# University of Alberta

## Evaluating Shear Capacity of Concrete Members with Deficient Shear Reinforcement

by

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## Abstract

This study assesses the suitability of four sectional shear methods for predicting the shear capacity of reinforced concrete members which do not comply with S6-06 Section 14 stirrup spacing and area requirements. The results of the evaluations indicate that the sectional shear provisions in S6-06, AASHTO LRFD-05 and software Response 2000 appropriately account for variations in stirrup spacing and area detailing, and present with good agreement between predicted and tested shear capacities for member with deficient shear reinforcement. However, shear capacities calculated using ACI 318-08 do not agree well with tested capacities for members with less than minimum stirrups. Two modified shear methods are proposed, which revise the diagonal crack spacing and concrete contribution area assumed by S6-06. The modified shear methods improve predictions of shear capacity relative to predictions calculated using S6-06 and eliminate the issue of non-convergent shear capacity predictions which can result from evaluation using S6-06.

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## Latin Lower Case Symbols

| | |
|---|---|
| $a$ | shear span length (mm) |
| $a_g$ | maximum size of coarse aggregate (mm) |
| $b$ | compression zone width (mm) |
| $b_v$ | effective shear width, determined in accordance with S6-06 Clause 8.9.1.6 (mm) |
| $c$ | diagonal distance to the nearest reinforcement in the section (mm) |
| $c_x$ | distance from midsection to longitudinal reinforcement (mm) |
| $c_y$ | distance from midsection to transverse reinforcement (mm) |
| $d$ | flexural depth (mm) |
| $d_b$ | bar or strand diameter (mm) |
| $d_{hx}$ | bar diameter of longitudinal reinforcement (mm) |
| $d_{hy}$ | bar diameter of transverse reinforcement (mm) |
| $d_v$ | shear depth, calculated as the greater of $0.9 \cdot d$ and $0.72 \cdot h$ (mm) |
| $f_1$ | principal tensile stress in the concrete (MPa) |
| $f_2$ | principal compressive stress in the concrete (MPa) |
| $f_{2,\max}$ | maximum average compressive stress (MPa) |
| $f_c'$ | specified concrete strength from test documentation (MPa) |
| $f_{cr}$ | tensile cracking strength of concrete (MPa) |
| $f_{pe}$ | stress in prestressing after losses (MPa) |

| | |
|---|---|
| $f_{po}$ | stress in prestressed reinforcement when stress in surrounding concrete is zero (MPa) |
| $f_{pu}$ | guaranteed ultimate tensile strength of prestressing steel (MPa) |
| $f_{sx}$ | average reinforcement response in the longitudinal direction (MPa) |
| $f_{sxcr}$ | local reinforcement response at a crack in the longitudinal direction (MPa) |
| $f_{sy}$ | average reinforcement response in the transverse direction (MPa) |
| $f_{sycr}$ | local reinforcement response at a crack in the transverse direction (MPa) |
| $f_v$ | yield strength of web reinforcement (MPa) |
| $f_{xx}$ | stress applied to an element in the x direction (MPa) |
| $f_y$ | yield strength of longitudinal reinforcement (MPa) |
| $f_{yy}$ | stress applied to an element in the y direction (MPa) |
| $h$ | overall section height (mm) |
| $jd$ | approximation of flexural lever arm (mm) |
| $k_1$ | coefficient of bond characteristic from CEB-FEP (1978) |
| $kd$ | depth to the cracked neutral axis (mm) |
| $l_{bearing}$ | bearing length of the girder (mm) |
| $l_d$ | development length of the longitudinal reinforcement (mm) |
| $r$ | $\frac{A_v}{s \cdot b}$, ratio for shear reinforcement using is ACI 318-56 |
| $s, s_y$ | longitudinal stirrup spacing (mm) |
| $s_{mx}$ | spacing of cracks in the longitudinal direction (mm) |

| | |
|---|---|
| $s_{my}$ | spacing of cracks in the transverse direction (mm) |
| $s_{m\theta}$ | spacing of cracks in the $\theta$ direction (mm) |
| $s_{m1}$ | S6-06 Section 14 maximum stirrup spacing limit (mm) |
| $s_{m2}$ | S6-06 Section 14 maximum stirrup spacing limit (mm) |
| $s_{\max}$ | AASHTO LRFD-05/ACI 318-08 maximum stirrup spacing limit (mm) |
| $s_x$ | spacing of the longitudinal reinforcement (mm) |
| $s_z$ | crack spacing term (mm) |
| $s_{ze}$ | effective crack spacing term (mm) |
| $t_f$ | thickness of flange (mm) |
| $v$ | applied shear stress (MPa) |
| $v_c$ | shear resistance attributed to the concrete (MPa) |
| $v_{ci}$ | shear stress that can be transmitted across a crack (MPa) |
| $v_u$ | average factored shear stress (MPa) |
| $w$ | crack width (mm) |
| $w_D$ | uniform dead load (kN/m) |

**Latin Upper Case Symbols**

| | |
|---|---|
| $A$ | gross area of concrete section (mm$^2$) |
| $A_{ct}$ | area of concrete in tension (mm$^2$) |
| $A_{cv}$ | area of concrete attributed to shear capacity including flange contributions (mm$^2$) |
| $A_{flange}$ | area of concrete flange attributed to shear capacity (mm$^2$) |

| | |
|---|---|
| $A_p$ | prestressing steel area on tension half of section ($mm^2$) |
| $A_s$ | longitudinal steel area on tension half of section ($mm^2$) |
| $A_v$ | cross-sectional area of web reinforcement ($mm^2$) |
| $A_{v,\min}$ | minimum permissible area of transverse reinforcement ($mm^2$) |
| $A_{web}$ | area of concrete web attributed to shear capacity ($mm^2$) |
| $DP$ | demerit points allotted to a test specimen |
| $E_c$ | modulus of elasticity of concrete (MPa) |
| $E_s$ | modulus of elasticity of steel reinforcement (MPa) |
| $E_p$ | modulus of elasticity of prestressing reinforcement (MPa) |
| $F$ | live load capacity factor |
| $L$ | member length (mm) |
| $M_{applied}$ | externally applied moment at a section (N·mm) |
| $M_D$ | dead load moment at a section (N·mm) |
| $M_f$ | factored moment on a section (N·mm) |
| $N_f$ | axial force applied on a section (N) |
| $P$ | prestressing force (N) |
| $T_r$ | tensile resistance of the longitudinal reinforcement (N) |
| $U$ | S6-06 Section 14 reliability factor |
| $V'$ | ACI Standard Specification No. 23 Force Carried by the Stirrups, calculated as $V_f - 0.02 \cdot f_c' \cdot b \cdot jd$ (N) |
| $V_{anchorage}$ | anchorage capacity of the longitudinal reinforcement (N) |
| $V_{applied}$ | externally applied shear force at a section (N) |

| | |
|---|---|
| $V_c$ | shear resistance attributed to the concrete (N) |
| $V_{calc}$ | calculated shear capacity at a critical section (N) |
| $V_D$ | dead load shear force at a section (N) |
| $V_f, V_u$ | factored shear force on a section (N) |
| $V_n$ | nominal shear resistance (N) |
| $V_s$ | shear resistance attributed to the stirrups (N) |
| $V_p$ | prestressing shear resistance contribution (N) |
| $V_r$ | shear resistance of the concrete at a section (N) |
| $V_{test}$ | shear force at critical section determined from test results (N) |

**Greek Symbols**

| | |
|---|---|
| $\alpha$ | angle between transverse and longitudinal steel |
| $\alpha_{LF}$ | S6-06 Section 14 load factor |
| $\beta_r$ | S6-06 Section 14 reliability index |
| $\varepsilon_1$ | principal tensile strain |
| $\varepsilon_2$ | principal compressive strain |
| $\varepsilon_c$ | strain in concrete |
| $\varepsilon_c^{'}$ | strain in the concrete corresponding to the peak compression stress |
| $\varepsilon_{prestrain}$ | strain in prestressing steel at transfer |
| $\varepsilon_x$ | idealized longitudinal strain at mid-depth |
| $\gamma$ | term derived to account for S6-06 Clause 14.14.1.6.2 stirrup area interpolation |

| | |
|---|---|
| $\phi$ | material resistance factor |
| $\phi_c$ | concrete resistance factor |
| $\phi_p$ | prestressing steel resistance factor |
| $\phi_s$ | steel reinforcement resistance factor |
| $\rho_x$ or $\rho$ | longitudinal reinforcing ratio $\frac{A_s}{b \cdot d}$ (in decimal format for calculations) |
| $\rho_r$ | longitudinal reinforcement ratio as determined by Response 2000 |
| $\rho_y$ | transverse reinforcing ratio $\frac{A_v}{s \cdot b}$ (in decimal format for calculations) |
| $\theta$ | calculated angle of the principal compression stress field (°) |
| $\theta_T$ | angle of compression flange area contributing to shear capacity |

# Chapter 1
# Introduction

## 1.1 Background

Structural evaluation for one-way shear capacity of existing reinforced and prestressed concrete structures is a process vital to public safety. The way in which structures carry shear is complex, particularly in composites such as reinforced concrete. Over the last century it has become clear that some form of regulation must be provided to ensure that all structural concrete members are designed and evaluated to strict standards. In order to develop these standards, committees of governing organizations rely upon academic and industrial research to produce simplified provisions. These provisions provide guidance to practicing engineers for the design and evaluation of reinforced concrete structures. In North America these governing bodies include the Canadian Standards Association (CSA), American Concrete Institute (ACI) and the American Association of State Highway and Transportation Officials (AASHTO).

Canadian bridges in service today are routinely evaluated for load carrying capabilities, both as part of scheduled maintenance programs and to accommodate loads which exceed standard legal limits. Changes in design/evaluation provisions can also necessitate load evaluations, to assess capacity according to the revised standards. Evaluating the flexural capacity of reinforced and prestressed concrete members is simplified by the fact that a rational theory exists for bending forces (Rusch, 1960), which is based on the well known hypothesis that plane sections remain plane in Bernoulli regions of members. However, for shear forces, a universally accepted rational theory does not yet exist although not for lack of effort. Over the past century several thousand tests of concrete members fabricated and loaded to fail in shear have been carried out with the intent of developing such a rational theory. As these tests were typically focused more on assessing shear capacity than determining the behavior of concrete critically loaded in shear, the goal of developing a rational theory for shear resistance has so far been left largely unachieved. Instead, multiple models suggesting how shear is carried in reinforced concrete have been developed. Two models are examined in this study: the shear model for sectional behavior given by Joint ASCE-ACI

Committee 426 (ASCE-ACI 426, 1973) and the Modified Compression Field Theory (Vecchio and Collins, 1986).

Based on the Modified Compression Field Theory and the Joint ASCE-ACI Committee 426 shear models, four simplified methods for evaluating shear capacity in reinforced concrete members have been selected for predicting shear capacity in this study. The following three sectional shear methods which have been selected for evaluating shear capacities in this study are based on the relationships from the Modified Compression Field Theory:

- the sectional design method for shear (Bentz and Collins, 2006) used in the Canadian Highway Bridge Design Code S6-06 (CSA, 2006),
- the General Method (Collins et al., 1996) for shear used in the American Association of State Highway and Transportation Officials AASHTO LRFD-05 Design Code (AASHTO, 2005), and
- software Response 2000 (Bentz, 2000)

The fourth method used in this study is the sectional shear provisions in the Building Code Requirements for Structural Concrete ACI 318-08 (ACI, 2008), which are empirical and use the shear model provided in ASCE-ACI 426 (1973). This method was derived based on research carried out primarily in the 1950's and 1960's. For concrete members with stirrups, a transverse reinforcement contribution to shear capacity based on Ritter's $45^{\circ}$ Truss Model was employed (as cited in Hognestad, 1951).

The focus of this study is to assess how well predictions of shear capacity for members not complying with S6-06 Section 14 stirrup spacing and area requirements agree with corresponding tested shear capacities, based on calculations using the four sectional shear methods. Because S6-06 Section 14 contains the provisions used in Canada which are specific to evaluation of existing bridge structures, members that were non-compliant with respect to this Section of S6-06 were selected for evaluation in this study. It should be noted that the non-compliant members evaluated in this study could have been compliant with respect to the provisions for which they were designed and fabricated to. To provide a quantitative comparison, members that comply with S6-06

Section 14 spacing and area requirements as well as members without stirrups were also evaluated.

Typical industrial practice has always been to use governing design provisions for evaluating the shear capacity of existing structures. This practice has two major shortcomings, the first being the discrepancy between the objectives of design and evaluation and the second being that as new research is developed and published, design standards will be revised to reflect this new knowledge. Consequently, structural details consistent with previous editions of a standard may not comply with the requirements of new versions.

The goal of design is to always have a capacity which is greater than or equal to the loading demands on the structure. Achieving this design goal ensures that the structure will have a load path capable of carrying the predicted loads and will thus prevent failure. Evaluating the load capacity of an existing structure, however, requires having a method which is able to accurately predict the capacity of the structure. Accurate evaluations of load carrying capacity are essential in practice. Over-predicting the capacity of a structure is inherently dangerous, as it may lead to the failure of the structure, while under-predicting capacity can result in unnecessary replacement or strengthening of an already adequate structure. In order to accomplish accurate predictions of member capacity, the method used for evaluation must take into account as many of the significant parameters of the load-carrying mechanisms as possible. The evaluation method must also be applicable to details which do not comply with provisions primarily intended for design of new structures. If an evaluation tool is successful, it will calculate predicted-to-test capacity ratios which are close to unity and which have low scatter for an overall set of studied members.

One complication which can be encountered during load evaluations occurs when details of design incorporated into a structure based on a previous standard do not comply with provisions for evaluation in the current standard. Shear reinforcement provisions pertaining to stirrup spacing and area requirements are two details which have continually changed over the past half century. Due to changes in standards with respect to these two details, many members that were constructed based on older codes do not comply with stirrup spacing and area requirements contained in the current standards. Evaluation provisions are typically derived from academic publications; however academic research seldom evaluates details that are not in conformance with prevailing design/evaluation standards. Applicability of the four sectional shear methods used in this study to evaluate the capacity of members which do not comply with S6-06 Section 14 requirements for stirrup spacing and area is an important consideration, as the suitability of these methods to predict the shear capacity of such members has not been well established and non-compliant members are found in numerous structures throughout Canada's roadway system.

## 1.2 Objectives and Scope of Work

The following objectives have been set for this study in consideration of the issues previously discussed:

- Illustrate why members typically become non-compliant with respect to S6-06 Section 14 stirrup spacing and area requirements.
- Assemble a dataset of non-prestressed and prestressed members containing stirrups which do not comply with S6-06 Section 14 stirrup spacing and area requirements. In order to allow a comparison of shear capacity predictions to be made, a collection of non-prestressed and prestressed members with compliant stirrup details, as well as members without stirrups has been assembled.
- Evaluate the shear capacity of members not complying with S6-06 Section 14 stirrup spacing and area requirements using the four sectional shear methods identified in Section 1.1. Although ductility of the failure mode is an important issue, the lack of information in the reviewed test documentation regarding ductility of non-compliant members renders study of this issue difficult, and as such it has been deemed out of scope. Flexural resistance, web crushing and longitudinal reinforcement anchorage capacity of the members has been checked using S6-06 Clause 8.8, S6-06 Clause 8.9.3.3 and S6-06 Clause 8.9.3.14 respectively to ensure that these modes of failure do not govern for the members studied.
- Evaluate the shear capacity of members complying with S6-06 Section 14 stirrup spacing and area requirements and members without stirrups using the four sectional shear capacity methods outlined in Section 1.1.

- Assess the agreement between predicted and tested shear capacities calculated using the four sectional shear methods for members which do not comply with S6-06 Section 14 stirrup spacing and area requirements by comparing predicted to tested shear capacities of the non-compliant members with the compliant members evaluated in this and other studies.
- Identify and explain the issue of non-convergence of shear capacity predictions that can arise when using S6-06 Section 8 and Section 14 shear provisions.
- Develop and validate modifications to the sectional shear provisions in S6-06 Section 8 to improve shear capacity predictions and eliminate the issue of non-convergent shear evaluations.
- Provide an evaluation tool in the form of a flowchart to assist engineers with the shear capacity evaluation of existing reinforced concrete structures presenting with non-compliant stirrup spacing and area details. The method illustrated in the flowchart is equally applicable to compliant members and members without stirrups.

## 1.3 Organization of Thesis

Chapter 2 of this study presents a review of how design provisions relating to stirrup spacing and area requirements have changed over time. This discussion is followed by a summary of previous research specifically focused on evaluation of members with non-compliant stirrup details. Chapter 2 also provides a review of the shear models from which the sectional methods used in this study are derived. The criteria identified as being critical for a method to address in order to declare that the method is suitable for predicting sectional shear capacity are discussed.

Chapter 3 discusses the four sectional shear methods used in this study and the assumptions and procedures used for evaluation. Shear capacity ratios ($V_{calc}/V_{test}$) and corresponding coefficients of variation (COV) derived from other studies using these methods are provided. The ratio $V_{calc}/V_{test}$ has been used in this study because it disperses members with unsafe predictions of shear capacity ($V_{calc} > V_{test}$) further away from unity ($V_{calc}/V_{test}$ equal to 1.00) than members with safe predictions. For example, if $V_{calc}$ = 2.00 and $V_{test}$ = 1.00, the ratio $V_{calc}/V_{test}$ equals 2.00 while the reciprocal ratio $V_{test}/V_{calc}$ equals

0.50. Essentially the ratio $V_{calc}/V_{test}$ statistically 'favors' conservative predictions while the ratio $V_{test}/V_{calc}$ statistically 'favors' unsafe predictions.

Chapter 4 provides the results of the shear evaluation of the members identified for analysis using S6-06, AASHTO LRFD-05, Response 2000 and ACI 318-08 in this study. A demerit point model is proposed, which is based on the demerit point model presented by Collins (2001). This discussion is followed by a description of the criteria for member selection used in this study. Chapter 4 also identifies deficiencies from evaluation using S6-06 shear provisions which warrant further investigation in this study.

Chapter 5 presents two modified sectional shear evaluation methods which are based on the provisions in S6-06 Section 8. The same members evaluated in Chapter 4 are re-evaluated using the two modified shear methods. The results of the evaluations are used to indicate improvements to the accuracy of the shear capacities predictions relative to those from the S6-06 sectional shear. Chapter 5 concludes with a flow chart which provides a method for evaluating the sectional shear capacity of concrete members.

Chapter 6 discusses the conclusions from this study and proposes future research related to evaluating the shear capacity of concrete members with stirrups which do not comply with S6-06 Section 14 stirrup spacing and area requirements. This study focuses specifically on the *strength* of members with deficient shear reinforcement. Future research will be useful for determining the *behavior* of shear critical non-compliant members; specifically ductility and diagonal crack spacing $s_z$ (see Section 2.4.2.i). Future research will also be useful for validating the findings in this study.

Chapter 7 provides the references used in this study.

Appendix A provides the geometric, reinforcing and loading properties of the members evaluated in this study. Appendix B provides sample calculations illustrating how sectional shear methods used in this study were applied. Appendix C provides a case study which examines the issue of non-convergent shear capacity predictions.

# Chapter 2
# Literature Review

## 2.1 Introduction

Chapter 2 discusses the literature relevant to evaluating the sectional shear capacity of concrete members with non-compliant stirrup spacing and area details. Section 2.2 reviews changes in design provisions for shear reinforcement over time in order to demonstrate how reinforced concrete members are found to be non-compliant with respect to S6-06 Section 14 stirrup spacing and area requirements. Section 2.3 discusses previous research specific to evaluation of members with non-compliant stirrup spacing and area details. Section 2.3 is limited in its discussion as research specific to members with these non-compliant details was found to be scarce. Section 2.4 discusses the shear capacity models from which the four sectional shear design/evaluation methods detailed in Section 1.1 were derived. The two shear models reviewed are the model presented in ASCE-ACI 426 (1973) for sectional shear analysis, and the Modified Compression Field Theory (Vecchio and Collins, 1986), from which simplifications produced the shear provisions in S6-06 and AASHTO LRFD-05. Software Response 2000 (Bentz, 2000) also uses the relationships from the Modified Compression Field Theory for sectional shear analysis. Section 2.5 discusses significant parameters which are known to affect shear capacity of concrete members. Section 2.6 details the Demerit Point model proposed by Collins (2001). Section 2.7 presents criteria critical for a shear method to address in order to be able to declare that the method is suitable for predicting the sectional shear capacity of concrete members.

## 2.2 Occurrence of Members with Non-Compliant Stirrup Spacing and Area Details due to Changes in Shear Provisions

This section examines the current maximum stirrup spacing and minimum stirrup area requirements for evaluation of shear capacity using Section 14 of the Canadian Highway Bridge Design Code S6-06, and provides the stirrup design requirements from previous standards. This comparison illustrates the primary reason members in service today may not comply with current stirrup spacing and area requirements.

*Current Stirrup Spacing and Area Requirements for Canadian Bridge Shear Capacity Evaluations*

The current requirements for stirrup spacing and area used for evaluations of bridge structures in Canada are contained in Section 14 of the Canadian Highway Bridge Design Code S6-06. Evaluation of concrete members using the sectional provisions in S6-06 Section 8 is discussed in depth in Section 3.2.

S6-06 Section 14 stirrup spacing limits are determined based on shear demand and overall member height using Figure 2.1. The maximum stirrup spacing is determined from each of the two plots in Figure 2.1, with the lesser of the two values governing as the maximum permissible stirrup spacing. These limits were established based on the experience and judgment of the CSA S6-06 Section 14 technical committee as opposed to direct research results. The $s_{m1}$ line provides the maximum spacing limit below which members are considered compliant in stirrup spacing. The $s_{m2}$ line provides the limit above which S6-06 Section 14 requires using no stirrup contribution to shear capacity. The stirrup spacing limit $s_{m2}$ is discussed further in Section 4.5.1. S6-06 Clause 14.14.1.6.2 requires that the predicted shear resistance be determined by interpolation between these limits, although it is not specific on how or what to interpolate.

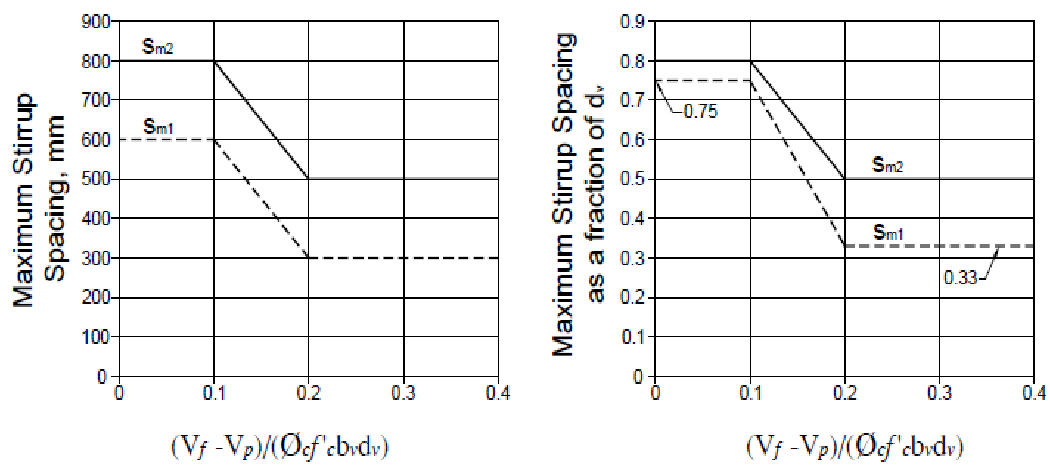



Figure 2.1 – S6-06 Section 14 Maximum Stirrup Spacing Requirements

S6-06 Section 14 also provides two limits for minimum permissible stirrup area. Eqn. (2.1) provides the minimum stirrup area whereby members containing equal or greater stirrup area are considered compliant for this detail.

$$A_v \geq 0.15 \cdot f_{cr} \cdot \frac{b_v s}{f_v} \qquad (\text{mm}^2) \qquad \text{Eqn. (2.1)}$$

Eqn. (2.2) provides the limit below which S6-06 Section 14 requires using no contribution to shear resistance from the stirrups. Linear interpolation of the stirrup contribution to shear capacity $V_s$ is required between the limits in Eqn. (2.1) and Eqn. (2.2), although S6-06 is not specific on how to interpolate. A means of interpolating shear resistance when the provided stirrup area is between Eqn. (2.1) and Eqn. (2.2) is provided in the flowchart found in Section 3.2. Discussion of the shear capacity attributed to the stirrups by the sectional method in S6-06 Section 8 is also found in Section 3.2 of the present study. Eqns. (2.1) and (2.2) are based on experience, as opposed to research and testing (Bentz, 2005).

$$A_v \leq 0.05 \cdot f_{cr} \cdot \frac{b_v s}{f_v} \qquad (\text{mm}^2) \qquad \text{Eqn. (2.2)}$$

S6-06 Section 14 does not provide any interpolation for the concrete contribution to shear capacity $V_c$ as members transition from being compliant to non-compliant with respect to stirrup spacing and area details. However, the assumption for diagonal crack spacing changes based on whether the member complies with minimum stirrup requirements, which can in turn affect the concrete contribution to shear capacity. Determining the assumed diagonal crack spacing and concrete contribution to shear resistance using the provisions in S6-06 Section 8 is discussed further is Section 3.2 of the present study.

*Previous Stirrup Detail Design Requirements*

Many reinforced concrete structures were constructed before generally accepted expressions for calculating shear capacity were introduced. The design of these structures was based on the judgment and practicality of the engineer, as well as on analytical methods and reinforcement details which had traditionally resulted in good performance. However, the emergence of more and more reinforced concrete structures in the years just prior to 1900 necessitated the development of a rational design theory. The problem at that time with producing design provisions for shear in reinforced concrete was that theories that are now known to be correct were not understood by most engineers. Two different schools of thought on how shear is carried by reinforced concrete structures existed in the early 1900's. Some engineers felt that shear in concrete was an issue of diagonal tension, while others felt that the concern was related to horizontal shear. This debate between diagonal tension and horizontal shear was concluded around 1910 (Hognestad, 1953), at which time the diagonal tension approach became widely accepted in North American practice.

What was known was that the use of web reinforcement dramatically increased the shear that could be carried in concrete members. The first known paper to present an analysis method for members with web reinforcement was published in 1899 by W. Ritter. Ritter presented what is known today as the truss analogy for shear capacity, which determines the stirrup contribution to shear resistance using Eqn. (2.3) (as cited in Hognestad, 1953):

$$V_s = \frac{A_v f_v jd}{s} \quad \text{(N)} \qquad \text{Eqn. (2.3)}$$

The first National Association of Cement User's (NACU) report appeared in 1908 (Hognestad, 1953). This report is the basic foundation of what became Ultimate Load Design for reinforced concrete. Reinforced concrete sections were dimensioned on an ultimate basis for loads that were four times the working loads. It was specified that "the shearing strength of 2000 psi (13.8 MPa) concrete shall be taken as 200 psi (1.38 MPa)" and that "when the shearing stresses developed in any part of a reinforced concrete constructed building exceed, under the multiplied loads, the shearing strength as fixed in this section, a sufficient amount of steel shall be introduced in such a position that the deficiency in the resistance of the shear is overcome" (as cited in Hognestad, 1953). No formulas for determining either shear stresses or design of web reinforcement were included in the first NACU report. Details pertaining to stirrup spacing and area were left to the designer's judgment and experience. Overall member height was typically the governing factor for determining maximum stirrup spacing. The only inclusion of shear demand in the 1908 NACU standard was whether or not members required stirrups. This model for shear design was used until 1920.

A “Special Committee on Unit Values for Vertical Shear in Reinforced Concrete Design” reported to the American Concrete Institute in 1920 (Hognestad, 1953). ACI Standard Specification No. 23, published in 1920, was based on their recommendations. This ACI code permitted the following nominal shear stresses (ACI-ASCE 326, 1962).

1) Beams without web reinforcement $0.02f_c^{'} \leq 0.42MPa$
2) Beams without web reinforcement but with special anchorage of the longitudinal reinforcement $0.03f_c^{'} \leq 0.62MPa$
3) Beams with web reinforcement designed using $A_v f_v = \frac{V's}{jd}\sin\alpha$ $0.06f_c^{'} \leq 1.24MPa$
4) Beams with web reinforcement and special anchorage of the longitudinal reinforcement $0.12f_c^{'} \leq 2.48MPa$

Similar to the NACU report, ACI Standard No. 23 did not provide any requirements for maximum stirrup spacing or minimum stirrup area details. These were left to the designer’s judgment and experience. Hence, selected stirrup spacing could have been considerably greater than permitted today. Insufficient stirrup area could have been an issue as well, but is less likely as the concrete compressive strengths used at that time were low relative to current typical values, as a review of historical design drawings indicates.

When ACI Standard 318-56 (ACI, 1956) was introduced, Committee 318 altered the 1920 ceiling values for maximum allowable shear stress. ACI 318-56 put limits on the maximum shear stress in beams without web reinforcement as 0.62 MPa, 1.65 MPa for beams with stirrups or bent up bars, and 2.48 MPa for beams with both stirrups and bent up bars. ACI 318-56 was the first American shear design provision to give a requirement of the minimum web reinforcement quantity. ACI 318-56 called for a minimum of $r = \frac{A_v}{sb} = 0.15\%$ when stirrups were required, although this requirement appeared to have been inspired by practical experience as opposed to specific research.

The provisions in ACI 318-56 provided no guidelines for maximum stirrup spacing, although a common practice was to space stirrups at $d/2$. Based on an assumed $45^{\circ}$ crack angle, this practice would ensure that every diagonal crack was intercepted by a stirrup. A stirrup spacing of $d/2$ could be spaced up to 1.68 times further apart than permitted by S6-06 Section 14 ($\frac{0.5 \cdot d}{0.33 \cdot 0.9 \cdot d}$=1.68, where $0.33 \cdot 0.9 \cdot d$ is the S6-06 maximum allowable stirrup spacing based on a shear demand $\frac{V_f - V_p}{\phi_c \cdot f_c^{'} \cdot b_v \cdot d_v} > 0.20$ - see Figure 2.1).

ACI 318-63 (ACI, 1963) was the first ACI standard to specify a maximum stirrup spacing limit. This standard required that every diagonal crack be intercepted by a stirrup. Thus, based on an assumed crack angle of $45^{\circ}$, stirrups were not permitted to be spaced greater than a maximum distance of *d/2* apart (ACI-ASCE 326, 1962). Because this spacing limit is independent of shear demand, stirrups spaced at *d/2* could be as much as 1.68 times greater than permitted using the stirrup spacing limits provided in S6-06 Section 14. During the 1960's, concrete inverted channel sections became a standard shape used in Alberta bridges. It was not uncommon for the stirrups in each leg in these channel sections to be spaced at *d* apart in the longitudinal direction and for the stirrups to be offset in one leg by *d/2* respective to the other leg. Thus if the two legs could be viewed simultaneously, the stirrups would appear to be spaced at *d/2*. However, since each leg of the channel will form diagonal cracks independent of the other leg, stirrups spaced at *d* could be up to 3.34 times further apart than permitted by S6-06 Section 14. ACI 318-63 also required a minimum stirrup reinforcing ratio of $r = \frac{A_v}{sb} = 0.15\%$ when stirrups were required. Note that this area requirement was not related to concrete strength as in current standards. However members fabricated when ACI 318-63 was the governing standard typically comply with S6-06 Section 14 stirrup area requirements. This is related to the lower strength of concrete used during the time when ACI 318-63 governed design of concrete structures.

ACI 318-71 (ACI, 1971) presented new requirements for shear reinforcement. Stirrups were required when:

1) The beam depth exceeded 254 mm, 2.5 times the flange thickness or ½ the web width.
2) The applied ultimate shear, $\upsilon_u$, was greater than $\frac{1}{2} \cdot \phi_c \cdot \upsilon_c$, where $\upsilon_c$ was the shear resistance attributed to the concrete.

When shear reinforcement was needed according to ACI 318-71 requirements, members were required to have a minimum area of transverse steel $A_v$ capable of carrying 0.35 MPa of shear stress. The minimum required stirrup area was determined using Eqn (2.4):

$$A_{v,\min} = 0.35 \frac{b_v s}{f_v} \qquad (\text{mm}^2) \qquad \text{Eqn. (2.4)}$$

The origins of the requirement that stirrups be designed to carry a minimum shear stress of 0.35 MPa are not entirely clear, which suggests that Eqn. (2.4) was based more on practical experience and judgment than on specific experimental data (Krauthammer, 1992). The requirement that stirrups be designed to carry a minimum shear stress of 0.35 MPa was implemented to produce sufficient ductility to protect against sudden, brittle failure of the member after critical flexural-shear cracking (ASCE-ACI 426, 1973). These stirrups, when placed perpendicular to the flexural reinforcement, were required to have a longitudinal spacing not exceeding the lesser of 0.5*d* for non-prestressed members, 0.75*h* for prestressed members, or 600 mm. These stirrup spacing limits are maintained in the current ACI 318-08 Section 11 shear provisions. Since ACI 318-71 did not include shear demand as a factor for determining the maximum allowable stirrup spacing, stirrups detailed according to ACI 318-71 provisions could be spaced up to 1.68 times further apart than permitted by S6-06 Section 14.

The ACI 318-90 (ACI, 1990) design standard limited the value of $\sqrt{f_c^{'}}$ for shear calculations to 8.30 MPa unless the member met the minimum ACI 318-90 shear reinforcement requirements. Tests published by Roller and Russell (1990) indicate that as the concrete strength increases, the minimum area of shear reinforcement required to achieve a ductile failure also needs to increase. ACI 318-90 required the minimum stirrup area calculated using Eqn. (2.4) to be multiplied by the ratio $\frac{f_c^{'}}{35}$ for members with concrete strengths greater than 35 MPa. This ratio had an upper limit of 3.0. This requirement for minimum stirrup area remained in use until publication of the ACI 318-02 (ACI, 2002) design standard. ACI 318-02 provided Equation (2.5), which saw the minimum web reinforcement requirement for non-prestressed members increase gradually as the concrete strength increased, but at the same time maintained a minimum value:

$$A_{v,\min} = 0.06\sqrt{f_c^{'}}\frac{b_v s}{f_v} \text{ but not less than } 0.35\frac{b_v s}{f_v} \qquad (\text{mm}^2) \qquad \text{Eqn. (2.5)}$$

These two terms become equal when $f_c^{'}$ is equal to 34.0 MPa.

Equation (2.5) was later validated by Yoon et al. (1996) and found to be both safe and appropriate. There have been no other significant changes to the minimum stirrup requirements in ACI 318 shear provisions since the 2002 standard was released.

Canada's concrete design standard followed the ACI 318 code for many years, first as part of the National Building Code, and then in 1966 as the CSA A23.3 Design of Concrete Structures standard. In 1977, the Canadian Concrete Design Code A23.3-M77 (CSA, 1977) appeared for the first time in metric units. All subsequent sectional shear provisions in Canadian standards have been published in metric units, but many still resembled the sectional shear method in ACI 318. A23.3 minimum transverse reinforcement requirements from 1977 until 1984 were the same as in the ACI 318-71 shear method, although A23.3-84 contained alternative requirements based on the Compressive Field Theory (Mitchell and Collins, 1974).

The Canadian Highway Bridge Design Code S6-88 (CSA, 1988) used the same sectional shear method as ACI 318-71, including the same requirements for stirrup spacing and area. S6-88 required a minimum area of shear reinforcement and that every diagonal crack be intercepted by a stirrup in all flexural members except slabs and footings. Based on Ritter's 45° Truss Model, the maximum stirrup spacing was restricted to the smaller of:

- 0.50$d$
- 600 mm

S6-88 stirrup spacing requirements were based on member height, and did not consider shear demand. Stirrups spaced at $d/2$ could be up to 1.68 times larger than permitted using the Canadian Highway Bridge Design Code S6-06, assuming a shear demand $\frac{V_f - V_p}{\phi_c f_c^{'} b_v d_v} > 0.20$.

S6-88 required that stirrups carry a minimum shear stress of 0.35 MPa when shear reinforcement was required. The requirement for minimum shear reinforcement area in S6-88 is given as Eqn. (2.4).

S6-00 (CSA, 2000) was the first version of the Canadian Highway Bridge Design Code to consider shear demand within the maximum stirrup spacing requirement. For design, S6-00 Section 8 provided the following requirements for maximum longitudinal stirrup spacing:

- Lesser of 600 mm or 0.75$d_v$ if $\frac{v_u}{\phi_c f_c^{'}} < 1.0$
- Lesser of 300 mm or 0.33$d_v$ if $\frac{v_u}{\phi_c f_c^{'}} \geq 1.0$

The shear provisions in S6-00 assumed a variable truss angle based on the sectional forces and reinforcement configuration, as opposed to assuming a constant diagonal compression field angle of $45^{\circ}$. The stirrup spacing requirements and assumption of a varying truss angle remain the same in S6-06 Section 8, although the assumption used to predict the truss angle differs between the two methods. As both provisions are dependent on shear demand, it is improbable that members designed in accordance with S6-00 Section 8 will be non-compliant with respect to stirrup spacing requirements found in S6-06 Section 14. To determine the required minimum stirrup area, S6-00 used Eqn. (2.1). As Eqn. (2.1) is the same expression used in S6-06 Section 8, members designed using S6-00 will have a stirrup area which is compliant with S6-06 requirements.

## 2.3 Previous Research Focused on Shear Predictions of Concrete Members with Non-Compliant Stirrup Spacing and Area Details

Research specifically addressing shear capacity of members not complying with S6-06 Section 14 stirrup spacing and area requirements is sparse in current literature. Although there exist some published test results for members which do not meet S6-06 Section 14 stirrup spacing and area requirements, discussion in these publications was not focused on non-compliant details. As discussed in Section 1.1, members found to be non-compliant with respect to S6-06 Section 14 minimum transverse reinforcement requirements could have been compliant with respect to the stirrup spacing and area provisions in the design standard for which they were design to. This section discusses previous research which focused specifically on evaluation of members not complying with stirrup spacing and area details.

The Center for Frontier Engineering Research (CFER) presented a report entitled "Shear Tests of Type "E" Precast Concrete Bridge Girders" (DeGeer and Stephens, 1993) which they prepared for Alberta Transportation and Utilities. This report provided the test results of four decommissioned precast type 'E' girders loaded to fail by one-way shear. 'E' girders are inverted channel girders which were used to form the superstructure of some Alberta bridges. The first test specimen was a single 9 m precast 'E' girder, tested to failure under a single point load. The second test specimen was a single 12 m precast 'E' girder, tested to failure under a single point load. The last test specimen consisted of two 9 m precast 'E' girders shear connected together, and tested to failure under a single point load. The test specimens were all simply supported, and were analyzed for shear capacity using software Response Version 1 (Felber, 1990), which was based on the relationships in the Modified Compression Field Theory (Vecchio and Collins, 1986), and the sectional shear provisions in S6-88. Actual material properties of the concrete and rebar were tested at the University of Alberta, and were used for analysis.

The single 9 m girder was loaded with a shear span-to-depth ratio, *a/d,* of 5.84. Due to the fracture of a longitudinal reinforcing bar in the north leg, the member failed in flexure prior to reaching the critical shear load. Consequently this member was not considered in this study.

The single 12 m girder (PE 1) was loaded with a shear span to depth ratio of 3.67 and failed in shear with 'some but not a lot of ductility' (Yu, 1993). The first stirrup in the critical leg was located at 762 mm from the concentrated load in a member with a flexural depth of 528 mm, which is non-compliant with respect to current S6-06 Section 14 shear reinforcement provisions. This spacing is also significantly more than the specified stirrup spacing of 254 mm shown on the Alberta Transportation standard drawing (Alberta Transportation, 1962) for 12 m E Girders. Based on the actual concrete strength (as well as the specified design concrete strength), stirrup size and stirrup spacing, the member was also non-compliant with respect to S6-06 transverse reinforcement area requirements. The predicted shear capacity of PE 1 calculated by S6-88 and Response (Felber, 1990) was 318 kN and 363 kN respectively, which was in 'appropriate' agreement (see Table 4.1) with the tested shear capacity of 426 kN (DeGeer and Stephens, 1993). At failure, PE 1 had reached 80% of its flexural capacity predicted using provisions in S6-06 Section 8.

For the double 9 m girder test (PE 2), the interior legs were loaded with a shear span-to-depth ratio, *a/d*, of 3.60, and showed some ductility under maximum load while 'additional load was transferred to the exterior legs' (Yu, 1993). The first stirrup of the north interior girder leg was located at 982 mm from the concentrated load in a member with a flexural depth of 528 mm, which is non-compliant with respect to S6-06 Section 14 maximum stirrup spacing requirements. This spacing is significantly more than the specified stirrup spacing of 254 mm shown on the Alberta Transportation standard drawing (Alberta Transportation, 1962) for 9 m E girders. Based on the actual concrete strength (as well as the specified design concrete strength), stirrup size and spacing, the member was also non-compliant with respect to S6-06 transverse reinforcement area requirements. The predicted shear capacity of PE 2 calculated by S6-88 and Response (Felber, 1990) was 157 kN and 151 kN respectively, which was in good agreement with the tested shear capacity of 178 kN (DeGeer and Stephens, 1993). At failure, PE 2 had reached 79% of its flexural capacity, predicted using provisions in S6-06.

Despite the considerable spacing between stirrups, Yu (1993) concluded that the transverse reinforcement provided good support to the bottom bars and enhanced dowel action of the longitudinal reinforcement contributing to shear capacity. Discussion in DeGeer and Stephens (1993) concluded that sectional shear methods based on the MCFT (Vecchio and Collins, 1986) would be appropriate for calculating the shear capacity of members with non-compliant stirrup spacing and area details. Specimen PE1 was calculated to have $V_{calc}/V_{test}$ ratios of 0.75 and 0.85 from evaluation of shear capacity using S6-88 and Response (Felber, 1990) respectively. Specimen PE2 was calculated to have $V_{calc}/V_{test}$ ratios of 0.88 and 0.85 from evaluation of shear capacity using S6-88 and Response (Felber, 1990) respectively. There is no discussion in the CFER report (DeGeer and Stephens, 1993) explaining why member PE2 presented with better agreement between predicted and tested shear capacities than did member PE1.

Angelakos et al. (2001) used the shear provisions in AASHTO LRFD-00 (AASHTO, 2000) to evaluate the shear capacity of twenty-one large ($h$ = 1000 mm) rectangular members, five of which did not comply with AASHTO LRFD-00 minimum stirrup area requirements. AASHTO LRFD-05 provisions require a stirrup area which is 38 % to 84% greater than the stirrup area required by S6-06 Section 14, depending on the density of the concrete used to determine the concrete cracking strength $f_{cr}$ as per S6-06 Clause 8.4.1.8.1. S6-06 and AASHTO LRFD-05 minimum stirrup area requirements are provided in Section 3.2 and Section 3.3 of the present study respectively. The ratio $\frac{A_v \cdot f_v}{b_v \cdot s}$ for the 5 non-compliant members was kept constant at 0.401 MPa. Four of the non-compliant members had longitudinal reinforcing ratios $\rho$ of 1.01% while the other non-compliant member had a longitudinal reinforcement ratio of 0.76%. All members had a constant cross-section and length, and were loaded in the same manner with a single point load at an *a/d* ratio of 2.92. Of the remaining sixteen members, only one had stirrups and these complied with AASHTO LRFD-00 (and S6-06) minimum stirrup requirements. The authors proposed a method to determine shear capacity of members fabricated with less than AASHTO LRFD-00 minimum stirrup area requirements by interpolating between the shear capacity of those members assuming they complied with minimum stirrup area requirements, and assuming the same member had no stirrups. This proposed shear evaluation method is illustrated in Figure 2.2. Figure 2.3 shows that the actual shear capacity of the five members with non-compliant stirrup area fell between the shear capacity predictions assuming compliant stirrup details and assuming no shear reinforcement. This should be appreciated, as it suggests that tested shear capacities of members not complying with minimum stirrup area requirements will typically be bounded by AASHTO LRFD-05 shear capacity predictions assuming compliant stirrup details and shear capacity predictions assuming no stirrups. AASHTO LRFD-00 and ASHTO LRFD-05 have the same requirement for minimum stirrup area – this requirement is provided in Section 3.3 and varies from the minimum stirrup area requirements in S6-06 which are provided in Section 3.2.

The mean $V_{calc}/V_{test}$ ratio and coefficient of variation from the evaluation of the five members which were non-compliant with respect to AASHTO LRFD-00 stirrup area requirements were 1.04 and 19.5% respectively, calculated using the interpolation proposed by Angelakos et al. (2001). The mean $V_{calc}/V_{test}$ ratio and coefficient of variation of the twenty-one reinforced concrete members evaluated for shear capacity were 1.00 and 13.8% respectively. These statistical values have been derived in the present study based on predictions given in Angelakos et al. (2001). The authors' proposed method calculated shear capacities which were in good agreement with their corresponding actual capacities for members with deficient stirrup area. However this proposed shear method did not modify predictions of shear capacity for members with non-compliant stirrup spacing details relative to evaluation using AASHTO LRFD-00.

As discussed in Angelakos (1999), test specimen DB140M had a stirrup fail during testing, which accounts for this test specimen demonstrating less shear resistance than was predicted using the sectional shear method in AASHTO LRFD-98. Test documentation discussing specimen DB120M does not provide any indication as to why this member failed prior to reaching its predicted shear capacity. The proposed interpolation line in Figure 2.3 does not apply to specimen BM100 because the proposed interpolation line was plotted for a longitudinal reinforcement ratio of 1.01% and specimen BM100 had a reinforcement ratio of 0.76%. Figure 2.3 suggests that variations in concrete strength have a notable effect on the agreement between predicted and tested shear capacities calculated using the interpolation method proposed by Angelakos et al (2001). The effect of concrete strength on the tested shear capacity of concrete members is discussed in Section 2.5.1.

