f_2}{f_c^{'}\varepsilon_c^{'}}$$



a) Stress- Strain relationship for cracked Concrete in Compression

b) Failure Stress in Cracked Concrete

Figure 2.11
Stress- Strain relationship for cracked Concrete in Compression

In addition, the reinforcing steel is assumed to behaves elastically:

$$f_x = E_s\varepsilon_x \tag{2.14}$$

$$f_v = E_s\varepsilon_y \tag{2.15}$$

Thus we have three equilibrium equations, two strain compatible equations, and three stress-strain relations to solve three stress unknowns f_2, f_v, f_x, four strain unknowns ε_x, ε_y, ε_1, ε_2 and the angle of the diagonal compression. In other words a total of eight equations are there to solve for eight unknowns. With these relationships, it is possible to predict the shear strength as well as the load-deformation response of reinforced concrete members subjected to shear. Finally the shear capacity of the beam is given as:

$$V_f = \frac{A_v f_v}{s}\frac{d_v}{\tan\theta} \tag{2.16}$$

2.4.3.2 Modified Compression Field Theory

Modified Compression Field Theory (MCFT) (Vecchio and Collins 1986) is a further development of the CFT that account for the influence of tensile stresses in the concrete. The key simplifying assumption of the modified compression field theory is that the principal strain direction coincides with principal strain direction. This assumption is justified by the experimental measurements which show that the principal stress and strain are parallel within ± 10°. Also this method has recognized the effect of shear transfer across the cracks.

When it comes to the derivation of MCFT, the first two assumptions of CFT are taken as valid. That is, the average shear stress distribution of a cracked web can be represented as a uniformly distributed shear stress over the depth d_v as shown in Figure 2.8e where d_v is the distance between resultant compression and tension force due to flexure. And the web of a cracked beam as shown in Figure 2.8 can be approximated with the uniformly strained reinforced concrete panels having an ε_x and ε_y in longitudinal x and y directions.

For the Derivation of Modified Compression Field Theory Consider the cracked beam web shown in Figure 2.12.

Equilibrium – As in CFT, Consider the equilibrium at the section X-X of the reinforced concrete beam where the bending moment is zero, shown in Fig. 2.12. . Hear f_2 is the principal compressive stress act on a plane which is perpendicular to the cracks. The other principal stress f_1 is the average tensile stress in compression struts between the cracks. Shear in the section is resisted by the diagonal compressive stress f_2 together with the diagonal tensile strength f_1.

$f_2 = f_1 - (AB+BC)$

$AB = v \cot \theta$

$BC = v \tan \theta$

$f_2 = f_1 - v (\cot \theta + \tan \theta)$ (2.17) where $v = \dfrac{V}{b_w d_v}$

Above equation shows that diagonal compressive stresses push apart the flanges while diagonal tensile stresses pull them together. The vertical balance has to be carried by the tension in the web reinforcement. Considering Fig.2.12b, force in a stirrup can be written as:

$$A_v f_v = (f_2 \sin^2\theta - f_1 \cos^2\theta)\, b_w s \qquad (2.18)$$

Similarly the longitudinal imbalance force must be carried by the longitudinal steel.

$$A_x f_x = (f_2 \cos^2\theta - f_1 \sin^2\theta)\, b_w d_v$$

Equilibrium across crack-Failure of the reinforced concrete element may not be governed by the average stresses, but rather by local stress that occurs at a crack. In checking conditions at crack, the actual complex crack pattern is idealized as a series of parallel cracks all of which occurs at an angle θ

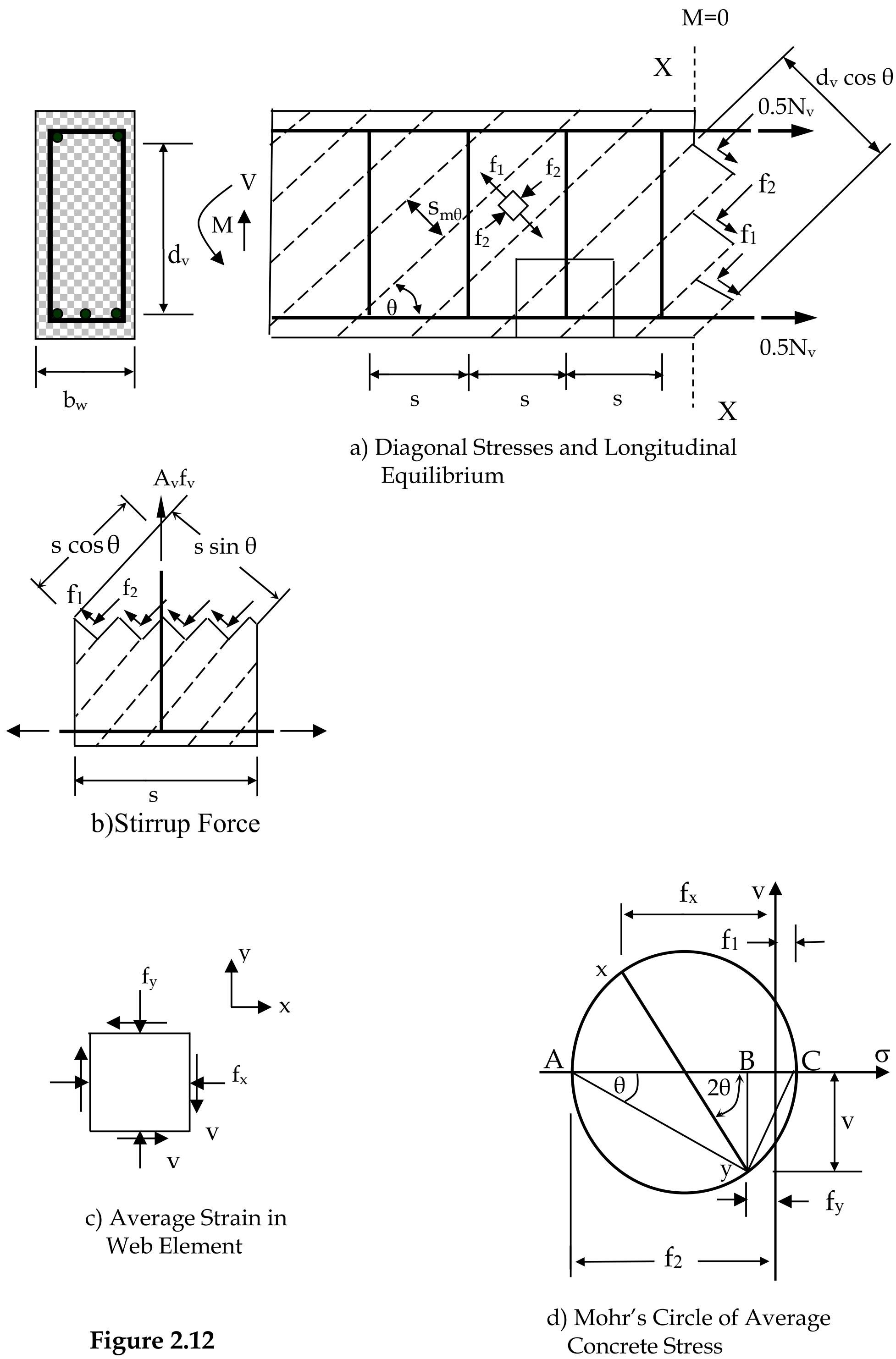


Figure 2.12
Equilibrium conditions of cracked web in Modified Compression Field theory

to the longitudinal reinforcement and space a distance $s_{m\theta}$ apart (Fig 2.12a). The two set of forces, at a crack and between cracks shown in Fig 2.13 a and b. must be statically equivalent to maintain the equilibrium.

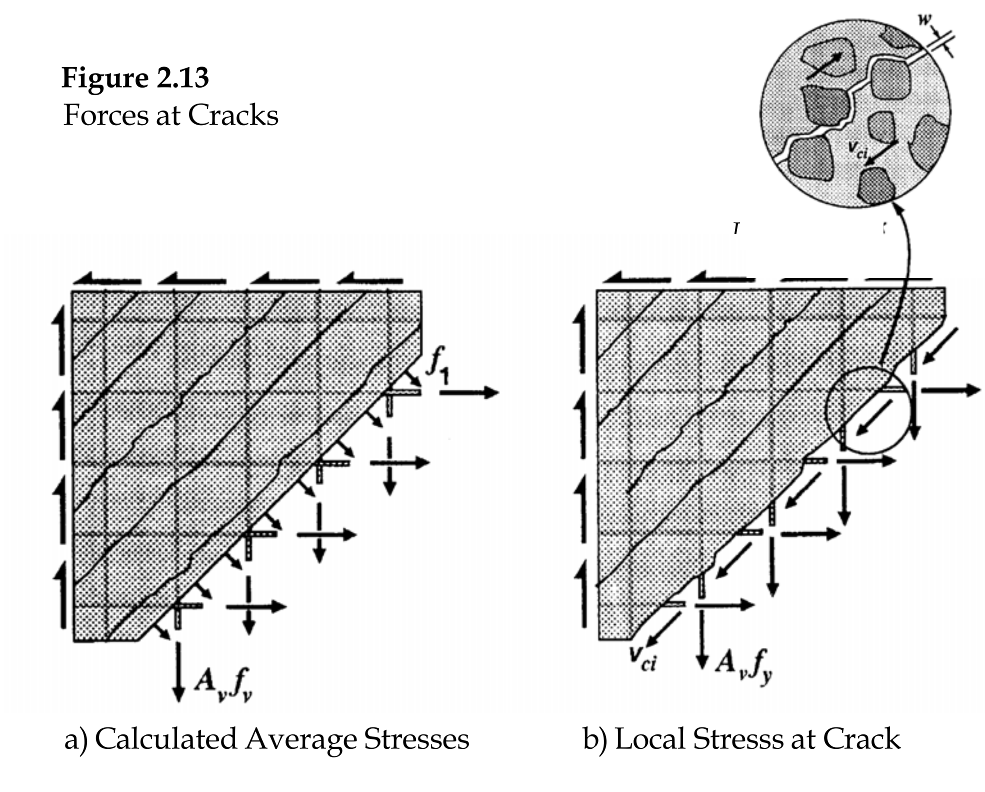
Considering the vertical equilibrium, it can be shown:

$$A_v f_v \left(\frac{d_v}{s \tan\theta} \right) + f_1 \frac{b_w d_v}{\sin\theta} \cos\theta = A_v f_{vy} \left(\frac{d_v}{s \tan\theta} \right) + v_{ci} b_w d_v \qquad (2.19)$$

To maintain this equality the average tensile stress, f_1 must be:

$$f_1 = v_{ci} \tan\theta + \frac{A_v}{b_w s} \left(f_{vy} - f_v \right) \qquad (2.20)$$

Figure 2.13
Forces at Cracks



a) Calculated Average Stresses b) Local Stresss at Crack

From above equation it can be seen that the value of f_1 is tied to the shear that can be transmitted across cracks by aggregate interlock. The ability of the crack interface to transmit the shear stress v_{ci} depends on the crack width w. Using the experimental data of Walraven (1981) Vecchio and Collins (1986) developed the following expression for the limiting value of v_{ci}:

$$v_{ci} = \frac{0.18\sqrt{f_c^{'}}}{0.3 + \frac{24w}{a+16}} \tag{2.21}$$

Where f'_c = Compressive Strength of Concrete in MPa

a = Maximum Aggregate Size

$w = \varepsilon_1 s_{m\theta}$

$$s_{m\theta} = \frac{1}{\frac{\sin\theta}{s_{mx}} + \frac{\cos\theta}{s_{mv}}} \tag{2.22}$$

S_{mx} is the spacing of vertical cracks which would occur in a beam subjected to an axial tension force and s_{mv} spacing of horizontal cracks in a member subjected to transverse tension force. These, in turn, are functions of spacing and cover of the horizontal and vertical reinforcements respectively.

Strain Compatibility : the compatibility equations for the average concrete are the same as described in Compression Field Theory.

$$\varepsilon_x + \varepsilon_y = \varepsilon_1 + \varepsilon_2 \tag{2.11}$$

$$\tan^2\theta = \frac{\varepsilon_x - \varepsilon_2}{\varepsilon_y - \varepsilon_2} = \frac{\varepsilon_1 - \varepsilon_y}{\varepsilon_1 - \varepsilon_x} = \frac{\varepsilon_1 - \varepsilon_y}{\varepsilon_y - \varepsilon_2} = \frac{\varepsilon_x - \varepsilon_2}{\varepsilon_1 - \varepsilon_x} \tag{2.12}$$

Stress Strain Relation ship of Cracked Concrete: Based on test of reinforced concrete panels Vecchio and Collins (1986) suggested

$$\frac{f_{2,max}}{f_c^{'}} = \frac{1}{0.8 + 170\varepsilon_1} \tag{2.23}$$

$$f_2 = f_{2,max}\left[2\left(\frac{\varepsilon_2}{\varepsilon_c^{'}}\right) - \left(\frac{\varepsilon_2}{\varepsilon_c^{'}}\right)^2\right] \tag{2.24}$$

Where $f_{2,max}$= Maximum Compressive Stress of Cracked Concrete

$\varepsilon_c^{'}$ = The Strain at Maximum Compressive Stress.

Collins and Mitchell (1991) suggested

If $\varepsilon_1 < \varepsilon_{cr}$ then $f_1 = E_c \varepsilon_1$ (2.25a)

If $\varepsilon_1 > \varepsilon_{cr}$ then $f_1 = \frac{\alpha_1 \alpha_2 f_{cr}}{1+\sqrt{500\varepsilon_1}}$ (2.25b)

Where ε_{cr}, f_{cr} = Cracking Strain and strength of Concrete

α_1, α_2 = factors accounting for bond characteristics (deformed or smooth bars) and type of loading (short term, cyclic or sustained)

Finally the shear capacity V, of the beam is given by the equation:

$$V = f_1 b_w d_v \cot\theta + \frac{A_v f_v}{s} d_v \cot\theta \qquad (2.26)$$

The above equation together with equations for equilibrium, compatibility and stress-strain properties provide a complete solution by which to predict shear strength of a beam.

2.4.4 Shear Friction Approach

The shear friction approach, as it is known today, was first introduced by Birkeland (1966). It was originally developed to deal with forces transfer cross joints in precast concrete construction. Eventually various researchers have develop this concept to apply even for the reinforced concrete. The method used in this evaluation was proposed by Loov (1998). And this method is presented here.

2.4.4.1 Shear Friction Method by Loov

The base equation used to calculate shear friction strength in this method is:

$$\mathbf{v = k(\sigma f'_c)^{\frac{1}{2}}} \quad (2.27)$$

This equation was developed by the Patnaik (1992) in which v is the average shear strength on potential shear failure plane, σ is the average normal stress on that potential shear failure plane, f'c is the compressive strength of the concrete and k is a constant. Based on various push-off tests done by Kumaraguru (1992), Loov (1998) suggested 0.6 for the k value.

For the inclined plane shown in Fig.2.14 with v = S/A and σ = R/A, the above equation becomes:

$$S = k(Rf'_c A)^{\frac{1}{2}} \quad (2.28)$$

where $A = \dfrac{b_w h}{\sin\theta}$

Solving for R and S using force equilibrium:

$$S = (T - N)\cos\theta - (V - \Sigma T_v)\sin\theta \quad (2.29)$$

$$R = (T - N)\sin\theta - (V - \Sigma T_v)\cos\theta \quad (2.30)$$

The basic equation for the shear strength V is derived using above three equation

$$\frac{V}{C} = 0.5k^2\left[\left(\frac{T - N}{0.25k^2C} + \cot^2\theta\right)^{\frac{1}{2}} - \cot\theta\right](1 + \cot^2\theta) - \frac{T - N}{C}\cot\theta + \Sigma\frac{T_v}{C} \quad (2.31)$$

Where $C = f'_c b_w h$

$T_v = A_v f_y$

For various θ values this equation gives shear strength of various crack planes.

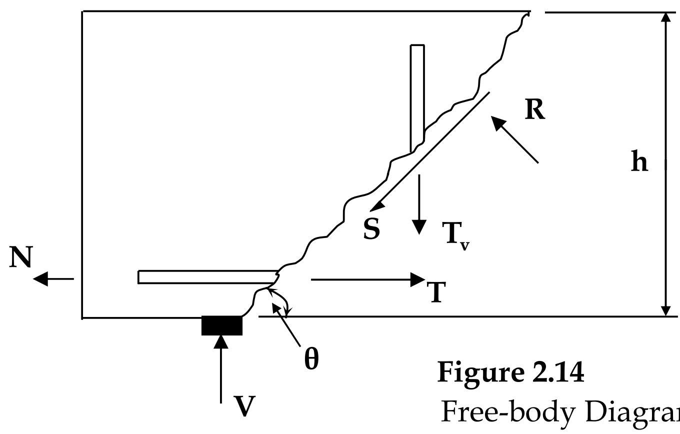


Figure 2.14
Free-body Diagram of a Inclined Plane Through a Crack.

For designing purposes it is necessary to calculate a suitable value for θ. Based on his work Loov (1998) proposed two more equations to use for the designing purposes.

Shear strength of the flattest plane bypassing stirrups occurs when (Fig.2.15):

$$\cot\theta = \frac{s}{d_s}$$

Where s – Spacing of stirrups

d_s – Length of stirrup centre to centre of bars forming top and bottom of stirrup

Then the corresponding shear strength can be calculated from:

$$V = 0.25Ck^2 \frac{d_s}{s} \tag{2.32}$$

Further he proposed the following equation to calculate the shear strength of weakest plane which passes trough stirrups;

$$V = k\left(CT_v \frac{d_s}{s}\right)^{1/2} - T_v \tag{2.33}$$

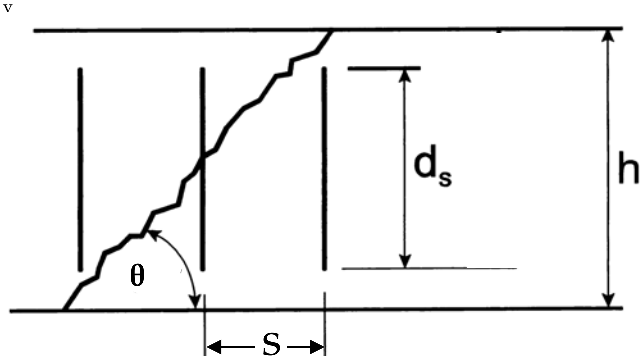


Figure 2.15

Possible Shear failure plane

3. Methodology

3.1 Introduction

As discussed earlier the main objective of this study is to present a critical evaluation of shear design procedures used in the industry. In addition to major codes of practices a recently developed shear design method based on Shear Friction, proposed by Loov (1998) has also been considered for this evaluation. The major codes that have been used for this study are American Code (ACI - 2002), Canadian Code (CSA A23.3 - 1994), British Code (BS 8110 - 1997), Australian Code (AS 3600 - 2001) and Japanese Code (JSCE SP-1 - 1986).

The first step of the methodology was to build a database which consists of beam test results compiled from technical literature. Then shear strength of each and every test beam was predicted using every design method separately. Then the predicted shear strength was compared with the test result to assess the accuracy. Also a Multinomial Logistic Regression analysis was formulated to identifying the key parameters influencing the accuracy of each design procedure.

3.2 Database Preparation

To evaluate the design procedures a data base of test results for more than 950 beams was compiled from technical literature. The test results included in this data base mainly consists of ACI research papers and the data base used by the Prof. R.E. Loov. All beams were selected from those reported to be failing in shear. General rules for selecting test results for the data base were as follows:

-Reinforced concrete beams (no limit on concrete strength)

-Rectangular, T or I beam sections

-No axial loads, no prestressing

-Steel reinforcement (no limit on yield strength)

-Deformed bars have used for the main tensile reinforcement

-Normal weight concrete

-Simply supported or continuous beams with point loads

-Beams are loaded on top chord and supported on the bottom chord

-Reported anchorage and bond failures were removed

-No geometrical limits on member size were used

For each beam, a total of 39 input entries were used to describe its material and geometrical properties, critical shear-span properties at the instant of failure. Fig. 3.1, 3.2 and 3.3 show the typical elevation, end anchorage properties, and cross-section of a possible beam in database. The definitions of the 39 input entries are also given below.

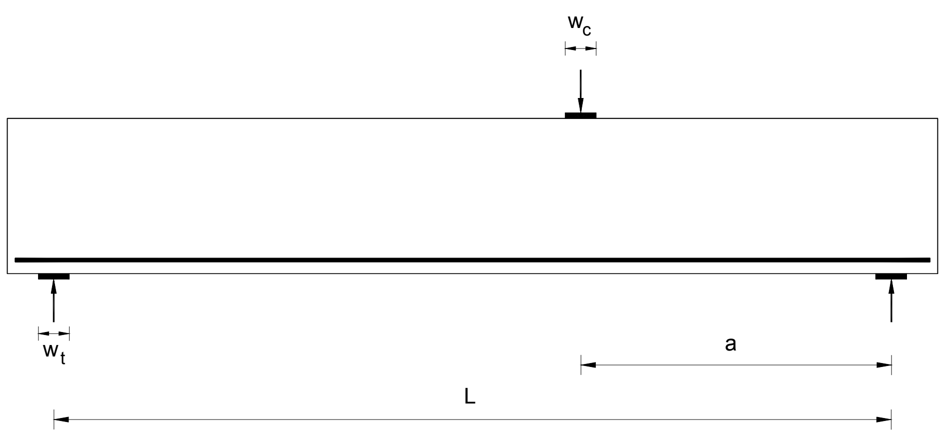


Figure 3.1

Elevation Layout for a typical beam in the database

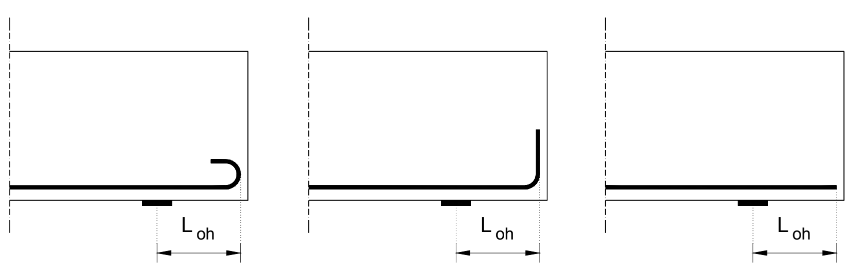


Figure 3.2
Anchorage Properties

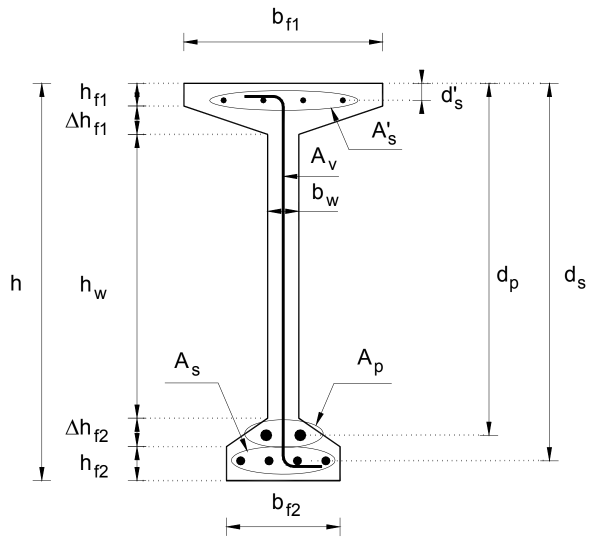


Figure 3.3
Cross-sectional details for a typical beam in the database

Beam elevation definitions (figures 3.4 and 3.5)

0 L (m) Span length centreline to centreline

1 L_{oh} (m) Anchorage length of tensile longitudinal reinforcement

2 a (m) Length of critical shear span

3 w_c (m) Width of load bearing plate

4 w_t (m) Width of support bearing plate

5 End anchor:

Flag = -1 tensile reinforcement anchored to end bearing plates

Flag = 0 tensile reinforcement ends in straight anchorage length

Flag = 1 tensile reinforcement ends in a 90° standard hook

Flag = 2 tensile reinforcement ends in a 180° standard hook

Concrete properties

6	f_c' (MPa)	Uniaxial compressive strength of 150 x 300 mm standard cylindrical specimens. Whenever the concrete strength was given for another size or shape of a specimen, the following modification factors were applied: for 2.75″ (70 mm) cube, multiply by 0.72 for 4″ (100 mm) cube, multiply by 0.76 for 6″ (150 mm) cube, multiply by 0.80 for 8″ (200 mm) cube, multiply by 1.00 for 3″ x 6″ (75 x 150 mm) cylinder, multiply by 0.92 for 4″ x 8″ (100 x 200 mm) cylinder, multiply by 0.95
7	a_g (mm)	Nominal maximum aggregate size for concrete mix, taken as 19 mm if not specified.

Concrete cross-section (figure 3.3)

8	h (m)	Total height of beam cross-section
9	h_{f1} (m)	Thickness of compression flange at its free edge
10	Δh_{f1} (m)	Increase in compression flange thickness at its junction with the web
11	h_w (m)	Clear web height between flanges
12	Δh_{f2} (m)	Increase in tension flange thickness at its junction with the web
13	h_{f2} (m)	Thickness of tension flange at its free edge
14	b_{f1} (m)	Width of compression flange
15	b_w (m)	Thickness of web
16	b_{f2} (m)	Width of tension flange
17	s_z (m)	Spacing between layers of crack control reinforcement
18	A_g (m^2)	Gross cross-sectional area

19	I_g (m^4)	Moment of inertia of gross cross-section about its c.g.
20	y_t (m)	Distance from extreme tension fibre to c.g. of gross cross-section
21	y_c (m)	Distance from extreme compression fibre to c.g. of gross cross section

Conventional longitudinal reinforcement

22	A_{s_fl} (mm^2)	Total area of tension steel at maximum moment section
23	A_{s_sh} (mm^2)	Total area of tension steel at critical shear section
24	d_s (m)	Depth to c.g. of tension steel measured from extreme compression fibre
25	d_b (mm)	Average diameter of tensile reinforcement bars
26	f_y (MPa)	Average yield strength of tensile reinforcing bars
27	A'_s (mm^2)	Total area of compression steel
28	d'_s (m)	Depth to c.g. of compression steel measured from extreme compression fibre
29	d'_b (mm)	Average diameter of compressive reinforcement bars
30	f'_y (MPa)	Average yield strength of compressive steel bars

Stirrups (uniformly distributed, perpendicular to the beam longitudinal axis)

31	A_v (mm^2)	Area of one stirrup
32	s (m)	Stirrup spacing
33	d_{bv} (mm)	Diameter of stirrup steel
34	f_{vy} (MPa)	Yield strength of stirrup steel
35	Stirrup	Flag = 0 Beam without stirrups Flag = 1 Beam with stirrups
36	c_c (m)	Clear concrete cover from stirrup to extreme compression fibre

37	c_t (m)	Clear concrete cover from stirrup to extreme tension fibre

Forces

38	V_t (MN)	Test shear strength
39	N (MN)	Applied concentric end axial force, positive if tension

3.3 Shear Strength of Slender Beams

Most of the design codes handle designing processes of slender beams and deep beams separately. In fact all design codes which have been selected for this study do the same. Different codes recognize Deep Beams and Slender Beams in different manner. Therefore every code has its own definition for Deep Beams and Slender Beams. The definition for Deep beams in each code will be given in Flow Charts given for each design method. But it can be found some common requirements that have to be satisfied by the slender beams which are being designed using any of the design methods considered.

1) Support reactions, in direction of applied shear introduce compression into the end region of the member;
2) Loads are applied at or near the top of the member;
3) No concentrated loads occur between the face of the support and the location of the critical section.

Shear strength of slender beams which satisfy the above requirements were adopt in each and every design method separately. It should be noted that except for the limitation on the maximum shear strength of a section, all the other limitations on material strength, reinforcement areas and spacing have been neglected in this study. This is to ensure that the study of shear behaviour is within range of the parameters defined.

3.3.1 ACI Code – Shear Design Provisions (ACI - 2002)

In the ACI code the basic equation for shear design of a beam is given as:

$V_u \leq \Phi V_n$ where V_u - Factored shear Force at a section

V_n - Shear Resistance at the same section

Φ – Strength reduction factor

In this equation, the shear resistance of the section is based on the parallel truss model with 45^0 constant inclination of diagonals supplemented by an experimentally obtained concrete contribution.

$V_n = V_c + V_s$ where V_c – Shear strength carried by Concrete (Based on Empirical Formula)

V_s - Shear strength carried by Stirrups (Based on Truss Model)

Shear strength carried by concrete V_c.

In the ACI Code, two equations have been given to Calculate Shear contribution of concrete: Equation 11.3 and: equation 11.5. The concrete contribution is considered to represent the net effect of three main shear transfer mechanisms namely Shear transfer across cracks due to aggregate interlock, Shear transfer across uncracked concrete in the compression chord and the dowel action of longitudinal reinforcement.

Equation 11-3

$$V_c = 2\sqrt{f_c^{'}}b_w d \qquad (3.1)$$

where $f_c^{'}$ = Compressive Strength of Concrete (psi)

b_w = Width of the Beam (in)

d = Effective Depth (in)

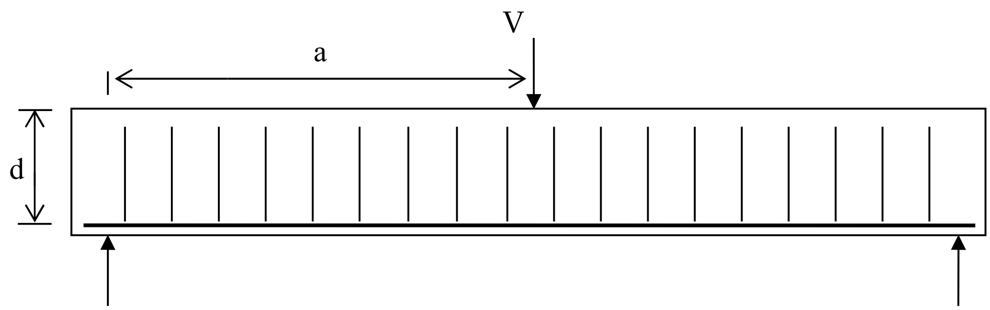
Equation 11-5

$$V_c = \left(1.9\sqrt{f_c'} + 2500\rho_w \frac{V_u d}{M_u}\right) b_w d \leq 3.5\sqrt{f_c'}\, b_w d \qquad (3.2)$$

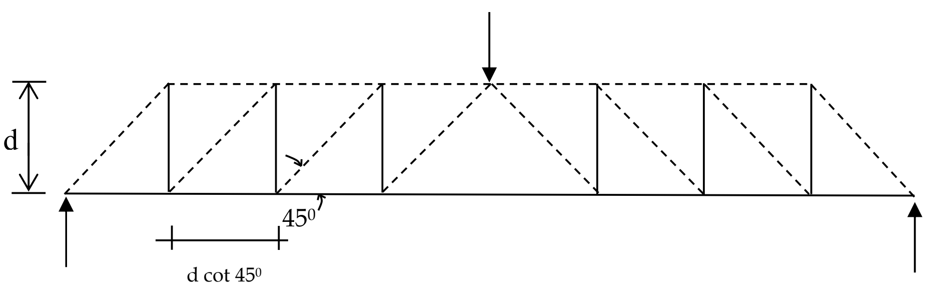
where $\rho_w = \dfrac{A_s}{b_w d}$ and $\dfrac{V_u d}{M_u} \leq 1$

A_s = Area of Longitiudinal Tensile Steel($1n^2$)
V_u and M_u are the ultimate shear force and the moment occurs simultaniously at section considered.

Shear strength provided by the shear reinforcement V_s.



a) Main Reinforcement



b) Simplified Truss Model with parallel compression chords

Figure 3.4
Simplified Truss Model for Slender Beams

For the calculation of shear carrying capacity of stirrups of a slender beams, the real beam is considered to be represented by a simplified parallel chord truss with depth equal to effective depth d (Fig. 3.1b). In this simplified truss model each vertical member represents a group of stirrups within a length d. And each diagonal member represent a zone of diagonal compression which is equal to $db_w \cos 45^0$, where b_w is the width of the web.

Then the total shear force carried by the stirrups within the length d is given by:

$$V_s = \frac{A_v f_{yt} d}{s} \tag{3.3}$$

where A_v = Area of Shear Reinforcement

s = Spacing of Stirrups

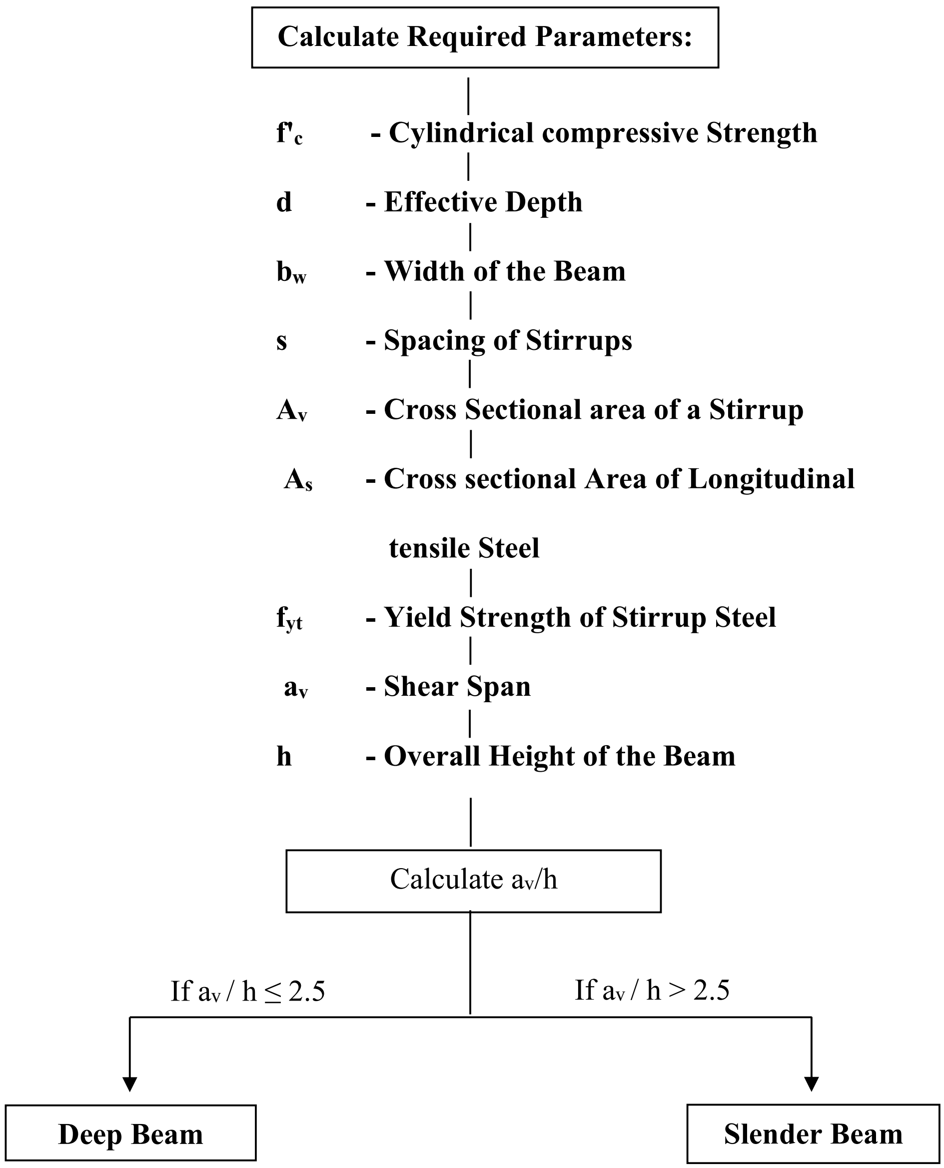
f_{yt} = Yield Strength of Stirrup

In the ACI Code maximum stirrup spacing has been limited to 0.25d and maximum concrete compressive strength of concrete has been limited to 10000 psi.

Flow chart for spread sheet calculation:

Step 1

Calculate Parameters and identify slender beams



Step 2

Predict shear strength (V_n) of beams without Shear reinforcement

Slender Beam without Shear reinforcement

For Beams without Shear Reinforcement $V_n = V_c$

Calculate the V_n

$$V_n = 2\sqrt{f_c'}\,b_w d$$

Predicted Shear Strength from **Equation 11-3**

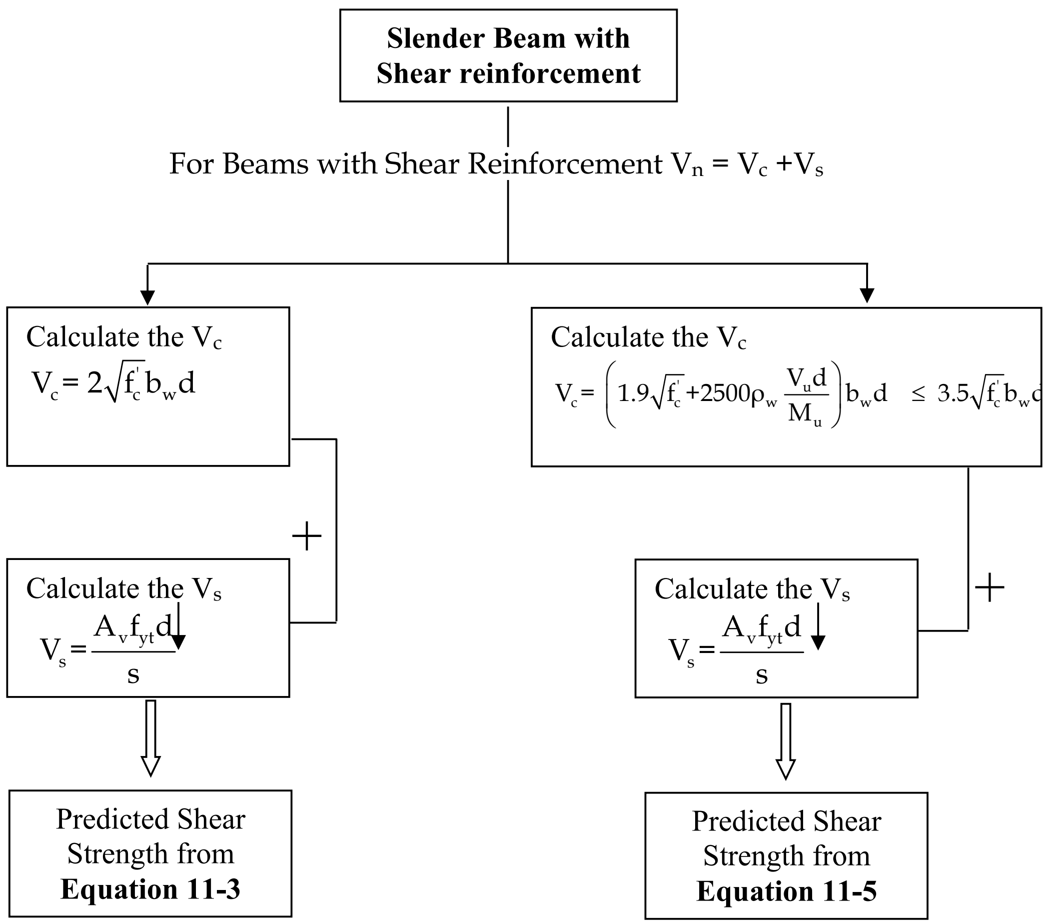
Calculate the V_n

$$V_n = \left(1.9\sqrt{f_c'} + 2500\rho_w \frac{V_u d}{M_u}\right) b_w d$$

Predicted Shear Strength from **Equation 11-5**

Step 3

Predict shear strength (V_n) of beams with Shear reinforcement



3.3.2 BS Code – Shear Design Provisions (BS 8110 - 1997)

Shear design procedure of the BS Code is also based on the parallel chord truss model with concrete contribution. As in the ACI Code, this method also assumes the inclination of compression struts to be equal to 45^0 to the axis of longitudinal tensile reinforcement. As a result, it can be said that this method is also based on a parallel truss model with 45^0 constant inclination diagonals supplemented by an experimentally obtained concrete contribution.

Shear strength carried by concrete V_c.

In the BS Code the shear contribution of concrete is given by the following empirical equation:

$$V_c = 0.79 \text{ x } \left(\frac{100A_s}{b_v d}\right)^{\frac{1}{3}} \text{ x } \left(\frac{400}{d}\right)^{\frac{1}{4}} \text{ x } \left(\frac{f_{cu}}{25}\right)^{\frac{1}{3}} b_v d \tag{3.4}$$

where A_s = Area of Longitiudinal Tensile Steel(mm)

f_{cu} = Compressive Strength of Concrete(MPa)

b_v = Width of the Beam(mm)

d = Effective Depth(mm)

Following limitations have been imposed for the above equation

The maximum value of $\frac{100A_s}{b_v d}$ limited to 3

For Beams without Shear reinforcement $\left(\frac{400}{d}\right)^{\frac{1}{4}} \geq 0.67$

For Beams with shear reinforcement $\left(\frac{400}{d}\right)^{\frac{1}{4}} \geq 1$

Also in BS Code maximum concrete compressive strength of concrete is limited to 40 MPa - Cubical Compressive Strength or 32 MPa - Cylindrical Compressive Strength

Shear strength provided by the shear reinforcement V_s.

Similar to the ACI code, shear carried by the shear reinforcement is given based on the truss model as:

$$V_s = \frac{A_{sv} f_{yv} d}{s_v} \tag{3.5}$$

where A_{sv} = Area of Shear Reinforcement

s_v = Spacing of Stirrups

f_{yv} = Yield Strength of Stirrup

In the BS Code maximum stirrup spacing has been limited to 0.75d and the Minimum Allowable area of shear reinforcement $A_{sv\,min}$ has been limited to:

$$A_{sv\,min} = \frac{0.4 b_v s_v}{f_{vy}} \tag{3.6}$$

Flow chart for spread sheet calculation:

Step 1

Calculate Parameters and identify slender beams

Calculate Required Parameters:

f_{cu} - Cubical compressive Strength

d - Effective Depth

b_v - Width of the Beam

s_v - Spacing of Stirrups

A_{sv} - Cross Sectional area of a Stirrup

A_s - Cross sectional Area of Longitudinal tensile Steel

f_{yv} - Yield Strength of Stirrup Steel

a - Shear Span

l_n - Clear Span

Calculate l_n/d ratios

If $l_n/d < 2$ → **Deep Beam**

IF $l_n/d \geq 2$ → **Slender Beam**

Step 2

Predict shear strength (V_n) of beams without Shear reinforcement

Slender Beam without Shear Reinforcement

For Beams without Shear Reinforcement $V_n = V_c$

Calculate the V_n

$$V_n = 0.79 \text{ x } \left(\frac{100A_s}{b_v d}\right)^{\frac{1}{3}} \text{ x } \left(\frac{400}{d}\right)^{\frac{1}{4}} \text{ x } \left(\frac{f_{cu}}{25}\right)^{\frac{1}{3}} b_v d$$

Predicted Shear Strength from **BS 8110**

Step 3

Predict shear strength (V_n) of beams with Shear reinforcement

Slender Beam with Shear Reinforcement

For Beams with Shear Reinforcement $V_n = V_c + V_s$

Calculate the V_s

$$V_s = \frac{A_{sv} f_{vy} d}{s_v}$$

+

Calculate the V_c

$$V_c = 0.79 \text{ x } \left(\frac{100A_s}{b_v d}\right)^{\frac{1}{3}} \text{ x } \left(\frac{400}{d}\right)^{\frac{1}{4}} \text{ x } \left(\frac{f_{cu}}{25}\right)^{\frac{1}{3}} b_v d$$

Predicted Shear Strength from **BS 8110**

3.3.3 Australian Code – Shear Design Provisions (AS 3600-2001)

Shear design procedure of the Australian Code is based on the variable angle truss model with concrete contribution. Similar to the previous two codes, concrete contribution of the Australian Code is also based on an empirical formula. But depending on the magnitude of the shear force the Australian Code allows designers to use different strut angles for the truss model when it comes to the calculation of shear carrying capacity of stirrups.

<u>*Shear strength carried by concrete* V_{uc}.</u>

In the Australian Code the shear contribution of concrete (V_{uc}) is given from the following empirical equation:

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_0 \frac{A_{st} f_c^{'}}{b_v d_0} \qquad (3.7)$$

where A_{st} = Area of Longitiudinal Tensile Steel(mm^2)

$f_c^{'}$ = Compressive Strength of Concrete (MPa)

b_v = Width of the Beam(mm)

d_0 = The distance from the extreme compression fibre of the concrete to the centroid of the outermost layer of tensile reinforcement(mm)

$\beta_1 = 1.1(1.6 - d_o/1000) > 1.1$

β_2 = 1; or

$= 1 - (N^* / 3.5A_g) > 0$ for members subject to significant axial tension; or

$= 1 + (N^* / 14A_g)$ for members subject to significant axial compression

β_3 = 1; or may be taken as -

$= 2d_0/a_v$ but not greater than 2, provided that the applied loads and the support are orientated so as to create diagonal compression over the length a_v

N^* = Magnitude of Axial Force

A_g = Gross Cross - Sectional Area of the Member

Shear strength provided by the shear reinforcement V_{us}.

Unlike in previous two codes, this method allows designer to select a suitable strut angle (θ_v) for the truss model given in Fig.2.7c. According to the code, depending on the applied shear force, θ_v can be a value between 30^0 to 45^0. Further it defines θ_v to vary linearly from 30^0 when $V^* = \Phi V_{u.min}$ and 45^0 when $V^* = \Phi V_{u.max}$ where Φ is a strength reduction factor and $V_{u.min}$ is the shear resistance of a beam with minimum shear reinforcement and $V_{u.max}$ is the maximum allowable shear for a section.

$$V_{u.min} = V_{uc} + 0.6 b_v d_0 \quad (3.8)$$

$$V_{u.max} = 0.2 f_c^{'} b_v d_0 \quad (3.9)$$

Then the total shear force carried by the stirrups within the length $d\cot\theta_v$ can be calculated:

$$V_{us} = \frac{A_{sv} f_{sy.f} d_0}{s} \cot\theta_v \quad (3.10)$$

where A_{sv} = Area of Shear Reinforcement

s = Spacing of Stirrups

$f_{sy.f}$ = Yield Strength of Stirrup

θ_v = Strut angle of the truss model

As we are dealing with ultimate shear capacity of beams, value of θ_v was taken as 45^0 for this study.

In order to prevent web crushing, the Australian Code limits the maximum shear ($V_{u.max}$) that can be carried by a beam to:

$$V_{u.max} = 0.2 f_c^{'} b_v d_0 \quad (3.11)$$

In the Australian Code, maximum stirrup spacing has limited to the lesser of 0.5D or 300 mm. When $V^* < \Phi V_{u.min}$, the maximum spacing limit can be increased to the lesser of 0.75D or 500 mm, where D is the overall depth of a cross-section in the plane of bending.

Also mminimum allowable area of shear reinforcement $A_{sv,min}$ is limited to:

$$A_{sv.min} = \frac{0.35 b_v s}{f_{sy.f}} \tag{3.12}$$

Flow chart for spread sheet calculation:

Step 1

Calculate Parameters and identify slender beams

Calculate Required Parameters:

f'_c - Cylindrical compressive Strength

d_0 - The distance from the extreme compression fibre of the concrete to the centroid of the outermost layer of tensile reinforcement

b_v - Width of the Beam

s - Spacing of Stirrups

A_{sy} - Cross Sectional area of a Stirrup

A_s - Cross sectional Area of Longitudinal tensile Steel

$f_{sy.f}$ - Yield Strength of Stirrup Steel

a_v - Shear Span

l_n - Clear Span

D - Overall Depth of the member

Calculate l_n/D ratios

If $l_n/D < 3$ → **Deep Beam**

IF $l_n/D \geq 3$ → **Slender Beam**

Step 2

Predict shear strength (V_u) of beams without Shear reinforcement

Slender Beam without Shear Reinforcement

For Beams without Shear Reinforcement $V_u = V_{uc}$

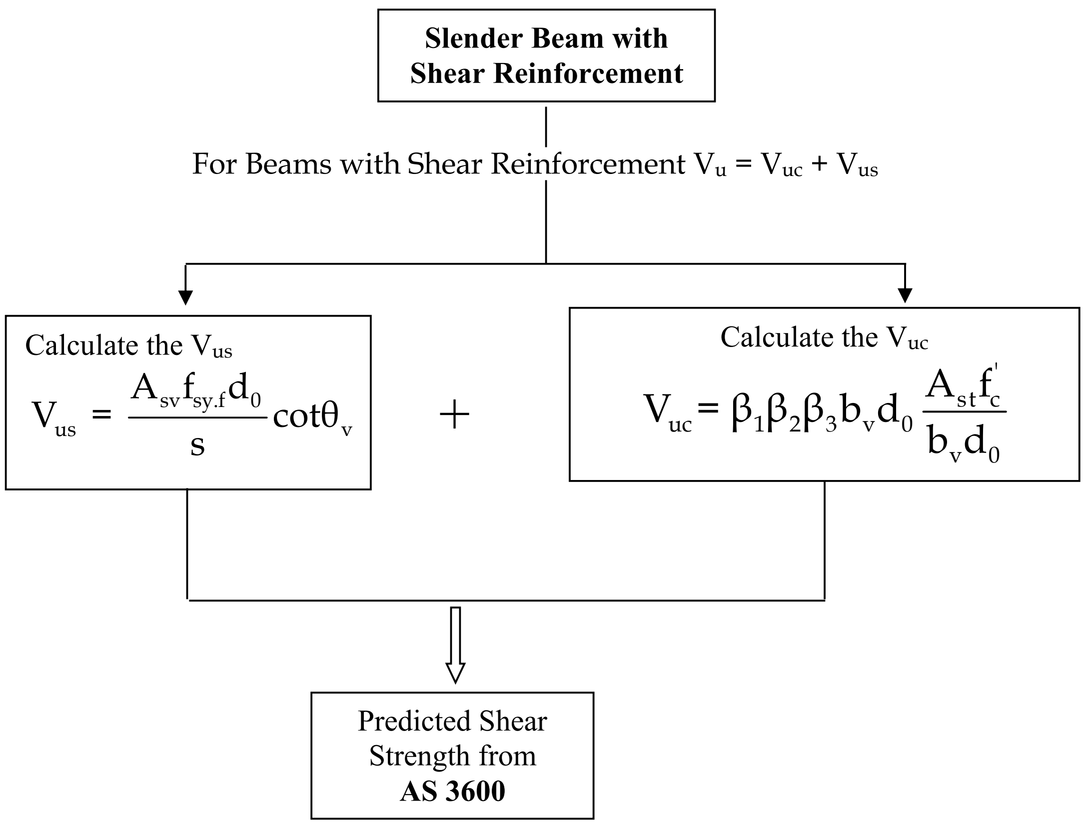
Calculate the V_c

$$V_u = \beta_1 \beta_2 \beta_3 b_v d_0 \frac{A_{st} f_c^{'}}{b_v d_0}$$

Predicted Shear Strength from **AS 3600**

Step 3

Predict shear strength (V_u) of beams with Shear reinforcement



3.3.4 Japanese Code –Shear Design Provisions (JSCE SP-1 - 1986)

The Japanese Code also uses the parallel chord truss model with concrete contribution to calculate shear strength of a slender beam. This method also assumes the inclination of compression struts to be equal to 45^0 to the axis of longitudinal tensile reinforcement and the concrete contribution is calculated using an empirical formula.

Shear strength carried by concrete V_{cd}.

The Japanese Code uses the following empirical equation to calculate the shear contribution of concrete:

$$V_{cd} = f_{vcd} b_w d \qquad (3.13)$$

where

$f_{vcd} = 0.9\beta_d\beta_p\beta_n\sqrt[3]{f'_{cd}} \quad (kgf/cm^2)$

$\beta_d = \sqrt[4]{100/d} \ \leq \ 1.5 \ \ (d : cm)$

$\beta_p = \sqrt[3]{100\rho_w} \ \leq \ 1.5 \ : \ \rho_w = (As/b_w d)$

$\beta_n = (1 + M_0/M_d) \ \leq \ 2 \quad \text{when} \quad (N'_d \geq 0)$

$\beta_n = (1 + 2M_0/M_d) \ \geq \ 0 \quad \text{when} \quad (N'_d < 0)$

f'_{cd} = Compressive Strength of Concrete (kgf/cm^2)

b_w = Width of the Beam (cm)

d = Effective Depth (cm)

A_s = Area of Longitiudinal Tensile Steel (cm^2)

M_d = Design moment

M_0 = Decompression moment necessory to cancel the fibre stress due to axial force at the tension fibre corresponding to design moment M_d

N'_d = Design axial force (compression +ve)

Shear strength provided by the shear reinforcement V_{sd}.

The shear carried by the stirrups within a distance of d is given from:

$$V_{sd} = \frac{A_w f_{wyd}}{s_s} z \quad \text{where } A_w = \text{Total Area of a Stirrup} \tag{3.14}$$

s_s = Spacing of Stirrups
f_{wyd} = Yield Strength of Stirrup
z = Distance from compression resultamt to centroid of tension steel
Generally, may be taken as d/1.15

Maximum Stirrup Spacing has been limited to 0.5d and the Minimum Allowable Area of Shear Reinforcement $A_{sv\ min}$ has been limited to:

$$A_{w\ min} = 0.0015\ b_w s \tag{3.15}$$

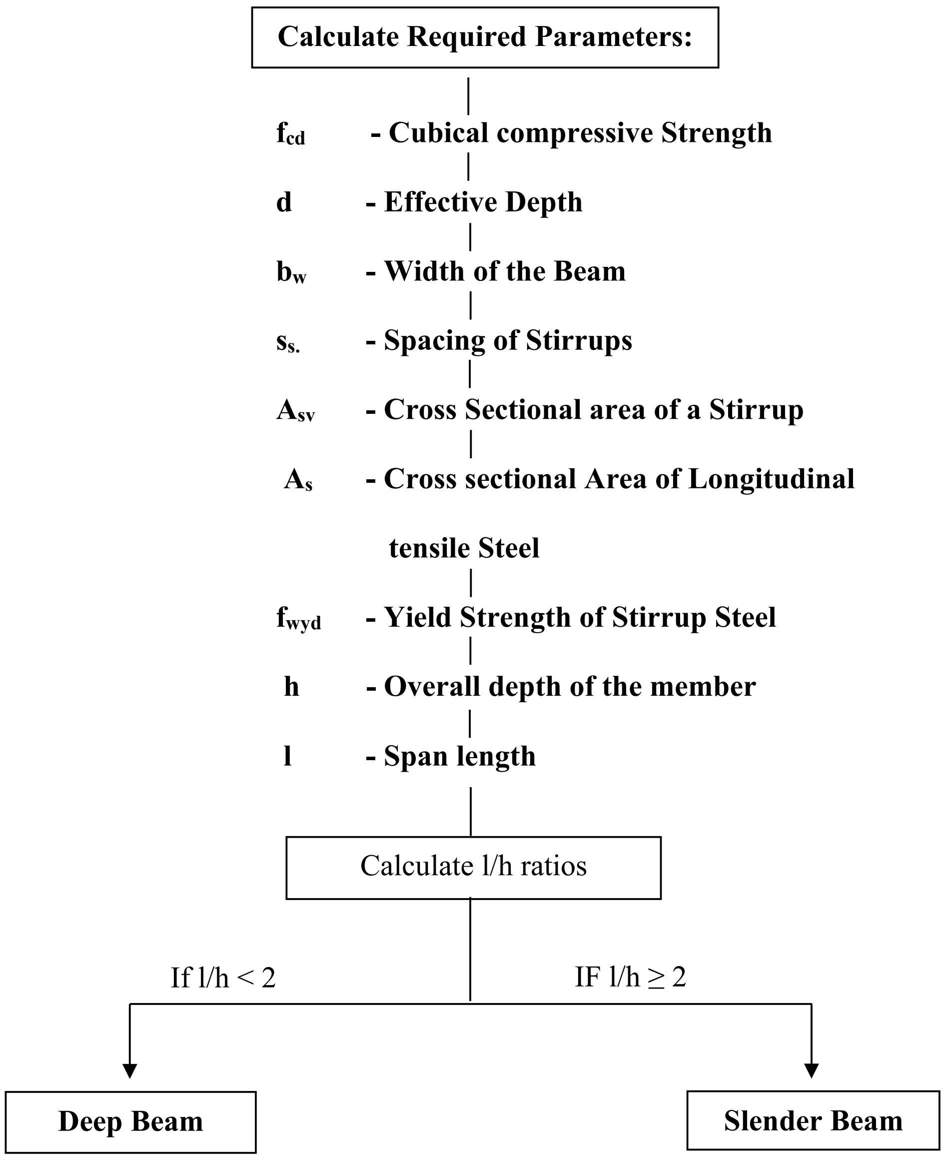
In order to prevent web crushing, the maximum shear carrying capacity of a cross section has been limited to;

$$V_{wcd} = f_{wcd} b_w d \quad \text{where} \quad f_{wcd} = 4\sqrt{f'_{cd}} \quad (\text{kgf/cm}^2) \tag{3.16}$$

Flow diagram for spread sheet calculation:

Step 1

Calculate Parameters and identify slender beams



Step 2

Predict shear strength (V_{yd}) of beams without Shear reinforcement

Slender Beam without Shear Reinforcement

For Beams without Shear Reinforcement $V_{yd} = V_{cd}$

↓

Calculate the V_{cd}

$$V_{yd} = 0.9\beta_d\beta_p\beta_n\sqrt[3]{f'_{cd}}\ b_w d$$

⇓

Predicted Shear Strength from **JSCE SP-1**

Step 3

Predict shear strength (V_{yd}) of beams with Shear reinforcement

Slender Beam with Shear Reinforcement

For Beams with Shear Reinforcement $V_{yd} = V_{cd} + V_{sd}$

Calculate the V_{sd}

$$V_{sd} = \frac{A_w f_{wyd}}{s_s} z$$

$+$

Calculate the V_{cd}

$$V_{cd} = f_{vcd} b_w d$$

Predicted Shear Strength from **JSCE SP-1**

3.3.5 Canadian Code – Shear Design Provisions (CAN A23.3 - 1994)

In the Canadian Code, two different procedures have been given for shear designs: *A Simplified method* and *A General method*. Simplified method is based on the parallel chord truss model with concrete contribution where as general method is based on modified compression field theory. The simplified method is permitted for flexural members which are not subjected to significant axial tension. And for other cases it is recommended to use the general method.

Simplified Method of Canadian Code

This method is also based on the parallel truss model with 45^0 constant inclination diagonals supplemented by an experimentally obtained concrete contribution.

Shear strength carried by concrete V_c.

In order to calculate shear contribution of concrete, Canadian code presents two different equations depending on the effective depth of the beam.

when $d \leq 300$mm

$$V_c = 0.167\sqrt{f_c'}b_w d \qquad (3.17)$$

$d > 300$mm

$$V_c = \left(\frac{260}{1000+d}\right)\sqrt{f_c'}b_w d \qquad (3.18)$$

where f_c' = Compressive Strength of Concrete (MPa)

b_w = Width of the Beam (mm)

d = Effective Depth (mm)

Here the first equation which is permitted to use when d ≤ 300mm and the equation 11.3 of ACI code, are based on the empirical equation proposed in ASCE-ACI - 318 committee report.

In the Canadian Code maximum value of f_c' has been limited to 80MPa .

Shear strength provided by the shear reinforcement V_s.

As mentioned above the shear contribution of shear reinforcement is based on the truss analogy. And it is given as:

$$V_s = \frac{A_v f_y d}{s} \tag{3.19}$$

where A_v = Area of Shear Reinforcement

s = Spacing of Stirrups

f_y = Yield Strength of Stirrup

The maximum spacing of stirrups is limited to the lesser of 0.35d or 300mm and the minimum area of shear reinforcement is limited to:

$$A_{v,min} = \frac{0.06\sqrt{f_c'}\, b_w s}{f_y} \tag{3.20}$$

Also it limits the maximum shear that can be carried by the shear reinforcement to:

$$V_{s,max} = 0.8\sqrt{f_c'}\, b_w d \tag{3.21}$$

This limitation is given to guard against excessive crack widths and to provide safety against the web crushing in reinforced concrete beams.

Flow chart for spread sheet calculation:

Step 1

Calculate Parameters and identify slender beams

Calculate Required Parameters:

f'_c - Cylindrical compressive Strength

d - Effective Depth

b_w - Width of the Beam

s - Spacing of Stirrups

A_v - Cross Sectional area of a Stirrup

A_s - Cross sectional Area of Longitudinal tensile Steel

f_y - Yield Strength of Stirrup Steel

a - Shear Span

l_n - Clear Span

h - Overall Height of the Beam

Calculate a/d ratio

If $a/d \leq 2.5$ → **Deep Beam**

If $a/d \geq 2.5$ → **Slender Beam**

Step 2

Predict shear strength (V_r)of beams without Shear reinforcement

Slender Beam without Shear Reinforcement

For Beams without Shear Reinforcement $V_r = V_c$

↓

Calculate the V_r

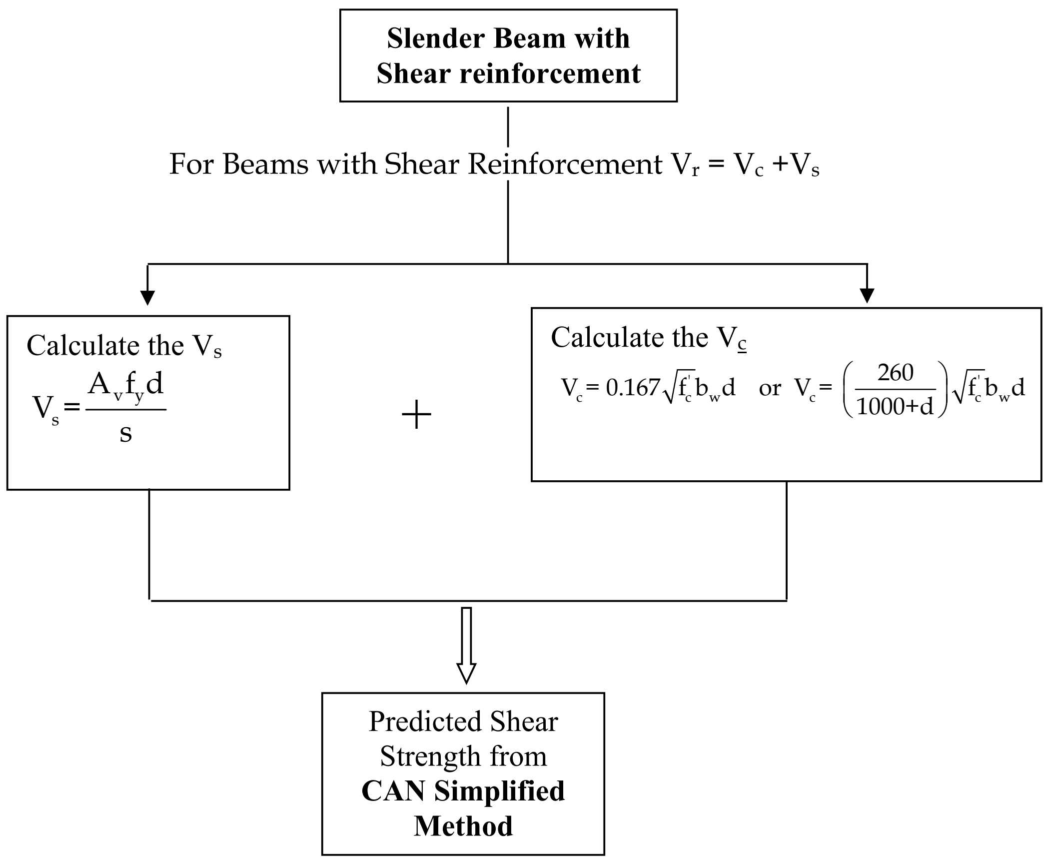
$$V_r = 0.167\sqrt{f_c^{'}}\,b_w d \quad \text{or} \quad V_r = \left(\frac{260}{1000+d}\right)\sqrt{f_c^{'}}\,b_w d$$

⇓

Predicted Shear Strength from **CAN Simplified Method**

Step 3

Predict shear strength (V_r) of beams with Shear reinforcement



General Method of Canadian Code

The General method is based on he modified compression field theory. According to the general method also, shear resistance (V_r) of a reinforced concrete beam is given by:

$V_r = V_c + V_s$

where V_c – Shear force carried by Concrete

V_s - Shear force carried by shear reinforcement.

The value of V_c is given from the equation:

$$V_c = \beta\sqrt{f_c^{'}}b_w d_v \tag{3.22}$$

where β = A funstion of principale tensile stress f_1 of the cracked beam

b_w = Width of the beam

d_v = Distance between resultant of compressive force and tension force

The value of Vs can be calculated using following equation:

$$V_s = \frac{A_v f_s d_v}{s}\cot\theta \tag{3.23}$$

where A_v = Cross sectional area of a stirrup

f_s = Stress of the shear reinforcement

θ = Inclination of the cracks to the longitudinal axis

Predicting the shear strength of a slender beam using above equation, separately for beams without and with shear reinforcement are illustrated below.