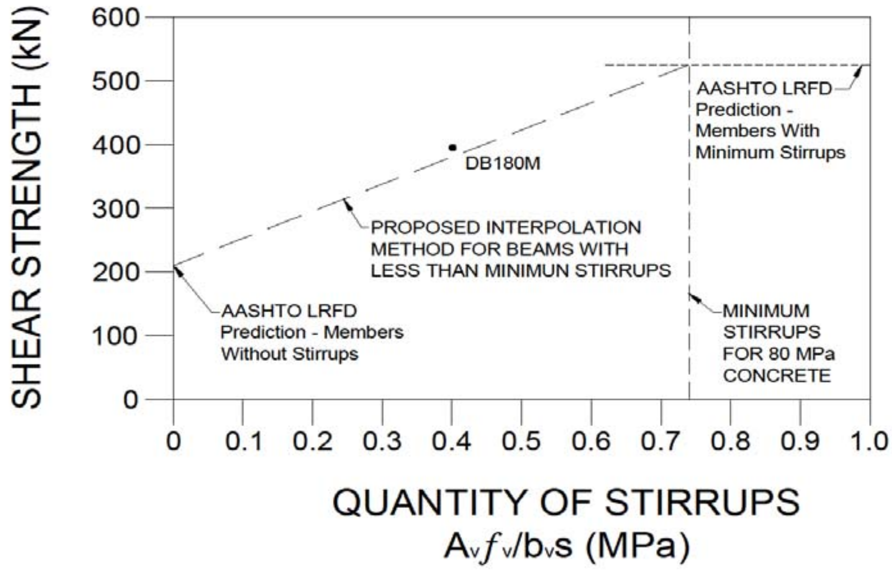



Figure 2.2 - Prediction of Shear Capacity by using Method Proposed by Angelakos et al. (2001) – Adapted from Angelakos et al (2001)

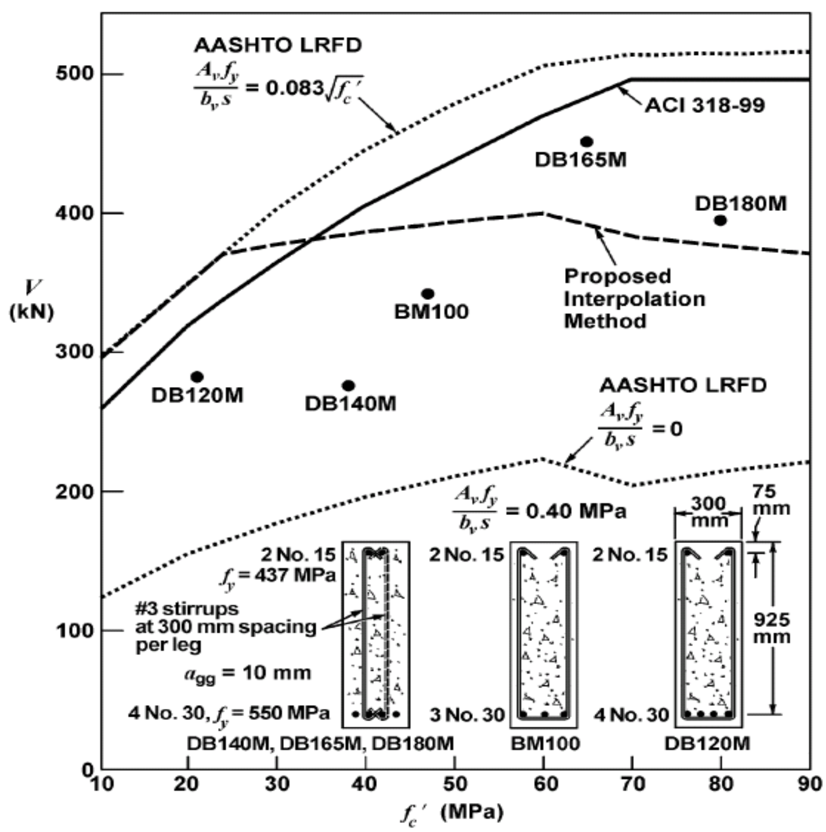



Figure 2.3 – Proposed Stirrup Area Interpolation (Angelakos et al., 2001)

## 2.4 Shear Models

The design/evaluation provisions used in this study are derived from two shear models – the empirical model presented in ASCE-ACI 426 (1973) and adopted by ACI Committee 318, and the Modified Compression Field Theory (Vecchio and Collins, 1986). These are the shear models which are most common in North American practice. This Section discusses these two models.

### 2.4.1 ACI 318 Shear Model

The shear capacity model used by ACI 318 consists of two distinct components contributing to shear capacity – a concrete contribution and a stirrup contribution.

*ACI 318 Concrete Contribution to Shear Capacity*

Research in the 1950's and 1960's was conducted to investigate the behavior of concrete members failing in one-way shear. ACI-ASCE Committee 326 (1962) recommended that the following concepts be considered to enable the development of a rational design standard:

1) Diagonal tension is a combined stress problem. Hence, the horizontal tensile stress component is the resultant of both bending and shearing stresses.
2) Failure by shear may result due to the formation of a critical shear crack or by deterioration of the compression zone due to shear if redistribution of internal forces is accomplished.
3) The load that caused the formation of the critical diagonal tension crack is to be considered as the design load for beams without web reinforcement. Tests of members with no transverse reinforcement are found to fail very close to this load.
4) Distribution of flexural and shear stresses over a cross section are not known. Concerning shear, the use of the average shear calculated as $v = \frac{V_f}{bd}$ was considered sufficient.

The criterion for the design method proposed by ACI-ASCE Committee 326 was that, using the expression $v = \frac{V_f}{bd}$ for the average shear stress, the usable ultimate shear strength of a member without shear reinforcement was the diagonal tension stress at cracking. Based on this criterion, a study of more than 440 specimens was conducted (ACI-ASCE 326, 1962), and parametric analysis indicated that the main factors influencing the nominal shear strength $v_n$ were:

1) the nominal shear strength $v_n$ increases as concrete strength increases;
2) the nominal shear strength $v_n$ decreases as the ratio $\frac{M_f}{V_f d}$ increases;
3) the nominal shear strength $v_n$ increases as the longitudinal reinforcement ratio $\rho$ increases.

ACI-ASCE Committee 326 (1962) recommended Eqn. (2.6) for calculating the shear capacity of non-prestressed concrete members subject to flexure and shear forces but no significant axial forces. This expression is still provided in ACI 318-08 as a method to predict the sectional shear capacity of non-prestressed members.

$$v_c = 0.16\sqrt{f_c^{'}} + 17 \cdot \rho \cdot \frac{V_f d}{M_f} \quad \text{(MPa)} \qquad \text{Eqn. (2.6)}$$

ACI 318 shear provisions from 1963 until the current 2008 standard allow Eqn. (2.6) to be simplified as given in Eqn. (2.7). As discussed in Section 3.5, Eqn. (2.7) is used in this study for predicting the shear capacity attributed by ACI 318-08 to the concrete for non-prestressed members.

$$v_c = \frac{1}{6} \cdot \sqrt{f_c^{'}} \quad \text{(MPa)} \qquad \text{Eqn. (2.7)}$$

MacGregor and Hanson (1969) recommended using Eqn. (2.8) for calculating the concrete contribution to shear capacity for members having an effective prestressing force $f_{pe}$ of at least 40% of the ultimate tensile strength of the prestressing steel $f_{pu}$. This expression was adopted by ASCE-ACI 426 (1973). As discussed in Section 3.5, Eqn. (2.8) is used in this study.

$$v_c = 0.05 \cdot \sqrt{f_c^{'}} + 5 \cdot \frac{V_f \cdot d}{M_f} \qquad \text{(MPa)} \qquad \text{Eqn. (2.8)}$$

*ACI 318 Stirrup Contribution to Shear Capacity*

In order to prevent sudden shear failure at the formation of diagonal cracking, ASCE-ACI Committee 426 (1973) recommended stirrups to be detailed in concrete members when the applied shear loading exceeded half of the calculated concrete contribution to shear capacity $(V_u > 0.5 \cdot \phi \cdot V_c)$. The stirrup contribution to shear resistance was based on Ritter's 45° Truss Model for evaluating the post-cracking capacity of reinforced concrete beams containing stirrups (ACI-ASCE 326, 1962). Ritter treated the longitudinal reinforcement as the bottom chord of the truss, the flexural compression zone as the top chord, the diagonal concrete compression struts as the diagonal members of the truss and the web reinforcement as the vertical truss members. In 1902 Morsch presented an independent version of the 45° Truss Model which was an improvement on Ritter's model, as Morsch allowed the diagonal struts to extend across more than one stirrup (Hognestad, 1953). The 45° Truss Models proposed by Ritter and Morsch neglected concrete's ability to carry tensile stresses after cracking, and assumed that the post-diagonal cracking compression field angle remained constant at 45°. Figure 2.4 shows the equilibrium condition used in the 45° Truss Model. This equilibrium condition led to Eqn. (2.9), which predicts the shear force carried by transverse reinforcement in reinforced concrete. The truss model was introduced to American literature in 1907 by Withey (as cited in Hognestad, 1953).

$$V_s = \frac{A_v \cdot f_v \cdot jd}{s} \qquad \text{(N)} \qquad \text{Eqn. (2.9)}$$

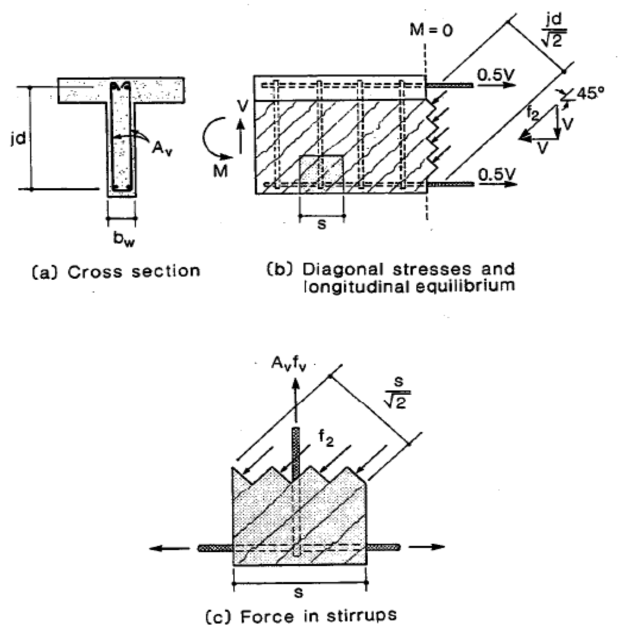



Figure 2.4 - 45° Truss Model Equilibrium (Collins and Mitchell, 1991)

## 2.4.2 Modified Compression Field Theory

The Modified Compression Field Theory (MCFT) (Vecchio and Collins, 1986) is a model which predicts the load-deformation behavior of reinforced concrete members subject to in-plane shear and normal forces. An essential assumption to the MCFT is that cracked concrete in reinforced members can be treated as a new material with an empirically derived stress-strain behavior. Based on the assumption of having a cracked reinforced concrete element large enough to include several cracks, Vecchio and Collins (1986) presented equilibrium and compatibility expressions and stress-strain relationships for average stress/average strain conditions. These relationships are presented in Figure 2.5. The average stresses and strains implicitly include longitudinal and transverse stresses and strains in the concrete and reinforcement over a length large enough to include several cracks. The Modified Compression Field Theory makes the reasonable assumption that the axes of the principal stresses and principal strains in the cracked concrete coincide. Analysis by Vecchio and Collins (1986) indicates that this assumption is typically accurate to within ±10°.

Average stresses and strains do not give information specific to local variations. Tensile stresses in reinforcement vary from a maximum at a crack to minimum between cracks, while concrete tensile stresses are zero at a crack and maximum between cracks. Modified Compression Field Theory (Vecchio and Collins, 1986) requires that a crack check be made to ensure that the average crack stresses are compatible with the local conditions at a crack. This crack check limits the principal tensile stress in the concrete to a maximum permissible value based on the ability of the cracked concrete surface and the steel reinforcement at the crack to transmit shear stresses. This local crack check also limits the shear stresses that can be transferred across a cracked surface (see Section 2.4.2.ii).

Figure 2.5 provides the equilibrium equations, geometric conditions and average stress-strain relationships used by the Modified Compression Field Theory (Vecchio and Collins, 1986).

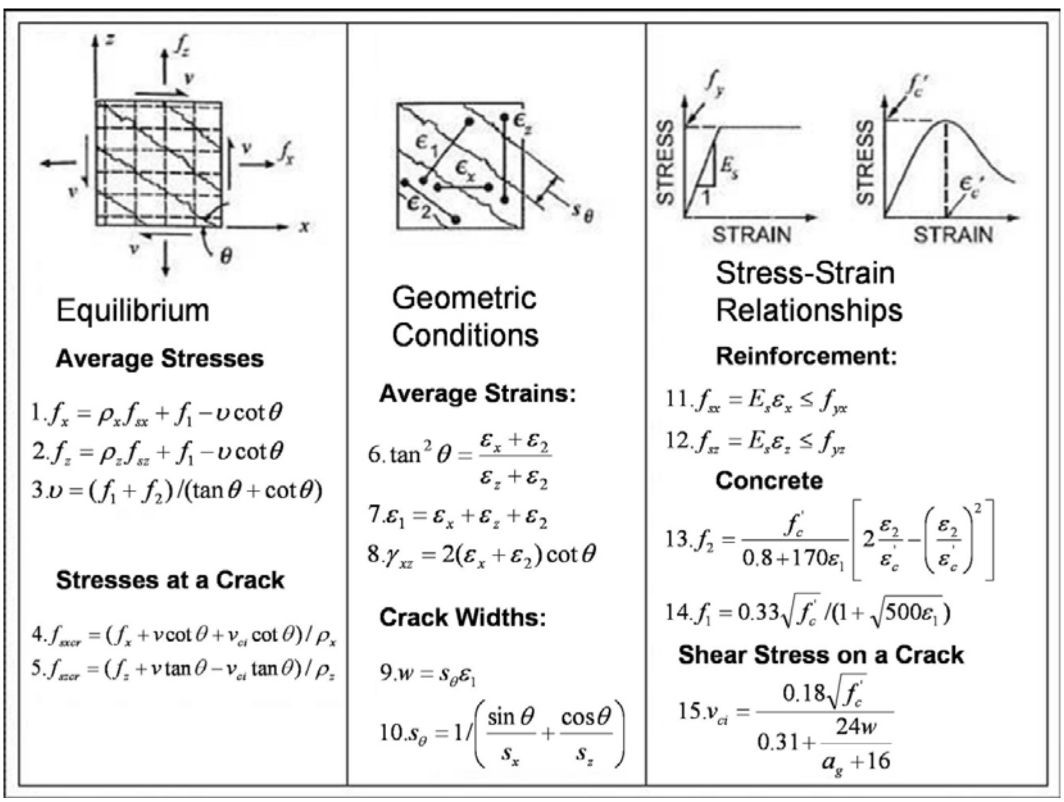



Figure 2.5 – Modified Compression Field Theory Equations (Bentz and Collins, 2006)

As discussed by Collins and Mitchell (1991), the following six aspects are important for defining the Modified Compression Field Theory.

### 2.4.2.i Crack Width (*w*) and Crack Spacing ($s_z$)

The Modified Compression Field Theory (Vecchio and Collins, 1986) simplifies the complex cracking history of reinforced concrete members into a single set of parallel cracks formed along the predicted angle of the principal compressive stresses. The predicted angle of the principal compressive stresses is taken relative to the longitudinal axis of the member. The spacing of the parallel cracks is calculated using Eqn. (2.10):

$$s_{m\theta} = \frac{1}{\left(\frac{\sin\theta}{s_{mx}} + \frac{\cos\theta}{s_{my}}\right)} \qquad \text{(mm)} \qquad \text{Eqn. (2.10)}$$

$s_{mx}$ and $s_{my}$ are the average crack spacing in the longitudinal and vertical directions respectively, and are both taken from CEB-FIP (1978). Eqn. (2.11) and Eqn. (2.12) account for details such as bond, the spacing, quantity and size of reinforcing steel, maximum distance from the crack to the reinforcing steel, and tensile strain in the concrete embedment zone.

$$s_{mx} = 2 \cdot \left(c_x + \frac{s_x}{10}\right) + 0.25 \cdot k_1 \cdot \frac{d_{hx}}{\rho_x} \qquad \text{(mm)} \qquad \text{Eqn. (2.11)}$$

$$s_{my} = 2 \cdot \left(c_y + \frac{s_y}{10}\right) + 0.25 \cdot k_1 \cdot \frac{d_{hy}}{\rho_y} \qquad \text{(mm)} \qquad \text{Eqn. (2.12)}$$

The average crack width is calculated in the Modified Compression Field Theory as the product of the average principal tensile strain and the diagonal crack spacing using Eqn. (2.13):

$$w = s_{m\theta} \cdot \varepsilon_1 \qquad \text{(mm)} \qquad \text{Eqn. (2.13)}$$

Research has indicated that diagonal crack spacing for members without stirrups is approximately equal to the member depth (Base, 1982; Bentz and Buckley, 2005). This assumption of diagonal crack spacing has been incorporated into MCFT based sectional shear provisions.

Studies investigating the relationship between diagonal crack spacing and the longitudinal spacing of stirrups were found to be sparse. Dilger and Divakar (1987) examined rectangular members with heights of 300 mm and with stirrup spaced at either 130 mm or 150 mm. Although the stirrup spacing range was small, the authors concluded that a correlation exists between diagonal crack spacing and longitudinal stirrup spacing. This is evident in crack spacing diagrams presented in Dilger and Divakar (1987). The authors also tested members with the same geometry but without stirrups. The members with stirrups showed a more uniform crack spacing pattern compared to members without stirrups. Members with stirrups had a diagonal crack spacing to average diagonal crack spacing ratio ranging from 0.76 to 1.24, while the same ratio for members without stirrups ranged from 0.67 to 1.33.

As shown in Figure 2.6 the correlation between diagonal crack spacing and longitudinal spacing of stirrups is also apparent in other studies (Angelakos, 1999; Yoshida, 2000).

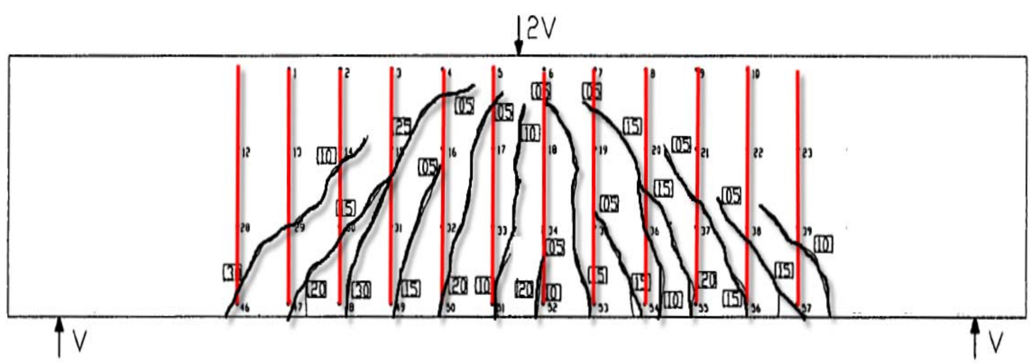



Figure 2.6 – DB120M Cracking Spacing Diagram (derived from Angelakos, 1999)

Angelakos (1999) presented diagonal crack spacing diagrams for five rectangular members with heights of 1000 mm and with stirrups spaced at either 300 mm or 600 mm. These diagrams indicate that diagonal crack spacing corresponded well with both longitudinal stirrup spacing details. Yoshida (2000) presented 'three' test specimens containing stirrups. These members were fabricated with heights of 2000 mm and with stirrups spaced at 590 mm, 1350 mm and 2700 mm. In each case, the crack drawings indicate that the spacing between diagonal cracks is similar to the corresponding longitudinal stirrup spacing. Properly scaled photographs would be more exact, but these crack diagrams can still give a reasonable indication of diagonal crack spacing.

Lubell (2006) proposed a modified shear method in which the spacing of diagonal cracks was assumed to be equal to the longitudinal spacing of the stirrups for members complying with A23.3-04 (CSA, 2004) minimum transverse reinforcement requirements. For members with less than minimum reinforcement Lubell assumed that the diagonal crack spacing was equal to the shear depth $d_v$. Using these assumptions for diagonal crack spacing, a modified method for evaluating sectional shear capacity, based on the provisions in A23.3-04 and titled CSA-M, was proposed and used to predict the shear capacity of 106 members with stirrups. Table 2.1 provides $V_{calc}/V_{test}$ ratios and corresponding COV values, derived in the present study, comparing predictions of shear capacity calculated using CSA-M with other sectional shear evaluation methods given in Lubell (2006). Comparison of values in Table 2.1 indicates that predictions of shear capacity calculated using Lubell's modified shear method CSA-M are in good agreement with tested shear capacities. This proposed shear method and assumption of diagonal crack spacing is discussed further in Section 5.2.

Table 2.1 –Comparison of Models derived from Lubell (2006)

| | CSA A23.3 | ACI 318 | R2K | CSA-M |
|---|---|---|---|---|
| $V_{calc}/V_{test}$ | 0.87 | 0.83 | 1.02 | 0.94 |
| COV | 16.5% | 23.1% | 15.1% | 15.3% |

### 2.4.2.ii Shear Transfer on Cracked Surface ($v_{ci}$)

The shear stress which can be carried along a cracked surface increases due to:

i) a decrease in crack width (Fenwick and Paulay, 1968),

ii) an increase in aggregate size (Walraven, 1981; Sherwood et al., 2007) and,

iii) an increase in concrete strength, up to approximately 60 MPa (Walraven, 1987).

Based on experimental work by Walraven (1981), Bhide and Collins (1989) presented Eqn. (2.14) to estimate the shear resistance resulting from aggregate interlock for the case where only shear stress is transmitted across a cracked surface. Eqn. (2.14) is a simplification of a similar expression presented in Vecchio and Collins (1986). It should be noted that Eqn. (2.14) neglects the beneficial effect of having compressive stress on the cracked surface. Discussion in Bhide and Collins (1989) suggests that neglecting compressive stresses on the crack surface is a reasonable assumption.

$$v_{ci} = \frac{0.18 \cdot \sqrt{f_c^{'}}}{0.31 + \dfrac{24 \cdot w}{a_g + 16}} \qquad \text{(MPa)} \qquad \text{Eqn. (2.14)}$$

Checking the shear resistance based on aggregate interlock using Eqn. (2.14) is a local stress check at a crack. The correlation between crack spacing and stirrup spacing indicates that members with non-compliant stirrup spacing may suffer from a reduced ability to transfer shear stress along a crack, as the crack spacing would be larger and less controlled. The correlation between diagonal crack spacing and the longitudinal stirrup spacing is discussed in Section 2.4.2.i and examined further in Section 5.2.

### 2.4.2.iii Principal Tensile ($f_1$) and Compressive ($f_2$) Stress Response of Concrete

Prior to diagonal cracking, the shear in the web of a beam is carried by a set of diagonal compressive stresses in one direction accompanied by a set of diagonal tensile stresses oriented at 90° to the compressive stresses (Collins and Mitchell, 1991). After The Modified Compression Field Theory assumes that, after cracking, the principal tensile stresses in the concrete equal zero at the diagonal cracks and equal peak values of tensile stress between cracks. The tensile stresses between cracks cause an apparent stiffening of the encased steel reinforcement (Gupta and Maestrini, 1990; Fields, 1998; Fields and Bischoff, 2004). This phenomenon of tension stiffening is largely an issue of bond performance (Fields, 1998). The bond performance is related to the concrete's ability between cracks to transfer load from the steel through bond force. This transfer of load causes local decreases in rebar stress between cracks. Collins and Mitchell (1987) derived Eqn. (2.15) for calculating the relationship between average principal tensile stress and average principal tensile strain in cracked concrete. The coefficient 500 in Eqn. (2.15) was taken as 200 in the original formulation (Vecchio and Collins, 1986). The change to 500 was recommended by Collins and Mitchell (1987) based on experimental results from larger panel elements than those initially tested by Vecchio (Vecchio and Collins, 1982).

$$f_1 = \frac{f_{cr}}{1+\sqrt{500\cdot\varepsilon_1}} \qquad \text{(MPa)} \qquad \text{Eqn. (2.15)}$$

Testing has indicated that the principal compressive stress in cracked concrete ($f_2$) is a function of both the principal compressive strain $\varepsilon_2$ and the co-existing principal tensile strain $\varepsilon_1$ (Vecchio and Collins, 1986). Vecchio and Collins (1986) used the parabolic relationship Eqn. (2.16) to determine the principal compressive stress at a section. Eqn. (2.17) was derived by Vecchio and Collins for determining the maximum permissible average principal compressive stress $f_{c2,max}$. The term $\varepsilon_c$' is a negative quantity representing the compression strain in the concrete which corresponds to the peak compression stress, typically taken as -0.002.

$$f_2 = f_{c2,\max}\cdot\left[2\cdot\left(\frac{\varepsilon_2}{\varepsilon_c^{'}}\right)-\left(\frac{\varepsilon_2}{\varepsilon_c^{'}}\right)^2\right] \qquad \text{(MPa)} \qquad \text{Eqn. (2.16)}$$

where

$$\frac{f_{c2,\max}}{f_c^{'}} = \frac{1.0}{0.8-0.34\,\varepsilon_1/\varepsilon_c^{'}} \le 1.0 \qquad \text{Eqn. (2.17)}$$

### 2.4.2.iv Average Reinforcement Stresses

The Modified Compression Field Theory assumes that the average behavior of steel can be approximated by the bare-bar response (Bentz, 2000). Numerical analysis by Porasz (1989) demonstrated that the error in assuming bare-bar behavior for the average stress-strain behavior is small. The average stresses in the reinforcement can be calculated using Eqn. (2.18) and Eqn. (2.19)

$$\rho_x \cdot f_{sx} = f_{xx} + \nu \cdot \cot\theta - f_1 \qquad \text{(MPa)} \qquad \text{Eqn. (2.18)}$$

$$\rho_y \cdot f_{sy} = f_{yy} + \nu \cdot \tan\theta - f_1 \qquad \text{(MPa)} \qquad \text{Eqn. (2.19)}$$

### 2.4.2.v Local Reinforcement Stresses at a Crack ($f_{sxcr}$, $f_{sycr}$)

At the cracked surface the tensile stress in the concrete becomes zero, causing the stresses in the reinforcement to increase substantially (Vecchio and Collins, 1986). The local reinforcement stresses can be calculated using Eqn. (2.20) and Eqn. (2.21). Eqns. (2.20) and (2.21) are determined using Mohr's Circle using the assumption that the diagonal cracks are parallel (Vecchio and Collins, 1986). The reinforcement stresses are limited to the yield strength of the steel (Vecchio and Collins, 1986).

$$\rho_x \cdot f_{sxcr} = f_{xx} + \nu \cdot \cot\theta + \nu_{ci} \cdot \cot\theta \qquad \text{(MPa)} \qquad \text{Eqn. (2.20)}$$

$$\rho_y \cdot f_{sycr} = f_{yy} + \nu \cdot \tan\theta - \nu_{ci} \cdot \tan\theta \qquad \text{(MPa)} \qquad \text{Eqn. (2.21)}$$

## 2.5 Parameters Affecting Shear Capacity

Collins (2001) presented the list of factors which affect the shear capacity of concrete members found in Table 2.2.

Table 2.2 – Parameters Influencing Shear Strength of Concrete Members (Collins, 2001)

| | | | |
|---|---|---|---|
| 1 | Beam Depth: $h, d, d_v$ | 11 | Proximity of Section to rigid support or to point load, clamping stress |
| 2 | Beam width: $b$, $b_v$ | 12 | Shear span-to-depth ratio, *a/d* Member length to depth ratio, *L/d* |
| 3 | Cross-sectional shape: rectangular, I, T | 13 | Type of test specimen: simple span or continuous |
| 4 | Amount of Transverse Reinforcement: $A_v f_v/(b_v s)$ | 14 | Stirrup spacing ratio s/d |
| 5 | Amount of Longitudinal Reinforcement $\rho = A_s/(b_v d)$ | 15 | Type of loading: uniform or point loads |
| 6 | Concrete Strength: $f_c^{'}$, $f_{cr}$ | 16 | Roughness of crack surface |
| 7 | Aggregate type and size | 17 | Anchorage of transverse reinforcement |
| 8 | Level of Prestress: *P/A* | 18 | Stress-strain characteristics of reinforcement |
| 9 | Magnitude of co-existing moment: $M_f/V_f$ | 19 | Anchorage of longitudinal reinforcement |
| 10 | Magnitude of co-existing axial load: $N_f/V_f$ | 20 | Variation of section properties along member length: $s$, $A_s$, $d$, etc. |

Aside from stirrup spacing and area details, this study focuses specifically on parametric sensitivity of shear capacity predictions for concrete members with respect to concrete strength $f_c^{'}$, shear span to depth ratio *a/d*, longitudinal reinforcement ratio $\rho$ and member shape (rectangular vs. flanged members). These parameters have been looked at in other published literature. This literature allows for a comparison of how these parameters influence the agreement between predicted and tested shear capacities calculated using the four sectional shear methods assessed in this study to be made. Understanding how these parameters can affect the shear resistance of reinforced concrete members is important for assessing whether a shear evaluation method appropriately accounts for variations in these parameters. If not appropriately considered, variations in these parameters will negatively affect the agreement between predicted and tested shear capacities.

### 2.5.1 Concrete Strength

Concrete strength has long been acknowledged as a parameter significant to the shear resistance of concrete members. Reinforced concrete carries shear through three primary mechanisms: shear carried by the compression zone of the member, dowel action of the longitudinal reinforcement and aggregate interlock (ACI-ASCE 326, 1962). In normal strength concrete, aggregate interlock has commonly been assumed to carry the greatest portion of shear stress, by some estimates up to 70% (Fenwick and Paulay, 1968). Figure 2.7 provides the contribution to shear capacity from aggregate interlock, dowel action of the longitudinal reinforcement and the concrete compression zone as determined by Fenwick and Paulay (1968) and cited in Collins and Mitchell (1991).

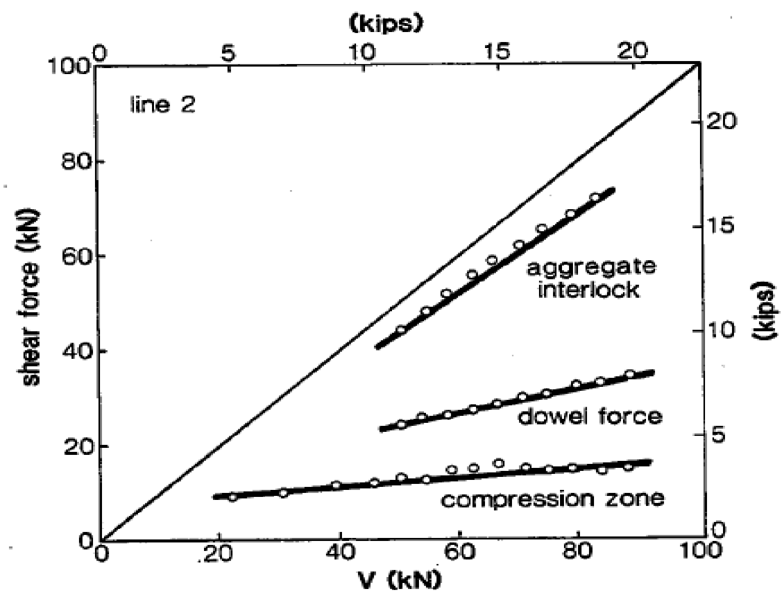



Figure 2.7 – Concrete Contributions to Shear Resistance (Collins and Mitchell, 1991)

The shear force carried in cracked concrete by aggregate interlock is dependent on the cracked surface's roughness and on the diagonal crack width, as well as on the normal force at the crack. Fenwick and Pauley (1968) studied this shear transfer mechanism by assessing the effect of both concrete strength and crack width. The concrete strengths ranged from 20 MPa to 60 MPa. Two conclusions were drawn from this research. The first conclusion was that the shear stress that could be carried by aggregate interlock decreased substantially as the crack width increased. The second conclusion was that as the concrete strength increased to 60 MPa, the stress carried by aggregate interlock increased. Walraven et al. (1987) carried out experiments of concrete push-off type specimen of various concrete strengths, which ranged from 17 MPa to 115 MPa. The authors concluded that in concrete with strengths greater than about 60 MPa cracks have a tendency to cleave through aggregates as opposed to around them, which reduces the surface roughness and thus the shear stress carried by aggregate interlock. Other research has been consistent with conclusions in Walraven et al. (Elzanaty et al., 1986; Johnson and Ramirez, 1989; Angelakos et al., 2001). Vecchio and Collins (1986) stated that the shear stress carried by cracked concrete through the aggregate interlock mechanism can be estimated using Eqn. (2.14), where the $a_g$ term is the maximum aggregate size used in the concrete.

Research investigating the influence of concrete strength on the shear capacity of members with transverse reinforcement has typically focused on the stirrup ratio required to produce a ductile failure as the concrete strength varies. Research has shown that members with higher concrete strength require larger stirrup ratios in order to ensure the post-diagonal cracking ductility necessary to achieve redistribution of internal forces (Johnson and Ramirez, 1989). The shear provisions in AASHTO LRFD-94 (AASHTO, 1994) and S6-00 (CSA, 2000) were the first to contain minimum stirrup area requirements which were a function of $\sqrt{f_c'}$. This has been carried on in subsequent shear design provisions. The minimum stirrup requirements in these two design standards were based on experience and practicality as opposed to testing (Bentz, 2005). Research by Yoon et al. (1996) has found that the design requirements for minimum stirrup area in S6-06, AASHTO-LRFD-05 and ACI 318-08 are appropriate to ensure adequate post-cracking ductility and shear capacity in concrete members as the specified concrete strength increases.

### 2.5.2 Shear Span to Depth Ratio, a/d

As early as 1909, Talbot (as cited in Hognestad, 1953) demonstrated that the shear capacity of reinforced concrete members increased as the loaded span length decreased. Since that time numerous other researchers have demonstrated that the shear span to depth ratio *a/d* is an important parameter which needs to be accounted for in sectional shear design/evaluation provisions (ASCE-ACI 426, 1973; Collins et al, 1996; Bentz and Collins, 2006).

The ratio *a/d* is known to dictate the mode of failure experienced by a member (Kani, 1967). Research by Kani (1967) demonstrated that members with *a/d* ratios between about 2.5 and 6.0 fail by diagonal tension. Kani termed this range the 'Valley of Diagonal Tension', which corresponded to members where one-way analysis of shear capacity was appropriate. Members loaded with *a/d* ratios less than 2.5 were found to carry a portion of shear through arching action, while members with *a/d* ratios greater than about 6.0 had a tendency to reach their flexural capacity prior to failing in shear. The shear-critical range, which corresponds to *a/d* ratios ranging from 2.5 to 6.0, is illustrated in Figure 2.8, which was developed based on numerous tests by Kani (1967) of members with varying *a/d* ratios and constant flexural capacity. Other research has confirmed that members with *a/d* ratios greater than 2.5 behaved differently than members with shorter shear spans, which indicated they should be treated separately (Leonhardt and Walther, 1964; Zsutty, 1971; Park and Pauley, 1975). Members of interest to the current study have *a/d* ratios between 2.5 and 6.0, the range which typically fails by diagonal tension as opposed to flexural failures or arching action.

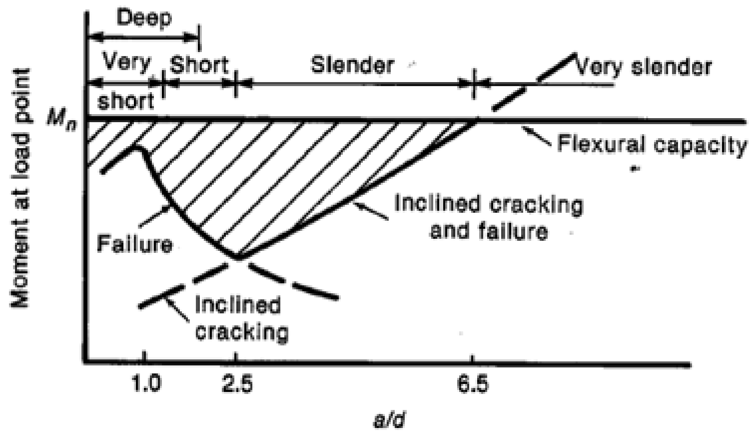



Figure 2.8 – Kani's Valley of Diagonal Tension (McGregor and Bartlett, 2000)

### 2.5.3 Longitudinal Reinforcement Ratio, ρ

Research in 1909 by Talbot (as cited in Hognestad, 1953) indicated that the shear capacity of reinforced concrete members increased as the percentage of longitudinal reinforcement increased. Subsequent research has also shown a strong correlation between increased shear strength and members with higher percentages of flexural reinforcement (Kani, 1967; Rajagopalan and Ferguson, 1969; Kong and Rangan, 1998; Angelakos, 1999; Tompos and Frosch, 2002). Figure 2.9 illustrates the increased shear stress carried by members without stirrups as the amount of longitudinal reinforcement increases.

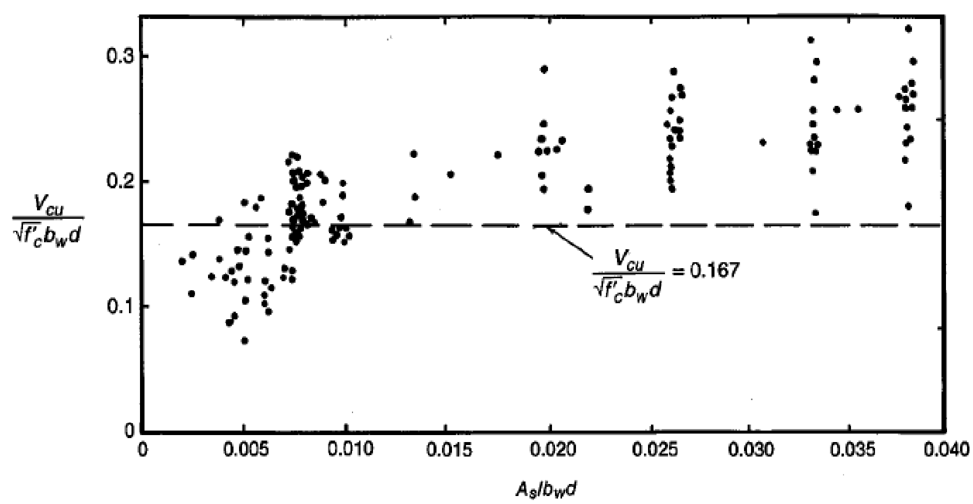



Figure 2.9 – Effect of Longitudinal Reinforcement on Shear Resistance (McGregor and Bartlett, 2000)

Greater flexural reinforcement ratios decrease the penetration of flexural cracking which in turn decreases the principal tensile stresses for a given load, enabling a section to carry greater shear stresses (Elzanaty el al., 1986).

Angelakos (1999) tested twelve large rectangular beams ($h$ = 1000 mm) and concluded that by increasing the longitudinal reinforcement ratio the shear capacity of beams without web reinforcement would increase. Kong and Rangan (1998) reported tests of members fabricated with web reinforcement containing the longitudinal reinforcement ratios ranging from 1.66% to 3.69%. A non-linear increase in shear capacity was found as the longitudinal reinforcing ratio increased.

Lubell et al. (2009) found that sectional shear evaluation methods which consider the strain in the longitudinal reinforcement present with predictions of shear capacity which better correlate to test results than do methods which account only for the percent of longitudinal reinforcement.

The longitudinal reinforcement ratio is calculated in the present study using Eqn. (2.22). The term $b$ is either the flange width for members with a compression flange or the section width for rectangular members.

$$\rho = \frac{A_s}{b \cdot d} \qquad \text{Eqn. (2.22)}$$

### 2.5.4 Member Shape

Although not widely recognized, member shape (rectangular vs. members with compression flanges) has been identified by some researchers as an important parameter affecting shear resistance of concrete members. Placas and Regan (1971) presented the results of twenty-four simply supported T-beams tested to fail in shear. One test series maintained a constant web width (150 mm) and varied the flange width from 150 mm to 1070 mm. All members had the same area of steel reinforcement in the longitudinal and transverse directions. As illustrated in Figure 2.10, flanged members had approximately 20% more shear capacity than did rectangular members with similar web widths. However, for flanged sections in the Placas and Regan study the width of the compression flange $b$ appeared to have a negligible effect on shear capacity. It should be noted from Figure 2.10 that flange thicknesses were not uniform. Placas and Regan (1971) concluded that only the portion of the flange adjacent to the web would carry a component of the shear in compression. Based on this concept, Zsutty (as cited in ASCE-ACI 426, 1973) proposed that the area of concrete carrying shear could be calculated using Eqn. (2.23):

$$A_{cv} = b_v \cdot d + 2 \cdot t_f^{\ 2} \qquad (\text{mm}^2) \qquad \text{Eqn. (2.23)}$$

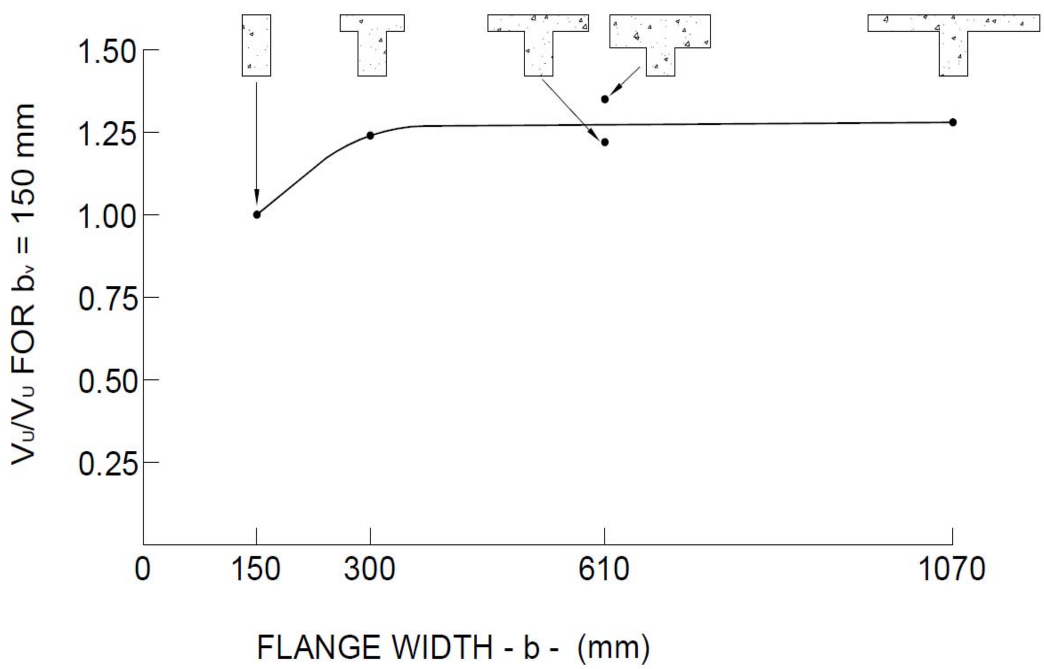



Figure 2.10 – Shear Stress Carried by Different Sections (adapted from ASCE-ACI 426, 1973)

Giaccio et al. (2002) tested fifteen T-beams to evaluate the effect of changes in flange geometry on sectional shear capacity. Details such as concrete strength, longitudinal reinforcement, web reinforcement, and member height were kept relatively constant. The details that were changed were flange depth and width. An increase in shear capacity was found as both the ratio of the flange depth to flexural depth $t_f/d$ and the flange width to effective web width $b/b_v$ increased. The correlation between the tested shear capacity and the two geometry ratios was unclear.

Tureyen et al. (2006) proposed a 'shear funnel' geometry, as shown in Figure 2.11, for calculating the shear capacity of flanged members. The angle $\theta_T$ for the extension of the concrete area contribution to shear capacity in the flange was assumed by the authors to be 45°. The depth of the concrete shear funnel was taken as the distance from the extreme compression fiber to the neutral axis (NA), which was calculated using the cracked elastic section depth *kd*. Although the proposed shear funnel geometry allowed for more accurate predictions of shear capacity, the authors recommended that for design the contribution of the flanges should be ignored. This was consistent with recommendations from ASCE-ACI 426 (1973).

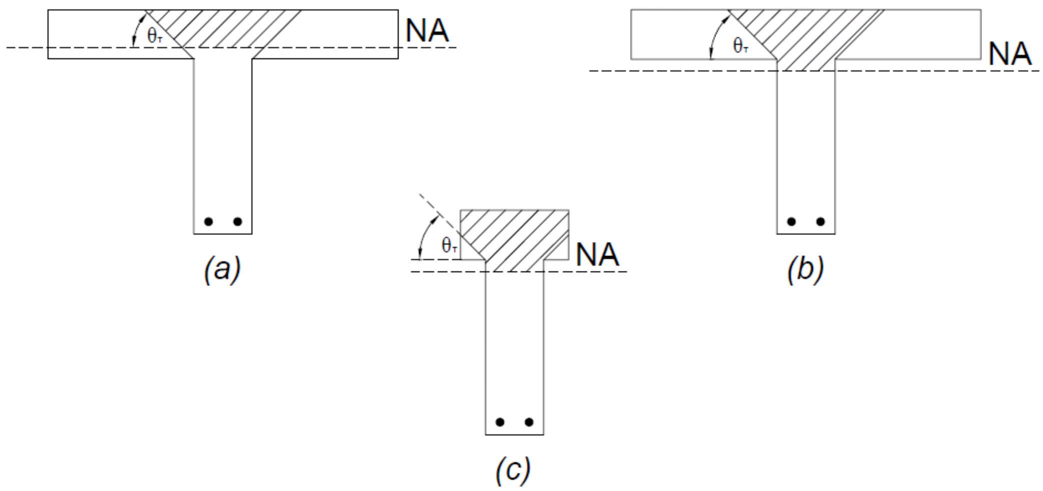



Figure 2.11 – 'Shear Funnel' Proposed by Tureyen et al. (2006)

## 2.6 Demerit Point Model

Collins (2001) presented a Demerit Point model as a quantitative tool for comparing shear evaluation methods. This model provided a tangible indication of agreement between tested and predicted shear capacities, with members having $V_{test}/V_{calc}$ ratios closer to unity receiving fewer demerit points. This Demerit Point model was used by Collins (2001) for the evaluation of a 413 member data set and by Kim (2004) for the evaluation of a 1353 member dataset. Collins' Demerit Point model is detailed as follows.

- The tested-to-predicted shear capacity ratio ($V_{test}/V_{calc}$) for each member of a dataset is determined using a given shear evaluation method.

- Based on its $V_{test}/V_{calc}$ ratio, each member is allotted a number of demerit points as given in Table 2.3. Since predicting an unsafe condition ($V_{test}/V_{calc}$ < 1.00) is more concerning than predicting a conservative condition ($V_{test}/V_{calc}$ > 1.00), members with $V_{test}/V_{calc}$ ratios less than 1.00 accrued demerit points at a faster rate than do members with $V_{test}/V_{calc}$ ratios greater than 1.00. Collins (2001) did not present a specific rationale as to how the ranges or demerit point allocations were determined.
- The demerit points for all members of a dataset are summed up for each shear evaluation method.
- The summation of demerit points is then used to indicate the performance of the shear evaluation method. A smaller summation indicates that a method is more appropriate for predicting one-way shear capacity of concrete members.

Table 2.3 – Collins (2001) Prediction Classifications and Demerit Points per Classification

| Classification | Extremely Dangerous | Dangerous | Low Safety | Appropriate Safety | Conservative | Extremely Conservative |
|---|---|---|---|---|---|---|
| $\frac{V_{test}}{V_{calc}}$ | < 0.50 | 0.50 – 0.65 | 0.65 – 0.85 | 0.85 – 1.30 | 1.30 – 2.00 | > 2.00 |
| Demerit Points | 10 | 5 | 2 | 0 | 1 | 2 |

The use of a reliability analysis was considered in this study for comparing the suitability of the four sectional shear methods. In order to properly perform a reliability analysis, it is important to have bias ratios and coefficients of variation for the geometric and reinforcing details required for calculating member capacity (MacGregor, 1976). These were not able to be produced for the specimens evaluated in this study from the information available in the documentation provided for each test member. Although a reliability analysis could have been conducted based solely on the average $V_{calc}/V_{test}$ ratios and corresponding COV, a review of other studies focusing on sectional shear capacity of concrete members indicates that such analysis is not typical. As Demerit Point models have been used in other studies (Collins, 2001; Kim, 2004), and because such models rely only on $V_{calc}/V_{test}$ ratios, it was decided to develop a Demerit Point model for this study.

A Demerit Point model, proposed for the current study and presented in Section 4.2, is based on Collins' (2001) Demerit Point model.