Beams without Shear Reinforcement

As mentioned above, In the general method, the shear resistance of a beam without web reinforcement is given as:

$$V_r = \beta\sqrt{f_c^{'}}\,b_w d_v \tag{3.22}$$

where $\beta = \dfrac{f_1 \cot\theta}{\sqrt{f_c^{'}}}$

Here f_1 is the principal tensile stress in cracked web and the value of f_1 is given as:

$$f_1 = \frac{\alpha_1 \alpha_2 f_{cr}}{1+\sqrt{500\varepsilon_1}} \le v_{ci}\tan\theta \tag{3.24}$$

where v_{ci} = shear stress transmit across a crack and

Bhide and Collins (1989) proposed: $v_{ci} = \dfrac{0.18\sqrt{f_c^{'}}}{0.3 + \dfrac{24\varepsilon_1 s_{m\theta}}{a+16}}$

α_1, α_2 = Factors accounting for bond characteristics and type of loading

f_{cr} = Tensile stress at cracking

θ = Crack Angle

ε_1 = Principal tensile strain of cracked concrete web

a = Maximum Aggregate Size

$$s_{m\theta} = \frac{1}{\dfrac{\sin\theta}{s_{mx}} + \dfrac{\cos\theta}{s_{mv}}}$$

s_{mx} = Spacing of vertical cracks which would occur in a beam subjected to an axial tension force

s_{mv} = Spacing of horizontal cracks in a member subjected to transverse tension force

Finding the value of f_1 and consequently the ultimate shear resistance is an iterative process. Steps involved in this procedure are given below:

Step1:

α_1 = 1, for deformed bars

α_2 = 1, for short term loading to failure

Step 2: Estimate a value for θ.

Step 3: Choose a value for ε_1. A good first estimation is the concrete strain at cracking ε_{cr}:

$$\varepsilon_1 = \frac{0.33\sqrt{f_c^{'}}}{E_c} \tag{3.25}$$

where E_c = Young's Moduls of concrete

Step 4 : Calculate mean spacing of inclined cracks, $S_{m\theta}$:

$$s_{m\theta} = \frac{1}{\dfrac{\sin\theta}{s_{mx}} + \dfrac{\cos\theta}{s_{mv}}} \tag{3.26}$$

For beams without shear reinforcement, due to the absence of stirrups $S_{m\theta}$ depends only on S_{mx} , which is the spacing of vertical cracks which would occur in a beam subjected to an axial tension force. Canadian code defines this parameter as the smaller of the d_v or the distance between layers of crack control reinforcement.

Step 5: Calculate Principal tensile stress in cracked concrete, f_1, using the constitutive relationship proposed by Collins and Mitchell (1991):

If $\varepsilon_1 < \varepsilon_{cr}$ then $f_1 = E_c\varepsilon_1$

If $\varepsilon_1 > \varepsilon_{cr}$ then $f_1 = \dfrac{\alpha_1\alpha_2 f_{cr}}{1 + \sqrt{500\varepsilon_1}}$

* the value of f_{cr} is assumed to be equal to $0.33\sqrt{f_c^{'}}$

Step 6: Calculate V_r from equation:

$$V_r = \beta\sqrt{f_c^{'}}\, b_w d_v \tag{3.27}$$

where $\beta = \dfrac{f_1\cot\theta}{\sqrt{f_c^{'}}}$

Step 7: Calculate the principal compressive stress, f_2 of the cracked beam using the stress relationship:

$$f_2 = f_1 - \frac{V_n}{b_w d_v}(\tan\theta + \cot\theta) \tag{3.28}$$

Step 8: Calculate the allowable maximum value of f_2 in order to safe guard against the web crushing using the following relationships:

$$\frac{f_{2,max}}{f_c^{'}} = \frac{1}{0.8 + 170\varepsilon_1} \tag{3.29}$$

If $f_2 \leq f_{2,max}$ then calculate ε_2 using the relationship:

$$\varepsilon_2 = -0.002\,(1 - \sqrt{(1 - f_2/f_{2,max}}\,)$$

If $f_2 > f_{2,max}$ then return to step select a smaller ε_1

Step 9 : Calculate the Strain in longitudinal tensile steel ε_x using the Equation:

$$\varepsilon_x = \frac{0.5V_n \cot\theta + \dfrac{M}{d_v}}{E_s A_s} \tag{3.30}$$

where M= Bending moment at the section concidered

E_s = Modulus of elasticity of steel

A_s = Area of longitudinal tensile steel.

If calculated value of ε_x is greater than the yield strain, ε_y of longitudinal tensile reinforcement then return to step 3 and choose a smaller ε_1 value.

Note: When calculating the yield strain of longitudinal tensile strain modulus of elasticity of steel was assumed to be equal to 250 GPa.

$$\varepsilon_y = \frac{f_y}{E_s} \tag{3.31}$$

Step 10: Calculate the principal compressive strain ε_1 using the relationship between strains:

$$\tan^2\theta = \frac{\varepsilon_x - \varepsilon_2}{\varepsilon_1 - \varepsilon_x} \tag{3.32}$$

Check weather the ε_1 value obtained from this equation matches with the estimated value of ε_1 in Step2. If not return to Step 2 and use this ε_1 value as the new initial estimation and continue the procedure.

Step 11: Calculate the shear stress transfer across the crack v_{ci} using:

$$v_{ci} = \frac{0.18\sqrt{f_c'}}{0.3 + \dfrac{24\varepsilon_1 s_{m\theta}}{a + 16}} \tag{3.33}$$

Step 12: In order to prevent slip alone the cracked, the maximum value of f_1 has been limited to:

$$f_1 \le v_{ci} \tan\theta$$

This has been done considering the equilibrium of forces along a crack and between two cracks.

If calculated value of f_1 satisfies above condition proceed to the next step else return to Step 2 and choose a higher value for θ.

Step 13: Predicted shear strength of the beam V_r;

$$V_r = \beta\sqrt{f_c'}\,b_w d_v \tag{3.22}$$

where $\beta = \dfrac{f_1 \cot\theta}{\sqrt{f_c'}}$

Note: for a particular beam shear strength was calculated for crack angle θ, varies from 15^0 to 60^0 and the maximum predicted shear strength was selected.

Beams with Shear Reinforcement

In the general method, the shear resistance of a beam with web reinforcement V_r, is given as:

$$V_r = \beta\sqrt{f_c'}\,b_w d_v + \frac{A_v f_s d_v}{s}\cot\theta \qquad \text{where } \beta = \frac{f_1 \cot\theta}{\sqrt{f_c'}} \tag{3.34}$$

Therefore it can be rearranged as:

$$V_r = f_1 b_w d_v \cot\theta + \frac{A_v f_s d_v}{s}\cot\theta \tag{3.35}$$

Predicting shear strength using above equation is also an iterative process. Steps involved in that procedure is given bellow:

Step1:

$\alpha_1 = 1$, for deformed bars

$\alpha_2 = 1$, for monolithic short term loading to failure

Step 2: Estimate initial value for θ value

Step 3: Chose a value for ε_1. A good first estimation is the concrete strain at cracking ε_{cr}:

Step 4: Estimate a value for the stress in the of the stirrups f_s. A good first estimation is the yield strength of the steel.

Step 5 : Calculate mean spacing of inclined cracks, $S_{m\theta}$:

$$s_{m\theta} = \frac{1}{\dfrac{\sin\theta}{s_{mx}} + \dfrac{\cos\theta}{s_{mv}}} \tag{3.26}$$

In the Canadian code spacing of inclined cracks has assumed as 305mm. it is believed that this value is appropriate for the full range of beams containing stirrups. Therefore $s_{m\theta}$ was taken equal to 305mm in this study too.

Step 6: Calculate Principal tensile stress in cracked concrete, f_1, using the constitutive relationship proposed:

If $\varepsilon_1 < \varepsilon_{cr}$ then $f_1 = E_c \varepsilon_1$

If $\varepsilon_1 > \varepsilon_{cr}$ then $f_1 = \dfrac{\alpha_1 \alpha_2 f_{cr}}{1 + \sqrt{500\varepsilon_1}}$

* the value of f_{cr} is assumed to be equal to $0.33\sqrt{f_c^{'}}$

Step 7: Calculate V_r from equation:

$$V_r = f_1 b_w d_v \cot\theta + \frac{A_v f_s d_v}{s} \cot\theta \tag{3.35}$$

Step 8: Calculate the principal compressive stress, f_2 of the cracked beam using the stress relationship:

$$f_2 = f_1 - \frac{V_n}{b_w d_v}(\tan\theta + \cot\theta) \tag{3.28}$$

Step 9: Calculate the allowable maximum value of f_2 in order to safe guard against the web crushing using the following relationships:

$$\frac{f_{2,max}}{f_c^{'}} = \frac{1}{0.8 + 170\varepsilon_1} \tag{3.29}$$

If $f_2 \leq f_{2,max}$ then calculate ε_2 using the relationship:

$$\varepsilon_2 = -0.002\,(1 - \sqrt{(1 - f_2/f_{2,max}})$$

If $f_2 \leq f_{2,max}$ then return to step select a smaller ε_1

Step 10 : Calculate the Strain in longitudinal tensile steel ε_x using the Equation:

$$\varepsilon_x = \frac{0.5V_n \cot\theta + \dfrac{M}{d_v}}{E_s A_s} \tag{3.30}$$

where M = Bending moment at the section concidered

E_s = Modulus of elasticity of steel

A_s = Area of longitudinal tensile steel.

If calculated value of ε_x is greater than the yield strain, ε_y of longitudinal tensile reinforcement then return to step 3 and choose a smaller ε_1 value.

Note: When calculating the yield strain of longitudinal tensile strain modulus of elasticity of steel was assumed to be equal 250 GPa.

$$\varepsilon_y = \frac{f_y}{E_s} \tag{3.31}$$

Step 11: Calculate the principal compressive strain ε_1, using the relationship between strains:

$$\tan^2\theta = \frac{\varepsilon_x - \varepsilon_2}{\varepsilon_1 - \varepsilon_x} \tag{3.32}$$

Check weather the ε_1 value obtained from this equation matches with the estimated value of ε_1 in the Step 2. If not return to step 2 and use this ε_1 value as the new initial estimation and continue with the procedure.

Step 12: Calculate the strain in stirrup steel ε_t, using the relationship between strains:

$$\varepsilon_t = \varepsilon_1 + \varepsilon_2 - \varepsilon_x$$

and calculate the stress in stirrup f_s:

$$f_s = E_v\,\varepsilon_t$$

here the modulus of elasticity of the steel is assumed to be equal to 250GPa.

Check weather the f_s value obtained from this equation matches with the estimated f_s value in the Step 3. If not return to step 3 and use this f_s value as the new initial estimation and continue the procedure.

If calculated value of f_s exceeds the yield stress of stirrups then return to step 2 and increase the θ.

Step 13: Calculate the shear stress transfer across the crack v_{ci} using:

$$v_{ci} = \frac{0.18\sqrt{f_c'}}{0.3 + \dfrac{24\varepsilon_1 s_{m\theta}}{a + 16}} \qquad (3.33)$$

Step 14: check whether $f_1 \leq v_{ci} \tan\theta$

If calculated value of f_1 satisfies above condition, precede next steps else return to step 2 and choose a higher value for θ.

Step 15: Predicted shear strength of the beam V_r;

$$V_r = f_1 b_w d_v \cot\theta + \frac{A_v f_s d_v}{s}\cot\theta \qquad (3.35)$$

Note: for a particular beam shear strength was calculated for crack angle θ, varies from 15^0 to 60^0 and the maximum predicted shear strength was selected.

Other than the design limitation for maximum allowable shear resistance of a section, all the other design limitations given for Simplified method is equally applicable to the General method.

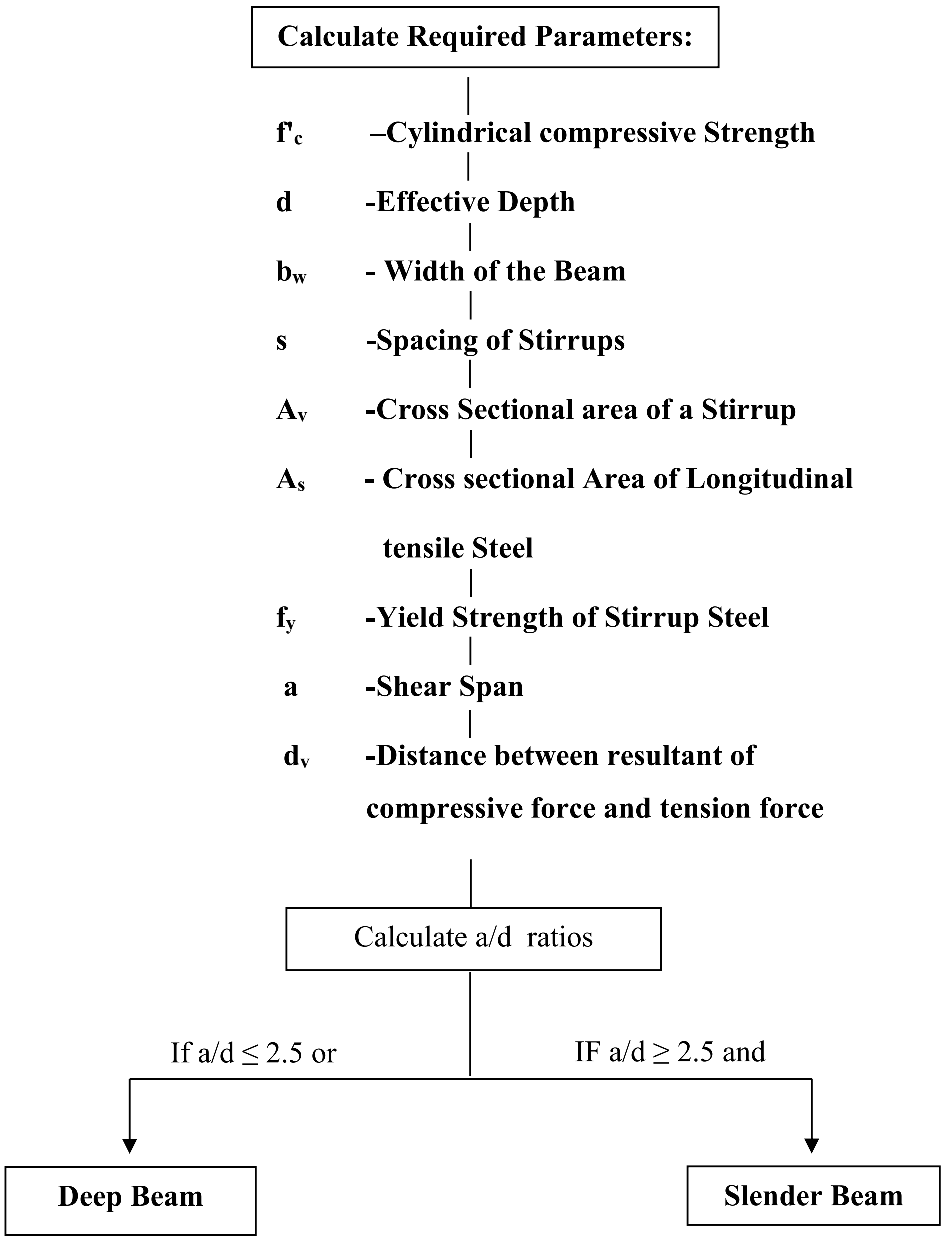
In the General method maximum shear resistance of the section is given as:

$$V_r = 0.25\, f_c' b_w d_v \qquad (3.36)$$

Flow chart for spread sheet calculation:

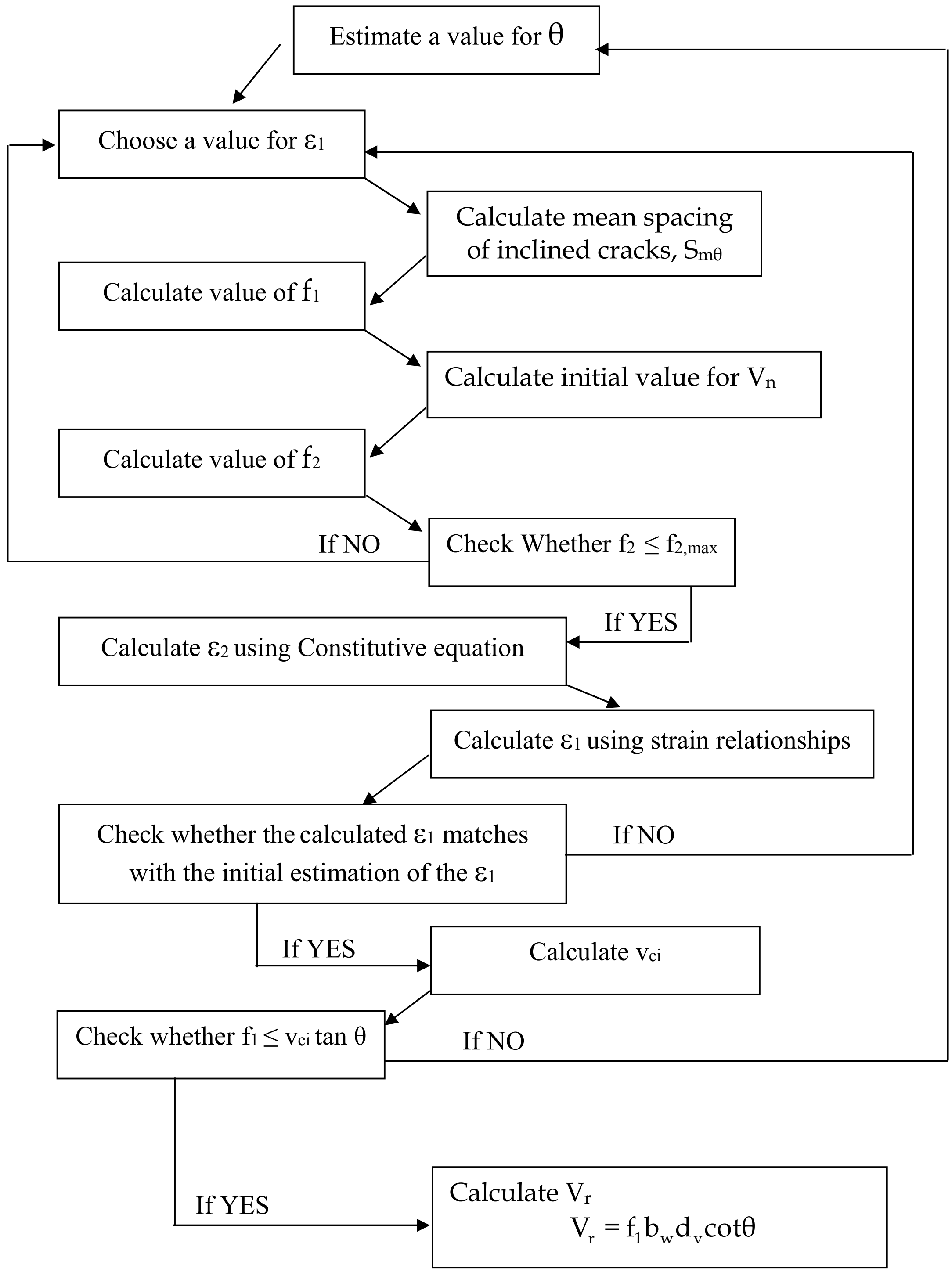
<u>Step 1</u>

<u>Calculate Parameters and identify slender beams</u>

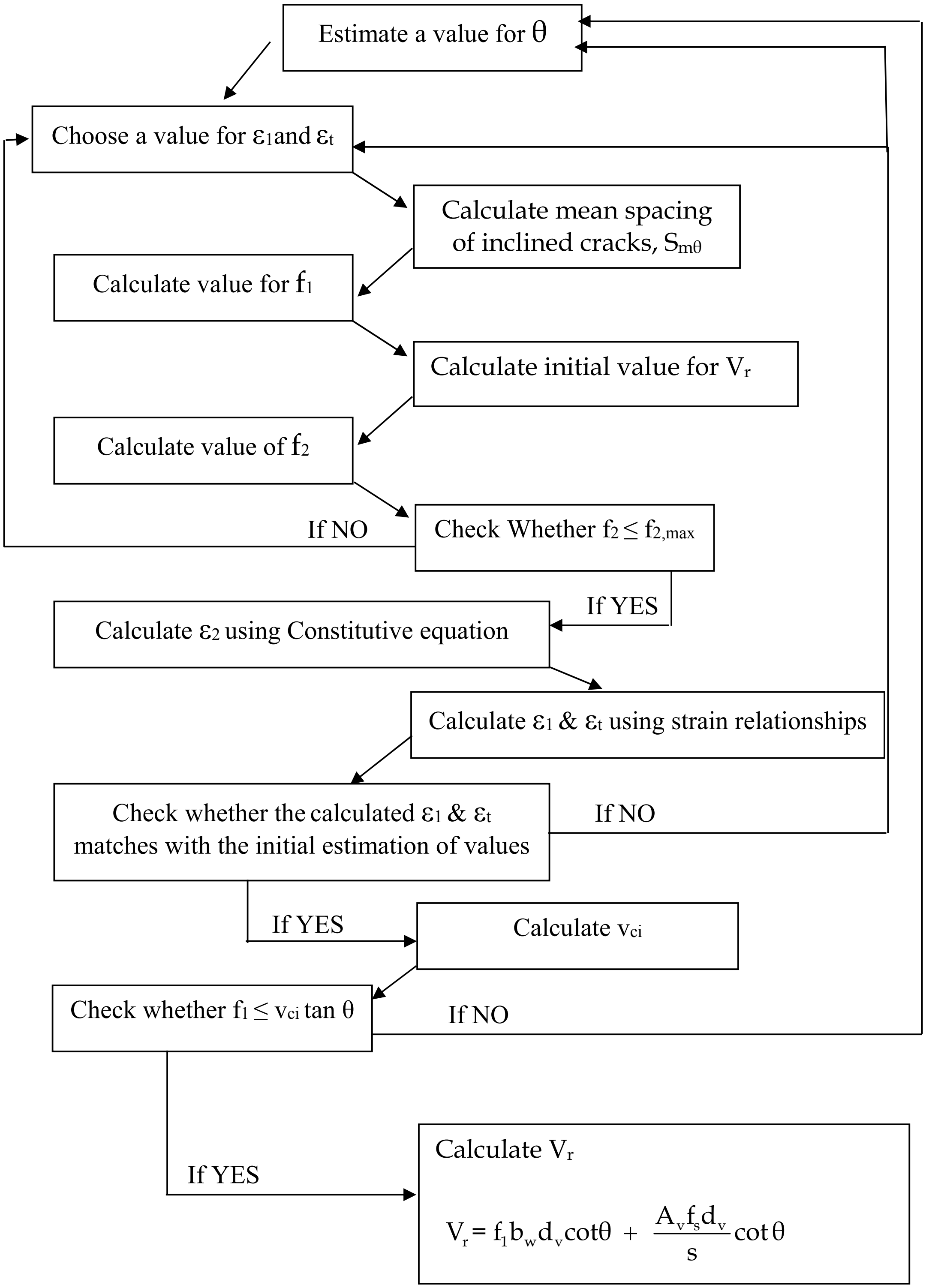


Step 2

Predict shear strength(Vn)of beams without Shear reinforcement



Predict shear strength (Vn) of beams with Shear reinforcement



3.3.6 Shear Friction Method – Loov (1998), El Metwally and Loov (2001)

The basic equation used in this method has been derived for beams with shear reinforcement which was published in 1998:

$$V_n = k\left(C\,T_v\,\frac{d_s}{s}\right)^{1/2} - T_v \quad \le \; 0.25\,C\,k^2\,\frac{d_s}{s} \qquad (3.37)$$

where $k = 0.6$

$C = f_c^{'} b_w h$

$T_v = A_v f_y$

$f_c^{'}$ = Compressive Strength of Concrete (MPa)

d_s = length of stirrup centre to centre bars formig top and bottom of the stirrup (mm).

s = Spacing of Stirrups(mm)

h = Height of the beam

For beams without shear reinforcement Loov and El Metwally (2001) proposed an equation to incorporate shear friction model:

$$V_c = V_{45}\,\frac{h}{a_{cl}} + \frac{M_{cr}}{a_{cl}} \qquad (3.38)$$

where $V_{45} = \beta_v \sqrt{f_c^{'}}\,A_w \quad : \quad \beta_v = 0.36\left(\frac{30}{f_c^{'}}\right)^{0.25}\left(\frac{500}{h}\right)^{0.25}$

M_{cr} = Cracking moment for the beam,according to CSA A23.3-94 clause 8.6.4

$$M_{cr} = \frac{f_r\,I_g}{y_t} \qquad (\text{A23.3 - 1994 Eq.9-2})$$

$f_c^{'}$ = Compressive Strength of Concrete (MPa)

a_{cl} = Clear distance between load and support (mm).

f_r = modulus of rapture = $0.6\sqrt{f_c^{'}}$

I_g = moment of inertia of uncracked beam section

y_t = Distance from centroid to extreme tension fibre

h = Height of the beam

A_w = Cross sectional area assumed to resist shear

As for all other methods, this method is also has design limitations to avoid unsafe designs. Loov has suggested to adopt all design limitations given for Simplified Method in Canadian code (1994) for his Shear Friction Model.

Flow chart for spread sheet calculation:

Step 1

Calculate Parameters and identify slender beams

Calculate Required Parameters:

f'_c - Cylindrical compressive Strength

d_s - Length of stirrup centre to centre bars forming top and bottom of the stirrup

b_w - Width of the Beam

s - Spacing of Stirrups

A_v - Cross Sectional area of a Stirrup

A_w - Cross sectional Area of beam

f_y - Yield Strength of Stirrup Steel

a_{cl} - Clear distance between load and support

y_t - Distance from centroid to extreme tension fibre

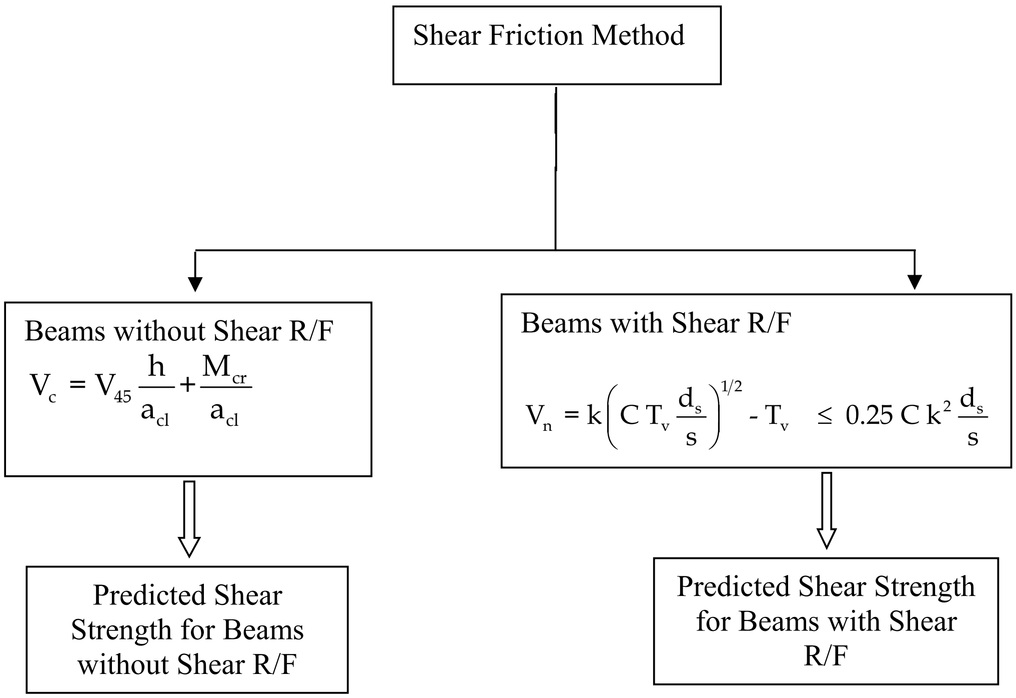
Calculate a/d ratios

If a/d < 2.5 → **Deep Beam**

IF a/d ≥ 2.5 → **Slender Beam**

Step 2

Predict shear strength (V_n)

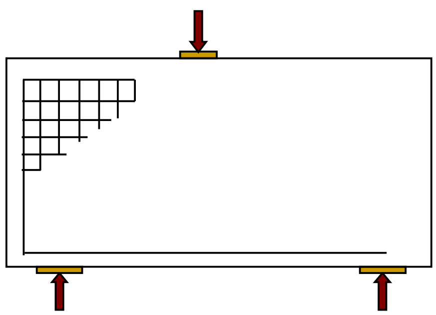


3.4 Shear Strength of Deep Beams

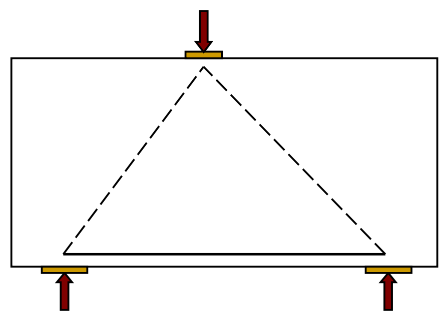
Among the selected design codes of practices and shear design method only US, Canadian, Australian, and Japanese codes have given guidelines for designing of deep beams. Except the Japanese code design procedures of other codes are based on Strut and Tie model. Japanese code uses empirical formula to predict the shear strength of deep beams.

3.4.1 Selection of Strut and Tie Model

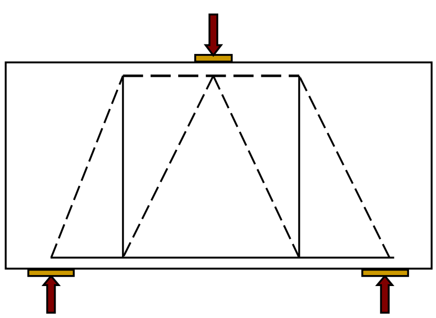
Ideally, it is possible to identify infinite number of suitable strut and tie models for a particular deep beam. Fig 3.3 shows some possible strut and tie models for the beam in Fig.3.3a.



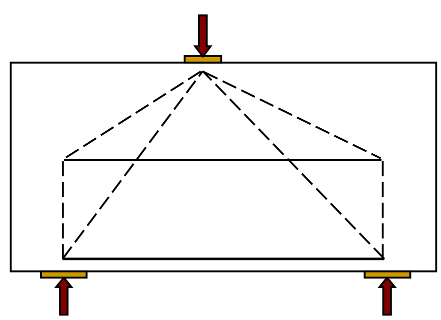
a. Deep Beam



b. Model 1



c. Model 2



d. Model 3

Figure 3.5
Strut and Tie models for a deep beam

All three strut and tie mechanisms shown in Fig3.3 satisfies equilibrium and compatibility relationships. But shear capacities calculated from different mechanisms will be different. Therefore care must be taken, such that the truss model chosen is appropriate for the beam. Based on the recommendations of Schlaich et al. the model that contains the least strain energy is likely to be the most appropriate model.

For this study strut and tie models shown in figure 3.3 b & c was selected as suitable models. For these two models based on the failure load of the specimen, total Strain energy of each model was calculated for each specimen. For each element in the truss model (struts and ties), the strain energy is calculated separately, then they were summed to determine strain energy stored in the entire truss.

The stepwise process of finding the strain energy has been illustrated below.

1) First of all dimensions of nodal zones at supports were determined.

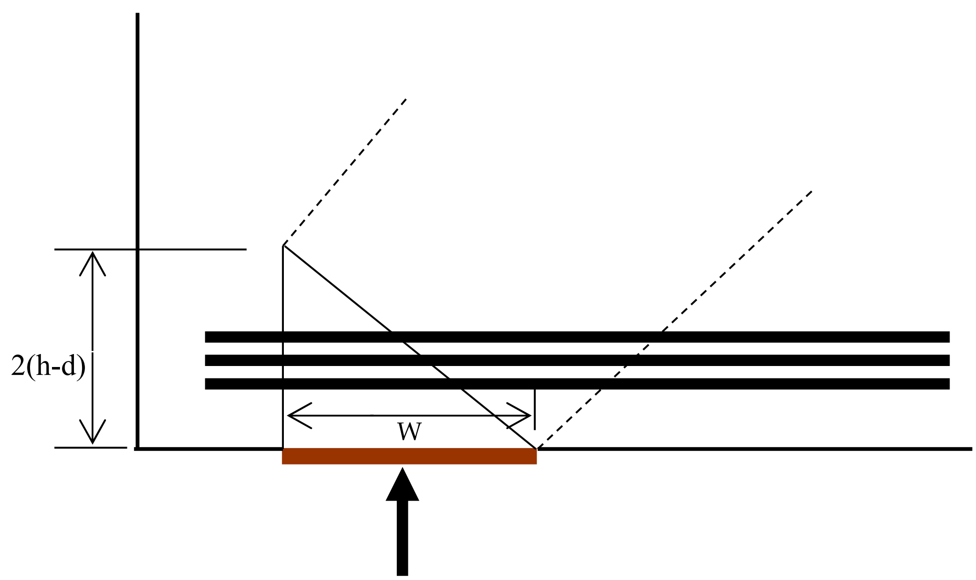


Figure 3.6
Dimensions of nodes at supports

Here h, d and w represent height of the beam, effective depth of the beam and width of the bearing plate respectively.

2) Dimensions of the node under load point and the other places on the top face were calculated using hydrostatic node.

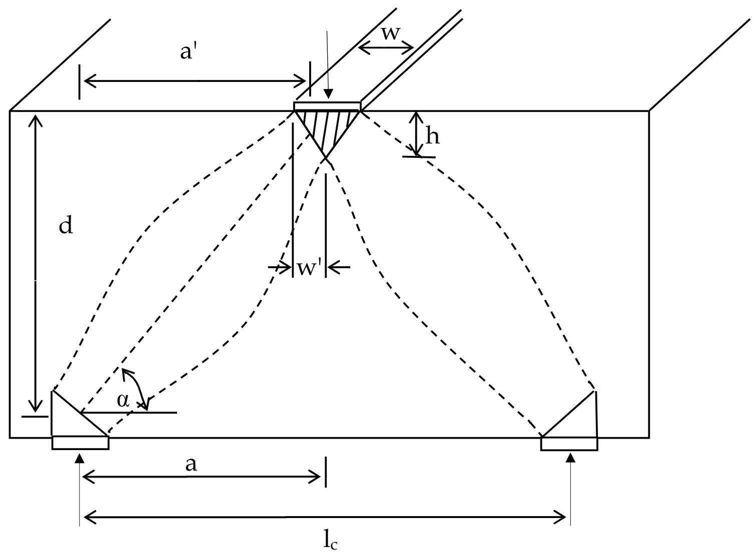


Figure 3.7
Height of the Hydrostatic Node

The height of the hydrostatic node, h can be calculated as

$$h = d - a\tan\alpha \tag{3.39}$$

where $\alpha = \tan^{-1}\left\{\frac{d}{2a'}\left(1 + \sqrt{(1-\frac{2w'a'}{d^2}}\right)\right\}$

$$a' = a + \frac{(lc-a)}{lc}\frac{w}{2} - \frac{w}{2}$$

$$w' = \frac{(lc-a)}{lc}w$$

3) Calculate the strain energy of elements.

Strain energy of a strut P_s:

$$P_s = \frac{1}{2}\frac{E_c F^2}{A_s} \tag{3.40}$$

Strain energy of a Tie P_t :

$$P_t = \frac{1}{2}\frac{E_t F^2}{A_t} \tag{3.41}$$

where E_c, E_t = Modulus of elasticity of Concrete and Reinforcement respectively
A_s, A_t = Cros sectional area of the strut and Tie respectively
F = Force in the element.

Due to the small volume, strain energy contain in the nodal zone is small compare to the other elements. Therefore when calculating the strain energy nodal zones were neglected.

Obviously for beams without shear reinforcements the strut and tie model shown in figure 3.3c is not suitable as there is no steel to form vertical ties. Therefore, for beams without shear reinforcement, shear capacity was calculated using model shown in Fig.3.3b. For beams with shear reinforcement both models in Fig.3.3b & c were considered.

4) Total strain energy of each strut and tie model was calculated for each and every beam. Then the model that contains the least strain energy was taken for the evaluation.

3.4.2 ACI Code –Shear Design Provisions for Deep Beams (ACI- 2002)

The capacity of strut and tie model is depends on three factors: Strength of Nodal zones; Strength of Struts; Strength of Tie. Each of these elements have been illustrated in Fig 3.4.

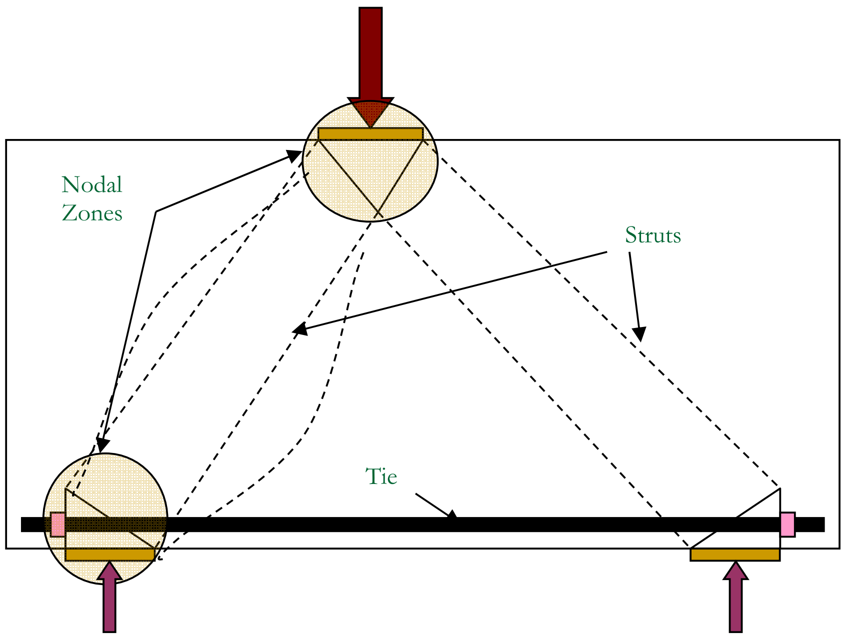


Figure 3.8
Elements of a Deep Beam

In the ACI Code, Strength of each element is defined separately.

Strength of Nodal Zones

Nodes are classifies according to the sign of forces meeting at that node. ACI Code identifies four types of nodes. Namely they are: CCC Node:, CCT Node:, CTT Node: and TTT Node. A CCC node resists three compressive forces; a CCT node resists two compressive forces and one tensile force and so on.

Strength of nodal zones, F_{nn}, is given as:

$$F_{nn} = f_{ce} A_{nz} \tag{3.42}$$

where A_{nz} = The area of the face of the nodal zone on which the force in strut or tie acts, taken perpendicular to the force.

f_{ce} = Effective compressive strength of concrete in nodal zones

$f_{ce} = 0.85\beta_n f'_c$

β_n = 1.0 for CCC nodes
= 0.80 for CCT nodes
= 0.60 for CTT and TTT nodes

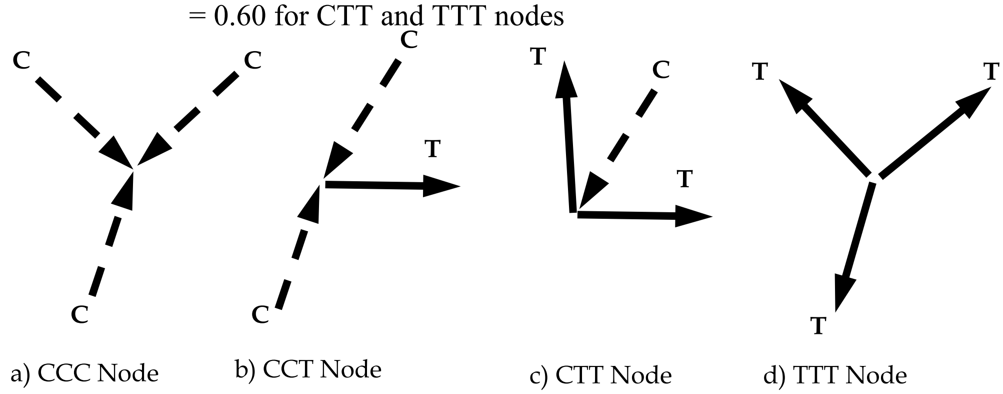


Figure 3.9
Classification of Nodes

Strength of Strut

The strength of strut F_{ns} is defined as:

$$F_{ns} = f_{ce}\, A_{cs} \tag{3.43}$$

where A_{ce} = smallest cross sectional area of the strut.

f_{ce} = taken as the smaller of a and b

a) Effective compressive strength of concrete in nodal zones

b) Effective compressive strength in nodal zones:

$f_{ce} = 0.85\beta_s f'_c$

βs = 1.0 for struts with uniform cross sections

0.75 for bottle shaped struts with crack control reinforcement.

0.60 for bottle shaped struts without crack control reinforcement.

Strength of Tie

The Strength of tie, F_{nt}, is defined as:

$$F_{nt} = A_{ts}\, f_y \tag{3.44}$$

where A_{ts} = Cross sectional area of Longitudinal tensile reinforcement.

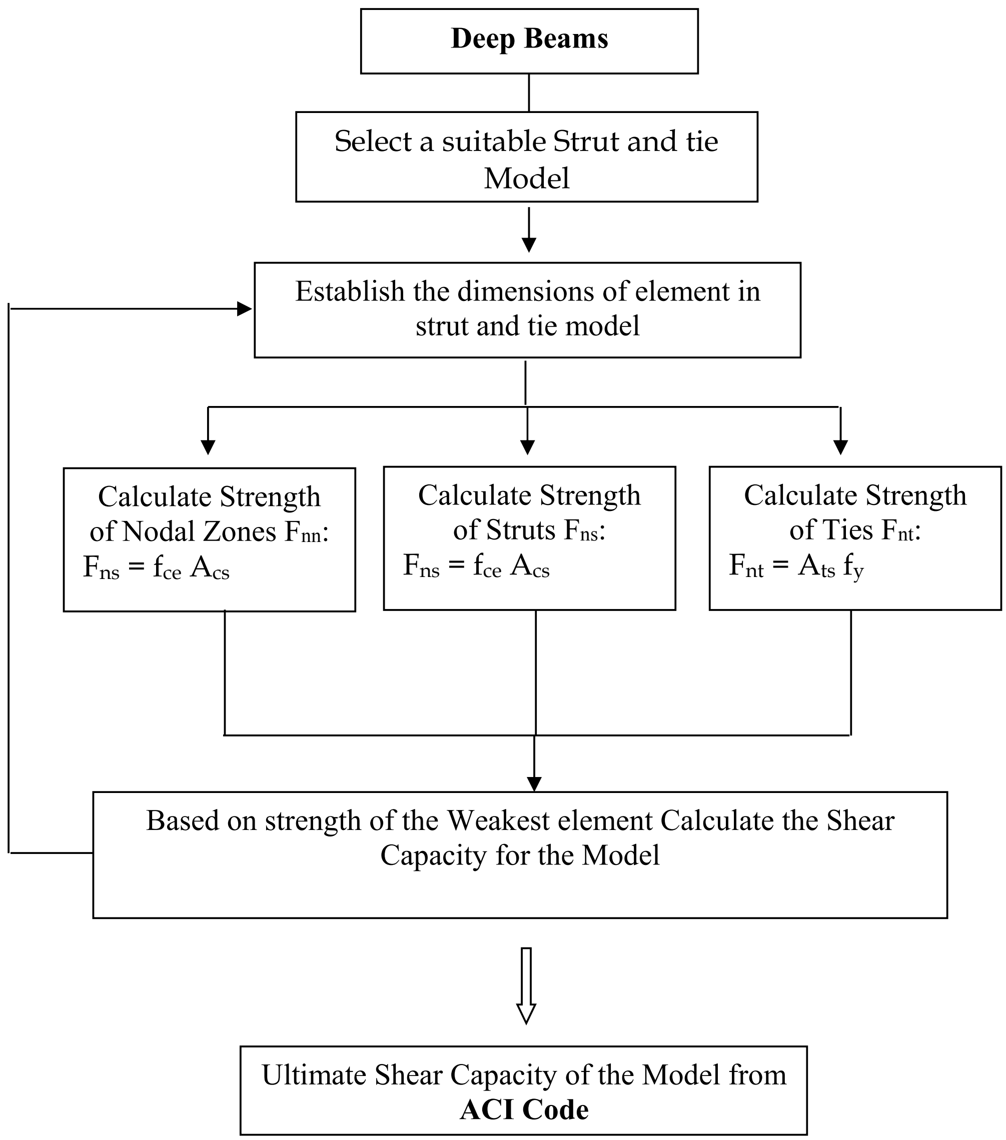
f_y = Yield strength of reinforcement.

Ultimate shear capacity of the beam is governed by the strength of the weakest element. The ultimate shear is calculated based on the strength of weakest element.

Initially the shear capacity is calculated using the strut and tie model which was used for calculating the strain energy. Then the strut angle α is increased and shear capacity of the modified strut and tie model is calculated. This process is carried out to find the optimum dimensions for the strut and tie model which gives the maximum shear capacity of the beam.

Flow chart for spread sheet calculation:

Predict shear strength (V_n) of beams without Shear reinforcement



3.4.3 Australian Code – Shear Design Provisions for Deep Beams (AS3600 - 2001)

Australian Code Method is also based on the strut and tie model. Therefore calculation procedure is similar to the ACI Code method. Calculation procedure is started with the same model used to determine the strain capacity. In Australian code the strength of nodal zones is given as:

$$F_{nn} = f_{c.cal}\, A_{nz} \qquad (3.45)$$

where A_{nz} = The area of the face of the nodal zone on which the force in strut or tie acts, taken perpendicular to the force.

$f_{c.cal} = (0.8 - f'_c/200)\, f'_c$

The strength of a strut is given as:

$$F_{ns} = f_{c.cal}\, b_c\, d_c \qquad (3.46)$$

where $f_{c.cal} = (0.8 - f'_c/200)\, f'_c$

b_c = The width of the compression strut

d_c = The depth of the compression strut

Finally the strength of a tie is given as:

$$F_{nt} = A_{ts}\, f_{y\ v} \qquad (3.47)$$

where A_{ts} = Cross sectional area of Longitudinal tensile reinforcement.

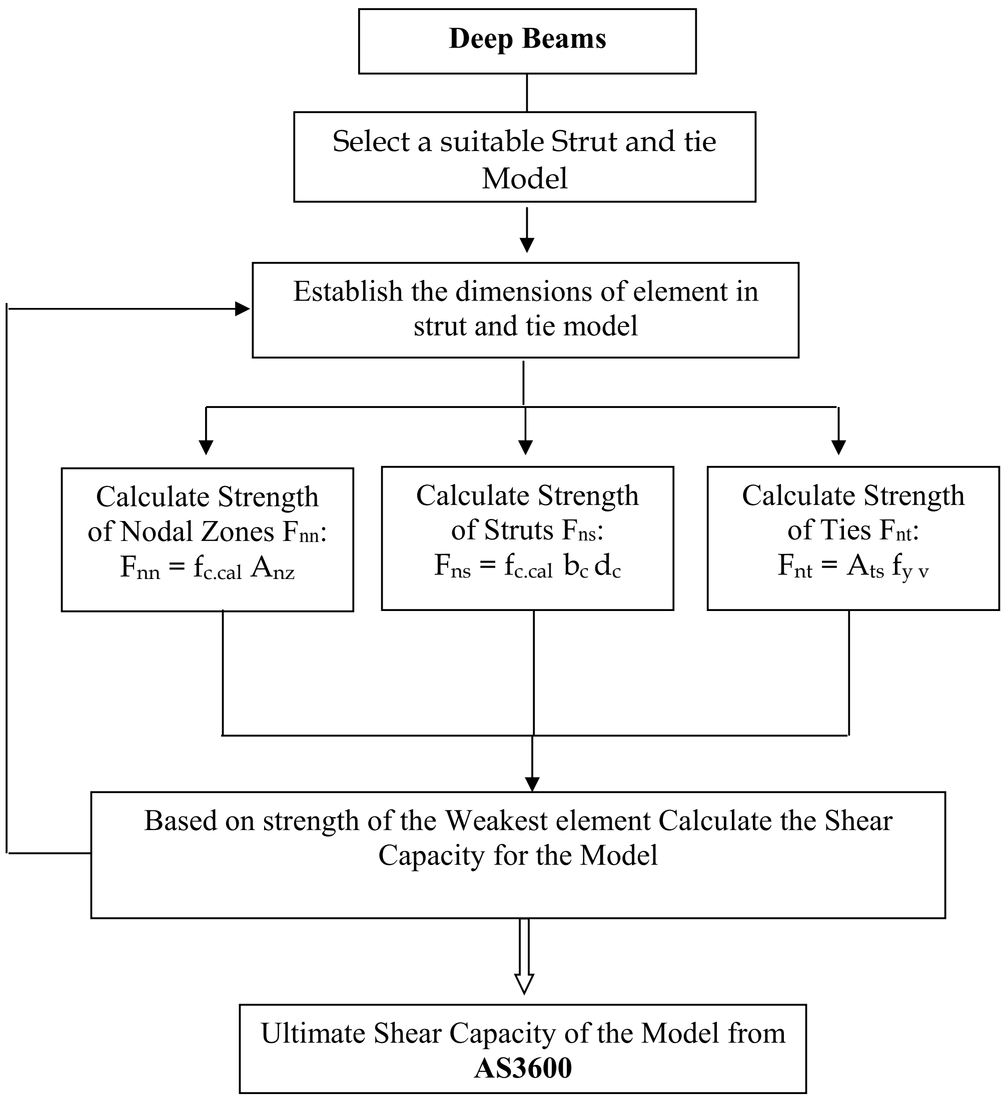
f_y = Yield strength of reinforcement.

Final shear capacity of the beam is calculated based on the strength of the weakest element.

As in the ACI Code, different dimensions for the strut and tie model is selected by increasing the initial strut angle α used for the strain energy calculation. Then the model with highest shear capacity has been taken to the study.

Flow chart for spread sheet calculation:

Predict shear strength (V_n) of beams without Shear reinforcement



3.4.4 Canadian Code – Shear Design Provisions for Deep Beams (CSA A23.3 - 1994)

Shear design procedure of the Canadian Code is also based on the Strut and Tie model. Therefore a suitable strut and tie model is selected based on the minimum strain energy. Dimensions of this model are initially selected to start the calculation procedure.

Similar to the previous two methods, Canadian code is also gives the strengths of the elements of the strut and tie model separately. Then the ultimate shear capacity of the beam is calculated based on the strength of the weakest element.

Strength of Nodal Zones

The strength of a Node, F_{nn}, is defined as:

$$F_{nn} = f_{ce} A_{nz} \tag{3.48}$$

where Anz = The area of the face of the nodal zone on which the force in strut or tie acts, taken perpendicular to the force.

f_{ce} = Effective compressive strength of concrete in nodal zones

$f_{ce} = \beta f'_c$ β = 0.85 for CCC nodes
= 0.75 for CCT nodes
= 0.65 for CTT and TTT nodes

Strength of Struts

The strength of a strut, F_{ns}, is defined as:

$$F_{ns} = f_{cu} A_{cs} \tag{3.49}$$

where A_{ce} = smallest cross sectional area of the strut.

$$f_{cu} = \frac{f_c^{'}}{0.8+170\varepsilon_1}$$

$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002) \cot^2\alpha_s$

ε_s = The strain in the reinforcement crossing the strut

α_s = The angle between the reinforcement and the axis of the strut

Strength of Ties

The Strength of a tie, F_{nt}, is defined as:

$$F_{nt} = A_{ts} f_y \tag{3.50}$$

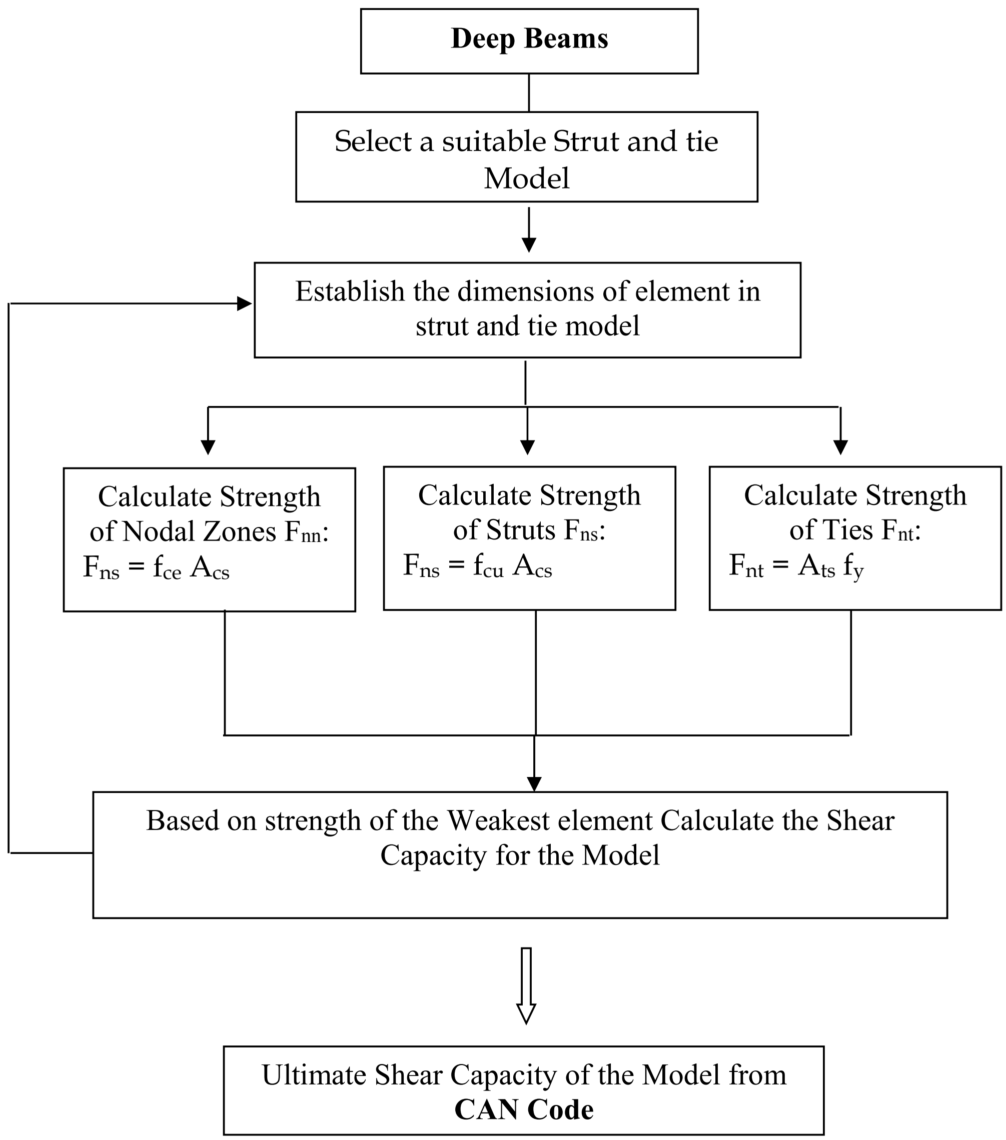
Where A_{ts} = Cross sectional area of Longitudinal tensile reinforcement.

f_y = Yield strength of reinforcement.

In order to find the maximum load that could be carried by the beam the dimensions of the elements were changed and the calculation was repeated. This is done by increasing the strut angle α from the initial value.

Flow chart for spread sheet calculation:

Predict shear strength (Vn) of beams without Shear reinforcement



3.4.5 Japanese Code – Shear Design Provisions for Deep Beams (JSCE SP-1 - 1986)

In contrast to previously described methods the Japanese Code uses a single empirical formula to predict the shear strength, V_{dd}, of deep beams.

$$V_{dd} = f_{dd} b_w d \tag{3.51}$$

where b_w = Width of the Beam (cm)

d = Effective Depth (cm)

$$f_{dd} = 0.6\beta_d \beta_p \beta_a \sqrt{f'_{cd}} \quad (kgf/cm^2)$$

$$\beta_d = \sqrt[4]{100/d} \le 1.5 \quad (d : cm)$$

$$\beta_p = \sqrt[3]{100\rho_w} \le 1.5 \;:\; \rho_w = (As/b_w d)$$

$$\beta_a = \frac{5}{\left(1 + (a_v/d)^2\right)}$$

a_v = Distance between face of the support and the load point

Flow diagram for spread sheet calculation:

Predict shear strength (V_{dd}) of deep beams

Deep Beams

Calculate parameters

f_{cd} - Cubical compressive Strength

d - Effective Depth

b_w - Width of the Beam

A_s - Cross sectional Area of Longitudinal tensile Steel

A_v - Distance between the face of the support and the load point

Calculate Shear Capacity V_{dd}:

$$V_{dd} = f_{dd} b_w d$$

Ultimate Shear Capacity of the Beam from **JSCE SP-1**

3.5 Application of Multinomial Logistic Regression

3.5.1 Introduction

When evaluating a shear design process it is essential to figure out critical parameters which critically influence design process and their effect on the accuracy of that process. Ideal way of doing this would be carrying out a laboratory experiment to find out the influence of a particular parameter at time while keeping all the other parameters constant. But this would be difficult when there are large numbers of parameters to be checked for the influence. A good possible solution to overcome this problem may be to use a suitable statistical model. Therefore a Multinomial Logistic Regression is used in this study to find the critical parameters influencing each design method and their effect.

3.5.2 Multinomial Logistic Regression

<u>The Logit and Logistic Transformations</u>

In multiple regression, a mathematical model of a set of explanatory variables is used to predict the mean of the dependent variable. In logistic regression, a mathematical model of a set of explanatory variables is used to predict a transformation of the dependent variable. This is the *logit* transformation.

Suppose the numerical values of 0 and 1 are assigned to the two categories of a binary variable. Often, the 0 represents a negative response and the 1 represents a positive response. The mean of this variable will be the proportion of positive responses. Because of this, one might try to model the relationship between the probability (proportion) of a positive response and the explanatory variables. If p is the proportion of observations with a response of 1, then 1-p is the probability of a response of 0. The ratio $p/(1\text{-}p)$ is call the *odds* and the *logit* is the logarithm of the odds, or just *log odds*. Mathematically, the logit transformation is written

$$l = \text{logit(p)} = \ln\left(\frac{\text{p}}{1 - \text{p}}\right) \tag{3.52}$$

The *logistic* transformation is the inverse of the logit transformation. It is written as:

$$\text{p} = \text{logistic}(l) = \ln\left(\frac{\text{e}^{l}}{1 + \text{e}^{l}}\right) \tag{3.53}$$

<u>The Logistic Regression and Logit Models</u>

In multiple-group logistic regression, a discrete dependent variable Y having G unique values ($G \geq 2$) is regressed on a set of p independent variables $X_1,X_2,\ldots X_p$. Y represents a way of partitioning the population of interest. For example, Y may be presence or absence of a disease, condition after surgery, or marital status. Since the names of these partitions are arbitrary and are referred to them by consecutive numbers. That is, in the discussion below, Y will take on the values 1, 2, …,G.

In the discussion to follow, let

$$\text{X} = \left(X_1, X_2, \ldots\ldots X_p\right)$$

$$B_p = \begin{pmatrix} \beta_{g1} \\ \vdots \\ \beta_{gp} \end{pmatrix}$$

The logistic regression model is given by the G equations

$$\begin{aligned} \ln\left(\frac{P_g}{P_1}\right) &= \ln\left(\frac{P_g}{P_1}\right) + \beta_{g1}X_1 + \beta_{g2}X_2 + \ldots\ldots + \beta_{gp}X_p \\ &= \ln\left(\frac{P_g}{P_1}\right) + XB_g \end{aligned} \tag{3.54}$$

Here, p_g is the probability that an individual with values , $X_1,X_2,\ldots X_p$ is in group g. That is,

$$p_g \; = \; \Pr\ \left(Y = g \mid X\right) \tag{3.55}$$

Usually $X_1 \equiv 1$ (that is, an intercept is included), but this is not necessary. The quantities $P_1,P_2,\ldots.P_G$ represent the prior probabilities of group membership. If these prior probabilities are assumed equal, then the term (P_g/P_1) becomes zero and drops out. If the priors are not assumed equal, they change the values of the intercepts in the logistic regression equation.

Group one is called the *reference group*. The regression coefficients $\beta_{11},\beta_{12},\ldots,\beta_{1p}$
for the reference group are set to zero. The choice of the reference group is arbitrary. Usually, it is the largest group or a control group to which the other groups are to be compared. This leaves *G*-1 logistic regression equations in the multinomial logistic model.

The β's are population regression coefficients that are to be estimated from the data. Their estimates are represented by *b*'s. The $\beta' s$ represents the unknown parameters, while the *b*'s are their estimates.

These equations are linear in the logits of *p*. However, in terms of the probabilities, they are nonlinear. The corresponding nonlinear equations are

$$p_g = \Pr\text{ob}\left(Y = g \mid X\right) = \frac{e^{XB_g}}{1 + e^{XB_2} + e^{XB_3} + \ldots\ldots e^{XB_G}} \qquad (3.56)$$

since $e^{XB_1} = 1$ because all of its regression coefficients are zero.

Estimation of Regression Coefficients

In logistic regression, regression coefficients are estimated using maximum likelihood. Roughly the idea behind the maximum likelihood is as follows. Consider the probability of a particular beam with set of parameters $X_1, X_2, \ldots X_p$ falling in to the group p is equal to P_g, Similarly P_1 can be interpret as the probability of that particular beam falling in to the group 1, logistic regression model was developed to the log value of the ratio P_g/P_1 ,

i.e. $\ln(\frac{P_g}{P_1})$

Maximum likelihood finds those coefficients $\beta_{g1},\beta_{g2},\ldots,\beta_{gp}$ which makes the value of $\ln(\frac{P_g}{P_1})$ as large as possible. The process of estimating regression coefficients is given bellow.

Consider N number of beams with each beam having p number of parameters, $X_1,X_2,\ldots X_p$ Then the probability of j[th] beam fall in to group g can be written as,

$$p_g = \Pr\left(Y = g \mid X_j\right) \tag{3.57}$$

To improve the notation, let

$$\begin{aligned} \pi_{gj} &= \Pr\left(Y = g \mid X_j\right) \\ &= \frac{e^{X_j B_g}}{1 + e^{X_j B_2} + e^{X_j B_3} + \ldots.. e^{X_j B_G}} \\ &= \frac{e^{X_j B_g}}{\sum_{s=1}^{N} e^{X_j B s}} \end{aligned} \tag{3.58}$$

The likelihood of sample of N observations is given by

$$l = \prod_{s=1}^{N} \prod_{g=1}^{G} \pi_{gj}^{\;y_{gj}}$$

where y_{gj} is one if the j^{th} beam is in the group g and zero otherwise

Using the fact that $\sum_{g=1}^{G} y_{gj} = 1$, the log likelihood, L, is given by

$$L = \ln(l) = \sum_{j=1}^{N} \sum_{g=1}^{G} y_{gj} \ln(\pi_{gj}) \tag{3.59}$$

Maximum likelihood estimates of the $\beta' s$ are found by finding those values that maximize this log likelihood equation. This is accomplished by calculating the partial derivatives and setting them to zero.

Interpretation of Regression Coefficients

The interpretation of the estimated regression coefficients is not as easy as in multiple regression. In multinomial logistic regression, not only is the relationship between *X* and *Y* nonlinear, but also, if the dependent variable has more than two unique values, there are several regression equations.

Consider the previous example, taking group 1 as the reference group, the regression equation was written as

$$\ln(\frac{P_g}{P_1}) = \ln(\frac{P_g}{P_1}) + \beta_1 X_1 + \beta_2 X_2 + ... + \beta_p X_p$$

Now consider impact of a unit increase in parameter X_1 while keeping all other coefficients constant. The logistic regression equation becomes

$$\ln(\frac{P_g^{'}}{P_1}) = \ln(\frac{P_g}{P_1}) + \beta_1 (X_1 + 1) + \beta_2 X_2 + ... + \beta_p X_p$$

Taking the difference between two equations β_1 can be written as

$$\beta_1 = \ln(\frac{P_g^{'}}{P_1^{'}}) - \ln(\frac{P_g}{P_1})$$

$$\beta_1 = \ln\left(\frac{\frac{P_g^{'}}{P_1^{'}}}{\frac{P_g}{P_1}}\right)$$

$$\beta_1 = \ln\left(\frac{Odds^{'}}{Odds}\right)$$

That is, β_1 is the log of the ratio of the odds for a unit increase in X_1 while keeping other variables constant. Removing the logarithm it can be written as:

$$e^{\beta_1} = \frac{Odds^{'}}{Odds}$$

The regression coefficient β_1 is interpreted as the log of the odds ratio comparing the odds after a one unit increase in X_1 to the original odds.

Note that, unlike multiple regression, the interpretation of β_1 depends on the particular set of *X* value of since the probability values, the *p*'s, will vary for different set of *X* values.

3.5.3 Application of Multinomial Logistic Regression

Our calculated results have to rearrange in a systematic manner in order it to use it for multinomial logistic regression. Therefore, first of all, the shear strength predictions were categorized into three different categories depending on their V_t/V_n ratio. Where V_t is the ultimate shear capacity observed in the beam testing and V_n is the predicted shear capacity from a particular method.

Table 3.1
Categories of Shear Strength Predictions

Category	Range	
C (Over Predictions)	$\mathbf{V_t/V_n < 0.9}$	Code has over Estimated the Shear Strength
B * (Accurate Predictions)	$\mathbf{0.9 \leq V_t/V_n \leq 1.1}$	Code has Accurately Estimate the Sear Strength
A (Under Predictions)	$\mathbf{V_t/V_n > 1.1}$	Code has under Estimated the Shear Strength

* Category B was selected as reference category

Selection of Parameters

From literature survey it was found there were about thirteen parameters which might influence the shear capacity of a reinforced concrete beam:

1) Concrete Grade - $\mathbf{f_c'}$
2) Effective Depth – **d**
3) Width of the beam - **b**
4) Percentage area of Longitudinal Tensile Steel for Shear – $\boldsymbol{\rho}$
5) Percentage area of transverse steel - $\boldsymbol{\rho_s}$
6) Spacing ratio of transverse reinforcement - **s/d**
7) Yield Strength of Long. Tensile Steel – $\mathbf{f_y}$
8) Anchorage of longitudinal Tensile Steel
9) Proximity of rigid support to point load – **a /d or α**
10) Cross Sectional Shape
11) Influence of Crack Control R/F
12) Maximum Aggregate Size - $\mathbf{a_g}$
13) Yield strength of transverse steel - $\mathbf{f_{vy}}$

These parameters were selected to for this study as explanatory variables in the Multinomial Logistic Regression analysis. Then the Multinomial Logistic Regression was carried out using SPSS software to study the influence of each parameter on the accuracy of the each design procedure.

3.6 Industrial survey

Parallel to this study a questionnaire survey was also carried out with several intentions: 1) Identify methods used for shear designs in the industry 2) Find out how comfortable they are with shear design method they use 3) Find out how often they use research information : 4) Find out their opinion of practicing designers on shear designs. A questionnaire was prepared with eighteen questions. Ninety eight designers were interviewed and their opinions were obtained. The model questionnaire paper is given in the next two pages.

Evaluation of Shear Design Procedures Adopted in Industry for Reinforced Concrete Design

QUESTIONNAIRE

1	What are the Shear Design methods you usually use? ☐ BS Code Method ☐ Australian Code Method ☐ Japanese Code Method ☐ Euro Code Method ☐ ACI Code Method ☐ Canadian Code Method ☐ Other ________________________________
2	How comfortable are you in using usual code equations, tables etc? ☐ Easy process ☐ Moderate process ☐ Tedious process
3	What is the most convenient code you use in the design of reinforced concrete beams for shear? ________________________________
4	How often do you come across situations in which shear design is governed by the limits imposed by the codes? ☐ Frequently ☐ Rarely ☐ Never
5	How often do you feel design guidelines are over conservative? ☐ Frequently ☐ Rarely ☐ Never
6	How often do you feel design guidelines are not adequate? ☐ Frequently ☐ Rarely ☐ Never
7	How have you overcome these situations of lack of sufficient guidelines? ☐ Go for Expertise ☐ Refer Hand Books ☐ Refer Research Papers ☐ Any other method ________________________________
8	How often do you use research information in design office practice? ☐ Frequently ☐ Rarely ☐ Never
9	Do current codes of practice provide sufficient guidelines to understand the shear behaviour as opposed to design? ☐ Yes ☐ To a certain Extent ☐ No
10	What are the structural elements you often design which require special attention on shear designing? ☐ Beams ☐ Slabs ☐ Deep Beams ☐ Footings ☐ Walls ☐ Columns Any other elements ________________________________

Figure 3.10 a
Questionnaire Page 1

Evaluation of Shear Design Procedures Adopted in Industry for Reinforced Concrete Design

QUESTIONNAIRE

11	If you use several Codes for shear designing what is the basis for selecting a particular design method for shear calculations? ☐ Based on the Element ☐ Based on the Parameters ☐ Use only one method ☐ Any other Basis Please give a brief explanation about your basis ________________
12	In your opinion what are the parameters that would influence shear design? ☐ Concrete Grade ☐ Depth of the Beam ☐ Amount of Flexural Tensile Steel ☐ Spacing of stirrups ☐ Area of Shear steel ☐ Width of the Beam ☐ Aggregate size ☐ Yield Strength of Flexural Tensile Steel Any other parameters ________________
13	Out of the following list have you heard a procedure outside the code? ☐ Truss model Compression Field Theory ☐ Shear Friction Modified Compression Field Theory ☐ None
14	Is shear design a critical problem in structural design practice for members without shear links? ☐ Yes ☐ No
15	Is shear design a critical problem in structural design practice for members with shear links? ☐ Yes ☐ No
16	Are we designing for shear or avoiding it by elimination of shear failures prior to flexural failure? ☐ Designing for Shear ☐ Avoiding Shear Failures
17	Have you experienced any shear failure in your career? ☐ Yes ☐ No
18	How often would you end up with problems related to shear being unresolved? ☐ Frequently ☐ Rarely ☐ Never

Figure 3.10 b
Questionnaire Page 2

4. Results and Discussion

4.1 Introduction

The main objective of this study is to critically review the shear design procedures used in the industry. Therefore accuracy of each shear design method and consequently the safeness of design method are assessed. Also critical parameters influencing the accuracy of the methods are identified and their influence on the accuracy of the design methods is discussed in this chapter.

Predicted shear strength was calculated using Microsoft Excel worksheet as illustrated in flow charts in Chapter 3. First of all, the set of 39 input values (Given in Chapter 3.2) describing the beam was entered into the Excel sheet. Then code equation was fed into the Excel sheet and predicted shear strength was calculated. It should be mentioned that all the safety factors including material as well as design were neglected during this calculation. Other than the design limitation on maximum shear carrying capacity of a section all the other limitations were neglected. As anchorage of longitudinal tensile reinforcement is very important on the shear strength, it was checked whether the tensile reinforcement is properly anchored or not. A sample of the final spread sheet is given in Appendix A.

Design procedures for slender beams and deep beams are assessed separately. Slender beams are further divided into two categories: beams without shear reinforcement and: beams with shear reinforcement. They will be discussed separately. Thirteen numbers of parameters were selected as explanatory variables in the Multinomial Logistic Regression analysis and their effect on the accuracy of shear design procedure has been studied. Description of selected parameters is also given in Appendix A.

After identifying the parameters affecting the accuracy of a particular design method, an attempt is made to estimate when they become critical on the accuracy of the method.

4.2 ACI Code

Among the test beams in our database 539 beams were classified as slender beams according to the ACI code recommendations. Out of 539 slender beams, 396 beams were found not to contain shear reinforcement. And 143 slender beams were found to contain shear reinforcement.

4.2.1 Slender Beams without Shear Reinforcement

In the ACI Code two separate equations have been given to calculate shear strength of a slender beam: Equation 11-3 and: Equation 11-5. The equation 11-3 is discussed first here.

***Equation* 11-3 (ACI Code - 2002)**

$$V_c = 2\sqrt{f_c^{'}}\,b_w d \qquad (4.1)$$

Figure 4.1 compares the Predicted shear strength (V_n) vs the Tested shear capacity(V_t). Straight line with unit gradient is the perfect fit line. The points above this line indicate safe predictions whereas points bellow indicate unsafe predictions. From this Figure it can be seen about 78% predictions of this equation has fallen into safe category.

Figure 4.2 has been plotted using the V_t/V_n ratio vs the Tested shear capacity. The horizontal line V_t/V_n = 1 is the perfect fit line. This graph gives a better idea about the deviation of predicted shear strength from tested value. This graph clearly shows that there are cases in which the predicted shear strength is more than twice the actual strength. Also there are cases where the predicted shear strength is less than half the actual value. This type of predictions may lead to very uneconomical designs or on the other hand very unsafe designs. Table 4.1 gives some descriptive statistics of V_t/V_n ratio.

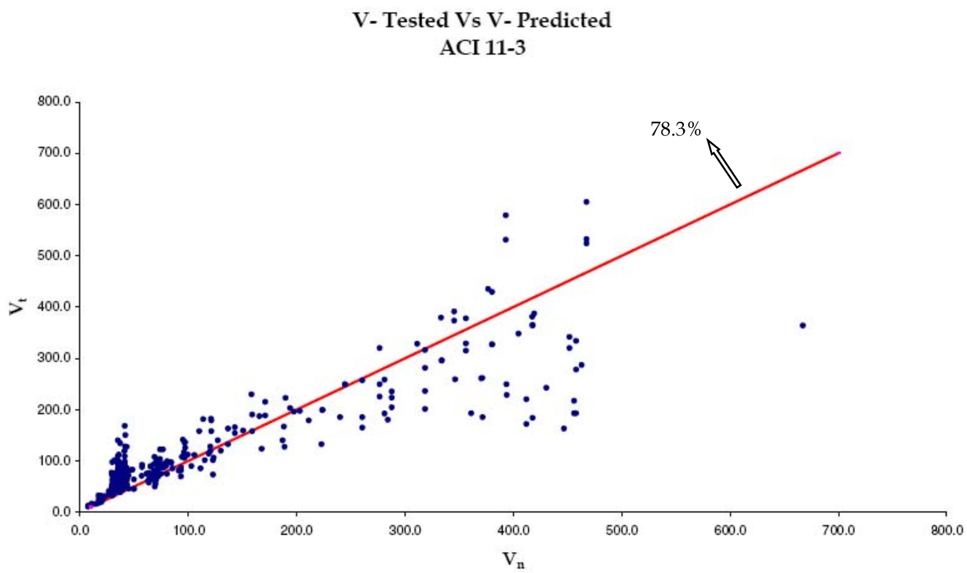


Figure 4.1
Tested Shear Strength Vt Vs Predicted Shear Strength Vn - ACI Equation 11-3
Beams without Shear Reinforcement

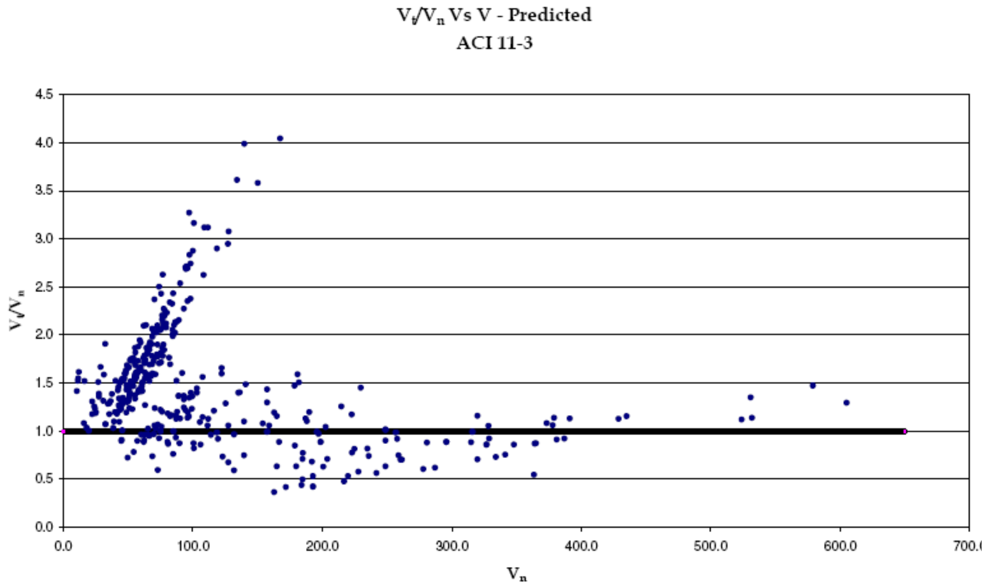


Figure 4.2
V_t/V_n Vs V_t Tested Shear Strength - ACI Equation 11-3
Beams without Shear Reinforcement

Table 4.1
Descriptive statistics of V_t/V_n –ACI-Equation11-3
Beams without Shear Reinforcement

	Vt/Vn
Mean	1.45
Median	1.38
Standard Deviation	0.58
Range	3.7
Minimum	0.36
Maximum	4.05

In order to find the critical parameters and their influence on the accuracy of the predicted shear strength a multinomial logistic regression was carried out using SPSS software. Ten parameters were selected as explanatory variables for study their influence on slender beams without shear reinforcement. Selected variables are given bellow:

Continuous Variables

1) Concrete Grade - **fc' -**

2) Effective Depth – **d** (dm)

3) Width of the Beam – **b** (dm)

4) % of Longitudinal Tensile Steel for Shear – **ρ**

5) Proximity of rigid support to point load – **a/d**

6) Yield Strength of Long. Tensile Steel – **fy**

7) Maximum Aggregate Size - $\mathbf{a_g}$

8) Cross Sectional Shape

9) Anchorage of longitudinal Tensile Steel

10) Presence of Crack Control R/F

A description about the parameters has given in Appendix A.

The Vt/Vn ratio was selected as the dependent variable (Y) and it was categorized into three categories: Under Predictions: Accurate Predictions: Over Predictions. Table 3.1 gives definitions for each category.

First, set of Continuous variables and Categorised variables were defined in the SPSS software and then inputs relevant to all 396 beam specimens were fed. It should be mentioned that the effective depth and the width of the web was entered in decimetres (dm). The outputs of SPSS Software are given below.

Table 4.2
Case Processing Summary – ACI Eqn 11-3
Beams without Shear Reinforcement

		N	Marginal Percentage
Y	A	282	71.2%
	B	51	12.9%
	C	63	15.9%
Cross Sectional Shape	2	136	34.3%
	3	260	65.7%
ag	1	49	12.4%
	2	261	65.9%
	3	86	21.7%
fy	1	13	3.3%
	2	50	12.6%
	3	49	12.4%
	4	116	29.3%
	5	121	30.6%
	6	47	11.9%
Crack control R/F	0	11	2.8%
	1	385	97.2%
Anchorage	0	274	69.2%
	1	122	30.8%
Valid		396	100.0%
Missing		0	
Total		396	
Subpopulation		381[a]	

a. The dependent variable has only one value observed in 379 (99.5%) subpopulations.

This table presents a summary about number of cases fallen in to each category of every categorize variables including the dependent variable Y. The definitions of category A,B and C are given in Table 3.1

Next SPSS presents three tables which can be used to describe the appropriateness of our regression model consisting of selected eleven explanatory variables for predicting the results.

Table 4.3
Output of SPSS for ACI 11-3 - Beams without Shear Reinforcement
a) Model Fitting Information b) Goodness of Fit c) Pseudo R-Square

a) **Model Fitting Information**

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	633.391	641.354	629.391			
Final	267.151	394.556	203.151	426.240	30	.000

b) **Goodness-of-Fit**

	Chi-Square	df	Sig.
Pearson	1504.684	730	.000
Deviance	200.378	730	1.000

c) **Pseudo R-Square**

Cox and Snell	.659
Nagelkerke	.827
McFadden	.674

Two models are referenced in the "Model Fitting Information" table above: (1) the "Intercept Only" model, also called the null model; it reflects the net effect of all variables not in the model plus error; and (2) the "Final Model", also called the fitted model, which is researcher's model comprised of the predictor variables; the logistic equation is the linear combination of predictor variables which maximizes the log likelihood that the dependent variable equals the predicted value/class/group. The difference in the -2 log likelihood (-2LL) measures how much the final model improves over the null model.

Therefore a well-fitting model is significant at the 0.05 level or better, as in the Model Fitting Information Table above, meaning researcher's model is significantly different from the one with the constant only. That is, a finding of significance ($p <= 0.05$ is the usual cutoff) leads to rejection of the null hypothesis that all of the predictor effects are zero. When this likelihood test is significant, at least one of the predictors is significantly related to the dependent variable. Alternatively, the likelihood ratio test tests the null hypothesis that all population logistic regression coefficients except the constant are zero. Finally it can be stated that, the likelihood ratio test reflects the difference between error not knowing the independents (initial chi-square) and error when the independents are included in the model (deviance). When probability (model chi-square) $<=0.05$, we reject the null hypothesis that knowing the independents makes no difference in predicting the dependent in logistic regression.

The "Goodness of Fit" table gives two similar overall model fit tests. Both are chi-square methods, but the Pearson statistic is based on traditional chi-square and the deviance statistic is based on likelihood ratio chi-square. The deviance test is preferred over the Pearson (Menard, 2002: 47). Adequate fit corresponds to a finding of non-significance for these tests. As the both goodness-of-fit test statistics are non-significant for our model, we fail to reject the null hypothesis that there is no difference between observed and model-predicted values, implying that the model's estimates fit the data at an acceptable level.

Next table called Psedo-R^2 attempt to measure the strength of association. But in logistic regression there is no widely-accepted direct analog to Ordinary Least Square regression's R^2. Therefore they are not widely accepted. They presented here give the complete out of the SPSS software.

Table 4.4
Output of SPSS for ACI 11-3 - Beams without Shear Reinforcement
Likelihood Ratio Test

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	267.151	394.556	2.032E2	.000	0	.
fcMPa	355.556	474.999	295.556	92.406	2	.000
bw_dm	263.895	383.337	203.895	.744	2	.689
ds_dm	323.774	443.216	263.774	60.623	2	.000
ad	283.789	403.231	223.789	20.638	2	.000
rhoshearlong.Steel	371.381	490.823	311.381	108.230	2	.000
CrossSectionalShape	266.829	386.271	206.829	3.678	2	.159
ag	268.247	379.726	212.247	9.096	4	.049
fy	257.825	345.416	213.825	10.675	10	.383
CrackcontrolRF	278.038	397.480	218.038	14.887	2	.001
Anchorage	263.601	383.043	203.601	.450	2	.798

Next table is called “likelihood Ratio Test”. In here likelihood ratio test has been used to identify the more important variable for the strength of the model. It has been done by comparing the different in -2LL for the overall model with a nested model. The nested model is created by dropping a variable from the full model. In this situation, the likelihood ratio test tests if the logistic regression coefficient for the dropped variable can be treated as 0, thereby justifying dropping the variable from the model. A non-significant likelihood ratio test indicates no difference between the full and the reduced models, hence justifying dropping the given variable so as to have a more parsimonious model that works just as well. In SPSS output, the "Likelihood Ratio Tests" table contains the likelihood ratio tests of individual model parameters and it shows that the models without b_w, f_y and Cross sectional shape are not significantly different from the full model and therefore Age should be dropped based on preference for the more parsimonious reduced model.

Next set gives an important table which contains the estimated parameter values for regression coefficients. The "b" values of Table 4.5 are the estimators of β values in the logistic regression model:

$$\text{Logit}\,(P_A) = \ln\left(\frac{P_A}{P_B}\right) = \beta_{A0} + \beta_{A1}X_1 + \beta_{A2}X_2 + \ldots\ldots + \beta_{A11}X_{11} \qquad (4.2)$$

Similarly the "b" values of Table 4.5a are the estimators of β values in the logistic regression model:

$$\text{Logit}\,(P_C) = \ln\left(\frac{P_C}{P_B}\right) = \beta_{C0} + \beta_{C1}X_1 + \beta_{C2}X_2 + \ldots\ldots + \beta_{C11}X_{11} \qquad (4.3)$$

It should be noted that the Category B of Dependent variable has been taken as the reference category. Interpretation of this "b" values will be discussed later.

The final table of the SPSS output is the "Classification table" (Table 4.6). The columns are the predicted values of the dependent, while the rows are the observed (actual) values of the dependent. In a perfect model, all cases will be on the diagonal and the overall percent correct will be 100%. This table indicates that more than 90% predictions of our model is correct. It should be mentioned here when classifying the predicted category the cutoff probability has been taken as 0.5. For example, all beams for which the predicted probability of a beam fallen in to category "A" greater than 0.5 counts for the predictions of category "A".

Table 4.5a

Output of SPSS for ACI 11-3 - Beams without Shear Reinforcement

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)	95% Confidence Interval for Exp (B)	
								Lower Bound	Upper Bound
A	Intercept	6.210	2.480	6.268	1	.012			
	bw_dm	-.004	.258	.000	1	.988	.996	.601	1.651
	ds_dm	-.746	.165	20.544	1	.000	.474	.343	.655
	ad	-1.251	.330	14.348	1	.000	.286	.150	.547
	rhoshearlong.Steel	3.550	.645	30.317	1	.000	34.825	9.841	123.240
	fcMPa	-.092	.020	22.095	1	.000	.912	.878	.948
	[CrossSectionalShape=2]	2.617	1.585	2.727	1	.099	13.697	.613	305.957
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.	.	.
	[ag=1]	-.503	1.735	.084	1	.772	.605	.020	18.108
	[ag=2]	.570	.698	.666	1	.414	1.768	.450	6.945
	[ag=3]	0[b]	.	.	0	.	.	.	.
	[fy=1]	-2.445	2.036	1.442	1	.230	.087	.002	4.694
	[fy=2]	-1.796	2.045	.771	1	.380	.166	.003	9.137
	[fy=3]	.227	2.076	.012	1	.913	1.255	.021	73.344
	[fy=4]	-1.167	1.923	.368	1	.544	.311	.007	13.486
	[fy=5]	-1.840	2.025	.826	1	.364	.159	.003	8.406
	[fy=6]	0[b]	.	.	0	.	.	.	.
	[CrackcontrolRF=0]	3.106	1.817	2.921	1	.087	22.331	.634	786.513
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.	.	.
	[Anchorage=0]	.408	.680	.361	1	.548	1.504	.397	5.703
	[Anchorage=1]	0[b]	.	.	0	.	.	.	.

a. The reference category is: B.

b. This parameter is set to zero because it is redundant.

Table 4.5b

Output of SPSS for ACI 11-3- Beams without Shear Reinforcement

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)	95% Confidence Interval for Exp (B)	
								Lower Bound	Upper Bound
C	Intercept	-20.180	2.122	90.424	1	.000			
	bw_dm	.155	.176	.775	1	.379	1.168	.827	1.649
	ds_dm	.563	.156	13.028	1	.000	1.755	1.293	2.383
	ad	.190	.335	.322	1	.570	1.209	.628	2.330
	rhoshearlong.Steel	-2.255	.728	9.594	1	.002	.105	.025	.437
	fcMPa	.075	.018	17.026	1	.000	1.078	1.040	1.117
	[CrossSectionalShape=2]	-12.544	4601.040	.000	1	.998	3.565E-6	.000	.[c]
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.	.	.
	[ag=1]	.854	1.383	.381	1	.537	2.349	.156	35.336
	[ag=2]	-1.317	.840	2.462	1	.117	.268	.052	1.388
	[ag=3]	0[b]	.	.	0	.	.	.	.
	[fy=1]	14.888	1.506	97.719	1	.000	2922551.719	152684.042	55941069.242
	[fy=2]	15.362	1.061	209.484	1	.000	4697027.400	586605.301	37609729.008
	[fy=3]	16.447	1.152	203.673	1	.000	13895385.413	1451792.247	1.330E8
	[fy=4]	15.944	1.051	230.183	1	.000	8401004.726	1071056.710	65894625.158
	[fy=5]	17.055	.000	.	1	.	25508650.166	25508650.166	25508650.166
	[fy=6]	0[b]	.	.	0	.	.	.	.
	[CrackcontrolRF=0]	-3.534	1.270	7.741	1	.005	.029	.002	.352
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.	.	.
	[Anchorage=0]	-.161	.842	.037	1	.848	.851	.164	4.432
	[Anchorage=1]	0[b]	.	.	0	.	.	.	.

a. The reference category is: B.

b. This parameter is set to zero because it is redundant.

Table 4.6

Output of SPSS for ACI 11-3- Beams without Shear Reinforcement

Classification Table

Observed	Predicted			
	A	B	C	Percent Correct
A	273	8	1	96.8%
B	18	25	8	49.0%
C	1	8	54	85.7%
Overall Percentage	73.7%	10.4%	15.9%	88.9%

As it was discussed earlier likelihood ratio can be used to identify more important variables for the strength of the model. When discussing about the parameters affecting accuracy of predictions it is convenient to have a more parsimonious model consisting only of significant variables that works just as well as the full model. There are two possible ways to have a nested model: first one is to manually identify the more significant parameters in the full model using likelihood Ratio Test and then selected them to have the nested model: second method is to use Stepwise regression offered by the SPSS. In Stepwise regression using the Likelihood ratio Test SPSS determines automatically which variable to add or drop from the model. As it is a data driven method, the first method is preferred over the second method. Therefore in our study when discussing about the critical parameters affecting the accuracy the full model or the manually created nested model has been preferred whenever possible. But in some situations, when calculating the full model, numerical problems occur in calculating likelihood ratio for some variables. In such situations stepwise logistic regression method has been used to identify significant parameters. SPSS output for nested model is given bellow.

Table 4.7
Case Processing Summary –Nested Model– ACI Eqn 11-3
- Beams without Shear Reinforcement

		N	Marginal Percentage
Y	A	282	71.2%
	B	51	12.9%
	C	63	15.9%
ag	1	49	12.4%
	2	261	65.9%
	3	86	21.7%
Crack control R/F	0	11	2.8%
	1	385	97.2%
Valid		396	100.0%
Missing		0	
Total		396	
Subpopulation		380[a]	

And other tables are also presented below.

Table 4.8
Output of SPSS for ACI 11-3 - Beams without Shear Reinforcement -Nested Model
a) Model Fitting Information b) Goodness of Fit c) Pseudo R-Square

a) **Model Fitting Information**

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	633.391	641.354	629.391			
Final	250.878	314.581	218.878	410.513	14	.000

b) **Goodness-of-Fit**

	Chi-Square	df	Sig.
Pearson	1220.269	746	.000
Deviance	216.106	746	1.000

c) **Pseudo R-Square**

Cox and Snell	.645
Nagelkerke	.809
McFadden	.649

Table 4.9
Output of SPSS for ACI 11-3 - Nested Model
Likelihood Ratio Test
- Beams without Shear Reinforcement

	Model Fitting Criteria			Likelihood Ratio Tests		
Effect	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	250.878	314.581	2.189E2	.000	0	.
fcMPa	369.696	425.436	341.696	122.818	2	.000
ad	264.300	320.039	236.300	17.421	2	.000
CrackcontrolRF	268.053	323.793	240.053	21.175	2	.000
ag	254.070	301.847	230.070	11.192	4	.024
ds_dm	364.340	420.079	336.340	117.462	2	.000
rhoshearlong.Steel	400.984	456.724	372.984	154.106	2	.000

The next table of SPSS output which lists the b coefficients also lists the standard error of b, the Wald statistic and its significance and the odds ratio (labeled Exp(b)) as well as confidence limits on the odds ratio. This is the "Parameter Estimates" in SPSS (Table 4.11). The values are the "b" estimates of the regression coefficients, "β". The Wald statistic is use to test the significant of the individual logistic coefficients for each independent variable (that is, to test the null hypothesis in logistic regression that a particular logistic coefficient is zero). There fore in order to make global statement about the significant of an variable the parameter estimated "b" should be significant.

In section 3.5.2 the interpretation was given as: The regression coefficient β_1 is the log of the odds ratio comparing the odds after a one unit increase in X_1 to the original odds where X_1 is the parameter associate with the regression coefficient β_1. For example, the "b" value estimated for the variable: Percentage longitudinal tensile reinforcement-ρ in the regression equation for category A, ln(P_A/P_B) is 3.875. This means:

"1% increase of percentage of Longitudinal Tensile Steel, 3.875 times increases the Log (P_A/P_B)"

$$1\% \uparrow \rho \Longrightarrow 3.875 \uparrow \mathrm{Log}(P_A/P_B)$$

As we know: when $\log\left(\dfrac{P_A}{P_B}\right) > 0 \Leftrightarrow \dfrac{P_A}{P_B} < 1 \Leftrightarrow P_A > P_B$

$$\log\left(\frac{P_A}{P_B}\right) < 0 \Leftrightarrow \frac{P_A}{P_B} < 1 \Leftrightarrow P_A < P_B$$

Alternatively, for a particular beam one increase of the percentage of longitudinal tensile steel while keeping other parameters constant, increases the log value of the ratio of the probability that the beam can fall in to under predicted category to the probability that it can fall in to the accurately predicted category.

Therefore we can conclude that ACI Equation 11-3 tends to under predict the shear strength of beams when percentage of longitudinal tensile steel increases.

Table 4.10
Output of SPSS for ACI 11-3-Nested Model
Parameter Estimates
- Beams without Shear Reinforcement

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)	95% Confidence Interval for Exp (B)	
								Lower Bound	Upper Bound
A	Intercept	4.532	1.190	14.513	1	.000			
	fcMPa	-.105	.018	33.407	1	.000	.900	.869	.933
	ad	-.868	.232	13.952	1	.000	.420	.266	.662
	[CrackcontrolRF=0]	4.589	1.664	7.609	1	.006	98.404	3.775	2565.117
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.	.	.
	[ag=1]	-.389	.864	.203	1	.652	.678	.125	3.682
	[ag=2]	.468	.566	.683	1	.408	1.596	.527	4.836
	[ag=3]	0[b]	.	.	0	.	.	.	.
	ds_dm	-.830	.134	38.147	1	.000	.436	.335	.568
	rhoshearlong.Steel	3.806	.600	40.231	1	.000	44.953	13.869	145.701
C	Intercept	-4.187	1.663	6.339	1	.012			
	fcMPa	.074	.017	19.750	1	.000	1.077	1.042	1.113
	ad	.244	.297	.672	1	.412	1.276	.712	2.285
	[CrackcontrolRF=0]	-3.645	1.222	8.891	1	.003	.026	.002	.287
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.	.	.
	[ag=1]	.090	.867	.011	1	.918	1.094	.200	5.988
	[ag=2]	-1.660	.669	6.161	1	.013	.190	.051	.705
	[ag=3]	0[b]	.	.	0	.	.	.	.
	ds_dm	.595	.139	18.404	1	.000	1.813	1.381	2.379
	rhoshearlong.Steel	-2.000	.584	11.735	1	.001	.135	.043	.425

a. The reference category is: B.

This factor is further emphasized from the corresponding regression coefficient of category C, ln(P_c/P_B). The "b" value for the variable of Percentage longitudinal tensile reinforcement-ρ is -1.888. which implies that: 1% increase of percentage of Longitudinal Tensile Steel, 1.888 times decreases the Log (P_C/P_B).

From Table 4.11 six critical parameters can be identified to have influence on the accuracy of the ACI Equation 11-3. Other than the percentage of longitudinal tensile steel, there are three other variables on which we can make global statements on their effect, based on the results of this study. Other three parameters and their effects are given below.

1) *Effective depth of the beam* **(d) – With the increase of the effective depth of the beam ACI Equation 11-3 tends to over predict the shear strength of a beam.**
2) *Compression capacity of concrete* **(f'c) –ACI 11-3 Equation tends to over estimate the shear strength of beams when compression capacity of concrete increases.**
3) *Crack control reinforcement* **– It can be identified that the ACI – Equation 11-3 has underestimated the influence of crack control reinforcement on shear strength of a beam.**

Estimated regression coefficients for other three variables are either significant only in one regression equation out of the two regression equations for two categories of the dependent variable or they are not significant in any regression equation. Therefore the corresponding parameters are not taken to make global statement on the accuracy of the method.

Finally SPSS output present the Classification table (Table 4.11). According to which 89% predictions of over regression model are correct.

Table 4.11
Output of SPSS for ACI 11-3
Classification Table
- Beams without Shear Reinforcement

Observed	Predicted			
	A	B	C	Percent Correct
A	274	7	1	97.2%
B	16	28	7	54.9%
C	4	7	52	82.5%
Overall Percentage	74.2%	10.6%	15.2%	89.4%

Estimating Critical Values for parameters affecting the Accuracy.

After identifying the significant variables affecting the accuracy of the shear strength predictions it is important to estimate when that particular parameter begin to influence the accuracy of the prediction. Multinomial logistic regression can be used to have a rough estimation for this turning point. The steps involves in this process is given below.

1) Sort the database according to the parameter selected to study the influence.

Note:- Data is sorted in ascending order.

2) Database is divided into blocks depending on the value of the selected parameter. (Ex:- For the parameter d, beams with effective depth 0 to 3 dm is categorised as the first block, next for every 1 dm increment a new block is created. Beams with effective depth greater than 1000 dm is taken as last block)

3) Then multinomial logistic regression is carried out for the data in the first block and then the next block is added. Likewise blocks are added one at a time until effect of that parameter become significant.
4) Value of the starting point of the last block is taken as the estimation for the critical value of that parameter.

A complete set of multinomial logistic regression results for estimating the critical value for Effective Depth for the ACI Equation 11-3 are given in Appendix C. Approximated critical values for variables with significant effect is given below:

1) ACI Equation 11-3 tends to over predict the shear strength of a beam without shear reinforcement when effective depth, d is approximately greater than 500mm.
2) ACI Equation 11-3 tends to under predict the shear strength of a beam without shear reinforcement when Percentage of Longitudinal Tensile Steel, ρ is approximately greater than 1.0%.
3) ACI Equation 11-3 tends to over predict the shear strength of a beam without shear reinforcement when Compression capacity of concrete, (f'c) is approximately greater than 50 MPa

***Equation* 11-5 (ACI Code-2002)**

$$Vc=\left(1.9\sqrt{f_c^{'}}+2500\rho_w\frac{V_u d}{M_u}\right)b_w d \qquad (4.4)$$

Figure 4.3 shows the predicted shear strength using above equation Vs the Tested shear strength. This graph shows that only about 75% of predictions of this equation are under predictions.

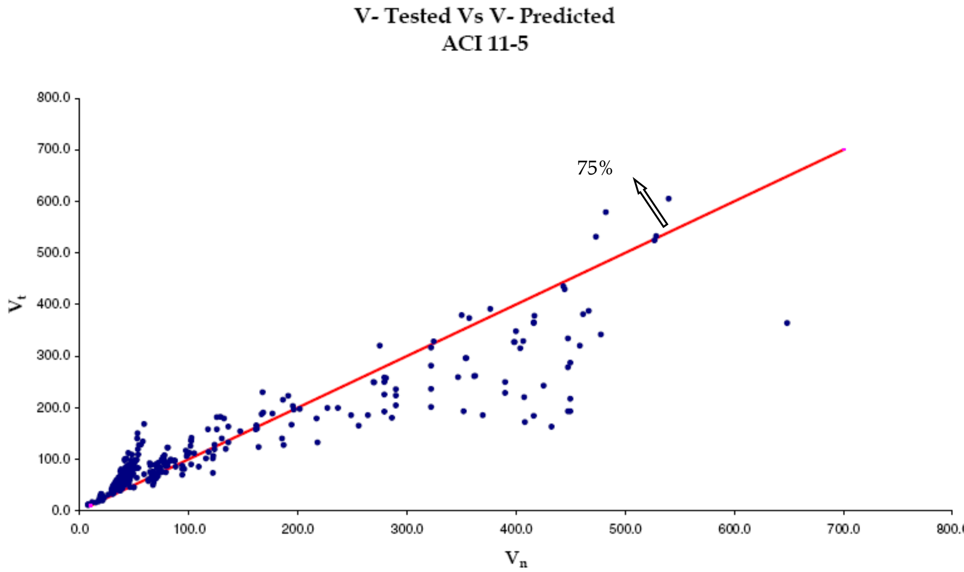


Figure 4.3
Tested Shear Strength V_t Vs Predicted Shear Strength Vn - ACI Equation 11-5 - Beams without Shear Reinforcement

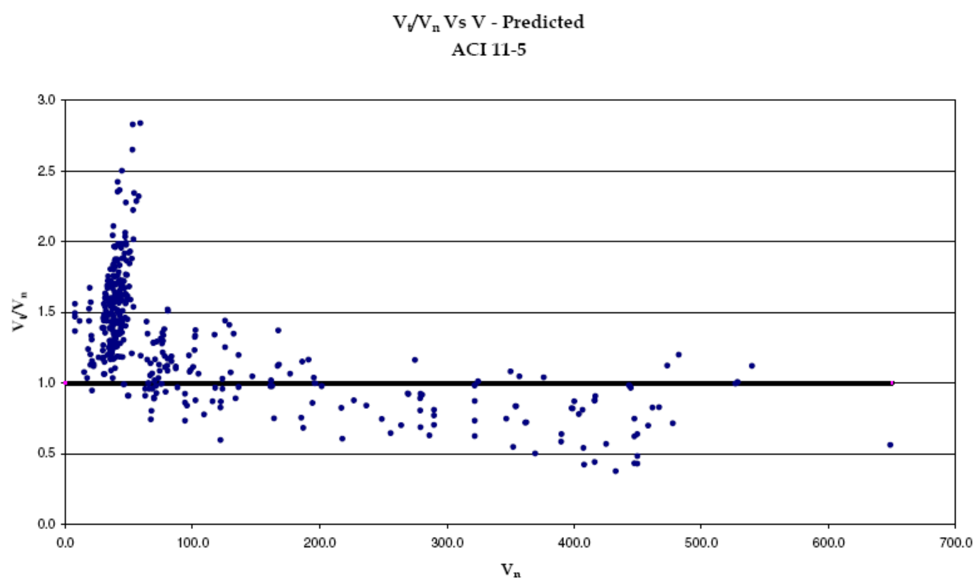


Figure 4.4
V_t/V_n Vs V_t Tested Shear Strength - ACI Equation 11-5 - Beams without Shear Reinforcement

Figure 4.4 plots the V_t/V_n value vs the predicted shear strength. It shows when predicted shear strength is high most of the time it gives an over prediction. Descriptive statistics of the V_t/V_n ratio are given in Table 4.12.

Table 4.12
Descriptive statistics of V_t/V_n –ACI-Equation11-5
- Beams without Shear Reinforcement

	Vt/Vn
Mean	1.32
Median	1.30
Standard Deviatio	0.42
Range	2.46
Minimum	0.38
Maximum	2.84

These Figures and table highlight inaccuracy as well as the unsafe practices of the predictions of the ACI Equation 11-5. As we did early, Multinomial Logistic Regression can be carried out to identify the critical parameters influencing the accuracy and their effect. Explanatory variables are same as what we used in previous case and are explained in Appendix A.

Results of Multinomial Logistic Regression have been given in Appendix B and only the “Parameter Estimates” table (Table 4.13) is presented here.

Table 4.13
Parameter Estimates –ACI-Equation11-5
- Beams without Shear Reinforcement

Y^a		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	4.886	1.248	15.317	1	.000	
	fcMPa	-.106	.018	34.259	1	.000	.899
	ds_dm	-1.018	.157	42.185	1	.000	.361
	rhoshearlong.Steel	3.316	.541	37.544	1	.000	27.537
	ad	-.540	.219	6.097	1	.014	.583
	[ag=1]	-.836	.860	.944	1	.331	.433
	[ag=2]	.000	.566	.000	1	.999	1.000
	[ag=3]	0^b	.	.	0	.	.
	[CrackcontrolRF=0]	5.083	1.861	7.462	1	.006	161.246
	[CrackcontrolRF=1]	0^b	.	.	0	.	.
C	Intercept	-4.051	1.861	4.736	1	.030	
	fcMPa	.081	.019	17.562	1	.000	1.085
	ds_dm	.802	.180	19.974	1	.000	2.231
	rhoshearlong.Steel	-2.153	.604	12.707	1	.000	.116
	ad	-.123	.362	.116	1	.734	.884
	[ag=1]	.860	1.020	.712	1	.399	2.364
	[ag=2]	-2.061	.731	7.952	1	.005	.127
	[ag=3]	0^b	.	.	0	.	.
	[CrackcontrolRF=0]	-4.820	1.415	11.613	1	.001	.008
	[CrackcontrolRF=1]	0^b	.	.	0	.	.

a. The reference category is: B.

Four parameters namely: Compression Capacity of Concrete, Effective depth, Percentage Area of Tensile Reinforcement and Presence of Crack Control Reinforcement are significantly affect the accuracy of the ACI equation 11-5 in the similar manner as for Equation 11-3. Their effects can be listed as:

1) ***Percentage Area of Tensile Reinforcement ρ*** **- ACI Equation 11-5 tends to under predict the shear strength of beams when percentage of longitudinal tensile steel increases.**
2) ***Effective depth of the beam*** **(d) – With the increase of the effective depth of the beam ACI Equation 11-5 tends to over predict the shear strength of a beam.**

3) ***Compression capacity of concrete* (f'$_c$) –ACI 11-5 Equation tends to over estimate the shear strength of beams when compression capacity of concrete increases.**
4) ***Crack control reinforcement* – It can be identified that the ACI – Equation 11-5 has underestimated the influence of crack control reinforcement on shear strength of a beam.**

Other parameters influencing the accuracy are: Maximum Aggregate size: Presence of Compression Reinforcement and: a/d ratio of the beam. Unfortunately the effect of these parameters can not be clearly identified.

Influence of above parameters are not discussed here as the discussion is similar the discussion of the ACI Equation 11-3.

Estimating Critical Values for parameters affecting the Accuracy.

As explained in earlier an estimation for the critical value of the parameter after which it become significant for the accuracy of the equation is done using the Multinomial Logistic Regression. The relevant SPSS results have given in Appendix C. The estimated critical values for significant parameters are given below:

1) ACI Equation 11-5 tends to over predict the shear strength of a beam without shear reinforcement when effective depth, d is approximately greater than 400 mm.
2) ACI Equation 11-5 tends to under predict the shear strength of a beam without shear reinforcement when Percentage of Longitudinal Tensile Steel, ρ is approximately greater than 1.25 %.
3) ACI Equation 11-5 tends to over predict the shear strength of a beam without shear reinforcement when Compression capacity of concrete, (f'c) is approximately greater than 45 MPa

4.2.2 Slender Beams with Shear Reinforcement

According to the guidelines given in ACI Code 2002, 143 numbers of slender beams can be identified in our database. All thirteen parameters given in Appendix A have used in the Multinomial Logistic Analysis for the slender beams without shear reinforcement.

***Equation* 11-3 (ACI Code-2002)**

$$V_n = 2\sqrt{f_c'}\,b_w d \quad + \frac{A_v f_{yt} d}{s} \tag{4.5}$$

The predicted shear strength from above equation vs the tested shear strength given in Fig.4.5. Also descriptive statistics have been in Table 4.15.

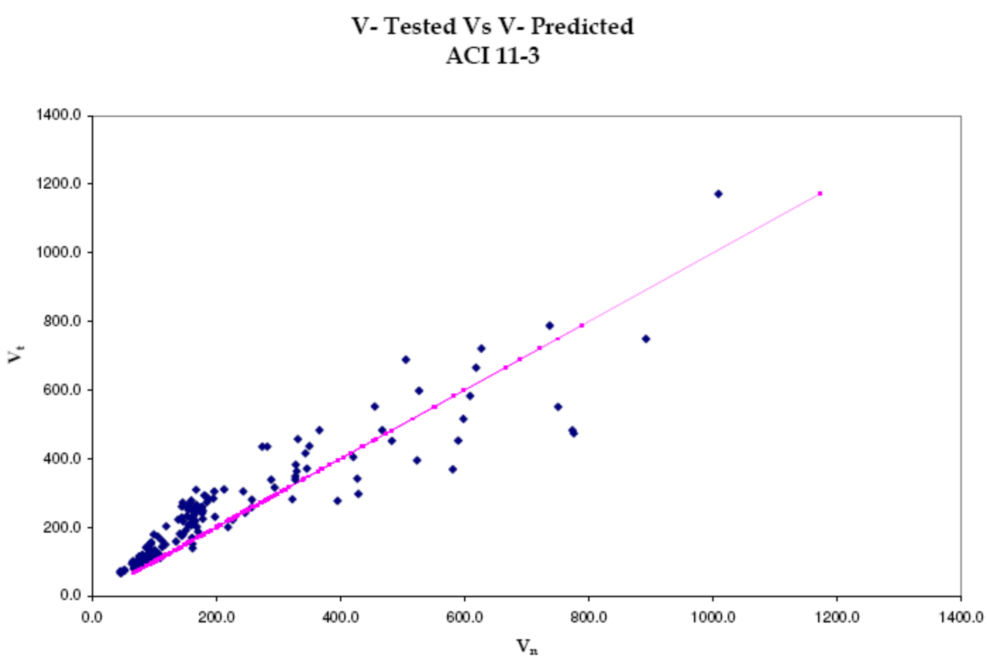


Figure 4.5
Tested Shear Strength V_t Vs Predicted Shear Strength V_n - ACI Equation 11-3 - Beams with Shear Reinforcement

Table 4.14
Descriptive statistics of V_t/V_n –ACI-Equation11-3
- Beams with Shear Reinforcement

	V_t/V_n
Mean	1.29
Median	1.31
Standard Deviatio	0.27
Range	1.26
Minimum	0.61
Maximum	1.87

Then the results were further analysed using SPSS. Due to the numerical problems occur in the likelihood ratio test calculation for some variables stepwise logistic regression has been used to identify the significant variables. Complete SPSS output of the full model and the model obtained using stepwise process have been given in Appendix B. only the "Likelihood Ratio Test" Table and a part of "Parameter Estimates" table of the stepwise process have given below.

Table 4.15
Likelihood Ratio Test – ACI-113-3 - Beams with Shear Reinforcement

	Model Fitting Criteria			Likelihood Ratio Tests		
Effect	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	121.484	174.815	85.484[a]	.000	0	.
dsm	181.024	228.429	1.490E2	63.540	2	.000
CrackControalCheck	126.865	174.270	94.865	9.381	2	.009
ag	129.441	176.847	97.441	11.957	2	.003
fy	134.241	163.869	114.241	28.757	8	.000
rhostirrup	124.254	171.660	92.254	6.770	2	.034

Table 4.16

Parameter Estimates – ACI-113-3 - Beams with Shear Reinforcement

Y[a]		B	Std. Error	Wald	df	Sig.
A	Intercept	10.753	2.447	19.310	1	.000
	dsm	-1.500	.354	17.936	1	.000
	[CrackControalCheck=0]	-19.123	6723.876	.000	1	.998
	[CrackControalCheck=1]	0[c]	.	.	0	.
	[ag=2]	1.770	.886	3.995	1	.046
	[ag=3]	0[c]	.	.	0	.
	[fy=1]	11.577	1619.903	.000	1	.994
	[fy=2]	-5.202	1.561	11.104	1	.001
	[fy=3]	-2.307	1.349	2.928	1	.087
	[fy=4]	-4.693	1.565	8.990	1	.003
	[fy=5]	0[c]	.	.	0	.
	rhostirrup	-.057	.089	.412	1	.521
C	Intercept	-20.146	1193.393	.000	1	.987
	dsm	.797	.630	1.600	1	.206
	[CrackControalCheck=0]	-24.322	.000	.	1	.
	[CrackControalCheck=1]	0[c]	.	.	0	.
	[ag=2]	-3.688	2.564	2.069	1	.150
	[ag=3]	0[c]	.	.	0	.
	[fy=1]	19.339	2449.211	.000	1	.994
	[fy=2]	14.042	1193.387	.000	1	.991
	[fy=3]	17.849	1193.387	.000	1	.988
	[fy=4]	3.333	1272.669	.000	1	.998
	[fy=5]	0[c]	.	.	0	.
	rhostirrup	.122	.090	1.810	1	.179

a. The reference category is: B.

Six parameters namely: Effective depth: Percentage Area of Tensile Reinforcement, Presence of Crack Control Reinforcement, Maximum aggregate size and Yield strength of longitudinal tensile steel have been identified to affect the accuracy of this method. But their influences cannot be clearly identified.

Equation **11-5 (ACI Code-2002)**

$$V_n = \left(1.9\sqrt{f_c^{'}} + 2500\rho_w \frac{V_u d}{M_u}\right) b_w d \quad + \quad \frac{A_v f_{yt} d}{s} \qquad (4.6)$$

Here also the graph predicting shear strength from above equation vs the tested shear strength is presented first (Fig.4.6). Also descriptive statistics have been displayed in Table 4.17.

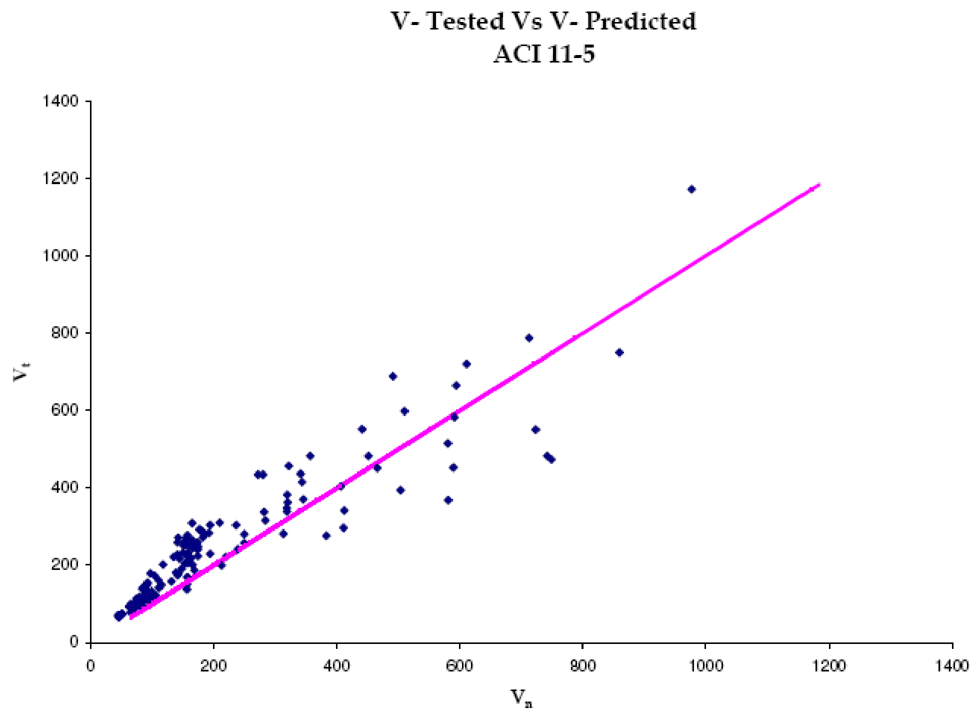


Figure 4.6
Tested Shear Strength V_t Vs Predicted Shear Strength V_n - ACI Equation 11-5 - Beams with Shear Reinforcement

Table 4.17
Descriptive statistics of V_t/V_n –ACI-Equation11-5 - Beams with Shear Reinforcement

	V_t/V_n
Mean	1.05
Median	1.03
Standard Deviation	0.20
Range	1.00
Minimum	0.56
Maximum	1.57

As usual the complete SPSS output of the full model and the reduced model is given in Appendix B. only the "Likelihood Ratio Test" Table and a part of "Parameter Estimates" table are given below.

Table 4.18

Parameter Estimates – ACI - 13-5 - Beams with Shear Reinforcement

Y^a		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	22.896	6.302	13.201	1	.000	
	ad	-2.127	.704	9.137	1	.003	.119
	ds_dm	-1.116	.433	6.641	1	.010	.328
	[fy=1]	11.969	7539.928	.000	1	.999	157850.503
	[fy=2]	-4.643	2.278	4.154	1	.042	.010
	[fy=3]	-11.021	4.085	7.279	1	.007	1.636E-5
	[fy=4]	-12.385	4.110	9.082	1	.003	4.182E-6
	[fy=5]	0^c	.	.	0	.	.
	[fvy=1]	-2.967	1.259	5.552	1	.018	.051
	[fvy=2]	-.830	.830	1.001	1	.317	.436
	[fvy=3]	-10.215	3.036	11.321	1	.001	3.662E-5
	[fvy=4]	-1.943	.943	4.246	1	.039	.143
	[fvy=5]	0^c	.	.	0	.	.
C	Intercept	-1.573	3.096	.258	1	.611	
	ad	-.232	.592	.153	1	.695	.793
	ds_dm	.611	.235	6.765	1	.009	1.841
	[fy=1]	.642	.000	.	1	.	1.901
	[fy=2]	1.400	1.079	1.683	1	.194	4.056
	[fy=3]	.188	1.009	.035	1	.852	1.207
	[fy=4]	-.540	1.455	.138	1	.710	.582
	[fy=5]	0^c	.	.	0	.	.
	[fvy=1]	-3.139	1.671	3.528	1	.060	.043
	[fvy=2]	-.577	1.115	.268	1	.605	.562
	[fvy=3]	-2.176	1.490	2.133	1	.144	.114
	[fvy=4]	-1.491	1.309	1.299	1	.254	.225
	[fvy=5]	0^c	.	.	0	.	.

a. The reference category is: B.

Four parameters namely: a/d ratio, Effective depth, Percentage Area of Tensile Reinforcement, Yield strength of longitudinal tensile steel and Yield strength of transverse tensile steel have been identified to affect the accuracy of this method. Influence of the Effective depth can be identified as:

***Effective depth of the beam* (d) – With the increase of the effective depth of the beam ACI Equation 11-5 tends to over predict the shear strength of a beam.**

4.2.3 Deep Beams

According to the ACI guidelines 66 beams could be categorised as deep beams. The predicted shear strength by the ACI Code Vs the tested shear strength is plotted below in Fig.4.7

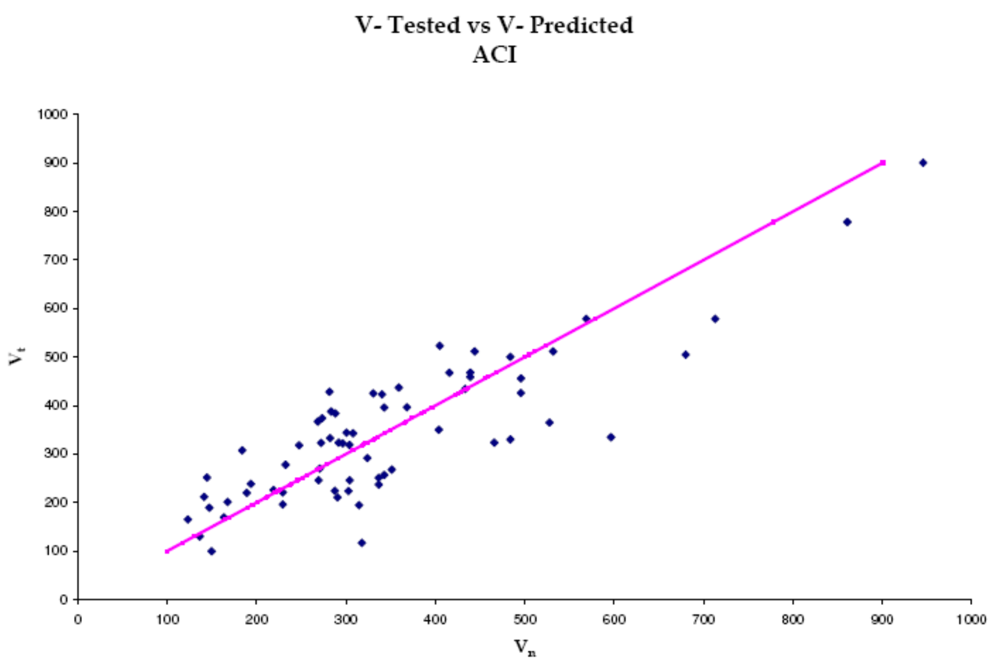


Figure 4.7
Tested Shear Strength V_t Vs Predicted Shear Strength V_n - ACI –Deep Beam

Table 4.19
Descriptive statistics of V_t/V_n –ACI-Deep Beam

	Vt/Vc
Mean	1.03
Standard Deviation	0.27
Range	1.37
Minimum	0.37
Maximum	1.74

The complete multinomial logistic regression results for this model has been given in Appendix B and only the "Parameters Estimate" tables has been presented here.

Table 4.20
Parameter Estimates – ACI-Deep Beam

Y^a		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	-204.830	6270.777	.001	1	.974	
	fcMPa	-.714	.550	1.682	1	.195	.490
	[fy=2]	27.262	6227.078	.000	1	.997	6.911E11
	[fy=3]	-59.128	6270.799	.000	1	.992	2.095E-26
	[fy=4]	42.075	6271.825	.000	1	.995	1.874E18
	[fy=6]	0^c	.	.	0	.	.
	[CrackControlcheck=0]	25.117	739.003	.001	1	.973	8.094E10
	[CrackControlcheck=1]	0^c	.	.	0	.	.
	bwm	977.184	.000	.	1	.	$.^b$
C	Intercept	23.095	2.506	84.903	1	.000	
	fcMPa	.073	.093	.611	1	.435	1.075
	[fy=2]	-21.186	1.762	144.500	1	.000	6.294E-10
	[fy=3]	-20.527	1.909	115.571	1	.000	1.217E-9
	[fy=4]	-23.491	.000	.	1	.	6.280E-11
	[fy=6]	0^c	.	.	0	.	.
	[CrackControlcheck=0]	-.297	1.200	.061	1	.805	.743
	[CrackControlcheck=1]	0^c	.	.	0	.	.
	bwm	-19.432	19.602	.983	1	.322	3.636E-9

a. The reference category is: B.

Four parameters namely: Compressive Strength of Concrete, Yield Strength of Longitudinal Tensile steel, Presence of Crack Control reinforcement and Width of the beam can be identified to influence the accuracy of the ACI deep beam guide lines.

4.3 BS Code

According to the guide lines given in BS 8110:

Total number of Slender Beams – 705

Slender beams without shear reinforcement – 524

Slender beams with shear reinforcement - 181

4.3.1 Slender Beams without Shear Reinforcement

$$V_n = 0.79 \text{ x } \left(\frac{100A_s}{b_v d}\right)^{\frac{1}{3}} \text{ x } \left(\frac{400}{d}\right)^{\frac{1}{4}} \text{ x } \left(\frac{f_{cu}}{25}\right)^{\frac{1}{3}} b_v d \tag{4.7}$$

Predicted shear strength (V_n) vs the Tested shear capacity(V_t). has been presented first and then the descriptive statistics of V_t/V_n ratio is given in the table below.

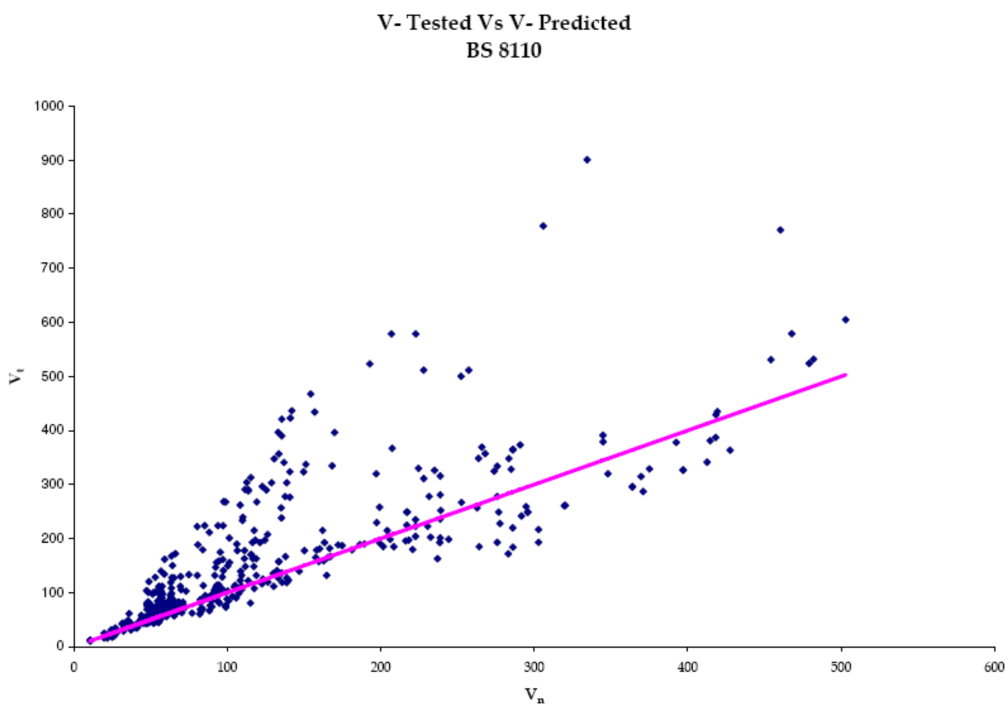


Figure 4.8
Tested Shear Strength V_t Vs Predicted Shear Strength V_n – BS 8110 - Beams with Shear r/f

Table 4.21
Descriptive statistics of V_t/V_n –BS 8110-Beams without Shear r/f

	Vt/Vn
Mean	1.29
Standard Deviation	0.50
Range	2.50
Minimum	0.61
Maximum	3.11

Complete SPSS output has been given in Appendix B. Only the Parameter Estimated table containing significant variables is displayed here.

Seven numbers of parameters have been found to effect the shear strength prediction of BS8110: Compressive Capacity of Concrete (f'_c), a/d ratio, Percentage Area of Tensile Reinforcement (ρ), Cross Sectional Shape, Yield strength of the Tensile Steel, Presence of Crack Control Reinforcement and Anchorage condition of the Tensile Reinforcement.

The influence of four parameters can be clearly identified as:

1) *Compression capacity of concrete* **(f'_c) –BS 8110 tends to over estimate the shear strength of beams when compression capacity of concrete increases**
2) *Effective depth to shear span ratio* **(a/d) – With the increase of the a/d ratio BS 8110 tends to over predict the shear strength of a beam.**
3) *Percentage Area of Tensile Reinforcement ρ* **– BS 8110 tends to under predict the shear strength of beams when percentage of longitudinal tensile steel increases.**
4) *Cross Sectional shape* **– It can be identified that the BS 8110 has a tendency of underestimating shear strength of T beams compared to rectangular beams.**

Table 4.22
Parameter Estimates – BS 8110 - Beams without Shear r/f

Y[a]		B	Std. Error	Wald	df	Sig.
A	Intercept	1.597	.764	4.368	1	.037
	fcMPa	-.034	.009	13.004	1	.000
	ad	-.685	.098	49.025	1	.000
	rhoshearlong.Steel	1.060	.139	57.915	1	.000
	[CrossSectionalShape=2]	.737	.380	3.758	1	.053
	[CrossSectionalShape=3]	0[b]	.	.	0	.
	[fy=1]	1.750	.629	7.750	1	.005
	[fy=2]	-.066	.620	.011	1	.915
	[fy=3]	1.075	.530	4.110	1	.043
	[fy=4]	1.037	.470	4.875	1	.027
	[fy=5]	.876	.477	3.373	1	.066
	[fy=6]	0[b]	.	.	0	.
	[CrackcontrolRF=0]	3.677	.958	14.727	1	.000
	[CrackcontrolRF=1]	0[b]	.	.	0	.
	[AnchorageCheck=0]	-1.619	.641	6.377	1	.012
	[AnchorageCheck=1]	0[b]	.	.	0	.
C	Intercept	-18.016	.835	465.668	1	.000
	fcMPa	.056	.010	33.419	1	.000
	ad	.782	.193	16.469	1	.000
	rhoshearlong.Steel	-.719	.240	9.000	1	.003
	[CrossSectionalShape=2]	-3.837	1.320	8.450	1	.004
	[CrossSectionalShape=3]	0[b]	.	.	0	.
	[fy=1]	-1.536	1.069	2.067	1	.151
	[fy=2]	1.430	.714	4.012	1	.045
	[fy=3]	-1.150	.840	1.873	1	.171
	[fy=4]	-.485	.691	.493	1	.483
	[fy=5]	.393	.715	.302	1	.583
	[fy=6]	0[b]	.	.	0	.
	[CrackcontrolRF=0]	-21.972	5795.959	.000	1	.997
	[CrackcontrolRF=1]	0[b]	.	.	0	.
	[AnchorageCheck=0]	13.519	.000	.	1	.
	[AnchorageCheck=1]	0[b]	.	.	0	.

a. The reference category is: B.

b. This parameter is set to zero because it is redundant.

Estimating Critical Values for parameters affecting the Accuracy.

As done for the ACI Code, critical values for parameters affecting the accuracy of this method are also estimated using the SPSS software. Complete output of this process is given in Appendix C. The estimated critical values have given below:

1) BS 8110 tends to over predict the shear strength of a beam without shear reinforcement when Compression capacity of concrete, (f'c) is approximately greater than 60 MPa
2) BS 8110 tends to over predict the shear strength of a beam without shear reinforcement when a/d ratio is approximately greater than 3.5
3) BS 8110 tends to under predict the shear strength of a beam without shear reinforcement when Percentage of Longitudinal Tensile Steel, ρ is approximately greater than 4.0 %.

4.3.2 Slender Beams with Shear Reinforcement

$$V_n = 0.79 \text{ x } \left(\frac{100A_s}{b_v d}\right)^{\frac{1}{3}} \text{ x } \left(\frac{400}{d}\right)^{\frac{1}{4}} \text{ x } \left(\frac{f_{cu}}{25}\right)^{\frac{1}{3}} b_v d + \frac{A_{sv} f_{vy} d}{s_v} \qquad (4.8)$$

As usual the graph predicting shear strength from above equation vs the tested shear strength has been presented first (Fig.4.9). Also descriptive statistics have been given in Table 4.23.

The SPSS output of “Log Likelihood Test” table and the “Parameter Estimates” table have also been presented below. Complete output of SPSS is given in Appendix B.