## 2.7 Elements Critical to a Sectional Shear Evaluation Method

This study identifies the following criteria as being critical for a shear method to address in order to be able to declare that the method is suitable for predicting sectional shear capacity:

- Ability to calculate shear capacity quickly and efficiently at various vertical cross sections along the length of a member;
- Predicted-to-test shear capacity ratios appropriately close to 1.00 and with a low coefficient of variation (COV);
- Low integer of assigned average demerit points per member. As discussed in Section 2.6, a revised Demerit Point model proposed for this study is presented in Section 4.2.
- No influence on $V_{calc}/V_{test}$ ratios due to variations of the stirrup detail ratios $s/s_{m1}$ and $A_{v,min}/A_v$; and
- No influence on $V_{calc}/V_{test}$ ratios due to variations in the four parameters discussed in Section 2.5.

Evaluating the shear capacity of a concrete member requires that the member be efficiently checked at various sections along its length. The most effective method of accomplishing these analyses along the length of a member is to have a shear evaluation method that can be incorporated into spreadsheets or other software.

Predicted-to-test shear capacity ratios ($V_{calc}/V_{test}$) consistently close to 1.00 are indicative of a method which is able to correctly predict the shear capacity of members. The coefficient of variation (COV) refers to the scatter in the predicted-to-test ratios relative to the average predicted-to-test ratio demonstrated by the method studied. Methods with lower scatter are considered more reliable, as lower COV indicates that the method better accounts for factors affecting shear capacity.

As discussed earlier, the two main details of interest in this study are the influence of non-compliant stirrup spacing and stirrup area on shear capacity predictions of concrete members. For a method to provide appropriate agreement between predicted and tested shear capacities for concrete members with these two non-compliant details, there should be no change in $V_{calc}/V_{test}$ ratios with respect to the average $V_{calc}/V_{test}$ ratio as the stirrup terms $s/s_{m1}$ and $A_{v,min}/A_v$ vary. These stirrup detail ratios are based on S6-06 Section 14 sectional shear provisions and are chosen for simplicity: $s/s_{m1}$ and $A_{v,min}/A_v$ ratios greater than 1.00 indicate member non-compliancy, while ratios less than 1.00 indicate that the member complied with the minimum stirrup detail requirements. To indicate that a method appropriately predicts the shear capacity of members with these non-compliant details, average demerit points per member, average $V_{calc}/V_{test}$ ratios and corresponding COV of the non-compliant and compliant datasets evaluated in this study must be in agreement with one another. A limited dataset is available for this study, as test results of members with non-compliant stirrup spacing and area details are scarce in test literature.

In addition to assessing the effect of the stirrup details $s/s_{m1}$ and $A_{v,min}/A_v$ on predictions of shear capacity this study also examines the influence of variations in concrete strength, longitudinal reinforcement ratio, shear span-to-depth ratio, and member shape (rectangular vs. members with compression flanges) on the ratio of the predicted-to-tested shear capacity. The methods examined in this study are considered to adequately account for a given parameter if the figure showing the relationship $V_{calc}/V_{test}$ vs. that parameter demonstrates no change in $V_{calc}/V_{test}$ ratios away from the mean $V_{calc}/V_{test}$ ratio as the parameter under consideration changes.

# Chapter 3
# Sectional Shear Evaluation Methods

## 3.1 Introduction

Four methods for evaluating the sectional shear capacity of concrete members are used in this study. Of these four shear evaluation methods, three were developed based on the Modified Compression Field Theory (Vecchio and Collins, 1986), discussed in Section 2.4.2. The other shear evaluation method was empirically derived by ACI Committee 318 from a collected database of test results primarily from the 1950's and 1960's (ASCE-ACI 426, 1973), as discussed in Section 2.4.1. The shear evaluation methods used are in this study are:

- the sectional design method for shear (Bentz and Collins, 2006) presented in S6-06 Section 8 (CSA, 2006) in combination with the stirrup spacing and area requirements in S6-06 Section 14;
- the General Method for Shear (Collins et al., 1996) in AASHTO LRFD-05 Section 5 (AASHTO, 2005);
- software Response 2000 (Bentz, 2000) and;
- the shear method for beams in ACI 318-08 Section 11 (ACI, 2008).

This chapter discusses these methods and how they are used in this study.

## 3.2 Canadian Highway Bridge Design Code S6-06 – Sectional Shear Method in Section 8 and Stirrup Requirements in Section 14

S6-06 adopted a sectional shear evaluation method (Bentz and Collins, 2006) based on simplifications to the Modified Compression Field Theory. The shear method used in S6-06 is an update to the General Method for shear (Collins et al., 1996) found in A23.3-94 (CSA, 1994) and S6-00 (CSA, 2000). S6-06 Section 8 contains the provisions for the design of concrete members, including the sectional shear provisions for both design of new structures and evaluation of existing structures. The maximum stirrup spacing and minimum stirrup area requirements contained in S6-06 Section 8 are for the design of new structures only. S6-06 Section 14 contains the provisions for evaluation of existing structures regardless of material, and includes requirements for maximum stirrup spacing and minimum stirrup area. Provisions specific for evaluation of existing structures are included in S6-06 Section 14 to 'avoid some of the conservatism that, in the interests of simplicity, may have been incorporated into the design provisions' (S6.1-06, 2006). S6-06 Section 14 provisions for maximum stirrup spacing and minimum stirrup area are provided in Section 2.2 and are used in this study for evaluation using the S6-06 sectional shear method.

The shear capacity at a section is evaluated by S6-06 using the general expression given in Eqn. (3.1). In accordance with S6-06 Clause 8.9.3.3 this study limits Eqn. (3.1) in order to assess whether web crushing is the expected mode of failure. As discussed in Section 4.3 members exceeding the web crushing limit $0.25 \cdot \phi_c \cdot f_c^{'} \cdot b_v \cdot d_v + \phi_p \cdot V_p$ are not included for evaluation in this study.

$$V_r = V_c + V_s + V_p \leq 0.25 \cdot \phi_c \cdot f_c^{'} \cdot b_v \cdot d_v + \phi_p \cdot V_p \qquad \text{(N)} \qquad \text{Eqn. (3.1)}$$

The terms $V_c$ and $V_s$ represent the shear resistance attributed to the concrete and to the stirrups respectively. The term $V_p$ represents the vertical force component of the prestressing steel which reduces the required shear resistance contributions from the concrete and the stirrups at a section. The actual material properties are used for evaluation in this study; therefore the resistance factors $\phi_c$, $\phi_s$, and $\phi_p$ are taken as 1.00. $V_c$ and $V_s$ are calculated using Eqn. (3.2) and Eqn. (3.3) respectively.

$$V_c = 2.5 \cdot \phi_c \cdot \beta \cdot f_{cr} \cdot b_v \cdot d_v \qquad \text{(N)} \qquad \text{Eqn. (3.2)}$$

$$V_s = \frac{\phi_s \cdot A_s \cdot f_v \cdot d_v}{\tan\theta \cdot s} \qquad \text{(N)} \qquad \text{Eqn. (3.3)}$$

The cracking resistance of the concrete $f_{cr}$ found in Eqn. (3.2) is limited in this study to 3.2 MPa in accordance with S6-06 Clause 8.9.3.4 and is calculated in this study in accordance with S6-06 Clause 8.4.1.4.1 as follows:

i. $0.40 \cdot \sqrt{f_c^{'}}$ for normal density concrete

ii. $0.34 \cdot \sqrt{f_c^{'}}$ for semi-low density concrete

iii. $0.30 \cdot \sqrt{f_c^{'}}$ for low density concrete

*Shear Terms β and θ – Overview of Derivation*

The $\beta$ term used in Eqn. (3.2) is a coefficient related to the ability of concrete to transfer shear across a cracked plane by means of aggregate interlock. Eqn. (3.4) is obtained by applying the following simplifications to Eqn. (2.14) (Bentz and Collins, 2006):

- maximum aggregate size $a_g$ = 20 mm
- crack widths $w$ calculated as $w = 0.2 + 1000 \cdot \varepsilon_x$:

$$v_{ci} = \frac{0.4 \cdot \sqrt{f_c^{'}}}{1 + 1500 \cdot \varepsilon_x} \quad \text{(MPa)} \qquad \text{Eqn. (3.4)}$$

Factoring the $\sqrt{f_c^{'}}$ term out of Eqn. (3.4) and applying a size correction factor to account for effective crack spacing, Bentz and Collins (2006) recommend calculating $\beta$ using Eqn. (3.5). The $\sqrt{f_c^{'}}$ term that is factored out of Eqn. (3.4) is accounted for in the concrete cracking stress term $f_{cr}$ found in Eqn. (3.2).

$$\beta = \frac{0.4}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} \qquad \text{Eqn. (3.5)}$$

The expression used to calculate the predicted angle of the compression field $\theta$ was fit based on two limits (Bentz and Collins, 2006), as shown in Figure 3.1. For a plastic truss mechanism to have sufficient ductility to allow redistribution of shear stresses to different angles, the concrete must be able to resist the applied shear stresses without crushing and the stirrups must be able to yield prior to shear failure of the member. Based on Figure 3.1, Eqn (3.6) provides the fit proposed by Bentz and Collins (2006) for predicting $\theta$. In order to improve the agreement between values of $\theta$

calculated using Eqn. (3.6) and values of $\theta$ obtained from the Modified Compression Field Theory (Vecchio and Collins, 1986) for large members without stirrups, the term $\left(0.88+\frac{s_{ze}}{2500}\right)$ is multiplied by Eqn. (3.6), resulting in Eqn. (3.7) (Bentz, and Collins, 2006). In accordance with S6-06 Clause 8.9.3.7, Eqn. (3.7) is used in this study for predicting the angle of the compression field. It should be noted that the expression $\left(0.88+\frac{s_{ze}}{2500}\right)$ has no effect on sectional shear capacity predictions of members without stirrups, as their predicted shear capacity is not dependent on the $\theta$ term. This expression does affect longitudinal reinforcement anchorage capacity calculations of these members. Anchorage capacity of the longitudinal reinforcement is checked in this study in accordance with S6-06 Clause 8.9.3.14.

$$\theta = 29^{\circ} + 7000 \cdot \varepsilon_x \qquad \text{Eqn. (3.6)}$$

$$\theta = \left(29 + 7000 \cdot \varepsilon_x\right) \cdot \left(0.88 + \frac{s_{ze}}{2500}\right) \qquad \text{Eqn. (3.7)}$$

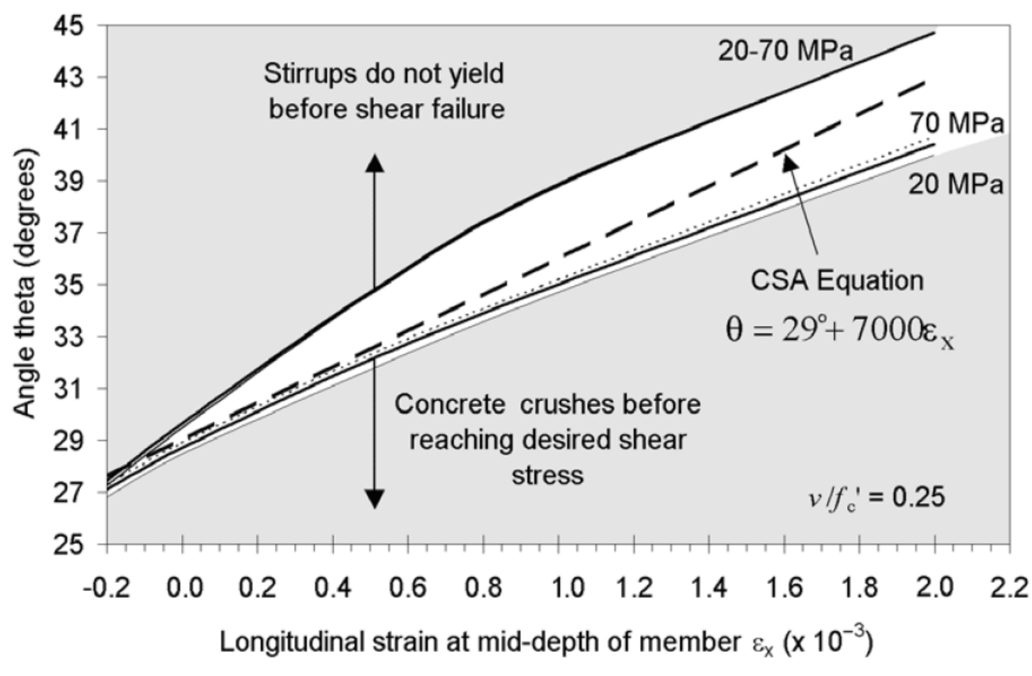



Figure 3.1 – Assumption of Linear fit for Angle of Principal Compression Field (Bentz and Collins, 2006)

*Calculating Parameters Required to Obtain Shear Terms β and θ*

Calculation of the $\beta$ and $\theta$ values relies on three main parameters:

i) longitudinal strain $\varepsilon_x$;

ii) normalized shear demand $\dfrac{V_f - V_p}{\phi_c \cdot f_c^{'} \cdot b_v \cdot d_v}$ Eqn. (3.8)

This term does not show up explicitly in Eqn. (3.5) or Eqn. (3.7). However, as shown in Figure 2.1 of the present study, Eqn. (3.8) is integral in determining the maximum allowable stirrup spacing, and thus the ratios $s/s_{m1}$, which in turn impacts the $s_z$ and $s_{ze}$ terms.

iii) effective crack spacing $s_{ze}$

S6-06 calculates the longitudinal strain $\varepsilon_x$ at the mid height of a member, using Eqn. (3.9). The mid height is selected as the location for calculating longitudinal strain because it corresponds with the location at which diagonal crack spacing is found to be largest (Bentz, 2006). According to Bentz it is appropriate to consider diagonal crack spacing and coexisting longitudinal strains at the same location. Research (Bentz and Collins, 2006) has shown that calculations of diagonal crack widths at mid-height using the expression $w = 0.2 + 1000 \cdot \varepsilon_x$ are in good agreement with predictions of mid height crack widths calculated using the Modified Compression Field Theory (Vecchio and Collins, 1986). Eqn. (3.9) accounts for effects of prestressing, quantity of longitudinal reinforcement, and applied axial and shear forces and bending moment at a section, as depicted in Figure 3.2. The factored moment $M_f$ and factored shear force $V_f$ are taken as positive values, while the factored axial load $N_f$ is taken as positive for tension forces. Although the longitudinal force in the bottom chord due to the diagonal tension should be rigorously taken as $0.5 \cdot \left(V_f - V_p\right) \cdot \cot\theta$ as shown in Figure 3.2, S6-06 shear provisions made the simplifying assumption that $0.5 \cdot \cot\theta$ would be equal to 1.00 (Bentz and Collins, 2006). The term $f_{po}$, found in Eqn. (3.9) and Eqn. (3.10), represents the stress in the prestressing tendons when the stress in the surrounding concrete is zero. This study calculates $f_{po}$ as $0.70 \cdot f_{pu}$ in accordance with S6-06 Clause 8.9.3.8 (d). This assumption for $f_{po}$ is appropriate for use in this study, as all prestressed members evaluated had effective tendon stresses after losses $f_{pe}/f_{pu}$ in the range of 0.45 to 0.65, which are typical values for prestressed members having a transfer stress of approximately 70% of $f_{pu}$.

$$\varepsilon_x = \frac{M_f/d_v + \left(V_f - V_p\right) + 0.5 \cdot N_f - A_p \cdot f_{po}}{2 \cdot \left(A_s \cdot E_s + A_p \cdot E_p\right)} \qquad \text{(mm/mm)} \qquad \text{Eqn. (3.9)}$$

If the numerator of Eqn. (3.9) is calculated to be negative, the sectional shear method in S6-06 permits the longitudinal strain to either be conservatively taken as zero or calculated using Eqn. (3.10) as is done in this study. S6-06 Clause 8.9.3.8 limits the value of $\varepsilon_x$ to a maximum value of 0.003 and the minimum value of -0.0002 when Eqn. (3.10) is used.

$$\varepsilon_x = \frac{M_f/d_v + \left(V_f - V_p\right) + 0.5 \cdot N_f - A_p \cdot f_{po}}{2 \cdot \left(A_s \cdot E_s + A_p \cdot E_p + A_{ct} \cdot E_c\right)} \qquad \text{(mm/mm)} \qquad \text{Eqn. (3.10)}$$

The longitudinal strain term $\varepsilon_x$ indirectly provides a good indication of demand and sufficiency of the quantity of longitudinal reinforcement. The concept is that larger strains result in larger crack widths, which in turn reduces shear capacity as less shear force can be transferred across the cracked surface by aggregate interlock (Bentz and Collins, 2006). Large compression or prestressing forces result in lower longitudinal strains, thereby decreasing the crack widths.

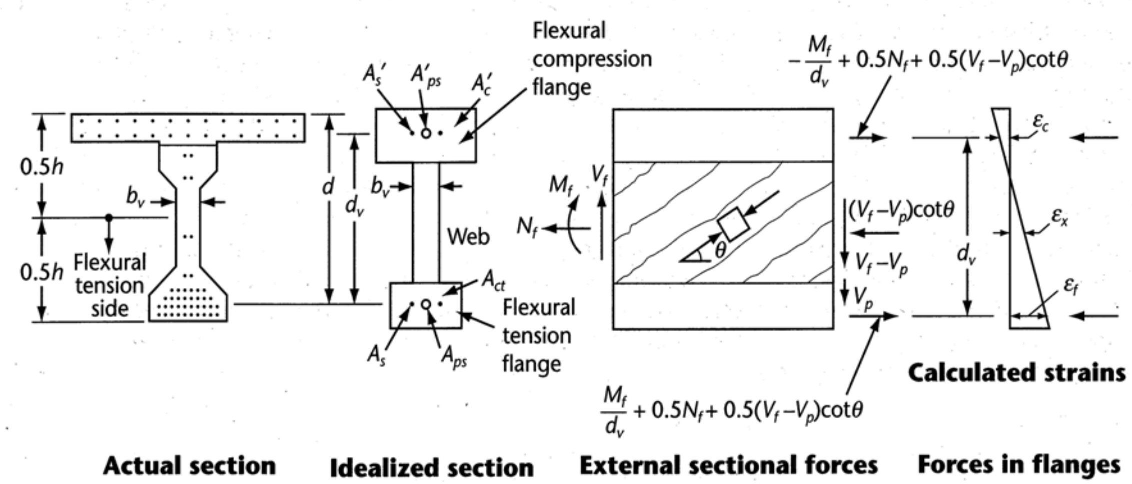



Figure 3.2 – S6-06 Idealized Section and Forces for Calculating $\varepsilon_x$ (S6-06, 2006)

Another major parameter for calculating the shear terms $\beta$ and $\theta$ is the spacing of diagonal cracks $s_z$ along the length of the member. As discussed in Section 2.4.2.ii, crack widths are used to predict the shear stress that the concrete is able to carry by aggregate interlock (Vecchio and Collins, 1986). S6-06 Section 8 assumes the crack spacing $s_z$ of members meeting stirrup spacing and area requirements to be 300 mm. This is applied consistently in this study for all compliant members, regardless of stirrup spacing or member depth. The diagonal crack spacing in members without stirrups has been shown to be approximately equal to the member depth (Base, 1982). Research indicates that the relationship between crack spacing and section depth for members without stirrups remains valid as member depth increases (Shioya et al., 1989; Bentz and Buckley, 2005). S6-06 Clause 8.9.3.7 assumes the diagonal crack spacing $s_z$ of members without stirrups to be equal to the shear depth $d_v$; this assumption is used in the present study.

S6-06 Clause 8.9.3.7 requires that, for members not complying with stirrup spacing and area provisions, the diagonal crack spacing $s_z$ be determined in the same manner as for members without stirrups. Thus non-compliant members evaluated using the shear provisions in S6-06 Section 8 are assumed to present with diagonal cracks spaced equal to the shear depth $d_v$. This assumption for diagonal crack spacing of members with non-compliant stirrup spacing and area details is used in this study when evaluating shear capacity using provisions in S6-06. This approach may be somewhat punitive, as the presence of stirrups spaced more tightly together than the shear depth of a member should provide better cracking control. As discussed in Section 2.4.2.i the diagonal crack spacing shows a correlation with the longitudinal spacing of stirrups. The assumption that crack spacing $s_z$ is equal to the longitudinal stirrup spacing is examined further in Section 5.2.

Figure 3.3, which is used to determine maximum permissible stirrup spacing, also illustrates how the assumed crack spacing $s_z$ is determined using S6-06 Section 14. It should be noted that both the assumed crack spacing and the maximum permissible longitudinal stirrup spacing are highly dependent on the normalized shear demand, calculated using Eqn. (3.8). Larger values of normalized shear demand require more closely spaced stirrups, which in turn can cause the actual stirrup spacing of existing members to exceed their maximum permissible stirrup spacing. This in turn affects the assumed crack spacing, as shown in Figure 3.3.

A special case for diagonal crack spacing exists when sufficient longitudinal reinforcement is distributed over the depth of a member. Collins et al. (1996) state that if an adequate quantity of longitudinal reinforcement is distributed over the depth of a member, the diagonal crack spacing can be taken as the maximum vertical distance between longitudinal bars. Collins et al. (1996) recommended a minimum bar area per layer of longitudinal reinforcement of $0.003 \cdot b_v \cdot s_v$, which is incorporated into S6-06 Clause 8.9.3.6. None of the members evaluated in this study were fabricated with longitudinal steel distributed vertically over the depth of the specimen.

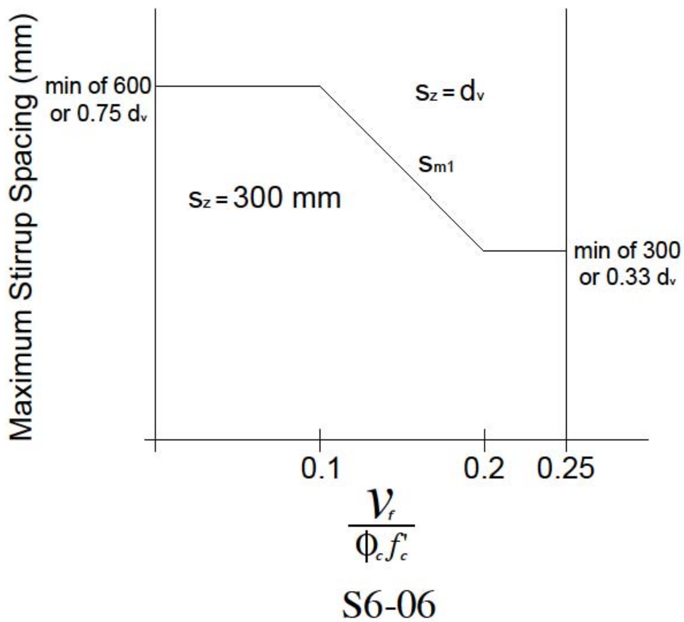



Figure 3.3 - $s_z$ used for S6-06 Section 14 Sectional Shear Analysis

The effective diagonal crack spacing $s_{ze}$ calculated by S6-06 Section 8 modifies the assumed diagonal crack spacing $s_z$ by accounting for variations in the maximum specified aggregate size $a_g$. The aggregate size term $a_g$ is dependent on the concrete compressive strength. S6-06 Clause 8.9.3.7 requires that for concrete strengths up to 60 MPa, the specified aggregate size shall be used for shear capacity evaluation and that for concrete strengths greater than 70 MPa the aggregate size shall be taken as 0 mm. S6-06 Clause 8.9.3.7 requires the aggregate size to be linearly interpolated from its specified value to a value of 0 mm as the concrete strength $f_c^{'}$ transitions from 60 MPa to 70 MPa. The effective crack spacing is calculated using Eqn. (3.11).

$$s_{ze} = \frac{35 \cdot s_z}{15 + a_g} \geq 0.85 \cdot s_z \qquad \text{(mm)} \qquad \text{Eqn. (3.11)}$$

*Application of S6-06 Shear Method*

Evaluating the sectional shear capacity of reinforced and prestressed concrete members using provisions in S6-06 requires the capacity to be checked at numerous locations along the length. These locations are necessitated by changes in geometry and reinforcement in the Bernoulli regions of the member along its length and by variations in the sectional moment to shear ratios of the member. In this study the critical section for shear is taken at $d_v$ away from the center of the externally applied load on the side nearest the support, as shown in Figure 3.4. This location is used as an estimate of the section at which the critical moment-shear interaction is produced.

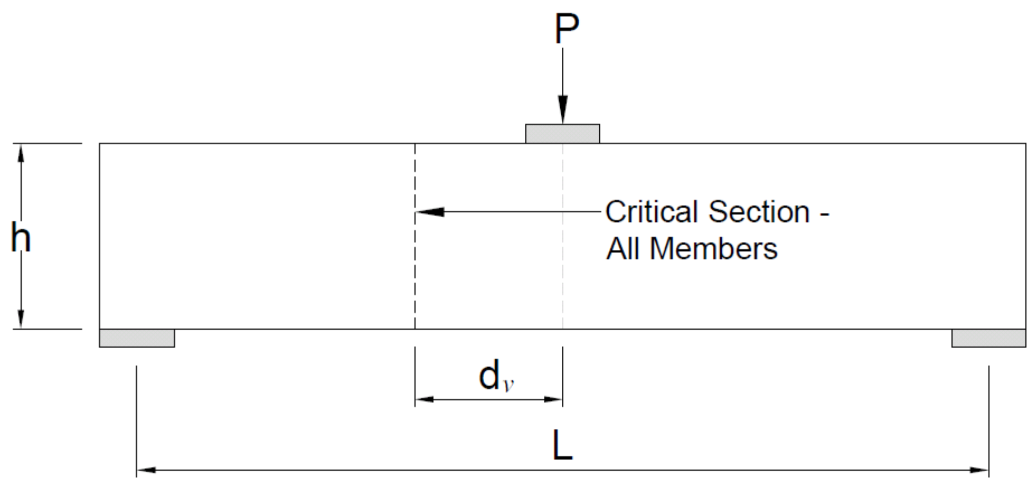



Figure 3.4 – Shear critical section for evaluation using S6-06

Other failure mechanisms such as flexural failures, crushing of the concrete web and anchorage capacity of the longitudinal reinforcement need to be checked in order to ensure they do not govern prior to one-way shear failure. These are addressed in this study in Section 4.3.

The stirrup area $A_v$ is taken in this study as the cross sectional area per stirrup. To determine longitudinal stirrup spacing, this study calculates the ratio $A_v/s$ for a distance $d_v$ from the applied load in the direction of the nearest support. The longitudinal stirrup spacing $s$ is then determined by dividing the cross sectional area per stirrup $A_v$ by the ratio $A_v/s$. In the event that a stirrup has not been intercepted within this distance, the distance between the applied load and the nearest stirrup is used as the longitudinal stirrup spacing $s$. Anchorage of the stirrups is checked to ensure that they meet detailing requirements in A23.1-09 Clause 6.6.2.2 and Clause 6.6.2.4. This study uses the quantity of longitudinal reinforcement reported in the test literature when evaluating shear capacity using provisions in S6-06 Section 8. Development length of non-prestressed members is checked according to S6-06 Clause 8.14.2. Prestressing strands are checked to ensure that they meet a transfer length of $50 \cdot d_s$ in accordance with traditional Canadian Precast/Prestressed Concrete Institute (CPCI, 2005) provisions.

Simplifications in the sectional shear method (Bentz and Collins, 2006) in S6-06 Section 8 have reduced the work required to converge on the predicted shear capacity, compared to the shear provisions in S6-00 and A23.3-94. However, iteration of the applied loads is still required for converging on the predicted shear capacity which is achieved when the applied shear equals the calculated shear resistance at a section. This study varies the externally applied load to achieve the iteration of forces required to predict the sectional shear capacity.

Successive iterations of applied shear vary the sectional shear demand, which in turn affects the maximum permitted stirrup spacing (see Figure 2.1). As discussed earlier in this Section, the diagonal crack spacing assumed by the S6-06 shear method is dependent on how the actual stirrup spacing compares to the permitted stirrup spacing. This is illustrated in Figure 3.3. The sudden change in assumed crack spacing $s_z$ at the stirrup spacing limit $s_{m1}$ can cause a discontinuity in the predicted shear capacity of a member. This can make determining the predicted shear capacity of the member impossible, because the predicted shear capacity at iteration $n$-1 may never equal the predicted shear capacity in iteration $n$. As such the applied shear force will never converge on the calculated shear resistance of a member. This source of ambiguity for predicting sectional shear capacity using the S6-06 evaluation method is more pronounced as the member depth increases. In this study the issue of non-convergent predicted shear capacities is addressed by iterating the externally applied load until the actual stirrup spacing at the critical section equals the maximum allowable stirrup spacing. This is the point at which the cusp of non-convergence in shear capacity is reached. Due to the ambiguous nature of predicting shear capacities of members presenting with this non-convergence issue, the lower predicted shear capacity at this iteration of externally applied load is selected as the calculated shear resistance. Using the lower predicted shear capacity at the point of non-convergence is more likely to assure a safe prediction of shear capacity than using the larger value of predicted shear capacity at non-convergence. An example of this approach is provided in Appendix C.

The flowchart included at the end of this Section provides the process used in this study for evaluating the sectional shear capacity of members using S6-06 Section 8 and Section 14. This flowchart as provided is suitable for simple spans subject to point loads – other cases require Step 2 in the flowchart to be modified. All members evaluated for shear capacity in this study were tested using 1 or 2 point loading. In order for the flow chart at the end of this Section to be applicable for members subjected to a uniformly distributed load, Step 2 should be revised as follows. Instead of using an assumed critical section at a distance $d_v$ away from the applied load, the shear capacity should be checked at numerous sections along the member length. The section which produces the highest $V_{calc}/V_{test}$ ratio should be selected as the governing section.

<u>*Other Studies using the Shear Method in S6-06*</u>

The following predictions of shear capacity taken or derived from other studies are restated based on the author(s)' predictions and have not been checked in this study.

Kim (2004) used the sectional shear method in A23.3-04 (CSA, 2004) to predict the shear capacity of 1363 concrete members tested to fail in one-way shear. The shear method in A23.3-04 is similar to the shear method in S6-06 Section 8, except that A23.3-04 uses different requirements for maximum permissible stirrup spacing (see discussion of CSA-M in Section 5.2) and Eqn (3.6) is used in A23.3-04 instead of Eqn (3.7) for calculating the predicted angle of the compression field. This typically impacts only the predicted shear capacities of members with stirrups not complying with stirrup spacing and area requirements for the following reasons:

- Shear capacity for members with stirrups not complying with stirrup spacing and area requirements includes a stirrup contribution $V_s$, which is dependent on the predicted angle of the compression field $\theta$. As discussed in this Section, the diagonal crack spacing of these members is assumed to be equal to the shear depth $d_v$. For members with large overall heights the expression $\left(0.88+\frac{s_{ze}}{2500}\right)$ in Eqn. (3.7) can have a considerable impact on the predicted angle of the compression field, which in turn impacts the predicted shear capacity attributed to the stirrups.

- As discussed in this Section, the term $\left(0.88+\frac{s_{ze}}{2500}\right)$ in Eqn. (3.7) does not affect one-way shear capacity predictions of members without stirrups.
- As discussed in this Section, for members complying with stirrup spacing and area requirements, diagonal crack spacing $s_z$ is assumed to be 300 mm. Assuming a specified aggregate size of 20 mm, the effective diagonal crack spacing $s_{ze}$, calculated using Eqn. (3.11), is also 300 mm. This makes the expression $\left(0.88+\frac{s_{ze}}{2500}\right)$ in Eqn. (3.7) equal to 1.00. Other aggregate sizes will vary the $s_{ze}$ term, which in turn will affect the shear capacity attributed to the stirrups.

The majority of members studied by Kim (2004) either contained no stirrups or were compliant with respect to maximum stirrup spacing and minimum stirrup area. A few members did contain stirrups which were non-compliant with respect to the previously mentioned stirrup details. Kim's (2004) dissertation did not provide individual values of $V_{test}/V_{calc}$ ratios for the data set, so the reciprocal values could not be duplicated to be consistent with the $V_{calc}/V_{test}$ ratios used in this study. Table 3.1 summarizes the mean $V_{test}/V_{calc}$ ratios and COV values derived from Kim (2004).

Table 3.1 – Summary of Predictions by Kim (2004) using A23.3-04

| Type | $V_{test}/V_{calc}$ | *COV (%)* |
|---|---|---|
| Non-Prestressed – Total Data Set (878) | 1.25 | 27.0 |
| Non-Prestressed – Members without Stirrups (718) | 1.27 | 28.0 |
| Non-Prestressed – Members with Stirrups (160) | 1.19 | 21.0 |
| Prestressed – Total Data Set (475) | 1.41 | 26.0 |
| Prestressed – Members without Stirrups (321) | 1.46 | 29.0 |
| Prestressed – Members with Stirrups (164) | 1.30 | 13.0 |

Lubell (2006) used the shear method in A23.3-04 to evaluate 106 members with transverse reinforcement. Table 3.2 provides the $V_{calc}/V_{test}$ ratios and COV values derived from Lubell (2006).

Table 3.2 – Summary of Predictions by Lubell (2006) using A23.3-04

| Type | $V_{calc}/V_{test}$ | *COV (%)* |
|---|---|---|
| Members with Stirrups (106) | 0.87 | 16.5 |

Flowchart - S6-06 Sectional Shear Method

Step 1: Determine sectional geometry and material properties for the member being evaluated.

Step 2: Calculate moments and shears from the externally applied load and member self weight at a section $d_v$ away from the centerline of the applied load. $d_v$ is calculated as the larger of 0.9$d$ and 0.72$h$. This applies to simple span members subject to point loading.

Step 3: Determine Stirrup Spacing and Area Requirements

S6-06 Clause 14.14.1.6.2 contains an in lieu clause to those found in Section 8 Clause 8.9.1.3.

To be considered compliant, stirrups are required to meet the minimum area requirements given in Eqn. (2.1).

$$A_v \geq 0.15 \cdot f_{cr} \cdot \frac{b_v s}{f_v} \text{ (mm}^2\text{)} \qquad \text{Eqn. (2.1)}$$

Members with an area of transverse steel less than that given in Eqn. (2.2) are required by S6-06 Section 14 to not use a stirrup contribution to shear capacity.

$$A_v < 0.05 \cdot f_{cr} \cdot \frac{b_v s}{f_v} \qquad \text{(mm}^2\text{)} \qquad \text{Eqn. (2.2)}$$

For $A_v$ values falling between Eqn. (2.1) and Eqn. (2.2), Eqn (3.12) is used in this study to accomplish the interpolation of stirrup area required by S6-06 Clause 14.14.1.6.2.

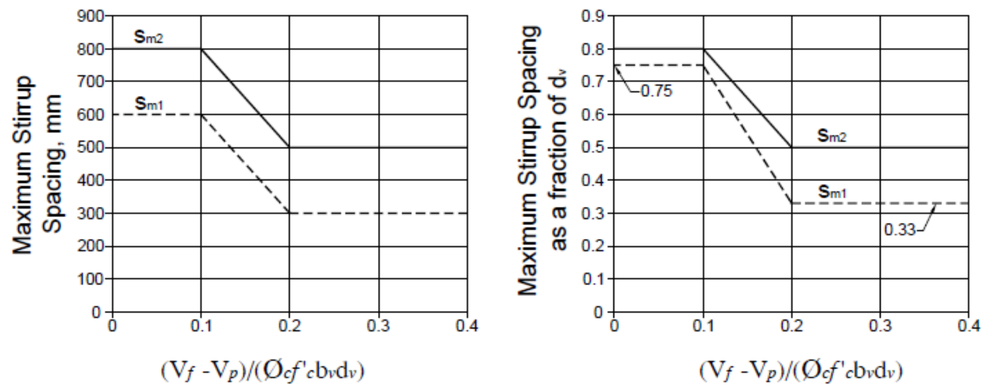
$$\gamma = 10 \cdot \frac{A_v \cdot f_v}{f_{cr} \cdot b_v \cdot s} - 0.5 \qquad 0 \leq \gamma \leq 1.00 \qquad \text{Eqn. (3.12)}$$

The stirrup spacing limits are a function of the shear demand and member depth at a section. The shear demand is calculated using Eqn. (3.8).

$$\frac{v_f}{\phi_c \cdot f_c^{'}} = \frac{V_f - V_p}{\phi_c \cdot f \cdot b_v \cdot d_v} \qquad \text{Eqn. (3.8)}$$

The maximum stirrup spacing $s_{m1}$ is then interpolated and taken as the minimum from the two graphs in Figure 2.1 (see following figure):

Step 3 cont:




As the shear capacity $V_r$ converges on the applied shear $V_f$, the normalized shear demand varies, which in turn changes the maximum permissible stirrup spacing. It needs to be rechecked for each change in applied shear force.

Step 4: Determine Effective Crack Spacing Term $s_{ze}$

The sectional shear method in S6-06 Section 8 assumes a diagonal crack spacing $s_z$ equal to 300 mm for members with stirrups complying with Section 14 spacing and area requirements, and $d_v$ for members with non-compliant stirrups or with no stirrups. For members with longitudinal reinforcement distributed over the depth of the member with minimum bar area of $0.003 \cdot b_v \cdot s_v$ (Collins et al., 1996) the diagonal crack spacing is assumed to be equal to the maximum vertical spacing of the longitudinal bars. S6-06 Clause 8.9.3.6 calculates the effective crack spacing $s_{ze}$ using Eqn. (3.11):

$$s_{ze} = \frac{35 \cdot s_z}{15 + a_g} \geq 0.85 \cdot s_z \qquad \text{(mm)} \qquad \text{Eqn. (3.11)}$$

S6-06 Clause 8.9.3.7 specifies that for concrete strengths up to 60 MPa the specified aggregate size shall be used for shear capacity evaluation, and for concrete strengths greater than 70 MPa the aggregate size shall be taken as 0 mm. S6-06 Clause 8.9.3.7 requires the aggregate size to be linearly interpolated from its specified value to 0 mm as the concrete strength $f_c$' transitions from 60 MPa to 70 MPa.

Step 5: Calculate the longitudinal strain $\varepsilon_x$

The longitudinal strain at a section is calculated by S6-06 using Eqn. (3.10):

$$\varepsilon_x = \frac{M_f / d_v + \left(V_f - V_p\right) + 0.5 \cdot N_f - A_p \cdot f_{po}}{2 \cdot \left(A_s \cdot E_s + A_p \cdot E_p + A_{ct} \cdot E_c\right)} \qquad \text{Eqn. (3.10)}$$

NOTE: The term $A_{ct}E_c$ is taken as 0 if $\varepsilon_x$ is positive. $\varepsilon_x$ is limited to a maximum value of 0.003 and a minimum value of -0.0002 as per S6-06 Clause 8.9.3.8. This study calculates $f_{po}$ as $0.70 \cdot f_{pu}$ in accordance with S6-06 Clause 8.9.3.8 (d).

Step 6: Calculate the Shear Terms $\beta$ and $\theta$

S6-06 Clause 8.9.3.7 calculates the concrete contribution term $\beta$ using Eqn.(3.5).

$$\beta = \frac{0.4}{1+1500\cdot\varepsilon_x}\cdot\frac{1300}{1000+s_{ze}} \qquad \text{Eqn. (3.5)}$$

S6-06 Clause 8.9.3.7 calculates the stirrup contribution term $\theta$ using Eqn. (3.7).

$$\theta = (29+7000\cdot\varepsilon_x)\cdot\left(0.88+\frac{s_{ze}}{2500}\right) \qquad \text{Eqn. (3.7)}$$

Step 7: Calculate the Shear Resistance $V_r$

S6-06 Clause 8.9.3.4 calculates the concrete resistance using Eqn. (3.2):

$$V_c = 2.5\cdot\phi_c\cdot\beta\cdot f_{cr}\cdot b_v\cdot d_v \qquad \text{(N)} \qquad \text{Eqn. (3.2)}$$

S6-06 Clause 8.9.3.5 calculates the stirrup resistance, modified with $\gamma$, using Eqn. (3.3):

$$V_s = \gamma\cdot\frac{\phi_s\cdot A_v\cdot f_v\cdot d_v}{s\cdot\tan\theta} \qquad \text{(N)} \qquad \text{Eqn. (3.3)}$$

The sectional shear resistance $V_r$ is then calculated using Eqn. (3.1)

$$V_r = V_c + V_s + V_p \leq 0.25\cdot\phi_c\cdot f_c^{'}\cdot b_v\cdot d_v + \phi_p\cdot V_p \qquad \text{( N)} \quad \text{Eqn. (3.1)}$$

$V_p$ is the vertical force component of the prestressing steel. The resistance factors $\phi_c$, $\phi_s$ and $\phi_p$ were set to 1.00 for evaluations in this study. The calculation $0.25\cdot\phi_c\cdot f_c^{'}\cdot b_v\cdot d_v$ is included in Eqn. (3.1) to check whether web crushing is expected to occur prior to flexural-shear failure.

Step 8: Converge Predicted Shear Capacity

Converging on the predicted shear capacity of concrete members according to the S6-06 shear method requires iterating shear predictions until the applied shear forces at iteration *n* equals the calculated shear capacity at iteration *n*. This study iterates the predicted shear capacity by varying the externally applied load at Step 2, which in turn varies the sectional moments and shears.

## 3.3 American Association of State Highway and Transportation Officials AASHTO LRFD-05

The General Method (Collins et al., 1996) for sectional shear in reinforced concrete, found in AASHTO LRFD-05 Section 5 (AASHTO, 2005), was derived based on simplifications to the Modified Compression Field Theory. The presentation of the AASHTO LRFD-05 shear method is similar to the sectional shear method in the Canadian Highway Bridge Code S6-06 Section 8 described in Section 3.2. However, the calculations of $(\varepsilon_x)_A$, $\beta$ and $\theta$ differ.

The general expression for shear in AASTHO LRFD-05 Section 5 is calculated as

$$V_n = V_c + V_s + V_p \leq 0.25 \cdot f_c^{'} \cdot b_v \cdot d_v + V_p \qquad \text{(N)} \qquad \text{Eqn. (3.13)}$$

The concrete contribution is calculated using Eqn. (3.14) while the stirrup contribution is calculated using Eqn. (3.15).

$$V_c = 0.083 \cdot \beta \cdot \sqrt{f_c^{'}} \cdot b_v \cdot d_v \qquad \text{(N)} \qquad \text{Eqn. (3.14)}$$

$$V_s = \frac{A_v \cdot f_v \cdot d_v}{s \cdot \tan\theta} \qquad \text{(N)} \qquad \text{Eqn. (3.15)}$$

AASHTO LRFD-05 Section 5 does not provide a specific limit for the value of $f_c^{'}$. However AASHTO LRFD-05 Clause 5.4.2.1 states that values of $f_c^{'}$ greater than 70 MPa are only permitted for use when testing is used to establish the relationships because the concrete compressive strength and the other properties of the concrete (eg. cracking strength $f_{cr}$). Thus, for evaluation using the sectional shear method in AASHTO LRFD-05, 70 MPa is considered as a practical limit for concrete compressive strength in this study.

Similar to the sectional shear method in S6-06 as discussed in Section 3.2, the shear terms $\beta$ and $\theta$ are functions of three parameters:

i) longitudinal strain $(\varepsilon_x)_A$

ii) normalized shear demand $\dfrac{v_u}{f_c^{'}} = \dfrac{V_f - V_p}{b_v \cdot d_v \cdot f_c^{'}}$

iii) crack spacing parameter $(s_{ze})_A$

AASHTO LRFD-05 uses Eqn. (3.16) for calculating the longitudinal strain in members with compliant stirrup details (with respect to spacing and area), while Eqn. (3.17) is used for members without stirrups or for members with stirrups not meeting minimum stirrup requirements. The sectional shear methods in AASTHO LRFD-05 and S6-06, which are both based on the relationships from the Modified Compression Field Theory (Vecchio and Collins, 1986), use the same idealized cross section and sectional force details; thus Eqn (3.16) to Eqn. (3.18) were derived based on Figure 3.1. As discussed in AASHTO LRFD-05 Clause 5.8.3.4.2, for members not meeting minimum stirrup requirements the longitudinal strain is calculated at the level of the longitudinal reinforcement. This accounts for the reduced ability of members not meeting stirrup spacing and area requirements to redistribute internal forces compared to compliant members. Thus the '2' in the denominator of Eqn. (3.17) was removed compared to Eqn. (3.16). In accordance with AASHTO LRFD-05 Clause 5.8.3.4.2, this study calculates $f_{po}$ as $0.70 \cdot f_{pu}$. This assumption for $f_{po}$ is discussed in Section 3.2.

$$(\varepsilon_x)_A = \frac{\dfrac{|M_f|}{d_v} + 0.5 \cdot N_f + 0.5 \cdot |V_p - V_p| \cdot \cot\theta - A_p \cdot f_{po}}{2 \cdot (E_s A_s + E_p A_p)} \qquad \text{Eqn. (3.16)}$$

$$(\varepsilon_x)_A = \frac{\dfrac{|M_f|}{d_v} + 0.5 \cdot N_f + 0.5 \cdot |V_p - V_p| \cdot \cot\theta - A_p \cdot f_{po}}{(E_s A_s + E_p A_p)} \qquad \text{Eqn. (3.17)}$$

If the numerator is calculated by either Eqn. (3.16) or Eqn. (3.17) to be negative, the longitudinal strain is calculated using Eqn. (3.18).

$$(\varepsilon_x)_A = \frac{\dfrac{\left|M_f\right|}{d_v} + 0.5 \cdot N_f + 0.5 \cdot \left|V_p - V_p\right| \cdot \cot\theta - A_p \cdot f_{po}}{2 \cdot \left(E_s A_s + E_p A_p + E_c A_{ct}\right)} \qquad \text{Eqn. (3.18)}$$

As can be seen in Eqn. (3.16) through Eqn. (3.18), the angle of the diagonal compression field is required to calculate the longitudinal strain. The interpolation required to obtain $\theta$ at each iteration of applied load increases the difficultly in determining the longitudinal strain, and is one of the primary complaints about this method (Hawkins et al., 2005).

AASHTO LRFD-05 determines the effective crack spacing term $(s_{ze})_A$ using Eqn. (3.19). In accordance with AASHTO LRFD-05 Clause C5.8.3.4.2, this study assumes a diagonal crack spacing $(s_z)_A$ of 300 mm for members complying with AASHTO LRFD-05 stirrup spacing and area requirements, and a diagonal crack spacing $(s_z)_A$ equal to the shear depth $d_v$ for members without stirrups or with non-compliant stirrup spacing and area details.

$$(s_{ze})_A = \frac{35 \cdot (s_z)_A}{16 + a_g} \leq 2000mm \qquad \text{(mm)} \qquad \text{Eqn. (3.19)}$$

Using the parameters $\upsilon_u/f_c'$ and $(\varepsilon_x)_A$, AASHTO LRFD-05 determines the shear terms $\beta$ and $\theta$ for members with stirrups complying with spacing and area requirements, using Table 3.3. For members without stirrups or with less than specified minimum stirrup requirements, the shear terms $\beta$ and $\theta$ are determined from Table 3.4 using the parameters $(\varepsilon_x)_A$ and $(s_{ze})_A$.

AASHTO LRFD-05 does not explicitly provide values of $\beta$ and $\theta$ for sections complying with minimum transverse reinforcement requirements and with longitudinal strains $(\varepsilon_x)_A$ greater than $1.00\text{x}10^{-3}$. AASHTO LRFD-05 Clause C5.8.3.4.2 states that for $(\varepsilon_x)_A$ values larger than provided in Table 3.3, smaller values of $\beta$ and larger values of $\theta$ should be used for predictions of sectional shear capacity. Because AASTHO LRFD-05 is not specific about values of $\beta$ and $\theta$ to be used when $(\varepsilon_x)_A$ exceeds $1.00\text{x}10^{-3}$, this study uses the $\beta$ and $\theta$ values given in AASHTO LRFD-00 Table 5.8.3.4.2-1. These values are provided in italics in Table 3.3 of the present study.