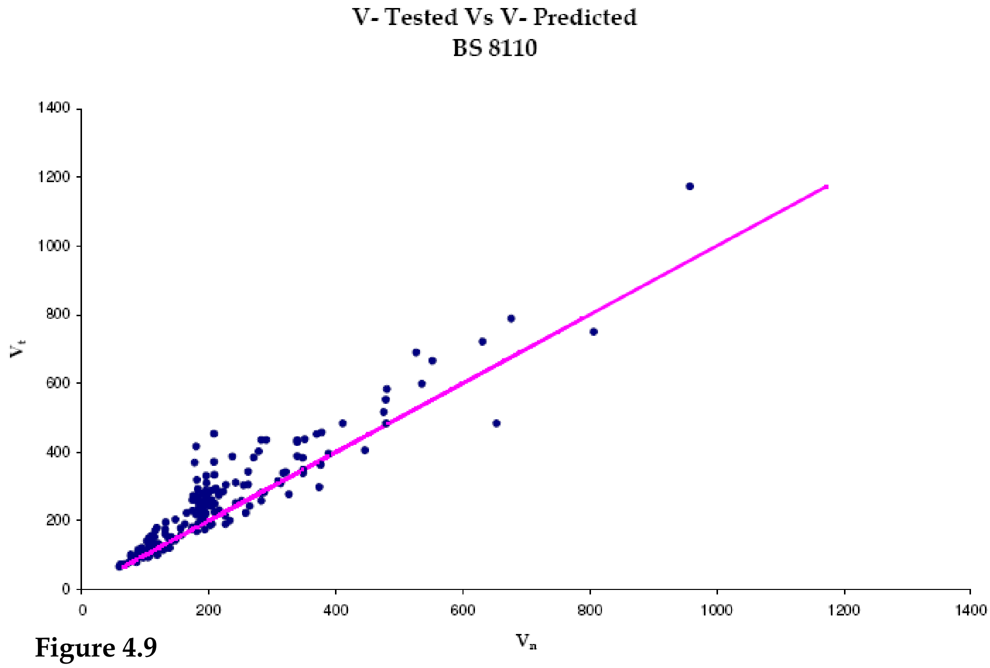


Figure 4.9
Tested Shear Strength V_t Vs Predicted Shear Strength V_n – BS 8110-Beams with Shear r/f

Table 4.23
Descriptive statistics of V_t/V_n –BS 8110-Beams with Shear r/f

	V_t/V_n
Mean	1.19
Standard Deviation	0.24
Range	1.57
Minimum	0.74
Maximum	2.31

Table 4.24
Likelihood Ratio Test –BS 8110-Beams with Shear r/f

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	305.114	445.848	2.171E2	.000	0	.
fcMPa	301.803	436.140	2.178E2	.689	2	.709
bw_dm	301.829	436.165	2.178E2	.715	2	.700
ds_dm	281.589	415.926	1.976E2	.	2	.
ad	310.988	445.325	2.270E2	9.875	2	.007
sd	290.458	424.795	2.065E2	.	2	.
rhoshearlong.Steel	288.169	422.506	2.042E2	.	2	.
rhostirrup	307.941	442.277	2.239E2	6.827	2	.033
CrossSection	299.595	427.534	2.196E2	2.481	4	.648
ag	296.951	431.288	2.130E2	.	2	.
fy	301.863	410.612	2.339E2	16.750	10	.080
fvy	293.732	408.878	2.217E2	4.618	8	.797
CrackControalCheck	301.173	435.510	2.172E2	.059	2	.971
Anchorage	284.485	418.822	200.485	.	2	.

Table 4.25

Parameter Estimates – BS 8110 - Beams with Shear reinforcement

Y^a		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	-4.611	2.252	4.192	1	.041	
	fcMPa	-.008	.012	.518	1	.471	.992
	bw_dm	.344	.394	.760	1	.383	1.410
	ds_dm	-.063	.213	.088	1	.767	.939
	ad	-1.168	.526	4.923	1	.026	.311
	sd	-.331	.160	4.258	1	.039	.718
	rhoshearlong.Steel	.823	.414	3.955	1	.047	2.278
	rhostirrup	-2.235	1.051	4.525	1	.033	.107
	[CrossSection=1]	.576	2.269	.064	1	.800	1.778
	[CrossSection=2]	1.635	1.092	2.244	1	.134	5.131
	[CrossSection=3]	0^b	.	.	0	.	.
	[ag=2]	.507	.708	.513	1	.474	1.660
	[ag=3]	0^b	.	.	0	.	.
	[fy=1]	2.186	2.197	.989	1	.320	8.897
	[fy=2]	-.832	1.136	.536	1	.464	.435
	[fy=3]	.249	1.045	.057	1	.811	1.283
	[fy=4]	-1.581	1.127	1.967	1	.161	.206
	[fy=5]	.426	1.275	.111	1	.739	1.531
	[fy=6]	0^b	.	.	0	.	.
	[fvy=1]	-.917	1.697	.292	1	.589	.400
	[fvy=2]	.062	.983	.004	1	.950	1.064
	[fvy=3]	-1.344	1.201	1.254	1	.263	.261
	[fvy=4]	-1.077	.841	1.639	1	.200	.340
	[fvy=5]	0^b	.	.	0	.	.
	[CrackControalCheck=0]	-.062	1.699	.001	1	.971	.940
	[CrackControalCheck=1]	0^b	.	.	0	.	.
	[Anchorage=0]	8.736	.000	.	1	.	6225.297
	[Anchorage=1]	0^b	.	.	0	.	.
C	Intercept	2.684	14.315	.035	1	.851	
	fcMPa	-.001	.019	.003	1	.955	.999
	bw_dm	.702	.673	1.086	1	.297	2.017
	ds_dm	.421	.309	1.855	1	.173	1.524
	ad	-.328	1.106	.088	1	.767	.721
	sd	.235	.216	1.174	1	.279	1.264
	rhoshearlong.Steel	.415	.850	.239	1	.625	1.515
	rhostirrup	-.693	2.999	.053	1	.817	.500
	[CrossSection=1]	-.693	7.385	.009	1	.925	.500
	[CrossSection=2]	.209	2.799	.006	1	.940	1.233
	[CrossSection=3]	0^b	.	.	0	.	.
	[ag=2]	1.431	1.306	1.200	1	.273	4.181
	[ag=3]	0^b	.	.	0	.	.
	[fy=1]	.631	4.181	.023	1	.880	1.879
	[fy=2]	-.598	2.265	.070	1	.792	.550
	[fy=3]	.650	2.012	.105	1	.746	1.916
	[fy=4]	-.624	2.289	.074	1	.785	.536
	[fy=5]	-2.062	3.050	.457	1	.499	.127
	[fy=6]	0^b	.	.	0	.	.
	[fvy=1]	-1.869	7.090	.069	1	.792	.154
	[fvy=2]	.099	1.856	.003	1	.958	1.104
	[fvy=3]	-1.580	2.342	.455	1	.500	.206
	[fvy=4]	-1.086	1.864	.339	1	.560	.338
	[fvy=5]	0^b	.	.	0	.	.
	[CrackControalCheck=0]	-3.038	4.522	.452	1	.502	.048
	[CrackControalCheck=1]	0^b	.	.	0	.	.
	[Anchorage=0]	-9.005	13.482	.446	1	.504	.000
	[Anchorage=1]	0^b	.	.	0	.	.

a. The reference category is: B.

4.4 Australian Code (AS 3600)

According to the guide lines given in AS3600:

Total number of Slender Beams – 658

Slender beams without shear reinforcement – 517

Slender beams with shear reinforcement - 141

Total number of Deep beams – 24

4.4.1 Slender Beams without Shear Reinforcement

$$V_n = \beta_1\beta_2\beta_3 b_v d_0 \frac{A_{st} f_c^{'}}{b_v d_0} \tag{4.9}$$

Predicted shear strength (V_n) Vs the Tested shear capacity (V_t). and the descriptive statistics of V_t/V_n ratio has given in the table bellow.

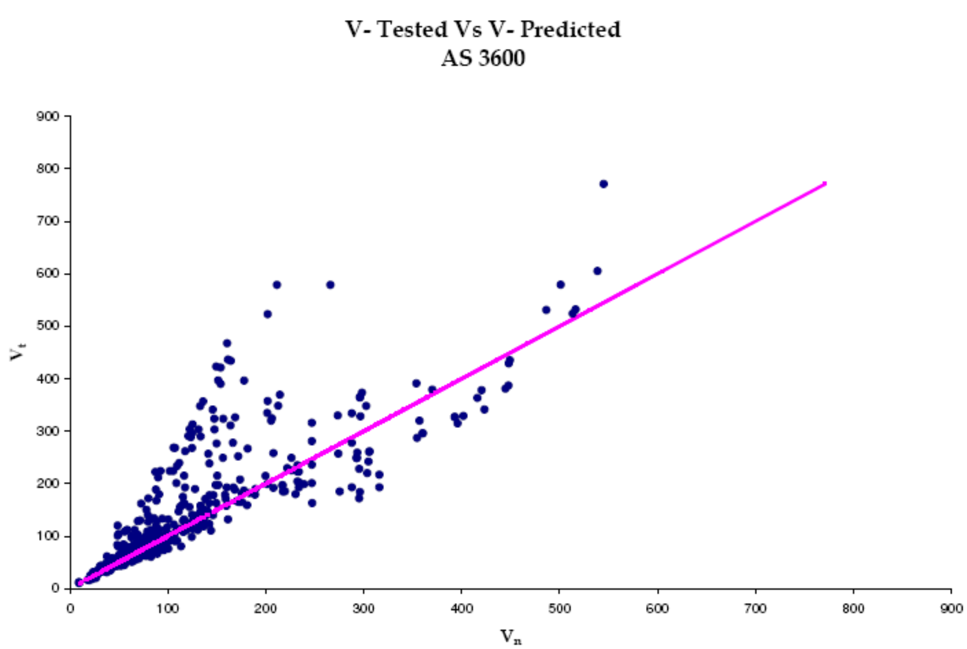


Figure 4.10
Tested Shear Strength V_t Vs Predicted Shear Strength V_n – AS 3600 - Beams without Shear r/f

Table 4.26
Descriptive statistics of V_t/V_n – AS 3600 - Beams without Shear r/f

	V_t/V_n
Mean	1.19
Standard Deviation	0.43
Range	2.33
Minimum	0.58
Maximum	2.91

Multinomial logistic regression results of this method is also given in Appendix B. Only the "Parameter Estimates" table has presented here.

Table 4.27
Parameter Estimates – AS 3600 Beams without Shear reinforcement

Y[a]		B	Std. Error	Wald	df	Sig.
A	Intercept	7.072	.881	64.395	1	.000
	fcMPa	-.026	.010	7.174	1	.007
	dsm	-.135	.065	4.368	1	.037
	ad	-1.956	.198	97.899	1	.000
	[CrossSectionalShape=2]	2.644	.442	35.728	1	.000
	[CrossSectionalShape=3]	0[b]	.	.	0	.
	[ag=1]	-1.320	.467	7.996	1	.005
	[ag=2]	-.115	.385	.090	1	.765
	[ag=3]	0[b]	.	.	0	.
	[CrackcontrolRF=0]	2.087	.895	5.433	1	.020
	[CrackcontrolRF=1]	0[b]	.	.	0	.
C	Intercept	-5.789	.825	49.234	1	.000
	fcMPa	.041	.008	27.076	1	.000
	dsm	.301	.063	22.671	1	.000
	ad	.498	.103	23.316	1	.000
	[CrossSectionalShape=2]	-1.861	.681	7.464	1	.006
	[CrossSectionalShape=3]	0[b]	.	.	0	.
	[ag=1]	.606	.526	1.326	1	.250
	[ag=2]	.467	.398	1.380	1	.240
	[ag=3]	0[b]	.	.	0	.
	[CrackcontrolRF=0]	-21.927	.000	.	1	.
	[CrackcontrolRF=1]	0[b]	.	.	0	.

a. The reference category is: B.

b. This parameter is set to zero because it is redundant.

Above tables indicates six numbers of parameters which effect the shear strength prediction from AS 3600. Namely they are: Compressive Capacity of Concrete (f’c), Effective depth of the beam (d), Effective depth to shear span, a/d ratio, Cross Sectional Shape, Maximum Aggregate size and Presence of Crack Control Reinforcement. The influence of four parameters can be clearly identified as:

1) ***Compression capacity of concrete*** **(f'_c) –AS 3600 tends to over estimate the shear strength of beams when compression capacity of concrete increases**
2) ***Effective depth*** **(d) – With the increase of the Effective depth AS 3600 tends to over predict the shear strength of a beam.**
2) ***Effective depth to shear span ratio*** **(a/d) – With the increase of the a/d ratio AS 3600 tends to over predict the shear strength of a beam.**
4) ***Cross Sectional shape*** **– It can be identified that the AS 3600 has a tendency of underestimating shear strength of T beams compare to rectangular beams.**

Estimating Critical Values for parameters affecting the Accuracy.

Estimation of critical values of parameters affecting the accuracy of this method is given below. Complete SPSS output of this process has been given in Appendix C.

1) AS 3600 tends to over predict the shear strength of a beam without shear reinforcement when Compression capacity of concrete, (f'_c) is approximately greater than 60 MPa
2) AS 3600 tends to over predict the shear strength of a beam without shear reinforcement Effective depth, d is approximately greater than 900 mm.

2) AS 3600 tends to over predict the shear strength of a beam without shear reinforcement when a/d ratio is approximately greater than 4.5

4.4.2 Slender Beams with Shear Reinforcement

$$V_n = \beta_1\beta_2\beta_3 b_v d_0 \left(\frac{A_{st} f_c^{'}}{b_v d_0}\right)^{1/3} + \frac{A_{sv} f_{sy.f} d_0}{s} \cot\theta_v \tag{4.10}$$

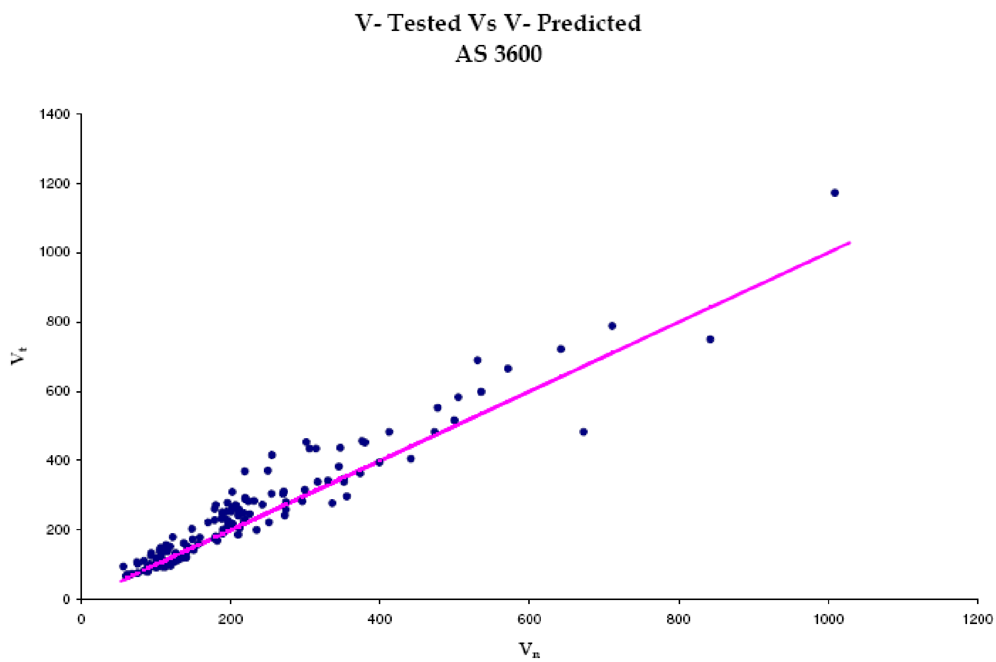


Figure 4.11
Tested Shear Strength V_t Vs Predicted Shear Strength V_n – AS 3600 - Beams with Shear r/f

Table 4.28
Descriptive statistics of V_t/V_n – AS 3600 - Beams with Shear r/f

	V_t/V_n
Mean	1.12
Standard Deviation	0.19
Range	0.97
Minimum	0.72
Maximum	1.69

The predicted shear strength from above equation (V_n) Vs the tested shear strength (V_t) given in Fig.4.11. Also descriptive statistics are given in Table 4.28.

The "Likelihood ratio Test" and "Parameter Estimates" tables of the SPSS output are also presented below. Complete SPSS output has been given in Appendix B.

Table 4.29
Likelihood Ratio Test – AS 3600 Beams with Shear reinforcement

	Model Fitting Criteria			Likelihood Ratio Tests		
Effect	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	232.785	356.633	1.488E2	.000	0	.
fcMPa	231.940	349.890	151.940	3.155	2	.206
ad	231.956	349.906	151.956	3.171	2	.205
bw_dm	231.241	349.191	151.241	2.456	2	.293
ds_dm	230.362	348.313	150.362	1.578	2	.454
sd	233.708	351.659	153.708	4.924	2	.085
rhoshearlong.Steel	228.875	346.825	148.875	.090	2	.956
rhostirrup	230.817	348.767	150.817	2.032	2	.362
ag	237.499	355.449	157.499	8.714	2	.013
fy	242.181	342.439	174.181	25.397	8	.001
Anchorage	238.335	356.286	158.335	9.551	2	.008
CrossSection	231.937	343.990	155.937	7.153	4	.128
fvy	230.444	330.702	162.444	13.660	8	.091
CrackControalCheck	229.061	347.011	149.061	.276	2	.871

From the results of multinomial logistic regression analysis it can be seen that Ratio of Stirrup spacing to Effective depth, s/d, Maximum Aggregate size, Yield strength of longitudinal tensile steel, Anchorage of longitudinal tensile steel are the parameters affecting the accuracy of the shear strength predictions of AS 3600 for beams with shear reinforcement. But unfortunately the effect of any of the above parameter could not be clearly identified.

Table 4.30. Parameter Estimates – AS 3600 Beams with Shear r/f

Y[a]		B	Std. Error	Wald	df	Sig.
A	Intercept	1.706	4.766	.128	1	.720
	fcMPa	.012	.017	.465	1	.495
	ad	-1.389	.947	2.152	1	.142
	bw_dm	.938	.707	1.758	1	.185
	ds_dm	-.237	.261	.826	1	.363
	sd	-.531	.256	4.316	1	.038
	rhoshearlong.Steel	-.142	.635	.050	1	.823
	rhostirrup	4.083	3.532	1.337	1	.248
	[ag=2]	3.684	1.593	5.346	1	.021
	[ag=3]	0[b]	.	.	0	.
	[fy=1]	15.139	5305.589	.000	1	.998
	[fy=2]	-4.900	1.767	7.688	1	.006
	[fy=3]	-3.923	2.250	3.039	1	.081
	[fy=4]	-4.842	2.457	3.883	1	.049
	[fy=5]	0[b]	.	.	0	.
	[Anchorage=0]	3.400	2.110	2.598	1	.107
	[Anchorage=1]	0[b]	.	.	0	.
	[CrossSection=1]	2.605	2.743	.902	1	.342
	[CrossSection=2]	21.782	6146.116	.000	1	.997
	[CrossSection=3]	0[b]	.	.	0	.
	[fvy=1]	2.615	2.051	1.626	1	.202
	[fvy=2]	3.424	1.708	4.019	1	.045
	[fvy=3]	-1.122	2.260	.247	1	.620
	[fvy=4]	-.211	1.142	.034	1	.854
	[fvy=5]	0[b]	.	.	0	.
	[CrackControalCheck=0]	-.071	1.826	.002	1	.969
	[CrackControalCheck=1]	0[b]	.	.	0	.
C	Intercept	-19.818	7.388	7.195	1	.007
	fcMPa	.038	.023	2.864	1	.091
	ad	.803	1.185	.459	1	.498
	bw_dm	.732	.788	.864	1	.353
	ds_dm	.263	.438	.360	1	.548
	sd	-.106	.302	.124	1	.725
	rhoshearlong.Steel	-.253	1.101	.053	1	.818
	rhostirrup	-3.473	6.067	.328	1	.567
	[ag=2]	.147	1.836	.006	1	.936
	[ag=3]	0[b]	.	.	0	.
	[fy=1]	16.655	9877.346	.000	1	.999
	[fy=2]	18.087	2.683	45.461	1	.000
	[fy=3]	18.007	2.023	79.217	1	.000
	[fy=4]	16.282	.000	.	1	.
	[fy=5]	0[b]	.	.	0	.
	[Anchorage=0]	-4.530	2.505	3.270	1	.071
	[Anchorage=1]	0[b]	.	.	0	.
	[CrossSection=1]	1.169	3644.457	.000	1	1.000
	[CrossSection=2]	.117	.000	.	1	.
	[CrossSection=3]	0[b]	.	.	0	.
	[fvy=1]	-15.633	3640.765	.000	1	.997
	[fvy=2]	-2.755	3.192	.745	1	.388
	[fvy=3]	-2.321	2.987	.604	1	.437
	[fvy=4]	.788	1.616	.238	1	.626
	[fvy=5]	0[b]	.	.	0	.
	[CrackControalCheck=0]	-16.817	.000	.	1	.
	[CrackControalCheck=1]	0[b]	.	.	0	.

a. The reference category is: B.

4.4.2 Deep Beams

According to the Australian Code there are only 22 number of deep beams in our database. Fig. 4.12 plots the predicted shear strength from Australian Code (V_n) Vs the tested shear strength (V_t). Also descriptive statistics have been presented in Table 4.31.

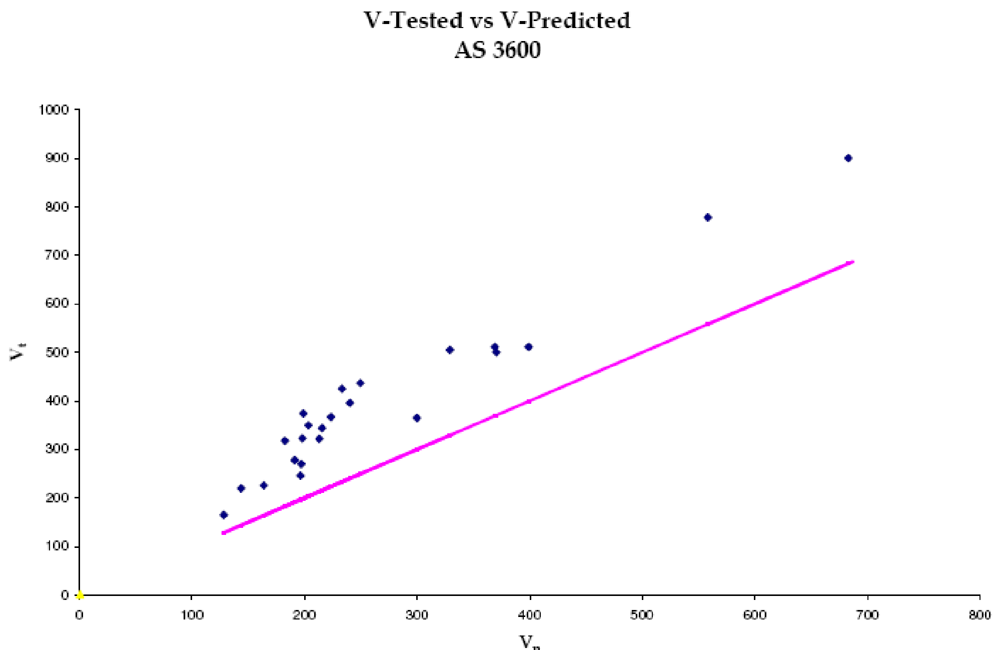


Figure 4.12
Tested Shear Strength V_t Vs Predicted Shear Strength V_n – AS 3600- Deep Beams

Table 4.31
Descriptive statistics of V_t/V_n – AS 3600 – Deep Beams

	V_t/V_n
Mean	1.51
Standard Deviatio	0.19
Range	0.66
Minimum	1.22
Maximum	1.88

It should be noted that all the points in Fig.4.11 has plotted above the perfect fit line and also the minimum value for the V_t/V_n is 1.22. This gives an indication of the conservativeness of Australian Code guidelines on deep beam design.

Unfortunately the multinomial logistic regression could not be used for this method due to the lack of data.

4.5 Japanese Code (JSCE SP-1)

According to the guide lines given in JSCE beams in the data base can be classified in to two categories and the number of beams in each category is given bellow:

Total number of Slender Beams – 698

Slender beams without shear reinforcement - 519

Slender beams with shear reinforcement - 179

Total number of Deep beams – 16

4.5.1 Slender Beams without Shear Reinforcement

$$V_n = 0.9\beta_d\beta_p\beta_n\sqrt[3]{f'_{cd}}\ b_w d \tag{4.11}$$

Predicted shear strength (V_n) Vs the Tested shear capacity (V_t). and the descriptive statistics of V_t/V_n ratio is given in the table below.

Complete results of Multinomial logistic regression of this method is also given in Appendix B. Only the "Parameter Estimates" table has been presented here (Table 4.33).

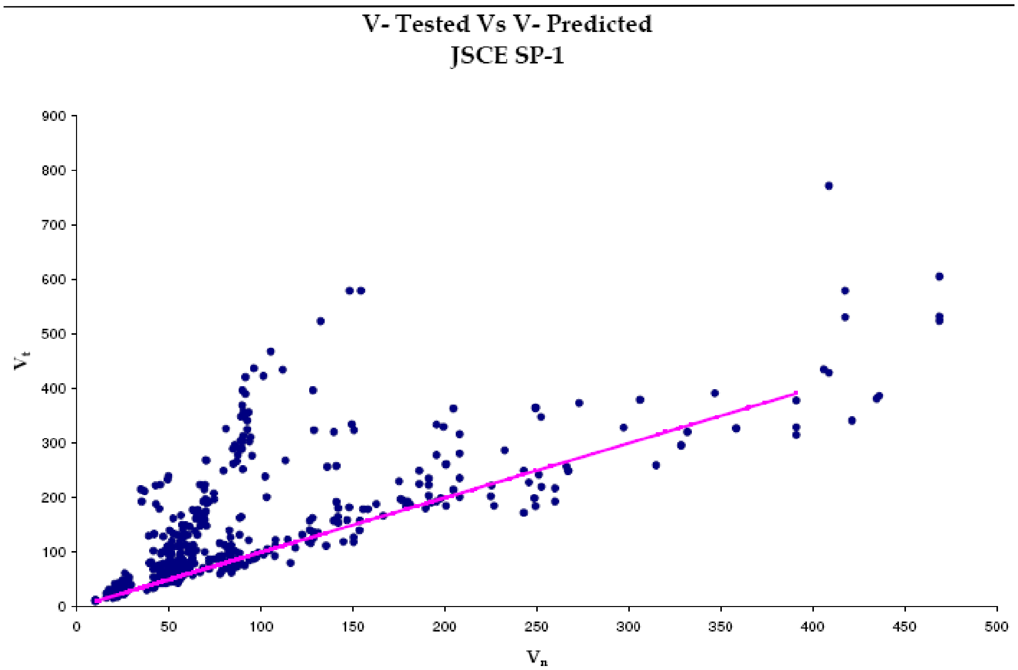


Figure 4.13

Tested Shear Strength V_t Vs Predicted Shear Strength V_n – JSCE-Beams without Shear r/f

Table 4.32

Descriptive statistics of V_t/V_n – JSCE- Beams without Shear r/f

	V_t/V_n
Mean	1.61
Standard Deviation	0.90
Range	5.46
Minimum	0.69
Maximum	6.15

Table 4.33

Parameter Estimates – JSCE beams without Shear reinforcement

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	-.551	1.176	.220	1	.639	
	ad	-.509	.096	27.898	1	.000	.601
	fcMPa	-.032	.008	17.462	1	.000	.969
	[CrackcontrolRF=0]	20.624	5387.605	.000	1	.997	9.059E8
	[CrackcontrolRF=1]	0^c	.	.	0	.	.
	[ag=1]	.866	.429	4.073	1	.044	2.377
	[ag=2]	-.100	.345	.084	1	.772	.905
	[ag=3]	0^c	.	.	0	.	.
	[CrossSectionalShape=2]	1.258	.430	8.548	1	.003	3.517
	[CrossSectionalShape=3]	0^c	.	.	0	.	.
	[fy=1]	2.371	1.159	4.184	1	.041	10.712
	[fy=2]	2.167	1.180	3.376	1	.066	8.736
	[fy=3]	1.895	1.107	2.929	1	.087	6.652
	[fy=4]	1.005	1.071	.881	1	.348	2.732
	[fy=5]	.366	1.057	.120	1	.729	1.442
	[fy=6]	0^c	.	.	0	.	.
	rhoshearlong.Steel	1.121	.155	52.146	1	.000	3.068
C	Intercept	-20.161	1.171	296.643	1	.000	
	ad	.329	.168	3.827	1	.050	1.38
	fcMPa	.014	.008	2.804	1	.094	1.01
	[CrackcontrolRF=0]	-.538	9250.617	.000	1	1.000	.58
	[CrackcontrolRF=1]	0^c	.	.	0	.	.
	[ag=1]	-.310	.811	.146	1	.702	.73
	[ag=2]	.486	.459	1.121	1	.290	1.62
	[ag=3]	0^c	.	.	0	.	.
	[CrossSectionalShape=2]	-1.350	.974	1.922	1	.166	.25
	[CrossSectionalShape=3]	0^c	.	.	0	.	.
	[fy=1]	17.912	.996	323.351	1	.000	60144671.49
	[fy=2]	18.017	.994	328.538	1	.000	66760558.84
	[fy=3]	16.846	.900	350.305	1	.000	20698226.65
	[fy=4]	16.855	.854	389.475	1	.000	20885420.12
	[fy=5]	16.246	.000	.	1	.	11364089.85
	[fy=6]	0^c	.	.	0	.	.
	rhoshearlong.Steel	-.060	.264	.051	1	.821	.94

a. The reference category is: B.

Seven numbers of parameters have been identified to effect the shear strength prediction from JSCE. Namely they are: Compressive Capacity of Concrete (f'_c), Presence of Crack Control Reinforcement, Maximum Aggregate size, Cross Sectional Shape, Yield strength of tensile reinforcement and Percentage Area of Tensile Reinforcement (ρ). The influence of only one parameter can be clearly identified as:

1) ***Effective depth to shear span ratio* (a/d) – With the increase of the a/d ratio JSCE SP-1 tends to over predict the shear strength of a beam.**

Estimating Critical Values for parameters affecting the Accuracy.

Estimation for critical values of a/d ratio affecting the accuracy of this method are given below. Complete output of this process is given in Appendix C.

1) JSCE SP-1 tends to over predict the shear strength of a beam without shear reinforcement when a/d ratio is approximately greater than 6.0

4.5.2 Slender Beams with Shear Reinforcement

$$V_n = 0.9\beta_d\beta_p\beta_n\sqrt[3]{f'_{cd}}\ b_w d + \frac{A_w f_{wyd}}{s_s} z \qquad (4.12)$$

The predicted shear strength from above equation (V_n) Vs the tested shear strength (V_t) are given in Fig.4.14. Also descriptive statistics have been presented in Table 4.34.

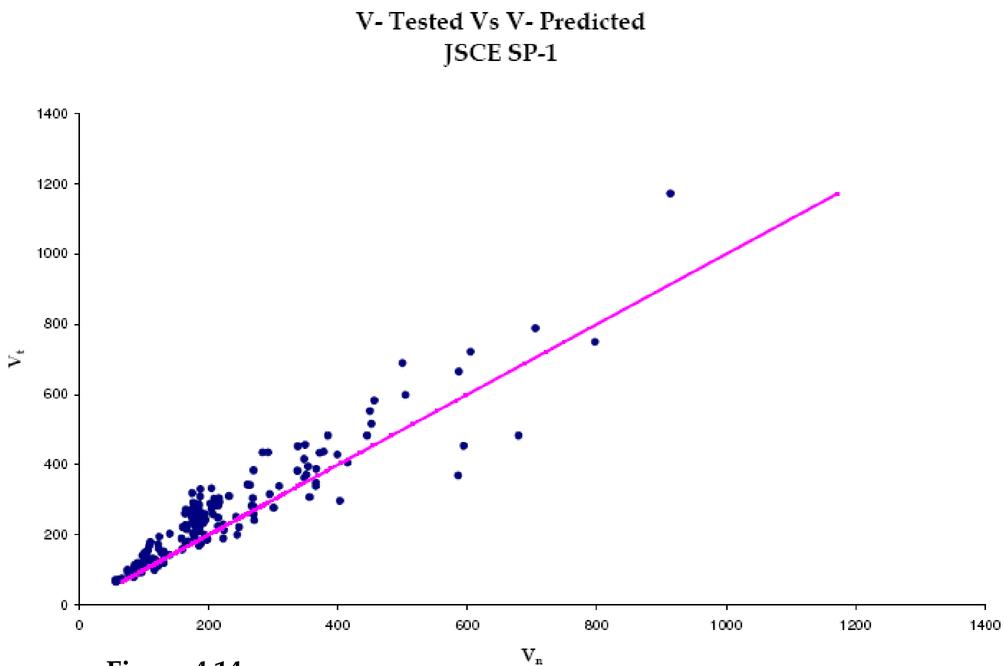


Figure 4.14
Tested Shear Strength V_t Vs Predicted Shear Strength V_n – JSCE-Beams with Shear r/f

Table 4.34
Descriptive statistics of V_t/V_n – JSCE - Beams with Shear r/f

	V_t/V_n
Mean	1.21
Standard Deviation	0.22
Range	1.19
Minimum	0.63
Maximum	1.82

Complete results of the Multinomial logistic regression using SPSS software is given in Appendix B. The "Likelihood Ratio Test" table and "Parameter Estimates" table of the SPSS output have been presented below.

Table 4.35

Likelihood Ratio Test – JSCE- Beams with Shear r/f

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	230.699	370.944	1.427E2	.000	0	.
rhostirrup	262.210	396.080	178.210	35.511	2	.000
rhoshearlong.Steel	254.231	388.101	170.231	27.532	2	.000
ad	239.225	373.095	155.225	12.526	2	.002
sd	243.707	377.577	159.707	17.008	2	.000
ds_dm	227.853	361.723	143.853	1.154	2	.562
bw_dm	228.969	362.839	144.969	2.270	2	.321
fcMPa	234.391	368.261	150.391	7.691	2	.021
CrackControalCheck	228.224	362.094	144.224	1.524	2	.467
Anchorage	234.949	368.819	150.949	8.250	2	.016
fvy	241.078	355.824	169.078	26.379	8	.001
fy	229.672	331.668	165.672	22.972	12	.028
ag	228.967	362.837	144.967	2.267	2	.322
CrossSection	233.958	367.828	149.958	7.259	2	.027

The multinomial logistic regression results shows eight parameter which affect the accuracy of the Japanese Code shear predictions of beams with shear reinforcement. But unfortunately their effect on the accuracy of the method can not be clearly identified. Parameters influencing the accuracy are : Concrete Grade, f'_c, Effective Depth, Percentage of Longitudinal Tensile Steel for Shear , ρ, Proximity of rigid support to point load, a/d, Yield Strength of Long. Tensile Steel, fy, Cross Sectional Shape and Anchorage of longitudinal Tensile Steel.

Table 4.36, Parameter Estimates – JSCE Beams with Shear r/f

Y^a		B	Std. Error	Wald	df	Sig.
A	Intercept	7.608	3.155	5.816	1	.016
	rhostirrup	-8.910	2.181	16.693	1	.000
	rhoshearlong.Steel	2.208	.585	14.243	1	.000
	ad	-2.198	.698	9.924	1	.002
	sd	-.823	.237	12.050	1	.001
	ds_dm	.237	.235	1.020	1	.312
	bw_dm	-.230	.418	.304	1	.582
	fcMPa	-.014	.015	.837	1	.360
	[CrackControalCheck=0]	17.007	4309.385	.000	1	.997
	[CrackControalCheck=1]	0^c	.	.	0	.
	[Anchorage=0]	.481	.759	.401	1	.527
	[Anchorage=1]	0^c	.	.	0	.
	[fvy=1]	-4.633	1.913	5.865	1	.015
	[fvy=2]	.331	1.259	.069	1	.792
	[fvy=3]	-2.655	1.639	2.626	1	.105
	[fvy=4]	.125	1.104	.013	1	.910
	[fvy=5]	0^c	.	.	0	.
	[fy=0]	19.059	5898.757	.000	1	.997
	[fy=1]	22.032	2030.892	.000	1	.991
	[fy=2]	-.642	1.508	.181	1	.671
	[fy=3]	.999	1.369	.532	1	.466
	[fy=4]	-1.894	1.568	1.458	1	.227
	[fy=5]	.791	1.670	.224	1	.636
	[fy=6]	0^c	.	.	0	.
	[ag=2]	1.145	.858	1.779	1	.182
	[ag=3]	0^c	.	.	0	.
	[CrossSection=1]	-6.738	3.886	3.007	1	.083
	[CrossSection=3]	0^c	.	.	0	.
C	Intercept	-16.653	1728.748	.000	1	.992
	rhostirrup	4.287	2.975	2.076	1	.150
	rhoshearlong.Steel	-2.443	1.819	1.805	1	.179
	ad	.483	1.830	.070	1	.792
	sd	-.316	.444	.506	1	.477
	ds_dm	-.168	.600	.079	1	.779
	bw_dm	2.079	1.688	1.517	1	.218
	fcMPa	.074	.039	3.610	1	.057
	[CrackControalCheck=0]	7.312	8092.570	.000	1	.999
	[CrackControalCheck=1]	0^c	.	.	0	.
	[Anchorage=0]	-4.558	2.396	3.619	1	.057
	[Anchorage=1]	0^c	.	.	0	.
	[fvy=1]	3.610	2410.746	.000	1	.999
	[fvy=2]	14.738	1728.730	.000	1	.993
	[fvy=3]	19.985	1728.732	.000	1	.991
	[fvy=4]	20.546	1728.732	.000	1	.991
	[fvy=5]	0^c	.	.	0	.
	[fy=0]	-1.103	.000	.	1	.
	[fy=1]	3.424	5299.096	.000	1	.999
	[fy=2]	-3.902	3.269	1.425	1	.233
	[fy=3]	-8.390	5.227	2.576	1	.108
	[fy=4]	-3.220	2.519	1.634	1	.201
	[fy=5]	-22.545	3161.814	.000	1	.994
	[fy=6]	0^c	.	.	0	.
	[ag=2]	-1.578	3.202	.243	1	.622
	[ag=3]	0^c	.	.	0	.
	[CrossSection=1]	15.465	10.379	2.220	1	.136
	[CrossSection=3]	0^c	.	.	0	.

a. The reference category is: B.

4.5.2 Deep Beams

According to the Japanese Code there are only 16 number of deep beams in our database. Fig. 4.15 plots the predicted shear strength from Australian Code (V_n) Vs the tested shear strength (V_t). Also descriptive statistics have been used in Table 4.37.

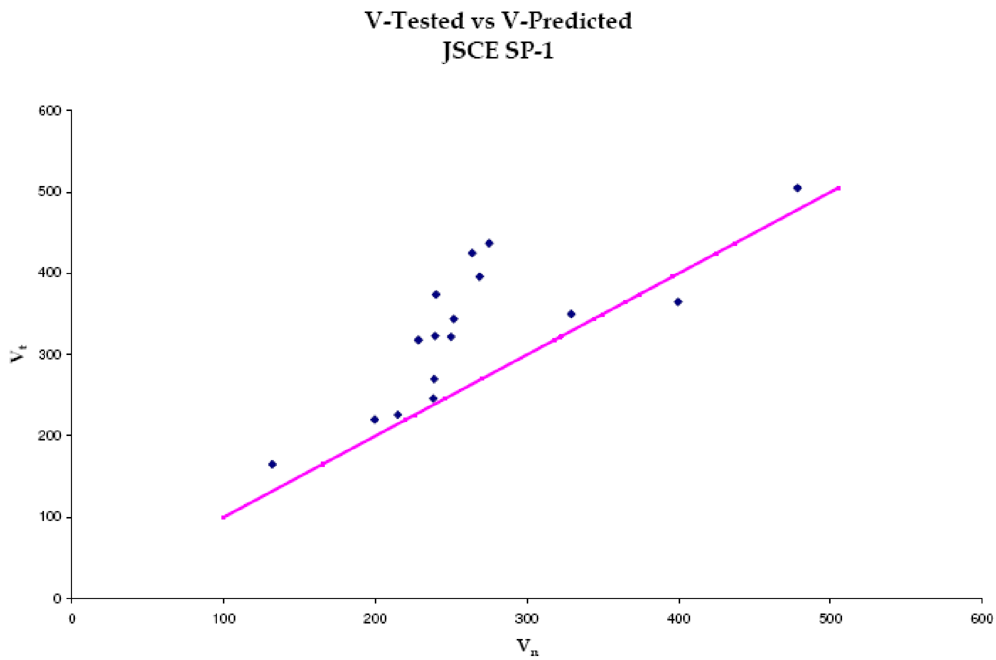


Figure 4.15
Tested Shear Strength V_t Vs Predicted Shear Strength V_n – JSCE- Deep Beams

Table 4.37
Descriptive statistics of V_t/V_n – JSCE – Deep Beams

	V_t/V_n
Mean	1.26
Standard Deviation	0.22
Range	0.70
Minimum	0.91
Maximum	1.61

Unfortunately due to the lack of data multinomial logistic regression could not be used for this method for the identification of the parameters affecting the accuracy of this method.

4.6 Canadian Code (CSA A23.3)

According to the guide lines CSA A23.1:

Total number of Slender Beams – 507

Slender beams without shear reinforcement – 381

Slender beams with shear reinforcement - 126

Total number of Deep beams – 75

4.6.1 Slender Beams without Shear Reinforcement

4.6.1.1 Simplified Method

when d $\leq$ 300mm

$$V_c = 0.167\sqrt{f_c^{'}}b_w d \tag{4.13a}$$

d > 300mm

$$V_c = \left(\frac{260}{1000+d}\right)\sqrt{f_c^{'}}b_w d \tag{4.13b}$$

Predicted shear strength using above equations (V_n) Vs the tested shear strength (V_t) given in Fig.4.16. Also descriptive statistics have been presented in Table 4.38.

Complete results of multinomial logistic regression analysis using SPSS software is given in Appendix B and only the "Parameter Estimates" table has been presented here (Table 4.30).

Table 4.38
Descriptive statistics of V_t/V_n – CSA A23.3 - Simplified Method - Beams with Shear r/f

	V_t/V_n
Mean	1.48
Standard Deviation	0.61
Range	3.88
Minimum	0.45
Maximum	4.33

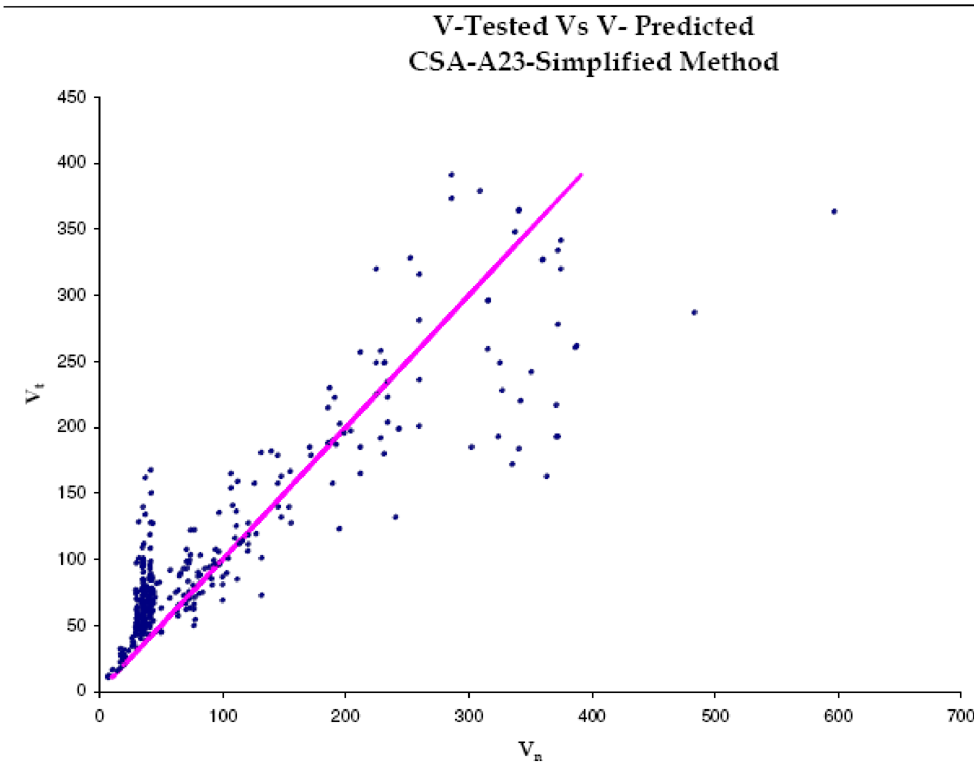


Figure 4.16
Tested Shear Strength Vt Vs Predicted Shear Strength Vn – CSA A23.3 - Simplified Method – Beams without Shear r/f

Six parameters have been identified to influence the accuracy of this method. Namely they are: Compressive Capacity of Concrete (f'_c), Effective depth to shear span, a/d ratio, Percentage Area of Tensile Reinforcement (ρ), Cross Sectional Shape, Maximum Aggregate size and Presence of Crack Control Reinforcement. The influence of only three parameters can be clearly identified as:

1) ***Compression capacity of concrete*** **(f'_c) – CSA – A23.3 Simplified Method tends to over estimate the shear strength of beams when compression capacity of concrete increases**
2) ***Percentage Area of Tensile Reinforcement ρ*** **– CSA A23.3 tends to under predict the shear strength of beams when percentage of longitudinal tensile steel increases.**

3) ***Crack control reinforcement* – It can be identified that the CSA - A23.3 Simplified method has underestimated the influence of crack control reinforcement on shear strength of a beam.**

Table 4.39, Parameter Estimates – CSA A23.3 - Simplified Method- Beams without Shear r/f

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	1.254	.961	1.703	1	.192	
	fcMPa	-.077	.015	27.389	1	.000	.926
	ad	-.820	.269	9.308	1	.002	.440
	rhoshearlong.Steel	2.754	.437	39.720	1	.000	15.707
	[CrossSectionalShape=2]	3.330	1.295	6.607	1	.010	27.940
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	.210	.598	.123	1	.725	1.234
	[ag=2]	.246	.457	.290	1	.590	1.279
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	2.626	.982	7.152	1	.007	13.819
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
C	Intercept	.533	1.319	.163	1	.686	
	fcMPa	.066	.015	19.106	1	.000	1.068
	ad	.198	.381	.270	1	.604	1.219
	rhoshearlong.Steel	-3.559	.770	21.345	1	.000	.028
	[CrossSectionalShape=2]	-14.682	.000	.	1	.	4.204E-7
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	.682	.741	.847	1	.357	1.978
	[ag=2]	-1.657	.676	6.009	1	.014	.191
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	-2.501	1.186	4.445	1	.035	.082
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.

a. The reference category is: B.

Estimating Critical Values for parameters affecting the Accuracy.

Estimation for critical values of a/d ratio affecting the accuracy of this method has been given below. Complete SPSS output of this process is given in Appendix C.

1) CSA A23.3 tends to over predict the shear strength of a beam without shear reinforcement when Compression capacity of concrete, (f'_c) is approximately greater than 35 MPa
2) CSA A23.3 tends to under predict the shear strength of a beam without shear reinforcement when Percentage Area of Tensile Reinforcement, ρ is approximately greater than 1.25%.

4.6.1.2 General Method

$$V_n = \beta\sqrt{f_c'}\,b_w d_v \tag{4.14}$$

Predicted shear strength using above equations (V_n) Vs the tested shear strength (V_t) are given in Fig.4.17. Also descriptive statistics have been presented in Table 4.40.

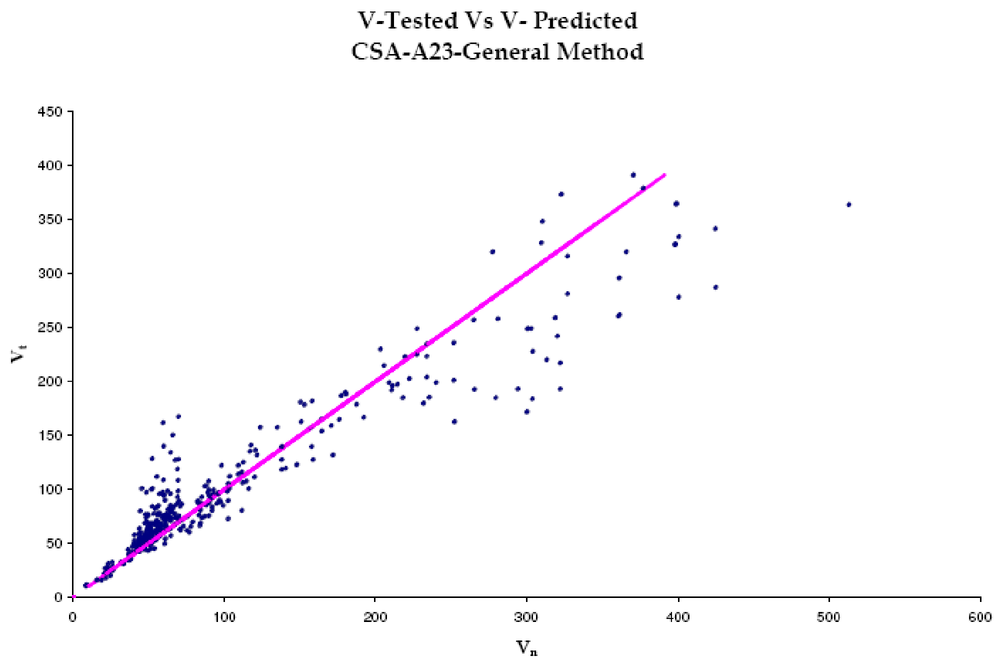


Figure 4.17
Tested Shear Strength V_t Vs Predicted Shear Strength V_n – CSA A23.3 - General Method- Beams without Shear r/f

Table 4.40
Descriptive statistics of V_t/V_n – CSA A23.3 - General Method- Beams without Shear r/f

	V_t/V_n
Mean	1.12
Standard Deviation	0.29
Range	2.15
Minimum	0.57
Maximum	2.72

Complete results of multinomial logistic regression analysis using SPSS software is given in Appendix B and only the "Parameter Estimates" table has been presented here (Table 4.30).

Table 4.41, Parameter Estimates – CSA A23.3 - General Method-Beams without Shear r/f

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	3.286	.757	18.853	1	.000	
	fcMPa	-.033	.011	9.601	1	.002	.967
	ds_dm	-.348	.112	9.601	1	.002	.706
	ad	-.776	.130	35.453	1	.000	.460
	rhoshearlong.Steel	.881	.168	27.521	1	.000	2.412
	[CrossSectionalShape=2]	2.496	.510	23.993	1	.000	12.132
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	-1.087	.533	4.167	1	.041	.337
	[ag=2]	-.953	.419	5.185	1	.023	.386
A	[ag=3]	0[b]	.	.	0	.	.
C	Intercept	-5.527	1.142	23.429	1	.000	
	fcMPa	.063	.011	35.599	1	.000	1.065
	ds_dm	.205	.077	6.979	1	.008	1.227
	ad	.638	.216	8.704	1	.003	1.894
	rhoshearlong.Steel	-1.088	.277	15.373	1	.000	.337
	[CrossSectionalShape=2]	-1.421	1.360	1.092	1	.296	.241
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	1.788	.600	8.885	1	.003	5.979
	[ag=2]	.170	.501	.115	1	.735	1.185
	[ag=3]	0[b]	.	.	0	.	.

a. The reference category is: B.

Six parameters have been identified to influence the accuracy of this method, namely: Compressive Capacity of Concrete (f'c), Effective depth, d, Effective depth to shear span, a/d ratio, Percentage Area of Tensile Reinforcement (ρ), Cross Sectional Shape, Maximum Aggregate size and The influence of four parameters that can be clearly identified as:

1) ***Compression capacity of concrete* (f'_c) – CSA A23.3 General Method tends to over estimate the shear strength of beams when compression capacity of concrete increases**
2) ***Effective Depth (d)* – CSA A23.3 General Method tends to over predict the shear strength of beams when Effective depth increases.**
3) ***Effective depth to shear span ratio (a/d)* – CSA A23.3 General Method tends to over predict the shear strength of beams when a/d ratio increases.**
4) ***Percentage Area of Tensile Reinforcement ρ* – CSA A23.3 General Method tends to under predict the shear strength of beams when percentage of longitudinal tensile steel increases.**

Estimating Critical Values for parameters affecting the Accuracy.

Estimation for critical values of a/d ratio affecting the accuracy of this method has been given below. Complete SPSS output of this process is given in Appendix C.

1) CSA A23.3 General Method tends to over predict the shear strength of a beam without shear reinforcement when Compression capacity of concrete, (f'_c) is approximately greater than 50 MPa.
2) CSA A23.3 General Method tends to over predict the shear strength of a beam without shear reinforcement when Effective Depth (d) is approximately greater than 600mm.
3) CSA A23.3 General Method tends to over predict the shear strength of a beam without shear reinforcement when a/d ratio is approximately greater than 4.0.

4) CSA A23.3 General Method tends to under predict the shear strength of a beam without shear reinforcement when Percentage Area of Tensile Reinforcement, ρ is approximately greater than 1.75%.

4.6.2 Slender Beams with Shear Reinforcement

4.6.2.1 Simplified Method

$$V_n = 0.167\sqrt{f_c^{'}}b_w d + \frac{A_v f_y d}{s} \tag{4.15}$$

The predicted shear strength from above equation (V_n) Vs the tested shear strength (V_t) given in Fig.4.18. Also descriptive statistics have been included in Table 4.42.

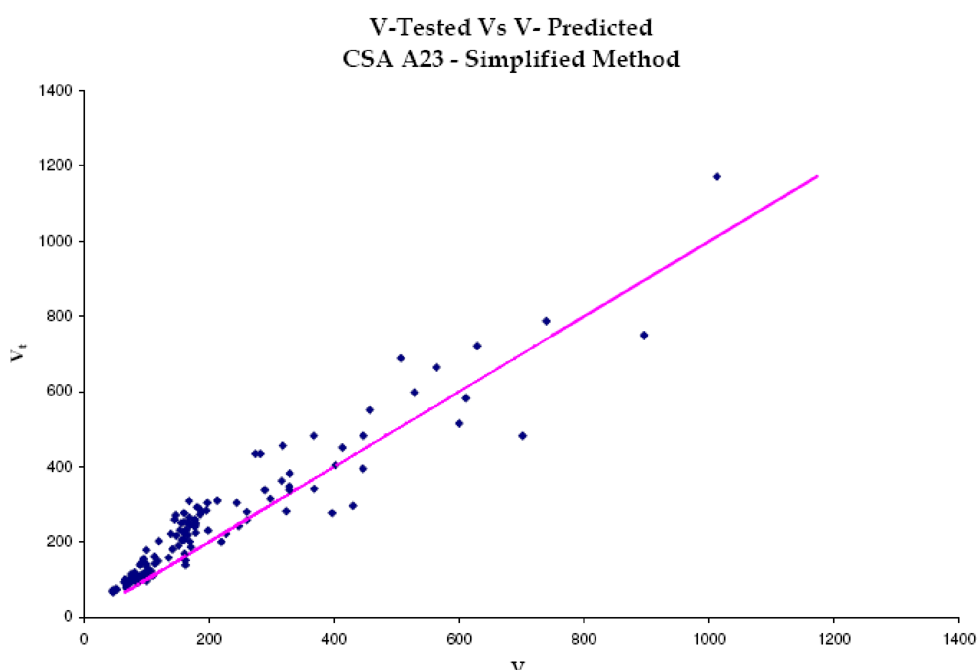


Figure 4.18
Tested Shear Strength V_t Vs Predicted Shear Strength V_n – CSA A23.3 - Simplified Method - Beams with Shear r/f

Table 4.42
Descriptive statistics of V_t/V_n – CSA A23.3 - Simplified Method- Beams with Shear r/f

	V_t/V_n
Mean	1.29
Standard Deviation	0.25
Range	1.17
Minimum	0.69
Maximum	1.86

Complete results of the Multinomial logistic regression using SPSS software is given in Appendix B. The "Likelihood Ratio Test" table and the "Parameter Estimates" table of the SPSS output have been presented below.

Table 4.43
Likelihood Ratio Test – CSA A23.3 - Beams wit Shear r/f

	Model Fitting Criteria			Likelihood Ratio Tests		
Effect	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	106.725	135.088	86.725[a]	.000	0	.
fcMPa	118.512	141.202	102.512	15.787	2	.000
CrackControalCheck	109.662	132.352	93.662	6.937	2	.031
ad	114.600	137.291	98.600	11.876	2	.003
rhoshearlong.Steel	166.433	189.123	150.433	63.708	2	.000

Table 4.44, Parameter Estimates – CSA A23.3 - Simplified Method-Beams with Shear r/f

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	2.879	1.878	2.349	1	.125	
	fcMPa	-.035	.015	5.314	1	.021	.965
	[CrackControalCheck=0]	-15.230	4438.502	.000	1	.997	2.431E-7
	[CrackControalCheck=1]	0[c]	.	.	0	.	.
	ad	-2.239	.695	10.363	1	.001	.107
	rhoshearlong.Steel	2.990	.712	17.633	1	.000	19.879
C	Intercept	13.359	14.286	.875	1	.350	
	fcMPa	.057	.027	4.513	1	.034	1.059
	[CrackControalCheck=0]	-23.411	.000	.	1	.	6.804E-11
	[CrackControalCheck=1]	0[c]	.	.	0	.	.
	ad	-4.418	4.911	.809	1	.368	.012
	rhoshearlong.Steel	-3.053	1.337	5.217	1	.022	.047

a. The reference category is: B.

Four parameters can be identified to affect the accuracy of the general method for beams with shear reinforcement. They are: Compressive Capacity of Concrete (f'_c), Effective depth to shear span, a/d ratio, Percentage Area of Tensile Reinforcement (ρ) and Presence of Crack Control Reinforcement. The influence of Compressive Capacity of Concrete can be clearly identified as:

1) When of Compressive Capacity of Concrete increases Canadian Simplified method tends to over predict the shear strength of beams with shear reinforcement.

4.6.2.2 General Method

$$V_n = \beta\sqrt{f'_c}\,b_w d_v + \frac{A_v f_s d_v}{s}\cot\theta \qquad (4.16)$$

The predicted shear strength from above equation (V_n) Vs the tested shear strength (V_t) are given in Fig.4.19. Also descriptive statistics have been used in Table 4.45.

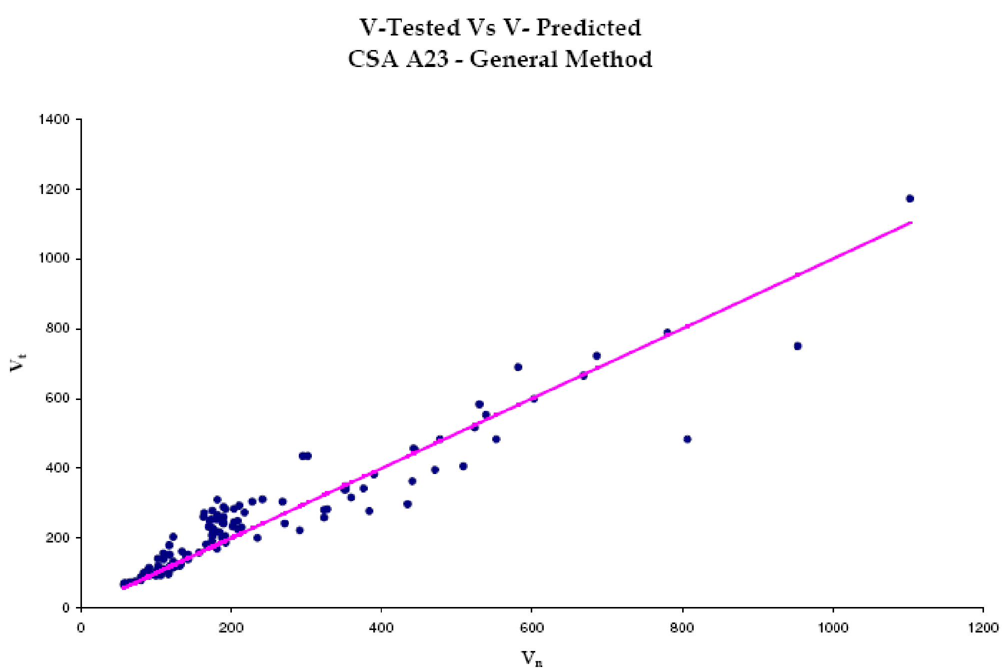


Figure 4.19
Tested Shear Strength Vt Vs Predicted Shear Strength Vn – CSA A23.3 - General Method - Beams with Shear r/f

Table 4.45
Descriptive statistics of V_t/V_n – CSA A23.3 - General Method- Beams with Shear r/f

	V_t/V_n
Mean	1.12
Standard Deviation	0.22
Range	1.10
Minimum	0.60
Maximum	1.70

Complete results of the Multinomial logistic regression using SPSS software is given in Appendix B. The "Likelihood Ratio Test" table and "Parameter Estimates" table of the SPSS output for Stepwise Logistic Regression are presented below.

Table 4.46
Likelihood Ratio Test – CSA A23.3 - General Method Beams with Shear r/f

	Model Fitting Criteria			Likelihood Ratio Tests		
Effect	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	174.655	248.398	1.227E2	.000	0	.
rhostirrup	182.396	250.467	134.396	11.741	2	.003
sd	183.141	251.212	135.141	12.486	2	.002
ds_dm	209.815	277.885	161.815	39.160	2	.000
fy	193.022	244.075	157.022	34.367	8	.000
fvy	181.000	232.053	145.000	22.345	8	.004
ad	178.917	246.988	130.917	8.262	2	.016

Four parameters have been identified to influence the accuracy of this method namely: Effective depth, d, Stirrup spacing to Effective depth, s/d ratio, Percentage Area of Shear Reinforcement (ρ_v), Yield strength of longitudinal tensile steel), Yield strength of Shear steel and Shear Span to

Effective depth, a/d ratio. Influence of Percentage Area of Shear Reinforcement (ρ_v), can be clearly identified from this study. Interestingly, the estimated values for regression coefficients in logistic regression Model A and Model B have become a negative value for this parameter. This implies when the Percentage Area of Shear Reinforcement increases the probability of shear strength being under predicted as well as the probability of shear strength being over predicted decreases. So we can conclude:

> ***1) Percentage Area of Shear Reinforcement ρ_v*** **– CSA A23.3 General Method tends to accurately predict the shear strength of beams when percentage of longitudinal tensile steel increases.**

Table 4.47, Parameter Estimates – CSA A23.3 - General Method-Beams with Shear r/f

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	41.540	124.827	.111	1	.739	
	rhostirrup	-8.216	3.506	5.493	1	.019	.000
	sd	-.786	.282	7.761	1	.005	.455
	ds_dm	-3.435	1.043	10.850	1	.001	.032
	[fy=1]	7.139	6484.825	.000	1	.999	1260.071
	[fy=2]	-11.317	4.141	7.470	1	.006	1.217E-5
	[fy=3]	-16.780	124.338	.018	1	.893	5.159E-8
	[fy=4]	-19.774	124.351	.025	1	.874	2.584E-9
	[fy=5]	0[c]	.	.	0	.	.
	[fvy=1]	3.977	1.598	6.198	1	.013	53.369
	[fvy=2]	.107	.961	.012	1	.912	1.112
	[fvy=3]	-9.475	124.216	.006	1	.939	7.673E-5
	[fvy=4]	-1.537	1.039	2.190	1	.139	.215
	[fvy=5]	0[c]	.	.	0	.	.
	ad	-2.051	.871	5.541	1	.019	.129
C	Intercept	-6.119	9.389	.425	1	.515	
	rhostirrup	-23.306	10.900	4.572	1	.032	7.556E-11
	sd	.034	.306	.012	1	.912	1.034
	ds_dm	.925	.668	1.919	1	.166	2.523
	[fy=1]	5.469	.000	.	1	.	237.292
	[fy=2]	2.135	1.548	1.902	1	.168	8.454
	[fy=3]	.257	1.295	.039	1	.843	1.293
	[fy=4]	6.518	5.193	1.575	1	.209	676.939
	[fy=5]	0[c]	.	.	0	.	.
	[fvy=1]	-11.979	6378.725	.000	1	.999	6.275E-6
	[fvy=2]	-1.960	1.810	1.173	1	.279	.141
	[fvy=3]	2.596	3.277	.628	1	.428	13.409
	[fvy=4]	-.792	1.532	.267	1	.605	.453
	[fvy=5]	0[c]	.	.	0	.	.
	ad	-.202	.815	.061	1	.804	.817

a. The reference category is: B.

4.6.3 Deep Beams

According to the Canadian Code guidelines 62 beams could be categorised as deep beams. The predicted shear strength by the Canadian Code Vs the Tested shear strength is plotted below in Fig.4.20.

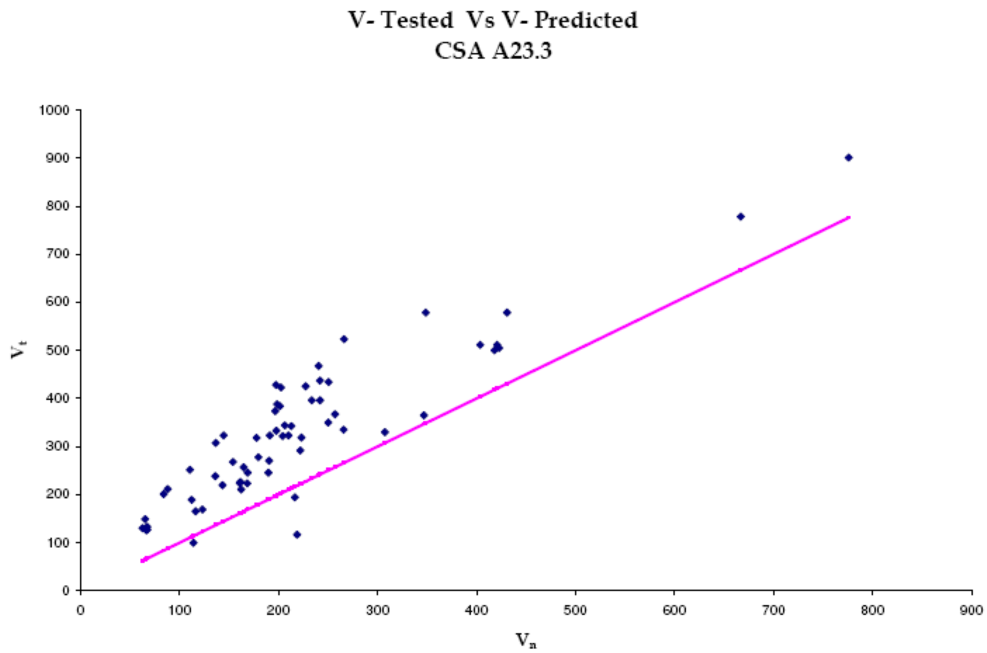


Figure 4.20
Tested Shear Strength V_t Vs Predicted Shear Strength V_n – CSA A23.3-Deep Beams

Table 4.48
Descriptive statistics of V_t/V_n – CSA A23.3 - Deep Beam

	V_t/V_n
Mean	1.61
Standard Deviation	0.39
Range	1.87
Minimum	0.54
Maximum	2.40

The complete multinomial logistic regression results for this model has been given in Appendix B and only the "Case Processing Summary" and "Model Fitting Information" tables have been presented here.

Table 4.49, Model Fitting Information – CSA A23.3 - Deep Beams

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	45.493	49.747	41.493			
Final	58.509	109.561	10.509	30.983	22	.096

As we can see from the Case Processing Summary table more than 90% of the predictions have fallen into the category "A" of the dependent variable. Only two and three cases are there in category "B" and "C" respectively. This type of situations weakens the logistic regression model. Therefore the non-significant value in the Model Fitting Information table above indicates that our regression model can not be considered as a good model. Therefore we can not identify parameters affecting the accuracy of the method. But it can be seen that more than 90% of predictions are in the conservative side.

Table 4.50, Case Processing Summary – CSA A23.3 – Deep Beams

		N	Marginal Percentage
Y	A	57	91.9%
	B	2	3.2%
	C	3	4.8%
Cross Sectional Shape	0	43	69.4%
	1	8	12.9%
	2	7	11.3%
	3	4	6.5%
ag	0	4	6.5%
	2	31	50.0%
	3	27	43.5%
Anchorage	0	20	32.3%
	1	42	67.7%
Overall Crack Control Check	1	62	100.0%
fy	2	15	24.2%
	3	27	43.5%
	4	16	25.8%
	6	4	6.5%
Valid		62	100.0%
Missing		0	
Total		62	
Subpopulation		61[a]	

4.6 Shear Friction

According to the guide lines Proposed by Loov (1998):

Total number of Slender Beams – 507

Slender beams without shear reinforcement – 381

Slender beams with shear reinforcement - 126

4.6.1 Slender Beams without Shear Reinforcement

$$V_n = V_{45}\frac{h}{a_{cl}} + \frac{M_{cr}}{a_{cl}} \tag{4.17}$$

Predicted shear strength using above equations (V_n) Vs the tested shear strength (V_t) given in Fig.4.21. Also descriptive statistics have been in Table 4.51.

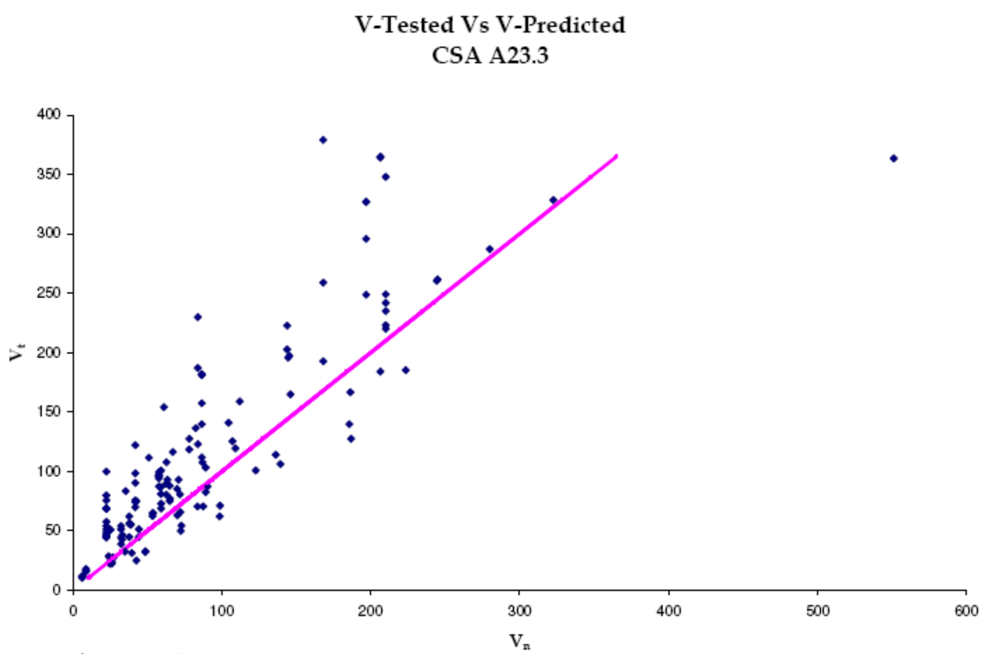


Figure 4.21
Tested Shear Strength V_t Vs Predicted Shear Strength V_n – Shear Friction method-Beams without Shear r/f

Table 4.51
Descriptive statistics of V_t/V_n – Shear Friction Method - Beams without Shear r/f

	V_t/V_n
Mean	1.53
Standard Deviation	0.64
Range	3.84
Minimum	0.59
Maximum	4.43

Complete results of multinomial logistic regression analysis using SPSS software is given in Appendix B and only the "Parameter Estimates" table has been presented here (Table 4.42).

Table 4.52, Parameter Estimates – Shear Friction Method - Beams without Shear r/f

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	-4.235	1.274	11.055	1	.001	
	rhoshearlong.Steel	2.122	.624	11.555	1	.001	8.346
	fcMPa	.072	.025	8.145	1	.004	1.075
C	Intercept	2.972	1.313	5.127	1	.024	
	rhoshearlong.Steel	-2.704	1.049	6.639	1	.010	.067
	fcMPa	-.003	.033	.008	1	.928	.997

a. The reference category is: B.

Two parameters have been identified to influence the accuracy of this method. Namely they are: Area of Tensile Reinforcement (ρ) and Compressive Capacity of Concrete (f'_c). The influence of one parameter can be clearly identified as:

1) ***Percentage Area of Tensile Reinforcement ρ* – Shear Friction Method tends to under predict the shear strength of beams when percentage of longitudinal tensile steel increases.**

Estimating Critical Values for parameters affecting the Accuracy.

Estimation for critical values of a/d ratio affecting the accuracy of this method is given below. Complete SPSS output of this process is given in Appendix C.

1) Shear Friction Method tends to over predict the shear strength of a beam without shear reinforcement when Area of Tensile Reinforcement (ρ) is approximately greater than 1.5%.

4.7.2 Slender Beams with Shear Reinforcement

$$V_n = k\left(CT_v \frac{d_s}{s}\right)^{1/2} - T_v \tag{4.18}$$

Predicted shear strength using above equations (V_n) Vs the tested shear strength (V_t) given in Fig.4.20. Also descriptive statistics have been in Table 4.43.

Table 4.53
Descriptive statistics of V_t/V_n – Shear Friction Method - Beams with Shear r/f

	V_t/V_n
Mean	3.08
Standard Deviation	1.12
Range	4.61
Minimum	0.81
Maximum	5.43

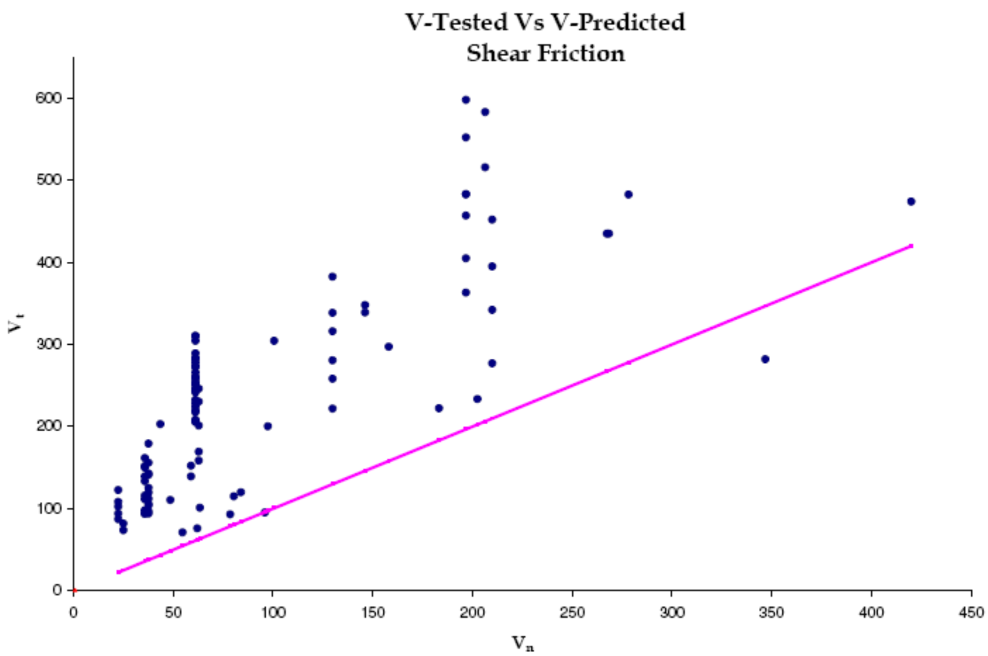


Figure 4.22
Tested Shear Strength V_t Vs Predicted Shear Strength V_n – Shear Friction method - Beams with Shear r/f

As we can see in the Figure above this method tends to give excessively conservative shear predictions. As there are very few data points in the category B and C of the dependent variable, it is not possible to do a multinomial logistic regression analysis.

4.8 Results of Industrial Survey

Evaluation of Shear Design Procedures Adopted in Industry for Reinforced Concrete Design

QUESTIONNAIRE

1	What are the Shear Design methods you usually use? BS Code Method-100% Australian Code Method-4% Japanese Code Method-0% Euro Code Method-7% ACI Code Method-0% Canadian Code Method-0% Other ____
2	How comfortable are you in using usual code equations, tables etc? Easy process-43% Moderate process-56% Tedious process-1%
3	What is the most convenient code you use in the design of reinforced concrete beams for shear? BS 8110 (100%) ____
4	How often do you come across situations in which shear design is governed by the limits imposed by the codes? Frequently-51% Rarely-37% Never-12%
5	How often do you feel design guidelines are over conservative? Frequently-27% Rarely-57% Never-16%
6	How often do you feel design guidelines are not adequate? Frequently-7% Rarely-49% Never-44%
7	How have you overcome these situations of lack of sufficient guidelines? Go for Expertise-50% Refer Hand Books-69% Refer Research Papers-7% Any other method – All three methods above-3%, Internet search- 3%, ___
8	How often do you use research information in design office practice? Frequently-4% Rarely-75% Never-21%
9	Do current codes of practice provide sufficient guidelines to understand the shear behaviour as opposed to design? Yes-24% To a certain Extent-64% No-12%
10	What are the structural elements you often design which require special attention on shear designing? Beams-81% Slabs-26% Deep Beams-32% Footings-60% Walls-8% Columns-8% Any other elements ____________________________________

Figure 4.23 a
Results of Industrial - Survey Page 1

Evaluation of Shear Design Procedures Adopted in Industry for Reinforced Concrete Design

QUESTIONNAIRE

11	If you use several Codes for shear designing what is the basis for selecting a particular design method for shear calculations? Based on the Element-28% Based on the Parameters- 22% Use only one method-56% Any other Basis-3% Please give a brief explanation about your basis _________________ ____ ____
12	In your opinion what are the parameters that would influence shear design? Concrete Grade-74% Depth of the Beam-90% Amount of Flexural Tensile Steel-62% Spacing of stirrups-53% Area of Shear steel-56% Width of the Beam-74% Aggregate size-1% Yield Strength of Flexural Tensile Steel-24% Any other parameters _______________________________________ ____
13	Out of the following list have you heard a procedure outside the code? Truss model-28% Compression Field Theory-6% Shear Friction -3% Modified Compression Field Theory-0% None-72%
14	Is shear design a critical problem in structural design practice for members without shear links? Yes-69% No-31%
15	Is shear design a critical problem in structural design practice for members with shear links? Yes-38% No-62%
16	Are we designing for shear or avoiding it by elimination of shear failures prior to flexural failure? Designing for Shear-64% Avoiding Shear Failures-36%
17	Have you experienced any shear failure in your career? Yes-25% No-75%
18	How often would you end up with problems related to shear being unresolved? Frequently-1% Rarely-58% Never-40%

Figure 4.23 b
Results of Industrial - Survey Page 2

4.9 Discussion

Beams without Shear Reinforcement

There are four major parameters affecting the shear strength of reinforced concrete beams; Longitudinal Reinforcement Ratio, ρ; Tensile Strength of Concrete; Shear Span to Depth Ratio, a/d ; and Depth of the Beam. Most of the design methods have identified these parameters as key parameters governing the shear strength of a reinforced concrete beam. But unfortunately, this study shows that still these parameters significantly affect the accuracy of the shear strength predictions, indicating the inability of design methods to properly identify the influence of these parameters on the shear strength. Apart from that several other parameters have been identified to influence the accuracy of the shear strength predictions namely: Maximum Aggregate size, Presence of crack control reinforcement: Cross sectional shape and Yield strength of longitudinal tensile steel. Influence of these parameters on the accuracy of shear strength predictions of each design method are discussed below.

First, the Area Ratio of Longitudinal Tensile Reinforcement has a pronounced effect on the basic shear transfer mechanisms. An important factor that affects the rate at which a flexural crack develops into an inclined one is the magnitude of the shear stress near the top of the crack. The intensity of principal stress above flexural crack depends on the depth of the penetration of the crack. The greater the value of ρ the lesser the penetration of the flexural cracks resulting in lesser principal stresses for a given applied load. Consequently the greater must be the shear required to cause the principal stresses that will result in diagonal tension cracking (Elzanaty, Nilson and Slate, 1986). Increase of ρ also increases the dowel capacity of the member which will also give rise to higher shear capacity of the beam. Also increasing ρ affects the aggregate interlocking capacity. Beams with low ρ will have wide, long cracks in contrast to the shorter narrower cracks found in beams with high ρ. As aggregate interlock capacity depends on the crack width, increasing ρ will increase the aggregate interlocking capacity. Most of the design methods considered in this study under estimates these favourable effects of longitudinal tensile reinforcement on the shear strength of reinforced concrete beams. ACI Code, BS Code, Canadian Code-General Method and Shear Friction Method are examples for it.

It has been identified by various researchers that the shear strength of a beam decreases with the increase of the depth of the beam (d or h). This effect can be seen clearly from this study too. As mentioned in the literature review, the reason for this may be that, as the depth of the beam increases the crack widths at points above the main reinforcement tend to increase. This leads to reduction in aggregate interlock across the crack, resulting a lower shear capacity. Results of this study shows that the ACI Code, Australian Code and Canadian Code-General methods have failed to properly identify this effect while the BS Code, Japanese Code and Shear Friction Methods seems to identify this effect properly. Figure 4.23 illustrate the variation of shear stress of a beam section with the effective depth according to several design methods.

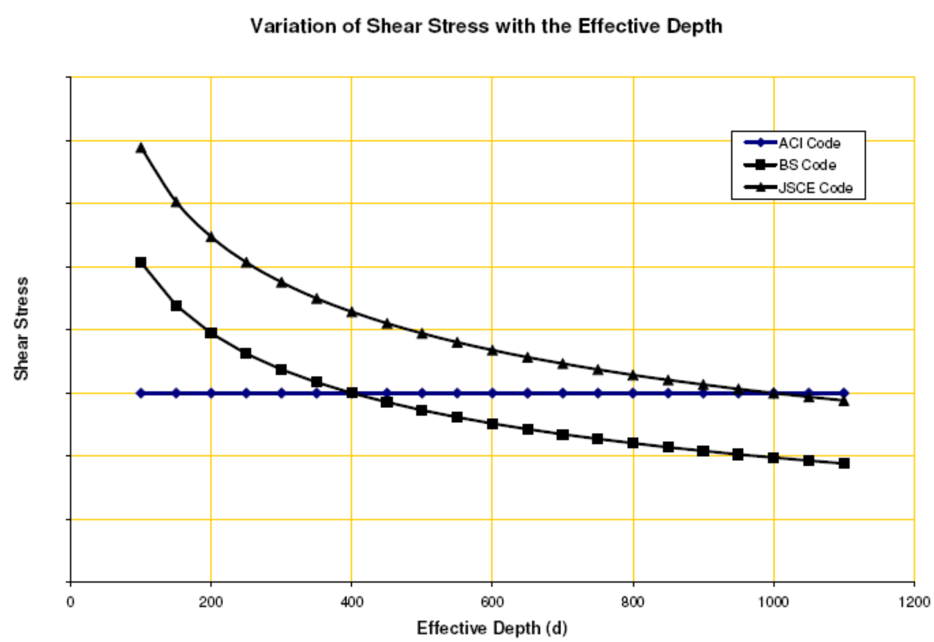


Figure 4.24
Variation of Predicted Shear Stress at the Critical Section with the Effective Depth

It can be seen from Fig.4.33, the predicted shear stress at the critical section from BS and Japanese Code methods decrease with the effective depth while the shear stress predicted from the ACI Code method remains a constant. In fact, both BS and Japanese Codes take shear capacity $V \, \alpha \left(\frac{1}{d}\right)^{1/4}$. Results of this study shows that this approach is much accurately predict the actual behaviour of the beams.

The inclined cracking load is a function of the tensile strength of the concrete. As a result of this, the shear strength of concrete depends on the compressive strength of concrete, f'_c. Tensile strength of concrete increases with the compressive strength of the concrete. But the relationship is not linearly proportional. Also up to cracking load, shear is resisted mostly by the shear stresses in concrete. After cracking, shear is resisted by aggregate interlock, dowel action of main reinforcement and the resistance of uncracked concrete at top of the beam. The aggregate interlock capacity depends strongly on the crack width and the surface roughness of the crack. Various researchers have reported that for the higher concrete strength crack surfaces were distinctly smoother, indicating that the shear force carried by the aggregate interlock decreases with the increase of compressive strength of concrete. Figure 4.24 illustrate the variation of predicted shear stress of a beam section from different methods with Compressive capacity of concrete. Results of this study shows that none of the design methods have been able to correctly identify the influence of concrete compressive strength on the shear capacity of a reinforced concrete beam for the entire range of the concrete compressive strengths. However it can be seen from the results that it is a much better approach to correlate V to $\sqrt[3]{f'_c}$, i.e. $V \, \alpha \, \sqrt[3]{f'_c}$ as it has done in BS, Australian and Japanese Codes.

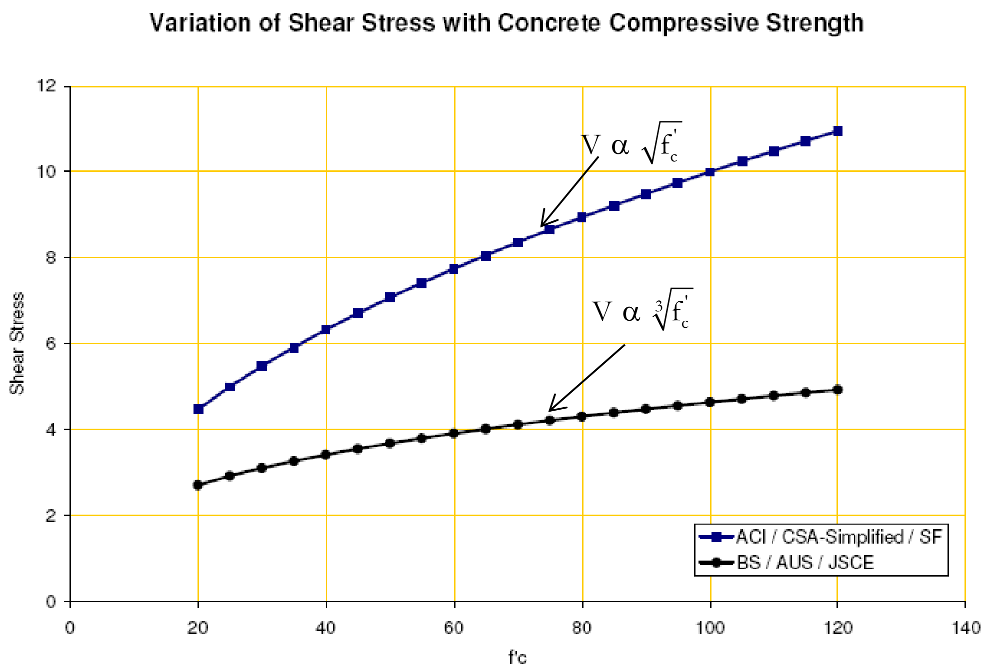


Figure 4.24
Variation of Predicted Shear Stress at the Critical Section with the Compressive Strength of Concrete

In general, it has been recognized that increasing a/d decreases the shear strength. For the same applied load, larger a/d ratios result in higher bending moments in the shear span; thus the depth of the penetration of the flexural cracks increases, reducing the shear capacity. This effect is found to significantly influence the accuracy of BS, Australian, Japanese Codes and Canadian Code General Method. As a result of this, these methods tend to over predict shear strength of beams when a/d ratio increases.

This study also shows that the maximum size of the coarse aggregate has an effect on the accuracy of the shear strength predictions of many design methods. But unfortunately the effect could not be directly identified. But when we find estimation for the critical value when f'_c it can be observed that the effect of the size of aggregate is significant particularly for higher strength grades. In normal strength concrete cracks go around the aggregate forming a rough crack surface which transfer shear by the aggregate interlock. But when concrete strength is high cracks may go through aggregates forming much smoother surfaces; thus affecting the aggregate interlock capacity. This phenomenon should be studied separately.

The presence of crack control reinforcement in the web, resist the widening of shear cracks in the web. This leads to a higher aggregate interlock capacity along the crack surface. And ultimately it increases the shear capacity of the beam. Most of the design methods have not been able to identify this favourable effect. Therefore their accuracy is found to be affected by the presence of crack-control reinforcement except the Canadian Code General Method and Shear Friction Method. In the Canadian Code General Method crack control reinforcement is considered to reduce the crack spacing. This reduces the average crack width hence increase the shear capacity of the beam.

When calculating the shear strength of a flanged beam, generally shear capacity of flange is conservatively neglected. Thus under estimate the shear strength of T and I beams when compare to the rectangular beams. This effect can be clearly seen on the accuracy of BS and Australian Code Methods.

Even though its effect could not be clearly identified it was seen that the yield strength of longitudinal tensile steel, f_y affect the accuracy of the design methods. As discussed earlier the intensity of principal stress above flexural crack depends on the depth of the penetration of the crack. For a given value of Area of Longitudinal tensile steel ratio ρ, higher the value of f_y, the lesser the penetration of the flexural cracks, resulting lesser principal stresses for a given applied load. Therefore much higher shear is required to cause the principal stresses that will result in diagonal tension cracking which will ultimately lead to the failure of the beam without shear reinforcement.

Beams with Shear Reinforcement

The presence of stirrups in beams increase the shear carrying capacity of a beam not only by carrying shear by themselves but also enhancing the other shear transfer mechanisms. The stirrups provide support to longitudinal steel to prevent bars being split from the surrounding concrete, hence increasing the dowel action. At the same time stirrups contain the crack, limiting its propagation and keeping its width small. These effects increase the aggregate interlocking capacity and the shear strength of uncracked concrete zone. Also shear reinforcement increases the strength of compression concrete by providing confinement. Although stirrups have a little effect on the diagonal cracking load, they enhance the shear capacity of concrete by increasing different shear transfer mechanisms.

Therefore the presence of stirrups in a beam found to rectify effect of many parameters of each and every design method. Also it was found in several occasions that the Ratio of Stirrup spacing to Effective depth, s/d affect the accuracy of the design, in several design methods. It is obvious when the number of stirrups crossed by an inclined crack is high then the shear carried by the stirrups as well as the enhancement of other shear carrying mechanisms by the stirrups will be high.

It should be noted that the Shear Friction method for beams with shear reinforcement has produce over conservative results. Predicted shear strength from this method depends only on the shear carrying capacity of the stirrups and the shear force transfer along cracked surfaces. It neglects the shear contribution from dowel action of longitudinal tensile steel and the shear contribution of uncracked concrete in flexural compression zone. This may be a reason for this method to give over conservative shear predictions.

Deep Beams

Among the selected design codes of practices only four of them have given guide lines for deep beam design. BS 8110 does not have provisions for deep beams. Results of this study shows that all the design methods have produces conservative results for deep beams when compared with the results of slender beams. The Truss model has been the key tool for analysing deep beams in ACI, Canadian and Australian Codes. Truss model neglects the shear contribution of concrete. This leads for over conservative results. Even though Japanese Code uses an empirical formula for prediction shear strength of deep beams this study shows that this method also produces excessively conservative results. Due to lack of experimental data Multinomial Logistic Regression analysis was not very successful for deep beams.

Industrial Survey

BS 8110 is widely used for shear designs in the Sri Lankan industry. Very few designers were found to be familiar with ACI and Australian Codes. Designers were found to be comfortable with the shear design process given in the BS 8110. But BS 8110 has not been able to provide a better idea about the shear mechanisms of a reinforced concrete beam. It was found that most of the times their design is governed by the design limitations imposed from Design Code.

Also it was found that they rarely use research information for design. Therefore most of the designers' knowledge about the theory behind the shear design process was found to be very weak and also it is not getting updated.

Majority of the designers consider that the shear designing is an important section of a beam designing process. Some of the senior designers were not satisfied with the limitations on the minimum shear reinforcement of the BS 8110. They considered those limitations were not conservative. But unfortunately there are designers who do not consider shear designing as a critical section especially among the young design engineers. Most of them consider that shear design for beams with shear reinforcement is not a critical although they may critical for beams without shear reinforcement. Most of them seem to worry only about punching Shear.

5. Conclusions and Recommendations

5.1 Introduction

The first analytical model for shear behaviour of reinforced concrete beams was published by Ritter (1988) and Mörsch (1902). Prediction of shear behaviour of the reinforced concrete beam was been uncertain from the beginning. Despite of the extensive research, still none of the methods seems to predict the shear strength accurately to match the test results. On the other hand shear failure is brittle which usually occurs with little warning hence the soundness of shear design procedure is vital.

The principle objective of this study is to carry out a state-of-the-art review of the shear design approaches. Under that we have selected five major codes including the BS 8110 and a developing method based on the shear friction concept and successfully found when these theories are justified for reinforced concrete.

5.2 Conclusions

From the literature survey thirteen numbers of parameters were identified to influence the shear capacity of reinforced concrete beams which are not subjected to axial forces. Unfortunately the influence of each parameter on the shear capacity of a reinforced concrete beam could not be verified directly using the test results due to the inadequacy of the database. Therefore a statistical method called Multinomial Logistic Regression has been used.

This method can be used to identify the influence of explanatory variables on a categorized dependent variable. In our case; dependent variable is the accuracy of predictions from each design method. Therefore using the Multinomial Logistic Regression parameters affecting the accuracy of each method was identified.

Various parameters could be identified to affect different design methods. For example: For beams without shear reinforcement the accuracy of shear strength predictions from equation 11-3 of ACI Code has been found to be significantly affected by six number of parameters. Namely they are: Percentage of Longitudinal Tensile Steel, Effective depth, Compression Capacity of Concrete, Crack Control Reinforcement, Maximum Aggregate size, and Shear Span to effective Depth Ratio, a/d. As discussed earlier, the effect of first four parameters on the accuracy of that method could be clearly identified using the Multinomial Logistic Regression Analysis results. But unfortunately the effect of the other two parameters could not be clearly identified. Possible reason for this may be that the effect of these parameters might depend on the value of another parameter. For example as we discussed earlier the effect of Maximum aggregate size was found to depend on the Compressive capacity of the concrete. In order to study this type of relationship, interaction term of parameters should be studied. But it is not covered in this study.

In the previous example of ACI Code equation 11-3, there were five variables which were found not be significant for the accuracy of the method while others were found to be significant. There are two possible reasons for a parameter to become none significant for that particular method. First one is that the shear design method has correctly identified the effect of that parameter on the shear strength of a beam. And the other one is that the particular parameter does not affect the shear strength of a beam at all.

After identifying the significant parameters for the accuracy of the method an attempt was made to statistically find when these parameters become significant for the accuracy of the method. Therefore a critical value for each parameter whose effect could be clearly identified was estimated using the Multinomial Logistic regression Analysis.

It should be noted that the Multinomial Logistic Regression can not be used to compare the accuracy of two different methods. Rather it can be used to identify the weaknesses in the design methods. Results of this study have identified reasons for each design method to loose the accuracy. Parameters identified in this study to influence the accuracy of each design method and their safe limits are important for practicing designers to minimize the risk of doing unsafe or over conservative shear design. On the other hand researchers can use these identifications to improve the accuracy of design methods.

All together twelve numbers of parameters were found to affect the accuracy of shear design methods, namely they are:

1. Percentage of Longitudinal Tensile Steel - ρ
2. Effective depth - **d**
3. Compression Capacity of Concrete - f_c'
4. Shear Span to effective Depth Ratio - **a/d**
5. Ratio of Stirrup spacing to Effective Depth - **s/d**
6. Percentage area of Shear Reinforcement - ρ_s
7. Yield strength of longitudinal tensile steel - f_y
8. Yield strength of transverse steel - f_{vy}
9. Anchorage of the Tensile Reinforcement
10. Presence of Crack Control Reinforcement
11. Maximum Aggregate size - a_g
12. cross sectional shape

Following table presents the percentage of predictions falling into each category for particular design method. This table gives an idea about the accuracy of each design method.

Table 5.1, Percentage of Predictions Falling into Each Category

	Beams without Shear r/f			Beams with Shear r/f			Overall accuracy		
	A	B	C	A	B	C	A	B	C
ACI -11.3	71.2%	12.9%	15.9%	77.6%	13.3%	9.1%	72.9%	13.0%	14.1%
ACI -11.5	68.7%	14.4%	16.9%	35.0%	44.1%	21.0%	59.8%	22.3%	18.0%
BS 8110	53.4%	33.0%	13.5%	59.7%	34.3%	6.1%	55.0%	33.3%	11.6%
AS 3600	43.9%	37.0%	18.8%	50.4%	37.6%	12.1%	45.3%	37.1%	17.4%
JSCE	67.4%	24.9%	7.7%	67.6%	26.8%	5.6%	68.6%	25.4%	6.0%
CSA-Simplified	69.3%	17.3%	13.4%	77.0%	16.7%	6.3%	71.2%	17.2%	11.6%
CSA-General	46.2%	36.0%	17.8%	42.9%	43.7%	13.5%	45.4%	37.9%	16.7%
Shear Friction	74.0%	11.6%	14.4%	98.1%	0.9%	0.9%	84.2%	7.1%	8.7%

Above table shows that Canadian Code General Method has the highest overall percentage of accurate predictions.

From the industrial survey it could be seen that most of the designers in Sri Lanka are familiar only with BS 8110. They seem to be comfortable with using Shear design procedure given in BS 8110. Whenever the shear design guidelines of BS8110 are not adequate most of the times they refer handbooks or go for expertise to overcome the problem. Few designers were found to read research information on the shear designing. Therefore most of the designers have a little knowledge about the shear behaviour of reinforced concrete beams. And unfortunately considerable percentage of designers considered that shear designing is not a critical parameter in the beam designing process.

5.3 Recommendations

This study has identified the parameters affecting the accuracy of each design procedure. Among them some are new parameters for that method. Others are already used in the design process but their effects have not been correctly represented by the design equations. Further experiments should be carried out to clarify the effect of newly identified variables and include them in design equation. And also research should be carried out to improve the shear design procedure by rectifying the effect of parameters which are already in the design equation but their effect has not properly recognized by the design method. This study should be further extended to cover deep beams with continuous spans and deep beams with complex loading systems.

Behaviour of beams without shear reinforcement has a larger dispersion compared to once with shear reinforcement. Code approaches such as: Australian Code, Japanese Code, British Code, Canadian Code, American Code (ACI 11-5) give safe values for design predictions for beams with shear reinforcement with a narrow dispersion. Shear Friction method with shear links is excessively conservative for test results considered in this study. For most of the cases, deep beams give conservative results but require more data to verify confidently.

Within the limitations specified in the code, Canadian General Method and Australian Code give most accurate results. The next best method is British standard which can also be recommended for application. Japanese Code gives conservative results without any restrictions on parameters. However this method is less accurate compared to Canadian General and Australian Code approaches.

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CHAPTER 4

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Appendix A

EXPLANATORY VARIABLES

1) Concrete Grade - **fc′**
2) Effective Depth – **d**
3) Width of the Beam – **b**
4) Percentage area of Longitudinal Tensile Steel for Shear – **ρ**
5) Yield Strength of Long. Tensile Steel – **fy**
6) Influence of Crack Control R/F
7) Maximum Aggregate Size - **ag**
8) Proximity of rigid support to point load – **a /d or α**
9) Cross Sectional Shape
10) Anchorage of longitudinal Tensile Steel - **ρ**
11) Percentage area of transverse steel - **ρ_s**
12) Spacing ratio of transverse reinforcement - **s/d**
13) Yield strength of transverse steel - **f_{vy}**

For beams with shear reinforcement all thirteen variables are used in the multinomial logistic regression analysis. But for beams without shear reinforcement only first ten numbers of variables are used. And for deep beams only first ten numbers of variables are used.

These explanatory parameters are further divided into two categories:

1) Continuous Variables

1) Concrete Grade - **fc′**
2) Effective Depth – **d**
3) Width of the Beam – **b**
4) % of Longitudinal Tensile Steel for Shear – **ρ**
5) Proximity of rigid support to point load – **a/d or α**
6) Amount of transverse steel -**ρ_s**
7) Spacing ratio of transverse reinforcement - **s/d**

2) Categorized Variables

1) Yield Strength of Long. Tensile Steel – **fy**

1 – 500 MPa < fy ≤ 600 MPa

2 – 450 MPa < fy ≤ 500 MPa

3 – 400 MPa < fy ≤ 450 MPa

4 – 350 MPa < fy ≤ 400 MPa

5 – 300 MPa < fy ≤ 350 MPa

2) Maximum Aggregate Size - $\mathbf{a_g}$

0 – 30mm < a_g

1 - 20 mm< a_g ≤ 30 mm

2 - 10 mm< a_g ≤ 20 mm

3 - 0 mm< a_g ≤ 10 mm

3) Cross Sectional Shape

1 – I Beams

2 – T Beams

3 - Rectangular Beams

4) Anchorage of longitudinal Tensile Steel

0 – longitudinal tensile is R/F fully anchored

1 - longitudinal tensile is not R/F fully anchored

5) Long. Compression Steel

0 – Beams with Long. Compression R/F

1 - Beams without Long. Compression R/F

6) Crack Control R/F

0 – Beams with Crack Control R/F

1 - Beams without Crack Control R/F

RESULTS

Beams without Shear Reinforcement

Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	V_n (kN) BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Kani et al.	147	3	16.8	19	0.152	306	0.287	0	0	0.000	42	31	30	30			31	21	
Kani et al.	149	3	18.0	19	0.152	323	0.272	0	0	0.000	44	32	29	30			32	23	
Kani et al.	151	3	19.3	19	0.152	326	0.273	0	0	0.000	36	33	30	31			33	24	
Kani et al.	152	3	19.7	19	0.152	319	0.270	0	0	0.000	32	33	30	31	30	30	33	23	49
Kani et al.	153	3	19.7	19	0.152	317	0.273	0	0	0.000	33	33	31	31	31	30	33	23	49
Kani et al.	102	3	25.3	19	0.152	314	0.269	0	0	0.000	49	35					37	25	
Kani et al.	103	3	29.4	19	0.152	314	0.274	0	0	0.000	39	37	37	37	38	36	37	26	32
Kani et al.	105	3	26.2	19	0.152	320	0.272	0	0	0.000	42	36	35	36			36	26	
Kani et al.	106	3	28.8	19	0.152	314	0.268	0	0	0.000	45	37	36	37	37	35	37	26	32
Kani et al.	111	3	27.0	19	0.152	317	0.272	0	0	0.000	43	36	36	36			36	26	
Kani et al.	112	3	27.0	19	0.152	317	0.273	0	0	0.000	39	36	36	36			36	26	
Kani et al.	114	3	25.5	19	0.152	330	0.270	0	0	0.000	61	36					38	26	
Kani et al.	115	3	26.2	19	0.152	319	0.272	0	0	0.000	45	36	35	36			36	26	
Kani et al.	25	3	24.5	19	0.152	774	0.271	0	0	0.000	104	47					50	48	
Kani et al.	26	3	27.1	19	0.152	774	0.271	0	0	0.000	78	49					52	50	
Kani et al.	27	3	29.8	19	0.152	774	0.271	0	0	0.000	51	51	37	41	38	45	51	52	32
Kani et al.	28	3	29.2	19	0.152	774	0.271	0	0	0.000	54	50	37	40	37	45	50	51	32
Kani et al.	29	3	24.5	19	0.152	774	0.271	0	0	0.000	43	47	34	35	34	42	48	48	33
Kani et al.	30	3	25.2	19	0.152	774	0.271	0	0	0.000	46	48	34	36	35	42	48	49	33
Kani et al.	35	3	26.1	19	0.152	761	0.269	0	0	0.000	45	48	35	37	35	42	48	49	44
Kani et al.	36	3	26.1	19	0.152	761	0.273	0	0	0.000	52	48	35	37	35	43	48	50	44
Kani et al.	81	3	27.5	19	0.152	1161	0.274	0	0	0.000	51	57	36	38	37	48	57	51	25
Kani et al.	83	3	27.4	19	0.152	1161	0.271	0	0	0.000	65	56	36	41	36	48	57	50	54
Kani et al.	84	3	27.4	19	0.152	1161	0.271	0	0	0.000	55	56	36	39	36	48	57	50	39
Kani et al.	85	3	25.5	19	0.152	1129	0.274	0	0	0.000	234	110					109	49	
Kani et al.	87	3	27.2	19	0.152	1129	0.269	0	0	0.000	240	110					112	50	
Kani et al.	91	3	27.4	19	0.152	1122	0.269	0	0	0.000	51	55	36	37	36	47	56	50	25
Kani et al.	94	3	25.3	19	0.152	1161	0.273	0	0	0.000	111	55					58	49	
Kani et al.	95	3	25.3	19	0.152	1161	0.275	0	0	0.000	73	55	35	41			55	49	
Kani et al.	96	3	25.3	19	0.152	1161	0.275	0	0	0.000	56	55	35	38	35	47	55	49	38
Kani et al.	97	3	27.2	19	0.152	1129	0.276	0	0	0.000	62	56	36	41	37	48	56	51	53
Kani et al.	99	3	26.2	19	0.152	1129	0.272	0	0	0.000	77	55	35	41	35	47	55	50	65
Kani et al.	100	3	27.2	19	0.152	1136	0.270	0	0	0.000	112	56					59	50	
Kani et al.	52	3	24.8	19	0.152	568	0.138	0	0	0.000	29	32	17	19	17	25	29	27	24
Kani et al.	55	3	25.1	19	0.152	568	0.135	0	0	0.000	33	32	17	19	17	25	29	26	35
Kani et al.	83	3	27.4	19	0.152	1161	0.271	0	0	0.000	65	56	36	41	36	48	57	50	54
Kani et al.	84	3	27.4	19	0.152	1161	0.271	0	0	0.000	55	56	36	39	36	51	57	50	39
Kani et al.	63	3	26.2	19	0.152	2323	0.543	0	0	0.000	93	93	70	77	71	91	88	83	64
Kani et al.	74	3	27.2	19	0.152	2265	0.523	0	0	0.000	108	92	69	78	71	90	88	82	87
Kani et al.	79	3	26.1	19	0.152	2316	0.556	0	0	0.000	84	94	72	74	72	92	87	85	35
Kani et al.	3043	3	26.9	19	0.152	4555	1.092	0	0	0.000	165	158	143	162	107	164	176	142	146
Kani et al.	3044	3	29.5	19	0.152	4555	1.097	0	0	0.000	159	163	150	163	112	171	181	147	112
Kani et al.	3046	3	26.7	19	0.152	4594	1.097	0	0	0.000	154		143	147	107	165		142	61
Moody et al.	A - A1	3	30.3	25	0.178	1006	0.262	0	0	0.000	60	61	43	46	43	54	62	59	
Moody et al.	A - A2	3	31.0	25	0.178	1020	0.267	0	0	0.000	67	62	44	48	44	55	63	60	
Moody et al.	A - A3	3	31.0	25	0.178	1058	0.268	0	0	0.000	76	63	44	48	44	56	63	61	
Moody et al.	A - A4	3	31.5	25	0.178	1136	0.270	0	0	0.000	71	65	45	49	45	58	65	61	
Moody et al.	A - B1	3	21.2	25	0.178	768	0.267	0	0	0.000	56	50	36	39	36	45	50	53	
Moody et al.	A - B2	3	21.6	25	0.178	774	0.268	0	0	0.000	60	50	37	39	37	45	51	54	
Moody et al.	A - B3	3	19.2	25	0.178	768	0.270	0	0	0.000	56	48	35	38	35	43	49	52	
Moody et al.	A - B4	3	16.8	25	0.178	800	0.272	0	0	0.000	56	47	33	36	33	42	47	50	
Moody et al.	A - C1	3	6.3	25	0.178	387	0.268	0	0	0.000	20	26	20	21	20	24	27	19	
Moody et al.	A - C2	3	6.1	25	0.178	400	0.272	0	0	0.000	25	27	20	21	20	24	27	20	
Moody et al.	A - C3	3	6.9	25	0.178	387	0.273	0	0	0.000	25	27	21	22	21	25	28	20	
Moody et al.	A - C4	3	6.8	25	0.178	400	0.274	0	0	0.000	25	28	21	22	21	25	28	20	43
Moody et al.	B - B1	3	36.7	25	0.152	774	0.268	0	0	0.000	58	54	41	43	41	50	55	55	

												V_n (kN)							
Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Moody et al.	B - B2	3	16.7	25	0.152	774	0.268	0	0	0.000	36	41	28	30	28	36	42	42	
Moody et al.	B - B3	3	25.8	25	0.152	774	0.268	0	0	0.000	52	48	34	37	35	43	48	49	
Moody et al.	B - B4	3	15.4	25	0.152	774	0.268	0	0	0.000	41	40	27	29	27	35	41	41	
Moody et al.	B - B5	3	30.7	25	0.152	774	0.268	0	0	0.000	52	51	38	40	38	46	51	52	
Moody et al.	B - B6	3	15.8	25	0.152	774	0.268	0	0	0.000	35	41	27	29	27	35	41	41	
Moody et al.	B - B7	3	30.9	25	0.152	774	0.268	0	0	0.000	51	51	38	40	38	46	51	52	
Moody et al.	B - B8	3	12.2	25	0.152	774	0.268	0	0	0.000	31	37	24	26	24	32	38	38	
Moody et al.	B - B9	3	41.2	25	0.152	774	0.268	0	0	0.000	53	56	43	45	44	52	57	57	
Moody et al.	B - B10	3	23.9	25	0.152	774	0.268	0	0	0.000	49	47	33	35	33	42	47	48	
Moody et al.	B - B11	3	38.1	25	0.152	774	0.268	0	0	0.000	60	55	42	44	42	50	55	56	
Moody et al.	B - B12	3	20.2	25	0.152	774	0.268	0	0	0.000	47	44	30	33	31	39	45	45	
Moody et al.	B - B13	3	37.8	25	0.152	774	0.268	0	0	0.000	56	54	42	43	42	50	55	55	
Moody et al.	B - B14	3	22.6	25	0.152	774	0.268	0	0	0.000	43	46	32	34	32	41	46	47	
Moody et al.	B - B15	3	37.4	25	0.152	774	0.268	0	0	0.000	51	54	41	43	42	50	55	55	
Moody et al.	B - B16	3	16.3	25	0.152	774	0.268	0	0	0.000	38	41	27	30	27	36	42	42	
Mphonde & Frantz	AO-3-3b	3	20.8	10	0.152	1530	0.298	0	0	0.000	64	56	34	40	35	46	59	50	
phonde & Fran	AO-3-3	3	27.1	10	0.152	1058	0.298	0	0	0.000	67	57	39	42	40	48	57	55	
phonde & Fran	AO-7-3	3	37.7	10	0.152	1530	0.298	0	0	0.000	82	69	46	51	47	59	72	61	
phonde & Fran	AO-7-3b	3	41.6	10	0.152	1530	0.298	0	0	0.000	83	71	49	54	49	62	75	63	
phonde & Fran	O-11-3	3	74.9	10	0.152	1530	0.298	0	0	0.000	89	87	65	69	66	78	91	77	
phonde & Fran	O-11-3	3	74.6	10	0.152	1530	0.298	0	0	0.000	89	86	65	69	66	78	91	77	
phonde & Fran	O-15-3	3	81.3	10	0.152	1530	0.298	0	0	0.000	93	89	68	72	68	80	93	79	
phonde & Fran	O-15-3	3	93.7	10	0.152	1530	0.298	0	0	0.000	100	93	73	77	74	85	98	83	
phonde & Fran	O-15-3	3	91.8	10	0.152	1530	0.298	0	0	0.000	97	93	72	76	73	84	97	82	
Ahmad et al.	A1	3	60.8	13	0.127	1020	0.203	0	0	0.000	58	50	33	36	34	45	57	45	23
Ahmad et al.	A2	3	60.8	13	0.127	1020	0.203	0	0	0.000	69	50	33	38	34	45	57	45	23
Ahmad et al.	A3	3	60.8	13	0.127	1020	0.203	0	0	0.000	69	50	33	38	34	45	57	45	23
Ahmad et al.	A8	3	60.8	13	0.127	471	0.208	0	0	0.000	49	43	34	35	34	38	44	46	23
Ahmad et al.	B1	3	67.0	13	0.127	1290	0.202	0	0	0.000	51	52	35	39	35	49	64	46	23
Ahmad et al.	B2	3	67.0	13	0.127	1290	0.202	0	0	0.000	69	52	35	40	35	49	64	46	23
Ahmad et al.	B3	3	67.0	13	0.127	1290	0.202	0	0	0.000	100	52	35	41	35	49	64	46	23
Ahmad et al.	B7	3	67.0	13	0.127	600	0.208	0	0	0.000	45	48	36	37	36	42	49	47	23
Ahmad et al.	B8	3	67.0	13	0.127	600	0.208	0	0	0.000	47	48	36	38	36	42	49	47	23
Ahmad et al.	B9	3	67.0	13	0.127	600	0.208	0	0	0.000	80	48	36	38	36	42	49	47	23
Ahmad et al.	C1	3	64.3	13	0.127	1548	0.184	0	0	0.000	54	48	31	36	31	47	67	42	23
Ahmad et al.	C2	3	64.3	13	0.127	1548	0.184	0	0	0.000	76	48	31	38	31	47	67	42	23
Ahmad et al.	C3	3	64.3	13	0.127	1548	0.184	0	0	0.000	69	48			31	47	67	42	23
Ahmad et al.	C7	3	64.3	13	0.127	852	0.207	0	0	0.000	45	52	35	37	35	44	55	46	23
Ahmad et al.	C8	3	64.3	13	0.127	852	0.207	0	0	0.000	44	52	35	38	35	44	55	46	23
Ahmad et al.	C9	3	64.3	13	0.127	852	0.207	0	0	0.000	45	52	35	39	35	44	55	46	23
Bresler & Scordelis	OA-1	3	22.5	19	0.310	2580	0.461	0	0	0.000	167	138							
resler & Scordel	OA-2	3	23.7	19	0.305	3225	0.466	0	0	0.000	178	150							
resler & Scordel	OA-3	3	37.6	19	0.307	3870	0.462	0	0	0.000	189	186							
Xie et al.	NNN-3	3	37.7	19	0.127	568	0.216	0	0	0.000	37	40	28	30	28	35	40	40	
Xie et al.	NHN-3	3	98.9	19	0.127	568	0.216	0	0	0.000	46	55	45	46	46	51	55	55	23
Yoon et al.	N1S	3	36.0	20	0.375	7000	0.655	0	0	0.000	249	296	245	269	232	284	293	267	
Yoon et al.	M1S	3	67.0	10	0.375	7000	0.655	0	0	0.000	296	364	334	354	316	342	360	329	
Yoon et al.	H1S	3	87.0	10	0.375	7000	0.655	0	0	0.000	327	397	380	398	360	379	393	359	197
Taylor, H. P. J.	A2	3	25.4	19	0.400	5026	0.930	0	0	0.000	328	285	311	325	253	294	297	297	323
Taylor, H. P. J.	B2	3	23.7	19	0.200	1257	0.465	0	0	0.000	87	83	75	79	80	80	80	86	90
Taylor, H. P. J.	B3	3	30.5	9	0.200	1257	0.465	0	0	0.000	85	90	85	88	91	84	87	94	70
Taylor, H. P. J.	C1	3	24.4	19	0.100	314	0.233	0	0	0.000	23	25	19	20	19	22	24	26	26
Taylor, H. P. J.	C2	3	24.4	9	0.100	314	0.233	0	0	0.000	24	25	19	20	19	21	24	26	26
Taylor, H. P. J.	C3	3	26.2	9	0.100	314	0.233	0	0	0.000	28	25	20	21	20	22	25	27	27
Taylor, H. P. J.	C4	3	19.8	9	0.100	314	0.233	0	0	0.000	23	23	17	18	17	20	22	24	25
Taylor, H. P. J.	C5	3	21.3	9	0.100	314	0.233	0	0	0.000	27	24	18	19	18	20	23	25	25

												V_n (kN)							
Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Taylor, H. P. J.	C6	3	27.4	2	0.100	314	0.233	0	0	0.000	28	26	20	21	20	21	25	27	27
Taylor, H. P. J.	D1	3	28.8	2	0.060	113	0.140	0	0	0.000	12	11	7	8	8	8	9	10	6
Taylor, H. P. J.	D2	3	28.8	2	0.060	113	0.140	0	0	0.000	12	11	7	8	8	8	9	10	6
Taylor, H. P. J.	D3	3	28.8	2	0.060	113	0.140	0	0	0.000	11	11	7	8	8	8	9	10	6
Taylor, H. P. J.	D4	3	28.8	2	0.060	113	0.140	0	0	0.000	11	11	7	8	8	8	9	10	6
Podgorniak-Sta	B100	3	36.0	10	0.300	2800	0.925	0	0	0.000	225	217	276	279	225	217	226	186	
Podgorniak-Sta	B100-R	3	36.0	10	0.300	2800	0.925	0	0	0.000	249	217	276	279	225	217	226	186	
Podgorniak-Sta	B100B	3	39.0	10	0.300	2800	0.925	0	0	0.000	204	223	288	290	234	224	232	191	
Podgorniak-Sta	B100L	3	39.0	10	0.300	2800	0.925	0	0	0.000	223	223	288	290	234	226	232	191	210
Podgorniak-Sta	B100L-R	3	39.0	10	0.300	2800	0.925	0	0	0.000	235	223	288	290	234	226	232	191	210
Podgorniak-Sta	BN100	3	37.2	10	0.300	2100	0.925	0	0	0.000	192	199	281	279	229	205	208	141	
Podgorniak-Sta	BRL100	3	94.0	10	0.300	1400	0.925	0	0	0.000	163	237	447	433	363	251	247	128	
Podgorniak-Sta	B100H	3	98.0	10	0.300	2800	0.925	0	0	0.000	193	303	456	450	371	318	316	260	
Podgorniak-Sta	B100HE	3	98.0	10	0.300	2800	0.925	0	0	0.000	217	303	456	450	371	318	316	260	
Podgorniak-Sta	BH100	3	98.8	10	0.300	2100	0.925	0	0	0.000	193	276	458	448	373	292	288	196	
Podgorniak-Sta	BN50	3	37.2	10	0.300	1100	0.450	0	0	0.000	132	119	137	136	148	118	117	89	
Podgorniak-Sta	BH50	3	98.8	10	0.300	1100	0.450	0	0	0.000	132	165	223	218	241	170	162	123	
Podgorniak-Sta	BN25	3	37.2	10	0.300	600	0.225	0	0	0.000	73	73	68	68	69	67	71	57	
Podgorniak-Sta	BH25	3	98.8	10	0.300	600	0.225	0	0	0.000	85	101	111	109	112	98	99	80	
Podgorniak-Sta	BN12	3	37.2	10	0.300	300	0.110	0	0	0.000	40		33	33	34	36		30	
Podgorniak-Sta	B100D	3	36.0	10	0.300	2100	0.925	0	0	0.000	320	197	276	275	225	268	206	140	
Podgorniak-Sta	BND100	3	37.2	10	0.300	2100	0.925	0	0	0.000	258	199	281	279	229	272	208	141	
Podgorniak-Sta	BHD100	3	98.8	10	0.300	2100	0.925	0	0	0.000	278	276	458	448	373	401	288	196	
Podgorniak-Sta	HD100-	3	98.8	10	0.300	2100	0.925	0	0	0.000	334	276	458	448	373	401	288	196	
Podgorniak-Sta	BND50	3	37.2	10	0.300	1100	0.450	0	0	0.000	163	119	137	136	148	147	117	89	
Collins & Kuch	E100A-	3	50.0	10	0.295	2800	0.920	0	0	0.000	201	239	319	322	260	250	247	208	
Collins & Kuch	100A-4	3	50.0	10	0.295	2800	0.920	0	0	0.000	236	239	319	322	260	250	247	208	
Collins & Kuch	E100A-	3	86.0	10	0.295	2800	0.920	0	0	0.000	184	286	418	416	341	301	296	249	207
Collins & Kuch	E50A-4	3	52.5	10	0.169	800	0.459	0	0	0.000	69	83	93	94	100	83	80	72	59
Collins & Kuch	E50A-45	3	52.5	10	0.169	800	0.459	0	0	0.000	81	83	93	94	100	83	80	72	59
Collins & Kuch	E50A-8	3	91.0	10	0.169	800	0.459	0	0	0.000	73	99	123	122	132	101	96	86	59
Collins & Kuch	E100B-	3	50.0	10	0.295	2800	0.920	0	0	0.000	281	239	319	322	260	326	247	208	
Collins & Kuch	100B-4	3	50.0	10	0.295	2800	0.920	0	0	0.000	316	239	319	322	260	326	247	208	
Collins & Kuch	E100B-	3	86.0	10	0.295	2800	0.920	0	0	0.000	365	286	418	416	341	398	296	249	207
Collins & Kuch	100B-8	3	86.0	10	0.295	2800	0.920	0	0	0.000	364	286	418	416	341	398	296	249	207
Collins & Kuch	E50B-4	3	52.5	10	0.169	800	0.459	0	0	0.000	87	83	93	94	100	95	80	72	59
Collins & Kuch	E50B-8	3	91.0	10	0.169	800	0.459	0	0	0.000	101	99	123	122	132	116	96	86	59
Angelakos et a	DB120	3	21.0	10	0.300	2800	0.925	0	0	0.000	179	181	211	217	172	179	189	156	
Angelakos et a	DB0.53	3	32.0	10	0.300	1400	0.925	0	0	0.000	165	166	261	256	212	173	173	90	
Angelakos et a	DB130	3	32.0	10	0.300	2800	0.925	0	0	0.000	185	209	261	264	212	208	218	179	
Angelakos et a	DB230	3	32.0	10	0.300	5600	0.925	0	0	0.000	257	263	261	281	212	252	274	266	
Angelakos et a	DB140	3	38.0	10	0.300	2800	0.925	0	0	0.000	180	221	284	286	231	222	230	190	
Angelakos et a	DB165	3	65.0	10	0.300	2800	0.925	0	0	0.000	185	264	372	369	302	273	276	227	
Angelakos et a	DB180	3	80.0	10	0.300	2800	0.925	0	0	0.000	172	283	412	408	335	295	295	243	
Adebar & Collin	ST1	3	52.5	19	0.360	1570	0.278	0	0	0.000	128	139	120	124	121	135	137	151	78
Adebar & Collin	ST2	3	52.5	19	0.360	1570	0.278	0	0	0.000	119	139	120	124	121	135	137	151	78
Adebar & Collin	ST3	3	49.3	19	0.290	1570	0.278	0	0	0.000	108	118	94	99	95	112	116	119	63
Adebar & Collin	ST8	3	46.2	19	0.290	1570	0.278	0	0	0.000	81	115	91	96	92	109	114	116	63
Adebar & Collin	ST16	3	51.5	19	0.290	1570	0.178	0	0	0.000	75	99	62	64	62	84	94	84	43
Adebar & Collin	ST17	3	51.5	19	0.290	1570	0.378	0	0	0.000	119	136					133	145	
Adebar & Collin	ST23	3	58.9	19	0.290	800	0.278	0	0	0.000	90		103	102	103	101	99	83	63
ddadin & Matto	A1	3	58.9	19	0.290	800	0.278	0	0	0.000	90						99	78	
ddadin & Matto	C1	3	25.9	19	0.178	2580	0.381	0	0	0.000	87		57	65	65	82	95	75	
ddadin & Matto	E1	3	25.9	19	0.178	2580	0.381	0	0	0.000	87						77	61	
Ghannoum	N220 - S	3	34.2	20	0.400	900	0.190	0	0	0.000	103	91	74	76	74	82	90	85	
Ghannoum	N220 - M	3	34.2	20	0.400	1500	0.190	0	0	0.000	122	108	74	80	74	93	106	108	

												V_n (kN)							
Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Ghannoum	N350 -	3	34.2	20	0.400	1500	0.313	0	0	0.000	158	133	121	126	145	128	134	127	
Ghannoum	N350 -	3	34.2	20	0.400	2500	0.313	0	0	0.000	179	158	121	133	145	144	158	158	
Ghannoum	N485 -	3	34.2	20	0.400	2100	0.440	0	0	0.000	188	172	171	177	186	171	169	163	
Ghannoum	N485 -	3	34.2	20	0.400	3500	0.440	0	0	0.000	215	204	171	187	186	194	200	205	
Ghannoum	N960 -	3	34.2	20	0.400	4200	0.889	0	0	0.000	373	291	345	357	286	307	298	273	
Ghannoum	N960 -	3	34.2	20	0.400	7000	0.889	0	0	0.000	391	345	345	376	286	350	354	347	
Ghannoum	H220 -	3	58.6	10	0.400	900	0.190	0	0	0.000	106	109	97	98	97	97	107	104	
Ghannoum	H220 -	3	58.6	10	0.400	1500	0.190	0	0	0.000	135	130	97	102	97	111	127	131	
Ghannoum	H350 -	3	58.6	10	0.400	1500	0.313	0	0	0.000	158	160	159	161	190	151	160	154	
Ghannoum	H350 -	3	58.6	10	0.400	2500	0.313	0	0	0.000	190	189	159	168	190	172	190	192	
Ghannoum	H485 -	3	58.6	10	0.400	2100	0.440	0	0	0.000	199	206	224	227	243	202	202	198	
Ghannoum	H485 -	3	58.6	10	0.400	3500	0.440	0	0	0.000	199	244	224	237	243	231	239	249	
Ghannoum	H960 -	3	58.6	10	0.400	4200	0.889	0	0	0.000	320	348	452	458	375	355	357	332	
Ghannoum	H960 -	3	58.6	10	0.400	7000	0.889	0	0	0.000	341	413	452	478	375	411	423	422	
Elmetwally	BH-10(	3	88.0	14	0.300	2800	0.921	0	0	0.000	242	292	430	425	351	315	305	251	210
Elmetwally	BH-10(	3	80.0	14	0.300	2800	0.884	0	0	0.000	228	278	394	390	327	298	295	246	
Elmetwally	BH-10(	3	87.0	14	0.300	2800	0.887	0	0	0.000	220	286	412	407	342	308	304	253	210
Elmetwally	BH-10(	3	86.0	14	0.300	2800	0.876	0	0	0.000	348	283	405	400	338	305	303	252	210
Elmetwally	BH-10(	3	78.0	14	0.300	2800	0.895	0	0	0.000	249	277	394	390	325	298	293	243	210
Elzanty et al.	F11	3	20.7	13	0.178	568	0.267	0	0	0.000	44	45	36	37	36	40	45	43	
Elzanty et al.	F12	3	20.7	13	0.178	1161	0.267	0	0	0.000	53	57	36	39	36	47	57	54	
Elzanty et al.	F8	3	40.0	13	0.178	458	0.267	0	0	0.000	45	52	50	49	50	49	52	43	38
Elzanty et al.	F13	3	40.0	13	0.178	568	0.267	0	0	0.000	45	55	50	50	50	52	56	53	
Elzanty et al.	F14	3	40.0	13	0.178	1161	0.267	0	0	0.000	63	70	50	52	50	61	71	67	
Elzanty et al.	F1	3	65.5	13	0.178	568	0.267	0	0	0.000	57	65	64	63	64	63	66	63	
Elzanty et al.	F2	3	65.6	13	0.178	1161	0.267	0	0	0.000	66	83	64	66	64	73	84	79	
Elzanty et al.	F10	3	65.5	13	0.178	1530	0.267	0	0	0.000	77	89	64	67	64	78	92	79	
Elzanty et al.	F9	3	79.3	13	0.178	768	0.267	0	0	0.000	62	77	70	70	71	73	78	84	38
Elzanty et al.	F15	3	79.3	13	0.178	1161	0.267	0	0	0.000	67	88	70	72	71	79	89	84	
Elzanty et al.	F3	3	69.0	13	0.178	568	0.267	0	0	0.000	79	67					72	64	
Elzanty et al.	F4	3	69.0	13	0.178	1161	0.267	0	0	0.000	113	84					91	80	
Elzanty et al.	F6	3	63.4	13	0.178	1161	0.267	0	0	0.000	60	82	63	63	63	73	83	78	
Walraven & Le	A1	3	22.9	16	0.200	208	0.125	0	0	0.000	32	26	20	20	20	22	24	17	40
Walraven & Le	A2	3	22.9	16	0.200	622	0.420	0	0	0.000	71	62	67	67	74	62	61	43	84
Walraven & Le	A3	3	23.2	16	0.200	1138	0.720	0	0	0.000	101	95	115	116	105	99	91	70	123
Mathey and W	II-3	3	21.9	25	0.203	1538	0.403	0	0	0.000	262	108					118	85	
Mathey and W	II-4	3	26.4	25	0.203	1538	0.403	0	0	0.000	313	115					125	91	
Mathey and W	III-5	3	25.7	25	0.203	1513	0.403	0	0	0.000	289	114					124	90	
Mathey and W	III-6	3	25.6	25	0.203	1513	0.403	0	0	0.000	291	114					123	90	
Mathey and W	IV-7	3	24.1	25	0.203	1522	0.403	0	0	0.000	291	112					121	88	
Mathey and W	IV-8	3	24.9	25	0.203	1522	0.403	0	0	0.000	304	113					122	89	
Mathey and W	V-9	3	23.1	25	0.203	949	0.403	0	0	0.000	224	94					102	67	
Mathey and W	V-10	3	27.0	25	0.203	949	0.403	0	0	0.000	268	99					107	71	
Mathey and W	VI-11	3	25.4	25	0.203	957	0.403	0	0	0.000	224	97					106	70	
Mathey and W	VI-12	3	25.7	25	0.203	957	0.403	0	0	0.000	268	98					106	70	
Mathey and W	V-13	3	22.4	25	0.203	614	0.403	0	0	0.000	222	80					87	43	
Mathey and W	V-14	3	26.7	25	0.203	614	0.403	0	0	0.000	224	85					93	45	
Mathey and W	VI-15	3	25.5	25	0.203	614	0.403	0	0	0.000	180	84					91	45	
Mathey and W	VI-16	3	22.8	25	0.203	614	0.403	0	0	0.000	189	81					88	43	
Mathey and W	IIIa-17	3	29.2	25	0.203	2078	0.403	0	0	0.000	88	100	73	79	82	92	100	94	65
Mathey and W	IIIa-18	3	25.2	25	0.203	2078	0.403	0	0	0.000	81	95	68	74	76	87	95	89	72
Mathey and W	Va-19	3	23.5	25	0.203	761	0.403	0	0	0.000	63	66	66	66	73	67	67	54	70
Mathey and W	Va-20	3	25.6	25	0.203	761	0.403	0	0	0.000	66	68	69	69	77	69	68	56	72
Mathey and W	VIb-21	3	26.1	25	0.203	687	0.403	0	0	0.000	71	67	69	70	77	68	67	51	99
Mathey and W	VIb-22	3	25.8	25	0.203	687	0.403	0	0	0.000	62	66	69	70	77	67	66	50	99
Mathey and W	VIb-23	3	30.6	25	0.203	687	0.403	0	0	0.000	75	70	75	76	84	72	70	53	65