Table 3.3 - $\theta$ and $\beta$ Values for Members with Greater than Minimum Stirrups (AASHTO, 2005)

| $\frac{\upsilon_f}{f_c^{'}}$ | | $(\varepsilon_x)_A$ x 1000 | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ≤ -0.20 | ≤ -0.10 | ≤ -0.05 | ≤0 | ≤0.125 | ≤0.25 | ≤0.50 | ≤0.75 | ≤1.00 | *<1.50* | *<2.00* |
| ≤0.075 | θ | 22.3 | 20.4 | 21.0 | 21.8 | 24.3 | 26.6 | 30.5 | 33.7 | 36.4 | *40.8* | *43.9* |
| | β | 6.32 | 4.75 | 4.10 | 3.75 | 3.23 | 2.94 | 2.59 | 2.38 | 2.23 | *1.95* | *1.67* |
| ≤0.100 | θ | 18.1 | 20.4 | 21.4 | 22.5 | 24.9 | 27.1 | 30.8 | 34.0 | 36.7 | *40.8* | *43.1* |
| | β | 3.79 | 3.38 | 3.24 | 3.14 | 2.91 | 2.75 | 2.50 | 2.32 | 2.18 | *1.93* | *1.69* |
| ≤0.125 | θ | 19.9 | 21.9 | 22.8 | 23.7 | 25.9 | 27.9 | 31.4 | 34.4 | 37.0 | *41.0* | *43.2* |
| | β | 3.18 | 2.99 | 2.94 | 2.87 | 2.74 | 2.62 | 2.42 | 2.26 | 2.08 | *1.90* | *1.67* |
| ≤0.150 | θ | 21.6 | 23.3 | 24.2 | 25.0 | 26.9 | 28.8 | 32.1 | 34.9 | 37.3 | *40.5* | *42.8* |
| | β | 2.88 | 2.79 | 2.78 | 2.72 | 2.60 | 2.52 | 2.36 | 2.21 | 2.08 | *1.82* | *1.61* |
| ≤0.175 | θ | 23.2 | 24.7 | 25.5 | 26.2 | 28.0 | 29.7 | 32.7 | 35.2 | 36.8 | *39.7* | *42.2* |
| | β | 2.73 | 2.66 | 2.65 | 2.60 | 2.52 | 2.44 | 2.28 | 2.14 | 1.96 | *1.82* | *1.54* |
| ≤0.200 | θ | 24.7 | 26.1 | 26.7 | 27.4 | 29.0 | 30.6 | 32.8 | 34.5 | 36.1 | *39.2* | *41.7* |
| | β | 2.63 | 2.59 | 2.52 | 2.51 | 2.43 | 2.37 | 2.14 | 1.94 | 1.79 | *1.61* | *1.47* |
| ≤0.225 | θ | 26.1 | 27.3 | 27.9 | 28.5 | 30.0 | 30.8 | 32.3 | 34.0 | 35.7 | *38.8* | *41.4* |
| | β | 2.53 | 2.45 | 2.42 | 2.40 | 2.34 | 2.14 | 1.86 | 1.73 | 1.64 | *1.51* | *1.39* |
| ≤0.250 | θ | 27.5 | 28.6 | 29.1 | 29.7 | 30.6 | 31.3 | 32.8 | 34.3 | 35.8 | *38.6* | *41.2* |
| | β | 2.39 | 2.39 | 2.33 | 2.33 | 2.12 | 1.93 | 1.70 | 1.58 | 1.50 | *1.38* | *1.29* |

Table 3.4 - $\theta$ and $\beta$ Values for Members with Less than Minimum Stirrups (AASHTO, 2005)

| $(s_{ze})_A$ (mm) | | $(\varepsilon_x)_A$ x 1000 | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ≤-0.20 | ≤-0.10 | ≤-0.05 | ≤0 | ≤0.25 | ≤0.50 | ≤0.75 | ≤1.00 | ≤1.50 | ≤2.00 |
| ≤130 | θ | 25.4 | 25.5 | 25.9 | 26.4 | 28.9 | 30.9 | 32.4 | 33.7 | 35.6 | 37.2 |
| | β | 6.36 | 6.06 | 5.56 | 5.15 | 3.91 | 3.26 | 2.86 | 2.58 | 2.21 | 1.96 |
| ≤250 | θ | 27.6 | 27.6 | 28.3 | 29.3 | 33.5 | 36.3 | 38.4 | 40.1 | 42.7 | 44.7 |
| | β | 5.78 | 5.78 | 5.38 | 4.89 | 3.52 | 2.88 | 2.50 | 2.23 | 1.88 | 1.65 |
| ≤380 | θ | 29.5 | 29.5 | 29.7 | 31.1 | 36.5 | 39.9 | 42.4 | 44.4 | 47.4 | 49.7 |
| | β | 5.34 | 5.34 | 5.27 | 4.73 | 3.28 | 2.64 | 2.26 | 2.01 | 1.68 | 1.46 |
| ≤500 | θ | 31.2 | 31.2 | 31.2 | 32.3 | 38.8 | 42.7 | 45.5 | 47.6 | 50.9 | 53.4 |
| | β | 4.99 | 4.99 | 4.99 | 4.61 | 3.09 | 2.46 | 2.09 | 1.85 | 1.52 | 1.31 |
| ≤750 | θ | 34.1 | 34.1 | 34.1 | 34.2 | 42.3 | 46.9 | 50.1 | 52.6 | 56.3 | 59.0 |
| | β | 4.46 | 4.46 | 4.46 | 4.43 | 2.82 | 2.19 | 1.84 | 1.60 | 1.30 | 1.10 |
| ≤1000 | θ | 36.6 | 36.6 | 36.6 | 36.6 | 45.0 | 50.2 | 53.7 | 56.3 | 60.2 | 63.0 |
| | β | 4.06 | 4.06 | 4.06 | 4.06 | 2.62 | 2.00 | 1.66 | 1.43 | 1.14 | 0.95 |
| ≤1500 | θ | 40.8 | 40.8 | 40.8 | 40.8 | 49.2 | 55.1 | 58.9 | 61.8 | 65.8 | 68.6 |
| | β | 3.50 | 3.50 | 3.50 | 3.50 | 2.32 | 1.72 | 1.40 | 1.18 | 0.92 | 0.75 |
| ≤2000 | θ | 44.3 | 44.3 | 44.3 | 44.3 | 52.3 | 58.7 | 62.8 | 65.7 | 69.7 | 72.4 |
| | β | 3.10 | 3.10 | 3.10 | 3.10 | 2.11 | 1.52 | 1.21 | 1.01 | 0.76 | 0.62 |

AASHTO LRFD-05 uses the following requirements for maximum stirrup spacing.

If $v_f < 0.125 f_c'$

$$s_{max} = 0.8 d_v \leq 600 \text{ mm} \qquad \text{Eqn. (3.20)}$$

If $v_f \geq 0.125 f_c'$

$$s_{max} = 0.4 d_v \leq 300 \text{ mm} \qquad \text{Eqn. (3.21)}$$

In accordance with AASHTO LRFD-05 Clause 5.8.2.9, this study calculates $d_v$ as the greater of $0.9 \cdot d$ and $0.72 \cdot h$. This is identical to the calculation of $d_v$ using S6-06, as discussed in Section 3.2.

AASHTO LRFD-05 also sets a minimum amount of shear reinforcement area when stirrups are required. This limit is expressed as:

$$A_{v,\min} \geq 0.083 \sqrt{f_c'} \frac{b_v s}{f_v} \qquad (\text{mm}^2) \qquad \text{Eqn. (3.22)}$$

The process involved in predicting shear capacity using the shear method in AASHTO LRFD-05 is similar to the process required when using the sectional shear provisions in S6-06, as discussed in Section 3.2. The main difference in calculating shear capacity between the two shear methods is the use of Table 3.3 and Table 3.4 for obtaining the $\beta$ and $\theta$ values using AASHTO LRFD-05, as opposed to using Eqn. (3.5) and Eqn. (3.7) when using S6-06 shear provisions. Because the shear method in AASHTO LRFD-05 requires iterating the sectional forces to converge on the predicted shear capacity, obtaining the shear terms $\beta$ and $\theta$ requires considerably more work due to the required interpolation of values from Table 3.3 and Table 3.4. To simplify this process, Bentz (1999) provided Excel spreadsheets to automate calculating shear capacity using the sectional shear method in AASHTO LRFD-00. These spreadsheets are applicable for evaluation of sectional shear capacity using AASTHO LRFD-05 and are used in this study.

*Other Studies using the Shear Method in AASHTO LRFD-05*

The following predictions of shear capacity taken or derived from other studies are restated based on the author(s)' predictions and have not been checked in this study.

Kim (2004) used the sectional shear method found in AASHTO LRFD-98 (AASHTO, 1998) to predict the shear capacity of 1363 concrete members tested to fail in one-way shear. Because of the assumption for $\beta$ and $\theta$ values corresponding to longitudinal strains greater than $1.00 \times 10^{-3}$ used in this study the expressions for predicting shear capacity employed in AASHTO LRFD-98 are identical to those in AASHTO LRFD-05. The majority of members in Kim's (2004) dataset either contained no stirrups or were compliant with respect to AASHTO LRFD-05 minimum stirrup requirements. However a few members evaluated in Kim's (2004) study contained stirrups which do not comply with AASHTO LRFD-05 (or S6-06) stirrup spacing and area requirements. Table 3.5 summarizes the mean $V_{test}/V_{calc}$ ratios and COV values derived from his evaluations. It should be noted that Kim's (2004) dissertation did not provide the individual $V_{test}/V_{calc}$ ratios for the data set, so the reciprocal values could not be duplicated in this study.

Table 3.5 – Summary of Predictions by Kim (2004) using AASHTO LRFD-98

| Type | $V_{test}/V_{calc}$ | *COV (%)* |
|---|---|---|
| Non-Prestressed – Total Data Set (878) | 1.37 | 26.0 |
| Non-Prestressed – Members without Stirrups (718) | 1.39 | 27.0 |
| Non-Prestressed – Members with Stirrups (160) | 1.28 | 20.0 |
| Prestressed – Total Data Set (485) | 1.40 | 26.0 |
| Prestressed – Members without Stirrups (321) | 1.44 | 29.0 |
| Prestressed – Members with Stirrups (164) | 1.31 | 13.0 |

Angelakos et al. (2001) presented the results of twenty-one non-prestressed rectangular members evaluated using the shear provisions in AASHTO LRFD-98. Of the twenty-one members, five were non-compliant with respect to AASHTO LRFD-98 minimum stirrup area requirements, and were evaluated using the proposed method discussed in Section 2.3. Table 3.6 summarizes the mean $V_{calc}/V_{test}$ ratios and COV values derived from the authors' evaluations.

Table 3.6 – Summary of Predictions by Angelakos et al. (2001) using AASHTO LRFD-98

| Type | $V_{calc}/V_{test}$ | *COV (%)* |
|---|---|---|
| Non-Prestressed – Total Data Set (21) | 1.02 | 15.1 |
| Non-Prestressed – Members without Stirrups (15) | 1.00 | 13.2 |
| Non-Prestressed – Members with Stirrups (6) | 1.08 | 18.8 |

Collins (2001) evaluated 273 members meeting the criteria for member selection in this study using the sectional shear provisions in AASHTO LRFD-98. Member selection criteria used in this study is discussed in Section 4.3. Table 3.7 summarizes the mean $V_{calc}/V_{test}$ ratios and COV values derived from Collins' evaluations.

Table 3.7 – Summary of Predictions by Collins (2001) using AASHTO LRFD-98

| Type | $V_{calc}/V_{test}$ | *COV (%)* |
|---|---|---|
| Total Data Set (273) | 0.86 | 15.3 |
| Non-Prestressed – Members without Stirrups (128) | 0.89 | 16.0 |
| Non-Prestressed – Members with Stirrups (94) | 0.87 | 15.3 |
| Prestressed – Members without Stirrups (10) | 0.85 | 19.4 |
| Prestressed – Members with Stirrups (41) | 0.75 | 12.2 |

## 3.4 Response 2000

Software Response 2000 (Bentz, 2000), developed by Bentz as part of his Ph.D. research work, is a two dimensional sectional analysis program based on the Modified Compression Field Theory (Vecchio and Collins, 1986). The program separates member cross sections into concrete layers and longitudinal steel elements, and determines the longitudinal and shear stress distributions using a flexibility approach and Modified Compression Field Theory relationships.

Similar to other methods based on the Modified Compression Field Theory (Vecchio and Collins, 1986), Response 2000 (Bentz, 2000) requires a prediction of member crack spacing. Response 2000 allows the crack spacing to be specified by the user or defaults to the crack spacing given by CEB-FIP (1978) and provided as Eqn. (3.23). This study predicts crack spacing of members with stirrups complying and not complying with minimum shear reinforcement requirements and members without stirrups using the default crack spacing option. For reinforced sections that are subject to bending, Response 2000 limits the crack spacing to the depth of the member. This is an appropriate assumption for members subject to flexure (Base, 1982).

$$\text{crack spacing} = 2 \cdot c + 0.1 \cdot {}^{d_b}\!/_{\rho_r} \leq d \qquad \text{(mm)} \qquad \text{Eqn. (3.23)}$$

where

$c$ = diagonal distance from midsection to the nearest layer of longitudinal reinforcement in the section

$d_b$ = diameter of the nearest longitudinal bar

$\rho_r$ = longitudinal reinforcement ratio

Response 2000 calculates the longitudinal reinforcement ratio $\rho$ as the percentage of steel within a concrete area $7.5d_b$ above and below the longitudinal bar(s) nearest the mid-depth of the section. The concrete area defined by the distance $7.5d_b$ and below the longitudinal bar(s) is the effective embedment area given to CEB-FIP (1978).

Response 2000 (Bentz, 2000) provides options to predict the shear capacity at a section or to predict the full member response. Evaluation of shear capacity in this study uses the predictions made utilizing the sectional analysis option, which considers moment-shear interaction, cross section geometry and longitudinal and transverse reinforcement details at a section of interest. Longitudinal and transverse reinforcing details used for evaluation of shear capacity using Response 2000 are determined as discussed in Section 3.2. In this study the critical section for shear using Response 2000 is taken at a distance $d_v$ away from the externally applied load on the side of the closest support. This is identical to the location of the critical section for shear capacity evaluations using S6-06, as shown in Figure 3.4. All members evaluated in this study are simple span members tested using 1 or 2 point loading.

The concrete strength and aggregate size specified in the test literature is used for the Response 2000 (Bentz, 2000) concrete material properties input. Other than aggregate size, this study uses the Response 2000 concrete material defaults. An elastic, perfectly plastic stress-strain curve is assumed for the longitudinal and transverse reinforcement. The yield stress for the longitudinal reinforcement ($f_y$) and stirrups ($f_v$) reported in the test literature is used in this study as the stress limits. For prestressing reinforcement this study uses the guaranteed ultimate tensile strength $f_{pu}$ reported in the test literature. Response 2000 requires input of prestrain when calculating the sectional shear capacity of prestressed members. This study calculates the prestrain using Eqn. (3.24) as recommended by Bentz (2000).

$$\varepsilon_{prestrain} = \frac{0.7 \cdot f_{pu}}{E_p} \qquad \text{Eqn. (3.24)}$$

Evaluation of shear capacity using Response 2000 (Bentz, 2000) is applied in the same manner for members compliant and non-compliant with respect to S6-06 Section 14 stirrup spacing and area requirements and for members without stirrups analyzed in this study. It should be noted that Response 2000 is not a sectional shear design/evaluation provision, and as such it does not provide minimum transverse reinforcement requirements. This study uses the stirrup detail ratios $s/s_{m1}$ and $A_{v,min}/A_v$ calculated for evaluation using the sectional shear provisions in S6-06 Section 14. Eqn (3.12), proposed in this study to interpolate the effective stirrup area for members with a stirrup area between Eqn. (2.1) and Eqn. (2.2) in accordance with S6-06 Clause 14.14.1.6.2, is not included in this study for evaluation of shear capacity using Response 2000.

The manual for Response 2000 can be found on the world wide web (Bentz, 2000). For this reason no flow chart describing how Response 2000 is used in this study for evaluating shear capacity is provided. Bentz (2000) also provides a detailed description of the principles and use of software Response 2000.

*Other Studies using Response 2000*

The following predictions of shear capacity taken or derived from other studies are restated based on the author(s)' predictions and have not been checked in this study.

Bentz (2000) presented the results of the evaluation of 534 members with and without shear reinforcement using Response 2000. Members with transverse reinforcement were primarily compliant with respect to S6-06 Section 14 stirrup spacing and area requirements. Table 3.8 summarizes the mean $V_{calc}/V_{test}$ ratios and COV values derived from his evaluations.

Table 3.8 – Summary of Predictions by Bentz (2000) using Response 2000

| Type | $V_{calc}/V_{test}$ | *COV (%)* |
|---|---|---|
| Total Data Set (534) | 0.96 | 12.2 |
| Non-Prestressed Members without Stirrups (201) | 0.99 | 12.5 |
| Prestressed Members without Stirrups (36) | 0.99 | 12.5 |
| Non-Prestressed Members with Stirrups (192) | 0.97 | 11.7 |
| Prestressed Members with Stirrups (105) | 0.91 | 10.9 |

Lubell (2006) used Response 2000 to predict the shear capacity of 106 members with shear reinforcement typically compliant with respect to S6-06 Section 14 stirrup spacing and area requirements. Table 3.9 summarizes the mean $V_{calc}/V_{test}$ ratios and COV values derived from his evaluations.

Table 3.9 – Summary of Predictions by Lubell (2006) using Response 2000

| Type | $V_{calc}/V_{test}$ | *COV (%)* |
|---|---|---|
| Members with Stirrups (106) | 1.02 | 15.1 |

Collins (2001) evaluated 273 members meeting the criteria of this study using Response 2000. Table 3.10 summarizes the mean $V_{calc}/V_{test}$ ratios and COV values derived from his evaluations. Member selection criteria used in this study is discussed in Section 4.3.

Table 3.10 – Summary of Predictions by Collins (2001) using Response 2000

| Type | $V_{calc}/V_{test}$ | *COV (%)* |
|---|---|---|
| Total Data Set (273) | 0.96 | 12.6 |
| Non-Prestressed – Members without Stirrups (128) | 0.98 | 12.4 |
| Non-Prestressed – Members with Stirrups (94) | 0.96 | 13.0 |
| Prestressed – Members without Stirrups (10) | 1.04 | 12.2 |
| Prestressed – Members with Stirrups (41) | 0.91 | 12.4 |

## 3.5 Building Code Requirements for Structural Concrete ACI 318-08

ACI 318-08 contains separate approaches for non-prestressed and prestressed members. The equations provided in this section are the ones most commonly used in practice and in other studies (Angelakos, 1999; Collins, 2001; Kim, 2004). The ACI 318 sectional shear method used for evaluation in this study is the metric equivalent to ACI 318, and not ACI 318M.

The ACI 318-08 shear method predicts the shear capacity of concrete beams using Eqn. (3.25). The upper limit of $V_n$ is different for non-prestressed and prestressed members. Upper limits for $V_c$ and $V_s$ are provided in this Section.

$$V_n = V_c + V_s \qquad \text{(N)} \qquad \text{Eqn. (3.25)}$$

The concrete contribution term $V_c$ used by ACI 318-08 attempts to predict the sectional shear force corresponding to significant diagonal cracking for members without transverse reinforcement (ACI-ASCE 326, 1962). Members without stirrups are found to fail at or near this condition. ACI 318-08 calculates the concrete contribution $V_c$ for non-prestressed members using Eqn. (3.26).

$$V_c = \left(0.16\sqrt{f_c^{'}} + 17 \cdot \rho \; \frac{V_f d}{M_f}\right) \cdot b_v d \qquad \text{(N)} \qquad \text{Eqn. (3.26)}$$

To simplify this expression, ACI 318-08 allows the use of Eqn. (3.27) for shear capacity evaluation of non-prestressed members not subject to significant axial loading. Eqn. (3.27) is used for analysis in this study.

$$V_c = \frac{1}{6} \cdot \sqrt{f_c^{'}} \cdot b_v \cdot d \qquad \text{(N)} \qquad \text{Eqn. (3.27)}$$

ACI 318 uses a different expression for predicting the concrete shear capacity contribution $V_c$ for prestressed members. For members having an effective prestressing force accounting for at least 40% of the tensile strength of flexural reinforcement, ACI 318-08 calculates $V_c$ using Eqn. (3.28).

$$V_c = \left( 0.05\sqrt{f_c^{'}} + 5\frac{V_f d}{M_f} \right) \cdot b_v d \qquad \text{(N)} \qquad \text{Eqn. (3.28)}$$

This expression was limited to $0.17\sqrt{f_c^{'}} \cdot b_v \cdot d \le V_c \le 0.4\sqrt{f_c^{'}} \cdot b_v \cdot d$.

In accordance with ACI 318-08 Clause 11.1.2.1 for members complying with ACI 318-08 stirrup area requirements, this study does not limit the value of $\sqrt{f_c^{'}}$. For members without stirrups or with less stirrup area than required by Eqn. (2.5) and Eqn. (3.30) for non-prestressed and prestressed members respectively, this study limits $\sqrt{f_c^{'}}$.to 8.30 MPa. This is consistent with the requirements of ACI 318-08 Clause 11.1.2.

The web reinforcement contribution used by ACI 318-08 is based on the 45° Truss Model, and is calculated for both non-prestressed and prestressed members using Eqn. (3.29). This expression is checked against the limit $V_s \le \frac{2}{3} \cdot \sqrt{f_c^{'}} \cdot b_v \cdot d$ to assess whether web crushing is expected to occur prior to sectional shear failure.

$$V_s = \frac{A_v \cdot f_v \cdot d}{s} \qquad \text{(N)} \qquad \text{Eqn. (3.29)}$$

ACI 318-08 limits the longitudinal stirrup spacing in members to the smallest of the following spacing limits:

- $d/2$ for non-prestressed members
- $0.75h$ for prestressed members
- 24 in (600mm)

For members where $V_s$ exceeds $\frac{1}{3} \cdot \sqrt{f_c^{'}} \cdot b_v \cdot d$, the maximum stirrup spacing is reduced by a factor of 2.

As discussed in Section 2.2, ACI 318-08 determines the minimum required stirrup area for prestressed members using Eqn (2.5). For members with an effective prestressing force equal to 40% of the total longitudinal reinforcement strength, ACI 318-08 determines the minimum permissible stirrup area using Eqn. (3.30).

$$A_{v,\min} = 0.06 \cdot \sqrt{f_c^{'}} \cdot \frac{b_v \cdot s}{f_v} \qquad (\text{mm}^2) \qquad \text{Eqn. (2.5)}$$

$$A_{v,\min} = \frac{A_{ps} \cdot f_{pu} \cdot s}{80 \cdot f_v \cdot d} \cdot \sqrt{\frac{d}{b_v}} \qquad (\text{mm}^2) \qquad \text{Eqn. (3.30)}$$

*Application of ACI 318-05 Shear Method*

The flowchart presented in this Section demonstrates how the sectional shear provisions in ACI 318-08 are used in this study to predict the shear capacity of simple spans subject to point loads. Predicting the shear capacity of non-prestressed members using ACI 318-08 is not an iterative process when using Eqn. (3.27). However Eqn. (3.28), used to predict the concrete contribution to shear capacity for prestressed members, is dependent on the sectional moments and shear forces and requires the externally applied load to be iterated until the resulting applied shear force equals the predicted shear capacity $V_n$, calculated using Eqn. (3.25).

In accordance with ACI 318-08 Clause 11.1.3 the critical location used in this study for evaluating the shear capacity of non-prestressed members is taken at a distance $d$ away from the member support. This section is appropriate for evaluation of shear capacity using Eqn. (3.27), because this equation is not a function of the sectional forces. The critical shear location for evaluation of prestressed members using ACI 318-08 varies from the critical location for evaluation of non-prestressed members, because Eqn. (3.28) is dependant on the moment to shear ratio. The critical shear location for evaluation of prestressed members in this study is taken at a distance $d_v$ from the externally applied load. Similar to evaluation using S6-06, $d_v$ is calculated as the larger of $0.9 \cdot d$ or $0.72 \cdot h$. This location is consistent with evaluation in this study using the shear methods derived from the Modified Compression Field Theory (Vecchio and Collins, 1986), as discussed in Section 3.2. ACI 318-08 shear critical sections are provided in Figure 3.5. For prestressed members subjected to a uniformly distributed load, the shear capacity should be evaluated at numerous sections along the member length, and the section which is determined to have the highest $V_{test}/V_{calc}$ should be selected as the shear critical section. This would be implemented at Step 2 in the flow chart at the end of this Section.

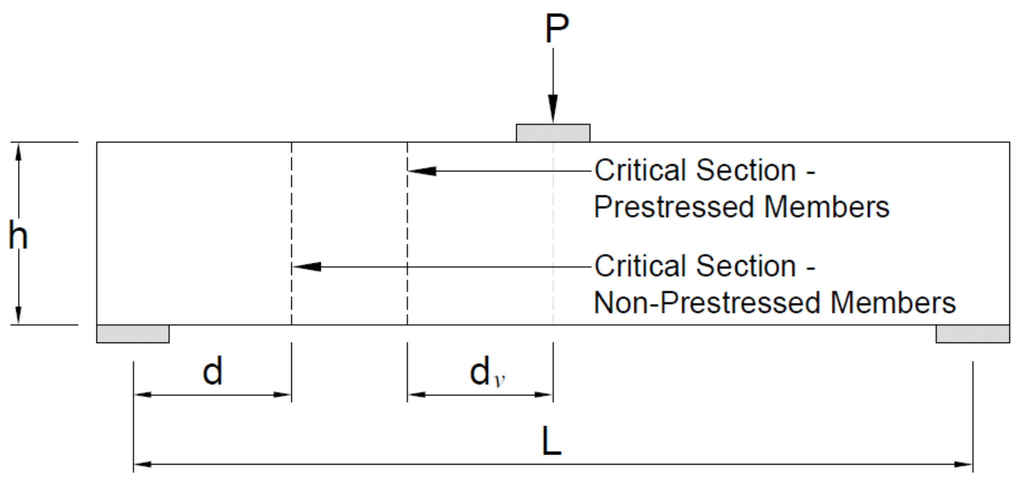



Figure 3.5 – ACI 318-08 Shear Critical Sections

ACI 318-08 does not contain provisions for evaluating members with stirrups not complying with minimum stirrup requirements. This study predicts the shear capacity of non-compliant members using the longitudinal stirrup spacing specified in the test literature and assuming fully effective stirrup area. Longitudinal and transverse reinforcement details are determined as discussed in Section 3.2 when evaluating shear capacity using AC8 318-08 shear provisions.

*Other Studies using the ACI 318-05 Shear Method*

The following predictions of shear capacity taken or derived from other studies are restated based on the author(s)' predictions and have not been checked in this study.

The method for predicting shear capacity using ACI 318 has remained the same from the 1977 publication through to the current 2008 provisions. As such evaluation of shear capacity discussed in this subsection is relevant to shear capacity predictions using ACI 318-08.

Kim (2004) used the sectional shear method found in ACI 318-02 (ACI, 2002) to predict the shear capacity of 1363 concrete members with and without shear reinforcement tested to fail in diagonal shear. The members with stirrups were primarily compliant with respect to S6-06 minimum stirrup requirements. Table 3.11 summarizes the mean $V_{test}/V_{calc}$ ratios and COV values derived from his evaluations. It should be noted that Kim's (2004) dissertation did not provide the individual $V_{test}/V_{calc}$ ratios for the data set, so the reciprocal values could not be duplicated in this study.

Table 3.11 - Summary of Predictions by Kim (2004) using ACI 318-02

| Type | $V_{test}/V_{calc}$ | *COV (%)* |
|---|---|---|
| Non-Prestressed – Total Data Set (878) | 1.51 | 40.0 |
| Non-Prestressed – Members without Stirrups (718) | 1.54 | 42.0 |
| Non-Prestressed – Members with Stirrups (160) | 1.35 | 26.0 |
| Prestressed – Total Data Set (485) | 1.33 | 24.0 |
| Prestressed – Members without Stirrups (321) | 1.38 | 25.0 |
| Prestressed – Members with Stirrups (164) | 1.24 | 20.0 |

Lubell (2006) evaluated 106 concrete members with stirrups tested to fail in shear using the sectional shear provisions in ACI 318-05. Table 3.12 summarizes the mean $V_{calc}/V_{test}$ ratios and COV values derived from his evaluations.

Table 3.12 – Summary of Predictions by Lubell (2006) using ACI 318-05

| Type | $V_{calc}/V_{test}$ | *COV (%)* |
|---|---|---|
| Members with Stirrups (106) | 0.83 | 23.1 |

Angelakos et al (2001) presented the shear capacity evaluation results of twenty-one rectangular members using the provisions in ACI 318-95 (ACI, 1995). Table 3.13 summarizes the mean $V_{calc}/V_{test}$ ratios and COV values derived from the authors' evaluations.

Table 3.13 – Summary of Predictions by Angelakos et al. (2001) using ACI 318-95

| Type | $V_{calc}/V_{test}$ | *COV (%)* |
|---|---|---|
| Non-Prestressed – Total Data Set (21) | 1.56 | 24.1 |
| Non-Prestressed – Members without Stirrups (15) | 1.68 | 13.2 |
| Non-Prestressed – Members with Stirrups (6) | 1.28 | 18.8 |

Bentz (2000) presented the shear capacity evaluation results of 448 members evaluated using the shear provisions in ACI 318-99. Table 3.14 summarizes the mean $V_{calc}/V_{test}$ ratios and COV values derived from the authors' evaluations.

Table 3.14 – Summary of Predictions by Bentz (2000) using ACI 318-99

| Type | $V_{calc}/V_{test}$ | *COV (%)* |
|---|---|---|
| Total Data Set (448) | 0.90 | 31.8 |
| Members without Stirrups (217) | 0.96 | 37.7 |
| Members with Stirrups (231) | 0.85 | 21.4 |

Collins (2001) evaluated the shear capacity of 273 members meeting the criterion of this study (see Section 4.3) using the provisions in ACI 318-95. Table 3.15 summarizes the mean $V_{calc}/V_{test}$ ratios and COV percentages derived from his evaluations.

Table 3.15 – Summary of Predictions by Collins (2001) using ACI 318-95

| Type | $V_{calc}/V_{test}$ | *COV (%)* |
|---|---|---|
| Total Data Set (273) | 0.94 | 30.6 |
| Non-Prestressed – Members without Stirrups (128) | 0.98 | 37.0 |
| Non-Prestressed – Members with Stirrups (94) | 0.94 | 26.9 |
| Prestressed – Members without Stirrups (10) | 0.95 | 14.9 |
| Prestressed – Members with Stirrups (41) | 0.79 | 23.0 |

Flowchart – ACI 318-08 Sectional Shear Method

Step 1: Determine section geometry and material properties for the member to be evaluated. The same process is used for design and evaluations.

Step 2: Calculate moments and shears from the externally applied load and member self weight at a section $d$ away from the support for non-prestressed members and $d_v$ away from the externally applied load for prestressed members. $d_v$ is calculated as the larger of $0.9 \cdot d$ and $0.72 \cdot h$.

Step 3: Determine Stirrup Spacing and Area Requirements

ACI 318-08 limits the maximum stirrup spacing to the smallest of the following requirements.

- $d/2$ for non-prestressed members
- $0.75h$ for prestressed members
- 24 in (600mm)

Members with a stirrup contribution to shear $V_s$ greater than $0.33 \cdot \sqrt{f_c^{'}} \cdot b_v \cdot d$ should reduce the above maximum stirrup spacing by a factor of 2.

The minimum stirrup area for non-prestressed members required by ACI 318-08 is calculated using Eqn. (2.5), while the minimum stirrup area for prestressed members is determined using Eqn. (3.30).

$$A_{v,\min} = 0.06\sqrt{f_c^{'}}\frac{b_v s}{f_v} \text{, but not less than } \frac{0.35 b_v s}{f_v} \qquad (\text{mm}^2) \quad \text{Eqn. (2.5)}$$

$$A_{v,\min} = \frac{A_{ps} \cdot f_{pu} \cdot s}{80 \cdot f_v \cdot d} \cdot \sqrt{\frac{d}{b_v}} \qquad (\text{mm}^2) \qquad \text{Eqn. (3.30)}$$

This study assumed the stirrups to be fully effective regardless of whether or not the member complied with longitudinal stirrup spacing and stirrup area requirements. As such, the process in Step 4 is the same for members complying with stirrup spacing and area requirements, members not complying with stirrup spacing and/or area requirements, and members without stirrups.

Step 4: Calculate the Shear Resistance $V_n$

The concrete contribution to shear resistance for non-prestressed members was calculated using Eqn. (3.27).

$$V_c = \frac{1}{6} \cdot \sqrt{f_c^{'}} \cdot b_v \cdot d \qquad \text{(N)} \qquad \text{Eqn. (3.27)}$$

The concrete contribution to shear resistance for prestressed members was calculated using Eqn. (3.28).

$$V_c = \left( 0.05\sqrt{f_c^{'}} + 5\frac{V_f d}{M_f} \right) \cdot b_v d \qquad \text{(N)} \qquad \text{Eqn. (3.28)}$$

Eqn. (3.28) is limited to $0.17\sqrt{f_c^{'}} \cdot b_v d \leq V_c \leq 0.4\sqrt{f_c^{'}} \cdot b_v d$

The stirrup contribution to shear resistance was calculated using Eqn. (3.29):

$$V_s = \frac{A_v \cdot f_v \cdot d}{s} \qquad \text{(N)} \qquad \text{Eqn. (3.29)}$$

Eqn. (3.28) is limited to $V_s \leq \frac{2}{3} \cdot \sqrt{f_c^{'}} \cdot b_v \cdot d$

The shear capacity was then calculated using Eqn. (3.25).

$$V_n = V_c + V_s \qquad \text{(N)} \qquad \text{Eqn. (3.25)}$$

Step 5: Converge Predicted Shear Capacity

For non-prestressed members, no iteration is required when calculating shear capacity using Eqn. (3.27), as this equation is independent of sectional forces. Predicting shear capacity of prestressed members using Eqn. (3.28) is an iterative process, because this equation is dependent on the sectional forces. This study varies the externally applied load at Step 2, which in turn varies the moments and shears at the critical section, until the applied shear force equals the calculated shear capacity.

# Chapter 4
# Evaluation using Sectional Shear Provisions

## 4.1 Introduction

Chapter 4 provides the evaluations of shear capacity calculated using the four sectional shear methods discussed in Chapter 3 for the members identified as suitable for this study. The purpose of these analyses is to assess the agreement between predicted and tested shear capacities calculated using the beam shear provisions in S6-06 (CSA, 2006), AASHTO LRFD-05 (AASHTO, 2005), Response 2000 (Bentz, 2000) and ACI 318-08 (ACI, 2008) for concrete members with excessive stirrup spacing and inadequate stirrup area according to S6-06 Section 14 provisions. Section 4.2 presents the Demerit Point model proposed in this study. This demerit point model provides a quantitative method for assessing and comparing predicted and tested shear capacities determined using the four sectional shear methods. Section 4.3 provides the criteria for member selection used in this study. Section 4.4 provides tables which give the predicted to tested shear capacity ratios ($V_{calc}/V_{test}$) of the members identified for analysis in this study, calculated using the four shear evaluation methods discussed in Chapter 3. These tables also provide the stirrup detail ratios $s/s_{m1}$ (or $s/s_{max}$) and $A_{v,min}/A_v$ for the identified members. Section 4.5 to Section 4.8 respectively provide the following discussions for evaluations of shear capacity of the members identified for this study using S6-06, AASHTO LRFD-05, Response 2000 and ACI 318-08:

- Allocation of shear capacity predictions into the classification ranges presented in Table 4.1.
- Allocation of average demerit points per member. These demerit points are allotted using the model detailed in Section 4.2.
- Influence of variations in the stirrup detail ratios $s/s_{m1}$ (or $s/s_{max}$) and $A_{v,min}/A_v$ on the agreement between predicted and tested shear capacities.
- Influence of concrete strength, shear span to depth ratio, longitudinal reinforcing ratio and member shape on predicted to tested shear capacities.
- Mean $V_{calc}/V_{test}$ ratios and corresponding COV.

Section 4.9 compares the results from evaluation using the four sectional shear methods against the criteria identified in Section 2.7 as being critical for a method to address in order to be able to declare that the method is suitable for predicting sectional shear capacity of concrete girders. Section 4.10 identifies details which evaluation in Section 4.5 indicates adversely affect the agreement between predicted and tested shear capacities calculated using the sectional shear method in S6-06, and introduces the modified shear methods proposed in Chapter 5 which address these details.

## 4.2 Demerit Point Model

A Demerit Point model, adapted from Collins' (2001) Demerit Point concept described in Section 2.6, is proposed in this study to provide a system for quantifying the relative performance of each shear evaluation method in order to augment the statistical data presented. Similar to Collins' (2001) Demerit Point concept, demerit points assigned to a test specimen in this study are a function of the specimen's $V_{calc}/V_{test}$ ratio. More demerit points are assigned to a test specimen as its $V_{calc}/V_{test}$ ratio deviates further from unity.

As discussed in Section 2.6, Collins (2001) did not provide a specific rational for the value of demerit points allotted to each prediction classification. However, because the demerit points allotted to each member are a function of that members $V_{calc}/V_{test}$ ratio, a Demerit Point model can be considered as an extension of basic statistics. Collins allocation of demerit points also uses logical considerations. As it is of greater concern to have an unsafe prediction compared to an overly conservative prediction the allotted quantity of demerit points increases faster for $V_{calc}/V_{test}$ ratios greater than 1.00 than for $V_{calc}/V_{test}$ ratios less than 1.00 (see Figure 4.1). Demerit Point models also provide a quick and efficient method of assessing the agreement because predicted and tested shear capacities and because the demerit points allotted to a member using the model proposed in this study are a function of that members $V_{calc}/V_{test}$ ratio, COV in demerit points will be related to the COV in the $V_{calc}/V_{test}$ ratios.

Using the Demerit Point model proposed in this study, the demerit points allotted to each member are a linear function of the member's corresponding $V_{calc}/V_{test}$ ratio. The rationale for making allotted demerit points a linear function of $V_{calc}/V_{test}$ ratios, as opposed to allotting demerit points using Collins' (2001) step distribution, is that the demerit points assigned to a member should continually increase as its corresponding $V_{calc}/V_{test}$ ratio deviates from 1.00. In this way assessing agreement between predicted and tested shear capacities using the Demerit Point model proposed in this study is methodically consistent with using average $V_{calc}/V_{test}$ ratios. Both models suggest progressively worse agreement between predicted and tested shear capacities as $V_{calc}/V_{test}$ ratios deviate from 1.00. This being said, the Demerit Point model proposed in this study recognizes a range of $V_{calc}/V_{test}$ ratios appropriately close to unity, for which no demerit points are assigned. In this way quantifying agreement between predicted and tested shear capacities using the Demerit Point model proposed in this study differs from quantifying this agreement based on average $V_{calc}/V_{test}$ ratios.

Figure 4.1 illustrates how demerit points are assigned in this study, and how they were assigned by Collins (2001).

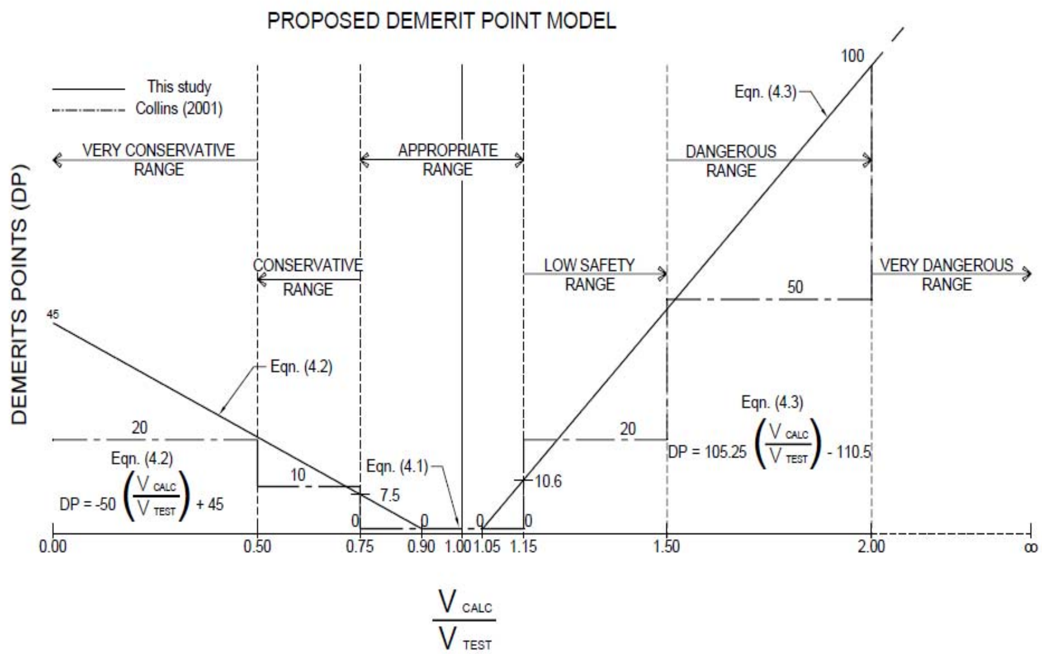



Figure 4.1 – Demerit Point System

$$DP = 0 \qquad \text{for} \qquad 0.90 < \frac{V_{calc}}{V_{test}} \leq 1.05 \qquad \text{Eqn. (4.1)}$$

$$DP = -50 \cdot \left( V_{calc} \big/ V_{test} \right) + 45 \quad \text{for} \quad 0 < \frac{V_{calc}}{V_{test}} \leq 0.90 \qquad \text{Eqn. (4.2)}$$

$$DP = 105.25 \cdot \left( V_{calc} \big/ V_{test} \right) - 110.5 \quad \text{for } 1.05 > \frac{V_{calc}}{V_{test}} \qquad \text{Eqn. (4.3)}$$

It should be noted in Figure 4.1 that demerit points allotted by Collins (2001), as given in Table 2.3, have been exaggerated by a factor of 10 (see Section 2.6). The linear functions for calculating allotted demerit points by the model proposed in this study are ‘fit’ to Collins’ (2001) exaggerated ‘steps’ as follows:

- $V_{calc}/V_{test}$ ratios between 0.90 and 1.05 are allotted zero demerit points as described by Eqn. 4.1.
- For $V_{calc}/V_{test}$ ratios between 0 and 0.90 the Demerit Point model proposed in this study matches Collins’ *exaggerated* Demerit Point model at $V_{calc}/V_{test}$ equal to 0.90 and 0.50. Based on these points, a linear function is interpolated between $V_{calc}/V_{test}$ equal to 0 and 0.90. Eqn (4.2) provides the expression used in this study for calculating the demerit points allotted to members with predicted to tested shear capacity ratios in this range.
- For $V_{calc}/V_{test}$ ratios greater than 1.05 the Demerit Point model proposed in this study matched Collins’ *exaggerated* Demerit Point model at $V_{calc}/V_{test}$ equal to 1.05 and 2.00 and a linear function is interpolated between these points. Eqn. (4.3) provides the expression used in this study for calculating the demerit points allotted to members with predicted to tested shear capacity ratios greater than 1.05.

The various data categories (eg. non-compliant non-prestressed members, non-compliant prestressed members, etc.) evaluated in this study have differing numbers of members. Therefore the average demerit points per member are used to assess the agreement between predicted and tested shear capacities for the sectional shear evaluation methods examined in this study. Figure 4.1 indicates that sectional shear methods which present with fewer than 7.5 average demerit points per member will typically provide predictions of shear capacity which are in good agreement with tested capacities.

Table 4.1 provides the classifications for $V_{calc}/V_{test}$ ratio ranges used in this study. These correspond with the reciprocal values of Collins (2001) ranges as given in Table 2.3.

Table 4.1 - $V_{calc}/V_{test}$ Classification Ranges (adapted from Collins, 2001)

| Range Designation | $V_{calc}/V_{test}$ Range |
|---|---|
| Very Conservative | <0.50 |
| Conservative | 0.50 – 0.75 |
| Appropriate | 0.75 – 1.15 |
| Low Safety | 1.15 – 1.50 |
| Dangerous | 1.50 – 2.00 |
| Very Dangerous | >2.00 |

## 4.3 Member Criteria and Selection

The primary members of interest to this study are concrete girders with transverse reinforcement details not complying with S6-06 Section 14 stirrup spacing and area requirements. Such details are commonly encountered during shear capacity evaluations, particularly for members designed according to code provisions which did not include the shear demand magnitude in stirrup spacing requirements. Factors leading to members being classified as non-compliant are discussed in Section 2.2. Members with stirrups complying with S6-06 Section 14 minimum transverse reinforcement requirements and members without stirrups are included in the dataset evaluated in this study to facilitate a comparison of the agreement between predicted and tested shear capacities with the non-compliant members.

The following criteria were used when identifying members for evaluation in this study:

- Test specimens which failed in beam action for one-way shear, as opposed to flexural failures, web-crushing, anchorage failures, etc. The flexural capacities of all members evaluated in this study were checked using the flexural capacity method in S6-06 Section 8 to ensure that this mode of failure did not govern. To account for scatter in flexural predictions, members which mobilized more than 95% of their predicted moment capacity were not included. The crushing capacity of the concrete webs was also checked in accordance with S6-06 Clause 8.9.3.3 to determine whether this was the expected mode of failure (see Eqn. (3.1)). This web crushing check is a component of the sectional shear method given in S6-06 Section 8 as discussed in Section 3.2. The anchorage capacity of the longitudinal reinforcement was determined in accordance with S6-06 Clause 8.9.3.14 to check whether this mode of failure was expected to govern.

- Total member height *h* greater than 300 mm. This limit on minimum section height is used so that specimens examined in this study can be considered representative of members encountered in service. Discussion by Leonhardt (as cited in Collins, 2001) indicates that larger members yield more productive research for shear in concrete members. It should be noted that section height is not a criteria used by S6-06 Section 8 for determining whether a member requires stirrups, although as discussed earlier for members with stirrups section height is a criteria for determining maximum stirrup spacing (see Section 2.2). Heights of members evaluated in this study range from 300 mm to 2000 mm.
- Shear span to depth ratio *a/d* not less than 2.5 and not greater than 6.0. As discussed in Section 2.5.2 this range is appropriate for studying one-way shear behavior of reinforced concrete members. Shear span-to-depth ratios of members evaluated in this study range from 2.58 to 5.57.
- Concrete compressive strength not exceeding 60 MPa. Concrete with strengths greater than 60 MPa were not examined in this study because high strength concrete has become more common only in recent years. Members with concrete strengths greater than 60 MPa are typically designed using provisions for detailing minimum transverse reinforcement in which maximum stirrup spacing is a function of the normalized shear demand. Current provisions for designing members which are comprised of high strength concrete also account for concrete strength when determining minimum stirrup area requirements. As such, members fabricated using high strength concrete typically comply with minimum stirrup requirements. Concrete strengths of members evaluated in this study range from 15.7 MPa to 60 MPa.

Figure 4.2 provides a breakdown by category of the members identified for evaluation in this study, while Table 4.2 provides the parametric detail ranges for each category. The $s/s_{m1}$ and $A_{v,min}/A_v$ ranges provided in Table 4.2 are based on S6-06 Section 14 provisions as described in Section 3.2, and are used to define each member as compliant or non-compliant with respect to stirrup spacing and area requirements. As indicated in Table A2, all prestressed members evaluated in this study had an effective prestressing force greater than 40% of the total prestressing force. None of the prestressed members had harped or sloped strands.