												V_n (kN)							
Author	Beam	Cross Sectional	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Kani et al.	147	3	16.8	19	0.152	306	0.287	0	0	0.000	42	31	30	30			31	21	
Kani et al.	149	3	18.0	19	0.152	323	0.272	0	0	0.000	44	32	29	30			32	23	
Kani et al.	151	3	19.3	19	0.152	326	0.273	0	0	0.000	36	33	30	31			33	24	
Kani et al.	152	3	19.7	19	0.152	319	0.270	0	0	0.000	32	33	30	31	30	30	33	23	49
Kani et al.	153	3	19.7	19	0.152	317	0.273	0	0	0.000	33	33	31	31	31	30	33	23	49
Kani et al.	102	3	25.3	19	0.152	314	0.269	0	0	0.000	49	35					37	25	
Kani et al.	103	3	29.4	19	0.152	314	0.274	0	0	0.000	39	37	37	37	38	36	37	26	32
Kani et al.	105	3	26.2	19	0.152	320	0.272	0	0	0.000	42	36	35	36			36	26	
Kani et al.	106	3	28.8	19	0.152	314	0.268	0	0	0.000	45	37	36	37	37	35	37	26	32
Kani et al.	111	3	27.0	19	0.152	317	0.272	0	0	0.000	43	36	36	36			36	26	
Kani et al.	112	3	27.0	19	0.152	317	0.273	0	0	0.000	39	36	36	36			36	26	
Kani et al.	114	3	25.5	19	0.152	330	0.270	0	0	0.000	61	36					38	26	
Kani et al.	115	3	26.2	19	0.152	319	0.272	0	0	0.000	45	36	35	36			36	26	
Kani et al.	25	3	24.5	19	0.152	774	0.271	0	0	0.000	104	47					50	48	
Kani et al.	26	3	27.1	19	0.152	774	0.271	0	0	0.000	78	49					52	50	
Kani et al.	27	3	29.8	19	0.152	774	0.271	0	0	0.000	51	51	37	41	38	45	51	52	32
Kani et al.	28	3	29.2	19	0.152	774	0.271	0	0	0.000	54	50	37	40	37	45	50	51	32
Kani et al.	29	3	24.5	19	0.152	774	0.271	0	0	0.000	43	47	34	35	34	42	48	48	33
Kani et al.	30	3	25.2	19	0.152	774	0.271	0	0	0.000	46	48	34	36	35	42	48	49	33
Kani et al.	35	3	26.1	19	0.152	761	0.269	0	0	0.000	45	48	35	37	35	42	48	49	44
Kani et al.	36	3	26.1	19	0.152	761	0.273	0	0	0.000	52	48	35	37	35	43	48	50	44
Kani et al.	81	3	27.5	19	0.152	1161	0.274	0	0	0.000	51	57	36	38	37	48	57	51	25
Kani et al.	83	3	27.4	19	0.152	1161	0.271	0	0	0.000	65	56	36	41	36	48	57	50	54
Kani et al.	84	3	27.4	19	0.152	1161	0.271	0	0	0.000	55	56	36	39	36	48	57	50	39
Kani et al.	85	3	25.5	19	0.152	1129	0.274	0	0	0.000	234	110					109	49	
Kani et al.	87	3	27.2	19	0.152	1129	0.269	0	0	0.000	240	110					112	50	
Kani et al.	91	3	27.4	19	0.152	1122	0.269	0	0	0.000	51	55	36	37	36	47	56	50	25
Kani et al.	94	3	25.3	19	0.152	1161	0.273	0	0	0.000	111	55					58	49	
Kani et al.	95	3	25.3	19	0.152	1161	0.275	0	0	0.000	73	55	35	41			55	49	
Kani et al.	96	3	25.3	19	0.152	1161	0.275	0	0	0.000	56	55	35	38	35	47	55	49	38
Kani et al.	97	3	27.2	19	0.152	1129	0.276	0	0	0.000	62	56	36	41	37	48	56	51	53
Kani et al.	99	3	26.2	19	0.152	1129	0.272	0	0	0.000	77	55	35	41	35	47	55	50	65
Kani et al.	100	3	27.2	19	0.152	1136	0.270	0	0	0.000	112	56					59	50	
Kani et al.	52	3	24.8	19	0.152	568	0.138	0	0	0.000	29	32	17	19	17	25	29	27	24
Kani et al.	55	3	25.1	19	0.152	568	0.135	0	0	0.000	33	32	17	19	17	25	29	26	35
Kani et al.	83	3	27.4	19	0.152	1161	0.271	0	0	0.000	65	56	36	41	36	48	57	50	54
Kani et al.	84	3	27.4	19	0.152	1161	0.271	0	0	0.000	55	56	36	39	36	51	57	50	39
Kani et al.	63	3	26.2	19	0.152	2323	0.543	0	0	0.000	93	93	70	77	71	91	88	83	64
Kani et al.	74	3	27.2	19	0.152	2265	0.523	0	0	0.000	108	92	69	78	71	90	88	82	87
Kani et al.	79	3	26.1	19	0.152	2316	0.556	0	0	0.000	84	94	72	74	72	92	87	85	35
Kani et al.	3043	3	26.9	19	0.152	4555	1.092	0	0	0.000	165	158	143	162	107	164	176	142	146
Kani et al.	3044	3	29.5	19	0.152	4555	1.097	0	0	0.000	159	163	150	163	112	171	181	147	112
Kani et al.	3046	3	26.7	19	0.152	4594	1.097	0	0	0.000	154		143	147	107	165		142	61
Moody et al.	A - A1	3	30.3	25	0.178	1006	0.262	0	0	0.000	60	61	43	46	43	54	62	59	
Moody et al.	A - A2	3	31.0	25	0.178	1020	0.267	0	0	0.000	67	62	44	48	44	55	63	60	
Moody et al.	A - A3	3	31.0	25	0.178	1058	0.268	0	0	0.000	76	63	44	48	44	56	63	61	
Moody et al.	A - A4	3	31.5	25	0.178	1136	0.270	0	0	0.000	71	65	45	49	45	58	65	61	
Moody et al.	A - B1	3	21.2	25	0.178	768	0.267	0	0	0.000	56	50	36	39	36	45	50	53	
Moody et al.	A - B2	3	21.6	25	0.178	774	0.268	0	0	0.000	60	50	37	39	37	45	51	54	
Moody et al.	A - B3	3	19.2	25	0.178	768	0.270	0	0	0.000	56	48	35	38	35	43	49	52	
Moody et al.	A - B4	3	16.8	25	0.178	800	0.272	0	0	0.000	56	47	33	36	33	42	47	50	
Moody et al.	A - C1	3	6.3	25	0.178	387	0.268	0	0	0.000	20	26	20	21	20	24	27	19	
Moody et al.	A - C2	3	6.1	25	0.178	400	0.272	0	0	0.000	25	27	20	21	20	24	27	20	
Moody et al.	A - C3	3	6.9	25	0.178	387	0.273	0	0	0.000	25	27	21	22	21	25	28	20	
Moody et al.	A - C4	3	6.8	25	0.178	400	0.274	0	0	0.000	25	28	21	22	21	25	28	20	43
Moody et al.	B - B1	3	36.7	25	0.152	774	0.268	0	0	0.000	58	54	41	43	41	50	55	55	

Author	Beam	Cross Sectional Shape	f'c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	V_n (kN) BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Moody et al.	II-20b	3	20.4	25	0.178	4032	0.533	0	0	0.000	369	266					214	90	
Moody et al.	IV-g	3	23.4	25	0.178	516	0.305	0	0	0.000	80	48					58	39	
Moody et al.	IV-h	3	25.9	25	0.178	798	0.305	0	0	0.000	89	57					69	63	
Moody et al.	IV-i	3	24.1	25	0.178	1140	0.305	0	0	0.000	86	63					75	63	
Moody et al.	IV-j	3	24.8	25	0.178	1553	0.305	0	0	0.000	106	70					84	63	
Moody et al.	IV-k	3	25.0	25	0.178	1498	0.305	0	0	0.000	112	69					83	63	
Moody et al.	IV-l	3	27.0	25	0.178	2584	0.305	0	0	0.000	103	73					101	65	
Moody et al.	V-b	3	26.0	25	0.178	798	0.305	0	0	0.000	95	86					104	63	
Moody et al.	V-d	3	24.8	25	0.178	1553	0.305	0	0	0.000	113	105					126	63	
Moody et al.	V-f	3	23.3	25	0.178	2584	0.305	0	0	0.000	111	105					144	62	
Moody et al.	VI-a	3	28.2	25	0.178	516	0.305	0	0	0.000	103	57					69	42	
Moody et al.	VI-b	3	28.7	25	0.178	798	0.305	0	0	0.000	172	66					80	65	
Morrow	B14-B2	3	14.6	6	0.305	2078	0.368	0	0	0.000	367	207							
Morrow	-E	3	12.7	6	0.305	651	0.375	0	0	0.000	278	138							
Morrow	A	3	22.6	6	0.305	2760	0.362	0	0	0.000	512	257							
Morrow	B	3	26.3	6	0.305	2078	0.368	0	0	0.000	500	252							
Morrow	E	3	28.9	6	0.305	1393	0.368	0	0	0.000	512	228							
Morrow	A	3	45.4	6	0.305	4154	0.356	0	0	0.000	901	335							
Morrow	B6	3	46.8	6	0.305	2078	0.368	0	0	0.000	778	306							
Morrow	B21B2	3	13.9	6	0.305	2081	0.367	0	0	0.000	239	135					143	102	
Morrow	E2	3	11.3	6	0.305	651	0.375	0	0	0.000	212	88					90	37	
Morrow	A4	3	29.8	6	0.305	2763	0.368	0	0	0.000	523	193					202	132	
Morrow	B4	3	27.1	6	0.305	2078	0.368	0	0	0.000	396	170					178	128	
Morrow	E4	3	24.2	6	0.305	1381	0.365	0	0	0.000	423	141					150	102	
Morrow	E4R	3	31.9	6	0.305	1393	0.368	0	0	0.000	434	157					164	112	
Morrow	F4	3	31.4	6	0.305	1320	0.370	0	0	0.000	468	154					161	105	
Morrow	A6	3	45.3	6	0.305	4154	0.356	0	0	0.000	579	223					266	148	
Morrow	B6	3	45.5	6	0.305	2080	0.375	0	0	0.000	579	207					212	155	
Morrow	B28B2	3	14.7	6	0.305	2075	0.362	0	0	0.000	201	101					109	103	
Morrow	E2	3	13.7	6	0.305	646	0.372	0	0	0.000	130	70					72	39	
Morrow	A4	3	27.5	6	0.305	2763	0.368	0	0	0.000	324	141					147	129	
Morrow	B4	3	32.3	6	0.305	2078	0.368	0	0	0.000	257	135					142	136	
Morrow	E4	3	33.1	6	0.305	1393	0.368	0	0	0.000	268	119					125	113	
Morrow	A6	3	47.2	6	0.305	4118	0.353	0	0	0.000	335	168					202	149	
Morrow	B6	3	43.9	6	0.305	2078	0.368	0	0	0.000	324	150					157	151	
Morrow	B40B4	3	34.8	6	0.305	2078	0.368	0	0	0.000	158	134	110	118	126	117	134	140	87
Morrow	B56B2	3	14.7	6	0.305	2078	0.368	0	0	0.000	103	100	72	77	82	83	100	105	89
Morrow	E2	3	14.7	6	0.305	652	0.368	0	0	0.000	83	68	72	71	82	63	68	40	89
Morrow	A4	3	25.0	6	0.305	2754	0.375	0	0	0.000	141	133	95	103	108	111	131	127	104
Morrow	B4	3	27.2	6	0.305	2078	0.368	0	0	0.000	125	123	97	102	111	106	123	129	107
Morrow	E4	3	28.4	6	0.305	1393	0.368	0	0	0.000	112	109	99	101	114	98	109	108	87
Morrow	A6	3	39.9	6	0.305	4111	0.356	0	0	0.000	181	160	114	126	131	142	176	142	87
Morrow	B6	3	45.7	6	0.305	2074	0.372	0	0	0.000	140	147	127	130	145	130	146	154	87
Morrow	B70B2	3	16.3	6	0.305	2072	0.365	0	0	0.000	93	103	75	78	86	86	104	108	71
Morrow	A4	3	27.2	6	0.305	2763	0.368	0	0	0.000	136	135	97	102	111	114	135	129	83
Morrow	A6	3	45.0	6	0.305	4154	0.356	0	0	0.000	182	167	121	129	140	149	183	148	87
Morrow	B84B4	3	27.2	6	0.305	2084	0.363	0	0	0.000	116	123	96	97	110	106	123	127	67
Morrow	B113B	3	32.6	6	0.305	2072	0.365	0	0	0.000	112	130	106	105	121	113	131	136	51
Bhal	B1	3	23.2	30	0.240	898	0.297	0	0	0.000	71	69	57	59	57	66	71	69	88
Bhal	B2	3	29.6	30	0.240	1814	0.600	0	0	0.000	120	127	130	134	127	133	118	126	109
Bhal	B3	3	27.5	30	0.240	2722	0.900	0	0	0.000	167	167	188	194	155	182	171	167	186
Bhal	B4	3	25.2	30	0.240	3629	1.200	0	0	0.000	185	202	240	249	171	223	220	201	224
Bhal	B5	3	26.6	30	0.240	907	0.600	0	0	0.000	106	97	123	122	121	109	90	61	139
Bhal	B6	3	24.7	30	0.240	907	0.600	0	0	0.000	114	95	119	118	116	106	88	59	136
Bhal	B7	3	27.2	30	0.240	1361	0.900	0	0	0.000	140	132	187	186	154	153	135	83	186
Bhal	B8	3	27.7	30	0.240	1361	0.900	0	0	0.000	128	133	189	187	156	155	136	84	187

												V_n (kN)							
Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Aster & Koch	8	3	31.1	30	1.000	3150	0.500	0	0	0.000	287	371	463	450	483	411	355	233	280
Aster & Koch	9	3	19.9	30	1.000	3150	0.500	0	0	0.000	261	320	370	362	387	342	306	201	244
Aster & Koch	10	3	20.0	30	1.000	3150	0.500	0	0	0.000	262	320	371	363	388	342	306	201	245
Aster & Koch	17	3	28.7	30	1.000	3150	0.750	0	0	0.000	364	428	667	649	597	503	416	205	551
Chana	2.1a	3	49.3	20	0.203	1221	0.356	0	0	0.000	96	95	84	87	97	91	96	102	58
Chana	2.1b	3	49.3	20	0.203	1221	0.356	0	0	0.000	97	95	84	87	97	91	96	102	58
Chana	2.2a	3	41.6	20	0.203	1221	0.356	0	0	0.000	87	89	77	81	89	85	91	96	58
Chana	2.2b	3	41.6	20	0.203	1221	0.356	0	0	0.000	94	89	77	81	89	85	91	96	58
Chana	2.3a	3	45.2	20	0.203	1221	0.356	0	0	0.000	99	92	81	84	93	88	93	99	58
Chana	2.3b	3	45.2	20	0.203	1221	0.356	0	0	0.000	96	92	81	84	93	88	93	99	58
Heger, F.J., an	SW14-	3	49.0	19	0.914	1624	0.191	0	0	0.000	197	218	203	202	204	205	217	176	145
Heger, F.J., an	SW14-	3	49.0	19	0.914	1632	0.186	0	0	0.000	196	216	198	196	199	202	217	177	145
Heger, F.J., an	SW1B-	3	48.3	19	0.914	2085	0.184	0	0	0.000	203	233	194	195	195	210	234	225	144
Heger, F.J., an	SW1B-	3	48.3	19	0.914	2089	0.180	0	0	0.000	223	231	190	191	191	207	234	226	144
Yoon, Cook, M	N1-S	3	36.0	20	0.375	7000	0.655	0	0	0.000	249	296	245	270	232	284	293	267	197
Yoon, Cook, M	M1-S	3	67.0	10	0.375	7000	0.655	0	0	0.000	296	364	334	355	316	342	360	329	197
Yoon, Cook, M	H1-S	3	87.0	10	0.375	7000	0.655	0	0	0.000	327	397	380	399	360	379	393	359	197
Konig grimm &	s1.1	3	90.1	16	0.300	615	0.153	0	0	0.000	70	84	72	72	73	76	85	82	42
Konig grimm &	s4.1	3	110.9	16	0.300	615	0.153	0	0	0.000	74	90	80	79	81	83	91	88	42
Konig grimm &	s2.3	3	93.7	16	0.300	981	0.348	0	0	0.000	123	140	168	164	195	146	142	115	84
Konig grimm &	s3.3	3	94.4	16	0.300	1858	0.746	0	0	0.000	193	239	361	352	324	263	234	180	168
Konig grimm &	s1.2	3	91.2	16	0.300	1008	0.152	0	0	0.000	76	99	72	73	73	85	101	91	42
Konig grimm &	s4.2	3	110.9	16	0.300	1008	0.152	0	0	0.000	90	106	80	80	80	93	107	97	42
Konig grimm &	s2.2	3	91.3	16	0.300	1963	0.348	0	0	0.000	187	175	166	167	192	171	178	181	84
Konig grimm &	s3.2	3	93.7	16	0.300	3705	0.718	0	0	0.000	259	295	346	347	316	309	294	315	168
Konig grimm &	s1.3	3	93.7	16	0.300	1848	0.146	0	0	0.000	99	107	70	75	71	96	124	88	42
Konig grimm &	s4.3	3	110.9	16	0.300	1848	0.146	0	0	0.000	122	114	77	81	77	103	132	94	42
Konig grimm &	s2.4	3	94.1	16	0.300	3700	0.328	0	0	0.000	230	197	159	168	187	191	222	175	84
Konig grimm &	s3.4	3	94.1	16	0.300	7390	0.690	0	0	0.000	379	345	333	350	309	356	370	306	168
Pendyala and N	inf(30)(	3	34.0	10	0.080	516	0.140	0	0	0.000	25	20					21	16	
Pendyala and N	inf(30)(	3	34.0	10	0.080	340	0.140	0	0	0.000	17	20	11	11	11	15	18	16	9
Pendyala and N	inf(60)(	3	63.0	10	0.080	470	0.140	0	0	0.000	31	24					25	20	
Pendyala and N	inf(60)(	3	63.0	10	0.080	243	0.140	0	0	0.000	16	22	15	15	15	18	20	20	9
Pendyala and N	inf(100	3	87.0	10	0.080	507	0.140	0	0	0.000	35	27					28	22	
Pendyala and N	inf(100	3	87.0	10	0.080	261	0.140	0	0	0.000	18	25	17	17	17	21	23	22	9
Adebar	DF-1	3	21.0	20	0.500	11250	1.000	0	0	0.000	429	418	380	445			448	409	
Adebar	DF-2	3	18.4	20	0.500	8850	1.000	0	0	0.000	315	370	356	404			396	391	
Adebar	DF-2R	3	18.4	20	0.500	10600	1.000	0	0	0.000	378	392	356	417			420	391	
Adebar	DF-3	3	18.4	20	0.500	9250	1.000	0	0	0.000	329	375	356	407			402	391	
Adebar	DF-4	3	25.5	20	0.500	9250	1.000	0	0	0.000	387	418	419	467			448	436	
Adebar	DF-5	3	25.5	20	0.500	9064	0.996	0	0	0.000	381	415	418	462			445	435	
Adebar	DF-6	3	21.0	20	0.500	20250	1.000	0	0	0.000	771	460					545	409	
Adebar	DF-7	3	20.6	20	0.500	11550	1.000	0	0	0.000	435	419	377	443			449	406	
Adebar	DF-8	3	22.4	20	0.500	13500	1.000	0	0	0.000	531	454	393	473			487	418	
Adebar	DF-8R	3	22.4	20	0.500	14750	1.000	0	0	0.000	579	468	393	482			501	418	
Adebar	DF-9	3	31.7	20	0.500	11400	1.000	0	0	0.000	532	482	467	528			516	469	
Adebar	DF-10	3	31.7	20	0.500	11200	1.000	0	0	0.000	524	479	467	527			513	469	
Adebar	DF-1 C	3	31.7	20	0.500	12950	1.000	0	0	0.000	605	503	467	540			539	469	
Adebar	DF-11	3	19.5	20	0.250	9000	1.000	0	0	0.000	330	225					274	199	
Oh and Shin	H4300	3	49.1	16	0.130	5798	0.500	0	0	0.000	337	151							
Oh and Shin	H4500	3	49.1	16	0.130	1931	0.500	0	0	0.000	113	94					98	84	
Kani T-Beams	4850	2	17.9	19	0.155	2323	0.273	0	0	0.000	71	51		38	30	49	63	45	
Kani T-Beams	4851	2	17.9	19	0.155	2319	0.273	0	0	0.000	52	51		32	30	49	63	45	
Kani T-Beams	4852	2	17.9	19	0.157	2298	0.270	0	0	0.000	97	51		41	30	49	63	46	
Kani T-Beams	4853	2	17.9	19	0.155	2298	0.273	0	0	0.000	62	51		36	30	48	62	45	
Kani T-Beams	4854	2	17.9	19	0.156	2304	0.272	0	0	0.000	56	51		33	30	49	63	46	

												V_n (kN)							
Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Kani T-Beams	4857	2	17.6	19	0.153	2285	0.275	0	0	0.000	77	50		39	29	48	61	45	
Kani T-Beams	4858	2	17.6	19	0.154	2287	0.276	0	0	0.000	74	51		40	30	48	62	45	
Kani T-Beams	4830	2	19.0	19	0.155	1134	0.272	0	0	0.000	64	50		36	31	44	50	46	
Kani T-Beams	4831	2	19.0	19	0.155	1134	0.271	0	0	0.000	59	50		34	31	44	50	46	
Kani T-Beams	4832	2	19.0	19	0.156	1133	0.272	0	0	0.000	56	50		33	31	44	50	46	
Kani T-Beams	4833	2	19.2	19	0.155	1134	0.271	0	0	0.000	47	50		32	31	44	50	46	
Kani T-Beams	4834	2	18.9	19	0.155	1134	0.271	0	0	0.000	51	50	30	32	30	44	50	46	
Kani T-Beams	4836	2	18.5	19	0.153	1144	0.272	0	0	0.000	49	49	30	32	30	43	50	45	
Kani T-Beams	4837	2	18.5	19	0.154	1148	0.272	0	0	0.000	49	50	30	31	30	43	50	45	
Kani T-Beams	4838	2	18.0	19	0.152	1103	0.279	0	0	0.000	52	49	30	32	30	43	48	45	
Kani T-Beams	4839	2	18.0	19	0.152	1125	0.273	0	0	0.000	44	49	29	31	29	42	49	45	
Kani T-Beams	4840	2	18.0	19	0.152	1141	0.275	0	0	0.000	45	49	29	31	30	43	49	45	
Kani T-Beams	4840A	2	17.6	19	0.153	1132	0.274	0	0	0.000	49	49	29	33	29	42	49	45	
Kani T-Beams	4840C	2	20.6	19	0.153	1137	0.295	0	0	0.000	45	53	34	35	34	48	51	50	
Kani T-Beams	4840D	2	20.6	19	0.152	1145	0.269	0	0	0.000	54	51	31	35	31	44	51	46	
Kani T-Beams	4840E	2	20.6	19	0.153	1153	0.271	0	0	0.000	60	51	31	35	31	45	52	47	
Kani T-Beams	4810	2	18.2	19	0.154	762	0.272	0	0	0.000	47	43	30	30	30	40	43	45	
Kani T-Beams	4812	2	19.3	19	0.155	753	0.273	0	0	0.000	58	44	31	34	31	41	44	47	
Kani T-Beams	4813	2	19.3	19	0.154	762	0.269	0	0	0.000	48	44	30	32	30	40	44	46	
Kani T-Beams	4814	2	19.3	19	0.154	764	0.271	0	0	0.000	51	44	30	32	31	40	44	46	
Kani T-Beams	4815	2	19.2	19	0.156	765	0.271	0	0	0.000	53	44	31	34	31	41	44	46	
Kani T-Beams	4818	2	17.6	19	0.155	738	0.272	0	0	0.000	43	42	29	32	30	39	42	45	
Kani T-Beams	461	2	30.1	19	0.153	3412	0.275	0	0	0.000	73	60	38	42	39	65	84	54	
Kani T-Beams	462	2	30.1	19	0.153	3400	0.275	0	0	0.000	80	60	38	46	39	65	84	54	
Kani T-Beams	463	2	25.4	19	0.153	3392	0.273	0	0	0.000	109	57	35	48	35	60	79	50	
Kani T-Beams	464	2	26.3	19	0.153	3412	0.276	0	0	0.000	69	58	36	39	36	61	80	51	
Kani T-Beams	466	2	26.8	19	0.155	3412	0.279	0	0	0.000	77	59	37	47	37	63	81	53	
Kani T-Beams	467	2	26.8	19	0.156	3394	0.273	0	0	0.000	75	59	37	42	37	62	82	52	
Kani T-Beams	469	2	25.6	19	0.154	3401	0.271	0	0	0.000	140	57	35	53	35	60	80	51	
Kani T-Beams	471	2	25.4	19	0.153	3416	0.269	0	0	0.000	98	56	34	47	35	59	79	50	
Kani T-Beams	472	2	26.2	19	0.154	3396	0.267	0	0	0.000	95	57	35	48	35	60	80	50	
Kani T-Beams	473	2	26.2	19	0.154	3401	0.268	0	0	0.000	95	57	35	48	35	60	80	50	
Kani T-Beams	474	2	26.6	19	0.154	3401	0.270	0	0	0.000	90	57	36	48	36	61	81	51	
Kani T-Beams	475	2	26.6	19	0.153	3379	0.269	0	0	0.000	95	57	35	48	35	60	80	51	
Kani T-Beams	476	2	28.3	19	0.152	3381	0.269	0	0	0.000	65	58	36	38	36	62	82	51	
Kani T-Beams	478	2	27.1	19	0.152	3312	0.271	0	0	0.000	72	57	36	40	36	61	80	51	
Kani T-Beams	480	2	27.2	19	0.152	3368	0.277	0	0	0.000	77	58	36	48	37	62	80	52	
Kani T-Beams	481	2	27.2	19	0.152	3344	0.276	0	0	0.000	77	58	36	47	37	62	80	52	
Kani T-Beams	443	2	24.4	19	0.155	2329	0.250	0	0	0.000	129	53			32	52	69	47	
Kani T-Beams	444	2	24.4	19	0.155	2334	0.251	0	0	0.000	101	53	32	43	32	53	70	47	
Kani T-Beams	447	2	26.2	19	0.155	2305	0.278	0	0	0.000	63	59	37	42	37	58	71	52	
Kani T-Beams	448	2	29.9	19	0.154	2337	0.279	0	0	0.000	63	61	39	42	39	62	74	54	
Kani T-Beams	449	2	29.0	19	0.155	2284	0.266	0	0	0.000	63	59	37	39	37	59	73	52	
Kani T-Beams	450	2	29.0	19	0.154	2283	0.270	0	0	0.000	162	59			37	59	73	53	
Kani T-Beams	451	2	26.1	19	0.154	2317	0.273	0	0	0.000	57	58	36	37	36	57	71	51	
Kani T-Beams	453	2	25.9	19	0.154	2300	0.274	0	0	0.000	96	58	36	47	36	57	70	51	
Kani T-Beams	454	2	25.4	19	0.153	2313	0.270	0	0	0.000	70	56	35	43	35	56	70	50	
Kani T-Beams	455	2	26.3	19	0.154	2294	0.266	0	0	0.000	79	57	35	43	35	56	71	50	
Kani T-Beams	456	2	26.3	19	0.153	2292	0.268	0	0	0.000	85	57	35	43	35	56	70	50	
Kani T-Beams	457	2	26.6	19	0.153	2330	0.272	0	0	0.000	78	57	36	45	36	57	71	51	
Kani T-Beams	458	2	27.1	19	0.152	2277	0.268	0	0	0.000	63	57	35	37	35	57	71	50	
Kani T-Beams	459	2	27.1	19	0.152	2277	0.268	0	0	0.000	63	57	35	37	35	57	71	50	
Kani T-Beams	423	2	27.0	19	0.153	1132	0.276	0	0	0.000	109	56					56	52	
Kani T-Beams	424	2	30.1	19	0.153	1136	0.276	0	0	0.000	69	58	38	43	39	53	58	54	
Kani T-Beams	426	2	32.7	19	0.155	1138	0.275	0	0	0.000	56	60	40	41	41	55	60	56	
Kani T-Beams	428	2	27.4	19	0.154	1131	0.269	0	0	0.000	58	56	36	37	36	51	57	51	

												V_n (kN)							
Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Kani T-Beams	429	2	27.4	19	0.154	1131	0.273	0	0	0.000	64	56	37	39	37	51	57	52	
Kani T-Beams	430	2	26.3	19	0.153	1144	0.271	0	0	0.000	66	55	35	39	36	50	56	51	
Kani T-Beams	431	2	26.3	19	0.153	1140	0.270	0	0	0.000	58	55	35	37	35	50	56	51	
Kani T-Beams	432	2	26.7	19	0.153	1119	0.272	0	0	0.000	58	55	36	37	36	50	56	51	
Kani T-Beams	433	2	26.7	19	0.153	1115	0.273	0	0	0.000	57	55	36	37	36	50	56	51	
Kani T-Beams	403	2	27.4	19	0.152	752	0.275	0	0	0.000	120	49					49	52	
Kani T-Beams	404	2	26.3	19	0.154	757	0.276	0	0	0.000	56	49	36	39	36	46	49	52	
Kani T-Beams	405	2	29.4	19	0.154	754	0.275	0	0	0.000	50	51	38	40	38	48	51	53	
Kani T-Beams	406	2	26.9	19	0.154	755	0.280	0	0	0.000	49	50	37	38	37	47	49	53	
Kani T-Beams	407	2	30.1	19	0.154	754	0.275	0	0	0.000	51	51	39	39	39	49	51	54	
Kani T-Beams	409	2	31.3	19	0.155	754	0.275	0	0	0.000	43	52	40	39	40	50	52	55	
Kani T-Beams	412	2	26.5	19	0.155	759	0.269	0	0	0.000	52	49	36	36	36	46	49	51	
Kani T-Beams	4134	2	26.5	19	0.153	763	0.274	0	0	0.000	46	49	36	36	36	46	49	51	
Kani T-Beams	141	2	26.3	19	0.153	770	0.269	0	0	0.000	101	48			35	45	49	50	
Kani T-Beams	415	2	26.3	19	0.153	764	0.270	0	0	0.000	55	48	35	37	35	45	49	51	
Kani T-Beams	416	2	26.5	19	0.153	746	0.274	0	0	0.000	83	48					48	51	
Kani T-Beams	417	2	27.2	19	0.152	741	0.274	0	0	0.000	55	49	36	38	36	46	49	51	
Kani T-Beams	418	2	27.2	19	0.155	729	0.275	0	0	0.000	49	49	37	39	37	47	49	52	
Kani T-Beams	4892	2	35.0	19	0.154	3401	0.274	0	0	0.000	168	64	41	59	42	70	89	57	
Kani T-Beams	4893	2	35.0	19	0.154	3372	0.271	0	0	0.000	119	63	41	53	41	69	88	56	
Kani T-Beams	4894	2	35.0	19	0.154	3376	0.271	0	0	0.000	93	63	41	51	41	69	88	56	
Kani T-Beams	4895	2	35.7	19	0.155	3377	0.270	0	0	0.000	89	64	42	47	42	70	89	57	
Kani T-Beams	4896	2	35.0	19	0.155	3377	0.273	0	0	0.000	75	64	42	44	42	70	89	57	
Kani T-Beams	4897	2	36.6	19	0.208	3357	0.274	0	0	0.000	92	87	57	64	58	91	109	78	
Kani T-Beams	4898	2	36.6	19	0.155	3377	0.271	0	0	0.000	85	65	42	45	42	71	90	57	
Kani T-Beams	4900	2	34.8	19	0.155	3369	0.267	0	0	0.000	87	63	41	44	41	69	88	56	
Kani T-Beams	4901	2	38.3	19	0.153	3375	0.272	0	0	0.000	87	65	43	46	43	72	91	58	
Kani T-Beams	4902	2	35.1	19	0.154	3401	0.271	0	0	0.000	88	63	41	44	41	69	89	56	
Kani T-Beams	4903	2	35.1	19	0.154	3359	0.273	0	0	0.000	109	64	41	54	42	70	88	56	
Kani T-Beams	4904	2	34.5	19	0.155	3376	0.274	0	0	0.000	99	64	41	52	42	70	88	57	
Kani T-Beams	4905	2	35.0	19	0.155	3339	0.273	0	0	0.000	128	64	42	56	42	70	88	57	
Kani T-Beams	4882	2	35.4	19	0.155	2289	0.274	0	0	0.000	150	64	42	53	42	66	78	57	
Kani T-Beams	4882A	2	38.3	19	0.154	2304	0.273	0	0	0.000	127	65	43	54	43	68	80	58	
Kani T-Beams	4883	2	37.4	19	0.155	2293	0.271	0	0	0.000	85	65	43	50	43	67	80	58	
Kani T-Beams	4883A	2	34.5	19	0.154	2291	0.271	0	0	0.000	86	63	41	49	41	64	77	56	
Kani T-Beams	4884	2	37.4	19	0.155	2293	0.269	0	0	0.000	78	65	42	47	43	67	80	57	
Kani T-Beams	4885	2	37.4	19	0.156	2300	0.273	0	0	0.000	77	66	43	46	43	68	80	58	
Kani T-Beams	4887	2	37.7	19	0.156	2295	0.273	0	0	0.000	74	66	43	46	44	68	80	59	
Kani T-Beams	4888	2	37.7	19	0.156	2300	0.272	0	0	0.000	68	66	43	45	44	68	80	58	
Kani T-Beams	4888A	2	35.0	19	0.154	2287	0.273	0	0	0.000	73	64	41	43	42	65	78	56	
Kani T-Beams	4889	2	34.5	19	0.154	2283	0.272	0	0	0.000	96	63	41	50	41	65	77	56	
Kani T-Beams	4890	2	34.5	19	0.154	2286	0.268	0	0	0.000	72	62	40	42	40	64	77	55	
Kani T-Beams	4890A	2	34.5	19	0.155	2281	0.272	0	0	0.000	85	63	41	48	41	65	77	56	
Kani T-Beams	4872	2	34.8	19	0.152	1153	0.269	0	0	0.000	71	61	40	45	40	55	61	55	
Kani T-Beams	4872A	2	34.5	19	0.154	1139	0.271	0	0	0.000	77	61	41	45	41	56	61	56	
Kani T-Beams	4872B	2	34.5	19	0.154	1135	0.273	0	0	0.000	77	61	41	46	41	56	61	56	
Kani T-Beams	4873	2	34.8	19	0.153	1144	0.270	0	0	0.000	70	61	40	43	41	56	61	55	
Kani T-Beams	4873A	2	34.1	19	0.153	1136	0.271	0	0	0.000	71	60	40	43	40	55	61	55	
Kani T-Beams	4873B	2	34.1	19	0.154	1139	0.274	0	0	0.000	62	61	41	44	41	56	61	56	
Kani T-Beams	4874	2	34.8	19	0.154	1144	0.271	0	0	0.000	63	61	41	42	41	56	61	56	
Kani T-Beams	4878	2	35.4	19	0.153	1144	0.272	0	0	0.000	61	61	41	45	41	56	62	56	
Kani T-Beams	4878A	2	36.7	19	0.154	1147	0.278	0	0	0.000	69	63	43	47	43	58	63	58	
Kani T-Beams	4878B	2	36.7	19	0.155	1146	0.279	0	0	0.000	74	63	44	48	44	59	63	59	
Kani T-Beams	4879	2	35.4	19	0.154	1139	0.271	0	0	0.000	65	61	41	43	41	56	62	56	
Kani T-Beams	4880B	2	35.7	19	0.155	1138	0.271	0	0	0.000	65	62	42	45	42	57	62	57	
Kani T-Beams	4880B	2	35.7	19	0.153	1144	0.273	0	0	0.000	64	62	41	45	42	57	62	56	

												V_n (kN)							
Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Kani T-Beams	4862	2	35.4	19	0.153	761	0.269	0	0	0.000	64	53	41	43	41	51	54	56	
Kani T-Beams	4863	2	35.4	19	0.154	765	0.273	0	0	0.000	61	54	42	43	42	52	54	57	
Kani T-Beams	1864	2	35.4	19	0.153	775	0.271	0	0	0.000	49	54	41	41	41	51	54	56	
Kani T-Beams	1865	2	37.0	19	0.155	763	0.269	0	0	0.000	84	55			42	52	55	57	
Kani T-Beams	1867	2	37.0	19	0.156	765	0.274	0	0	0.000	61	55	43	44	43	53	55	58	
Kani T-Beams	1868A	2	34.4	19	0.154	775	0.272	0	0	0.000	65	54	41	43	41	51	54	56	
Kani T-Beams	4868B	2	34.4	19	0.155	770	0.270	0	0	0.000	58	54	41	43	41	51	54	56	
Kani T-Beams	4869A	2	34.4	19	0.154	772	0.271	0	0	0.000	61	53	41	42	41	51	54	56	
Kani T-Beams	4869B	2	34.4	19	0.154	765	0.270	0	0	0.000	69	53	40	42	41	51	54	56	
Kani T-Beams	6041	2	27.7	19	0.152	3875	0.275	0	0	0.000	75	58	37	44	37	63	85	52	
Kani T-Beams	6042	2	26.8	19	0.152	3880	0.271	0	0	0.000	77	57	35	41	36	62	84	51	
Kani T-Beams	6043	2	26.8	19	0.152	3870	0.268	0	0	0.000	67	57	35	39	35	61	84	50	
Kani T-Beams	6044	2	28.7	19	0.152	3879	0.275	0	0	0.000	134	59	37	58	37	65	86	52	
Kani T-Beams	6045	2	28.7	19	0.152	3869	0.277	0	0	0.000	79	59	37	49	38	65	86	53	
Kani T-Beams	6046	2	26.8	19	0.152	3875	0.275	0	0	0.000	99	58	36	51	36	63	84	51	
Kani T-Beams	6048	2	25.8	19	0.152	3872	0.271	0	0	0.000	77	56	35	37	35	61	83	50	
Kani T-Beams	6050	2	25.4	19	0.152	3876	0.273	0	0	0.000	73	56	35	39	35	61	82	50	
Kani T-Beams	6021	2	27.4	19	0.152	1874	0.274	0	0	0.000	84	58	36	43	36	56	66	51	
Kani T-Beams	6022	2	27.4	19	0.152	1849	0.274	0	0	0.000	63	58	36	40	36	56	66	51	
Kani T-Beams	6023	2	27.4	19	0.152	1865	0.272	0	0	0.000	60	58	36	38	36	56	66	51	
Kani T-Beams	6024	2	25.7	19	0.152	1856	0.275	0	0	0.000	82	57	35	42	35	54	65	50	
Kani T-Beams	6025	2	25.7	19	0.152	1859	0.273	0	0	0.000	67	57	35	40	35	54	65	50	
Kani T-Beams	6026	2	25.7	19	0.152	1864	0.275	0	0	0.000	62	57	35	37	35	55	65	50	
Kani T-Beams	6027	2	27.5	19	0.152	1862	0.271	0	0	0.000	112	58	36	45	36	56	66	51	
Kani T-Beams	6028	2	27.5	19	0.152	1859	0.273	0	0	0.000	67	58	36	38	36	56	66	51	
Kani T-Beams	6030	2	24.9	19	0.152	1874	0.271	0	0	0.000	78	56	34	40	34	53	64	49	

Beams with Shear Reinforcement

												V_n (kN)							
Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Hsiung & Frant	A	3	43.0	10	0.152	1161	0.419	36	283	0.107	110	116	109	106	110	119	117	122	49
Hsiung & Frant	B	3	43.0	10	0.305	2322	0.419	71	303	0.114	200	233	219	213	200	239	235	244	98
Hsiung & Frant	C	3	43.0	10	0.457	3483	0.419	107	283	0.107	339	348	327	319	339	358	352	366	146
Hsiung & Frant	D	3	43.0	10	0.457	3483	0.419	107	283	0.107	348	348	327	319	348	358	352	366	146
Debaiky & Elni	A1	3	24.5	19	0.120	900	0.260	48	318	0.200	72	60	45	45	72	58	62	57	
Debaiky & Elni	B1	3	24.5	19	0.120	900	0.260	48	318	0.200	68	60	45	45	68	58	62	57	
Debaiky & Elni	C1	3	28.0	19	0.120	900	0.260	48	318	0.200	71	62	47	46	71	61	63	58	55
Debaiky & Elni	D1	3	29.8	19	0.120	900	0.260	48	318	0.100	82	82	68	67	82	80	85	79	25
Debaiky & Elni	D2	3	30.6	19	0.120	900	0.260	57	318	0.200	74	67	52	51	74	66	68	63	25
Debaiky & Elni	F5	3	20.2	19	0.120	750	0.260	57	314	0.200	66	59	47	46	66	57	60	58	
Mphonde	B50-3-	3	22.1	19	0.152	1530	0.298	16	303	0.089	76	73	52	51	76	72	76	66	62
Mphonde	B50-7-	3	39.8	19	0.152	1530	0.298	16	303	0.089	94	85	64	62	94	87	89	77	36
Mphonde	B50-11	3	59.7	19	0.152	1530	0.298	16	303	0.089	98	96	74	72	98	100	100	86	36
Mphonde	B50-15	3	83.0	19	0.152	1530	0.298	16	303	0.089	111	105	85	82	111	113	109	94	36
Mphonde	B100-3	3	27.9	19	0.152	1530	0.298	36	266	0.089	95	93	72	70	95	93	97	86	96
Mphonde	B100-7	3	47.1	19	0.152	1530	0.298	36	266	0.089	94	104	84	82	94	107	109	97	36
Mphonde	B100-1	3	68.6	19	0.152	1530	0.298	36	266	0.089	152	114	94	92	152	120	119	105	36
Mphonde	B100-1	3	81.9	19	0.152	1530	0.298	36	266	0.089	116	119	100	98	116	127	125	110	36
Mphonde	B150-3	3	28.7	19	0.152	1530	0.298	51	284	0.089	139	110	90	88	139	111	115	104	36
Mphonde	B150-7	3	46.6	19	0.152	1530	0.298	51	284	0.089	133	121	101	99	133	123	127	114	36
Mphonde	B150-1	3	69.5	19	0.152	1530	0.298	51	284	0.089	161	131	112	110	161	137	138	123	36
Mphonde	B150-1	3	82.7	19	0.152	1530	0.298	51	284	0.089	150	136	118	115	150	143	143	127	36
Elzanty et al.	G6	3	20.7	13	0.178	1161	0.267	59	379	0.191	79	86	67	65	79	80	89	85	
Johnson & Rar	NO. 1	3	36.4	19	0.305	4095	0.539	64	479	0.133	339	317	288	282	339	363	316	309	130
Johnson & Rar	No. 2	3	36.4	19	0.305	4095	0.539	64	479	0.267	222	258	226	220	222	301	252	247	130
Johnson & Rar	No. 4	3	72.3	19	0.305	4095	0.539	64	479	0.267	316	310	294	284	316	376	300	295	130
Johnson & Rar	No. 5	3	55.8	19	0.305	4095	0.539	64	479	0.133	383	348	328	319	383	407	345	338	130
Johnson & Rar	No. 7	3	51.3	19	0.305	4095	0.539	64	479	0.267	281	282	257	249	281	337	274	270	130
Johnson & Rar	No. 8	3	51.3	19	0.305	4095	0.539	64	479	0.267	258	282	257	249	258	337	274	270	130
Roller & Russe	1	3	120.1	13	0.356	3276	0.559	64	407	0.216	297	374	429	411	297	453	356	403	158
Roller & Russe	6	3	72.4	13	0.457	6036	0.762	142	445	0.381	665	552	619	594	665	683	572	587	278
Roller & Russe	7	3	72.4	13	0.457	6552	0.762	142	445	0.197	788	676	737	712	788	791	711	705	278
Roller & Russe	8	3	125.3	13	0.457	6552	0.762	142	445	0.381	483	653	774	741	483	837	673	679	278
Roller & Russe	9	3	125.3	13	0.457	8190	0.762	142	445	0.197	749	807	892	860	749	980	843	798	278
Roller & Russe	10	3	125.3	13	0.457	10060	0.762	142	445	0.133	1172	958	1009	976	1172	1122	1009	914	278
Bresler & Scord	A-1	3	24.1	19	0.307	2580	0.466	64	325	0.210	234	185	163	158	234	202	189	192	203
Bresler & Scord	B-1	3	24.8	19	0.231	2580	0.461	64	325	0.191	222	165	138	135	222	177	170	160	183
Xie et al.	NNW-3	3	40.7	19	0.127	826	0.203	64	324	0.102	87	84	69	68	87	81	90	80	23
Xie et al.	NHW-3	3	98.3	19	0.127	1136	0.198	64	324	0.099	102	98	83	82	102	103	113	92	23
Xie et al.	NHW-3	3	90.0	19	0.127	1136	0.198	64	324	0.076	108	108	94	93	108	112	125	103	23
Xie et al.	NHW-3	3	103.2	19	0.127	1136	0.198	64	324	0.064	123	121	107	106	123	125	140	116	23
Xie et al.	NHW-4	3	98.8	19	0.127	1136	0.198	64	324	0.099	94	98	83	82	94	103	113	92	23
Yoon et al.	N1N	3	36.0	20	0.375	7000	0.655	100	430	0.325	457	378	331	322	457	443	376	350	197
Yoon et al.	N2S	3	36.0	20	0.375	7000	0.655	140	430	0.465	363	376	330	320	363	441	374	348	197
Yoon et al.	N2N	3	36.0	20	0.375	7000	0.655	140	430	0.325	483	411	366	357	483	478	412	384	197
Yoon et al.	M1N	3	67.0	10	0.375	7000	0.655	100	430	0.325	405	446	421	407	405	509	442	415	197
Yoon et al.	M2S	3	67.0	10	0.375	7000	0.655	140	430	0.325	552	479	455	441	552	539	478	450	197
Yoon et al.	M2N	3	67.0	10	0.375	7000	0.655	140	430	0.230	689	527	505	491	689	582	531	500	197
Yoon et al.	H1N	3	87.0	10	0.375	7000	0.655	100	430	0.325	483	479	467	451	483	560	473	445	197
Yoon et al.	H2S	3	87.0	10	0.375	7000	0.655	140	430	0.270	598	536	526	510	598	611	536	505	197
Yoon et al.	H2N	3	87.0	10	0.375	7000	0.655	140	430	0.160	721	631	627	611	721	696	642	605	197
Kriski	1	3	27.5	14	0.360	2500	0.345	51	600	0.150	249	209	178	174	249	209	218	215	
Kriski	3	3	27.5	14	0.360	2500	0.345	51	600	0.150	225	209	178	174	225	209	218	215	
Kriski	5	3	28.6	14	0.360	2500	0.345	51	600	0.150	293	211	180	176	293	211	220	217	
Kriski	7	3	70.6	14	0.360	2500	0.345	51	600	0.150	305	262	243	236	305	276	271	268	101
Kriski	9	3	73.2	14	0.360	2500	0.345	51	600	0.150	242	265	246	239	242	279	273	271	

												V_n (kN)							
Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Peng	B-1	3	29.7	14	0.280	2100	0.274	50	587	0.355	114	128	92	90	114	121	130	120	
Peng	B-2	3	30.2	14	0.280	2100	0.274	50	587	0.300	119	132	97	95	119	126	135	125	
Peng	B-3	3	31.1	14	0.280	2100	0.274	50	587	0.250	121	139	103	101	121	132	141	131	
Peng	B-4	3	31.4	14	0.280	2100	0.274	50	587	0.195	143	147	113	110	143	141	151	140	
Peng	B-5	3	30.8	14	0.280	2100	0.274	200	456	0.355	181	174	141	139	181	167	180	169	
Peng	B-6	3	27.8	14	0.280	2100	0.274	200	456	0.300	191	183	150	148	191	175	190	178	
Peng	B-7	3	30.6	14	0.280	2100	0.274	200	456	0.250	187	202	170	168	187	193	211	198	
Clark	A1 - 1	3	24.6	19	0.203	2457	0.393	142	331	0.183	224	194						187	
Clark	A1 - 2	3	23.6	19	0.203	2457	0.393	142	331	0.183	211	193						186	
Clark	A1 - 3	3	23.4	19	0.203	2457	0.393	142	331	0.183	224	192						185	
Clark	A1 - 4	3	24.8	19	0.203	2457	0.393	142	331	0.183	246	194						187	
Clark	B1 - 1	3	23.4	19	0.203	2457	0.393	142	331	0.183	281	195						185	
Clark	B1 - 2	3	25.4	19	0.203	2457	0.393	142	331	0.183	258	198						188	
Clark	B1 - 3	3	23.7	19	0.203	2457	0.393	142	331	0.183	287	196						186	
Clark	B1 - 4	3	23.3	19	0.203	2457	0.393	142	331	0.183	269	195						185	
Clark	B1 - 5	3	24.6	19	0.203	2457	0.393	142	331	0.183	243	197						187	
Clark	C1 - 1	3	25.6	19	0.203	1638	0.393	142	331	0.203	280	199						178	
Clark	C1 - 3	3	24.0	19	0.203	1638	0.393	142	331	0.203	247	197						176	
Clark	C1 - 4	3	29.0	19	0.203	1638	0.393	142	331	0.203	288	204						181	
Clark	D1 - 1	3	26.2	19	0.203	1290	0.393	142	331	0.152	303	255						209	
Clark	D1 - 3	3	24.5	19	0.203	1290	0.393	142	331	0.152	258	252						207	
Lyngberg	5A-0	1	25.7	19	0.120	2515	0.540	100	674	0.160	435	291	282	280	435	327	315	293	267
Lyngberg	5B-0	1	26.6	19	0.120	2515	0.540	100	647	0.160	435	283	274	272	435	320	306	284	269
Rangan	I-1	1	34.7	19	0.074	3480	0.563	101	485	0.050	453	208	590	590			301	595	
Rangan	I-2	1	28.7	19	0.074	3480	0.563	57	485	0.050	371	208	346	346			250	352	
Rangan	I-3	1	29.6	19	0.063	3480	0.563	101	485	0.050	369	177	581	581			219	586	
Rangan	I-4	1	33.9	19	0.064	3480	0.563	57	485	0.050	416	180	343	344			256	348	
Kong & Rangan	S1-1	3	60.4	7	0.250	2046	0.292	40	569	0.102	228	189	159	157	228	175	196	181	61
Kong & Rangan	S1-2	3	60.4	7	0.250	2046	0.292	40	569	0.102	208	189	159	157	208	175	196	181	61
Kong & Rangan	S1-3	3	60.4	7	0.250	2046	0.292	40	569	0.102	206	189	159	157	206	175	196	181	61
Kong & Rangan	S1-4	3	60.4	7	0.250	2046	0.292	40	569	0.102	278	189	159	157	278	175	196	181	61
Kong & Rangan	S1-5	3	60.4	7	0.250	2046	0.292	40	569	0.102	253	189	159	157	253	175	196	181	61
Kong & Rangan	S1-6	3	60.4	7	0.250	2046	0.292	40	569	0.102	224	189	159	157	224	175	196	181	61
Kong & Rangan	S2-1	3	68.9	7	0.250	2046	0.292	40	569	0.152	260	174	144	141	260	163	179	164	61
Kong & Rangan	S2-2	3	68.9	7	0.250	2046	0.292	40	569	0.127	233	183	153	150	233	170	188	173	61
Kong & Rangan	S2-3	3	68.9	7	0.250	2046	0.292	40	569	0.102	253	195	166	163	253	181	201	186	61
Kong & Rangan	S2-4	3	68.9	7	0.250	2046	0.292	40	569	0.102	219	195	166	163	219	181	201	186	61
Kong & Rangan	S2-5	3	68.9	7	0.250	2046	0.292	40	569	0.077	282	215	187	185	282	193	224	207	61
Kong & Rangan	S3-2	3	64.0	7	0.250	1232	0.297	25	632	0.099	178	155	146	142			159	167	
Kong & Rangan	S3-3	3	64.0	7	0.250	2046	0.293	25	632	0.099	229	174	144	141			179	165	
Kong & Rangan	S3-4	3	64.0	7	0.250	2046	0.293	25	632	0.099	175	174	144	141			179	165	
Kong & Rangan	S4-4	3	82.9	7	0.250	2046	0.292	40	569	0.102	258	203	176	172	258	191	210	194	61
Kong & Rangan	S4-6	3	82.9	7	0.250	1380	0.198	40	569	0.102	203	147	119	117	203	129	149	140	44
Kong & Rangan	S5-1	3	84.9	7	0.250	2046	0.292	40	569	0.102	242	204	177	173	242	192	211	195	61
Kong & Rangan	S5-2	3	84.9	7	0.250	2046	0.292	40	569	0.102	260	204	177	173	260	192	211	195	61
Kong & Rangan	S5-3	3	84.9	7	0.250	2046	0.292	40	569	0.102	244	204	177	173	244	192	211	195	61
Kong & Rangan	S7-1	3	71.1	7	0.250	3284	0.294	40	569	0.152	217	180	147	144	217	185	203	166	61
Kong & Rangan	S7-2	3	71.1	7	0.250	3284	0.294	40	569	0.127	205	188	156	153	205	193	212	175	61
Kong & Rangan	S7-3	3	71.1	7	0.250	3284	0.294	40	569	0.102	247	200	169	166	247	204	226	188	61
Kong & Rangan	S7-4	3	71.1	7	0.250	3284	0.294	40	569	0.082	274	216	185	182	274	218	243	205	61
Kong & Rangan	S7-5	3	71.1	7	0.250	3284	0.294	40	569	0.071	304	227	197	194	304	228	255	216	61
Kong & Rangan	S7-6	3	71.1	7	0.250	3284	0.294	40	569	0.061	311	242	212	210	311	242	271	232	61
Kong & Rangan	S8-1	3	70.9	7	0.250	2046	0.292	40	569	0.152	272	176	146	143	272	165	180	165	61
Kong & Rangan	S8-2	3	70.9	7	0.250	2046	0.292	40	569	0.127	251	184	154	151	251	172	189	174	61
Kong & Rangan	S8-3	3	70.9	7	0.250	2046	0.292	40	569	0.102	310	196	167	164	310	182	203	187	61
Kong & Rangan	S8-4	3	70.9	7	0.250	2046	0.292	40	569	0.102	266	196	167	164	266	182	203	187	61

												V_n (kN)							
Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Kong & Ranga	S8-5	3	70.9	7	0.250	2046	0.292	40	569	0.082	289	211	183	181	289	190	220	203	61
Kong & Ranga	S8-6	3	70.9	7	0.250	2046	0.292	40	569	0.071	284	223	195	192	284	204	232	215	61
Ozcebe et al.	ACI 56	3	58.0	15	0.150	1600	0.310	25	255	0.120	94	96	75	73	94	99	102	88	38
Ozcebe et al.	TH 56	3	63.0	15	0.150	1600	0.310	25	255	0.100	104	102	81	79	104	105	108	93	38
Ozcebe et al.	ACI 59	3	82.0	15	0.150	2000	0.310	25	255	0.120	97	106	86	84	97	117	120	97	38
Ozcebe et al.	TH 59	3	75.0	15	0.150	2000	0.310	25	255	0.090	119	109	89	86	119	119	123	100	38
Ozcebe et al.	TS 59	3	82.0	15	0.150	2000	0.310	25	255	0.060	125	122	103	100	125	133	138	113	38
Ozcebe et al.	ACI 36	3	75.0	15	0.150	1200	0.310	25	255	0.120	105	99	83	81	105	103	102	94	38
Ozcebe et al.	TH 36	3	75.0	15	0.150	1200	0.310	25	255	0.100	141	102	87	84	141	106	105	98	38
Ozcebe et al.	TS 36	3	75.0	15	0.150	1200	0.310	25	255	0.070	156	110	95	93	156	113	114	106	38
Ozcebe et al.	ACI 39	3	73.0	15	0.150	1400	0.310	25	255	0.120	112	103	82	80	112	105	105	94	38
Ozcebe et al.	TH 39	3	73.0	15	0.150	1400	0.310	25	255	0.080	143	110	91	88	143	113	114	102	38
Ozcebe et al.	TS 39	3	73.0	15	0.150	1400	0.310	25	255	0.060	179	118	99	97	179	120	123	110	38
Podgorniak-Sta	BM100	3	47.0	10	0.300	2100	0.925	142	508	0.600	342	321	427	412	342	394	331	264	210
Collins & Kuch	SE100	3	71.0	10	0.295	2800	0.920	200	522	0.440	516	476	598	581	516	554	500	452	207
Collins & Kuch	SE100	3	75.0	10	0.295	2800	0.920	200	522	0.440	583	481	609	591	583	561	505	456	207
Collins & Kuch	SE50A	3	74.0	10	0.169	800	0.459	51	593	0.276	139	141	161	156	139	151	142	131	59
Collins & Kuch	SE50B	3	74.0	10	0.169	800	0.459	51	593	0.276	152	141	161	156	152	151	142	131	59
Yoshida	YB2000 / 9		36.0	10	0.300	4200	1.890	645	467	2.700			776	749					420
Yoshida	YB2000 / 6		36.0	10	0.300	4200	1.890	284	467	1.350			751	723					
Angelakos et a	DB120	3	21.0	10	0.300	2800	0.925	142	508	0.600	282	287	322	313	282	328	296	267	347
Angelakos et a	DB140	3	38.0	10	0.300	2800	0.925	71	508	0.300	277	326	395	382	277	395	336	301	210
Angelakos et a	DB165	3	65.0	10	0.300	2800	0.925	71	508	0.300	452	370	483	465	452	465	380	338	210
Angelakos et a	DB180	3	80.0	10	0.300	2800	0.925	71	508	0.300	395	389	523	504	395	494	399	354	210
Adebar & Colli	ST4	3	49.3	19	0.290	1570	0.278	100	460	0.313	158	156	135	131	158	165	156	160	63
Adebar & Colli	ST5	3	49.3	19	0.290	1570	0.278	100	460	0.192	169	181	161	157	169	190	182	186	63
Adebar & Colli	ST6	3	49.3	19	0.290	1570	0.278	100	460	0.123	230	216	198	194	230	224	219	223	63
Adebar & Colli	ST18	3	49.8	19	0.290	1570	0.278	100	460	0.172	246	189	169	165	246	197	190	193	63
Adebar & Colli	ST19	3	49.8	19	0.290	1570	0.278	100	460	0.172	201	189	169	165	201	197	190	193	63
Gupta & Collin	PC1	3	62.2	10	0.489	2400	0.340	157	520	0.210	437	351	350	341			347	378	
Gupta & Collin	PC4	3	83.1	10	0.375	1800	0.335	112	509	0.210	401	279							
Gupta & Collin	PC7	3	39.9	10	0.375	1800	0.335	112	509	0.210	387	237							
Haddadin & Ma	A2	2	29.2	19	0.178	2580	0.381	64	359	0.190	195	133						124	
Haddadin & Ma	A3	2	30.1	19	0.178	2580	0.381	142	345	0.190	292	183						177	
Haddadin & Ma	A4	2	28.6	19	0.178	2580	0.381	142	345	0.102	343	262						260	
Haddadin & Ma	A5	2	26.3	19	0.178	2580	0.381	142	345	0.064	388	339						367	
Haddadin & Ma	B3	2	27.7	19	0.178	2580	0.381	142	345	0.190	272	180	157	156			207	174	
Haddadin & Ma	C2	2	27.8	19	0.178	2580	0.381	64	359	0.190	173	131	105	104			149	122	
Haddadin & Ma	C3	2	24.1	19	0.178	2580	0.381	142	345	0.190	260	177	154	152			203	171	
Haddadin & Ma	E2	2	15.2	19	0.178	2580	0.381	64	359	0.190	169	115						108	
Haddadin & Ma	E3	2	13.7	19	0.178	2580	0.381	142	345	0.190	189	162						158	
Haddadin & Ma	E4	2	13.4	19	0.178	2580	0.381	142	345	0.102	252	242						243	
Haddadin & Ma	E5	2	17.1	19	0.178	2580	0.381	142	345	0.064	308	313						356	
Haddadin & Ma	F3	2	44.9	19	0.178	2580	0.381	142	345	0.190	330	196						188	
Haddadin & Ma	G3	2	26.2	19	0.178	2580	0.381	142	456	0.190	333	209						205	
Haddadin & Ma	G4	2	26.8	19	0.178	2580	0.381	142	456	0.127	384	271						270	
Haddadin & Ma	G5	2	26.1	19	0.178	2580	0.381	142	456	0.076	428	339						399	
Haddadin & Ma	H2	2	28.2	19	0.178	3870	0.381	142	345	0.190	319	181						175	
Haddadin & Ma	J3	3	30.4	19	0.178	2580	0.381	142	345	0.190	263	183						177	
Haddadin & Ma	J5	3	32.5	19	0.178	2580	0.381	142	345	0.064	434	339						372	
Rodriguez et al	E2A1	3	25.5	19	0.152	1290	0.318	142	346	0.254	130	119						117	
Rodriguez et al	E2A2	3	19.3	19	0.152	1290	0.318	142	347	0.254	120	114						112	
Rodriguez et al	E2A3	3	20.1	19	0.156	1290	0.316	142	351	0.254	129	116						114	
Rodriguez et al	C2A1	3	22.6	19	0.154	1290	0.318	142	355	0.254	100	119						117	
Rodriguez et al	C2A2	3	22.1	19	0.157	1290	0.311	142	349	0.254	123	116						114	
Rodriguez et al	E3H1	3	24.8	19	0.152	1290	0.305	258	331	0.152	214	226						224	

												V_n (kN)							
Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	BS 8110	ACI 11-3	ACI 11-5	A23.3-Sim.	A23.3-Gen	As 3600	JSCE SP-1	SF
Rodriguez et al	E3H2	3	27.5	19	0.152	1290	0.326	258	316	0.191	190	205						197	
Rodriguez et al	C3H1	3	22.6	19	0.152	1290	0.316	258	318	0.152	190	226						223	
Rodriguez et al	C3H2	3	22.8	19	0.152	1290	0.315	258	318	0.191	174	193						189	
Gayed	300IB-	1	25.5	14	0.100	2000	0.264	50	608	0.185	95	77	65	65	95	84	57	75	
Gayed	300IB-	1	25.5	14	0.100	2000	0.264	50	608	0.185	101	77	65	65	101	84	92	75	64
Gayed	400IB-	1	25.7	14	0.100	2000	0.364	50	608	0.250	107	88	75	74	107	90	75	85	
Gayed	400IB-	1	25.7	14	0.100	2000	0.364	50	608	0.250	93	88	75	74	93	90	100	85	79
Gayed	400IB-	1	26.6	14	0.100	2000	0.364	50	608	0.250	115	89	75	75	115	91	101	86	81
Gayed	400IB-	1	27.3	14	0.100	2000	0.364	50	608	0.250	102	89	76	75	102	88	76	86	
Gayed	500IB-	1	22.7	14	0.100	2000	0.464	50	608	0.320	120	95	81	80	120	103	102	91	84
Gayed	500IB-	1	23.8	14	0.100	2000	0.464	50	608	0.320	110	96	82	81	110	104	85	92	
Gayed	600IB-	1	26.8	14	0.100	2000	0.564	50	608	0.390	133	107	92	91			94	102	
Gayed	600IB-	1	26.8	14	0.100	2000	0.564	50	608	0.390	127	107	92	91			107	102	
Gayed	600IB-	1	25.9	14	0.100	2000	0.564	50	608	0.390	149	106	92	90			107	101	
Gayed	600IB-	1	26.5	14	0.100	2000	0.564	50	608	0.390	126	107	92	91			94	101	
Gayed	600IB-	1	26.8	14	0.100	2000	0.564	50	608	0.390	127	107	92	91			107	102	
Gayed	600IB-	1	25.9	14	0.100	2000	0.564	50	608	0.390	149	106	92	90			107	101	
Gayed	600IB-	1	26.5	14	0.100	2000	0.564	50	608	0.390	126	107	92	91			94	101	

Deep Beams

												V_n			
Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_t (kN)	ACI	CAN A23.3	AS 3600	JSCE SP-1
Haddadin & Matt	A1	2	29.5	19	0.178	2580	0.381	0	0	0.000	117	318	218		
Haddadin & Matt	E1	2	13.9	19	0.178	2580	0.381	0	0	0.000	100	150	114		
Clark	A1 - 1	1	24.6	19	0.203	2457	0.393	142	331	0.183	224	303	168		
Clark	A1 - 2	1	23.6	19	0.203	2457	0.393	142	331	0.183	211	291	162		
Clark	A1 - 3	1	23.4	19	0.203	2457	0.393	142	331	0.183	224	288	161		
Clark	A1 - 4	1	24.8	19	0.203	2457	0.393	142	331	0.183	246	304	169		
Haddadin & Matt	A2	2	29.2	19	0.178	2580	0.381	64	359	0.190	195	315	217		
Haddadin & Matt	A3	2	30.1	19	0.178	2580	0.381	142	345	0.190	292	324	222		
Haddadin & Matt	A4	2	28.6	19	0.178	2580	0.381	142	345	0.102	343	308	213		
Haddadin & Matt	A5	2	26.3	19	0.178	2580	0.381	142	345	0.064	388	283	198		
Haddadin & Matt	E2	2	15.2	19	0.178	2580	0.381	64	359	0.190	169	163	123		
Haddadin & Matt	E3	2	13.7	19	0.178	2580	0.381	142	345	0.190	189	147	112		
Haddadin & Matt	E4	2	13.4	19	0.178	2580	0.381	142	345	0.102	252	144	110		
Haddadin & Matt	E5	2	17.1	19	0.178	2580	0.381	142	345	0.064	308	184	137		
Haddadin & Matt	F3	2	44.9	19	0.178	2580	0.381	142	345	0.190	330	484	308		
Haddadin & Matt	G3	2	26.2	19	0.178	2580	0.381	142	456	0.190	333	282	198		
Haddadin & Matt	G4	2	26.8	19	0.178	2580	0.381	142	456	0.127	384	288	201		
Haddadin & Matt	G5	2	26.1	19	0.178	2580	0.381	142	456	0.076	428	282	197		
Haddadin & Matt	H2	2	28.2	19	0.178	3870	0.381	142	345	0.190	319	304	223		
Gayed	600IB-39	3	26.8	14	0.100	2000	0.564	50	608	0.390			67		
Gayed	600IB-39	3	26.8	14	0.100	2000	0.564	50	608	0.390			67		
Gayed	600IB-39	3	25.9	14	0.100	2000	0.564	50	608	0.390			65		
Gayed	600IB-39	3	26.5	14	0.100	2000	0.564	50	608	0.390			67		
Walraven & Lehw	V011	1	16.1	8	0.200	814	0.360	0	0	0.000	226	219	161	164	215
Walraven & Lehw	V012	1	21.8	8	0.200	814	0.360	0	0	0.000	322	296	204	213	250
Walraven & Lehw	V013	1	22.1	8	0.200	814	0.360	0	0	0.000	344	301	206	216	252
Walraven & Lehw	V014	1	24.3	8	0.200	814	0.360	0	0	0.000	425	330	227	233	264
Walraven & Lehw	V021	1	13.9	16	0.200	814	0.360	0	0	0.000	220	189	143	144	200
Walraven & Lehw	V023	1	20.1	16	0.200	814	0.360	0	0	0.000	374	273	197	199	240
Walraven & Lehw	V024	1	25.2	16	0.200	814	0.360	0	0	0.000	396	343	234	240	269
Walraven & Lehw	V031	1	20.0	16	0.200	814	0.360	0	0	0.000	323	272	191	198	239
Walraven & Lehw	V032	1	18.2	32	0.200	814	0.360	0	0	0.000	318	248	178	183	228
Walraven & Lehw	V033	1	19.8	32	0.200	814	0.360	0	0	0.000	246	269	190	196	238
Walraven & Lehw	V034	1	26.4	32	0.200	814	0.360	0	0	0.000	437	359	242	250	275
Walraven & Lehw	V0711	1	18.1	32	0.200	486	0.160	0	0	0.000	165	123	116	128	132
Walraven & Lehw	V022	1	19.9	16	0.200	814	0.360	0	0	0.000	270	271	191	197	239
Walraven & Lehw	V511	1	19.8	16	0.200	1254	0.560	0	0	0.000	350	404	250	203	329
Walraven & Lehw	V411	1	19.4	16	0.200	1628	0.740	0	0	0.000	365	528	347	300	400
Walraven & Lehw	V211	1	20.0	16	0.200	2009	0.930	0	0	0.000	505	680	423	329	478
Morrow	B14-B2	1	14.6	6	0.305	2078	0.368	0	0	0.000	367	269	257	224	
Morrow	-E2	1	12.7	6	0.305	651	0.375	0	0	0.000	278	232	179	191	
Morrow	A4	1	22.6	6	0.305	2760	0.362	0	0	0.000	512	444	404	369	
Morrow	B4	1	26.3	6	0.305	2078	0.368	0	0	0.000	500	484	418	370	
Morrow	E4	1	28.9	6	0.305	1393	0.368	0	0	0.000	512	532	421	399	
Morrow	A6	1	45.4	6	0.305	4154	0.356	0	0	0.000	901	946	776	683	
Morrow	B6	1	46.8	6	0.305	2078	0.368	0	0	0.000	778	861	667	558	
Morrow	B21B2	1	13.9	6	0.305	2081	0.367	0	0	0.000	239	194	136		
Morrow	E2	1	11.3	6	0.305	651	0.375	0	0	0.000	212	141	88		
Morrow	A4	1	29.8	6	0.305	2763	0.368	0	0	0.000	523	405	266		
Morrow	B4	1	27.1	6	0.305	2078	0.368	0	0	0.000	396	368	242		
Morrow	E4	1	24.2	6	0.305	1381	0.365	0	0	0.000	423	340	202		
Morrow	E4R	1	31.9	6	0.305	1393	0.368	0	0	0.000	434	433	251		
Morrow	F4	1	31.4	6	0.305	1320	0.370	0	0	0.000	468	416	240		
Morrow	A6	1	45.3	6	0.305	4154	0.356	0	0	0.000	579	713	431		
Morrow	B6	1	45.5	6	0.305	2080	0.375	0	0	0.000	579	569	348		
Morrow	B28B2	1	14.7	6	0.305	2075	0.362	0	0	0.000	201	168	84		

												V_n			
Author	Beam	Cross Sectional Shape	f'_c (MPa)	a_g (mm)	b_w (m)	A_s [for shear] (mm^2)	d_s (m)	A_v (mm^2)	f_{vy} (MPa)	S (m)	V_f (kN)	ACI	CAN A23.3	AS 3600	JSCE SP-1
Morrow	E2	1	13.7	6	0.305	646	0.372	0	0	0.000	130	136	63		
Morrow	A4	1	27.5	6	0.305	2763	0.368	0	0	0.000	324	292	145		
Morrow	B4	1	32.3	6	0.305	2078	0.368	0	0	0.000	257	343	165		
Morrow	E4	1	33.1	6	0.305	1393	0.368	0	0	0.000	268	351	154		
Morrow	A6	1	47.2	6	0.305	4118	0.353	0	0	0.000	335	596	266		
Morrow	B6	1	43.9	6	0.305	2078	0.368	0	0	0.000	324	466	210		
Carlos G.	A1	1	22.0	10	0.150	1552	0.370	64	407	0.135	251	337			
Carlos G.	A2	1	22.0	10	0.150	1552	0.370	64	407	0.135	237	337			
Carlos G.	A3	1	22.0	10	0.150	1552	0.370	0	0	0.000	221	229			
Carlos G.	A4	1	22.0	10	0.150	1552	0.370	0	0	0.000	196	229			
Carlos G.	B1	1	32.4	10	0.150	1140	0.375	64	545	0.165	456	496			
Carlos G.	B2	1	32.4	10	0.150	1140	0.375	64	545	0.165	426	496			
Carlos G.	B3	1	32.4	10	0.150	1140	0.375	0	0	0.000	468	439			
Carlos G.	B4	1	32.4	10	0.150	1140	0.375	0	0	0.000	459	439			

Appendix B

B.1 SPSS output of ACI Code

B.1.1 ACI Equation 11-3

B.1.1.1 Beams without Shear Reinforcement

B.1.1.1.1 *Beams without Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	282	71.2%
	B	51	12.9%
	C	63	15.9%
Cross Sectional Shape	2	136	34.3%
	3	260	65.7%
ag	1	49	12.4%
	2	261	65.9%
	3	86	21.7%
fy	1	13	3.3%
	2	50	12.6%
	3	49	12.4%
	4	116	29.3%
	5	121	30.6%
	6	47	11.9%
Comp. R/F Check	0	31	7.8%
	1	365	92.2%
Crack control R/F	0	11	2.8%
	1	385	97.2%
Anchorage	0	274	69.2%
	1	122	30.8%
Valid		396	100.0%
Missing		0	
Total		396	
Subpopulation		381[a]	

a. The dependent variable has only one value observed in 379 (99.5%) subpopulations.

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	633.391	641.354	629.391			
Final	267.151	394.556	203.151	426.240	30	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	1504.684	730	.000
Deviance	200.378	730	1.000

Pseudo R-Square

Cox and Snell	.659
Nagelkerke	.827
McFadden	.674

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	267.151	394.556	2.032E2	.000	0	.
fcMPa	355.556	474.999	295.556	92.406	2	.000
bw_dm	263.895	383.337	203.895	.744	2	.689
ds_dm	323.774	443.216	263.774	60.623	2	.000
ad	283.789	403.231	223.789	20.638	2	.000
rhoshearlong.Steel	371.381	490.823	311.381	108.230	2	.000
CrossSectionalShape	266.829	386.271	206.829	3.678	2	.159
ag	268.247	379.726	212.247	9.096	4	.049
fy	257.825	345.416	213.825	10.675	10	.383
CrackcontrolRF	278.038	397.480	218.038	14.887	2	.001
Anchorage	263.601	383.043	203.601	.450	2	.798

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	273	8	1	96.8%
B	18	25	8	49.0%
C	1	8	54	85.7%
Overall Percentage	73.7%	10.4%	15.9%	88.9%

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)	95% Confidence Interval for Exp (B)	
								Lower Bound	Upper Bound
A	Intercept	6.210	2.480	6.268	1	.012			
	fcMPa	-.092	.020	22.095	1	.000	.912	.878	.948
	bw_dm	-.004	.258	.000	1	.988	.996	.601	1.651
	ds_dm	-.746	.165	20.544	1	.000	.474	.343	.655
	ad	-1.251	.330	14.348	1	.000	.286	.150	.547
	rhoshearlong.Steel	3.550	.645	30.317	1	.000	34.825	9.841	123.240
	[CrossSectionalShape=2]	2.617	1.585	2.727	1	.099	13.697	.613	305.957
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.	.	.
	[ag=1]	-.503	1.735	.084	1	.772	.605	.020	18.108
	[ag=2]	.570	.698	.666	1	.414	1.768	.450	6.945
	[ag=3]	0[b]	.	.	0	.	.	.	.
	[fy=1]	-2.445	2.036	1.442	1	.230	.087	.002	4.694
	[fy=2]	-1.796	2.045	.771	1	.380	.166	.003	9.137
	[fy=3]	.227	2.076	.012	1	.913	1.255	.021	73.344
	[fy=4]	-1.167	1.923	.368	1	.544	.311	.007	13.486
	[fy=5]	-1.840	2.025	.826	1	.364	.159	.003	8.406
	[fy=6]	0[b]	.	.	0	.	.	.	.
	[CrackcontrolRF=0]	3.106	1.817	2.921	1	.087	22.331	.634	786.513
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.	.	.
	[Anchorage=0]	.408	.680	.361	1	.548	1.504	.397	5.703
	[Anchorage=1]	0[b]	.	.	0	.	.	.	.

a. The reference category is: B.

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)	95% Confidence Interval for Exp (B)	
								Lower Bound	Upper Bound
C	Intercept	-20.180	2.122	90.424	1	.000			
	fcMPa	.075	.018	17.026	1	.000	1.078	1.040	1.117
	bw_dm	.155	.176	.775	1	.379	1.168	.827	1.649
	ds_dm	.563	.156	13.028	1	.000	1.755	1.293	2.383
	ad	.190	.335	.322	1	.570	1.209	.628	2.330
	rhoshearlong.Steel	-2.255	.728	9.594	1	.002	.105	.025	.437
	[CrossSectionalShape=2]	-12.544	4601.040	.000	1	.998	3.565E-6	.000	.[c]
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.	.	.
	[ag=1]	.854	1.383	.381	1	.537	2.349	.156	35.336
	[ag=2]	-1.317	.840	2.462	1	.117	.268	.052	1.388
	[ag=3]	0[b]	.	.	0	.	.	.	.
	[fy=1]	14.888	1.506	97.719	1	.000	2922551.726	152684.042	55941069.381
	[fy=2]	15.362	1.061	209.484	1	.000	4697027.411	586605.302	37609729.102
	[fy=3]	16.447	1.152	203.673	1	.000	13895385.448	1451792.251	1.330E8
	[fy=4]	15.944	1.051	230.183	1	.000	8401004.747	1071056.712	65894625.322
	[fy=5]	17.055	.000	.	1	.	25508650.229	25508650.229	25508650.229
	[fy=6]	0[b]	.	.	0	.	.	.	.
	[CrackcontrolRF=0]	-3.534	1.270	7.741	1	.005	.029	.002	.352
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.	.	.
	[Anchorage=0]	-.161	.842	.037	1	.848	.851	.164	4.432
	[Anchorage=1]	0[b]	.	.	0	.	.	.	.

a. The reference category is: B.

B.1.1.1.2 *Beams without Shear Reinforcement-SPSS output for selected explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	282	71.2%
	B	51	12.9%
	C	63	15.9%
ag	1	49	12.4%
	2	261	65.9%
	3	86	21.7%
Crack control R/F	0	11	2.8%
	1	385	97.2%
Valid		396	100.0%
Missing		0	
Total		396	
Subpopulation		380[a]	

Model Fitting Information

	Model Fitting Criteria			Likelihood Ratio Tests		
Model	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	633.391	641.354	629.391			
Final	250.878	314.581	218.878	410.513	14	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	1220.269	744	.000
Deviance	216.106	744	1.000

Pseudo R-Square

Cox and Snell	.645
Nagelkerke	.809
McFadden	.649

Classification

Observed	Predicted A	Predicted B	Predicted C	Percent Correct
A	274	7	1	97.2%
B	16	28	7	54.9%
C	4	7	52	82.5%
Overall Percentage	74.2%	10.6%	15.2%	89.4%

Likelihood Ratio Tests

	Model Fitting Criteria			Likelihood Ratio Tests		
Effect	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	250.878	314.581	2.189E2	.000	0	.
fcMPa	369.696	425.436	341.696	122.818	2	.000
ds_dm	364.340	420.079	336.340	117.462	2	.000
ad	264.300	320.039	236.300	17.421	2	.000
rhoshearlong.Steel	400.984	456.724	372.984	154.106	2	.000
ag	254.070	301.847	230.070	11.192	4	.024
CrackcontrolRF	268.053	323.793	240.053	21.175	2	.000

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	4.532	1.190	14.513	1	.000	
	fcMPa	-.105	.018	33.407	1	.000	.900
	ds_dm	-.830	.134	38.147	1	.000	.436
	ad	-.868	.232	13.952	1	.000	.420
	rhoshearlong.Steel	3.806	.600	40.231	1	.000	44.953
	[ag=1]	-.389	.864	.203	1	.652	.678
	[ag=2]	.468	.566	.683	1	.408	1.596
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	4.589	1.664	7.609	1	.006	98.404
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
C	Intercept	-4.187	1.663	6.339	1	.012	
	fcMPa	.074	.017	19.750	1	.000	1.077
	ds_dm	.595	.139	18.404	1	.000	1.813
	ad	.244	.297	.672	1	.412	1.276
	rhoshearlong.Steel	-2.000	.584	11.735	1	.001	.135
	[ag=1]	.090	.867	.011	1	.918	1.094
	[ag=2]	-1.660	.669	6.161	1	.013	.190
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	-3.645	1.222	8.891	1	.003	.026
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.

a. The reference category is: B.

B.1.1.2 Beams with Shear Reinforcement

B.1.1.2.1 *Beams with Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	111	77.6%
	B	19	13.3%
	C	13	9.1%
Cross sectional shape	1	18	12.6%
	3	125	87.4%
ag	2	90	62.9%
	3	53	37.1%
fy	1	4	2.8%
	2	19	13.3%
	3	65	45.5%
	4	46	32.2%
	5	9	6.3%
Crack Controal Check	0	2	1.4%
	1	141	98.6%
Anchorage	0	118	82.5%
	1	25	17.5%
fvy	1	17	11.9%
	2	47	32.9%
	3	35	24.5%
	4	22	15.4%
	5	22	15.4%
Valid		143	100.0%
Missing		0	
Total		143	
Subpopulation		128[a]	

Classification

Observed	Predicted A	Predicted B	Predicted C	Percent Correct
A	111	0	0	100.0%
B	0	18	1	94.7%
C	0	0	13	100.0%
Overall Percentage	77.6%	12.6%	9.8%	99.3%

Likelihood Ratio Tests

	Model Fitting Criteria			Likelihood Ratio Tests		
Effect	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	83.962	202.475	3.962[a]	.000	0	.
fcMPa	84.472	197.060	8.472[b]	4.510	2	.105
bwm	79.934	192.522	3.934[b]	.	2	.
sd	105.853	218.441	29.853[b]	25.891	2	.000
ad	80.084	192.672	4.084[c]	.122	2	.941
rhoshearlong.Steel	110.746	223.334	34.746[b]	30.784	2	.000
rhostirrup	119.410	231.998	43.410[d]	39.448	2	.000
dsm	80.836	193.424	4.836[b]	.875	2	.646
Crosssectionalshape	118.760	231.348	42.760[b]	38.798	2	.000
ag	80.918	193.506	4.918[b]	.956	2	.620
fy	68.500	163.311	4.500[b]	.538	8	1.000
CrackControalCheck	85.211	197.799	9.211[c]	5.250	2	.072
Anchorage	2.413E4	2.424E4	2.405E4	2.405E4	2	.000
fvy	115.278	210.089	51.278[b]	47.316	8	.000

B.1.1.2.2 *Beams with Shear Reinforcement-SPSS output for Stepwise Logistic Regression*

Case Processing Summary

		N	Marginal Percentage
Y	A	111	77.6%
	B	19	13.3%
	C	13	9.1%
Cross sectional shape	1	18	12.6%
	3	125	87.4%
ag	2	90	62.9%
	3	53	37.1%
fy	1	4	2.8%
	2	19	13.3%
	3	65	45.5%
	4	46	32.2%
	5	9	6.3%
Crack Controal Check	0	2	1.4%
	1	141	98.6%
Anchorage	0	118	82.5%
	1	25	17.5%
fvy	1	17	11.9%
	2	47	32.9%
	3	35	24.5%
	4	22	15.4%
	5	22	15.4%
Valid		143	100.0%
Missing		0	
Total		143	
Subpopulation		128[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	199.280	205.206	195.280			
Final	121.484	174.815	85.484	109.796	16	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	492.408	238	.000
Deviance	85.484	238	1.000

Pseudo R-Square

Cox and Snell	.536
Nagelkerke	.720
McFadden	.562

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	108	2	1	97.3%
B	6	10	3	52.6%
C	2	0	11	84.6%
Overall Percentage	81.1%	8.4%	10.5%	90.2%

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.
A	Intercept	10.753	2.447	19.310	1	.000
	dsm	-1.500	.354	17.936	1	.000
	[CrackControalCheck=0]	-19.123	6723.876	.000	1	.998
	[CrackControalCheck=1]	0[c]	.	.	0	.
	[ag=2]	1.770	.886	3.995	1	.046
	[ag=3]	0[c]	.	.	0	.
	[fy=1]	11.577	1619.903	.000	1	.994
	[fy=2]	-5.202	1.561	11.104	1	.001
	[fy=3]	-2.307	1.349	2.928	1	.087
	[fy=4]	-4.693	1.565	8.990	1	.003
	[fy=5]	0[c]	.	.	0	.
	rhostirrup	-.057	.089	.412	1	.521
C	Intercept	-20.146	1193.393	.000	1	.987
	dsm	.797	.630	1.600	1	.206
	[CrackControalCheck=0]	-24.322	.000	.	1	.
	[CrackControalCheck=1]	0[c]	.	.	0	.
	[ag=2]	-3.688	2.564	2.069	1	.150
	[ag=3]	0[c]	.	.	0	.
	[fy=1]	19.339	2449.211	.000	1	.994
	[fy=2]	14.042	1193.387	.000	1	.991
	[fy=3]	17.849	1193.387	.000	1	.988
	[fy=4]	3.333	1272.669	.000	1	.998
	[fy=5]	0[c]	.	.	0	.
	rhostirrup	.122	.090	1.810	1	.179

a. The reference category is: B.

B.1.2 ACI Equation 11-5

B.1.2.1 Beams without Shear Reinforcement

B.1.2.1.1 *Beams without Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	272	68.7%
	B	57	14.4%
	C	67	16.9%
Cross Sectional Shape	2	136	34.3%
	3	260	65.7%
ag	1	49	12.4%
	2	261	65.9%
	3	86	21.7%
fy	1	13	3.3%
	2	50	12.6%
	3	49	12.4%
	4	116	29.3%
	5	121	30.6%
	6	47	11.9%
Crack control R/F	0	11	2.8%
	1	385	97.2%
Anchorage	0	274	69.2%
	1	122	30.8%
Valid		396	100.0%
Missing		0	
Total		396	
Subpopulation		381[a]	

Model Fitting Information

Model	Model Fitting Criteria AIC	BIC	-2 Log Likelihood	Likelihood Ratio Tests Chi-Square	df	Sig.
Intercept Only	664.614	672.577	660.614			
Final	254.146	381.552	190.146	470.468	30	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	741.281	730	.378
Deviance	187.374	730	1.000

Pseudo R-Square

Cox and Snell	.695
Nagelkerke	.855
McFadden	.709

Likelihood Ratio Tests

Effect	Model Fitting Criteria AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Likelihood Ratio Tests Chi-Square	df	Sig.
Intercept	254.146	381.552	1.901E2	.000	0	.
fcMPa	335.229	454.672	275.229	85.083	2	.000
bw_dm	251.528	370.970	191.528	1.381	2	.501
ds_dm	325.653	445.095	265.653	75.506	2	.000
rhoshearlong.Steel	333.242	452.684	273.242	83.095	2	.000
ad	258.703	378.146	198.703	8.557	2	.014
CrossSectionalShape	255.534	374.977	195.534	5.388	2	.068
ag	264.200	375.680	208.200	18.054	4	.001
fy	246.865	334.456	202.865	12.718	10	.240
CrackcontrolRF	270.762	390.204	210.762	20.616	2	.000
Anchorage	253.600	373.042	193.600	3.453	2	.178

Classification

Observed	Predicted A	B	C	Percent Correct
A	263	8	1	96.7%
B	17	33	7	57.9%
C	1	6	60	89.6%
Overall Percentage	71.0%	11.9%	17.2%	89.9%

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)	95% Confidence Interval for Exp (B)	
								Lower Bound	Upper Bound
A	Intercept	6.442	2.678	5.787	1	.016			
	fcMPa	-.094	.020	22.034	1	.000	.910	.875	.946
	bw_dm	.194	.277	.489	1	.484	1.214	.705	2.090
	ds_dm	-.942	.201	22.019	1	.000	.390	.263	.578
	rhoshearlong.Steel	3.079	.603	26.099	1	.000	21.743	6.672	70.858
	ad	-.817	.314	6.772	1	.009	.442	.239	.817
	[CrossSectionalShape=2]	19.343	6313.327	.000	1	.998	2.515E8	.000	.[b]
	[CrossSectionalShape=3]	0[c]	.	.	0	.	.	.	.
	[ag=1]	-1.148	1.915	.359	1	.549	.317	.007	13.541
	[ag=2]	-.206	.726	.080	1	.777	.814	.196	3.381
	[ag=3]	0[c]	.	.	0	.	.	.	.
	[fy=1]	-4.746	1.945	5.953	1	.015	.009	.000	.393
	[fy=2]	-2.737	2.100	1.699	1	.192	.065	.001	3.967
	[fy=3]	-1.211	2.119	.327	1	.568	.298	.005	18.967
	[fy=4]	-1.691	2.012	.706	1	.401	.184	.004	9.518
	[fy=5]	-2.789	2.112	1.745	1	.186	.061	.001	3.854
	[fy=6]	0[c]	.	.	0	.	.	.	.
	[CrackcontrolRF=0]	4.412	2.074	4.524	1	.033	82.443	1.414	4807.740
	[CrackcontrolRF=1]	0[c]	.	.	0	.	.	.	.
	[Anchorage=0]	.955	.694	1.895	1	.169	2.599	.667	10.125
	[Anchorage=1]	0[c]	.	.	0	.	.	.	.

a. The reference category is: B.

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)	95% Confidence Interval for Exp (B) Lower Bound	95% Confidence Interval for Exp (B) Upper Bound
C	Intercept	-18.102	2.446	54.750	1	.000			
	fcMPa	.078	.020	14.919	1	.000	1.081	1.039	1.125
	bw_dm	.211	.211	1.007	1	.316	1.235	.817	1.867
	ds_dm	.693	.190	13.255	1	.000	1.999	1.377	2.903
	rhoshearlong.Steel	-2.344	.729	10.334	1	.001	.096	.023	.401
	ad	-.554	.545	1.032	1	.310	.575	.197	1.674
	[CrossSectionalShape=2]	7.362	8085.842	.000	1	.999	1575.646	.000	.[b]
	[CrossSectionalShape=3]	0[c]	.	.	0	.	.	.	.
	[ag=1]	.766	1.744	.193	1	.660	2.151	.071	65.607
	[ag=2]	-2.867	1.187	5.838	1	.016	.057	.006	.582
	[ag=3]	0[c]	.	.	0	.	.	.	.
	[fy=1]	13.750	1.663	68.402	1	.000	936853.891	36015.873	24369677.520
	[fy=2]	15.167	1.027	217.906	1	.000	3862674.881	515609.159	28937145.451
	[fy=3]	15.802	1.132	194.744	1	.000	7291996.374	792448.842	67099866.013
	[fy=4]	14.466	1.249	134.146	1	.000	1917311.018	165774.601	22175179.511
	[fy=5]	16.049	.000	.	1	.	9335381.376	9335381.376	9335381.376
	[fy=6]	0[c]	.	.	0	.	.	.	.
	[CrackcontrolRF=0]	-4.514	1.440	9.828	1	.002	.011	.001	.184
	[CrackcontrolRF=1]	0[c]	.	.	0	.	.	.	.
	[Anchorage=0]	1.549	1.234	1.575	1	.209	4.708	.419	52.903
	[Anchorage=1]	0[c]	.	.	0	.	.	.	.

a. The reference category is: B.

B.1.2.1.2 *Beams without Shear Reinforcement-SPSS output for selected explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	272	68.7%
	B	57	14.4%
	C	67	16.9%
ag	1	49	12.4%
	2	261	65.9%
	3	86	21.7%
Crack control R/F	0	11	2.8%
	1	385	97.2%
Valid		396	100.0%
Missing		0	
Total		396	
Subpopulation		380[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	664.614	672.577	660.614			
Final	246.065	309.768	214.065	446.549	14	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	1271.435	744	.000
Deviance	211.293	744	1.000

Pseudo R-Square

Cox and Snell	.676
Nagelkerke	.832
McFadden	.673

Classification

Observed	Predicted A	Predicted B	Predicted C	Predicted Percent Correct
A	265	7	0	97.4%
B	17	30	10	52.6%
C	1	10	56	83.6%
Overall Percentage	71.5%	11.9%	16.7%	88.6%

Likelihood Ratio Tests

	Model Fitting Criteria			Likelihood Ratio Tests		
Effect	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	246.065	309.768	2.141E2	.000	0	.
fcMPa	362.066	417.806	334.066	120.001	2	.000
ds_dm	414.424	470.164	386.424	172.359	2	.000
rhoshearlong.Steel	374.756	430.496	346.756	132.691	2	.000
ad	247.891	303.630	219.891	5.825	2	.054
ag	254.982	302.759	230.982	16.917	4	.002
CrackcontrolRF	268.032	323.771	240.032	25.966	2	.000

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	4.886	1.248	15.317	1	.000	
	fcMPa	-.106	.018	34.259	1	.000	.899
	ds_dm	-1.018	.157	42.185	1	.000	.361
	rhoshearlong.Steel	3.316	.541	37.544	1	.000	27.537
	ad	-.540	.219	6.097	1	.014	.583
	[ag=1]	-.836	.860	.944	1	.331	.433
	[ag=2]	.000	.566	.000	1	.999	1.000
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	5.083	1.861	7.462	1	.006	161.246
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
C	Intercept	-4.051	1.861	4.736	1	.030	
	fcMPa	.081	.019	17.562	1	.000	1.085
	ds_dm	.802	.180	19.974	1	.000	2.231
	rhoshearlong.Steel	-2.153	.604	12.707	1	.000	.116
	ad	-.123	.362	.116	1	.734	.884
	[ag=1]	.860	1.020	.712	1	.399	2.364
	[ag=2]	-2.061	.731	7.952	1	.005	.127
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	-4.820	1.415	11.613	1	.001	.008
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.

a. The reference category is: B.

B.1.2.2 Beams with Shear Reinforcement

B.1.2.2.1 *Beams with Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	50	35.0%
	B	63	44.1%
	C	30	21.0%
Cross sectional shape	1	18	12.6%
	3	125	87.4%
ag	2	90	62.9%
	3	53	37.1%
fy	1	4	2.8%
	2	19	13.3%
	3	65	45.5%
	4	46	32.2%
	5	9	6.3%
Crack Controal Check	0	2	1.4%
	1	141	98.6%
Anchorage	0	118	82.5%
	1	25	17.5%
fvy	1	17	11.9%
	2	47	32.9%
	3	35	24.5%
	4	22	15.4%
	5	22	15.4%
Valid		143	100.0%
Missing		0	
Total		143	
Subpopulation		128[a]	

Model Fitting Information

Model	Model Fitting Criteria AIC	BIC	-2 Log Likelihood	Likelihood Ratio Tests Chi-Square	df	Sig.
Intercept Only	293.717	299.643	289.717			
Final	227.439	345.953	147.439	142.277	38	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	188.757	216	.910
Deviance	138.286	216	1.000

Pseudo R-Square

Cox and Snell	.630
Nagelkerke	.717
McFadden	.471

Likelihood Ratio Tests

Effect	Model Fitting Criteria AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Likelihood Ratio Tests Chi-Square	df	Sig.
Intercept	227.439	345.953	1.474E2	.000	0	.
fcMPa	228.312	340.900	152.312	4.872	2	.087
bwm	224.673	337.261	148.673	1.233	2	.540
sd	227.523	340.111	151.523	4.084	2	.130
ad	229.706	342.294	153.706	6.267	2	.044
rhoshearlong.Steel	224.802	337.391	148.802	1.363	2	.506
rhostirrup	225.978	338.567	149.978	2.539	2	.281
dsm	238.584	351.173	162.584	15.145	2	.001
Crosssectionalshape	224.811	337.400	148.811	1.372	2	.504
ag	227.541	340.129	151.541	4.101	2	.129
fy	246.110	340.921	182.110	34.671	8	.000
CrackControalCheck	227.843	340.431	151.843	4.404	2	.111
Anchorage	226.822	339.410	150.822	3.382	2	.184
fvy	233.513	328.324	169.513	22.073	8	.005

Classification

Observed	Predicted A	B	C	Percent Correct
A	38	12	0	76.0%
B	8	49	6	77.8%
C	0	11	19	63.3%
Overall Percentage	32.2%	50.3%	17.5%	74.1%

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)	95% Confidence Interval for Exp (B)	
								Lower Bound	Upper Bound
A	Intercept	30.055	10.526	8.152	1	.004			
	fcMPa	-.016	.020	.668	1	.414	.984	.946	1.023
	bwm	-7.933	7.278	1.188	1	.276	.000	2.290E-10	562.317
	sd	-.339	.249	1.846	1	.174	.713	.437	1.162
	ad	-2.246	1.225	3.363	1	.067	.106	.010	1.167
	rhoshearlong.Steel	-.004	.831	.000	1	.996	.996	.195	5.082
	rhostirrup	-.230	.362	.403	1	.525	.795	.391	1.615
	dsm	-.947	.793	1.428	1	.232	.388	.082	1.834
	[Crosssectionalshape=1]	1.707	3.357	.259	1	.611	5.514	.008	3969.449
	[Crosssectionalshape=3]	0[b]	.	.	0	.	.	.	.
	[ag=2]	-1.277	1.152	1.228	1	.268	.279	.029	2.668
	[ag=3]	0[b]	.	.	0	.	.	.	.
	[fy=1]	10.206	7184.517	.000	1	.999	27071.159	.000	.[c]
	[fy=2]	-3.453	3.265	1.119	1	.290	.032	5.261E-5	19.031
	[fy=3]	-12.185	6.466	3.552	1	.059	5.106E-6	1.601E-11	1.628
	[fy=4]	-13.754	6.301	4.765	1	.029	1.063E-6	4.605E-12	.245
	[fy=5]	0[b]	.	.	0	.	.	.	.
	[CrackControalCheck=0]	-20.601	.000	.	1	.	1.130E-9	1.130E-9	1.130E-9
	[CrackControalCheck=1]	0[b]	.	.	0	.	.	.	.
	[Anchorage=0]	-.871	1.318	.437	1	.509	.418	.032	5.537
	[Anchorage=1]	0[b]	.	.	0	.	.	.	.
	[fvy=1]	-4.474	2.233	4.012	1	.045	.011	.000	.908
	[fvy=2]	-.566	1.791	.100	1	.752	.568	.017	19.008
	[fvy=3]	-10.877	4.741	5.263	1	.022	1.888E-5	1.739E-9	.205
	[fvy=4]	-1.605	1.065	2.270	1	.132	.201	.025	1.620
	[fvy=5]	0[b]	.	.	0	.	.	.	.
C	Intercept	-3.208	4.679	.470	1	.493			
	fcMPa	.036	.020	3.109	1	.078	1.036	.996	1.078
	bwm	-4.258	6.876	.383	1	.536	.014	1.987E-8	10083.701
	sd	.253	.227	1.242	1	.265	1.287	.826	2.007
	ad	.835	.967	.745	1	.388	2.305	.346	15.336
	rhoshearlong.Steel	-.879	.798	1.213	1	.271	.415	.087	1.985
	rhostirrup	.225	.177	1.616	1	.204	1.253	.885	1.774
	dsm	1.049	.467	5.041	1	.025	2.854	1.143	7.132
	[Crosssectionalshape=1]	5.357	5.710	.880	1	.348	212.038	.003	15361281.402
	[Crosssectionalshape=3]	0[b]	.	.	0	.	.	.	.
	[ag=2]	-2.541	1.430	3.158	1	.076	.079	.005	1.299
	[ag=3]	0[b]	.	.	0	.	.	.	.
	[fy=1]	.581	.000	.	1	.	1.787	1.787	1.787
	[fy=2]	4.228	2.190	3.729	1	.053	68.611	.938	5016.397
	[fy=3]	.322	1.806	.032	1	.859	1.379	.040	47.560
	[fy=4]	.629	2.511	.063	1	.802	1.876	.014	257.273
	[fy=5]	0[b]	.	.	0	.	.	.	.
	[CrackControalCheck=0]	-5.555	3.053	3.310	1	.069	.004	9.744E-6	1.536
	[CrackControalCheck=1]	0[b]	.	.	0	.	.	.	.
	[Anchorage=0]	-3.009	1.741	2.986	1	.084	.049	.002	1.497
	[Anchorage=1]	0[b]	.	.	0	.	.	.	.
	[fvy=1]	-5.516	4.956	1.239	1	.266	.004	2.432E-7	66.560
	[fvy=2]	-1.799	1.981	.825	1	.364	.165	.003	8.028
	[fvy=3]	-3.192	2.581	1.530	1	.216	.041	.000	6.464
	[fvy=4]	-1.962	1.602	1.500	1	.221	.141	.006	3.247
	[fvy=5]	0[b]	.	.	0	.	.	.	.

a. The reference category is: B.

B.1.3 ACI Deep Beams

B.1.3.1 *Deep beams-SPSS output for with all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	45	68.2%
	B	8	12.1%
	C	13	19.7%
CrossSectionalShape	1.00	15	22.7%
	2.00	51	77.3%
ag	0	4	6.1%
	2	27	40.9%
	3	35	53.0%
fy	2	15	22.7%
	3	27	40.9%
	4	20	30.3%
	6	4	6.1%
Anchorage	.00	28	42.4%
	1.00	38	57.6%
Crack Control check	0	18	27.3%
	1	48	72.7%
Valid		66	100.0%
Missing		531	
Total		597	
Subpopulation		62[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	111.702	116.082	107.702			
Final	73.358	130.289	21.358	86.344	24	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	611.707	98	.000
Deviance	18.586	98	1.000

Pseudo R-Square

Cox and Snell	.730
Nagelkerke	.898
McFadden	.782

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	73.358	130.289	21.358[a]	.000	0	.
fcMPa	70.030	122.582	22.030[b]	.672	2	.715
bwm	55.602	108.154	7.602[b]	.	2	.
dsm	55.635	108.187	7.635[b]	.	2	.
rhoshearlong.Steel	60.093	112.645	12.093[b]	.	2	.
StrutAngle	72.620	125.172	24.620[b]	3.262	2	.196
CrossSectionalShape	73.358	130.289	21.358[a]	.000	0	.
ag	136.837	185.009	92.837[c]	71.478	4	.000
fy	59.324	107.496	15.324[b]	.	4	.
Anchorage	75.106	127.658	27.106[c]	5.748	2	.056
CrackControlcheck	70.419	122.970	22.419[b]	1.060	2	.588

B.1.3.2 *Deep beams-SPSS output Stepwise Logistic regression.*

Case Processing Summary

		N	Marginal Percentage
Y	A	45	68.2%
	B	8	12.1%
	C	13	19.7%
CrossSectionalShape	1.00	15	22.7%
	2.00	51	77.3%
ag	0	4	6.1%
	2	27	40.9%
	3	35	53.0%
fy	2	15	22.7%
	3	27	40.9%
	4	20	30.3%
	6	4	6.1%
Anchorage	.00	28	42.4%
	1.00	38	57.6%
Crack Control check	0	18	27.3%
	1	48	72.7%
Valid		66	100.0%
Missing		531	
Total		597	
Subpopulation		62[a]	

Model Fitting Information

Model	Model Fitting Criteria AIC	BIC	-2 Log Likelihood	Likelihood Ratio Tests Chi-Square	df	Sig.
Intercept Only	111.702	116.082	107.702			
Final	59.636	90.291	31.636	76.067	12	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	36.730	110	1.000
Deviance	28.863	110	1.000

Pseudo R-Square

Cox and Snell	.684
Nagelkerke	.842
McFadden	.689

Likelihood Ratio Tests

Effect	Model Fitting Criteria AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Likelihood Ratio Tests Chi-Square	df	Sig.
Intercept	59.636	90.291	31.636[a]	.000	0	.
fcMPa	66.128	92.404	42.128	10.492	2	.005
fy	89.995	107.512	73.995	42.359	6	.000
CrackControlcheck	70.455	96.731	46.455	14.820	2	.001
bwm	106.662	132.938	82.662	51.026	2	.000

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	-204.830	6270.777	.001	1	.974	
	fcMPa	-.714	.550	1.682	1	.195	.490
	[fy=2]	27.262	6227.078	.000	1	.997	6.911E11
	[fy=3]	-59.128	6270.799	.000	1	.992	2.095E-26
	[fy=4]	42.075	6271.825	.000	1	.995	1.874E18
	[fy=6]	0[c]	.	.	0	.	.
	[CrackControlcheck=0]	25.117	739.003	.001	1	.973	8.094E10
	[CrackControlcheck=1]	0[c]	.	.	0	.	.
	bwm	977.184	.000	.	1	.	.[b]
C	Intercept	23.095	2.506	84.903	1	.000	
	fcMPa	.073	.093	.611	1	.435	1.075
	[fy=2]	-21.186	1.762	144.500	1	.000	6.294E-10
	[fy=3]	-20.527	1.909	115.571	1	.000	1.217E-9
	[fy=4]	-23.491	.000	.	1	.	6.280E-11
	[fy=6]	0[c]	.	.	0	.	.
	[CrackControlcheck=0]	-.297	1.200	.061	1	.805	.743
	[CrackControlcheck=1]	0[c]	.	.	0	.	.
	bwm	-19.432	19.602	.983	1	.322	3.636E-9

a. The reference category is: B.

B.2 SPSS output of BS 8110

B.2.1 Beams without Shear Reinforcement

B.2.1.1 *Beams without Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	280	53.4%
	B	173	33.0%
	C	71	13.5%
Cross Sectional Shape	2	143	27.3%
	3	381	72.7%
ag	1	129	24.6%
	2	284	54.2%
	3	111	21.2%
fy	1	24	4.6%
	2	51	9.7%
	3	144	27.5%
	4	128	24.4%
	5	131	25.0%
	6	46	8.8%
Crack control R/F	0	11	2.1%
	1	513	97.9%
AnchorageCheck	0	449	85.7%
	1	75	14.3%
Valid		524	100.0%
Missing		0	
Total		524	
Subpopulation		508[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	1.013E3	1.021E3	1.009E3			
Final	656.228	792.596	592.228	416.289	30	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	1377.064	984	.000
Deviance	583.571	984	1.000

Pseudo R-Square

Cox and Snell	.548
Nagelkerke	.640
McFadden	.409

Likelihood Ratio Tests

	Model Fitting Criteria			Likelihood Ratio Tests		
Effect	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	656.228	792.596	5.922E2	.000	0	.
fcMPa	731.548	859.393	671.548	79.320	2	.000
bw_dm	653.428	781.272	593.428	1.199	2	.549
ds_dm	652.602	780.447	592.602	.374	2	.830
ad	743.501	871.346	683.501	91.273	2	.000
rhoshearlong.Steel	769.578	897.423	709.578	117.350	2	.000
CrossSectionalShape	668.614	796.459	608.614	16.386	2	.000
ag	651.318	770.639	595.318	3.089	4	.543
fy	674.775	768.528	630.775	38.547	10	.000
CrackcontrolRF	691.324	819.168	631.324	39.095	2	.000
AnchorageCheck	660.283	788.128	600.283	8.054	2	.018

Classification

	Predicted			
Observed	A	B	C	Percent Correct
A	233	44	3	83.2%
B	35	122	16	70.5%
C	5	20	46	64.8%
Overall Percentage	52.1%	35.5%	12.4%	76.5%

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	1.406	.961	2.139	1	.144	
	fcMPa	-.033	.010	11.190	1	.001	.968
	bw_dm	-.037	.148	.062	1	.803	.964
	ds_dm	-.032	.083	.153	1	.696	.968
	ad	-.669	.099	45.869	1	.000	.512
	rhoshearlong.Steel	1.036	.140	54.664	1	.000	2.817
	[CrossSectionalShape=2]	.549	.408	1.808	1	.179	1.731
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	.299	.506	.350	1	.554	1.349
	[ag=2]	.509	.411	1.535	1	.215	1.663
	[ag=3]	0[b]	.	.	0	.	.
	[fy=1]	1.812	.661	7.509	1	.006	6.124
	[fy=2]	.189	.839	.051	1	.822	1.208
	[fy=3]	1.302	.587	4.920	1	.027	3.676
	[fy=4]	1.064	.520	4.183	1	.041	2.898
	[fy=5]	.872	.541	2.597	1	.107	2.392
	[fy=6]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	3.936	1.020	14.878	1	.000	51.196
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
	[AnchorageCheck=0]	-1.656	.656	6.370	1	.012	.191
	[AnchorageCheck=1]	0[b]	.	.	0	.	.
C	Intercept	-19.277	1.281	226.400	1	.000	
	fcMPa	.059	.010	33.275	1	.000	1.061
	bw_dm	.162	.167	.942	1	.332	1.176
	ds_dm	-.049	.091	.290	1	.590	.952
	ad	.802	.208	14.892	1	.000	2.231
	rhoshearlong.Steel	-.727	.252	8.328	1	.004	.483
	[CrossSectionalShape=2]	-3.922	1.370	8.198	1	.004	.020
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	.999	.779	1.643	1	.200	2.715
	[ag=2]	.413	.453	.830	1	.362	1.511
	[ag=3]	0[b]	.	.	0	.	.
	[fy=1]	-1.812	1.273	2.026	1	.155	.163
	[fy=2]	1.888	1.005	3.531	1	.060	6.608
	[fy=3]	-.621	1.021	.370	1	.543	.538
	[fy=4]	-.012	.833	.000	1	.989	.988
	[fy=5]	.899	.941	.913	1	.339	2.457
	[fy=6]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	-21.657	5934.815	.000	1	.997	3.931E-10
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
	[AnchorageCheck=0]	13.636	.000	.	1	.	835475.491
	[AnchorageCheck=1]	0[b]	.	.	0	.	.

a. The reference category is: B.

B.2.1.2 Beams without Shear Reinforcement-SPSS output for selected explanatory variables

Case Processing Summary

		N	Marginal Percentage
Y	A	280	53.4%
	B	173	33.0%
	C	71	13.5%
Cross Sectional Shape	2	143	27.3%
	3	381	72.7%
fy	1	24	4.6%
	2	51	9.7%
	3	144	27.5%
	4	128	24.4%
	5	131	25.0%
	6	46	8.8%
Crack control R/F	0	11	2.1%
	1	513	97.9%
AnchorageCheck	0	449	85.7%
	1	75	14.3%
Valid		524	100.0%
Missing		0	
Total		524	
Subpopulation		507[a]	

Model Fitting Information

	Model Fitting Criteria			Likelihood Ratio Tests		
Model	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	1.013E3	1.021E3	1.009E3			
Final	644.847	747.123	596.847	411.671	22	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	1325.805	990	.000
Deviance	588.189	990	1.000

Pseudo R-Square

Cox and Snell	.544
Nagelkerke	.635
McFadden	.404

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.
A	Intercept	1.597	.764	4.368	1	.037
	fcMPa	-.034	.009	13.004	1	.000
	ad	-.685	.098	49.025	1	.000
	rhoshearlong.Steel	1.060	.139	57.915	1	.000
	[CrossSectionalShape=2]	.737	.380	3.758	1	.053
	[CrossSectionalShape=3]	0[b]	.	.	0	.
	[fy=1]	1.750	.629	7.750	1	.005
	[fy=2]	-.066	.620	.011	1	.915
	[fy=3]	1.075	.530	4.110	1	.043
	[fy=4]	1.037	.470	4.875	1	.027
	[fy=5]	.876	.477	3.373	1	.066
	[fy=6]	0[b]	.	.	0	.
	[CrackcontrolRF=0]	3.677	.958	14.727	1	.000
	[CrackcontrolRF=1]	0[b]	.	.	0	.
	[AnchorageCheck=0]	-1.619	.641	6.377	1	.012
	[AnchorageCheck=1]	0[b]	.	.	0	.
C	Intercept	-18.016	.835	465.668	1	.000
	fcMPa	.056	.010	33.419	1	.000
	ad	.782	.193	16.469	1	.000
	rhoshearlong.Steel	-.719	.240	9.000	1	.003
	[CrossSectionalShape=2]	-3.837	1.320	8.450	1	.004
	[CrossSectionalShape=3]	0[b]	.	.	0	.
	[fy=1]	-1.536	1.069	2.067	1	.151
	[fy=2]	1.430	.714	4.012	1	.045
	[fy=3]	-1.150	.840	1.873	1	.171
	[fy=4]	-.485	.691	.493	1	.483
	[fy=5]	.393	.715	.302	1	.583
	[fy=6]	0[b]	.	.	0	.
	[CrackcontrolRF=0]	-21.972	5795.959	.000	1	.997
	[CrackcontrolRF=1]	0[b]	.	.	0	.
	[AnchorageCheck=0]	13.519	.000	.	1	.
	[AnchorageCheck=1]	0[b]	.	.	0	.

a. The reference category is: B.

Classification

Observed	Predicted A	Predicted B	Predicted C	Percent Correct
A	229	49	2	81.8%
B	34	124	15	71.7%
C	5	21	45	63.4%
Overall Percentage	51.1%	37.0%	11.8%	76.0%

B.2.2 Beams with Shear Reinforcement

B.2.2.1 *Beams with Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	108	59.7%
	B	62	34.3%
	C	11	6.1%
Cross Section	1	18	9.9%
	2	16	8.8%
	3	147	81.2%
ag	2	128	70.7%
	3	53	29.3%
fy	1	4	2.2%
	2	34	18.8%
	3	65	35.9%
	4	49	27.1%
	5	10	5.5%
	6	19	10.5%
fvy	1	17	9.4%
	2	49	27.1%
	3	36	19.9%
	4	57	31.5%
	5	22	12.2%
Crack Controal Check	0	2	1.1%
	1	179	98.9%
Anchorage	0	179	98.9%
	1	2	1.1%
Valid		181	100.0%
Missing		0	
Total		181	
Subpopulation		166[a]	

Model Fitting Information

Model	Model Fitting Criteria: AIC	BIC	-2 Log Likelihood	Likelihood Ratio Tests: Chi-Square	df	Sig.
Intercept Only	300.869	307.266	296.869			
Final	305.114	445.848	217.114	79.755	42	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	699.030	288	.000
Deviance	209.745	288	1.000

Pseudo R-Square

Cox and Snell	.356
Nagelkerke	.437
McFadden	.261

Likelihood Ratio Tests

Effect	Model Fitting Criteria: AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Likelihood Ratio Tests: Chi-Square	df	Sig.
Intercept	305.114	445.848	2.171E2	.000	0	.
fcMPa	301.803	436.140	2.178E2	.689	2	.709
bw_dm	301.829	436.165	2.178E2	.715	2	.700
ds_dm	281.589	415.926	1.976E2	.	2	.
ad	310.988	445.325	2.270E2	9.875	2	.007
sd	290.458	424.795	2.065E2	.	2	.
rhoshearlong.Steel	288.169	422.506	2.042E2	.	2	.
rhostirrup	307.941	442.277	2.239E2	6.827	2	.033
CrossSection	299.595	427.534	2.196E2	2.481	4	.648
ag	296.951	431.288	2.130E2	.	2	.
fy	301.863	410.612	2.339E2	16.750	10	.080
fvy	293.732	408.878	2.217E2	4.618	8	.797
CrackControalCheck	301.173	435.510	2.172E2	.059	2	.971
Anchorage	284.485	418.822	200.485	.	2	.

Classification

Observed	Predicted: A	B	C	Percent Correct
A	90	17	1	83.3%
B	19	38	5	61.3%
C	2	4	5	45.5%
Overall Percentage	61.3%	32.6%	6.1%	73.5%

B.2.2.2 *Beams with Shear Reinforcement-SPSS output of Stepwise Logistic Regression*

Case Processing Summary

		N	Marginal Percentage
Y	A	108	59.7%
	B	62	34.3%
	C	11	6.1%
Cross Section	1	18	9.9%
	2	16	8.8%
	3	147	81.2%
ag	2	128	70.7%
	3	53	29.3%
fy	1	4	2.2%
	2	34	18.8%
	3	65	35.9%
	4	49	27.1%
	5	10	5.5%
	6	19	10.5%
fvy	1	17	9.4%
	2	49	27.1%
	3	36	19.9%
	4	57	31.5%
	5	22	12.2%
Crack Controal Check	0	2	1.1%
	1	179	98.9%
Anchorage	0	179	98.9%
	1	2	1.1%
Valid		181	100.0%
Missing		0	
Total		181	
Subpopulation		166[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	300.869	307.266	296.869			
Final	266.968	330.938	226.968	69.901	18	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	273.626	312	.943
Deviance	219.599	312	1.000

Pseudo R-Square

Cox and Snell	.320
Nagelkerke	.393
McFadden	.228

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	266.968	330.938	2.270E2	.000	0	.
sd	288.665	346.238	252.665	25.697	2	.000
ds_dm	269.045	326.617	233.045	6.077	2	.048
fy	281.107	313.092	261.107	34.139	10	.000
CrossSection	276.899	328.075	244.899	17.931	4	.001

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	90	17	1	83.3%
B	18	43	1	69.4%
C	2	9	0	.0%
Overall Percentage	60.8%	38.1%	1.1%	73.5%

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	4.432	1.194	13.783	1	.000	
	sd	-.484	.127	14.445	1	.000	.616
	ds_dm	-.197	.151	1.693	1	.193	.821
	[fy=1]	17.181	6641.052	.000	1	.998	28944509.900
	[fy=2]	-1.968	.838	5.518	1	.019	.140
	[fy=3]	-.116	.673	.029	1	.864	.891
	[fy=4]	-2.496	.720	12.003	1	.001	.082
	[fy=5]	-.459	.939	.239	1	.625	.632
	[fy=6]	0[c]	.	.	0	.	.
	[CrossSection=1]	2.322	1.133	4.204	1	.040	10.200
	[CrossSection=2]	2.307	.984	5.504	1	.019	10.049
	[CrossSection=3]	0[c]	.	.	0	.	.
C	Intercept	-4.113	1.951	4.441	1	.035	
	sd	.212	.168	1.606	1	.205	1.237
	ds_dm	.321	.210	2.337	1	.126	1.379
	[fy=1]	.517	.000	.	1	.	1.677
	[fy=2]	-.787	1.500	.275	1	.600	.455
	[fy=3]	.364	1.261	.083	1	.773	1.439
	[fy=4]	.121	1.350	.008	1	.929	1.129
	[fy=5]	-18.774	9646.803	.000	1	.998	7.022E-9
	[fy=6]	0[c]	.	.	0	.	.
	[CrossSection=1]	-16.767	6478.621	.000	1	.998	5.225E-8
	[CrossSection=2]	-15.991	8028.115	.000	1	.998	1.136E-7
	[CrossSection=3]	0[c]	.	.	0	.	.

a. The reference category is: B.

B.3 SPSS output of AS 3600

B.3.1 Beams without Shear Reinforcement

B.3.1.1 *Beams without Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	227	43.9%
	B	193	37.3%
	C	97	18.8%
Cross Sectional Shape	2	146	28.2%
	3	371	71.8%
ag	1	129	25.0%
	2	284	54.9%
	3	104	20.1%
fy	1	52	10.1%
	2	48	9.3%
	3	177	34.2%
	4	102	19.7%
	5	126	24.4%
	6	12	2.3%
Crack control R/F	0	11	2.1%
	1	506	97.9%
Anchorage	0	412	79.7%
	1	105	20.3%
Valid		517	100.0%
Missing		0	
Total		517	
Subpopulation		501[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	1.076E3	1.084E3	1.072E3			
Final	694.681	830.618	630.681	441.044	30	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	1140.309	970	.000
Deviance	623.749	970	1.000

Pseudo R-Square

Cox and Snell	.574
Nagelkerke	.655
McFadden	.409

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	694.681	830.618	6.307E2	.000	0	.
bw_dm	695.115	822.556	635.115	4.434	2	.109
ds_dm	705.675	833.117	645.675	14.994	2	.001
rhoshearlong.Steel	691.530	818.971	631.530	.849	2	.654
fcMPa	740.044	867.485	680.044	49.363	2	.000
ad	937.217	1.065E3	877.217	246.536	2	.000
CrossSectionalShape	712.061	839.503	652.061	21.381	2	.000
ag	707.374	826.320	651.374	20.694	4	.000
fy	691.780	785.237	647.780	17.100	10	.072
CrackcontrolRF	717.498	844.940	657.498	26.818	2	.000
Anchorage	696.714	824.155	636.714	6.033	2	.069

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	197	26	4	86.8%
B	44	122	27	63.2%
C	5	35	57	58.8%
Overall Percentage	47.6%	35.4%	17.0%	72.7%

Parameter Estimates

Y^a		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	6.901	1.651	17.469	1	.000	
	bw_dm	-.270	.206	1.729	1	.189	.763
	ds_dm	.040	.094	.181	1	.670	1.041
	rhoshearlong.Steel	.040	.098	.166	1	.684	1.041
	fcMPa	-.023	.010	5.021	1	.025	.978
	ad	-2.091	.242	74.733	1	.000	.124
	[CrossSectionalShape=2]	2.104	.543	15.032	1	.000	8.196
	[CrossSectionalShape=3]	0^b	.	.	0	.	.
	[ag=1]	-1.842	.532	12.008	1	.001	.158
	[ag=2]	-.049	.424	.014	1	.907	.952
	[ag=3]	0^b	.	.	0	.	.
	[fy=1]	-.576	1.233	.218	1	.641	.562
	[fy=2]	-1.068	1.259	.720	1	.396	.344
	[fy=3]	.337	1.149	.086	1	.769	1.401
	[fy=4]	-.133	1.139	.014	1	.907	.875
	[fy=5]	.833	1.095	.578	1	.447	2.300
	[fy=6]	0^b	.	.	0	.	.
	[CrackcontrolRF=0]	2.129	.919	5.368	1	.021	8.409
	[CrackcontrolRF=1]	0^b	.	.	0	.	.
	[Anchorage=0]	.433	.425	1.037	1	.309	1.541
	[Anchorage=1]	0^b	.	.	0	.	.
C	Intercept	-5.313	1.401	14.387	1	.000	
	bw_dm	.138	.108	1.640	1	.200	1.148
	ds_dm	.299	.080	13.905	1	.000	1.349
	rhoshearlong.Steel	-.098	.128	.590	1	.442	.907
	fcMPa	.046	.009	26.653	1	.000	1.047
	ad	.545	.121	20.447	1	.000	1.725
	[CrossSectionalShape=2]	-1.350	.826	2.670	1	.102	.259
	[CrossSectionalShape=3]	0^b	.	.	0	.	.
	[ag=1]	.516	.594	.756	1	.385	1.676
	[ag=2]	.752	.446	2.841	1	.092	2.121
	[ag=3]	0^b	.	.	0	.	.
	[fy=1]	.305	1.192	.066	1	.798	1.357
	[fy=2]	.060	1.212	.002	1	.961	1.062
	[fy=3]	-1.083	1.142	.899	1	.343	.339
	[fy=4]	-.714	1.115	.410	1	.522	.490
	[fy=5]	-.375	.983	.145	1	.703	.687
	[fy=6]	0^b	.	.	0	.	.
	[CrackcontrolRF=0]	-22.332	.000	.	1	.	2.001E-10
	[CrackcontrolRF=1]	0^b	.	.	0	.	.
	[Anchorage=0]	-1.103	.553	3.982	1	.046	.332
	[Anchorage=1]	0^b	.	.	0	.	.

a. The reference category is: B.

B.3.1.2 Beams without Shear Reinforcement-SPSS output for selected explanatory variables

Case Processing Summary

		N	Marginal Percentage
Y	A	227	43.9%
	B	193	37.3%
	C	97	18.8%
Cross Sectional Shape	2	146	28.2%
	3	371	71.8%
ag	1	129	25.0%
	2	284	54.9%
	3	104	20.1%
Crack control R/F	0	11	2.1%
	1	506	97.9%
Valid		517	100.0%
Missing		0	
Total		517	
Subpopulation		473[a]	

Model Fitting Information

	Model Fitting Criteria			Likelihood Ratio Tests		
Model	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	1.062E3	1.071E3	1.058E3			
Final	682.925	750.894	650.925	407.277	14	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	937.807	930	.422
Deviance	632.196	930	1.000

Pseudo R-Square

Cox and Snell	.545
Nagelkerke	.622
McFadden	.378

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	682.925	750.894	6.509E2	.000	0	.
fcMPa	731.606	791.078	703.606	52.681	2	.000
dsm	717.839	777.312	689.839	38.914	2	.000
ad	965.363	1.025E3	937.363	286.438	2	.000
CrossSectionalShape	737.573	797.045	709.573	58.648	2	.000
ag	689.404	740.380	665.404	14.479	4	.006
CrackcontrolRF	703.959	763.432	675.959	25.034	2	.000

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.
A	Intercept	7.072	.881	64.395	1	.000
	fcMPa	-.026	.010	7.174	1	.007
	dsm	-.135	.065	4.368	1	.037
	ad	-1.956	.198	97.899	1	.000
	[CrossSectionalShape=2]	2.644	.442	35.728	1	.000
	[CrossSectionalShape=3]	0[b]	.	.	0	.
	[ag=1]	-1.320	.467	7.996	1	.005
	[ag=2]	-.115	.385	.090	1	.765
	[ag=3]	0[b]	.	.	0	.
	[CrackcontrolRF=0]	2.087	.895	5.433	1	.020
	[CrackcontrolRF=1]	0[b]	.	.	0	.
C	Intercept	-5.789	.825	49.234	1	.000
	fcMPa	.041	.008	27.076	1	.000
	dsm	.301	.063	22.671	1	.000
	ad	.498	.103	23.316	1	.000
	[CrossSectionalShape=2]	-1.861	.681	7.464	1	.006
	[CrossSectionalShape=3]	0[b]	.	.	0	.
	[ag=1]	.606	.526	1.326	1	.250
	[ag=2]	.467	.398	1.380	1	.240
	[ag=3]	0[b]	.	.	0	.
	[CrackcontrolRF=0]	-21.927	.000	.	1	.
	[CrackcontrolRF=1]	0[b]	.	.	0	.

a. The reference category is: B.

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	194	29	4	85.5%
B	50	121	22	62.7%
C	9	33	55	56.7%
Overall Percentage	48.9%	35.4%	15.7%	71.6%

B.3.2 Beams with Shear Reinforcement

B.3.2.1 *Beams with Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	71	50.4%
	B	53	37.6%
	C	17	12.1%
ag	2	90	63.8%
	3	51	36.2%
fy	1	4	2.8%
	2	19	13.5%
	3	63	44.7%
	4	46	32.6%
	5	9	6.4%
Anchorage	0	125	88.7%
	1	16	11.3%
Cross Section	1	18	12.8%
	2	3	2.1%
	3	120	85.1%
fvy	1	17	12.1%
	2	47	33.3%
	3	33	23.4%
	4	22	15.6%
	5	22	15.6%
Crack Controal Check	0	2	1.4%
	1	139	98.6%
Valid		141	100.0%
Missing		50	
Total		191	
Subpopulation		127[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	264.722	270.619	260.722			
Final	232.785	356.633	148.785	111.937	40	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	199.681	212	.718
Deviance	139.631	212	1.000

Pseudo R-Square

Cox and Snell	.548
Nagelkerke	.640
McFadden	.410

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	232.785	356.633	1.488E2	.000	0	.
fcMPa	231.940	349.890	151.940	3.155	2	.206
ad	231.956	349.906	151.956	3.171	2	.205
bw_dm	231.241	349.191	151.241	2.456	2	.293
ds_dm	230.362	348.313	150.362	1.578	2	.454
sd	233.708	351.659	153.708	4.924	2	.085
rhoshearlong.Steel	228.875	346.825	148.875	.090	2	.956
rhostirrup	230.817	348.767	150.817	2.032	2	.362
ag	237.499	355.449	157.499	8.714	2	.013
fy	242.181	342.439	174.181	25.397	8	.001
Anchorage	238.335	356.286	158.335	9.551	2	.008
CrossSection	231.937	343.990	155.937	7.153	4	.128
fvy	230.444	330.702	162.444	13.660	8	.091
CrackControalCheck	229.061	347.011	149.061	.276	2	.871

Classification

Observed	Predicted A	Predicted B	Predicted C	Percent Correct
A	59	10	2	83.1%
B	9	41	3	77.4%
C	1	5	11	64.7%
Overall Percentage	48.9%	39.7%	11.3%	78.7%

Parameter Estimates

Y^a		B	Std. Error	Wald	df	Sig.	Exp(B)	95% Confidence Interval for Exp (B) Lower Bound	Upper Bound
A	Intercept	1.706	4.766	.128	1	.720			
	fcMPa	.012	.017	.465	1	.495	1.012	.978	1.046
	ad	-1.389	.947	2.152	1	.142	.249	.039	1.595
	bw_dm	.938	.707	1.758	1	.185	2.554	.639	10.213
	ds_dm	-.237	.261	.826	1	.363	.789	.474	1.315
	sd	-.531	.256	4.316	1	.038	.588	.356	.970
	rhoshearlong.Steel	-.142	.635	.050	1	.823	.868	.250	3.014
	rhostirrup	4.083	3.532	1.337	1	.248	59.342	.059	60195.601
	[ag=2]	3.684	1.593	5.346	1	.021	39.815	1.753	904.524
	[ag=3]	0^b	.	.	0	.	.	.	.
	[fy=1]	15.139	5305.589	.000	1	.998	3755683.517	.000	$.^c$
	[fy=2]	-4.900	1.767	7.688	1	.006	.007	.000	.238
	[fy=3]	-3.923	2.250	3.039	1	.081	.020	.000	1.628
	[fy=4]	-4.842	2.457	3.883	1	.049	.008	6.391E-5	.974
	[fy=5]	0^b	.	.	0	.	.	.	.
	[Anchorage=0]	3.400	2.110	2.598	1	.107	29.970	.480	1872.685
	[Anchorage=1]	0^b	.	.	0	.	.	.	.
	[CrossSection=1]	2.605	2.743	.902	1	.342	13.537	.063	2929.493
	[CrossSection=2]	21.782	6146.116	.000	1	.997	2.882E9	.000	$.^c$
	[CrossSection=3]	0^b	.	.	0	.	.	.	.
	[fvy=1]	2.615	2.051	1.626	1	.202	13.669	.246	760.914
	[fvy=2]	3.424	1.708	4.019	1	.045	30.690	1.079	872.507
	[fvy=3]	-1.122	2.260	.247	1	.620	.326	.004	27.330
	[fvy=4]	-.211	1.142	.034	1	.854	.810	.086	7.601
	[fvy=5]	0^b	.	.	0	.	.	.	.
	[CrackControalCheck=0]	-.071	1.826	.002	1	.969	.931	.026	33.372
	[CrackControalCheck=1]	0^b	.	.	0	.	.	.	.
C	Intercept	-19.818	7.388	7.195	1	.007			
	fcMPa	.038	.023	2.864	1	.091	1.039	.994	1.086
	ad	.803	1.185	.459	1	.498	2.233	.219	22.797
	bw_dm	.732	.788	.864	1	.353	2.080	.444	9.740
	ds_dm	.263	.438	.360	1	.548	1.301	.551	3.069
	sd	-.106	.302	.124	1	.725	.899	.497	1.625
	rhoshearlong.Steel	-.253	1.101	.053	1	.818	.777	.090	6.719
	rhostirrup	-3.473	6.067	.328	1	.567	.031	2.127E-7	4523.274
	[ag=2]	.147	1.836	.006	1	.936	1.158	.032	42.308
	[ag=3]	0^b	.	.	0	.	.	.	.
	[fy=1]	16.655	9877.346	.000	1	.999	17105986.836	.000	$.^c$
	[fy=2]	18.087	2.683	45.461	1	.000	71659079.534	373104.423	1.376E10
	[fy=3]	18.007	2.023	79.217	1	.000	66121007.556	1253760.120	3.487E9
	[fy=4]	16.282	.000	.	1	.	11783994.327	11783994.327	11783994.327
	[fy=5]	0^b	.	.	0	.	.	.	.
	[Anchorage=0]	-4.530	2.505	3.270	1	.071	.011	7.950E-5	1.463
	[Anchorage=1]	0^b	.	.	0	.	.	.	.
	[CrossSection=1]	1.169	3644.457	.000	1	1.000	3.219	.000	$.^c$
	[CrossSection=2]	.117	.000	.	1	.	1.124	1.124	1.124
	[CrossSection=3]	0^b	.	.	0	.	.	.	.
	[fvy=1]	-15.633	3640.765	.000	1	.997	1.625E-7	.000	$.^c$
	[fvy=2]	-2.755	3.192	.745	1	.388	.064	.000	33.184
	[fvy=3]	-2.321	2.987	.604	1	.437	.098	.000	34.198
	[fvy=4]	.788	1.616	.238	1	.626	2.200	.093	52.234
	[fvy=5]	0^b	.	.	0	.	.	.	.
	[CrackControalCheck=0]	-16.817	.000	.	1	.	4.972E-8	4.972E-8	4.972E-8
	[CrackControalCheck=1]	0^b	.	.	0	.	.	.	.

a. The reference category is: B.