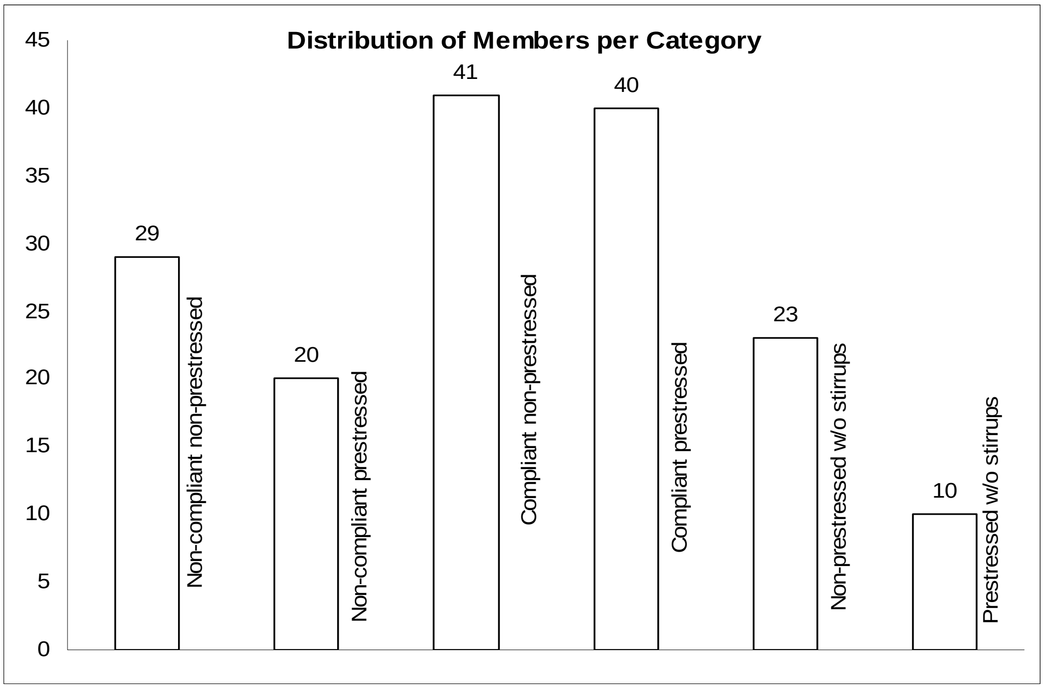



Figure 4.2 – Distribution per Category of Members Evaluated in this Study

Table 4.2 – Parametric Detail Ranges of 163 Members Evaluated in this Study

| Range | $f_c^{'}$ (MPa) | $a/d$ | $\rho$ (%) | $b/b_v$ | $d$ (mm) | $s/s_{m1}$ | $A_{v,min}/A_v$ | $\frac{v}{f_c^{'}}$ |
|---|---|---|---|---|---|---|---|---|
| Non-Compliant Non-Prestressed Members | | | | | | | | |
| Min-Max | 23.6–51.3 | 2.81-5.36 | 0.48-3.42 | 1.00-4.00 | 271-1890 | 0.74-4.50 | 0.26-1.77 | 0.012-0.088 |
| Non-Compliant Prestressed Members | | | | | | | | |
| Min-Max | 24.5–57.8 | 3.00-5.32 | 0.32-1.14 | 2.06-6.68 | 254-363 | 1.19-1.81 | 0.16-0.99 | 0.039-0.175 |
| Compliant Non-Prestressed Members | | | | | | | | |
| Min-Max | 15.7–55.8 | 2.78-5.36 | 0.36-3.46 | 1.00-13.9 | 279-925 | 0.27-0.90 | 0.06-0.99 | 0.033-0.245 |
| Compliant Prestressed Members | | | | | | | | |
| Min-Max | 27.5–60.0 | 2.58-5.57 | 0.30-1.14 | 2.98-5.93 | 269-1003 | 0.30-0.98 | 0.07-0.66 | 0.045-0.215 |
| Non-Prestressed Members without Stirrups | | | | | | | | |
| Min-Max | 19.9–55.0 | 2.86-5.50 | 0.42-2.85 | 1.00-4.00 | 279-1890 | - | - | 0.012-0.035 |
| Prestressed Members without Stirrups | | | | | | | | |
| Min-Max | 39.6–45.4 | 3.51-5.32 | 0.30-0.99 | 3.99-6.68 | 300-411 | - | - | 0.034-0.088 |

The members identified for evaluation in this study are found in the following abbreviated references. Full references are found in Chapter 7.

- Angelakos et al., 2001
- Aster and Koch (as cited in Kim, 2004)
- Bennett and Debaiky, 1974
- Collins and Kuchma, 1999
- DeGeer and Stephens, 1993
- Durham et al., 2003
- Elzanaty et al., 1986
- Frosch, 2000
- Higgins et al., 2004
- Johnson and Ramirez, 1989
- Kani et al, 1979
- Krefeld and Thurston, 1966
- Leonhardt and Walther, 1964
- Lubell, 2006
- Lyngberg, 1976
- MacGregor, 1960
- Moa et al., 1997
- Moayer and Reagan, 1974
- Shahaway and Batchelor, 1996
- ShenCao, 2001
- Tompos and Frosch, 2002
- Vecchio and Shim, 2004
- Yoon et al., 1996
- Yoshida, 2000

## 4.4 $V_{calc}/V_{test}$ and Stirrup Detail Ratios $s/s_{m1}$ and $A_{vmin}/A_v$ for Evaluated Members

Table 4.3 provides the $V_{calc}/V_{test}$, $s/s_{m1}$ (or $s/s_{max}$), and $A_{v,min}/A_v$ ratios from evaluation of shear capacity using the four sectional shear methods discussed in Chapter 3. The values in Table 4.3 for S6-06 use the method for shear evaluation discussed in Section 3.2. The stirrup spacing limit $s_{m2}$ is disregarded, as discussed in Section 4.5, and the interpolation for stirrup area required by S6-06 Clause 14.14.1.6.2 is accommodated using Eqn. (3.12). The values in Table 4.3 for shear evaluation using AASHTO-LRFD-05 provisions were calculated using the spreadsheets provided by Bentz (1999) and the process discussed in Section 3.3. The method used to determine the $V_{calc}/V_{test}$ values for Response 2000 (Bentz, 2000) was discussed in Section 3.4 while the process discussed in Section 3.5 was used to produce the values per ACI 318-08 in Table 4.3. As discussed in Section 3.4, this study uses the same stirrup detail ratios $s/s_{m1}$ and $A_{v,min}/A_v$ for Response 2000 as from evaluation using the provisions in S6-06 Section 14.

The stirrup spacing limit $s_{m2}$ is disregarded for shear capacity evaluation using Response 2000.

Table 4.4a provides the $V_{calc}/V_{test}$ and $s/s_{m2}$ ratios for the forty-nine non-compliant members with stirrups evaluated in this study using S6-06. These predictions use no stirrup contribution to shear capacity when the actual stirrup spacing $s$ exceeds the allowed stirrup spacing limit $s_{m2}$.

Table 4.4b provides the $V_{calc}/V_{test}$ and $A_{v,min}/A_v$ ratios for the twenty-nine non-prestressed non-compliant members evaluated in this study using S6-06 and assuming full stirrup contribution to shear capacity for members with non-compliant stirrup area. All prestressed members with stirrups evaluated in this study comply with S6-06 minimum stirrup area requirements, and as such are not included in Table 4.4b. Geometric and material properties for all specimens are provided in Appendix A Tables A.1 and A.2. References for all members evaluated in this study are found in Chapter 7.

The ratios $s/s_{m1}$ ($s/s_{max}$) and $A_{v,min}/A_v$ are used to identify compliant and non-compliant members. Members with $s/s_{m1}$ ($s/s_{max}$) and/or $A_{v,min}/A_v$ ratios greater than 1.00 are non-compliant, while ratios less than 1.00 are compliant with respect to the specific design provisions being assessed. Some members determined as compliant with respect to S6-06 Section 14 stirrup spacing and area provisions are non-compliant using other provisions, and vice versa. These members are included in their corresponding S6-06 classification to keep consistent data sets but have been marked with an asterisks in Table 4.3.

Table 4.3 – Evaluation Results using Four Sectional Shear Methods– 163 Member Data Set

| Members | *S6-06* $V_{calc}/V_{test}$ | | *AASHTO LRFD-05* $V_{calc}/V_{test}$ | | *Response 2000* $V_{calc}/V_{test}$ | | *ACI 318-08* $V_{calc}/V_{test}$ | |
|---|---|---|---|---|---|---|---|---|
| | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ |
| Non-Compliant Members – Non-Prestressed | | | | | | | | |
| YB2000/9 | 0.70 | | 0.71 | | 1.03 | | 1.61 | |
| | 4.50 | 0.94 | 4.50 | 1.29 | 4.50 | 0.94 | 4.50 | 0.941 |
| YB2000/6 | 0.58 | | 0.61 | | 0.86 | | 1.39 | |
| | 2.25 | 1.12 | 2.25 | 1.55 | 2.25 | 1.12 | 2.25 | 1.12 |
| YB2000/4 | 0.48 | | 0.50 | | 0.71 | | 1.13 | |
| | 0.98 | 1.08 | 0.98 | 1.49 | 0.98 | 1.08 | 0.98 | 1.08 |
| 5084 | 0.65 | | 0.54 | | 0.62 | | 0.48 | |
| | 2.55 | 0.44 | 2.39 | 0.61 | 2.55 | 0.44 | 3.44 | 0.50 |

Table 4.3 continued

| Members | *S6-06* $V_{calc}/V_{test}$ | | *AASHTO LRFD-05* $V_{calc}/V_{test}$ | | *Response 2000* $V_{calc}/V_{test}$ | | *ACI 318-08* $V_{calc}/V_{test}$ | |
|---|---|---|---|---|---|---|---|---|
| | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ |
| Non-Compliant Members – Non-Prestressed | | | | | | | | |
| 5063 | 0.61 | | 0.51 | | 0.59 | | 0.45 | |
| | 2.96 | 0.54 | 2.78 | 0.74 | 2.96 | 0.54 | 4.00 | 0.58 |
| 5053 | 0.67 | | 0.56 | | 0.62 | | 0.49 | |
| | 3.26 | 0.57 | 3.05 | 0.79 | 3.26 | 0.57 | 4.40 | 0.65 |
| 5052 | 0.66 | | 0.56 | | 0.62 | | 0.48 | |
| | 3.57 | 0.63 | 3.35 | 0.87 | 3.57 | 0.63 | 4.82 | 0.71 |
| 5051 | 0.63 | | 0.53 | | 0.60 | | 0.45 | |
| | 3.93 | 0.68 | 3.68 | 0.94 | 3.93 | 0.68 | 5.30 | 0.77 |
| N2-S | 0.98 | | 0.84 | | 0.92 | | 0.91 | |
| | 1.05 | 1.03 | 0.99 | 1.42 | 1.05 | 1.03 | 1.42 | 1.03 |
| N1-N | 0.78 | | 0.67 | | 0.74 | | 0.73 | |
| | 0.74 | 1.02 | 0.69 | 1.41 | 0.74 | 1.02 | 0.99 | 1.02 |
| P21 | 0.76 | | 0.66 | | 1.04 | | 0.71 | |
| | 1.22 | 0.90 | 1.14 | 1.24 | 1.22 | 0.90 | 1.64 | 0.90 |
| Ss2-321-3 | 1.08 | | 0.99 | | 1.12 | | 1.14 | |
| | 0.99 | 1.62 | 1.62 | 1.88 | 0.99 | 1.62 | 2.34 | 1.36 |
| Ss2-318-3 | 0.92 | | 0.81 | | 0.95 | | 0.95 | |
| | 1.73 | 1.36 | 1.39 | 1.61 | 1.73 | 1.36 | 2.00 | 1.17 |
| Ss2-313.5-3 | 0.84 | | 0.70 | | 0.86 | | 0.84 | |
| | 1.11 | 0.87 | 1.04 | 1.21 | 1.11 | 0.87 | 1.50 | 0.87 |
| Ss2-321-2 | 1.00 | | 0.84 | | 0.98 | | 0.97 | |
| | 1.73 | 1.00 | 1.62 | 1.39 | 1.73 | 1.00 | 2.34 | 1.00 |
| Ss2-318-2 | 0.98 | | 0.82 | | 0.96 | | 0.96 | |
| | 1.48 | 0.87 | 1.39 | 1.20 | 1.48 | 0.87 | 2.00 | 0.87 |
| Ss2-321-1 | 1.13 | | 0.94 | | 0.96 | | 1.12 | |
| | 1.73 | 0.69 | 1.62 | 0.95 | 1.73 | 0.69 | 2.34 | 0.69 |
| Ss2-318-1 | 0.89 | | 0.74 | | 0.91 | | 0.89 | |
| | 1.48 | 0.60 | 1.39 | 0.84 | 1.48 | 0.60 | 2.00 | 0.60 |
| Ss2-218a-2 | 0.83 | | 0.77 | | 0.87 | | 0.87 | |
| | 1.48 | 1.77 | 1.39 | 2.44 | 1.48 | 1.77 | 2.00 | 1.77 |
| Ss2-213.5-2 | 0.91 | | 0.82 | | 0.96 | | 0.93 | |
| | 1.11 | 1.31 | 1.04 | 1.82 | 1.11 | 1.31 | 1.50 | 1.32 |
| Ss2-213.5-1 | 0.97 | | 0.89 | | 1.05 | | 1.01 | |
| | 1.11 | 1.47 | 1.04 | 2.03 | 1.11 | 1.47 | 1.50 | 1.47 |
| J & R – 7 | 0.95 | | 0.83 | | 0.91 | | 0.92 | |
| | 0.73 | 1.14 | 0.69 | 1.58 | 0.73 | 1.14 | 0.99 | 1.14 |
| J & R – 8 | 1.03 | | 0.90 | | 0.99 | | 1.00 | |
| | 0.73 | 1.14 | 0.69 | 1.58 | 0.73 | 1.14 | 0.99 | 1.14 |
| BM100 | 0.73 | | 0.64 | | 0.92 | | 1.25 | |
| | 1.00 | 1.03 | 1.00 | 1.42 | 1.00 | 1.03 | 1.30 | 1.03 |
| SB 2012/6 | 0.58 | | 0.63 | | 0.98 | | 1.05 | |
| | 2.25 | 0.92 | 2.25 | 1.27 | 2.25 | 0.92 | 2.25 | 1.03 |
| SB 2003/6 | 0.72 | | 0.72 | | 0.87 | | 2.09 | |
| | 2.25 | 0.99 | 2.25 | 1.36 | 2.25 | 0.99 | 2.25 | 1.03 |
| 10T24 | 0.71 | | 0.61 | | 0.85 | | 0.85 | |
| | 1.02 | 0.91 | 1.02 | 1.26 | 1.02 | 0.91 | 1.10 | 1.09 |
| PE1 | 0.78 | | 0.68 | | 0.77 | | 0.81 | |
| | 2.14 | 1.14 | 2.00 | 1.57 | 2.14 | 1.14 | 2.89 | 1.14 |
| PE2 | 0.74 | | 0.69 | | 0.79 | | 0.85 | |
| | 2.74 | 1.46 | 2.57 | 2.02 | 2.74 | 1.46 | 3.70 | 1.46 |
| Non-Compliant Members –Prestressed | | | | | | | | |
| CH-6-240 | 0.72 | | 0.73 | | 0.62 | | 0.58 | |
| | 1.19 | 0.39 | 1.12 | 0.55 | 1.19 | 0.39 | 0.97 | 0.39 |

| Table 4.3 continued | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Members | *S6-06* $V_{calc}/V_{test}$ | | | *AASHTO LRFD-05* $V_{calc}/V_{test}$ | | | *Response 2000* $V_{calc}/V_{test}$ | | | *ACI 318-08* $V_{calc}/V_{test}$ | | |
| | $V_{calc}/V_{test}$ | $s/s_{m1}$ | $A_{v,min}/A_v$ | $V_{calc}/V_{test}$ | $s/s_{max}$ | $A_{v,min}/A_v$ | $V_{calc}/V_{test}$ | $s/s_{m1}$ | $A_{v,min}/A_v$ | $V_{calc}/V_{test}$ | $s/s_{max}$ | $A_{v,min}/A_v$ |
| Non-Compliant Members –Prestressed | | | | | | | | | | | | |
| CM-6-240 | 0.70 | 1.19 | 0.47 | 0.71 | 1.12 | 0.66 | 0.60 | 1.19 | 0.47 | 0.56 | 0.97 | 0.47 |
| CL-6-240 | 0.63 | 1.19 | 0.62 | 0.64 | 1.12 | 0.85 | 0.58 | 1.19 | 0.62 | 0.51 | 0.97 | 0.62 |
| PH-6-240 | 0.86 | 1.35 | 0.34 | 0.86 | 1.12 | 0.47 | 0.75 | 1.35 | 0.34 | 0.70 | 0.97 | 0.34 |
| PM-6-240 | 0.76 | 1.26 | 0.44 | 0.75 | 1.12 | 0.61 | 0.65 | 1.26 | 0.44 | 0.63 | 0.97 | 0.44 |
| PL-6-240 | 0.69 | 1.20 | 0.54 | 0.69 | 1.12 | 0.75 | 0.62 | 1.20 | 0.54 | 0.58 | 0.97 | 0.54 |
| NM-10-240 | 0.96 | 1.90 | 0.14 | 0.94 | 2.24 | 0.20 | 0.98 | 1.90 | 0.14 | 0.82 | 1.94 | 0.14 |
| NL-10-240 | 0.83 | 1.59 | 0.23 | 0.82 | 2.24 | 0.32 | 0.79 | 1.59 | 0.23 | 0.67 | 0.97 | 0.23 |
| NM-8-240 | 0.99 | 1.86 | 0.20 | 0.98 | 2.24 | 0.27 | 0.96 | 1.86 | 0.20 | 0.79 | 0.97 | 0.20 |
| NH-6-240 | 0.76 | 1.54 | 0.31 | 0.51 | 2.24 | 0.43 | 0.67 | 1.54 | 0.31 | 0.60 | 0.97 | 0.31 |
| NM-6-240 | 0.71 | 1.35 | 0.39 | 0.71 | 1.12 | 0.54 | 0.61 | 1.35 | 0.39 | 0.59 | 0.97 | 0.39 |
| NL-6-240 | 0.75 | 1.26 | 0.53 | 0.73 | 1.12 | 0.73 | 0.64 | 1.26 | 0.53 | 0.61 | 0.97 | 0.53 |
| CW12 | 0.83 | 1.77 | 0.16 | 0.82 | 1.93 | 0.22 | 0.92 | 1.77 | 0.16 | 0.58 | 1.48 | 0.16 |
| CW11 | 0.78 | 1.25 | 0.19 | 0.78 | 1.93 | 0.26 | 0.89 | 1.25 | 0.19 | 0.53 | 0.74 | 0.19 |
| CI12 | 1.00 | 1.55 | 0.19 | 0.99 | 1.98 | 0.26 | 1.06 | 1.55 | 0.19 | 0.56 | 0.76 | 0.19 |
| CI11 | 1.01 | 1.18 | 0.22 | 1.01 | 0.99 | 0.31 | 1.04 | 1.18 | 0.22 | 0.55 | 0.76 | 0.22 |
| P9 | 0.69 | 1.25 | 0.94 | 0.65 | 1.17 | 1.30 | 0.70 | 1.25 | 0.94 | 0.61 | 0.95 | 0.94 |
| P14 | 0.69 | 1.20 | 0.98 | 0.62 | 1.13 | 1.36 | 0.76 | 1.20 | 0.98 | 0.81 | 0.95 | 0.98 |
| P19 | 0.73 | 1.26 | 1.00 | 0.70 | 1.18 | 1.37 | 0.78 | 1.26 | 1.00 | 0.66 | 0.95 | 1.00 |
| BW.14.34 | 0.87 | 1.68 | 0.99 | 0.86 | 1.44 | 1.37 | 0.86 | 1.68 | 0.99 | 0.78 | 1.17 | 1.17 |
| Compliant Members – Non-Prestressed | | | | | | | | | | | | |
| V18-2 | 0.90 | 1.08 | 0.65 | 1.04 | 0.61 | 0.68 | 1.08 | 1.08 | 0.65 | 0.99 | 0.86 | 0.49 |
| V36-3* | 1.05 | 0.65 | 0.87 | 1.11 | 0.62 | 1.20* | 0.97 | 0.65 | 0.87 | 1.19 | 0.87 | 0.87 |
| V36-2* | 1.08 | 0.29 | 0.96 | 1.16 | 0.28 | 1.32* | 1.00 | 0.29 | 0.96 | 1.21 | 0.39 | 0.96 |
| V1* | 1.28 | 0.65 | 0.90 | 1.38 | 0.62 | 1.24* | 1.22 | 0.65 | 0.90 | 1.41 | 0.87 | 0.90 |
| V2* | 1.03 | 0.65 | 0.90 | 1.11 | 0.62 | 1.24* | 0.98 | 0.65 | 0.90 | 1.13 | 0.87 | 0.90 |
| A1 | 0.83 | 0.68 | 0.39 | 0.94 | 0.64 | 0.66 | 0.91 | 0.68 | 0.39 | 0.85 | 0.92 | 0.58 |
| A2 | 0.89 | 0.68 | 0.42 | 1.01 | 0.64 | 0.71 | 0.95 | 0.68 | 0.42 | 0.93 | 0.92 | 0.58 |

Table 4.3 continued

| Members | *S6-06* $V_{calc}/V_{test}$ | | *AASHTO LRFD-05* $V_{calc}/V_{test}$ | | *Response 2000* $V_{calc}/V_{test}$ | | *ACI 318-08* $V_{calc}/V_{test}$ | |
|---|---|---|---|---|---|---|---|---|
| | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ |
| Compliant Members – Non-Prestressed | | | | | | | | |
| B1 | 0.84 | | 0.93 | | 0.89 | | 0.82 | |
| | 0.62 | 0.27 | 0.58 | 0.45 | 0.62 | 0.27 | 0.83 | 0.40 |
| B2 | 0.96 | | 1.03 | | 0.91 | | 1.00 | |
| | 0.62 | 0.29 | 0.58 | 0.48 | 0.62 | 0.29 | 0.83 | 0.40 |
| C1 | 0.98 | | 1.01 | | 0.83 | | 0.99 | |
| | 0.68 | 0.20 | 0.64 | 0.33 | 0.68 | 0.20 | 0.92 | 0.29 |
| C2 | 1.05 | | 1.13 | | 0.98 | | 0.99 | |
| | 0.68 | 0.21 | 0.64 | 0.35 | 0.68 | 0.21 | 0.92 | 0.29 |
| P5 | 0.59 | | 0.65 | | 0.74 | | 0.65 | |
| | 0.53 | 0.37 | 0.50 | 0.51 | 0.53 | 0.37 | 0.72 | 0.37 |
| P20* | 0.61 | | 0.66 | | 0.71 | | 0.59 | |
| | 0.81 | 0.63 | 0.76 | 0.87 | 0.81 | 0.63 | 1.09 | 0.63 |
| P22* | 0.63 | | 0.69 | | 0.76 | | 0.71 | |
| | 0.81 | 0.55 | 0.76 | 0.76 | 0.81 | 0.55 | 1.09 | 0.55 |
| DBO530M* | 1.01 | | 0.94 | | 0.89 | | 1.44 | |
| | 0.50 | 0.85 | 0.50 | 1.17* | 0.50 | 0.85 | 0.65 | 0.87 |
| DB120M* | 1.10 | | 1.26 | | 1.14 | | 1.16 | |
| | 1.00 | 0.69 | 1.00 | 0.95 | 1.00 | 0.69 | 1.30* | 0.87 |
| DM140M* | 1.26 | | 1.47 | | 1.35 | | 1.45 | |
| | 0.50 | 0.92 | 0.50 | 1.28* | 0.50 | 0.92 | 0.65 | 0.92 |
| Ss2-29g-2 | 0.99 | | 1.01 | | 0.85 | | 0.84 | |
| | 0.74 | 0.58 | 0.70 | 0.80 | 0.74 | 0.58 | 1.00 | 0.85 |
| Ss2-29e-2* | 0.93 | | 0.99 | | 0.89 | | 0.90 | |
| | 0.74 | 0.99 | 0.70 | 1.37* | 0.74 | 0.99 | 1.00 | 0.99 |
| Ss2-29d-2* | 1.04 | | 1.09 | | 0.98 | | 0.95 | |
| | 0.74 | 0.80 | 0.70 | 1.11* | 0.74 | 0.80 | 1.00 | 0.85 |
| Ss2-29c-2 | 1.01 | | 1.06 | | 0.92 | | 0.90 | |
| | 0.74 | 0.71 | 0.70 | 0.99 | 0.74 | 0.71 | 1.00 | 0.85 |
| Ss2-29b-2* | 0.91 | | 0.96 | | 0.85 | | 0.86 | |
| | 0.74 | 0.94 | 0.70 | 1.30* | 0.74 | 0.94 | 1.00 | 0.94 |
| Ss2-29a-2* | 0.83 | | 0.87 | | 0.77 | | 0.77 | |
| | 0.74 | 0.89 | 0.70 | 1.23* | 0.74 | 0.89 | 1.00 | 0.89 |
| Ss2-29b-1* | 1.10 | | 1.16 | | 1.05 | | 1.03 | |
| | 0.74 | 0.97 | 0.70 | 1.35* | 0.74 | 0.97 | 1.00 | 0.97 |
| Ss2-29a-1* | 1.11 | | 1.16 | | 1.06 | | 1.04 | |
| | 0.74 | 0.99 | 0.70 | 1.37* | 0.74 | 0.99 | 1.00 | 0.99 |
| Ss2-26-1 | 0.96 | | 1.01 | | 0.96 | | 0.92 | |
| | 0.49 | 0.67 | 0.46 | 0.92 | 0.49 | 0.67 | 0.67 | 0.67 |
| 1 | 1.01 | | 1.05 | | 1.04 | | 0.87 | |
| | 0.37 | 0.47 | 0.34 | 0.65 | 0.37 | 0.47 | 0.49 | 0.47 |
| 5 | 0.96 | | 1.01 | | 1.03 | | 0.87 | |
| | 0.37 | 0.58 | 0.34 | 0.81 | 0.37 | 0.58 | 0.49 | 0.58 |
| 5A-0* | 0.77 | | 0.78 | | 0.82 | | 0.63 | |
| | 0.98 | 0.09 | 0.81 | 0.13 | 0.98 | 0.09 | 1.16* | 0.10 |
| 5B-0* | 0.75 | | 0.76 | | 0.83 | | 0.64 | |
| | 0.98 | 0.09 | 0.81 | 0.13 | 0.98 | 0.09 | 1.16* | 0.10 |
| Test 1.1* | 0.82 | | 0.78 | | 0.76 | | 0.76 | |
| | 0.87 | 0.27 | 0.81 | 0.38 | 0.87 | 0.27 | 1.17* | 0.27 |
| Test 2.1* | 0.95 | | 0.93 | | 0.87 | | 0.88 | |
| | 0.87 | 0.27 | 0.81 | 0.38 | 0.87 | 0.27 | 1.17 | 0.27 |
| Test 2.2* | 0.99 | | 0.98 | | 0.91 | | 0.92 | |
| | 0.87 | 0.27 | 0.81 | 0.38 | 0.87 | 0.27 | 1.17 | 0.27 |
| Test 2.3* | 0.96 | | 0.95 | | 0.85 | | 0.88 | |
| | 0.87 | 0.28 | 0.81 | 0.39 | 0.87 | 0.28 | 1.17 | 0.28 |

Table 4.3 continued

| Members | S6-06 $V_{calc}/V_{test}$ | | AASHTO LRFD-05 $V_{calc}/V_{test}$ | | Response 2000 $V_{calc}/V_{test}$ | | ACI 318-08 $V_{calc}/V_{test}$ | |
|---|---|---|---|---|---|---|---|---|
| | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ |
| Compliant Members – Non-Prestressed | | | | | | | | |
| T1 | 0.72 | | 0.81 | | 1.01 | | 0.49 | |
| | 0.32 | 0.03 | 0.68 | 0.09 | 0.32 | 0.03 | 0.97 | 0.07 |
| ET1 | 0.94 | | 0.99 | | 0.91 | | 0.93 | |
| | 0.54 | 0.56 | 0.51 | 0.77 | 0.54 | 0.56 | 0.73 | 0.61 |
| ET2 | 0.81 | | 0.83 | | 0.93 | | 0.70 | |
| | 0.54 | 0.28 | 0.51 | 0.39 | 0.54 | 0.28 | 0.73 | 0.31 |
| ET3* | 0.76 | | 0.76 | | 0.86 | | 0.61 | |
| | 0.65 | 0.19 | 1.02* | 0.26 | 0.65 | 0.19 | 0.73 | 0.20 |
| 2T10 | 0.98 | | 0.96 | | 1.10 | | 0.98 | |
| | 0.43 | 0.21 | 0.42 | 0.29 | 0.43 | 0.21 | 0.46 | 0.25 |
| 2T12 | 0.96 | | 0.95 | | 1.13 | | 0.94 | |
| | 0.51 | 0.25 | 0.51 | 0.35 | 0.51 | 0.25 | 0.55 | 0.30 |
| 1T18 | 0.96 | | 0.97 | | 1.12 | | 0.94 | |
| | 0.76 | 0.45 | 0.76 | 0.62 | 0.76 | 0.45 | 0.83 | 0.45 |
| Compliant Members – Prestressed | | | | | | | | |
| PL-6-160 | 0.69 | | 0.62 | | 0.62 | | 0.57 | |
| | 0.88 | 0.35 | 0.75 | 0.49 | 0.88 | 0.35 | 0.65 | 0.35 |
| NL-6-80 | 0.81 | | 0.78 | | 0.82 | | 0.68 | |
| | 0.59 | 0.17 | 0.75 | 0.24 | 0.59 | 0.17 | 0.65 | 0.17 |
| NH-6-80 | 0.88 | | 0.88 | | 0.96 | | 0.79 | |
| | 0.82 | 0.11 | 0.75 | 0.15 | 0.82 | 0.11 | 0.65 | 0.11 |
| NM-6-80 | 0.88 | | 0.86 | | 0.93 | | 0.76 | |
| | 0.76 | 0.13 | 0.75 | 0.18 | 0.76 | 0.13 | 0.65 | 0.13 |
| NL-6-160 | 0.73 | | 0.65 | | 0.66 | | 0.60 | |
| | 0.92 | 0.34 | 0.75 | 0.47 | 0.92 | 0.34 | 0.65 | 0.34 |
| NM-6-160* | 0.78 | | 0.72 | | 0.74 | | 0.63 | |
| | 1.00 | 0.26 | 1.49* | 0.36 | 1.00 | 0.26 | 0.65 | 0.26 |
| CH-6-80 | 0.75 | | 0.76 | | 0.86 | | 0.67 | |
| | 0.50 | 0.13 | 0.75 | 0.18 | 0.50 | 0.13 | 0.65 | 0.13 |
| CM-6-80 | 0.77 | | 0.78 | | 0.86 | | 0.68 | |
| | 0.45 | 0.16 | 0.37 | 0.22 | 0.45 | 0.16 | 0.65 | 0.16 |
| CL-6-80 | 0.77 | | 0.77 | | 0.81 | | 0.65 | |
| | 0.42 | 0.21 | 0.37 | 0.29 | 0.42 | 0.21 | 0.32 | 0.21 |
| CH-6-160 | 0.75 | | 0.73 | | 0.74 | | 0.61 | |
| | 0.81 | 0.27 | 0.75 | 0.37 | 0.81 | 0.27 | 0.65 | 0.27 |
| CL-6-160 | 0.71 | | 0.68 | | 0.62 | | 0.57 | |
| | 0.80 | 0.42 | 0.75 | 0.58 | 0.80 | 0.42 | 0.65 | 0.42 |
| PM-6-80 | 0.83 | | 0.82 | | 0.90 | | 0.73 | |
| | 0.59 | 0.14 | 0.75 | 0.19 | 0.59 | 0.14 | 0.65 | 0.14 |
| PM-6-160* | 0.77 | | 0.72 | | 0.75 | | 0.64 | |
| | 0.94 | 0.27 | 1.49* | 0.38 | 0.94 | 0.27 | 0.65 | 0.27 |
| PL-6-80 | 0.82 | | 0.79 | | 0.83 | | 0.70 | |
| | 0.53 | 0.18 | 0.75 | 0.25 | 0.53 | 0.18 | 0.65 | 0.18 |
| A1-00-1.5R-N | 1.17 | | 1.15 | | 1.34 | | 1.27 | |
| | 0.30 | 0.07 | 0.45 | 0.10 | 0.30 | 0.07 | 0.45 | 0.07 |
| B0-00-R-S | 0.72 | | 0.73 | | 0.77 | | 0.73 | |
| | 0.74 | 0.23 | 0.74 | 0.32 | 0.74 | 0.23 | 0.74 | 0.23 |
| B0-00-R-N | 0.96 | | 1.03 | | 1.12 | | 1.07 | |
| | 0.41 | 0.11 | 0.68 | 1.15 | 0.41 | 0.11 | 0.68 | 0.11 |
| A1-00-M-N | 1.18 | | 1.17 | | 1.15 | | 1.12 | |
| | 0.80 | 0.24 | 0.76 | 0.33 | 0.80 | 0.24 | 0.76 | 0.24 |
| A1-00-R_N | 0.99 | | 1.04 | | 1.15 | | 1.11 | |
| | 0.42 | 0.10 | 0.68 | 0.14 | 0.42 | 0.10 | 0.68 | 0.10 |

Table 4.3 continued

| Members | S6-06 $V_{calc}/V_{test}$ | | AASHTO LRFD-05 $V_{calc}/V_{test}$ | | Response 2000 $V_{calc}/V_{test}$ | | ACI 318-08 $V_{calc}/V_{test}$ | |
|---|---|---|---|---|---|---|---|---|
| | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ |
| Compliant Members – Prestressed | | | | | | | | |
| A1-00-M-S | 0.88 | | 0.90 | | 0.97 | | 0.90 | |
| | 0.76 | 0.24 | 0.76 | 0.33 | 0.76 | 0.24 | 0.76 | 0.24 |
| A1-00-0.5R-N | 1.03 | | 1.03 | | 1.00 | | 0.99 | |
| | 0.73 | 0.21 | 0.68 | 0.29 | 0.73 | 0.21 | 0.68 | 0.21 |
| A1-00-0.5R-S | 0.87 | | 0.88 | | 0.91 | | 0.71 | |
| | 0.74 | 0.23 | 0.74 | 0.32 | 0.74 | 0.23 | 0.74 | 0.23 |
| 2A-3 | 0.73 | | 0.71 | | 0.80 | | 0.71 | |
| | 0.92 | 0.10 | 0.81 | 0.15 | 0.92 | 0.10 | 0.70 | 0.11 |
| 2B-3 | 0.74 | | 0.72 | | 0.81 | | 0.72 | |
| | 0.88 | 0.10 | 0.88 | 0.14 | 0.88 | 0.10 | 0.70 | 0.10 |
| 3A-2 | 0.73 | | 0.74 | | 0.82 | | 0.76 | |
| | 0.95 | 0.10 | 0.81 | 0.13 | 0.95 | 0.10 | 0.70 | 0.10 |
| 3B-2 | 0.80 | | 0.80 | | 0.87 | | 0.81 | |
| | 0.98 | 0.09 | 0.81 | 0.13 | 0.98 | 0.09 | 0.70 | 0.11 |
| 4A-1 | 0.72 | | 0.73 | | 0.81 | | 0.78 | |
| | 0.80 | 0.10 | 0.81 | 0.14 | 0.80 | 0.10 | 0.70 | 0.10 |
| 4B-1 | 0.75 | | 0.76 | | 0.84 | | 0.81 | |
| | 0.89 | 0.09 | 0.81 | 0.13 | 0.89 | 0.09 | 0.70 | 0.10 |
| P4 | 0.66 | | 0.71 | | 0.86 | | 1.02 | |
| | 0.53 | 0.35 | 0.49 | 0.49 | 0.53 | 0.35 | 0.43 | 0.35 |
| P8 | 0.63 | | 0.62 | | 0.61 | | 0.65 | |
| | 0.83 | 0.59 | 0.78 | 0.81 | 0.83 | 0.59 | 0.63 | 0.59 |
| P13 | 0.62 | | 0.64 | | 0.71 | | 0.87 | |
| | 0.80 | 0.57 | 0.75 | 0.78 | 0.80 | 0.57 | 0.63 | 0.57 |
| P18 | 0.68 | | 0.67 | | 0.67 | | 0.70 | |
| | 0.84 | 0.60 | 0.79 | 0.83 | 0.84 | 0.60 | 0.63 | 0.60 |
| P24 | 0.68 | | 0.72 | | 0.86 | | 0.95 | |
| | 0.54 | 0.37 | 0.50 | 0.51 | 0.54 | 0.37 | 0.43 | 0.37 |
| P25 | 0.71 | | 0.73 | | 1.06 | | 0.89 | |
| | 0.80 | 0.55 | 0.75 | 0.76 | 0.80 | 0.55 | 0.63 | 0.55 |
| P26 | 0.70 | | 0.72 | | 0.82 | | 0.76 | |
| | 0.56 | 0.38 | 0.53 | 0.53 | 0.56 | 0.38 | 0.43 | 0.38 |
| P27 | 0.75 | | 0.77 | | 0.70 | | 0.75 | |
| | 0.84 | 0.56 | 0.79 | 0.78 | 0.84 | 0.56 | 0.63 | 0.56 |
| P28 | 0.64 | | 0.66 | | 0.73 | | 0.68 | |
| | 0.55 | 0.37 | 0.52 | 0.51 | 0.55 | 0.37 | 0.42 | 0.37 |
| P29 | 0.68 | | 0.70 | | 0.93 | | 0.65 | |
| | 0.83 | 0.57 | 0.78 | 0.79 | 0.83 | 0.57 | 0.63 | 0.57 |
| P49 | 0.68 | | 0.68 | | 0.75 | | 0.70 | |
| | 0.55 | 0.34 | 0.52 | 0.48 | 0.55 | 0.34 | 0.43 | 0.34 |
| P50 | 0.72 | | 0.74 | | 0.87 | | 0.75 | |
| | 0.57 | 0.19 | 0.51 | 0.26 | 0.57 | 0.19 | 0.42 | 0.19 |
| Non-Prestressed Members without Stirrups | | | | | | | | |
| YB2000/0 | 1.01 | | 1.09 | | 1.04 | | 2.16 | |
| | - | - | - | - | - | - | - | - |
| N1-S | 1.12 | | 1.00 | | 1.10 | | 0.99 | |
| | - | - | - | - | - | - | - | - |
| P41 | 0.75 | | 0.66 | | 0.77 | | 0.66 | |
| | - | - | - | - | - | - | - | - |
| SD-1 | 1.03 | | 0.92 | | 1.01 | | 1.14 | |
| | - | - | - | - | - | - | - | - |
| SD-2 | 1.12 | | 0.97 | | 1.10 | | 1.25 | |
| | - | - | - | - | - | - | - | - |

| Table 4.3 continued | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Members | *S6-06* $V_{calc}/V_{test}$ | | *AASHTO LRFD-05* $V_{calc}/V_{test}$ | | *Response 2000* $V_{calc}/V_{test}$ | | *ACI 318-08* $V_{calc}/V_{test}$ | |
| | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ |
| Non-Prestressed Members without Stirrups | | | | | | | | |
| A & S – 8 | 1.04 | | 0.97 | | 0.90 | | 1.66 | |
| | - | - | - | - | - | - | - | - |
| A & S – 9 | 1.01 | | 0.96 | | 0.90 | | 1.46 | |
| | - | - | - | - | - | - | - | - |
| A & S – 10 | 1.00 | | 0.95 | | 0.89 | | 1.46 | |
| | - | - | - | - | - | - | - | - |
| A & S – 11 | 1.09 | | 1.03 | | 0.99 | | 1.58 | |
| | - | - | - | - | - | - | - | - |
| A & S – 12 | 1.07 | | 0.98 | | 0.95 | | 1.34 | |
| | - | - | - | - | - | - | - | - |
| A & S – 16 | 1.03 | | 0.99 | | 0.94 | | 1.76 | |
| | - | - | - | - | - | - | - | - |
| A & S – 17 | 1.14 | | 1.08 | | 1.04 | | 1.92 | |
| | - | - | - | - | - | - | - | - |
| DB120 | 0.89 | | 0.85 | | 0.93 | | 1.21 | |
| | - | - | - | - | - | - | - | - |
| DB130 | 1.00 | | 0.95 | | 1.03 | | 1.44 | |
| | - | - | - | - | - | - | - | - |
| DB140 | 1.09 | | 1.04 | | 1.11 | | 1.62 | |
| | - | - | - | - | - | - | - | - |
| DB230 | 0.86 | | 0.84 | | 0.88 | | 1.00 | |
| | - | - | - | - | - | - | - | - |
| DB0.530 | 0.89 | | 0.84 | | 0.95 | | 1.62 | |
| | - | - | - | - | - | - | - | - |
| B100 | 0.86 | | 0.82 | | 0.88 | | 1.26 | |
| | - | - | - | - | - | - | - | - |
| B100L | 0.89 | | 0.85 | | 0.91 | | 1.32 | |
| | - | - | - | - | - | - | - | - |
| BN100 | 0.92 | | 0.88 | | 1.03 | | 1.49 | |
| | - | - | - | - | - | - | - | - |
| AW1 | 0.79 | | 0.72 | | 0.81 | | 1.11 | |
| | - | - | - | - | - | - | - | - |
| AW4 | 0.80 | | 0.72 | | 0.83 | | 0.89 | |
| | - | - | - | - | - | - | - | - |
| AW8 | 0.73 | | 0.65 | | 0.76 | | 0.81 | |
| | - | - | - | - | - | - | - | - |
| Prestressed Members without Stirrups | | | | | | | | |
| CI8 | 0.81 | | 0.67 | | 1.14 | | 0.34 | |
| | - | - | - | - | - | - | - | - |
| CW8 | 0.63 | | 0.50 | | 0.53 | | 0.42 | |
| | - | - | - | - | - | - | - | - |
| P12 | 0.80 | | 0.74 | | 0.88 | | 0.90 | |
| | - | - | - | - | - | - | - | - |
| P16 | 0.66 | | 0.63 | | 0.69 | | 0.50 | |
| | - | - | - | - | - | - | - | - |
| P17 | 0.66 | | 0.64 | | 0.72 | | 0.52 | |
| | - | - | - | - | - | - | - | - |
| P10 | 0.59 | | 0.60 | | 0.68 | | 0.52 | |
| | - | - | - | - | - | - | - | - |
| P11 | 0.60 | | 0.57 | | 0.65 | | 0.83 | |
| | - | - | - | - | - | - | - | - |
| P15 | 0.60 | | 0.61 | | 0.68 | | 0.54 | |
| | - | - | - | - | - | - | - | - |

| Table 4.3 continued | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Members | *S6-06* $V_{calc}/V_{test}$ | | *AASHTO LRFD-05* $V_{calc}/V_{test}$ | | *Response 2000* $V_{calc}/V_{test}$ | | *ACI 318-08* $V_{calc}/V_{test}$ | |
| | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ | $s/s_{m1}$ | $A_{v,min}/A_v$ | $s/s_{max}$ | $A_{v,min}/A_v$ |
| Prestressed Members without Stirrups | | | | | | | | |
| P47 | 0.61 | | 0.61 | | 0.68 | | 0.54 | |
| | - | - | - | - | - | - | - | - |
| P48 | 0.67 | | 0.67 | | 0.77 | | 0.59 | |
| | - | - | - | - | - | - | - | - |

Summaries of the average $V_{calc}/V_{test}$ ratios and coefficients of variation for each sectional shear evaluation method are provided in each method's corresponding section, along with the average demerit points per member.

Table 4.4a –S6-06 Predictions Adhering to $s_{m2}$ Limit – 49 Non-Compliant Members

| Member | *S6-06* $V_{calc}/V_{test}$ | $s/s_{m2}$ | $A_{v,min}/A_v$ |
|---|---|---|---|
| Non-Compliant Members – Non-Prestressed | | | |
| YB2000/9 | 0.54 | 3.38 | 0.94 |
| YB2000/6 | 0.48 | 1.69 | 1.12 |
| YB2000/4 | 0.74 | 0.74 | 1.10 |
| 5084 | 0.40 | 2.39 | 0.44 |
| 5053 | 0.46 | 3.06 | 0.57 |
| 5052 | 0.46 | 3.35 | 0.63 |
| 5051 | 0.45 | 3.68 | 0.68 |
| 5063 | 0.41 | 2.78 | 0.57 |
| N2-S | 0.98 | 0.99 | 1.03 |
| N1-N | 0.78 | 0.69 | 1.02 |
| P21 | 0.57 | 1.14 | 0.90 |
| Ss2-321-3 | 0.95 | 1.62 | 1.36 |
| Ss2-318-3 | 0.77 | 1.39 | 1.17 |
| Ss2-313.5-3 | 0.63 | 1.05 | 0.87 |
| Ss2-321-2 | 0.77 | 1.62 | 1.00 |
| Ss2-318-2 | 0.73 | 1.39 | 0.87 |
| Ss2-321-1 | 0.79 | 1.62 | 0.69 |
| Ss2-318-1 | 0.60 | 1.39 | 0.60 |
| Ss2-218a-2 | 0.78 | 1.39 | 1.77 |
| Ss2-213.5-2 | 0.79 | 1.05 | 1.32 |
| Ss2-313.5-1 | 0.88 | 1.05 | 1.47 |
| J & R - 7 | 0.95 | 0.69 | 1.14 |
| J & R - 8 | 1.03 | 0.69 | 1.14 |
| BM100 | 0.73 | 0.90 | 1.03 |
| SB 2012/6 | 0.44 | 1.69 | 0.92 |
| SB 2003/6 | 0.57 | 1.67 | 0.99 |
| 10T24 | 0.71 | 0.76 | 0.91 |
| PE1 | 0.64 | 2.00 | 1.14 |
| PE2 | 0.67 | 2.57 | 1.46 |
| Non-Compliant Members –Prestressed | | | |
| CH-6-240 | 0.49 | 1.12 | 0.39 |
| CM-6-240 | 0.51 | 1.12 | 0.47 |
| CL-6-240 | 0.49 | 1.12 | 0.62 |
| PH-6-240 | 0.60 | 1.12 | 0.34 |
| PM-6-240 | 0.55 | 1.12 | 0.44 |
| PL-6-240 | 0.48 | 1.12 | 0.51 |
| NM-10-240 | 0.50 | 1.12 | 0.14 |
| NL-10-240 | 0.51 | 1.12 | 0.23 |

| Member | *S6-06* $V_{calc}/V_{test}$ | $s/s_{m2}$ | $A_{v,min}/A_v$ |
|---|---|---|---|
| Non-Compliant Members –Prestressed | | | |
| NM-8-240 | 0.58 | 1.12 | 0.20 |
| NH-6-240 | 0.52 | 1.12 | 0.31 |
| NM-6-240 | 0.50 | 1.12 | 0.39 |
| NL-6-240 | 0.59 | 1.12 | 0.53 |
| CW12 | 0.91 | 1.00 | 0.16 |
| CW11 | 0.81 | 1.00 | 0.19 |
| CI12 | 1.14 | 1.00 | 0.19 |
| CI11 | 1.06 | 1.00 | 0.22 |
| P9 | 0.61 | 1.17 | 0.94 |
| P14 | 0.57 | 1.13 | 0.98 |
| P19 | 0.65 | 1.18 | 1.00 |
| BW.14.34 | 0.74 | 1.44 | 0.99 |

Table 4.4b –Full Stirrup Contribution Regardless of Stirrup Area– 29 Member Data Set

| Member | *S6-06* $V_{calc}/V_{test}$ | $A_{v,min}/A_v$ |
|---|---|---|
| Non-Compliant Members – Non-Prestressed | | |
| YB2000/9 | 0.70 | 0.94 |
| YB2000/6 | 0.60 | 1.12 |
| YB2000/4 | 0.49 | 1.08 |
| 5084 | 0.65 | 0.44 |
| 5053 | 0.65 | 0.52 |
| 5052 | 0.66 | 0.63 |
| 5051 | 0.63 | 0.68 |
| 5063 | 0.61 | 0.54 |
| N2-S | 0.99 | 1.03 |
| N1-N | 0.79 | 1.02 |
| P21 | 0.76 | 0.90 |
| Ss2-321-3 | 1.16 | 1.36 |
| Ss2-318-3 | 0.96 | 1.17 |
| Ss2-313.5-3 | 0.84 | 0.87 |
| Ss2-321-2 | 1.00 | 1.00 |
| Ss2-318-2 | 0.98 | 0.87 |
| Ss2-321-1 | 1.13 | 0.69 |
| Ss2-318-1 | 0.89 | 0.60 |
| Ss2-218a-2 | 0.91 | 1.77 |
| Ss2-213.5-2 | 0.97 | 1.32 |
| Ss2-313.5-1 | 1.05 | 1.47 |
| J & R - 7 | 0.98 | 1.14 |
| J & R - 8 | 1.07 | 1.14 |
| BM100 | 0.74 | 1.03 |
| SB 2012/6 | 0.58 | 0.92 |
| SB 2003/6 | 0.72 | 0.99 |
| 10T24 | 0.71 | 0.91 |
| PE1 | 0.82 | 1.14 |
| PE2 | 0.81 | 1.46 |

## 4.5 Canadian Highway Bridge Design Code S6-06

Section 4.5 assesses the agreement between predicted and tested shear capacities calculated using S6-06 Section 8 for members with non-compliant stirrup spacing and area details identified for this study using the member selection criteria discussed in

Section 4.3. Forty nine test specimens bearing these non-compliant stirrup details were evaluated using the process discussed in Section 3.2 and are compared against shear capacity evaluations of eighty-one members complying with S6-06 Section 14 minimum stirrup requirements and thirty-three members without stirrups. Variations in parameters known to affect shear capacity of concrete members (concrete strength $f'_c$, shear span to depth ratio $a/d$, longitudinal reinforcing ratio $\rho$ and member shape) are also studied in this section to assess any influence they have on predicted to tested shear capacity ratios which are calculated using S6-06 Section 8. The effect of these parameters on tested shear capacity is discussed in Section 2.5.

As discussed in Section 2.2, S6-06 Clause 14.14.1.6.2 provides in lieu requirements for maximum stirrup spacing ($s_{m2}$) and minimum stirrup area (Eqn 2.2), which are specific to evaluation of existing structures. This section examines the influence that adhering to the $s_{m2}$ stirrup spacing limit and to the linear interpolation for stirrup area, discussed in Section 3.2 (Eqn 3.12), has on predicted to tested shear capacity ratios calculated using provisions in S6-06.

### 4.5.1 Evaluation of Members with Stirrups not Complying with Minimum Stirrup Requirements using S6-06 Sectional Shear Provisions

<u>Evaluation of Members with Non-Compliant Stirrup Details using Shear Provisions in S6-06 Section 14 – Adhering to Stirrup Spacing Limit $s_{m2}$</u>

The forty-nine members not complying with S6-06 Section 14 stirrup spacing and area requirements were evaluated in this study by adhering to the $s_{m2}$ stirrup spacing limit in order to assess whether this limit is appropriate for predicting the shear capacity of non-compliant members. Thus the predicted shear capacity for any member with a stirrup spacing exceeding its corresponding $s_{m2}$ limit was calculated assuming zero stirrup contribution ($V_s = 0$). Figure 4.3a shows the relationship $V_{calc}/V_{test}$ *vs.* $s/s_{m2}$ for the twenty-nine non-prestressed members with non-compliant stirrup details evaluated in this study by using no stirrup contribution when the stirrup spacing $s$ exceeded the stirrup spacing limit $s_{m2}$. Figure 4.3b shows the same relationship for the same members evaluated using full stirrup contribution to shear capacity when the $s_{m2}$ limit was exceeded. Thus stirrups in a member violating the $s_{m2}$ spacing requirement were still considered fully effective, but the assumed crack spacing $s_z$ would change from 300 mm to $d_v$, as discussed in Section 3.2. Figure 4.4a and Figure 4.4b show the same information respectively for the twenty prestressed members with non-compliant stirrup details. The solid line in the following figures represents exact shear predictions, while the two dashed lines illustrate the range of 'appropriate' predictions as defined in Section 4.2.

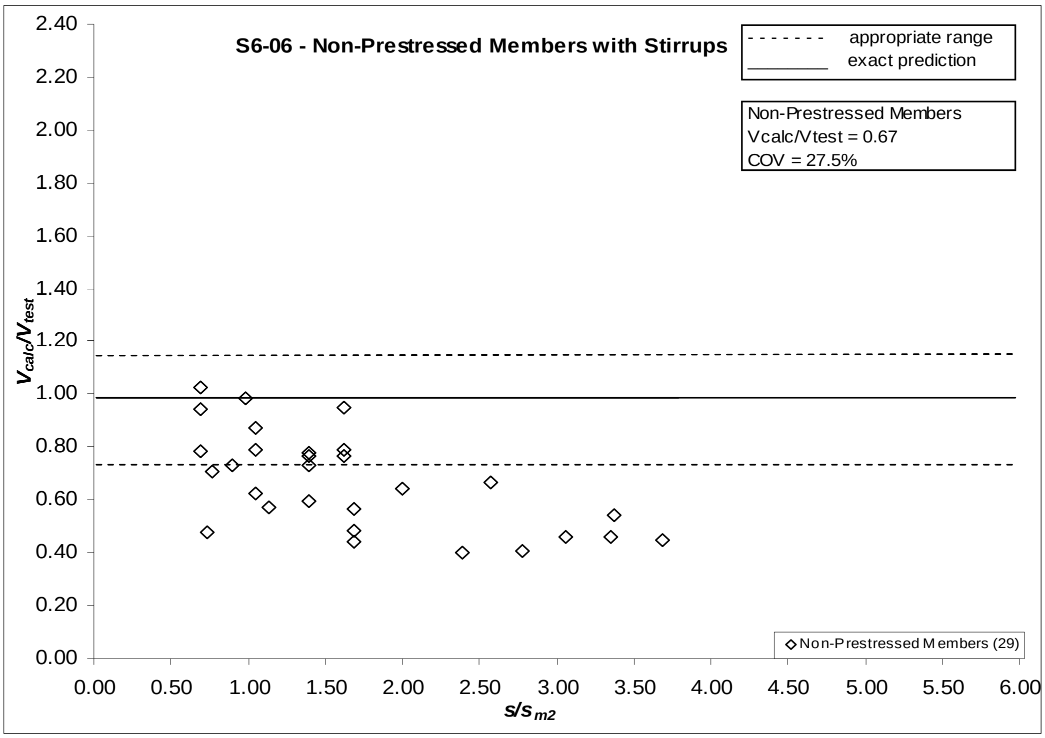



Figure 4.3a - $V_{calc}/V_{test}$ *vs.* $s/s_{m2}$ for the 29 Non-Compliant Non-Prestressed Members Evaluated using S6-06

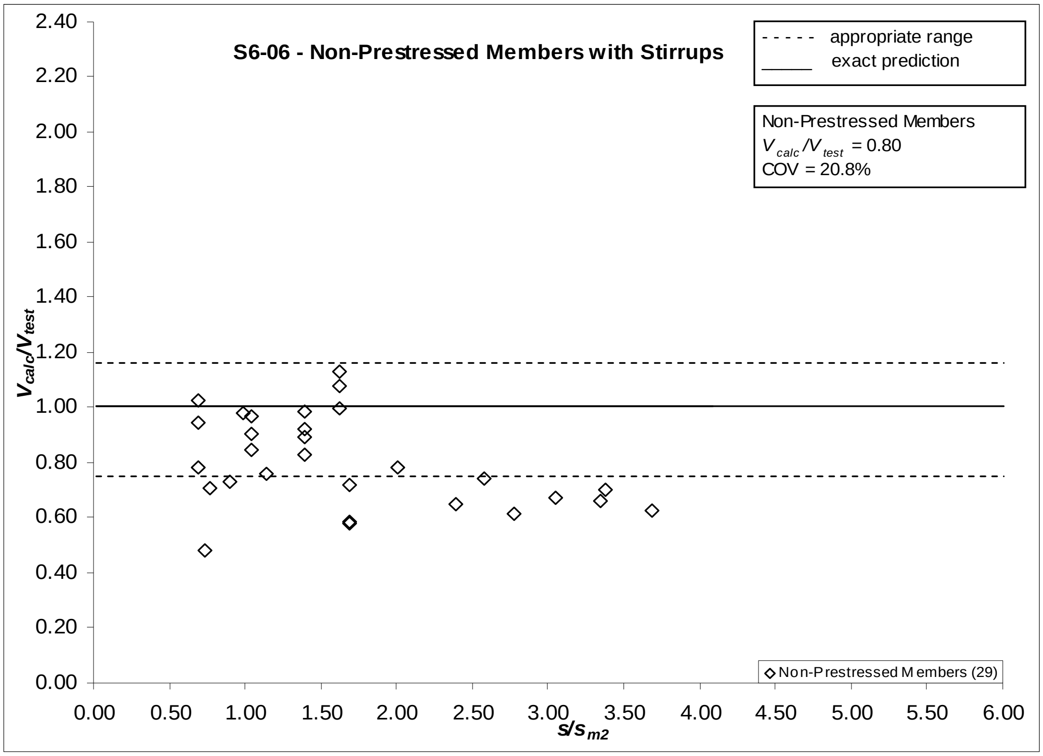



Figure 4.3b - $V_{calc}/V_{test}$ *vs.* $s/s_{m2}$ for the 29 Non-Compliant Non-Prestressed Members Evaluated using S6-06 (using full stirrup contribution)

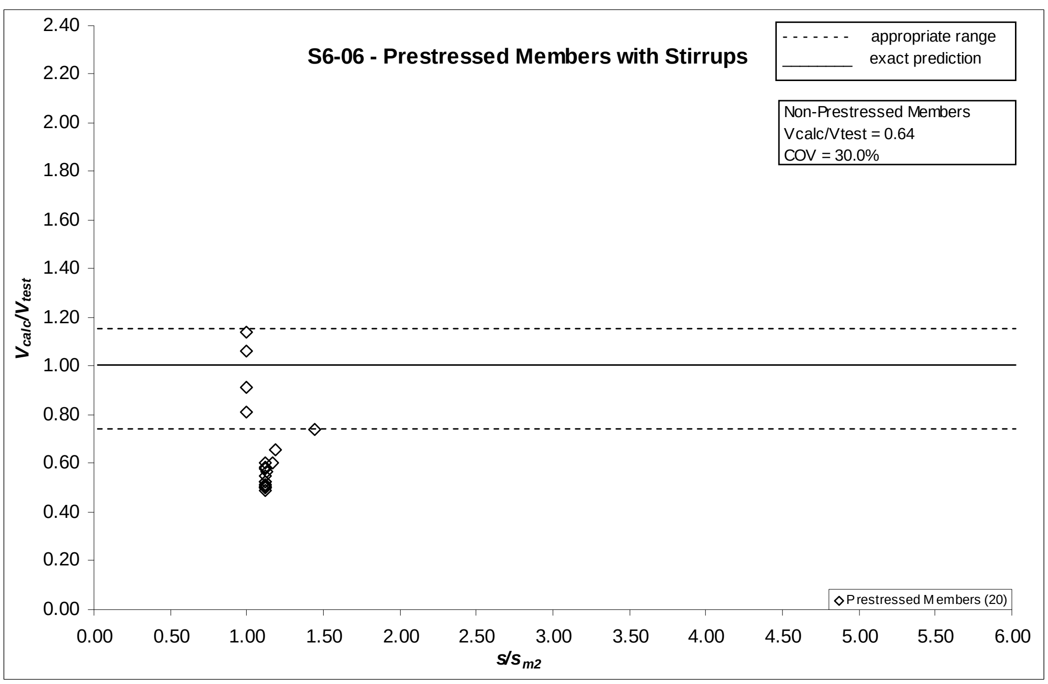



Figure 4.4a - $V_{calc}/V_{test}$ *vs.* $s/s_{m2}$ for the 20 Non-Compliant Prestressed Members Evaluated using S6-06

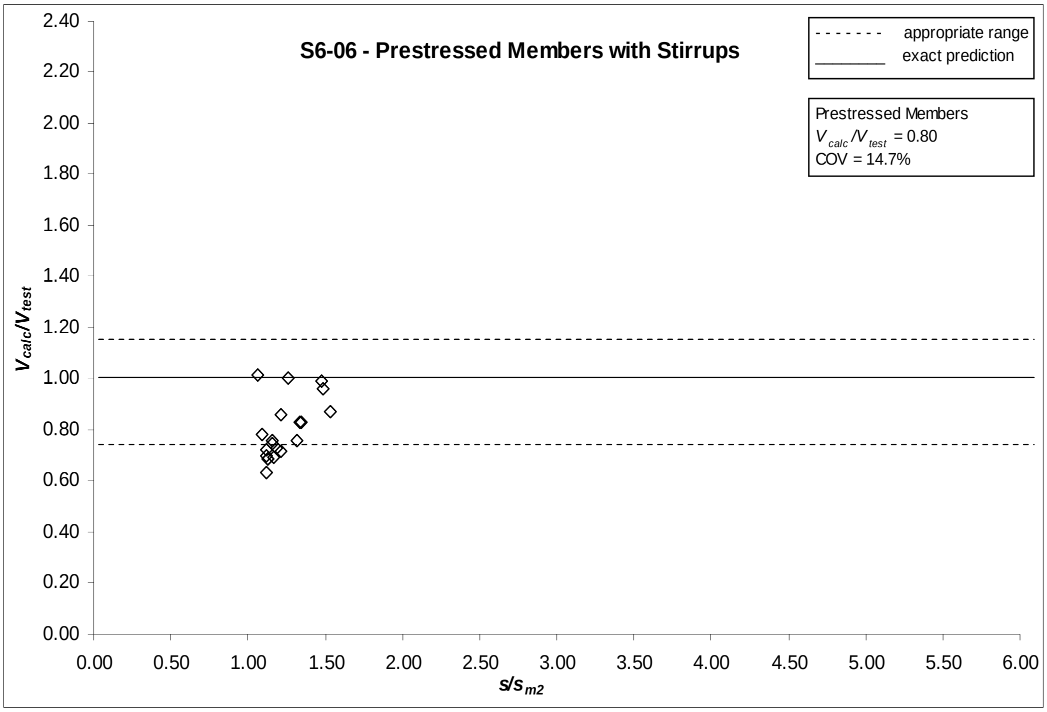



Figure 4.4b - $V_{calc}/V_{test}$ *vs.* $s/s_{m2}$ for the 20 Non-Compliant Prestressed Members Evaluated using S6-06 (using full stirrup contribution)

A comparison of Figure 4.3a and Figure 4.3b indicates that adhering to the stirrup spacing limit $s_{m2}$ causes shear capacity predictions calculated using the provisions in S6-06 Section 8 to become overly conservative for non-prestressed non-compliant members. $V_{calc}/V_{test}$ ratios were calculated as low as 0.40 (Specimen 5084) when adhering to the stirrup spacing limit $s_{m2}$, indicating predicted shear capacities for members that were nearly three times less than their corresponding tested shear capacity. This level of conservatism has a negative impact on the ability of engineers to make appropriate, economical decisions regarding the shear capacity of existing members. For example, as discussed in Section 1.1, excessively low predictions of shear strength may indicate that unnecessary strengthening is required. A comparison of Figure 4.4a and 4.4b provides similar results for non-compliant prestressed members evaluated by adhering to the $s_{m2}$ limit. $V_{calc}/V_{test}$ ratios as low as 0.49 were calculated, indicating predicted shear capacities which were less than half of their corresponding tested shear capacity when adhering to the $s_{m2}$ limit.

The other issue which arose more frequently when adhering to the $s_{m2}$ limit was the sudden discontinuity in shear strength predictions, which can make converging on the predicted shear capacity impossible. As discussed in Section 3.2, successive iterations of shear capacity can result in a case in which the predicted shear capacity at iteration *n* never equals the prediction from iteration *n-1*. This situation needs to be avoided in order to provide an unambiguous evaluation of the shear capacity for concrete members. The method used in this study to determine the shear capacities of members demonstrating the issue of non-convergence is discussed in Section 3.2 while a sample calculation demonstrating the non-convergent shear capacity prediction issue is provided in Appendix C.

Table 4.5 provides results from statistical analysis and the average demerit points per member calculated using the model proposed in Section 4.2 for the forty-nine non-compliant members evaluated using the shear provisions in S6-06, and adhering to the stirrup spacing limit $s_{m2}$. Table 4.6 provides the equivalent data for evaluation of the same members excluding the stirrup spacing $s_{m2}$ limit.

Table 4.5 - Results using S6-06 Sectional Shear Provisions ($s_{m2}$ limit considered)

| Test Group (number) | $V_{calc}/V_{test}$ Mean | COV (%) | Demerit Points / Member |
|---|---|---|---|
| Non-Prestressed Members With Stirrups (29) | 0.67 | 27.5 | 12.03 |
| Prestressed Members with Stirrups (20) | 0.64 | 30.0 | 14.40 |

Table 4.6 - Results using S6-06 Sectional Shear Provisions ($s_{m2}$ limit ignored)

| Test Group (number) | $V_{calc}/V_{test}$ Mean | COV (%) | Demerit Points / Member |
|---|---|---|---|
| Non-Prestressed Members With Stirrups (29) | 0.80 | 20.8 | 6.84 |
| Prestressed Members with Stirrups (20) | 0.80 | 14.7 | 6.02 |

Comparing results in Table 4.5 to those in Table 4.6, it can be seen that the mean $V_{calc}/V_{test}$ ratio decreases by 16.3% (0.67 compared to 0.80) for non-prestressed members and by 20.0% (0.64 compared to 0.80) for prestressed members when the stirrup contribution to shear capacity was ignored for members where $s > s_{m2}$. The corresponding coefficients of variation increase substantially, by 32.2% (27.5% compared to 20.8%) for non-prestressed members and by 104.1% (30.0% compared to

14.7%) for prestressed members when the stirrup contribution to shear capacity was ignored for members with $s > s_{m2}$. The average demerit points per member increases by 75.9% (12.03 compared to 6.84) and by 139.2% (14.40 compared to 6.02) for the non-prestressed and prestressed members respectively when shear capacity has been predicted by adhering to the stirrup spacing limit $s_{m2}$. This is due to the significant decrease in the $V_{calc}/V_{test}$ ratios which is a result of neglecting the stirrup contribution to shear capacity when the $s_{m2}$ limit was exceeded.

The considerable decreases in $V_{calc}/V_{test}$ ratios and increases in COV values and average demerit points per member combined with the increased likelihood of encountering the non-convergence issue discussed in Section 3.2 indicates that the $s_{m2}$ limit, used to define effectiveness of stirrups, is not appropriate for evaluating concrete members for one-way shear capacity. Thus the $s_{m2}$ stirrup spacing limit was not considered for subsequent evaluations of shear capacity in this study. The only stirrup spacing limit considered was the $s_{m1}$ limit and full stirrup contribution was used for evaluating shear capacity in this study regardless of whether or not the stirrup spacing $s$ was compliant with $s_{m1}$. Eqn. (3.12) was used to interpolate effective stirrup contribution based on how the actual stirrup area compared to the required stirrup area. This process of shear capacity evaluation is justified based on the similar agreement between predicted and tested shear capacities discussed in this section with those from other studies, which are discussed in Section 3.2.

Evaluation of Members with Non-Compliant Stirrup Details using Sectional Shear Provisions in S6-06 – Adhering to Stirrup Area Provisions in Clause 14.14.1.6.2

To assess the suitability of the stirrup area provisions in Section 14 of S6-06, the twenty-nine non-compliant non-prestressed members identified for this study were evaluated using two different approaches. The first approach adhered to the provisions in S6-06 Clause 14.14.1.6.2 and used the linear interpolation for stirrup contribution to shear capacity discussed in Section 3.2 (Eqn 3.12), while the second approach assumed full stirrup contribution to shear capacity regardless of how the required stirrup area compared to the actual stirrup area (Eqn 3.12 always equal to 1.00). Prestressed members were evaluated as well but their results are not included in this discussion because all prestressed members identified for this study have stirrup areas complying with S6-06 Section 14 requirements. It should be noted that the concrete contribution to shear capacity ($V_c$) is not directly affected by the interpolation for stirrup contribution proposed in Eqn (3.12). However since members having stirrup areas less than required by Eqn. (2.1) are non-compliant, their assumed crack spacing $s_z$ is taken as $d_v$ in accordance with S6-06 Clause 8.9.3.7. This in turn affects the $V_c$ term as discussed in Section 3.2. In both approaches the stirrups were assumed to be fully effective regardless of how the required stirrup spacing compared to the actual stirrup spacing, but the assumed crack spacing varied depending on how $s$ compared to $s_{m1}$, as discussed in Section 3.2.

Figure 4.5a provides the relationship $V_{calc}/V_{test}$ *vs.* $A_{v,min}/A_v$ for the twenty-nine non-compliant non-prestressed members evaluated in this study using the procedure discussed in Section 3.2. Figure 4.5b provides the relationship $V_{calc}/V_{test}$ *vs.* $A_{v,min}/A_v$ for the same members assuming full stirrup contribution regardless of provided stirrup area. Table 4.7 provides the statistical results and average demerit points per member from the analyses using the two different approaches for considering non-compliant stirrup area details. The demerit points were assigned according to the model proposed in Section 4.2.

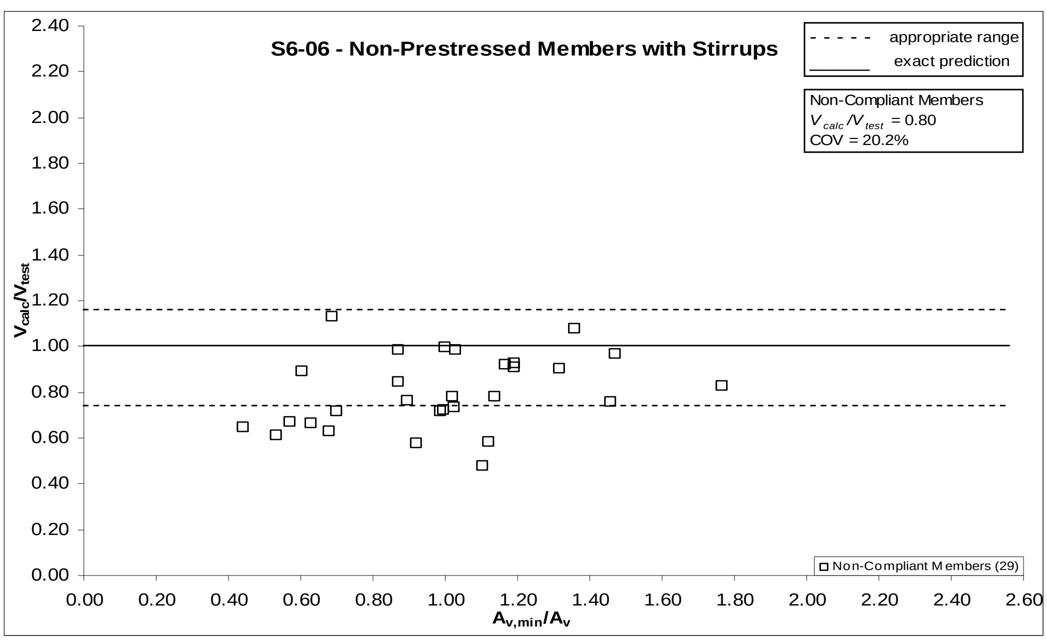



Figure 4.5a - $V_{calc}/V_{test}$ *vs.* $A_{v,min}/A_v$ for 29 Non-Prestressed Members adhering to Stirrup Area Provisions in S6-06 Section 14

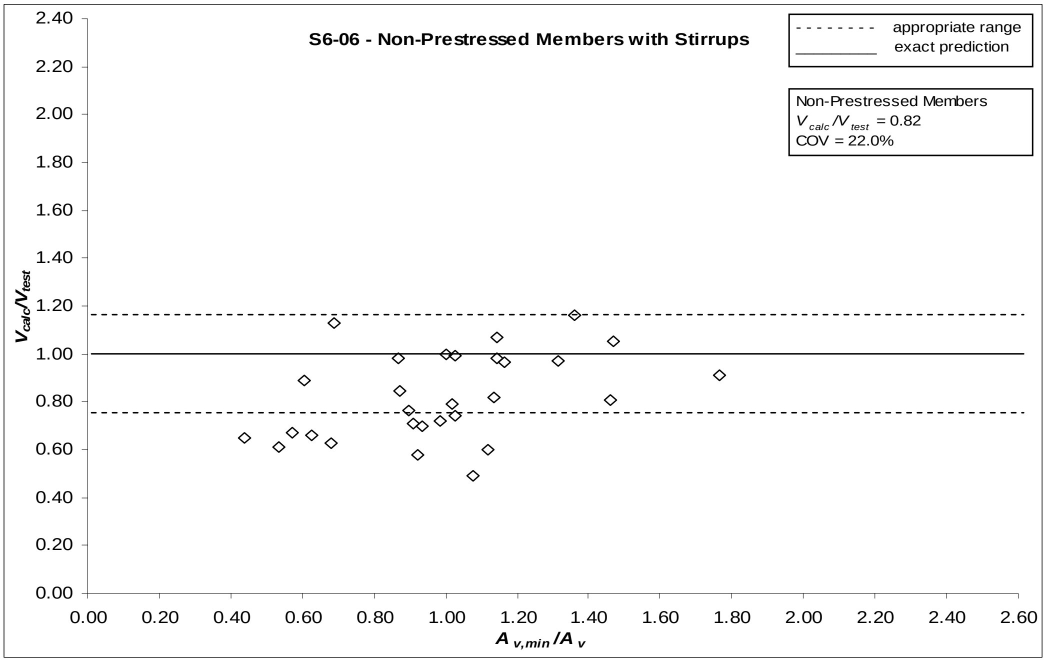



Figure 4.5b - $V_{calc}/V_{test}$ *vs.* $A_{v,min}/A_v$ for 29 Non-Prestressed Members Assuming Full Stirrup Contribution Evaluated using S6-06 Sectional Shear Method

Table 4.7 - Results for Members with Non-Compliant Stirrup Area Details using S6-06

| Test Group (number) | $V_{calc}/V_{test}$ Mean | $V_{calc}/V_{test}$ C.O.V (%) | Demerit Points / Member |
|---|---|---|---|
| Non-Prestressed Members With Stirrups (29) – Reduced as per S6-06 Section 14 | 0.80 | 20.8 | 6.84 |
| Non-Prestressed Members With Stirrups (29) – Full Contribution S6-06 | 0.82 | 22.0 | 5.43 |

A comparison of Figures 4.5a and 4.5b indicates that predicting shear capacity using the approach for interpolating stirrup contribution $V_s$ proposed in Section 3.2, or predicting shear capacity assuming fully effective stirrups regardless of the $A_{v,min}/A_v$ ratio, does not significantly impact the agreement between predicted and tested shear capacities. No skewed behavior with respect to the average $V_{calc}/V_{test}$ value is apparent in either figure as the stirrup area ratio varies.

Based on the twenty-nine non-compliant non-prestressed members evaluated in this study, Table 4.7 indicates that adhering to Eqn. (3.12) for interpolating stirrup contribution to shear capacity decreases the average $V_{calc}/V_{test}$ ratio and corresponding COV by 2.4% (0.80 compared to 0.82) and 5.5% (20.8% compared to 22.0%)

respectively, relative to evaluation using full stirrup capacity regardless of how the required stirrup area compared to the actual stirrup area. There is a 26.0% increase (6.84 compared to 5.43) in average assigned demerit points per member to evaluations made when adhering to Clause 14.14.1.6.2; this is due to predictions typically becoming more conservative. Based on the 'appropriate' agreement between predicted and tested shear capacities, Eqn (3.12), proposed in Section 3.2 for interpolating between the stirrup area requirements in S6-06 Section 14, is appropriate for evaluating the shear capacity of members with non-compliant stirrup area and is included in further calculations in this study.

Evaluation of Non-Prestressed Members not Complying with Stirrup Spacing and Area Details using S6-06 Sectional Shear Provisions ($s_{m2}$ limit excluded)

The twenty-nine non-prestressed members with stirrup details not complying with S6-06 Section 14 spacing and area requirements that were identified for this study were evaluated to identify any trends which may present for these non-compliant issues. Forty-one non-prestressed members meeting S6-06 Section 14 minimum stirrup requirements were evaluated to facilitate a comparison of the agreement between predicted and tested shear capacities with the non-compliant members. Figure 4.6 provides the relationship $V_{calc}/V_{test}$ vs. $s/s_{m1}$ for the seventy non-prestressed members with stirrups, while Figure 4.7 provides the relationship $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for the same members. The solid line in Figures 4.6 and 4.7 represents exact shear predictions, while the two dashed lines define the range of 'appropriate predictions' as given in Section 4.2. Members are classified as non-compliant if either of their stirrup detail ratios, $s/s_{m1}$ or $A_{v,min}/A_v$, is greater than 1.00. Table 4.8 distributes the member predictions into the ranges given in Table 4.1, and provides the average demerit points per member for the full data set of non-prestressed members with stirrups, as well as the compliant and non-compliant data categories.

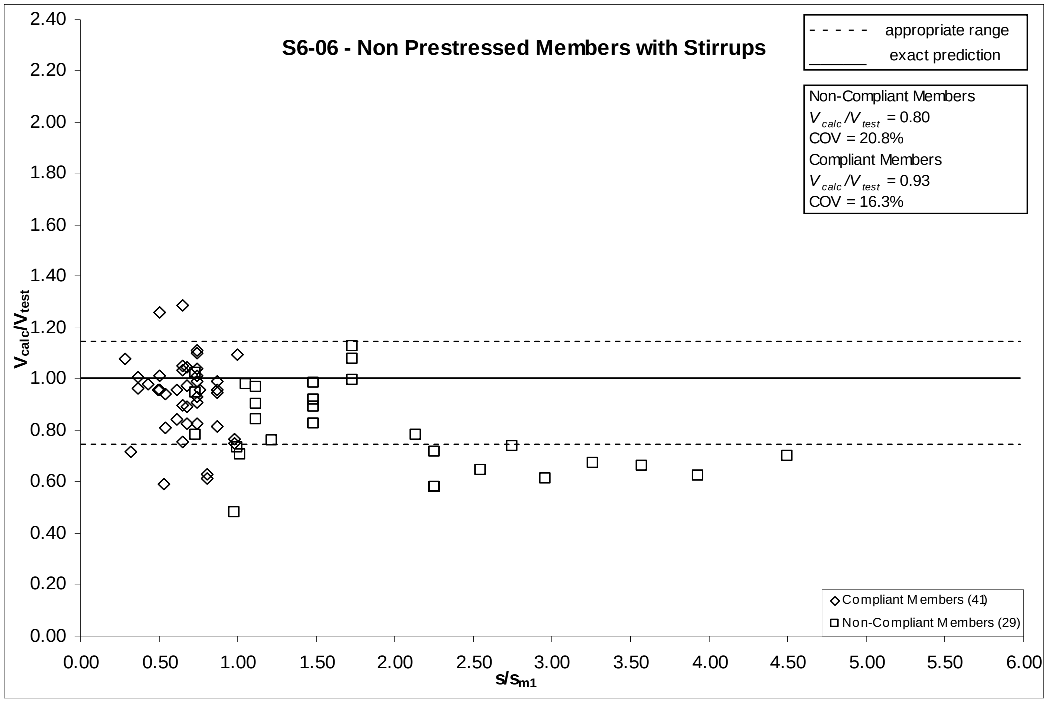



Figure 4.6 - $V_{calc}/V_{test}$ *vs.* $s/s_{m1}$ for 70 Non-Prestressed Members with Stirrups Evaluated using S6-06

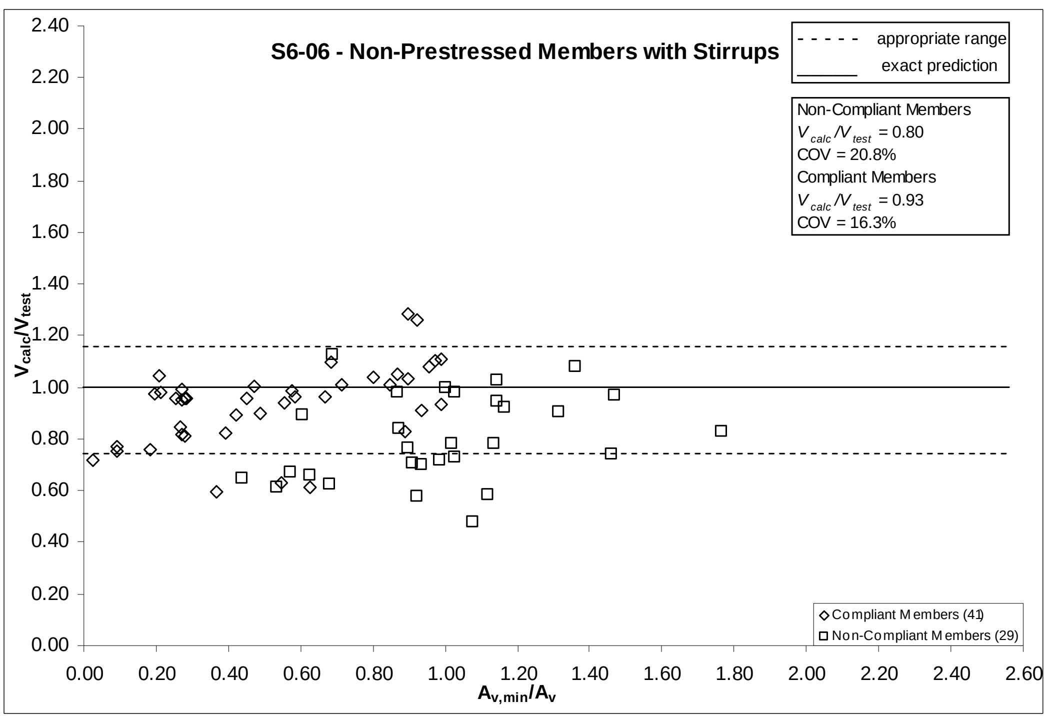



Figure 4.7 - $V_{calc}/V_{test}$ *vs.* $A_{v,min}/A_v$ for 70 Non-Prestressed Members with Stirrups Evaluated using S6-06

Table 4.8 – S6-06 Non-Prestressed Members - $V_{calc}/V_{test}$ Ranges and Demerit Points

| **S6-06 - Non-Prestressed Members with Stirrups** | **Full Data Set** | **Non-Compliant** | **Compliant** | **Demerits - Full Data Set** | **Demerits - Non-Compliant** | **Demerits - Compliant** |
|---|---|---|---|---|---|---|
| Total | 70 | 29 | 41 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 1<br>1.4% | 1<br>3.4% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 16<br>22.9% | 12<br>41.4% | 4<br>9.8% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 51<br>72.9% | 16<br>55.2% | 35<br>82.9% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 2<br>2.9% | 0<br>0.0% | 2<br>7.3% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 359 | 198 | 160 |
| Average Demerits/Members | | | | 5.12 | 6.84 | 3.91 |

65% of predictions for the 878 non-prestressed members evaluated by Kim (2004) using the sectional shear method in A23.3-04 were calculated to be in the ‘appropriate’ range. This percentage of predictions in the appropriate range is consistent with the percentage of ‘appropriate’ predictions for the twenty-nine non-compliant non-prestressed members evaluated in this study, as inferred from Table 4.8. A description of Kim’s members and the differences between the sectional shear methods in S6-06 and A23.3-04 are discussed in Section 3.2. The twelve non-compliant non-prestressed members evaluated in this study which are calculated to have $V_{calc}/V_{test}$ ratios in the ‘conservative’ range are all members with overall heights greater than 800 mm or members with compression flanges while the single non-compliant non-prestressed member presenting a ‘very conservative’ shear capacity prediction had an overall member height of 2000 mm. The influence of section height on predictions of shear capacity for members with non-compliant stirrup spacing and area details is a result of the assumption of diagonal crack spacing used by S6-06. This diagonal crack spacing assumption is discussed in Section 3.2 and studied further in Section 5.2. None of the non-compliant members evaluated using provisions in S6-06 Section 8 exhibit unsafe predictions of shear capacity, defined in this study as $V_{calc}/V_{test}$ ratios greater than 1.15. It should be noted from Table 4.8 that the compliant members evaluated in this study demonstrate a considerably higher percentage of shear capacity predictions in the 'appropriate' range than do the non-compliant members (82.9% compared to 55.2%). The four non-prestressed compliant members presenting with shear capacity predictions in the 'conservative' range are all sections fabricated with compression flanges, further indicating that this geometric detail has an influence on $V_{calc}/V_{test}$ ratios. Evaluation in this study indicates that variations in section height do not have a considerable influence on the agreement between predicted and tested shear capacities for members meeting S6-06 Section 14 minimum transverse reinforcement requirements. The two compliant non-prestressed members having $V_{calc}/V_{test}$ ratios in the 'low safety' range appear to deviate from the rest of the shear capacity predictions for compliant members. Member parameters and testing details related to these deviations are discussed later in this section.

The non-compliant non-prestressed category of members is allotted an average of 74.9% more demerit points per member (6.84 compared to 3.91) than are the members in the compliant category using the Demerit Point model proposed in Section 4.2. The increase in demerit points per non-compliant member is a result of the non-compliant members presenting with more conservative predictions of shear capacity. A significant cause for the increase in conservatism for the non-compliant members is the diagonal crack spacing assumption employed by the sectional shear method in S6-06. Average demerit points per member have been calculated in this study for the 106 members with stirrups evaluated in Lubell (2006). These members are calculated to have an average of 4.80 demerit points per member using the Demerit Point model described in Section 4.2. This value of allotted demerit points per member is consistent with the forty-one non-prestressed compliant members evaluated in this study.

A decrease in the value of $V_{calc}/V_{test}$ ratios as the $s/s_{m1}$ ratio increases is apparent in Figure 4.6, suggesting that members with $s/s_{m1}$ ratios greater than 2.00 produce more conservative predictions of shear capacity than do members with $s/s_{m1}$ ratios less than 2.00. However, this is believed to result from the following two causes. 1) The majority of members with $s/s_{m1}$ ratios greater than 2.00 are flanged members, which are known to present with conservative predictions of shear capacity (Placas and Regan, 1971; Giaccio et al., 2002). These flanged members carry higher shear forces with thinner webs which results in larger normalized shear stresses and thus tighter stirrup spacing requirements. This is discussed in Section 3.2. Therefore the smaller $V_{calc}/V_{test}$ values at $s/s_{m1}$ ratios greater than 2.00 are more likely a result of these members having compression flanges than the fact that actual stirrup spacing exceeds allowable stirrup spacing. 2) The rectangular members with $s/s_{m1}$ ratios greater than 2.00 have large overall member heights ($h$ > 800 mm) and are non-compliant with respect to S6-06 Section 14 stirrup spacing and area requirements. In accordance with S6-06 Clause 8.9.3.6 the diagonal crack spacing of these members is taken to be equal to their corresponding shear depth $d_v$. As discussed in Section 3.2, the shear capacity attributed to both the concrete and the stirrups decreases as the predicted diagonal crack spacing increases. Predictions of shear capacity for these taller members are calculated to have $V_{calc}/V_{test}$ ratios as low as 0.49 (YB2000/4) meaning that greater than 50% of a member's shear capacity is ignored. This indicates that the assumption for diagonal crack spacing used by S6-06 Section 8 can be punitive to shear predictions of tall members which do not comply with minimum transverse reinforcement requirements. These punitive predictions of shear capacity can lead engineers to make uneconomical decisions, such as requiring unnecessary strengthening or replacement of existing structures. Figure 4.7 shows that variations in the stirrup area ratio $A_{v,min}/A_v$ have no appreciable influence on $V_{calc}/V_{test}$ ratios, which indicates that stirrup area is appropriately accommodated by the S6-06 sectional shear method.

No trends in $V_{calc}/V_{test}$ ratios with respect to the average $V_{calc}/V_{test}$ ratio are well defined in either Figure 4.6 or 4.7 for the compliant members, further indicating that stirrup spacing and area details are appropriately accounted for in the S6-06 sectional shear method. As discussed earlier in this section two compliant members were determined to have $V_{calc}/V_{test}$ ratios in the 'low safety' range, which deviates from the rest of the compliant non-prestressed category of members. Both members had rectangular cross sections. The first member (V1) had a height of 914 mm, had stirrups spaced at 370 mm and had 1.00% longitudinal reinforcement. The member was loaded with a shear span-to-depth ratio $a/d$ of 3.00 and failed at 48% of its predicted flexural capacity. Literature describing the test (Frosch, 2000) indicates that the critical shear crack crossed only one stirrup, which likely led to the 'low safety' prediction. The diagonal compression field angle calculated using Eqn. (3.7) indicates that the diagonal crack should have crossed 2.77 stirrups (at least 2 stirrups engaged). The second member (DB140M) had a height of 1000 mm, stirrups spaced at 300 mm, and a longitudinal reinforcement ratio of 1.01%. The member was loaded with a shear span-to-depth ratio $a/d$ of 2.92 and failed at approximately 40% of its predicted flexural capacity. The crack pattern shown in the test documentation (Angelakos, 1999) indicates more than 2 effective stirrups being crossed by the critical shear crack. Discussion in Angelakos (1999) states that a stirrup fractured during the testing, which is believed to have caused the poor prediction.

Evaluation of Prestressed Members not Complying with Stirrup Spacing and Area Details using S6-06 Sectional Shear Provisions

The twenty prestressed members not complying with S6-06 Section 14 stirrup spacing requirements were evaluated to assess the influence that variations in the stirrup detail ratios $s/s_{m1}$ and $A_{v,min}/A_v$ have on the agreement between predicted and tested shear capacities calculated using S6-06 Section 8 provisions. All prestressed members evaluated in this study comply with S6-06 Section 14 stirrup area requirements. Forty prestressed members complying with S6-06 Section 14 minimum stirrup requirements were evaluated to facilitate a comparison of the agreement between predicted-to-tested shear capacities with the non-compliant members. Figure 4.8 provides the relationship $V_{calc}/V_{test}$ vs. $s/s_{m1}$ for the sixty prestressed members with stirrups, while Figure 4.9 provides the relationship $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for the same members. The solid line in the Figures 4.8 and 4.9 represent exact shear predictions, while the two dashed lines define the range of ‘appropriate predictions’ as defined in Section 4.2. Table 4.9 distributes the member predictions into the ranges provided in Table 4.1, and provides the average demerit points per member for the full data set of prestressed members with stirrups, as well as for the compliant and non-compliant data subsets.

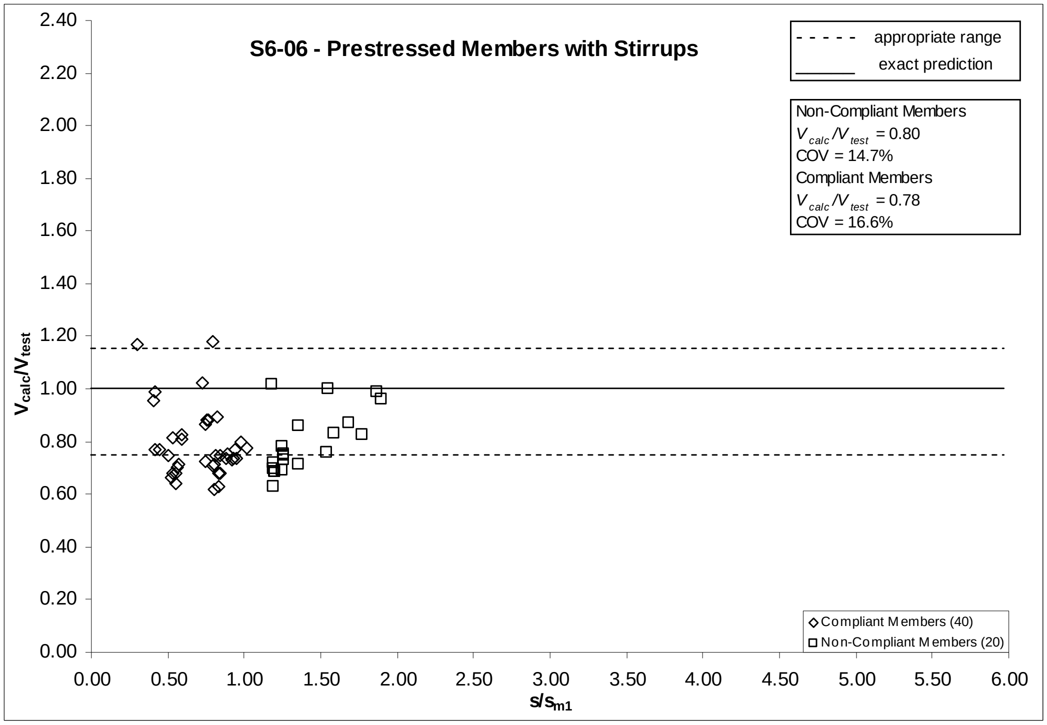



Figure 4.8 – $V_{calc}/V_{test}$ *vs.* $s/s_{m1}$ for 60 Prestressed Members with Stirrups Evaluated using S6-06

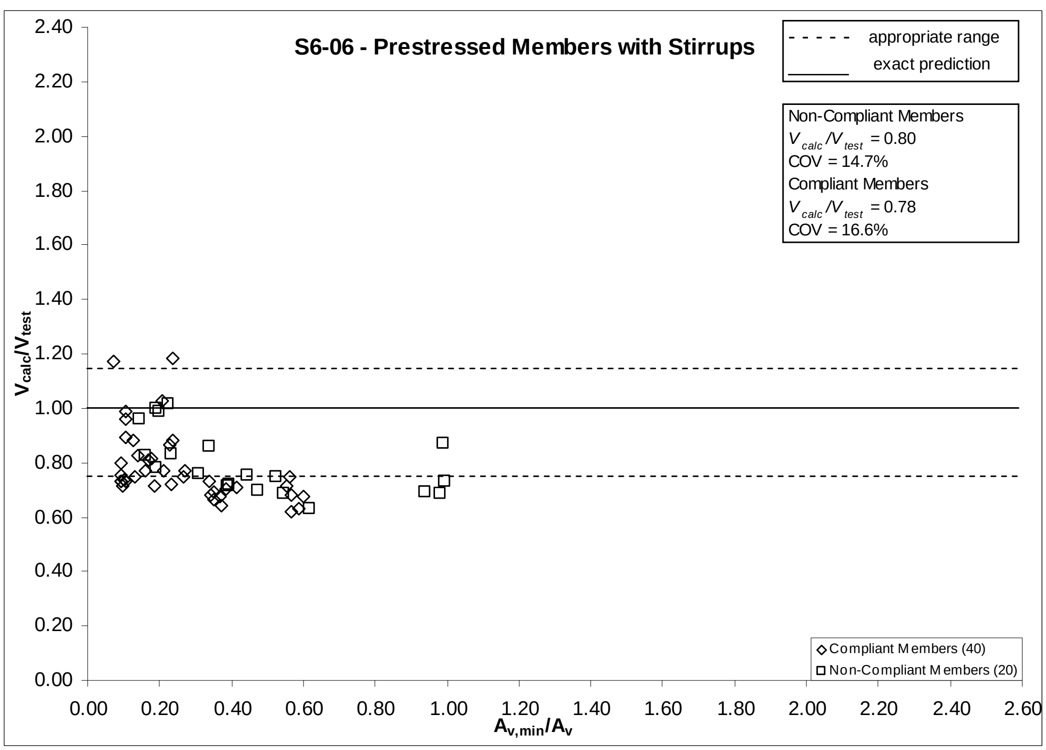



Figure 4.9 - $V_{calc}/V_{test}$ *vs.* $A_{v,min}/A_v$ for 60 Prestressed Members with Stirrups Evaluated using S6-06

Table 4.9 – S6-06 Prestressed Members - $V_{calc}/V_{test}$ Ranges and Demerit Points

| **S6-06 - Prestressed Members with Stirrups** | **Full Data Set** | **Non-Compliant** | **Compliant** | **Demerits - Full Data Set** | **Demerits - Non-Compliant** | **Demerits - Compliant** |
|---|---|---|---|---|---|---|
| Total | 60 | 20 | 40 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 31<br>51.7% | 9<br>45.0% | 22<br>55.0% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 27<br>45.0% | 11<br>55.0% | 16<br>40.0% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 2<br>3.3% | 0<br>0.0% | 2<br>5.0% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 417 | 120 | 300 |
| Average Demerits/Members | | | | 6.94 | 6.02 | 7.49 |

Evaluation of 485 prestressed members by Kim (2004) using the sectional shear method in A23.3-04 presents with 37.7% of predictions in the 'appropriate' range, which compares well with the percentage of 'appropriate' predictions for the twenty non-compliant prestressed members evaluated in this study, as shown in Table 4.9. A description of Kim's members and the differences between the sectional shear methods in S6-06 and A23.3-04 are discussed in Section 3.2. A comparison of non-compliant members in Table 4.8 and Table 4.9 indicates that the percentage of 'appropriate' shear capacity predictions for non-prestressed members compares well to the percentage of 'appropriate' shear capacity predictions for prestressed members (55.2% compared to 55.0%). The percentage of non-compliant prestressed members evaluated in this study with predicted to tested shear capacity ratios in the 'conservative' range compares well to 'conservative' predictions for the forty compliant members evaluated in this study as shown in Table 4.9, and to 'conservative' predictions for prestressed members in Kim (52.6%). None of the non-compliant prestressed member predictions of shear capacity calculated using S6-06 shear provisions were found to be unsafe. A comparison of compliant members in Tables 4.8 and 4.9 indicates that the percentage of 'appropriate' shear capacity predictions for non-prestressed members does not compare well to the percentage of 'appropriate' shear capacity predictions for prestressed members (82.9% compared to 40.0%). The majority of compliant prestressed members evaluated in this study are determined to have $V_{calc}/V_{test}$ in the 'conservative' range, which indicates that the sectional shear method in S6-06 has a tendency to calculate more conservative predictions of shear capacity for prestressed members than for non-prestressed members. The observation in this study that prestressed members calculate more conservative predictions of shear capacity than non-prestressed members is consistent with results in Kim (2004). Most of the prestressed members evaluated in this study are small, flanged sections. These two details are known to result in lower $V_{calc}/V_{test}$ predictions (Collins, 2001; Placas and Regan, 1971; Giaccio et al, 2002). The two compliant prestressed members determined to have $V_{calc}/V_{test}$ ratios in the 'low safety' range appear to deviate from the rest of the shear capacity predictions for compliant members. Member parameters and testing details are discussed later in this section to demonstrate that these deviations are not related to inappropriate member selection.