B.4 SPSS output of JSCE

B.3.1 Beams without Shear Reinforcement

B.4.1.1 *Beams without Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	350	67.4%
	B	129	24.9%
	C	40	7.7%
Cross Sectional Shape	2	146	28.1%
	3	373	71.9%
ag	1	129	24.9%
	2	285	54.9%
	3	105	20.2%
fy	1	53	10.2%
	2	48	9.2%
	3	177	34.1%
	4	103	19.8%
	5	126	24.3%
	6	12	2.3%
Crack control R/F	0	11	2.1%
	1	508	97.9%
Anchorage	0	318	61.3%
	1	201	38.7%
Valid		519	100.0%
Missing		0	
Total		519	
Subpopulation		503[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	838.436	846.940	834.436			
Final	653.977	790.038	589.977	244.459	30	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	760.506	974	1.000
Deviance	585.479	974	1.000

Pseudo R-Square

Cox and Snell	.376
Nagelkerke	.468
McFadden	.291

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	653.977	790.038	5.900E2	.000	0	.
fcMPa	681.873	809.430	621.873	31.896	2	.000
bw_dm	652.661	780.218	592.661	2.684	2	.261
ds_dm	650.203	777.760	590.203	.226	2	.893
rhoshearlong.Steel	748.555	876.113	688.555	98.578	2	.000
ad	698.247	825.804	638.247	48.269	2	.000
CrossSectionalShape	662.269	789.826	602.269	12.292	2	.002
ag	659.121	778.174	603.121	13.143	4	.011
fy	657.454	750.996	613.454	23.477	10	.009
CrackcontrolRF	679.700	807.257	619.700	29.723	2	.000
Anchorage	654.310	781.867	594.310	4.332	2	.115

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	313	35	2	89.4%
B	73	48	8	37.2%
C	13	20	7	17.5%
Overall Percentage	76.9%	19.8%	3.3%	70.9%

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	-.736	1.232	.357	1	.550	
	fcMPa	-.034	.008	18.472	1	.000	.967
	bw_dm	.125	.105	1.438	1	.231	1.134
	ds_dm	-.031	.066	.220	1	.639	.969
	rhoshearlong.Steel	1.139	.159	51.549	1	.000	3.123
	ad	-.532	.099	29.166	1	.000	.587
	[CrossSectionalShape=2]	1.147	.444	6.674	1	.010	3.150
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	.723	.440	2.704	1	.100	2.061
	[ag=2]	-.433	.387	1.255	1	.263	.648
	[ag=3]	0[b]	.	.	0	.	.
	[fy=1]	2.169	1.175	3.405	1	.065	8.748
	[fy=2]	1.797	1.218	2.176	1	.140	6.033
	[fy=3]	1.995	1.118	3.186	1	.074	7.353
	[fy=4]	.901	1.077	.700	1	.403	2.463
	[fy=5]	.458	1.064	.186	1	.667	1.581
	[fy=6]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	21.007	5360.166	.000	1	.997	1.327E9
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
	[Anchorage=0]	.653	.355	3.380	1	.066	1.921
	[Anchorage=1]	0[b]	.	.	0	.	.
C	Intercept	-20.134	1.260	255.198	1	.000	
	fcMPa	.014	.009	2.793	1	.095	1.014
	bw_dm	-.076	.148	.266	1	.606	.927
	ds_dm	-.006	.083	.006	1	.938	.994
	rhoshearlong.Steel	-.074	.267	.076	1	.783	.929
	ad	.359	.173	4.316	1	.038	1.432
	[CrossSectionalShape=2]	-1.448	.987	2.150	1	.143	.235
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	.050	.863	.003	1	.954	1.051
	[ag=2]	.677	.547	1.529	1	.216	1.967
	[ag=3]	0[b]	.	.	0	.	.
	[fy=1]	18.100	1.050	297.427	1	.000	72582613.483
	[fy=2]	18.228	1.061	295.208	1	.000	82498260.700
	[fy=3]	16.825	.918	335.618	1	.000	20270398.448
	[fy=4]	16.939	.887	364.736	1	.000	22733527.166
	[fy=5]	16.207	.000	.	1	.	10932837.471
	[fy=6]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	-.619	9469.621	.000	1	1.000	.538
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
	[Anchorage=0]	-.217	.533	.166	1	.684	.805
	[Anchorage=1]	0[b]	.	.	0	.	.

a. The reference category is: B.

B.4.1.2 Beams without Shear Reinforcement-SPSS output for selected explanatory variables

Case Processing Summary

		N	Marginal Percentage
Y	A	350	67.4%
	B	129	24.9%
	C	40	7.7%
Cross Sectional Shape	2	146	28.1%
	3	373	71.9%
ag	1	129	24.9%
	2	285	54.9%
	3	105	20.2%
fy	1	53	10.2%
	2	48	9.2%
	3	177	34.1%
	4	103	19.8%
	5	126	24.3%
	6	12	2.3%
Crack control R/F	0	11	2.1%
	1	508	97.9%
Valid		519	100.0%
Missing		0	
Total		519	
Subpopulation		503[a]	

Model Fitting Information

Model	Model Fitting Criteria: AIC	Model Fitting Criteria: BIC	Model Fitting Criteria: -2 Log Likelihood	Likelihood Ratio Tests: Chi-Square	Likelihood Ratio Tests: df	Likelihood Ratio Tests: Sig.
Intercept Only	838.436	846.940	834.436			
Final	649.156	759.706	597.156	237.280	24	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	789.901	980	1.000
Deviance	592.657	980	1.000

Pseudo R-Square

Cox and Snell	.367
Nagelkerke	.458
McFadden	.282

Likelihood Ratio Tests

Effect	Model Fitting Criteria: AIC of Reduced Model	Model Fitting Criteria: BIC of Reduced Model	Model Fitting Criteria: -2 Log Likelihood of Reduced Model	Likelihood Ratio Tests: Chi-Square	Likelihood Ratio Tests: df	Likelihood Ratio Tests: Sig.
Intercept	649.156	759.706	5.972E2	.000	0	.
ad	690.616	792.662	642.616	45.460	2	.000
fcMPa	676.386	778.432	628.386	31.230	2	.000
CrackcontrolRF	671.804	773.850	623.804	26.648	2	.000
ag	651.757	745.299	607.757	10.601	4	.031
CrossSectionalShape	659.526	761.572	611.526	14.370	2	.001
fy	654.217	722.248	622.217	25.061	10	.005
rhoshearlong.Steel	743.912	845.958	695.912	98.756	2	.000

Classification

Observed	Predicted: A	Predicted: B	Predicted: C	Predicted: Percent Correct
A	310	38	2	88.6%
B	76	45	8	34.9%
C	14	20	6	15.0%
Overall Percentage	77.1%	19.8%	3.1%	69.6%

Parameter Estimates

Y^a		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	-.551	1.176	.220	1	.639	
	ad	-.509	.096	27.898	1	.000	.601
	fcMPa	-.032	.008	17.462	1	.000	.969
	[CrackcontrolRF=0]	20.624	5387.605	.000	1	.997	905855355.630
	[CrackcontrolRF=1]	0^c	.	.	0	.	.
	[ag=1]	.866	.429	4.073	1	.044	2.377
	[ag=2]	-.100	.345	.084	1	.772	.905
	[ag=3]	0^c	.	.	0	.	.
	[CrossSectionalShape=2]	1.258	.430	8.548	1	.003	3.517
	[CrossSectionalShape=3]	0^c	.	.	0	.	.
	[fy=1]	2.371	1.159	4.184	1	.041	10.712
	[fy=2]	2.167	1.180	3.376	1	.066	8.736
	[fy=3]	1.895	1.107	2.929	1	.087	6.652
	[fy=4]	1.005	1.071	.881	1	.348	2.732
	[fy=5]	.366	1.057	.120	1	.729	1.442
	[fy=6]	0^c	.	.	0	.	.
	rhoshearlong.Steel	1.121	.155	52.146	1	.000	3.068
C	Intercept	-20.161	1.171	296.643	1	.000	
	ad	.329	.168	3.827	1	.050	1.389
	fcMPa	.014	.008	2.804	1	.094	1.014
	[CrackcontrolRF=0]	-.538	9250.617	.000	1	1.000	.584
	[CrackcontrolRF=1]	0^c	.	.	0	.	.
	[ag=1]	-.310	.811	.146	1	.702	.733
	[ag=2]	.486	.459	1.121	1	.290	1.625
	[ag=3]	0^c	.	.	0	.	.
	[CrossSectionalShape=2]	-1.350	.974	1.922	1	.166	.259
	[CrossSectionalShape=3]	0^c	.	.	0	.	.
	[fy=1]	17.912	.996	323.351	1	.000	60144671.498
	[fy=2]	18.017	.994	328.538	1	.000	66760558.845
	[fy=3]	16.846	.900	350.305	1	.000	20698226.659
	[fy=4]	16.855	.854	389.475	1	.000	20885420.124
	[fy=5]	16.246	.000	.	1	.	11364089.854
	[fy=6]	0^c	.	.	0	.	.
	rhoshearlong.Steel	-.060	.264	.051	1	.821	.942

a. The reference category is: B.

B.4.2 Beams with Shear Reinforcement

B.4.2.1 *Beams with Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	121	67.6%
	B	48	26.8%
	C	10	5.6%
Crack Controal Check	0	2	1.1%
	1	177	98.9%
Anchorage	0	133	74.3%
	1	46	25.7%
fvy	1	17	9.5%
	2	47	26.3%
	3	36	20.1%
	4	57	31.8%
	5	22	12.3%
fy	0	1	.6%
	1	4	2.2%
	2	33	18.4%
	3	63	35.2%
	4	49	27.4%
	5	10	5.6%
	6	19	10.6%
ag	2	128	71.5%
	3	51	28.5%
Cross Section	1	6	3.4%
	3	173	96.6%
Valid		179	100.0%
Missing		8	
Total		187	
Subpopulation		167[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	278.657	285.032	274.657			
Final	230.699	370.944	142.699	131.958	42	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	302.599	290	.293
Deviance	138.540	290	1.000

Pseudo R-Square

Cox and Snell	.522
Nagelkerke	.661
McFadden	.473

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	230.699	370.944	1.427E2	.000	0	.
rhostirrup	262.210	396.080	178.210	35.511	2	.000
rhoshearlong.Steel	254.231	388.101	170.231	27.532	2	.000
ad	239.225	373.095	155.225	12.526	2	.002
sd	243.707	377.577	159.707	17.008	2	.000
ds_dm	227.853	361.723	143.853	1.154	2	.562
bw_dm	228.969	362.839	144.969	2.270	2	.321
fcMPa	234.391	368.261	150.391	7.691	2	.021
CrackControalCheck	228.224	362.094	144.224	1.524	2	.467
Anchorage	234.949	368.819	150.949	8.250	2	.016
fvy	241.078	355.824	169.078	26.379	8	.001
fy	229.672	331.668	165.672	22.972	12	.028
ag	228.967	362.837	144.967	2.267	2	.322
CrossSection	233.958	367.828	149.958	7.259	2	.027

Classification

Observed	Predicted A	Predicted B	Predicted C	Percent Correct
A	113	8	0	93.4%
B	15	30	3	62.5%
C	1	2	7	70.0%
Overall Percentage	72.1%	22.3%	5.6%	83.8%

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)	95% Confidence Interval for Exp (B) Lower Bound	Upper Bound
A	Intercept	7.608	3.155	5.816	1	.016			
	rhostirrup	-8.910	2.181	16.693	1	.000	.000	1.881E-6	.010
	rhoshearlong.Steel	2.208	.585	14.243	1	.000	9.099	2.890	28.643
	ad	-2.198	.698	9.924	1	.002	.111	.028	.436
	sd	-.823	.237	12.050	1	.001	.439	.276	.699
	ds_dm	.237	.235	1.020	1	.312	1.268	.800	2.010
	bw_dm	-.230	.418	.304	1	.582	.794	.350	1.801
	fcMPa	-.014	.015	.837	1	.360	.986	.958	1.016
	[CrackControalCheck=0]	17.007	4309.385	.000	1	.997	24335355.162	.000	.[b]
	[CrackControalCheck=1]	0[c]	.	.	0	.	.	.	.
	[Anchorage=0]	.481	.759	.401	1	.527	1.617	.365	7.160
	[Anchorage=1]	0[c]	.	.	0	.	.	.	.
	[fvy=1]	-4.633	1.913	5.865	1	.015	.010	.000	.413
	[fvy=2]	.331	1.259	.069	1	.792	1.393	.118	16.413
	[fvy=3]	-2.655	1.639	2.626	1	.105	.070	.003	1.744
	[fvy=4]	.125	1.104	.013	1	.910	1.133	.130	9.862
	[fvy=5]	0[c]	.	.	0	.	.	.	.
	[fy=0]	19.059	5898.757	.000	1	.997	1.893E8	.000	.[b]
	[fy=1]	22.032	2030.892	.000	1	.991	3.700E9	.000	.[b]
	[fy=2]	-.642	1.508	.181	1	.671	.526	.027	10.121
	[fy=3]	.999	1.369	.532	1	.466	2.714	.185	39.743
	[fy=4]	-1.894	1.568	1.458	1	.227	.150	.007	3.254
	[fy=5]	.791	1.670	.224	1	.636	2.206	.084	58.261
	[fy=6]	0[c]	.	.	0	.	.	.	.
	[ag=2]	1.145	.858	1.779	1	.182	3.141	.584	16.887
	[ag=3]	0[c]	.	.	0	.	.	.	.
	[CrossSection=1]	-6.738	3.886	3.007	1	.083	.001	5.837E-7	2.406
	[CrossSection=3]	0[c]	.	.	0	.	.	.	.
C	Intercept	-16.653	1728.748	.000	1	.992			
	rhostirrup	4.287	2.975	2.076	1	.150	72.716	.214	24764.570
	rhoshearlong.Steel	-2.443	1.819	1.805	1	.179	.087	.002	3.069
	ad	.483	1.830	.070	1	.792	1.621	.045	58.598
	sd	-.316	.444	.506	1	.477	.729	.305	1.741
	ds_dm	-.168	.600	.079	1	.779	.845	.261	2.737
	bw_dm	2.079	1.688	1.517	1	.218	7.996	.293	218.508
	fcMPa	.074	.039	3.610	1	.057	1.077	.998	1.162
	[CrackControalCheck=0]	7.312	8092.570	.000	1	.999	1497.501	.000	.[b]
	[CrackControalCheck=1]	0[c]	.	.	0	.	.	.	.
	[Anchorage=0]	-4.558	2.396	3.619	1	.057	.010	9.581E-5	1.148
	[Anchorage=1]	0[c]	.	.	0	.	.	.	.
	[fvy=1]	3.610	2410.746	.000	1	.999	36.949	.000	.[b]
	[fvy=2]	14.738	1728.730	.000	1	.993	2515595.601	.000	.[b]
	[fvy=3]	19.985	1728.732	.000	1	.991	4.779E8	.000	.[b]
	[fvy=4]	20.546	1728.732	.000	1	.991	8.375E8	.000	.[b]
	[fvy=5]	0[c]	.	.	0	.	.	.	.
	[fy=0]	-1.103	.000	.	1	.	.332	.332	.332
	[fy=1]	3.424	5299.096	.000	1	.999	30.684	.000	.[b]
	[fy=2]	-3.902	3.269	1.425	1	.233	.020	3.335E-5	12.248
	[fy=3]	-8.390	5.227	2.576	1	.108	.000	8.071E-9	6.391
	[fy=4]	-3.220	2.519	1.634	1	.201	.040	.000	5.566
	[fy=5]	-22.545	3161.814	.000	1	.994	1.618E-10	.000	.[b]
	[fy=6]	0[c]	.	.	0	.	.	.	.
	[ag=2]	-1.578	3.202	.243	1	.622	.206	.000	109.836
	[ag=3]	0[c]	.	.	0	.	.	.	.
	[CrossSection=1]	15.465	10.379	2.220	1	.136	5205034.382	.008	3.560E15
	[CrossSection=3]	0[c]	.	.	0	.	.	.	.

a. The reference category is: B.

B.5 SPSS output of CSA A23.3

B.5.1 CSA A23.3 Simplified Method

B.5.1.1 Beams without Shear Reinforcement

B.5.1.1.1 *Beams without Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	264	69.3%
	B	66	17.3%
	C	51	13.4%
Cross Sectional Shape	2	140	36.7%
	3	241	63.3%
ag	1	49	12.9%
	2	246	64.6%
	3	86	22.6%
fy	1	42	11.0%
	2	33	8.7%
	3	86	22.6%
	4	84	22.0%
	5	124	32.5%
	6	12	3.1%
Crack control R/F	0	11	2.9%
	1	370	97.1%
Anchorage	0	274	71.9%
	1	107	28.1%
Valid		381	100.0%
Missing		1	
Total		382	
Subpopulation		366[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	630.072	637.958	626.072			
Final	299.545	425.715	235.545	390.527	30	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	548.336	700	1.000
Deviance	231.386	700	1.000

Pseudo R-Square

Cox and Snell	.641
Nagelkerke	.793
McFadden	.620

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	299.545	425.715	2.355E2	.000	0	.
fcMPa	359.256	477.540	299.256	63.711	2	.000
bwm	298.993	417.277	238.993	3.448	2	.178
dsm	299.120	417.404	239.120	3.575	2	.167
ad	308.506	426.790	248.506	12.961	2	.002
rhoshearlong.Steel	386.403	504.687	326.403	90.858	2	.000
CrossSectionalShape	306.846	425.130	246.846	11.301	2	.004
ag	303.082	413.480	247.082	11.537	4	.021
fy	289.316	376.058	245.316	9.771	10	.461
CrackcontrolRF	311.483	429.767	251.483	15.938	2	.000
Anchorage	297.798	416.082	237.798	2.253	2	.324

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	255	7	2	96.6%
B	16	38	12	57.6%
C	2	7	42	82.4%
Overall Percentage	71.7%	13.6%	14.7%	87.9%

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	17.762	1.657	114.860	1	.000	
	fcMPa	-.075	.016	21.728	1	.000	.928
	bwm	-.429	.268	2.567	1	.109	.651
	dsm	-.206	.112	3.378	1	.066	.814
	ad	-.854	.267	10.247	1	.001	.426
	rhoshearlong.Steel	2.560	.477	28.843	1	.000	12.942
	[CrossSectionalShape=2]	3.592	1.358	7.000	1	.008	36.316
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	-.414	1.054	.154	1	.694	.661
	[ag=2]	.668	.557	1.441	1	.230	1.951
	[ag=3]	0[b]	.	.	0	.	.
	[fy=1]	-13.747	1.131	147.830	1	.000	1.071E-6
	[fy=2]	-14.285	1.450	97.111	1	.000	6.252E-7
	[fy=3]	-13.771	1.051	171.612	1	.000	1.045E-6
	[fy=4]	-14.957	1.048	203.863	1	.000	3.193E-7
	[fy=5]	-13.534	.000	.	1	.	1.325E-6
	[fy=6]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	3.384	1.339	6.385	1	.012	29.478
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
	[Anchorage=0]	-.703	.549	1.637	1	.201	.495
	[Anchorage=1]	0[b]	.	.	0	.	.
C	Intercept	19.371	11964.982	.000	1	.999	
	fcMPa	.065	.017	13.947	1	.000	1.068
	bwm	-.085	.144	.354	1	.552	.918
	dsm	.008	.104	.006	1	.938	1.008
	ad	.228	.408	.311	1	.577	1.256
	rhoshearlong.Steel	-3.219	.944	11.639	1	.001	.040
	[CrossSectionalShape=2]	-11.314	3109.698	.000	1	.997	1.220E-5
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	1.529	1.069	2.044	1	.153	4.614
	[ag=2]	-1.237	.874	2.002	1	.157	.290
	[ag=3]	0[b]	.	.	0	.	.
	[fy=1]	-18.349	11964.982	.000	1	.999	1.074E-8
	[fy=2]	-19.219	11964.982	.000	1	.999	4.503E-9
	[fy=3]	-18.844	11964.982	.000	1	.999	6.552E-9
	[fy=4]	-19.758	11964.982	.000	1	.999	2.627E-9
	[fy=5]	-35.550	12443.456	.000	1	.998	3.639E-16
	[fy=6]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	-2.083	1.165	3.197	1	.074	.125
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
	[Anchorage=0]	-.690	.761	.821	1	.365	.502
	[Anchorage=1]	0[b]	.	.	0	.	.

a. The reference category is: B.

B.5.1.1.2 Beams without Shear Reinforcement-SPSS output for selected explanatory variables

Case Processing Summary

		N	Marginal Percentage
Y	A	264	69.3%
	B	66	17.3%
	C	51	13.4%
Cross Sectional Shape	2	140	36.7%
	3	241	63.3%
ag	1	49	12.9%
	2	246	64.6%
	3	86	22.6%
Crack control R/F	0	11	2.9%
	1	370	97.1%
Valid		381	100.0%
Missing		1	
Total		382	
Subpopulation		365[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	627.875	635.761	623.875			
Final	296.870	359.955	264.870	359.005	14	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	743.457	714	.216
Deviance	259.089	714	1.000

Pseudo R-Square

Cox and Snell	.610
Nagelkerke	.755
McFadden	.570

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	296.870	359.955	2.649E2	.000	0	.
fcMPa	385.220	440.419	357.220	92.350	2	.000
ad	304.522	359.721	276.522	11.652	2	.003
rhoshearlong.Steel	473.168	528.367	445.168	180.298	2	.000
CrossSectionalShape	304.211	359.411	276.211	11.341	2	.003
ag	299.639	346.953	275.639	10.769	4	.029
CrackcontrolRF	310.158	365.358	282.158	17.288	2	.000

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	1.254	.961	1.703	1	.192	
	fcMPa	-.077	.015	27.389	1	.000	.926
	ad	-.820	.269	9.308	1	.002	.440
	rhoshearlong.Steel	2.754	.437	39.720	1	.000	15.707
	[CrossSectionalShape=2]	3.330	1.295	6.607	1	.010	27.940
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	.210	.598	.123	1	.725	1.234
	[ag=2]	.246	.457	.290	1	.590	1.279
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	2.626	.982	7.152	1	.007	13.819
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
C	Intercept	.533	1.319	.163	1	.686	
	fcMPa	.066	.015	19.106	1	.000	1.068
	ad	.198	.381	.270	1	.604	1.219
	rhoshearlong.Steel	-3.559	.770	21.345	1	.000	.028
	[CrossSectionalShape=2]	-14.682	.000	.	1	.	4.204E-7
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	.682	.741	.847	1	.357	1.978
	[ag=2]	-1.657	.676	6.009	1	.014	.191
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	-2.501	1.186	4.445	1	.035	.082
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.

a. The reference category is: B.

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	255	8	1	96.6%
B	24	28	14	42.4%
C	3	6	42	82.4%
Overall Percentage	74.0%	11.0%	15.0%	85.3%

B.5.1.2Beams with Shear Reinforcement

B.5.1.2.1 *Beams with Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	97	77.0%
	B	21	16.7%
	C	8	6.3%
Cross Section	1	10	7.9%
	3	116	92.1%
ag	2	79	62.7%
	3	47	37.3%
fy	1	4	3.2%
	2	16	12.7%
	3	52	41.3%
	4	45	35.7%
	5	9	7.1%
fvy	1	10	7.9%
	2	46	36.5%
	3	29	23.0%
	4	19	15.1%
	5	22	17.5%
Crack Controal Check	0	2	1.6%
	1	124	98.4%
Anchorage	0	111	88.1%
	1	15	11.9%
Valid		126	100.0%
Missing		0	
Total		126	
Subpopulation		113[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	174.108	179.781	170.108			
Final	98.829	206.608	22.829	147.279	36	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	18.969	188	1.000
Deviance	22.829	188	1.000

Pseudo R-Square

Cox and Snell	.689
Nagelkerke	.930
McFadden	.866

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	98.829	206.608	22.829[a]	.000	0	.
fcMPa	96.161	198.267	24.161[b]	1.332	2	.514
bw_dm	106.569	208.675	34.569[b]	11.740	2	.003
ds_dm	115.107	217.214	43.107[b]	20.278	2	.000
ad	94.029	196.135	22.029[b]	.	2	.
sd	110.099	212.205	38.099[b]	15.270	2	.000
rhoshearlong.Steel	98.745	200.851	26.745[b]	3.915	2	.141
rhostirrup	122.737	224.843	50.737[b]	27.908	2	.000
CrossSection	98.829	206.608	22.829[a]	.000	0	.
ag	124.928	227.035	52.928[b]	30.099	2	.000
fy	112.763	197.852	52.763[b]	29.934	8	.000
fvy	118.149	208.910	54.149[b]	31.320	6	.000
CrackControalCheck	122.432	224.538	50.432	27.603	2	.000
Anchorage	120.136	222.242	48.136[b]	25.306	2	.000

B.5.1.2.1 *Beams with Shear Reinforcement-SPSS output for Stepwise Logistic Regression*

Case Processing Summary

		N	Marginal Percentage
Y	A	97	77.0%
	B	21	16.7%
	C	8	6.3%
Cross Section	1	10	7.9%
	3	116	92.1%
ag	2	79	62.7%
	3	47	37.3%
fy	1	4	3.2%
	2	16	12.7%
	3	52	41.3%
	4	45	35.7%
	5	9	7.1%
fvy	1	10	7.9%
	2	46	36.5%
	3	29	23.0%
	4	19	15.1%
	5	22	17.5%
Crack Controal Check	0	2	1.6%
	1	124	98.4%
Anchorage	0	111	88.1%
	1	15	11.9%
Valid		126	100.0%
Missing		0	
Total		126	
Subpopulation		113[a]	

Classification

Observed	Predicted A	Predicted B	Predicted C	Predicted Percent Correct
A	93	4	0	95.9%
B	14	5	2	23.8%
C	0	2	6	75.0%
Overall Percentage	84.9%	8.7%	6.3%	82.5%

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	174.108	179.781	170.108			
Final	106.725	135.088	86.725	83.383	8	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	92.284	216	1.000
Deviance	86.725	216	1.000

Pseudo R-Square

Cox and Snell	.484
Nagelkerke	.653
McFadden	.490

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	106.725	135.088	86.725[a]	.000	0	.
fcMPa	118.512	141.202	102.512	15.787	2	.000
CrackControalCheck	109.662	132.352	93.662	6.937	2	.031
ad	114.600	137.291	98.600	11.876	2	.003
rhoshearlong.Steel	166.433	189.123	150.433	63.708	2	.000

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	2.879	1.878	2.349	1	.125	
	fcMPa	-.035	.015	5.314	1	.021	.965
	[CrackControalCheck=0]	-15.230	4438.502	.000	1	.997	2.431E-7
	[CrackControalCheck=1]	0[c]	.	.	0	.	.
	ad	-2.239	.695	10.363	1	.001	.107
	rhoshearlong.Steel	2.990	.712	17.633	1	.000	19.879
C	Intercept	13.359	14.286	.875	1	.350	
	fcMPa	.057	.027	4.513	1	.034	1.059
	[CrackControalCheck=0]	-23.411	.000	.	1	.	6.804E-11
	[CrackControalCheck=1]	0[c]	.	.	0	.	.
	ad	-4.418	4.911	.809	1	.368	.012
	rhoshearlong.Steel	-3.053	1.337	5.217	1	.022	.047

a. The reference category is: B.

B.5.2 CSA A23.3 General Method

B.5.2.1 Beams without Shear Reinforcement

B.5.2.1.1 *Beams without Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	176	46.2%
	B	137	36.0%
	C	68	17.8%
Cross Sectional Shape	2	140	36.7%
	3	241	63.3%
ag	1	49	12.9%
	2	246	64.6%
	3	86	22.6%
fy	1	42	11.0%
	2	33	8.7%
	3	86	22.6%
	4	84	22.0%
	5	124	32.5%
	6	12	3.1%
Crack control R/F	0	11	2.9%
	1	370	97.1%
Anchorage	0	274	71.9%
	1	107	28.1%
Valid		381	100.0%
Missing		0	
Total		381	
Subpopulation		366[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	784.930	792.815	780.930			
Final	493.386	619.556	429.386	351.543	30	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	4565.054	700	.000
Deviance	423.841	700	1.000

Pseudo R-Square

Cox and Snell	.603
Nagelkerke	.690
McFadden	.447

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	493.386	619.556	4.294E2	.000	0	.
fcMPa	567.800	686.084	507.800	78.413	2	.000
bw_dm	489.801	608.085	429.801	.415	2	.813
ds_dm	500.351	618.634	440.351	10.964	2	.004
ad	554.330	672.614	494.330	64.944	2	.000
rhoshearlong.Steel	542.451	660.735	482.451	53.065	2	.000
CrossSectionalShape	518.099	636.383	458.099	28.713	2	.000
ag	495.151	605.549	439.151	9.765	4	.045
fy	487.661	574.402	443.661	14.274	10	.161
CrackcontrolRF	494.993	613.277	434.993	5.607	2	.061
Anchorage	489.805	608.089	429.805	.418	2	.811

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	144	31	1	81.8%
B	24	99	14	72.3%
C	0	20	48	70.6%
Overall Percentage	44.1%	39.4%	16.5%	76.4%

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	1.355	1.404	.930	1	.335	
	fcMPa	-.037	.012	10.334	1	.001	.963
	bw_dm	.137	.207	.441	1	.507	1.147
	ds_dm	-.281	.136	4.272	1	.039	.755
	ad	-.759	.132	32.955	1	.000	.468
	rhoshearlong.Steel	.875	.181	23.403	1	.000	2.400
	[CrossSectionalShape=2]	2.704	.578	21.856	1	.000	14.935
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	-.534	.779	.471	1	.493	.586
	[ag=2]	-.648	.493	1.724	1	.189	.523
	[ag=3]	0[b]	.	.	0	.	.
	[fy=1]	-.769	1.503	.262	1	.609	.463
	[fy=2]	.507	1.507	.113	1	.737	1.660
	[fy=3]	1.712	1.129	2.297	1	.130	5.537
	[fy=4]	1.032	1.033	.998	1	.318	2.807
	[fy=5]	1.080	.970	1.239	1	.266	2.945
	[fy=6]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	1.269	1.552	.669	1	.414	3.559
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
	[Anchorage=0]	.042	.480	.008	1	.931	1.043
	[Anchorage=1]	0[b]	.	.	0	.	.
C	Intercept	-20.542	1.431	206.120	1	.000	
	fcMPa	.076	.014	31.499	1	.000	1.079
	bw_dm	.016	.119	.018	1	.895	1.016
	ds_dm	.180	.087	4.301	1	.038	1.198
	ad	.715	.236	9.133	1	.003	2.044
	rhoshearlong.Steel	-1.123	.349	10.332	1	.001	.325
	[CrossSectionalShape=2]	-1.528	1.460	1.096	1	.295	.217
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	1.763	.755	5.459	1	.019	5.832
	[ag=2]	.070	.629	.012	1	.911	1.072
	[ag=3]	0[b]	.	.	0	.	.
	[fy=1]	15.091	.921	268.435	1	.000	3581823.149
	[fy=2]	14.436	1.105	170.828	1	.000	1859911.759
	[fy=3]	13.733	.926	219.828	1	.000	920525.457
	[fy=4]	15.098	.884	291.712	1	.000	3604753.204
	[fy=5]	14.291	.000	.	1	.	1608603.654
	[fy=6]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	-2.135	1.058	4.073	1	.044	.118
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
	[Anchorage=0]	-.342	.548	.388	1	.533	.711
	[Anchorage=1]	0[b]	.	.	0	.	.

a. The reference category is: B.

B.5.2.1.2 Beams without Shear Reinforcement-SPSS output for selected explanatory variables

Case Processing Summary

		N	Marginal Percentage
Y	A	176	46.2%
	B	137	36.0%
	C	68	17.8%
Cross Sectional Shape	2	140	36.7%
	3	241	63.3%
ag	1	49	12.9%
	2	246	64.6%
	3	86	22.6%
Valid		381	100.0%
Missing		0	
Total		381	
Subpopulation		358[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	778.913	786.799	774.913			
Final	476.814	539.899	444.814	330.099	14	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	2916.337	700	.000
Deviance	436.600	700	1.000

Pseudo R-Square

Cox and Snell	.580
Nagelkerke	.664
McFadden	.420

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	139	36	1	79.0%
B	23	104	10	75.9%
C	0	26	42	61.8%
Overall Percentage	42.5%	43.6%	13.9%	74.8%

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	476.814	539.899	4.448E2	.000	0	.
fcMPa	550.337	605.536	522.337	77.523	2	.000
ds_dm	497.723	552.923	469.723	24.909	2	.000
ad	540.582	595.781	512.582	67.768	2	.000
rhoshearlong.Steel	544.924	600.124	516.924	72.110	2	.000
CrossSectionalShape	505.251	560.450	477.251	32.437	2	.000
ag	487.808	535.122	463.808	18.994	4	.001

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	3.286	.757	18.853	1	.000	
	fcMPa	-.033	.011	9.601	1	.002	.967
	ds_dm	-.348	.112	9.601	1	.002	.706
	ad	-.776	.130	35.453	1	.000	.460
	rhoshearlong.Steel	.881	.168	27.521	1	.000	2.412
	[CrossSectionalShape=2]	2.496	.510	23.993	1	.000	12.132
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	-1.087	.533	4.167	1	.041	.337
	[ag=2]	-.953	.419	5.185	1	.023	.386
	[ag=3]	0[b]	.	.	0	.	.
C	Intercept	-5.527	1.142	23.429	1	.000	
	fcMPa	.063	.011	35.599	1	.000	1.065
	ds_dm	.205	.077	6.979	1	.008	1.227
	ad	.638	.216	8.704	1	.003	1.894
	rhoshearlong.Steel	-1.088	.277	15.373	1	.000	.337
	[CrossSectionalShape=2]	-1.421	1.360	1.092	1	.296	.241
	[CrossSectionalShape=3]	0[b]	.	.	0	.	.
	[ag=1]	1.788	.600	8.885	1	.003	5.979
	[ag=2]	.170	.501	.115	1	.735	1.185
	[ag=3]	0[b]	.	.	0	.	.

a. The reference category is: B.

B.5.2.2Beams with Shear Reinforcement

B.5.2.2.1 *Beams with Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	54	42.9%
	B	55	43.7%
	C	17	13.5%
Cross Section	1	10	7.9%
	3	116	92.1%
ag	2	79	62.7%
	3	47	37.3%
fy	1	4	3.2%
	2	16	12.7%
	3	52	41.3%
	4	45	35.7%
	5	9	7.1%
fvy	1	10	7.9%
	2	46	36.5%
	3	29	23.0%
	4	19	15.1%
	5	22	17.5%
Crack Controal Check	0	2	1.6%
	1	124	98.4%
Anchorage	0	111	88.1%
	1	15	11.9%
Valid		126	100.0%
Missing		0	
Total		126	
Subpopulation		113[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	250.638	256.311	246.638			
Final	191.315	299.094	115.315	131.323	36	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	228.069	188	.024
Deviance	111.156	188	1.000

Pseudo R-Square

Cox and Snell	.647
Nagelkerke	.750
McFadden	.524

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	191.315	299.094	1.153E2	.000	0	.
fcMPa	191.122	293.228	119.122	3.807	2	.149
bw_dm	187.496	289.602	115.496	.181	2	.913
ds_dm	204.913	307.019	132.913	17.598	2	.000
ad	191.367	293.474	119.367	4.052	2	.132
sd	196.333	298.439	124.333	9.018	2	.011
rhoshearlong.Steel	187.749	289.855	115.749	.434	2	.805
rhostirrup	196.747	298.853	124.747	9.432	2	.009
CrossSection	191.315	299.094	1.153E2	.000	0	.
ag	188.872	290.978	116.872	1.557	2	.459
fy	209.533	294.621	149.533	34.218	8	.000
fvy	188.761	279.522	124.761	9.446	6	.150
CrackControalCheck	187.613	289.719	115.613	.298	2	.861
Anchorage	188.507	290.614	116.507	1.192	2	.551

Classification

Observed	Predicted A	Predicted B	Predicted C	Predicted Percent Correct
A	43	11	0	79.6%
B	8	44	3	80.0%
C	0	7	10	58.8%
Overall Percentage	40.5%	49.2%	10.3%	77.0%

B.5.2.2.2 *Beams with Shear Reinforcement-SPSS output of Stepwise Logistic Regression*

Case Processing Summary

		N	Marginal Percentage
Y	A	54	42.9%
	B	55	43.7%
	C	17	13.5%
Cross Section	1	10	7.9%
	3	116	92.1%
ag	2	79	62.7%
	3	47	37.3%
fy	1	4	3.2%
	2	16	12.7%
	3	52	41.3%
	4	45	35.7%
	5	9	7.1%
fvy	1	10	7.9%
	2	46	36.5%
	3	29	23.0%
	4	19	15.1%
	5	22	17.5%
Crack Controal Check	0	2	1.6%
	1	124	98.4%
Anchorage	0	111	88.1%
	1	15	11.9%
Valid		126	100.0%
Missing		0	
Total		126	
Subpopulation		113[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	250.638	256.311	246.638			
Final	174.655	248.398	122.655	123.983	24	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	162.777	200	.975
Deviance	118.496	200	1.000

Pseudo R-Square

Cox and Snell	.626
Nagelkerke	.725
McFadden	.494

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	174.655	248.398	1.227E2	.000	0	.
rhostirrup	182.396	250.467	134.396	11.741	2	.003
sd	183.141	251.212	135.141	12.486	2	.002
ds_dm	209.815	277.885	161.815	39.160	2	.000
fy	193.022	244.075	157.022	34.367	8	.000
fvy	181.000	232.053	145.000	22.345	8	.004
ad	178.917	246.988	130.917	8.262	2	.016

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	42	12	0	77.8%
B	8	43	4	78.2%
C	0	6	11	64.7%
Overall Percentage	39.7%	48.4%	11.9%	76.2%

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	41.540	124.827	.111	1	.739	
	rhostirrup	-8.216	3.506	5.493	1	.019	.000
	sd	-.786	.282	7.761	1	.005	.455
	ds_dm	-3.435	1.043	10.850	1	.001	.032
	[fy=1]	7.139	6484.825	.000	1	.999	1260.071
	[fy=2]	-11.317	4.141	7.470	1	.006	1.217E-5
	[fy=3]	-16.780	124.338	.018	1	.893	5.159E-8
	[fy=4]	-19.774	124.351	.025	1	.874	2.584E-9
	[fy=5]	0[c]	.	.	0	.	.
	[fvy=1]	3.977	1.598	6.198	1	.013	53.369
	[fvy=2]	.107	.961	.012	1	.912	1.112
	[fvy=3]	-9.475	124.216	.006	1	.939	7.673E-5
	[fvy=4]	-1.537	1.039	2.190	1	.139	.215
	[fvy=5]	0[c]	.	.	0	.	.
	ad	-2.051	.871	5.541	1	.019	.129
C	Intercept	-6.119	9.389	.425	1	.515	
	rhostirrup	-23.306	10.900	4.572	1	.032	7.556E-11
	sd	.034	.306	.012	1	.912	1.034
	ds_dm	.925	.668	1.919	1	.166	2.523
	[fy=1]	5.469	.000	.	1	.	237.292
	[fy=2]	2.135	1.548	1.902	1	.168	8.454
	[fy=3]	.257	1.295	.039	1	.843	1.293
	[fy=4]	6.518	5.193	1.575	1	.209	676.939
	[fy=5]	0[c]	.	.	0	.	.
	[fvy=1]	-11.979	6378.725	.000	1	.999	6.275E-6
	[fvy=2]	-1.960	1.810	1.173	1	.279	.141
	[fvy=3]	2.596	3.277	.628	1	.428	13.409
	[fvy=4]	-.792	1.532	.267	1	.605	.453
	[fvy=5]	0[c]	.	.	0	.	.
	ad	-.202	.815	.061	1	.804	.817

a. The reference category is: B.

B.5.3 Canadian Code - Deep Beams

B.5.3.1 *Deep beams-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	57	91.9%
	B	2	3.2%
	C	3	4.8%
Cross Sectional Shape	0	43	69.4%
	1	8	12.9%
	2	7	11.3%
	3	4	6.5%
ag	0	4	6.5%
	2	31	50.0%
	3	27	43.5%
Anchorage	0	20	32.3%
	1	42	67.7%
Overall Crack Control Check	1	62	100.0%
fy	2	15	24.2%
	3	27	43.5%
	4	16	25.8%
	6	4	6.5%
Valid		62	100.0%
Missing		0	
Total		62	
Subpopulation		61[a]	

a. The dependent variable has only one value observed in 61 (100.0%) subpopulations.

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	45.493	49.747	41.493			
Final	40.479	48.987	32.479	9.014	2	.011

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	80.957	118	.996
Deviance	32.479	118	1.000

Pseudo R-Square

Cox and Snell	.135
Nagelkerke	.277
McFadden	.217

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	42.927	47.182	38.927	6.449	2	.040
StrutAngle	45.493	49.747	41.493	9.014	2	.011

Parameter Estimates

Y^a		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	3.237	2.527	1.641	1	.200	
	StrutAngle	.004	.078	.002	1	.963	1.004
C	Intercept	138.657	1.260	12113.782	1	.000	
	StrutAngle	-6.835	.000	.	1	.	.001

a. The reference category is: B.

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	57	0	0	100.0%
B	2	0	0	.0%
C	3	0	0	.0%
Overall Percentage	100.0%	.0%	.0%	91.9%

B.6 SPSS Output of Shear Friction

B.6.1 Beams without Shear Reinforcement

B.6.1.1 *Beams without Shear Reinforcement-SPSS output for all explanatory variables*

Case Processing Summary

		N	Marginal Percentage
Y	A	108	74.0%
	B	17	11.6%
	C	21	14.4%
Cross Sectional Shape	3	146	100.0%
ag	1	22	15.1%
	2	85	58.2%
	3	39	26.7%
fy	1	20	13.7%
	2	23	15.8%
	3	55	37.7%
	4	38	26.0%
	5	10	6.8%
Crack control R/F	0	4	2.7%
	1	142	97.3%
Anchorage	0	134	91.8%
	1	12	8.2%
Valid		146	100.0%
Missing		0	
Total		146	
Subpopulation		136[a]	

Model Fitting Information

	Model Fitting Criteria			Likelihood Ratio Tests		
Model	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	222.287	228.255	218.287			
Final	128.904	212.445	72.904	145.383	26	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	208.156	244	.953
Deviance	71.518	244	1.000

Pseudo R-Square

Cox and Snell	.631
Nagelkerke	.811
McFadden	.662

Likelihood Ratio Tests

	Model Fitting Criteria			Likelihood Ratio Tests		
Effect	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	128.904	212.445	72.904[a]	.000	0	.
fcMPa	137.816	215.389	85.816	12.912	2	.002
bw_dm	125.853	203.427	73.853	.949	2	.622
ds_dm	125.510	203.084	73.510	.606	2	.738
ad	126.559	204.132	74.559	1.655	2	.437
rhoshearlong.Steel	144.962	222.536	92.962	20.058	2	.000
CrossSectionalShape	128.904	212.445	72.904[a]	.000	0	.
ag	127.525	199.131	79.525	6.620	4	.157
fy	127.521	187.193	87.521	14.617	8	.067
CrackcontrolRF	126.300	203.874	74.300	1.396	2	.498
Anchorage	130.209	207.783	78.209	5.305	2	.070

a. This reduced model is equivalent to the final model because omitting the effect does not increase the degrees of freedom.

Classification

	Predicted			
Observed	A	B	C	Percent Correct
A	105	2	1	97.2%
B	7	9	1	52.9%
C	2	0	19	90.5%
Overall Percentage	78.1%	7.5%	14.4%	91.1%

Parameter Estimates

Y^a		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	42.430	4379.325	.000	1	.992	
	fcMPa	.084	.033	6.544	1	.011	1.087
	bw_dm	-.810	.926	.765	1	.382	.445
	ds_dm	-.094	.231	.165	1	.685	.911
	ad	.179	.683	.069	1	.793	1.196
	rhoshearlong.Steel	.334	.725	.212	1	.645	1.396
	[CrossSectionalShape=3]	0^b	.	.	0	.	.
	[ag=1]	-21.960	1621.495	.000	1	.989	2.904E-10
	[ag=2]	.619	1.031	.361	1	.548	1.857
	[ag=3]	0^b	.	.	0	.	.
	[fy=1]	-.479	3744.490	.000	1	1.000	.619
	[fy=2]	-24.422	3615.868	.000	1	.995	2.476E-11
	[fy=3]	-24.162	3615.868	.000	1	.995	3.209E-11
	[fy=4]	-22.114	3615.867	.000	1	.995	2.488E-10
	[fy=5]	0^b	.	.	0	.	.
	[CrackcontrolRF=0]	17.026	7058.145	.000	1	.998	24784069.568
	[CrackcontrolRF=1]	0^b	.	.	0	.	.
	[Anchorage=0]	-20.260	2470.619	.000	1	.993	1.589E-9
	[Anchorage=1]	0^b	.	.	0	.	.
C	Intercept	31.049	3066.302	.000	1	.992	
	fcMPa	.035	.048	.535	1	.465	1.036
	bw_dm	.105	.577	.033	1	.856	1.110
	ds_dm	.144	.251	.328	1	.567	1.155
	ad	-1.375	1.282	1.150	1	.284	.253
	rhoshearlong.Steel	-5.273	2.151	6.008	1	.014	.005
	[CrossSectionalShape=3]	0^b	.	.	0	.	.
	[ag=1]	-.110	2.833	.001	1	.969	.896
	[ag=2]	.957	2.016	.226	1	.635	2.605
	[ag=3]	0^b	.	.	0	.	.
	[fy=1]	-14.402	3066.299	.000	1	.996	5.561E-7
	[fy=2]	-18.429	3066.301	.000	1	.995	9.921E-9
	[fy=3]	-18.416	3066.300	.000	1	.995	1.005E-8
	[fy=4]	-13.640	3066.299	.000	1	.996	1.192E-6
	[fy=5]	0^b	.	.	0	.	.
	[CrackcontrolRF=0]	-.144	9799.050	.000	1	1.000	.866
	[CrackcontrolRF=1]	0^b	.	.	0	.	.
	[Anchorage=0]	-7.362	.000	.	1	.	.001
	[Anchorage=1]	0^b	.	.	0	.	.

a. The reference category is: B.

B.6.1.2 Beams without Shear Reinforcement-SPSS output for selected explanatory variables

Case Processing Summary

		N	Marginal Percentage
Y	A	108	74.0%
	B	17	11.6%
	C	21	14.4%
Valid		146	100.0%
Missing		0	
Total		146	
Subpopulation		121[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	218.704	224.671	214.704			
Final	130.202	148.103	118.202	96.502	4	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	216.280	236	.817
Deviance	113.807	236	1.000

Pseudo R-Square

Cox and Snell	.484
Nagelkerke	.622
McFadden	.439

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	170.434	182.368	162.434	44.232	2	.000
rhoshearlong.Steel	183.934	195.869	175.934	57.732	2	.000
fcMPa	152.614	164.548	144.614	26.412	2	.000

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	-4.235	1.274	11.055	1	.001	
	rhoshearlong.Steel	2.122	.624	11.555	1	.001	8.346
	fcMPa	.072	.025	8.145	1	.004	1.075
C	Intercept	2.972	1.313	5.127	1	.024	
	rhoshearlong.Steel	-2.704	1.049	6.639	1	.010	.067
	fcMPa	-.003	.033	.008	1	.928	.997

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	105	0	3	97.2%
B	12	0	5	.0%
C	5	0	16	76.2%
Overall Percentage	83.6%	.0%	16.4%	82.9%

Appendix C

SPSS OUTPUT FOR CRITICAL VALUE ESTIMATIONS

ACI Code Equation 11-3

Effective depth, d

d ≤ 3 dm

Case Processing Summary

		N	Marginal Percentage
Y	A	239	89.2%
	B	23	8.6%
	C	6	2.2%
ag	1	29	10.8%
	2	213	79.5%
	3	26	9.7%
Crack control R/F	1	268	100.0%
Valid		268	100.0%
Missing		0	
Total		268	
Subpopulation		264[a]	

Likelihood Ratio Tests

	Model Fitting Criteria			Likelihood Ratio Tests		
Effect	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	99.676	149.950	71.676[a]	.000	0	.
fcMPa	167.532	210.623	143.532	71.856	2	.000
ds_dm	116.303	159.395	92.303	20.627	2	.000
ad	109.711	152.803	85.711	14.035	2	.001
rhoshearlong.Steel	190.876	233.968	166.876	95.201	2	.000
ag	93.089	128.999	73.089	1.413	4	.842
CrackcontrolRF	99.676	149.950	71.676[a]	.000	0	.

Model Fitting Information

Model	Model Fitting Criteria AIC	BIC	-2 Log Likelihood	Likelihood Ratio Tests Chi-Square	df	Sig.
Intercept Only	217.286	224.468	213.286			
Final	99.676	149.950	71.676	141.610	12	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	692.436	514	.000
Deviance	71.676	514	1.000

Pseudo R-Square

Cox and Snell	.410
Nagelkerke	.748
McFadden	.664

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	8.341	2.966	7.908	1	.005	
	fcMPa	-.169	.040	18.002	1	.000	.844
	ds_dm	-2.236	1.041	4.608	1	.032	.107
	ad	-1.311	.417	9.903	1	.002	.269
	rhoshearlong.Steel	5.984	1.397	18.336	1	.000	396.959
	[ag=1]	.031	1.565	.000	1	.984	1.031
	[ag=2]	1.011	1.144	.782	1	.377	2.749
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
C	Intercept	-35.121	19.441	3.264	1	.071	
	fcMPa	.153	.073	4.342	1	.037	1.165
	ds_dm	10.940	6.101	3.215	1	.073	56366.858
	ad	.441	.773	.325	1	.569	1.554
	rhoshearlong.Steel	-4.305	2.191	3.861	1	.049	.014
	[ag=1]	-14.001	.000	.	1	.	8.306E-7
	[ag=2]	1.679	3.090	.295	1	.587	5.360
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.

a. The reference category is: B.

Classification

Observed	Predicted A	B	C	Percent Correct
A	237	2	0	99.2%
B	6	17	0	73.9%
C	1	1	4	66.7%
Overall Percentage	91.0%	7.5%	1.5%	96.3%

d ≤ 4 dm

Case Processing Summary

		N	Marginal Percentage
Y	A	262	88.8%
	B	26	8.8%
	C	7	2.4%
ag	1	29	9.8%
	2	225	76.3%
	3	41	13.9%
Crack control R/F	1	295	100.0%
Valid		295	100.0%
Missing		0	
Total		295	
Subpopulation		288[a]	

Model Fitting Information

	Model Fitting Criteria			Likelihood Ratio Tests		
Model	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	244.839	252.213	240.839			
Final	133.912	185.529	105.912	134.927	12	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	856.725	562	.000
Deviance	105.912	562	1.000

Pseudo R-Square

Cox and Snell	.367
Nagelkerke	.658
McFadden	.560

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	133.912	185.529	1.059E2	.000	0	.
fcMPa	191.269	235.512	167.269	61.357	2	.000
ds_dm	134.258	178.501	110.258	4.346	2	.114
ad	146.662	190.905	122.662	16.750	2	.000
rhoshearlong.Steel	220.878	265.121	196.878	90.966	2	.000
ag	127.576	164.445	107.576	1.664	4	.797
CrackcontrolRF	133.912	185.529	1.059E2	.000	0	.

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	3.751	1.839	4.158	1	.041	
	fcMPa	-.115	.025	20.864	1	.000	.891
	ds_dm	-.175	.499	.123	1	.726	.839
	ad	-1.080	.294	13.470	1	.000	.340
	rhoshearlong.Steel	4.261	.888	23.001	1	.000	70.856
	[ag=1]	-.545	1.314	.172	1	.678	.580
	[ag=2]	.158	.690	.053	1	.819	1.171
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
C	Intercept	-7.396	4.407	2.817	1	.093	
	fcMPa	.053	.026	4.051	1	.044	1.055
	ds_dm	1.802	1.063	2.870	1	.090	6.060
	ad	.191	.611	.097	1	.755	1.210
	rhoshearlong.Steel	-2.782	1.706	2.659	1	.103	.062
	[ag=1]	-14.200	.000	.	1	.	6.807E-7
	[ag=2]	1.607	1.531	1.101	1	.294	4.989
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.

a. The reference category is: B.

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	257	4	1	98.1%
B	12	14	0	53.8%
C	1	3	3	42.9%
Overall Percentage	91.5%	7.1%	1.4%	92.9%

d ≤ 5 dm

Case Processing Summary

		N	Marginal Percentage
Y	A	268	83.2%
	B	35	10.9%
	C	19	5.9%
ag	1	41	12.7%
	2	229	71.1%
	3	52	16.1%
Crack control R/F	0	3	.9%
	1	319	99.1%
Valid		322	100.0%
Missing		0	
Total		322	
Subpopulation		314[a]	

Model Fitting Information

	Model Fitting Criteria			Likelihood Ratio Tests		
Model	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	365.279	372.828	361.279			
Final	166.875	227.267	134.875	226.405	14	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	1765.153	612	.000
Deviance	134.875	612	1.000

Pseudo R-Square

Cox and Snell	.505
Nagelkerke	.749
McFadden	.627

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	166.875	227.267	1.349E2	.000	0	.
fcMPa	255.308	308.152	227.308	92.433	2	.000
ds_dm	182.629	235.472	154.629	19.754	2	.000
ad	181.636	234.479	153.636	18.761	2	.000
rhoshearlong.Steel	286.981	339.825	258.981	124.106	2	.000
ag	161.858	207.153	137.858	2.984	4	.561
CrackcontrolRF	169.665	222.509	141.665	6.791	2	.034

Parameter Estimates

Y^a		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	4.870	1.771	7.565	1	.006	
	fcMPa	-.134	.026	27.138	1	.000	.874
	ds_dm	-.738	.375	3.870	1	.049	.478
	ad	-1.106	.290	14.497	1	.000	.331
	rhoshearlong.Steel	4.823	.864	31.126	1	.000	124.301
	[ag=1]	-.885	1.002	.780	1	.377	.413
	[ag=2]	.554	.661	.702	1	.402	1.740
	[ag=3]	0^b	.	.	0	.	.
	[CrackcontrolRF=0]	3.289	1.801	3.333	1	.068	26.805
A	[CrackcontrolRF=1]	0^b	.	.	0	.	.
C	Intercept	-9.150	3.660	6.249	1	.012	
	fcMPa	.072	.023	9.521	1	.002	1.075
	ds_dm	1.864	.638	8.528	1	.003	6.449
	ad	.371	.412	.812	1	.367	1.450
	rhoshearlong.Steel	-2.454	1.177	4.352	1	.037	.086
	[ag=1]	-.276	1.225	.051	1	.822	.759
	[ag=2]	.593	1.101	.290	1	.590	1.809
	[ag=3]	0^b	.	.	0	.	.
	[CrackcontrolRF=0]	-2.865	2.161	1.758	1	.185	.057
	[CrackcontrolRF=1]	0^b	.	.	0	.	.

a. The reference category is: B.

Classification

Observed	Predicted A	Predicted B	Predicted C	Percent Correct
A	265	2	1	98.9%
B	12	20	3	57.1%
C	2	4	13	68.4%
Overall Percentage	86.6%	8.1%	5.3%	92.5%

$d \leq 6dm$

Case Processing Summary

		N	Marginal Percentage
Y	A	271	82.6%
	B	37	11.3%
	C	20	6.1%
ag	1	44	13.4%
	2	232	70.7%
	3	52	15.9%
Crack control R/F	0	3	.9%
	1	325	99.1%
Valid		328	100.0%
Missing		0	
Total		328	
Subpopulation		320[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	380.831	388.417	376.831			
Final	173.932	234.620	141.932	234.899	14	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	1890.462	624	.000
Deviance	141.932	624	1.000

Pseudo R-Square

Cox and Snell	.511
Nagelkerke	.749
McFadden	.623

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	173.932	234.620	1.419E2	.000	0	.
fcMPa	264.514	317.616	236.514	94.582	2	.000
ds_dm	187.744	240.846	159.744	17.812	2	.000
ad	189.587	242.689	161.587	19.655	2	.000
rhoshearlong.Steel	303.293	356.395	275.293	133.361	2	.000
ag	169.681	215.197	145.681	3.749	4	.441
CrackcontrolRF	176.170	229.273	148.170	6.239	2	.044

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	5.125	1.734	8.735	1	.003	
	fcMPa	-.137	.026	28.233	1	.000	.872
	ds_dm	-.827	.355	5.440	1	.020	.437
	ad	-1.093	.292	14.055	1	.000	.335
	rhoshearlong.Steel	4.861	.862	31.812	1	.000	129.101
	[ag=1]	-.972	.989	.966	1	.326	.378
	[ag=2]	.536	.665	.651	1	.420	1.710
	[ag=3]	0^b	.	.	0	.	.
	[CrackcontrolRF=0]	3.520	1.794	3.849	1	.050	33.787
A	[CrackcontrolRF=1]	0^b	.	.	0	.	.
C	Intercept	-6.548	2.837	5.325	1	.021	
	fcMPa	.064	.021	9.390	1	.002	1.066
	ds_dm	1.272	.482	6.964	1	.008	3.567
	ad	.487	.361	1.826	1	.177	1.628
	rhoshearlong.Steel	-2.755	1.135	5.894	1	.015	.064
	[ag=1]	-1.042	1.155	.814	1	.367	.353
	[ag=2]	.087	1.014	.007	1	.932	1.091
	[ag=3]	0^b	.	.	0	.	.
	[CrackcontrolRF=0]	-2.229	1.991	1.253	1	.263	.108
	[CrackcontrolRF=1]	0^b	.	.	0	.	.

a. The reference category is: B.

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	268	2	1	98.9%
B	13	21	3	56.8%
C	2	6	12	60.0%
Overall Percentage	86.3%	8.8%	4.9%	91.8%

d ≤ 7dm

Case Processing Summary

		N	Marginal Percentage
Y	A	272	81.2%
	B	39	11.6%
	C	24	7.2%
ag	1	44	13.1%
	2	235	70.1%
	3	56	16.7%
Crack control R/F	0	3	.9%
	1	332	99.1%
Valid		335	100.0%
Missing		0	
Total		335	
Subpopulation		327[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	411.607	419.235	407.607			
Final	198.182	259.208	166.182	241.424	14	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	1161.666	638	.000
Deviance	166.182	638	1.000

Pseudo R-Square

Cox and Snell	.514
Nagelkerke	.730
McFadden	.592

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	198.182	259.208	1.662E2	.000	0	.
fcMPa	284.378	337.776	256.378	90.196	2	.000
ds_dm	242.204	295.602	214.204	48.022	2	.000
ad	208.735	262.133	180.735	14.553	2	.001
rhoshearlong.Steel	321.413	374.811	293.413	127.231	2	.000
ag	193.150	238.920	169.150	2.968	4	.563
CrackcontrolRF	201.433	254.831	173.433	7.251	2	.027

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	5.229	1.463	12.773	1	.000	
	fcMPa	-.107	.019	30.515	1	.000	.898
	ds_dm	-1.068	.284	14.158	1	.000	.344
	ad	-.829	.251	10.924	1	.001	.436
	rhoshearlong.Steel	3.840	.658	34.042	1	.000	46.542
	[ag=1]	-.524	.912	.330	1	.566	.592
	[ag=2]	.335	.607	.305	1	.581	1.399
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	3.398	1.710	3.947	1	.047	29.891
A	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
C	Intercept	-6.306	2.222	8.057	1	.005	
	fcMPa	.066	.021	10.269	1	.001	1.068
	ds_dm	1.262	.389	10.532	1	.001	3.533
	ad	.449	.347	1.675	1	.196	1.566
	rhoshearlong.Steel	-2.509	.904	7.697	1	.006	.081
	[ag=1]	-1.312	1.146	1.310	1	.252	.269
	[ag=2]	-.674	.914	.544	1	.461	.510
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	-2.696	2.013	1.794	1	.180	.067
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.

a. The reference category is: B.

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	266	4	2	97.8%
B	14	21	4	53.8%
C	1	7	16	66.7%
Overall Percentage	83.9%	9.6%	6.6%	90.4%

d ≤ 8 dm

Case Processing Summary

		N	Marginal Percentage
Y	A	272	80.2%
	B	39	11.5%
	C	28	8.3%
ag	1	45	13.3%
	2	238	70.2%
	3	56	16.5%
Crack control R/F	0	3	.9%
	1	336	99.1%
Valid		339	100.0%
Missing		0	
Total		339	
Subpopulation		331[a]	

Model Fitting Information

Model	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	432.110	439.762	428.110			
Final	198.304	259.520	166.304	261.806	14	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	1195.761	646	.000
Deviance	166.304	646	1.000

Pseudo R-Square

Cox and Snell	.538
Nagelkerke	.750
McFadden	.612

Likelihood Ratio Tests

Effect	Model Fitting Criteria			Likelihood Ratio Tests		
	AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Chi-Square	df	Sig.
Intercept	198.304	259.520	1.663E2	.000	0	.
fcMPa	284.854	338.418	256.854	90.550	2	.000
ds_dm	252.385	305.949	224.385	58.081	2	.000
ad	208.905	262.469	180.905	14.600	2	.001
rhoshearlong.Steel	323.547	377.111	295.547	129.243	2	.000
ag	193.298	239.210	169.298	2.994	4	.559
CrackcontrolRF	201.672	255.236	173.672	7.368	2	.025

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	5.222	1.463	12.741	1	.000	
	fcMPa	-.107	.019	30.532	1	.000	.898
	ds_dm	-1.066	.284	14.090	1	.000	.345
	ad	-.830	.251	10.935	1	.001	.436
	rhoshearlong.Steel	3.840	.658	34.029	1	.000	46.507
	[ag=1]	-.524	.911	.330	1	.566	.592
	[ag=2]	.340	.607	.313	1	.576	1.404
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	3.394	1.710	3.941	1	.047	29.800
A	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
C	Intercept	-6.412	2.201	8.486	1	.004	
	fcMPa	.067	.021	10.470	1	.001	1.069
	ds_dm	1.296	.372	12.169	1	.000	3.655
	ad	.456	.347	1.725	1	.189	1.578
	rhoshearlong.Steel	-2.567	.886	8.388	1	.004	.077
	[ag=1]	-1.360	1.137	1.432	1	.231	.257
	[ag=2]	-.628	.898	.490	1	.484	.533
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	-2.742	2.013	1.855	1	.173	.064
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.

a. The reference category is: B.

Classification

Observed	Predicted			
	A	B	C	Percent Correct
A	266	4	2	97.8%
B	14	21	4	53.8%
C	1	7	20	71.4%
Overall Percentage	82.9%	9.4%	7.7%	90.6%

d > 8 dm

Case Processing Summary

		N	Marginal Percentage
Y	A	282	71.2%
	B	51	12.9%
	C	63	15.9%
ag	1	49	12.4%
	2	261	65.9%
	3	86	21.7%
Crack control R/F	0	11	2.8%
	1	385	97.2%
Valid		396	100.0%
Missing		0	
Total		396	
Subpopulation		380[a]	

Model Fitting Information

	Model Fitting Criteria			Likelihood Ratio Tests		
Model	AIC	BIC	-2 Log Likelihood	Chi-Square	df	Sig.
Intercept Only	633.391	641.354	629.391			
Final	250.878	314.581	218.878	410.513	14	.000

Goodness-of-Fit

	Chi-Square	df	Sig.
Pearson	1220.269	744	.000
Deviance	216.106	744	1.000

Pseudo R-Square

Cox and Snell	.645
Nagelkerke	.809
McFadden	.649

Likelihood Ratio Tests

Effect	Model Fitting Criteria: AIC of Reduced Model	BIC of Reduced Model	-2 Log Likelihood of Reduced Model	Likelihood Ratio Tests: Chi-Square	df	Sig.
Intercept	250.878	314.581	2.189E2	.000	0	.
fcMPa	369.696	425.436	341.696	122.818	2	.000
ds_dm	364.340	420.079	336.340	117.462	2	.000
ad	264.300	320.039	236.300	17.421	2	.000
rhoshearlong.Steel	400.984	456.724	372.984	154.106	2	.000
ag	254.070	301.847	230.070	11.192	4	.024
CrackcontrolRF	268.053	323.793	240.053	21.175	2	.000

Parameter Estimates

Y[a]		B	Std. Error	Wald	df	Sig.	Exp(B)
A	Intercept	4.532	1.190	14.513	1	.000	
	fcMPa	-.105	.018	33.407	1	.000	.900
	ds_dm	-.830	.134	38.147	1	.000	.436
	ad	-.868	.232	13.952	1	.000	.420
	rhoshearlong.Steel	3.806	.600	40.231	1	.000	44.953
	[ag=1]	-.389	.864	.203	1	.652	.678
	[ag=2]	.468	.566	.683	1	.408	1.596
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	4.589	1.664	7.609	1	.006	98.404
A	[CrackcontrolRF=1]	0[b]	.	.	0	.	.
C	Intercept	-4.187	1.663	6.339	1	.012	
	fcMPa	.074	.017	19.750	1	.000	1.077
	ds_dm	.595	.139	18.404	1	.000	1.813
	ad	.244	.297	.672	1	.412	1.276
	rhoshearlong.Steel	-2.000	.584	11.735	1	.001	.135
	[ag=1]	.090	.867	.011	1	.918	1.094
	[ag=2]	-1.660	.669	6.161	1	.013	.190
	[ag=3]	0[b]	.	.	0	.	.
	[CrackcontrolRF=0]	-3.645	1.222	8.891	1	.003	.026
	[CrackcontrolRF=1]	0[b]	.	.	0	.	.

a. The reference category is: B.

Classification

Observed	Predicted: A	B	C	Percent Correct
A	274	7	1	97.2%
B	16	28	7	54.9%
C	4	7	52	82.5%
Overall Percentage	74.2%	10.6%	15.2%	89.4%