Non-compliant prestressed members are allotted 19.6% fewer average demerit points per member (6.02 compared to 7.49) than are the compliant prestressed members. A possible explanation as to why this difference is smaller than for the non-prestressed members (74.9%) is the crack spacing assumption used by sectional shear method in S6-06. Prestressed members evaluated in this study are typically small and as such both compliant and non-compliant members had an assumed crack spacing of approximately 300 mm. Non-prestressed members evaluated in this study have larger variations in section height, and therefore the assumed diagonal crack spacing of these members is more variable. Discussion related to the affect of diagonal crack spacing on shear capacity predictions is found in Section 3.2.

Figure 4.8 presents with an increase in $V_{calc}/V_{test}$ ratios as the $s/s_{m1}$ ratio increases from 1.00 to 2.00. This increase in $V_{calc}/V_{test}$ ratios suggests that as prestressed members become more non-compliant with respect to stirrup spacing (the actual stirrup spacing becomes progressively larger than the maximum allowable stirrup spacing), predictions of shear capacity become less acceptable because the decisions made based on evaluations would be unsafe. This perceived behavior is based on a small data set however. None of the non-compliant prestressed member predictions vary significantly from the rest of that member category (eg. no outliers). Figure 4.9 demonstrates a decrease in $V_{calc}/V_{test}$ ratios for the non-compliant prestressed members as their $A_{v,min}/A_v$ ratio increases. This perceived behavior suggests that shear capacity predictions of prestressed members have a tendency to ignore a greater portion of their shear capacity as the actual stirrup area decreases with respect to the required stirrup area (eg. becomes more lightly reinforced with respect to transverse reinforcement). Other studies showing this behavior were not found during a literature review because there has been limited similar work on this topic using the S6-06 shear provisions. However, Bentz (2000) demonstrated comparable behavior for prestressed members tested by McGregor (1960) and evaluated using Response 2000. The behavior exhibited by the prestressed members in Bentz (2000) is relevant to evaluation using the sectional shear method in S6-06 Section 8 because both sectional methods are based on the relationships of the Modified Compression Field Theory (Vecchio and Collins, 1986).

Figure 4.8 does not indicate any specific skews in $V_{calc}/V_{test}$ values as the stirrup spacing ratio varies. Figure 4.9 demonstrates a decrease in $V_{calc}/V_{test}$ ratios for compliant members as the stirrup area ratio $A_{v,min}/A_v$ increases. This perceived behavior is similar to the behavior demonstrated by the non-compliant prestressed members in Figure 4.9. Two compliant prestressed members in this study have $V_{calc}/V_{test}$ ratios in the 'low safety' range, which deviates from the rest of the compliant prestressed data category. Both members were tested by Shahaway and Batchelor (1996). The members' cross sections were identical, with heights of 1118 mm, stirrups spaced at 135 mm and a longitudinal reinforcement ratio of 0.35%. The girders were loaded with *a/d* ratios of 2.60 and failed at 43% of their flexural capacity. No discussion was provided in the test documentation to indicate why these members failed prior to reaching their corresponding predicted shear capacity.

### 4.5.2 Evaluation of Members without Transverse Reinforcement using the Sectional Shear Provisions in S6-06

Based on the criteria for member selection discussed in Section 4.3, thirty-three members without stirrups were identified for evaluation in this study. The results are used to assess the agreement between predicted and tested shear capacities calculated using the sectional shear method in S6-06 Section 8 for members fabricated with no transverse reinforcement. The sectional shear method in S6-06 assumes that the diagonal cracks in members without stirrups will be spaced equal to the shear depth $d_v$ of the member (Bentz and Collins, 2006). This crack spacing assumption is discussed in Section 3.2. Figure 4.10 shows the relationship $V_{calc}/V_{test}$ vs. $d$ for the thirty-three members. The solid line represents the condition in which the predicted shear capacity is equal to the tested shear capacity while the upper and lower dashed lines define the boundaries that are considered 'appropriate' predictions in this study. Table 4.10 provides the quantity and percentage of predictions falling in the various statistical ranges (as provided in Table 4.1), as well as the average demerit points per member corresponding to the shear capacity predictions.

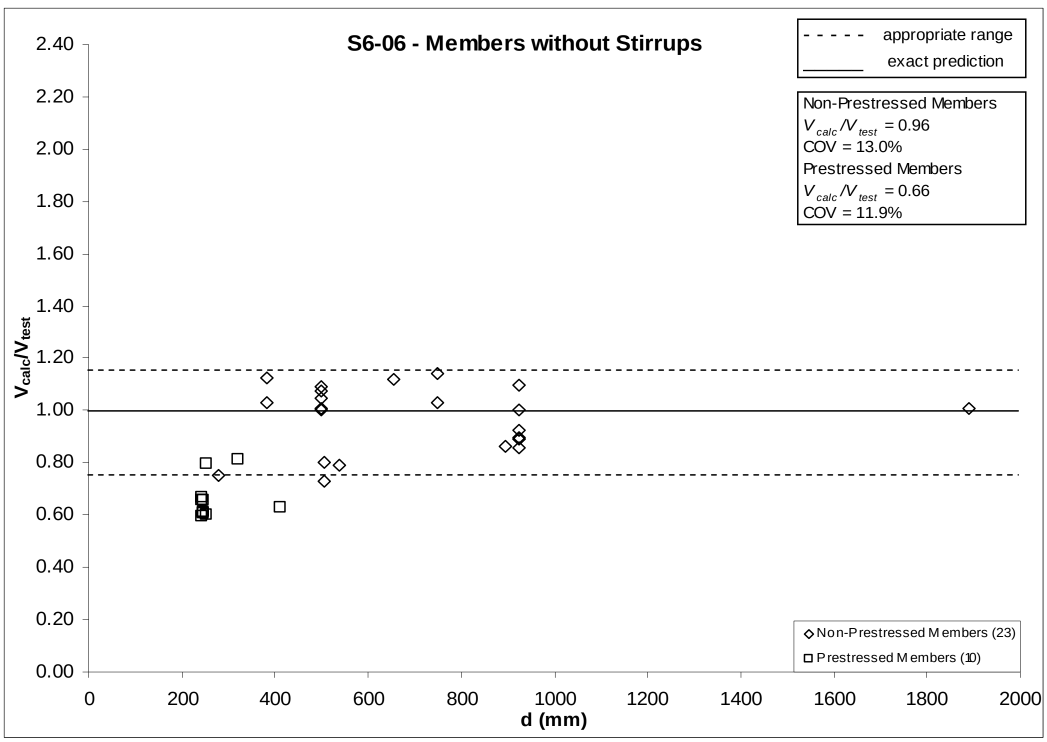



Figure 4.10 - $V_{calc}/V_{test}$ *vs. d* for 33 Members without Stirrups Evaluated Evaluated using S6-06

Table 4.10 – S6-06 Members without Stirrups - $V_{calc}/V_{test}$ Ranges and Demerit Points

| **S6-06 - Members without Stirrups** | **Full Data Set** | **Non-Prestressed** | **Prestressed** | **Demerits - Full Data Set** | **Non-Prestressed** | **Prestressed** |
|---|---|---|---|---|---|---|
| Total | 33 | 23 | 10 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 9<br>27.3% | 1<br>4.3% | 8<br>80.0% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 24<br>72.7% | 22<br>95.7% | 2<br>20.0% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 187 | 68 | 118 |
| Average Demerits/Members | | | | 5.65 | 2.96 | 11.84 |

Table 4.10 indicates a better agreement between predicted and tested shear capacities calculated using the sectional shear method in S6-06 Section 8 for non-prestressed members without stirrups than for prestressed members without stirrups. This is a result of the prestressed members presenting with considerably more conservative predictions of shear capacity than do the non-prestressed members. This observation is consistent with predictions of shear capacity for members without stirrups performed by Kim (2004).

Non-prestressed members are allotted 2.96 demerit points per member, which compares favorably to the prestressed members which are allotted 11.84 demerit points per member. This increase in average demerit points is a result of the increased conservatism related to S6-06 shear capacity predictions of prestressed members.

The absence of any specific skews of $V_{calc}/V_{test}$ ratios as the depth varies in Figure 4.10 indicates that member height is appropriately considered by the sectional shear method in S6-06 Section 8. The highest $V_{calc}/V_{test}$ ratio calculated for the non-prestressed members was 1.14, which is within the range deemed ‘acceptable’ in this study. This prediction (Specimen A & S – 17) is for a 750 mm deep slab with 0.42% longitudinal reinforcement, sectional details which are typical of members encountered in service.

### 4.5.3 Parametric Sensitivity Analysis of Shear Predictions Made using the Shear Provisions in S6-06

Section 4.5.3 assesses the influence that variations of the parameters discussed in Section 2.5 have on the agreement between predicted to tested shear capacities calculated using the sectional shear method in S6-06. As described in Section 2.5, variations in concrete strength $f'_c$, shear span to depth ratio $a/d$, longitudinal reinforcement ratio $\rho$, and member shape (flanged members vs. rectangular members) are known to affect the shear capacity of concrete members. These parameters are studied against the forty-nine non-compliant members evaluated in this study to assess whether they are appropriately accounted for by the S6-06 sectional shear method. The eighty-one compliant members and thirty-three members without stirrups are included to provide a comparison of $V_{calc}/V_{test}$ ratios with the non-compliant members.

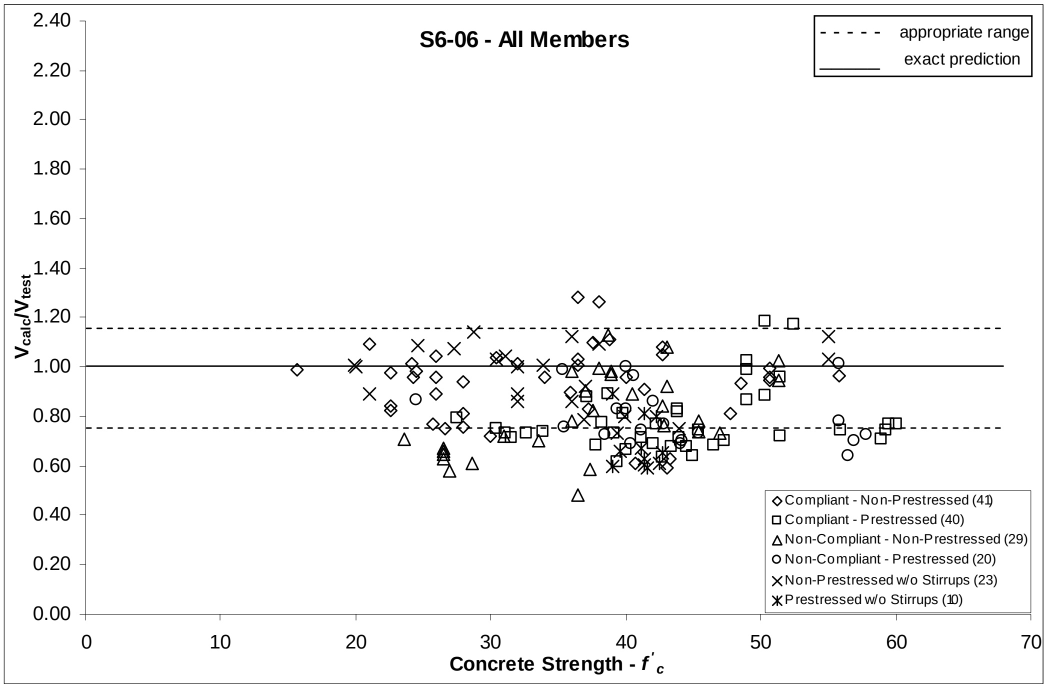



Figure 4.11 - $V_{calc}/V_{test}$ *vs.* $f'_c$ for 163 Members Evaluated using S6-06

Figure 4.11 shows the relationship between the predicted-to-tested shear capacities and the specified concrete strength for the 163 members evaluated in this study. This figure is devoid of any specific trends in $V_{calc}/V_{test}$ ratios as the concrete strength varies for any of the data categories, indicating that the shear method in S6-06 correctly accounts for this parameter.

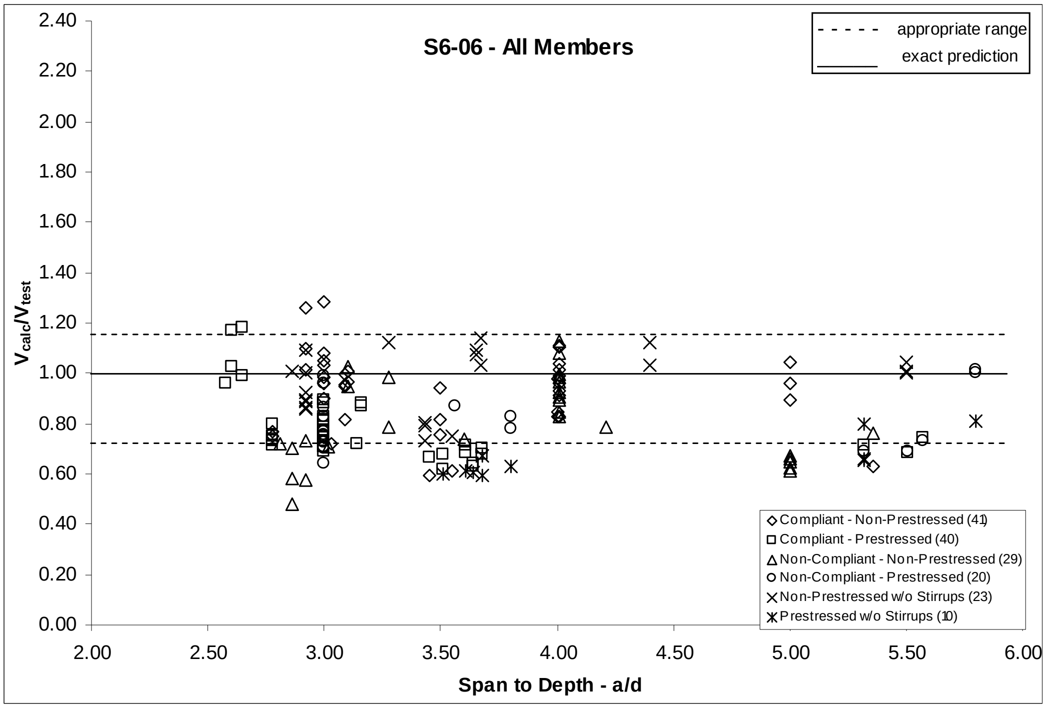



Figure 4.12 - $V_{calc}/V_{test}$ *vs. a/d* for 163 Members Evaluated using S6-06

Figure 4.12 provides the relationship between predicted-to-tested shear capacities and the shear span-to-depth ratio *a/d* for the 163 members identified for evaluation in this study. Figure 4.12 does not exhibit any defined behaviors in $V_{calc}/V_{test}$ ratios as the shear span-to-depth ratio changes for any of the member categories, indicating that the shear method in S6-06 appropriately accounts for the *a/d* ratio.

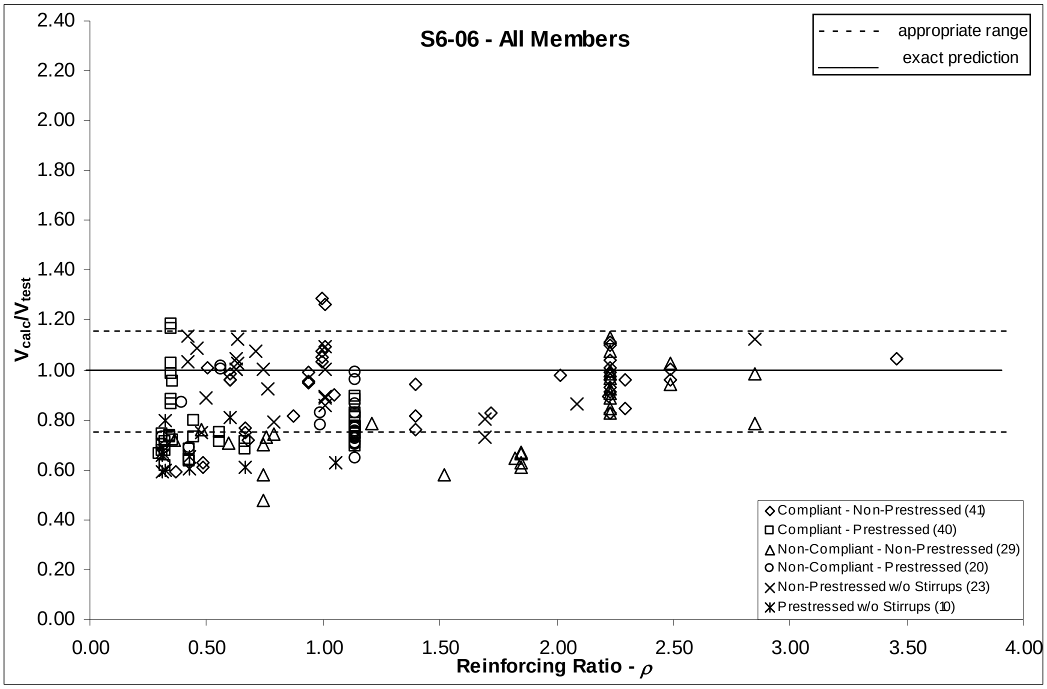



Figure 4.13 - $V_{calc}/V_{test}$ *vs.* $\rho$ for 163 Members Evaluated using S6-06

Figure 4.13 provides the relationship between $V_{calc}/V_{test}$ ratios and the longitudinal reinforcement ratio for the 163 members evaluated in this study. The horizontal distribution of $V_{calc}/V_{test}$ ratios shown in Figure 4.13 indicates that the sectional shear method in S6-06 Section 8 appropriately accounts for the quantity of longitudinal steel.

The lack of notable trends in $V_{calc}/V_{test}$ ratios due to variations in the concrete strength, shear span to depth ratio and longitudinal reinforcing ratio, as shown in Figures 4.11 through 4.13 respectively, is similar to observations presented in Kim (2004). Kim's shear capacity predictions were calculated using the provisions in A23.3-04.

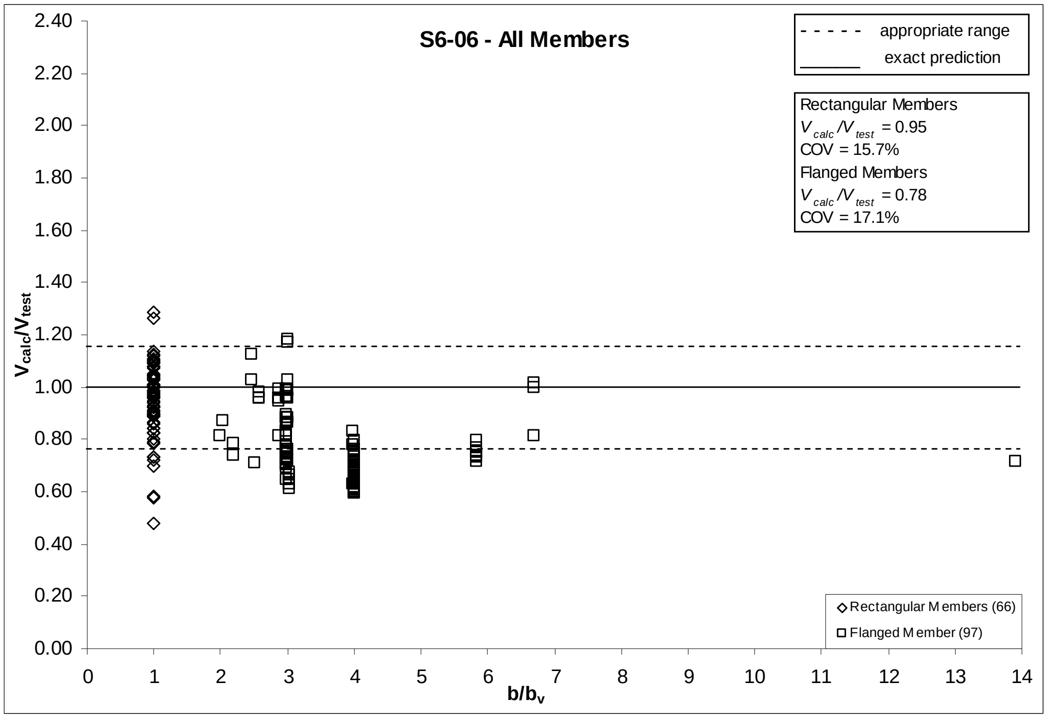



Figure 4.14 - $V_{calc}/V_{test}$ *vs.* $b/b_v$ for 163 Members Evaluated using S6-06

Figure 4.14 provides the relationship between predicted-to-tested shear capacities and the member flange width to web width ratio $b/b_v$ for the 163 members identified for evaluation in this study. This figure indicates that typical $V_{calc}/V_{test}$ ratios are smaller for flanged members than for rectangular members. This increased conservatism of shear capacity predictions for flanged members is similar to results found in other studies (Placas and Regan, 1971; Giaccio et al, 2002). Summary statistical results for the entire dataset are provided in Figure 4.14 while the non-compliant members alone have a mean $V_{calc}/V_{test}$ for rectangular and flanged members of 0.91 and 0.77 respectively with corresponding COV of 16.4% and 14.9%. No well defined variations in $V_{calc}/V_{test}$ ratios are identified for flanged members as the $b/b_v$ ratio varies. Figure 4.14 indicates that one member (T1) had a $b/b_v$ ratio of 13.9. This value represents the effective flange to web width ratio which for calculation purposes has been reduced from the actual $b/b_v$ value of 15.0 in accordance with S6-06 Clause 5.8.2.1.

### 4.5.4 Summary of Shear Predictions using the S6-06 Shear Method

Table 4.11 presents the statistical results and average demerit points per member from the evaluation of the 163 members evaluated in this study using the shear provisions in S6-06.

Table 4.11 - Evaluation Results using S6-06 Sectional Shear Provisions

| Test Group (number) | $V_{calc}/V_{test}$ Mean | $V_{calc}/V_{test}$ C.O.V (%) | Average Demerit Points / Member | $V_{test}/V_{calc}$ Mean | $V_{test}/V_{calc}$ COV (%) |
|---|---|---|---|---|---|
| All Members (163) | 0.85 | 19.2 | 5.90 | 1.23 | 19.4 |
| Non-Compliant Non-Prestressed Members (29) | 0.80 | 20.8 | 6.84 | 1.30 | 22.0 |
| Compliant Non-Prestressed Members (41) | 0.93 | 16.3 | 3.91 | 1.10 | 18.8 |
| Non-Prestressed Members without Stirrups (23) | 0.96 | 13.0 | 2.96 | 1.06 | 13.9 |
| Non-Compliant Prestressed Members (20) | 0.80 | 14.7 | 6.02 | 1.27 | 13.3 |
| Compliant Prestressed Members (40) | 0.78 | 16.6 | 7.49 | 1.30 | 14.0 |
| Prestressed Members without Stirrups (10) | 0.66 | 11.9 | 11.84 | 1.57 | 10.7 |
| All Rectangular Members (66) | 0.95 | 15.7 | 3.71 | 1.09 | 20.0 |
| All Flanged Members (97) | 0.78 | 17.1 | 7.39 | 1.32 | 15.4 |

NOTE: Vtest/Vcalc values for the data set evaluated in this study were included to allow direct comparison to results by Kim (2004) in Section 3.2.

The twenty-nine non-prestressed members with non-compliant stirrup spacing and area details evaluated using the sectional shear provisions in S6-06 are calculated to have a mean $V_{calc}/V_{test}$ ratio of 0.80 with a corresponding coefficient of variation of 20.8%. These statistical values are consistent with research focused on shear capacity predictions of members predominately compliant with respect to minimum transverse reinforcement requirements (Kim, 2004; Lubell, 2006). These studies are discussed in Section 3.2. The good agreement in statistical results for the non-compliant non-prestressed members evaluated in this study is reflected in the low value of average demerit points allotted to this member category (6.84). As discussed in Section 4.2, methods which are allotted fewer than 7.50 average demerit points per member are expected to provide shear capacity predictions which are in good agreement with tested capacities. It should be noted that the mean $V_{calc}/V_{test}$ ratio of the non-compliant non-prestressed members is considerably less than for the compliant non-prestressed members identified for this study as shown in Table 4.11. These $V_{calc}/V_{test}$ ratios (0.93 compared to 0.80) suggest that members with stirrups not complying with stirrup spacing and area requirements exhibit more conservative predictions than members meeting minimum transverse reinforcement provisions. Compliant and non-compliant member categories have similar ratios of flanged to rectangular members in this study; thus this detail is not believed to contribute to the difference in $V_{calc}/V_{test}$ ratios.

The twenty prestressed members with non-compliant stirrup spacing details evaluated in this study using the sectional shear provisions in S6-06 are calculated to have a mean $V_{calc}/V_{test}$ ratio of 0.80 and a coefficient of variation of 14.7%. These statistical values correspond well with the mean $V_{calc}/V_{test}$ ratio and corresponding COV for the compliant prestressed members evaluated in this study. The mean $V_{calc}/V_{test}$ ratio and COV of the non-compliant prestressed members evaluated in this study corresponds well with results from Kim (2004) for prestressed concrete members with predominantly compliant stirrup details. Kim's statistical results are discussed in Section 3.2. The good agreement between predicted and tested shear capacities for the non-compliant prestressed members evaluated in this study, demonstrated by the statistical analyses, is consistent with the low value of average demerit points per member for this member category (6.02).

The average $V_{calc}/V_{test}$ ratios calculated using the sectional shear method in S6-06 Section 8 for members without stirrups evaluated in this study compares well to $V_{calc}/V_{test}$ ratios of members without stirrups in Kim (2004); however the corresponding COV in this study was lower by a factor of about 2. In both studies prestressed members demonstrated more conservative predictions than did non-prestressed members.

Results from the evaluation of the members identified for this study indicate that the sectional shear provisions in S6-06 are appropriate for predicting the shear capacity of members with non-compliant stirrup spacing and area details. The typical lack of trends in $V_{calc}/V_{test}$ ratios as the stirrup ratios $s/s_{m1}$ and $A_{v,min}/A_v$ vary in Figures 4.6 through Figure 4.9 indicates that the shear method in S6-06 appropriately considers stirrup spacing and area details. Members with non-compliant stirrup details having heights greater than 800 mm and flanged members offer the most conservative predictions of shear capacity as discussed in Section 4.5.1. These issues are examined further in Chapter 5

## 4.6 American Association of State Highway and Transportation Officials AASHTO LRFD-05

The General Method for shear (Collins et al., 1996) in the American Association of State Highway and Transportation Officials (AASHTO, 2005) standard is derived from simplifications made to the Modified Compression Field Theory (Vecchio and Collins, 1986), as discussed in Section 3.3. Section 4.6 assesses the agreement between predicted and tested shear capacities calculated using AASTHO LRFD-05 Section 5 for members with non-compliant stirrup spacing and area details. Forty-nine test specimens with these non-compliant stirrup details were evaluated using spreadsheets provided by Bentz (1999) for calculating sectional shear capacity, as discussed in Section 3.3, and are compared against shear capacity evaluations of eighty-one members complying with S6-06 minimum stirrup requirements and thirty-three members without stirrups. The $s/s_{max}$ and $A_{v,min}/A_v$ ratios shown for the evaluations in Section 4.6 were calculated based on the stirrup spacing and area requirements in AASHTO LRFD-05. Variations in parameters known to affect shear capacity of concrete members (concrete strength $f'_c$, shear span to depth ratio $a/d$, longitudinal reinforcing ratio $\rho$ and member shape) are also studied in this Section to assess any influence they have on $V_{calc}/V_{test}$ ratios calculated using AASTHO LRFD-05. The effect of these parameters on actual shear capacities is discussed in Section 2.5.

### 4.6.1 Evaluation of Members with Stirrups not Complying with Minimum Transverse Reinforcement Requirements using AASHTO LRFD-05 Shear Provisions

Evaluation of Non-Prestressed Members not Complying with Stirrup Spacing and Area Details using AASHTO LRFD-05 Shear Provisions

Figure 4.15 and Figure 4.16 show the relationships $V_{calc}/V_{test}$ vs. $s/s_{max}$ and $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ respectively for the twenty-nine non-compliant non-prestressed members evaluated in this study. Shear capacity evaluations of forty-one compliant non-prestressed members are also included in these figures to facilitate a comparison of the agreement between predicted-to-tested shear capacities with the non-compliant members. These figures are used to investigate the influence that changes in the stirrup detail ratios $s/s_{max}$ and $A_{v,min}/A_v$ may have on $V_{calc}/V_{test}$ ratios. The solid line in Figures 4.15 and 4.16 represents exact shear predictions, while the two dashed lines define the range of 'appropriate' predictions as classified in Section 4.2. Members are identified as non-compliant if either of their stirrup detail ratios, $s/s_{max}$ or $A_{v,min}/A_v$, are greater than 1.00. As noted in Table 4.3, twelve of the non-prestressed members listed as compliant are actually non-compliant according to the stirrup spacing and area requirements in AASHTO LRFD-05 Section 5. These test specimens have been left among the compliant category so that similar datasets are compared for the four sectional shear methods. Table 4.12 distributes the member predictions into the ranges classified in Table 4.1, and provides the average demerit points per member for the full data set of non-prestressed members with stirrups, as well as the compliant and non-compliant data categories.

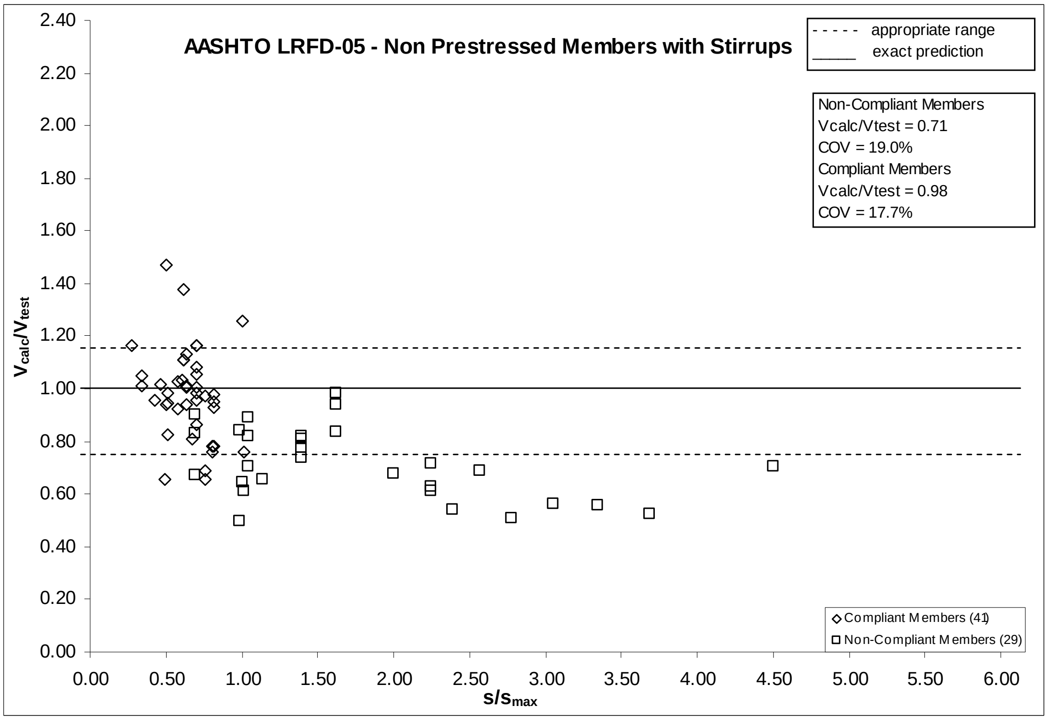



Figure 4.15 - $V_{calc}/V_{test}$ *vs.* $s/s_{max}$ for 70 Non-Prestressed Members with Stirrups Evaluated using AASHTO LRFD-05

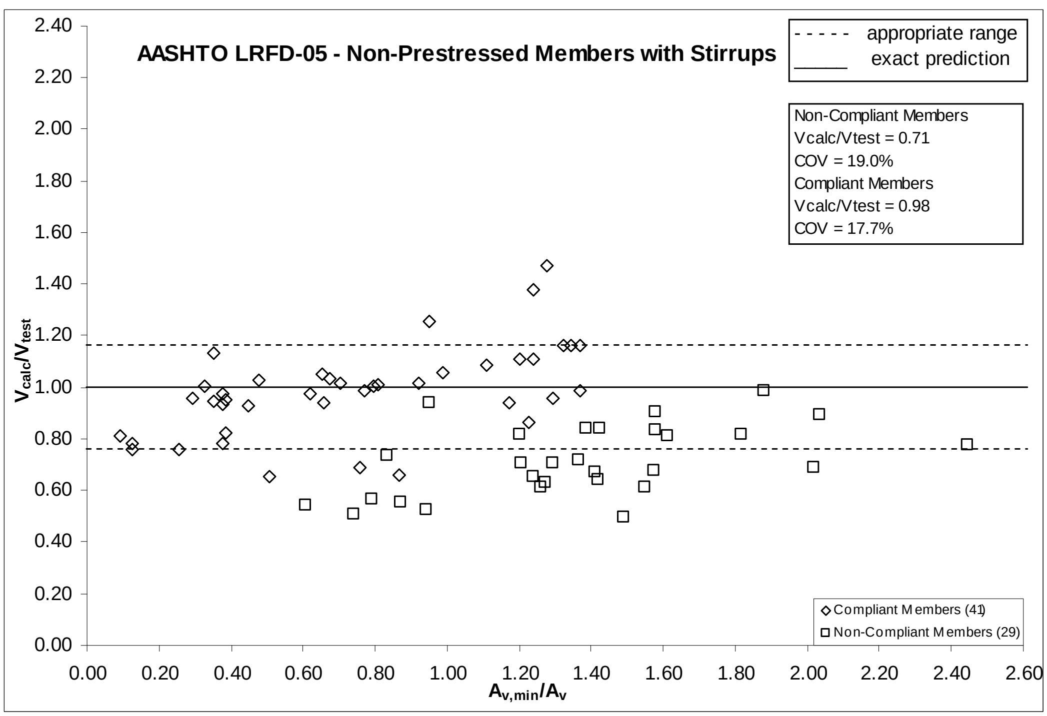



Figure 4.16 - $V_{calc}/V_{test}$ *vs.* $A_{v,min}/A_v$ for 70 Non-Prestressed Members with Stirrups Evaluated using AASHTO LRFD-05

Table 4.12 – AASHTO LRFD-05 Non-Prestressed Members - $V_{calc}/V_{test}$ Ranges and Average Demerit Points

| AASHTO LRFD-05 - Non-Prestressed Members with Stirrups | Full Data Set | Non-Compliant | Compliant | Demerits - Full Data Set | Demerits - Non-Compliant | Demerits - Compliant |
|---|---|---|---|---|---|---|
| Total | 70 | 29 | 41 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5 | 1 | 1 | 0 | | | |
| Percent of Total | 1.4% | 3.4% | 0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75 | 20 | 17 | 3 | | | |
| Percent of Total | 28.6% | 58.6% | 7.3% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15 | 43 | 11 | 32 | | | |
| Percent of Total | 61.4% | 37.9% | 78.0% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50 | 6 | 0 | 6 | | | |
| Percent of Total | 8.6% | 0.0% | 14.6% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00 | 0 | 0 | 0 | | | |
| Percent of Total | 0.0% | 0.0% | 0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0 | 0 | 0 | 0 | | | |
| Percent of Total | 0.0% | 0.0% | 0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 509 | 276 | 233 |
| Average Demerits/Members | | | | 7.27 | 9.53 | 5.68 |

45.8% of the 878 non-prestressed members evaluated by Kim (2004), using the sectional shear method in AASHTO LRFD-98, are calculated to have predictions of shear capacity in the 'appropriate' range, which is consistent with the percentage of 'appropriate' predictions for the twenty-nine non-compliant non-prestressed members evaluated in this study, as shown in Table 4.12. A description of Kim's members and the differences between the sectional shear methods in AASTHO LRFD-05 and AASTHO LRFD-98 are discussed in Section 3.3. The majority of the non-compliant non-prestressed members evaluated in this study which calculate shear capacities in the 'appropriate' range are rectangular members with depths of 800 mm or less. The non-compliant non-prestressed members with predicted to tested shear capacities in the 'conservative' and 'very conservative' ranges are mostly either members fabricated with compression flanges or members with depths greater than 800 mm. This observation is consistent with predictions using the shear method in S6-06 Section 8 as discussed in Section 4.5.1. Five non-compliant non-prestressed members evaluated as 'appropriate' using the sectional shear method in S6-06 were evaluated as 'conservative' using the sectional shear provisions in AASHTO LRFD-05, suggesting that AASHTO LRFD-05 produces more conservative shear capacity predictions than S6-06. None of the non-compliant members evaluated in this study have $V_{calc}/V_{test}$ ratios which are considered unsafe (see Table 4.1). It should be noted from Table 4.12 that evaluation of the compliant non-prestressed members in this study, using the sectional shear method in AASHTO LRFD-05, exhibit a considerably higher percentage of shear capacity predictions in the 'appropriate' range than do the non-compliant non-prestressed members (78.0% compared to 37.9%). The three compliant members with $V_{calc}/V_{test}$ ratios in the 'conservative' range are all flanged sections indicating that this detail has an influence on the agreement between predicted and tested shear capacities. This is consistent with research by others (Placas and Regan, 1971; Giaccio et al., 2002). From Table 4.12 it is evident that six compliant members have $V_{calc}/V_{test}$ values in the 'low safety' range, which is four more than resulting from evaluation using S6-06 shear provisions as shown in Table 4.8. These six members are all rectangular sections with heights greater than 500 mm.

Non-compliant non-prestressed members are calculated to have an average of 67.8% percent more demerit points per member (9.53 compared to 5.68) than are the compliant members evaluated in this study. The larger value of average demerit points per member for the non-compliant non-prestressed members compared to the compliant non-prestressed members is consistent with predictions in this study using S6-06 (see Table 4.8), and again is believed to be largely a result of the diagonal crack spacing assumption used for non-compliant members. AASHTO LRFD-05 diagonal crack spacing assumptions are discussed in Section 3.3. Collins (2001) evaluated ninety-four non-prestressed compliant members with stirrups matching the requirements defined for member selection in Section 4.3 of the present study. An average of 4.56 demerit points per member are allotted to Collins members using the Demerit Point model proposed in Section 4.2. This average is comparable to the average demerit points per member for the forty-one compliant non-prestressed members evaluated in this study.

The absence of any specific trends in $V_{calc}/V_{test}$ ratios as the ratio $s/s_{max}$ varies for non-compliant non-prestressed members, as shown in Figure 4.15, suggests that stirrup spacing is appropriately considered in the AASHTO LRFD-05 sectional shear method. Similar to evaluation using the sectional shear provisions in S6-06 Section 8 (see Figure

4.6), an apparent decrease in $V_{calc}/V_{test}$ ratios exists as the ratio $s/s_{max}$ increases past 2.00. As discussed in Section 4.5.1 the conservative predictions of shear capacity for members with $s/s_{max}$ ratios greater than 2.00 are believed to be a result of these members being fabricated with compression flanges or having overall section heights greater than 800 mm, rather than being a result of the non-compliant stirrup spacing detail. Figure 4.16 shows that variations in the stirrup area ratio $A_{v,min}/A_v$ have no appreciable influence on $V_{calc}/V_{test}$ ratios indicating that the stirrup area detail is appropriately accommodated for by the AASHTO LRFD-05 sectional shear method. All predicted shear capacities for the non-compliant members remained less than their tested capacities, even as the required stirrup area became 140% greater than the actual stirrup area. Neither Figure 4.15 or 4.16 show any outlying $V_{calc}/V_{test}$ ratios for the non-compliant non-prestressed category of members.

The absence of any specific trends in $V_{calc}/V_{test}$ values for compliant non-prestressed members as their corresponding stirrup detail ratios $s/s_{max}$ and $A_{v,min}/A_v$ vary, as shown in Figures 4.15 and 4.16, further indicates that changes in stirrup spacing and area are appropriately considered by the AASHTO LRFD-05 sectional shear method. Compliant members are determined in this study to have $V_{calc}/V_{test}$ ratios as high as 1.47 ('low safety' range) and as low as 0.66 ('conservative' range). Members with larger $V_{calc}/V_{test}$ ratios typically had rectangular cross sections, while test specimens with smaller $V_{calc}/V_{test}$ ratios were typically flanged members. Evaluation of predicted-to-tested shear capacities using AASHTO LRFD-05 provides the same outlier points (Specimens V1 and DB140M) as discussed in Section 4.5.1 for analysis using S6-06.

<u>Evaluation of Prestressed Members not Complying with Stirrup Spacing and Area Details using AASHTO LRFD-05 Shear Provisions</u>

The twenty prestressed members not complying with S6-06 stirrup spacing requirements were evaluated to assess the influence that variations in the stirrup detail ratios $s/s_{m1}$ and $A_{v,min}/A_v$ have on the agreement between predicted and tested shear capacities calculated using AASHTO LRFD-05 Section 5 provisions. All prestressed members evaluated in this study comply with S6-06 Section 14 stirrup area requirements. Forty prestressed members complying with S6-06 Section 14 minimum stirrup requirements were evaluated to facilitate a comparison of the agreement between predicted-to-tested shear capacities with the non-compliant members. Figure 4.17 provides the relationship $V_{calc}/V_{test}$ vs. $s/s_{max}$ for the sixty prestressed members with stirrups, while Figure 4.18 provides the relationship $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for the same members. The solid line in Figures 4.17 and 4.18 represent exact shear predictions, while the two dashed lines define the range of 'appropriate predictions' as given in Section 4.2. Table 4.13 distributes the member predictions into the ranges provided in Table 4.1, and provides the average demerit points per member for the full dataset of prestressed members with stirrups, as well as the compliant and non-compliant member categories.

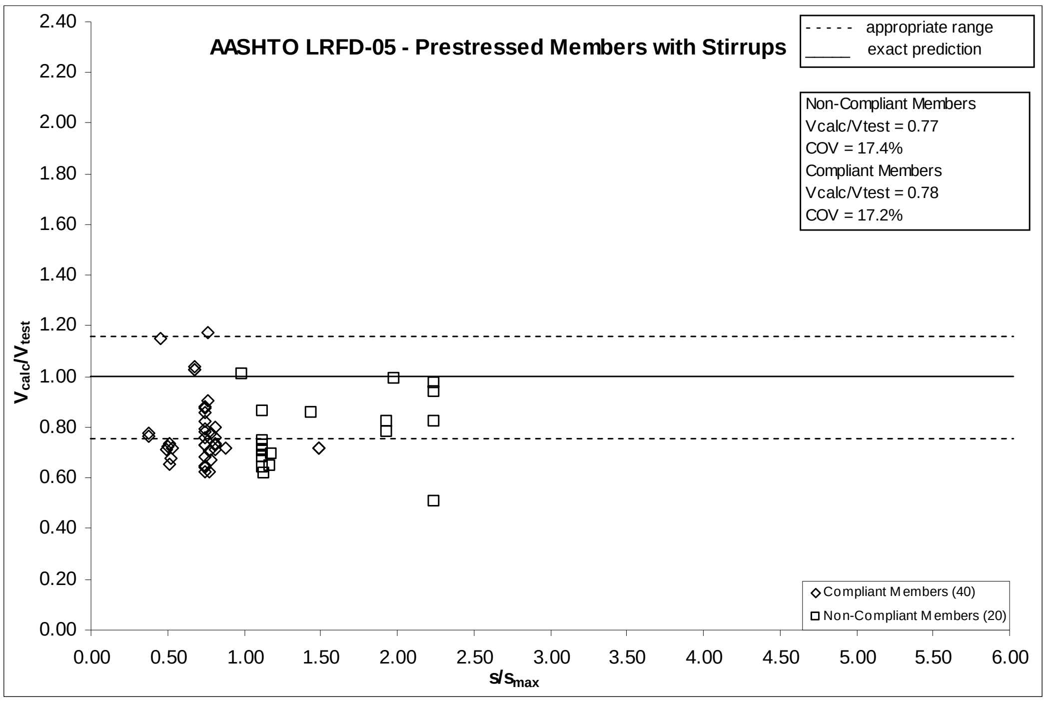



Figure 4.17 – $V_{calc}/V_{test}$ *vs.* $s/s_{max}$ for 60 Prestressed Members with Stirrups Evaluated using AASHTO LRFD-05 Shear Provisions

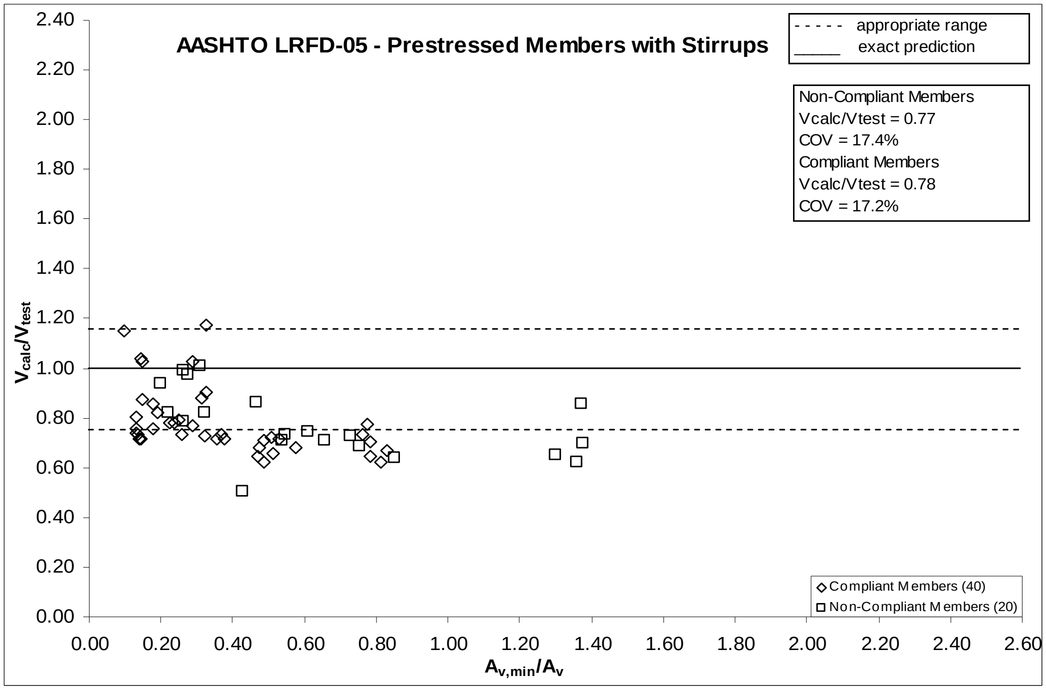



Figure 4.18 - $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for 60 Prestressed Members with Stirrups Evaluated using AASHTO LRFD-05 Shear Provisions

Table 4.13 – AASHTO LRFD-05 Prestressed Members - $V_{calc}/V_{test}$ Ranges and Demerit Points

| **AASHTO LRFD-05 - Prestressed Members with Stirrups** | **Full Data Set** | **Non-Compliant** | **Compliant** | **Demerits - Full Data Set** | **Demerits - Non-Compliant** | **Demerits - Compliant** |
|---|---|---|---|---|---|---|
| Total | 60 | 20 | 40 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 33<br>55.0% | 11<br>55.0% | 22<br>55.0% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 25<br>41.7% | 9<br>45.0% | 16<br>40.0% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 2<br>3.3% | 0<br>0.0% | 2<br>5.0% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 446 | 141 | 304 |
| Average Demerits/Members | | | | 7.43 | 7.06 | 7.61 |

38.4% of the 485 prestressed members evaluated by Kim (2004), using the sectional shear method in AASHTO LRFD-98, are calculated to have predictions of shear capacity in the 'appropriate' range, which compares well with the percentage of 'appropriate' predictions for the twenty non-compliant prestressed members evaluated in this study, as shown in Table 4.13. A description of Kim's members and the differences between the sectional shear methods in AASHTO LRFD-05 and AASHTO LRFD-98 are discussed in Section 3.3. A comparison of non-compliant members in Tables 4.12 and 4.13 indicates that the percentage of 'appropriate' shear capacity predictions for non-prestressed members compares well to the percentage of 'appropriate' shear capacity predictions for prestressed members (37.9% compared to 45.0%). The percentage of non-compliant prestressed members evaluated in this study with predicted to tested shear capacity ratios in the 'conservative' range compares well to 'conservative' predictions for the forty compliant members evaluated in this study as shown in Table 4.13, and to 'conservative' predictions for prestressed members by Kim (52.4%). None of the shear capacity predictions for the non-compliant prestressed members, calculated in this study using AASTHO LRFD-05 shear provisions, fall in the range deemed by this study to be unsafe (see Table 4.1). A comparison of compliant members in Table 4.12 and Table 4.13 indicates that the percentage of 'appropriate' shear capacity predictions for non-prestressed members does not compare well to the percentage of 'appropriate' shear capacity predictions for prestressed members (78.0% compared to 40%). The majority of compliant prestressed members are calculated to have $V_{calc}/V_{test}$ ratios in the 'conservative' range which indicates that the sectional shear method in AASHTO LRFD-05 has a tendency to provide more conservative predictions of shear capacity for prestressed members than for non-prestressed members. This is consistent with results derived from Collins (2001). Most of the prestressed members evaluated in the present study were small, flanged sections, two details that are known to result in more conservative predictions of shear capacity (Collin, 2001; Placas and Regan, 1971; Giaccio et al, 2002).

Compliant and non-compliant prestressed members are allotted nearly equal average demerit points per member, having 7.61 and 7.06 respectively. Using the sectional shear provisions in AASHTO LRFD-98, Collins (2001) evaluated forty-one prestressed members with stirrups meeting the criteria for member selection discussed in Section 4.3. These forty-one members are allotted 7.68 average demerit points per member, using the Demerit System proposed in Section 4.2, which is consistent with the prestressed member results in this study.

No deviation in $V_{calc}/V_{test}$ values for the non-compliant prestressed members is shown in Figure 4.17 as the stirrup detail ratio $s/s_{max}$ varies, indicating that changes in stirrup spacing are appropriately accounted for in the AASHTO LRFD-05 sectional shear method. As such, variations in stirrup spacing are not expected to influence the agreement between predicted and tested shear capacity calculated using AASHTO LRFD-05 Section 5 provisions for non-compliant prestressed members. Figure 4.18 demonstrates a decrease in $V_{calc}/V_{test}$ ratios as the stirrup area ratio $A_{v,min}/A_v$ increases. This behavior is similar to evaluation results for non-compliant prestressed members evaluated using the sectional shear provisions in S6-06 Section 8 and shown in Figure 4.9. Behavior exhibited by the sectional shear method in AASHTO LRFD-05 is relevant to behavior exhibited by the sectional shear method in S6-06 because both shear methods are derived based on simplifications to the Modified Compression Field Theory (Vecchio and Collins, 1986). None of the predictions exceed the 'acceptable' range, and none of the non-compliant prestressed member predictions vary significantly from the rest of that member category (ie. no outliers).

$V_{calc}/V_{test}$ ratios of compliant prestressed members in Figure 4.17 are clustered, which has the effect of reducing the possibility of defining specific trends. Figure 4.18 demonstrates a decrease in $V_{calc}/V_{test}$ ratios for compliant members as the stirrup area ratio $A_{v,min}/A_v$ increases. This is similar to behavior noted in Figure 4.9 for evaluation using the sectional shear method in S6-06 Section 8, as discussed in Section 4.5.1. Two predictions of shear capacity for the compliant prestressed members evaluated in this study fall in the in the 'low safety' range. These are the same two members that are discussed in Section 4.5.1 (Specimens A1-00-1.5R_N and A1-00-M-N).

### 4.6.2 Evaluation of Members without Transverse Reinforcement using the Shear Provisions in AASHTO LRFD-05

Based on the criteria for member selection discussed in Section 4.3, thirty-three members without stirrups were identified for evaluation in this study. The results have been used to assess the agreement between predicted and tested shear capacities calculated using the sectional shear method in AASHTO LRFD-05 Section 5 for members fabricated with no transverse reinforcement. Figure 4.19 shows the relationship $V_{calc}/V_{test}$ vs. $d$ for the thirty-three members. The sectional shear method in AASHTO LRFD-05 assumes that members without stirrups exhibit diagonal cracks spaced equal to the shear depth $d_v$ of the member (Collins et al., 1996). This diagonal crack spacing assumption is discussed in Section 3.3. Table 4.14 provides the quantity and percentage of predictions falling in the various statistical ranges (see Table 4.1), as well as the average demerit points per member corresponding to predictions.

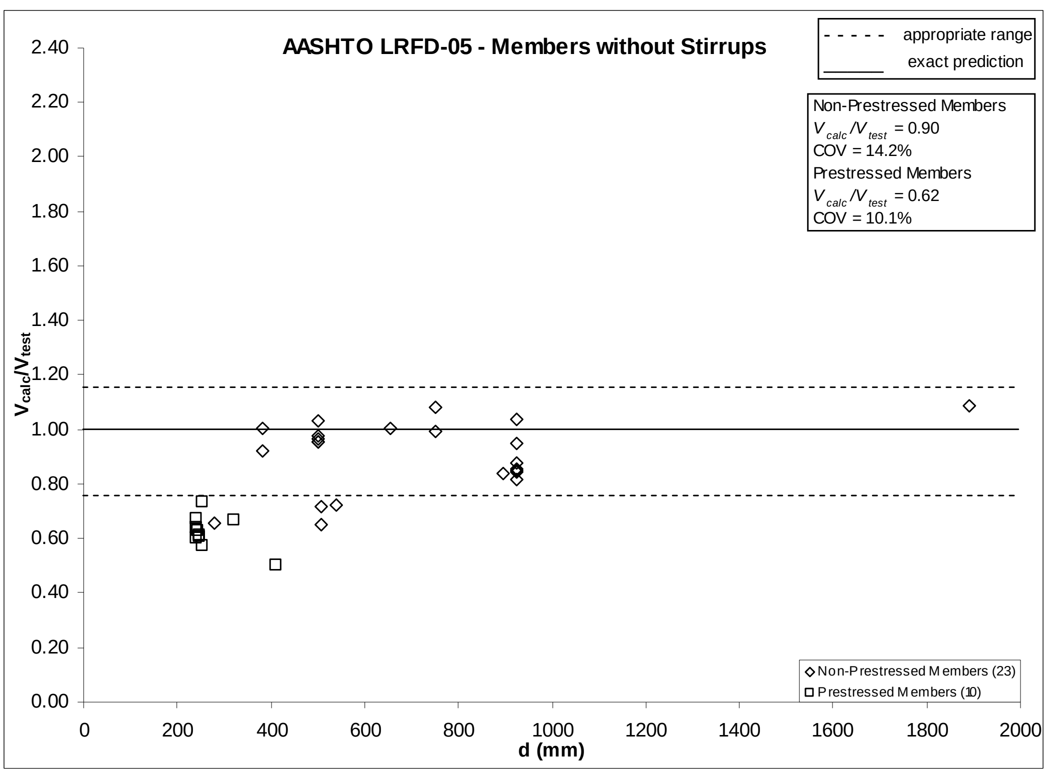



Figure 4.19 - $V_{calc}/V_{test}$ vs. $d$ for 33 Members without Stirrups using AASHTO LRFD-05 Shear Provisions

Table 4.14 – AASHTO LRFD-05 Members without Stirrups- $V_{calc}/V_{test}$ Ranges and Demerit Points

| **AASHTO LRFD-05 - Members without Stirrups** | **Full Data Set** | **Non-Prestressed** | **Prestressed** | **Demerits - Full Data Set** | **Non-Prestressed** | **Prestressed** |
|---|---|---|---|---|---|---|
| Total | 33 | 23 | 10 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 14<br>42.4% | 4<br>17.4% | 10<br>100.0% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 19<br>57.6% | 19<br>82.6% | 0<br>0.0% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 204 | 66 | 138 |
| Average Demerits/Members | | | | 6.19 | 2.89 | 13.80 |

Table 4.14 indicates a better agreement between predicted and tested shear capacities calculated using the sectional shear method in AASHTO LRFD-05 for non-prestressed members without stirrups than for prestressed members without stirrups. This is a result of the prestressed members exhibiting considerably more conservative predictions of shear capacity than did the non-prestressed members; this observation is consistent with predictions of shear capacity for members without stirrups performed by Kim (2004).

Non-prestressed members are allotted 2.89 average demerit points per member, compared to prestressed members which have 13.80 demerit points per member. The difference in average demerit points per members is a result of the increased conservatism related to AASHTO LRFD-05 predictions of prestressed members.

Consistent with results in Kim (2004), Figure 4.19 does not demonstrate any discernable increase in $V_{calc}/V_{test}$ ratios as member depth increases for non-prestressed members, indicating that the AASHTO LRFD-05 sectional shear method appropriately accounts for member depth. Four members have predictions in the 'conservative' range, which deviates from the rest of the non-prestressed data category. One of these members (Specimen P41) was a small, flanged section tested with a shear span-to-depth ratio a/d of 3.55 and having 0.48% longitudinal reinforcement. The other three (Specimens AW1, AW4, and AW8) were wide beam sections with depths of 590 mm, and longitudinal reinforcing ratios ranging from 0.79% to 1.01%. These members were loaded with *a/d* ratios ranging from 3.44 to 3.66. None of these four non-prestressed members with deviating predictions of shear capacity exceeded 70% of their flexural capacity at failure. No trends are discernable in Figure 4.19 for the ten prestressed members without stirrups evaluated in this study.

### 4.6.3 Parametric Sensitivity Analysis of Shear Predictions Made using the Shear Provisions in AASHTO LRFD-05

Section 4.6.3 assesses the influence that variations of the parameters discussed in Section 2.5 have on the agreement between predicted to tested shear capacities calculated using the sectional shear method in AASHTO LRFD-05. As described in Section 2.5, variations in concrete strength $f'_c$, shear span to depth ratio *a/d*, longitudinal reinforcement ratio $\rho$, and member shape (flanged members vs. rectangular members) are known to affect the shear capacity of concrete members. These parameters are studied against the forty-nine non-compliant members evaluated in this study to assess whether they are appropriately accounted for by the AASHTO LRFD-05 sectional shear method. The eighty-one compliant members and thirty-three members without stirrups are included to provide a comparison of their $V_{calc}/V_{test}$ ratios with those of the non-compliant members.

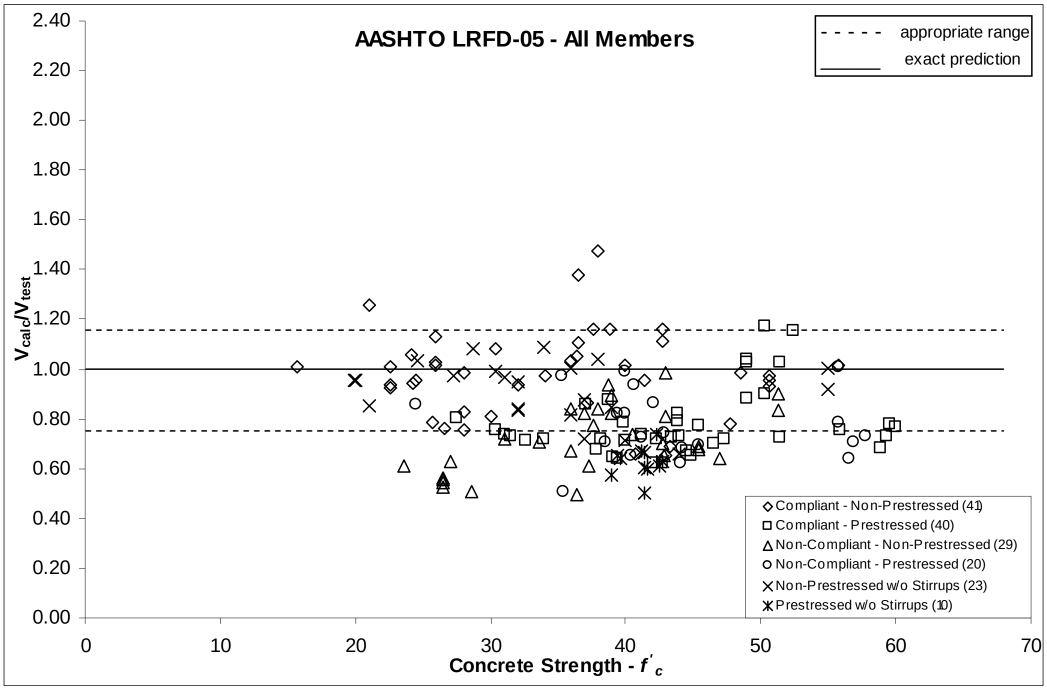



Figure 4.20 - $V_{calc}/V_{test}$ *vs.* $f'_c$ for 163 Members Evaluated using AASHTO LRFD-05

Figure 4.20 shows the relationship between the predicted-to-tested shear capacities and the specified concrete strength for the 163 members evaluated in this study. This figure is devoid of any specific trends in $V_{calc}/V_{test}$ ratios as the concrete strength varies for any of the data categories, indicating that the shear method in AASHTO LRFD-05 correctly accounts for changes in this parameter. This is similar to behavior seen in other research (Kim, 2004; Angelakos, 1999).

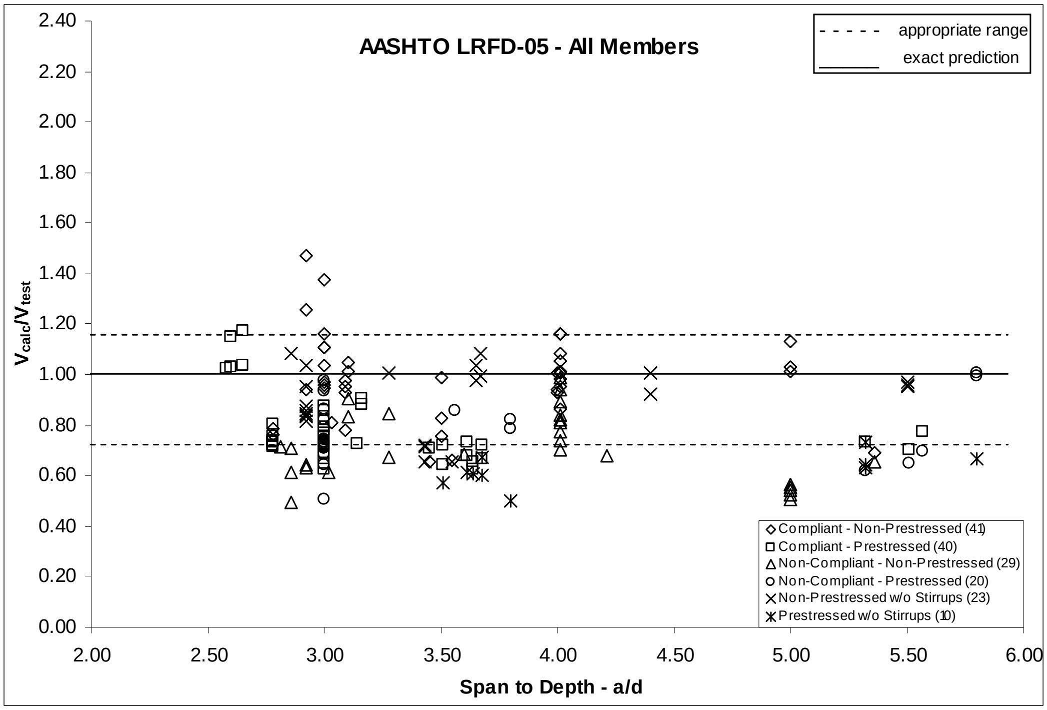



Figure 4.21 - $V_{calc}/V_{test}$ *vs. a/d* for 163 Members Evaluated using AASHTO LRFD-05

Figure 4.21 provides the relationship between predicted-to-tested shear capacities and the shear span-to-depth ratio *a/d* for the 163 members identified for evaluation in this study. The horizontal distribution of $V_{calc}/V_{test}$ ratios shown in Figure 4.21 indicates that the shear method in AASHTO LRFD-05 appropriately accounts for the *a/d* ratio. This is similar to observations in Kim (2004).

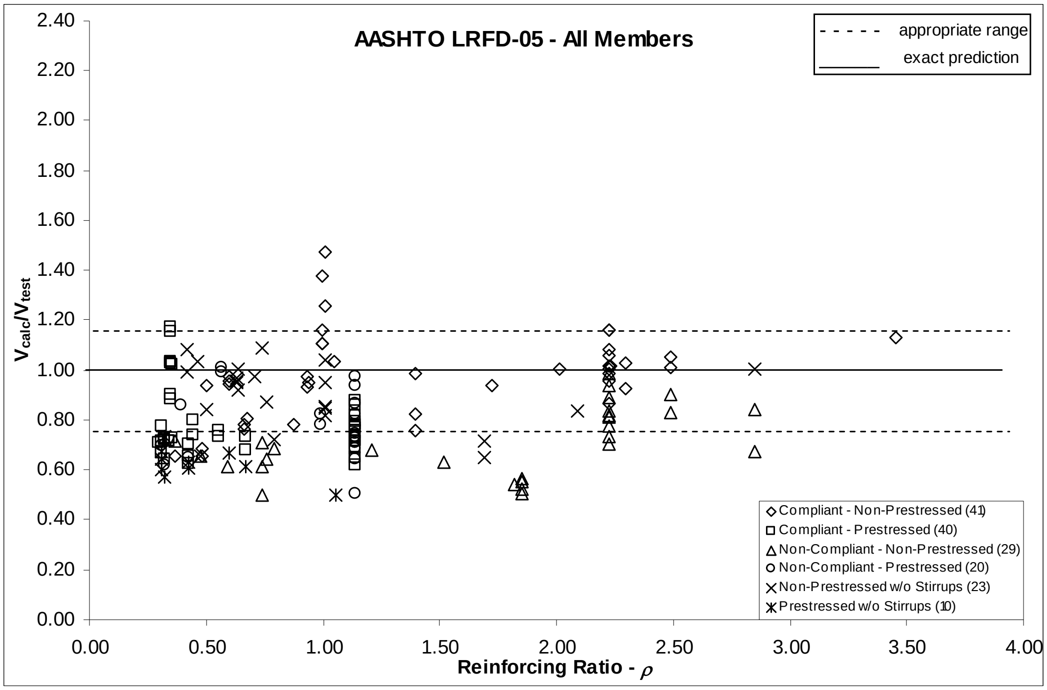



Figure 4.22 - $V_{calc}/V_{test}$ *vs.* $\rho$ for 163 Members Evaluated using AASHTO LRFD-05

Figure 4.22 provides the relationship between $V_{calc}/V_{test}$ ratios and the longitudinal reinforcement ratio for the 163 members evaluated in this study. This figure does not exhibit any trend in $V_{calc}/V_{test}$ ratios with respect to the average $V_{calc}/V_{test}$ ratio for any of the data categories as the longitudinal reinforcement percentage changes, indicating that the shear method in AASHTO LRFD-05 Section 5 appropriately accounts for the quantity of longitudinal steel. This corresponds with other studies (Kim, 2004; Angelakos, 1999).

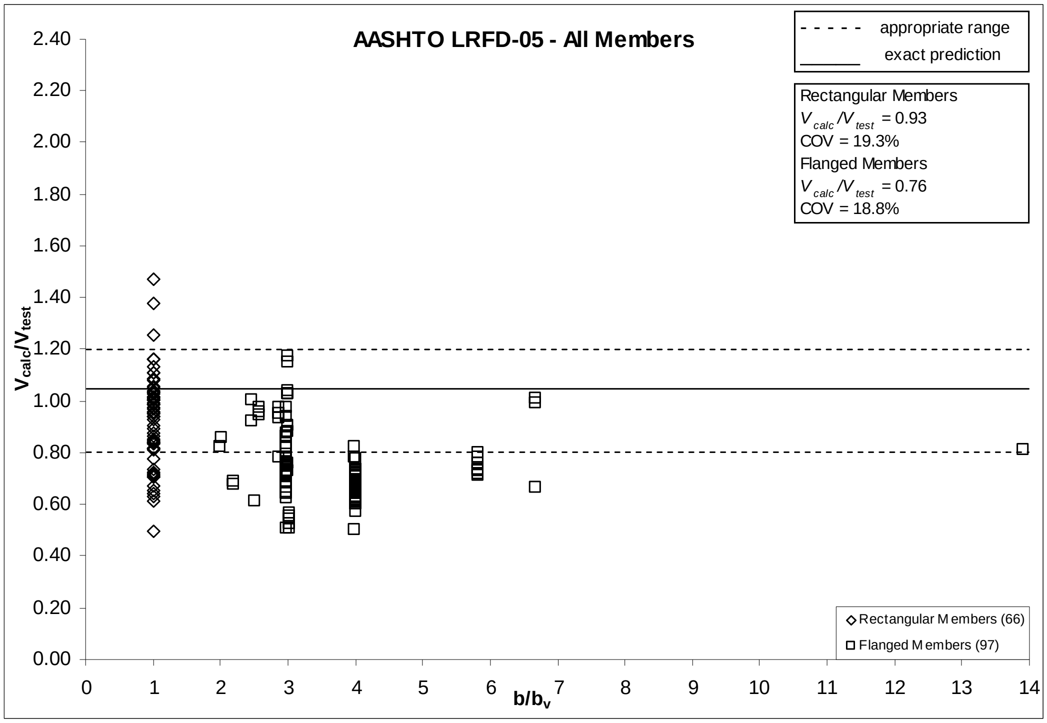



Figure 4.23 - $V_{calc}/V_{test}$ *vs.* $b/b_v$ for 163 Members Evaluated using AASHTO LRFD-05

Figure 4.23 provides the relationship between predicted-to-tested shear capacities calculated using the sectional shear method in AASTHO LRFD-05 and the member flange width to web width ratio $b/b_v$ for the 163 members identified for evaluation in this study. Similar to predictions using the Canadian Highway Bridge Design Code S6-06 (see Figure 4.14), Figure 4.23 indicates that typical $V_{calc}/V_{test}$ ratios are smaller for flanged members than for rectangular members. This increase in reserve capacity for members with compression flanges is similar to results in other studies (Placas and Regan, 1971; Giaccio et al, 2002). The rectangular non-compliant members are calculated to have a mean $V_{calc}/V_{test}$ ratio of 0.84 and a COV of 16.1%, while non-compliant flanged members have a mean $V_{calc}/V_{test}$ ratio of 0.72 and a COV of 20.4%. A summary of the statistical analyses is provided in Figure 4.23. As shown in Figure 4.23, there is no well defined variations in the $V_{calc}/V_{test}$ ratios for the flanged members as the $b/b_v$ ratio varies. Member T1 had its effective flange width reduced to 13.9 from its actual flange-to-web width ratio of 15 in accordance with S6-06 Clause 5.8.2.1, as discussed in Section 4.5.3.

### 4.6.4 Summary of Shear Predictions using the AASHTO LRFD-05 Shear Method

Table 4.15 summarizes the statistical results and average demerit points per member from the evaluation of the 163 members evaluated in this study using the shear provisions in AASHTO LRFD-05 Section 5.

Table 4.15 - Results using of AASHTO LRFD-05 Sectional Shear Provisions

| Test Group (number) | $V_{calc}/V_{test}$ Mean | $V_{calc}/V_{test}$ C.O.V (%) | Average Demerit Points / Member | $V_{test}/V_{calc}$ Mean | $V_{test}/V_{calc}$ COV (%) |
|---|---|---|---|---|---|
| All Members (163) | 0.83 | 21.7 | 7.11 | 1.26 | 21.5 |
| Non-Compliant Non-Prestressed Members (29) | 0.71 | 19.0 | 9.53 | 1.45 | 19.5 |
| Compliant Non-Prestressed Members (41) | 0.98 | 17.7 | 5.68 | 1.05 | 18.6 |
| Non-Prestressed Members without Stirrups (23) | 0.90 | 14.2 | 2.89 | 1.13 | 15.9 |
| Non-Compliant Prestressed Members (20) | 0.77 | 17.4 | 7.06 | 1.33 | 18.1 |
| Compliant Prestressed Members (40) | 0.78 | 17.2 | 7.61 | 1.31 | 14.4 |
| Prestressed Members without Stirrups (10) | 0.62 | 10.1 | 13.80 | 1.62 | 10.7 |
| All Rectangular Members (66) | 0.93 | 19.3 | 5.61 | 1.12 | 21.1 |
| All Flanged Members (97) | 0.76 | 18.8 | 8.13 | 1.37 | 18.1 |

NOTE: Vtest/Vcalc values for the data set evaluated in this study were included to allow direct comparison to results by Kim (2004) in Section 3.3.

As shown in Table 4.15 the mean $V_{calc}/V_{test}$ ratio of the twenty-nine non-compliant non-prestressed members is considerably less than for the forty-one compliant non-prestressed members identified for this study. These $V_{calc}/V_{test}$ ratios (0.71 compared to 0.98) indicate that members not complying with stirrup spacing and area requirements may provide more conservative predictions of shear capacity than members meeting minimum transverse reinforcement provisions. The statistical values calculated from the evaluation of the forty-one non-prestressed compliant members evaluated in this study are consistent with other research as discussed in Section 3.3. The compliant and non-compliant member categories have similar ratios of flanged to rectangular members in this study. Thus this detail is not believed to contribute to the difference in $V_{calc}/V_{test}$ ratios. The conservative predictions of shear capacity for the non-compliant non-prestressed members calculated using AASHTO LRFD-05 are reflected in the average demerit points per member, which exceed the 'appropriate' limit of average demerit points per member given in Section 4.2 by 27.1% (9.53 compared to 7.50).

The twenty prestressed members with non-compliant stirrup spacing details evaluated in this study using the shear method in AASHTO LRFD-05 have a mean $V_{calc}/V_{test}$ ratio of 0.77 and a coefficient of variation of 17.4%. This corresponds well with the mean $V_{calc}/V_{test}$ ratio and COV for the compliant prestressed members evaluated in this study. The mean $V_{calc}/V_{test}$ ratio and COV of the non-compliant prestressed members evaluated in this study correspond well with other research (Kim, 2004; Collins, 2001) for prestressed concrete members with predominantly compliant stirrup details, as discussed in Section 3.3. As shown in Table 4.15, the average demerit points per member for the non-compliant prestressed members is less than 7.50, which is the upper limit for 'appropriate' average demerit points recommended in Section 4.2. This is expected considering the statistical results.

The average $V_{calc}/V_{test}$ ratios and COV, calculated using the sectional shear method in AASHTO LRFD-05 Section 5, for members without stirrups evaluated in this study compares well to $V_{calc}/V_{test}$ ratios of members without stirrups in Kim (2004). In both studies prestressed members provided more conservative predictions than did non-prestressed members.

Results from the evaluation of the members identified for this study suggest that the sectional shear provisions in AASHTO LRFD-05 are appropriate for predicting the shear capacity of members with non-compliant stirrup spacing and area details. The typical lack of trends in $V_{calc}/V_{test}$ ratios as the stirrup details $s/s_{max}$ and $A_{v,min}/A_v$ vary in

Figures 4.15 through 4.18 indicates that the shear method in AASHTO LRFD-05 appropriately considers changes in stirrup spacing and area. Similar to results drawn from Section 4.5.4, which discusses evaluation using the sectional shear method in S6-06, members with non-compliant stirrup details having heights greater than 800 mm and members with compression flanges present with the most conservative predictions of shear capacity. These issues are examined further in Chapter 5.

Section 2.3 discusses a shear evaluation method proposed by Angelakos et al. (2001) for evaluating the shear capacity of members with stirrups not complying with AASTHO LRFD-00 (and AASTHO LRFD-05) transverse reinforcement area requirements. It is observed from Angelakos et al. that the shear capacity of members with non-compliant stirrup area were typically bound by predictions assuming compliant stirrup detailing and predictions assuming no stirrups. This observation was not checked for the non-compliant members in this study.

## 4.7 Response 2000

Section 4.7 assesses the agreement between predicted and tested shear capacities calculated using software Response 2000 (Bentz, 2000) for members with non-compliant stirrup spacing and area details. The shear capacity of forty-nine test specimens with these non-compliant stirrup details was evaluated using software Response 2000 following the procedure discussed in Section 3.4 and the results have been compared against shear capacity evaluations of eighty-one members complying with S6-06 minimum stirrup requirements and thirty-three members without stirrups. The $s/s_{m1}$ and $A_{v,min}/A_v$ ratios shown for the evaluations in Section 4.7 were calculated based on the stirrup spacing and area requirements in S6-06 Section 14, ignoring the $s_{m2}$ limit. Variations in parameters known to affect shear capacity of concrete members ( concrete strength $f'_c$, shear span to depth ratio $a/d$, longitudinal reinforcing ratio $\rho$ and member shape) are also studied in this section to assess any influence they have on $V_{calc}/V_{test}$ ratios calculated using Response 2000. The effect of these parameters on tested shear capacities is discussed in Section 2.5.

### 4.7.1 Evaluation of Members with Stirrups not Complying with Minimum Stirrup Requirements using Response 2000

Evaluation of Non-Prestressed Members not Complying with S6-06 Section 14 Stirrup Spacing and Area Details using Response 2000

Figure 4.24 and Figure 4.25 show the relationships $V_{calc}/V_{test}$ vs. $s/s_{m1}$ and $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ respectively for the twenty-nine non-compliant non-prestressed members evaluated in this study using software Response 2000 (Bentz, 2000). Shear capacity evaluations of forty-one compliant non-prestressed members are included in these figures to facilitate a comparison of the agreement between predicted-to-tested shear capacities with the non-compliant members. The solid line in Figures 4.24 and 4.25 represent exact shear predictions, while the two dashed lines define the range of 'appropriate predictions' classified in Section 4.2. Members evaluated in this study are identified as non-compliant if either of their stirrup detail ratios, $s/s_{m1}$ or $A_{v,min}/A_v$, are greater than 1.00. Table 4.16 distributes the member predictions into the ranges classified in Table 4.1, and provides the average demerit points per member for the full data set of non-prestressed members with stirrups, as well as for the compliant and non-compliant data categories.

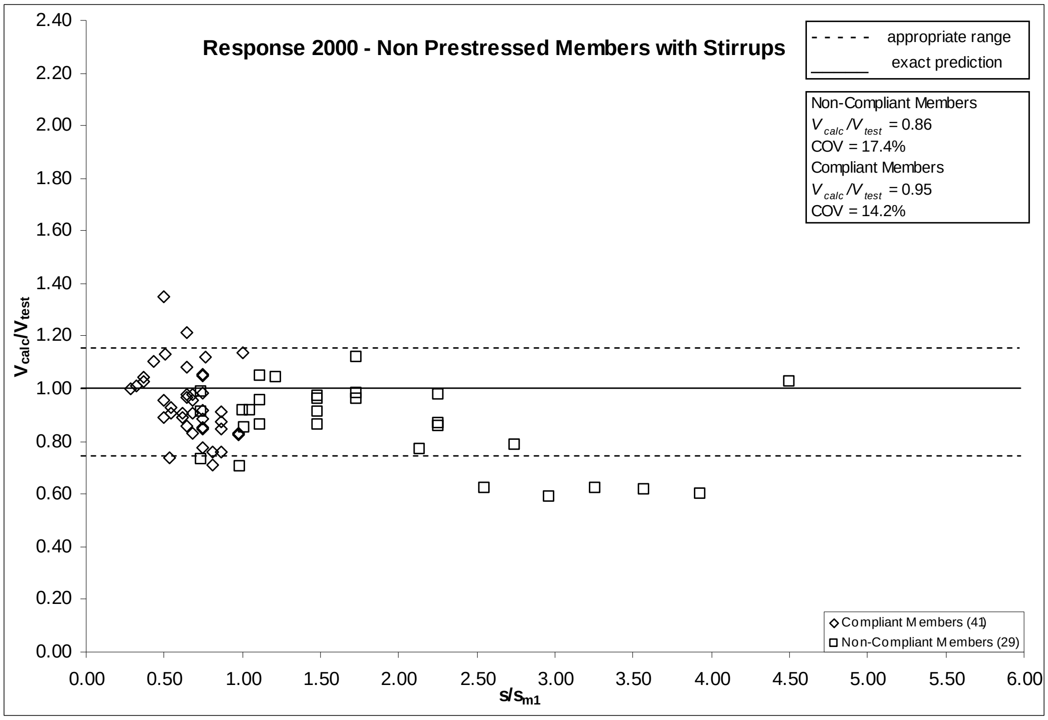



Figure 4.24 - $V_{calc}/V_{test}$ *vs.* $s/s_{m1}$ for 70 Non-Prestressed Members with Stirrups Evaluated using Response 2000

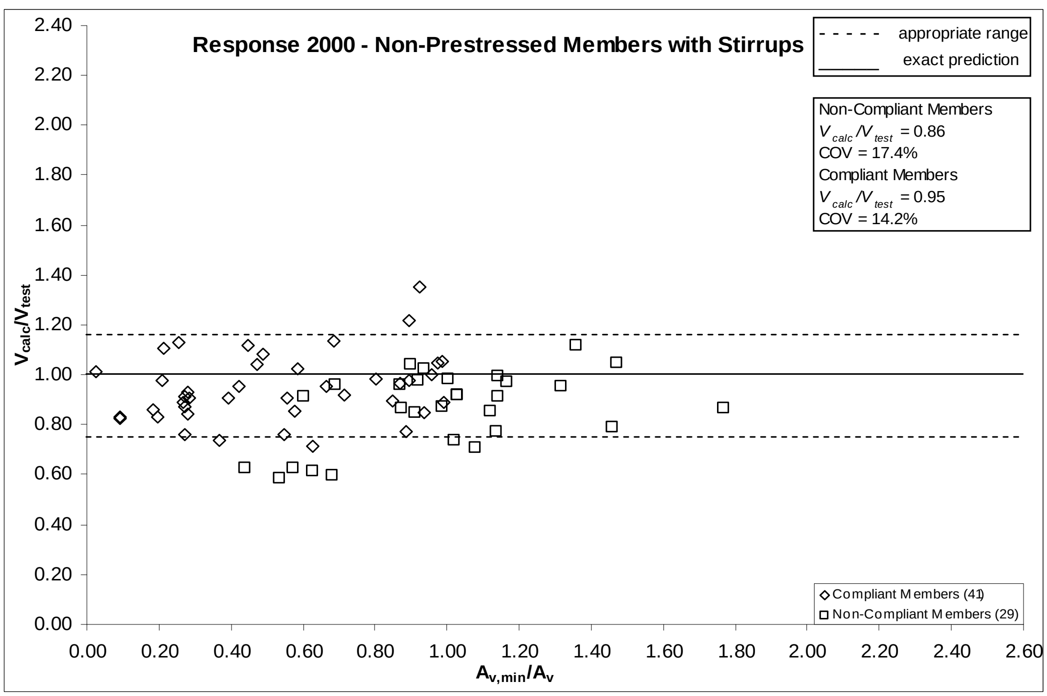



Figure 4.25 - $V_{calc}/V_{test}$ *vs.* $A_{v,min}/A_v$ for 70 Non-Prestressed Members with Stirrups Evaluated using Response 2000

Table 4.16 – Response 2000 Non-Prestressed Members - $V_{calc}/V_{test}$ Ranges and Demerit Points

| Response 2000 – Non-Prestressed Members with Stirrups | Full Data Set | Non-Compliant | Compliant | Demerits - Full Data Set | Demerits - Non-Compliant | Demerits - Compliant |
|---|---|---|---|---|---|---|
| Total | 70 | 29 | 41 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 9<br>12.9% | 7<br>24.1% | 2<br>4.9% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 59<br>84.3% | 22<br>75.9% | 37<br>90.2% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 2<br>2.9% | 0<br>0.0% | 2<br>4.9% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 265 | 120 | 146 |
| Average Demerits/Members | | | | 3.79 | 4.12 | 3.55 |

Table 4.16 indicates that 75.9% of predictions for the non-compliant non-prestressed members evaluated in this study are in the range considered 'appropriate'. This is an improvement over predictions using S6-06 shear provisions, were 55.2% of predictions for the same category of members are in the 'appropriate' range (see Table 4.8). The seven non-compliant non-prestressed members with 'conservative' predictions of shear capacity are all flanged members or members with heights greater than 800 mm. Evaluation of 192 non-prestressed members with stirrups by Bentz (2000) using Response 2000 resulted in 'appropriate' predictions for 93.2% of these members, while evaluation of ninety-four non-prestressed members with stirrups by Collins (2001) using Response 2000 resulted in 'appropriate' predictions for 89.4% of these members. As shown in Table 4.16, the percentage of 'appropriate' predictions derived from Bentz (2000) and Collins (2001) are consistent with the percentage of 'appropriate' predictions for the forty-one compliant non-prestressed members evaluated in this study. It should be noted that the percentage of compliant non-prestressed members with 'appropriate' predictions for shear capacity evaluation using Response 2000 compares favorably to

evaluation of the same members using the sectional shear method in S6-06 (90.2% compared to 82.9%).

The non-compliant non-prestressed member category averaged 16.1% more demerit points per member (4.12 compared to 3.55) than did the compliant non-prestressed members evaluated in this study. This is a result of the non-compliant members presenting with more conservative predictions of shear capacity. The average demerit points per member allotted to the compliant non-prestressed members evaluated in this study are consistent with values derived from Bentz (2000) and Collins (2001), which are allotted 2.47 and 3.25 average demerit points per member respectively, calculated using the model proposed in Section 4.2.

Similar to evaluation using the sectional shear provisions in S6-06 (see Figure 4.6) an apparent decrease in $V_{calc}/V_{test}$ ratios exists as the ratio $s/s_{m1}$ increases past 2.00. As discussed in Section 4.5.1, this is believed to be a result of members which have $s/s_{m1}$ ratios greater than 2.00 typically being flanged sections, a detail which has been shown to increase shear capacity (Placas and Regan, 1971; Giaccio et al, 2002). As such the absence of any specific trends in $V_{calc}/V_{test}$ ratios as the ratio $s/s_{m1}$ varies for non-compliant members in Figure 4.24 suggests that stirrup spacing is appropriately considered by software Response 2000 (Bentz, 2000). Response 2000 calculates $V_{calc}/V_{test}$ ratios closer to unity for members with non-compliant stirrup details and with heights greater than 800 mm than does evaluation using the sectional shear method in S6-06. This is believed to be due to the variation in assumption of diagonal crack spacing between the two sectional shear methods. The crack spacing assumptions used by the sectional shear method in S6-06 Section 8 and Response 2000 are provided in Section 3.2 and Section 3.4 respectively. Figure 4.25 shows that variations in the stirrup area ratio $A_{v,min}/A_v$ have no appreciable influence on $V_{calc}/V_{test}$ ratios, which indicates that stirrup area is appropriately accommodated for by software Response 2000. Neither Figures 4.24 nor 4.25 exhibit any outlying $V_{calc}/V_{test}$ ratios for the members in the non-compliant non-prestressed category.

The absence of any specific trend in $V_{calc}/V_{test}$ values for compliant non-prestressed members as their corresponding stirrup detail ratios $s/s_{m1}$ and $A_{v,min}/A_v$ vary in Figure 4.24 and Figure 4.25 further indicates that stirrup spacing and area is appropriately considered by software Response 2000 (Bentz, 2000). Evaluation of the forty-one compliant non-prestressed members resulted in two members appearing as outliers. These members (test specimens DB140M and V1) are discussed in Section 4.5.1.

Evaluation of Prestressed Members not Complying with Stirrup Spacing and Area Details using Response 2000

The twenty prestressed members not complying with S6-06 Section 14 stirrup spacing requirements were evaluated to assess the influence that variations in the stirrup detail ratios $s/s_{m1}$ and $A_{v,min}/A_v$ have on the agreement between predicted and tested shear capacities calculated using software Response 2000 (Bentz, 2000). Forty prestressed members complying with S6-06 Section 14 minimum stirrup requirements were evaluated to facilitate a comparison of the agreement between predicted-to-tested shear capacities with the non-compliant members. Figure 4.26 provides the relationship $V_{calc}/V_{test}$ vs. $s/s_{m1}$ for the sixty prestressed members with stirrups, while Figure 4.27 provides the relationship $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for the same members. The solid line in Figures 4.26 and 4.27 represent exact shear predictions, while the two dashed lines define the range of ‘appropriate predictions’ as given in Section 4.2. Table 4.17 distributes the member predictions into the ranges provided in Table 4.1, and provides the average demerit points per member for the full data set of prestressed members with stirrups, as well as the compliant and non-compliant member categories.

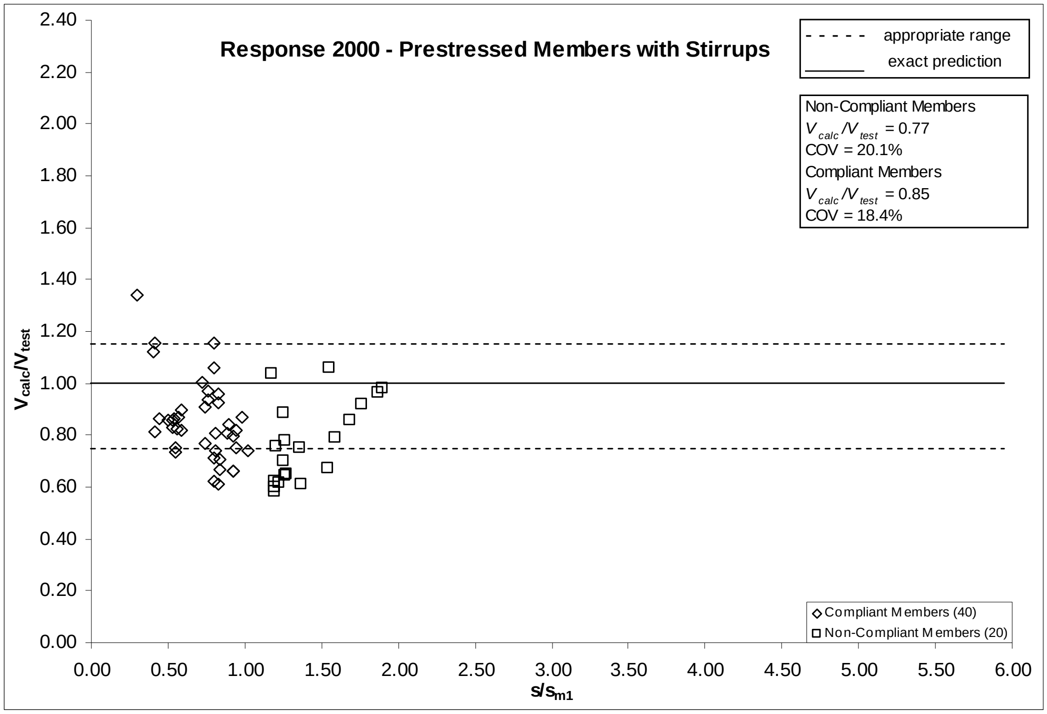



Figure 4.26 - $V_{calc}/V_{test}$ *vs.* $s/s_{m1}$ for 60 Prestressed Members with Stirrups Evaluated using Response 2000

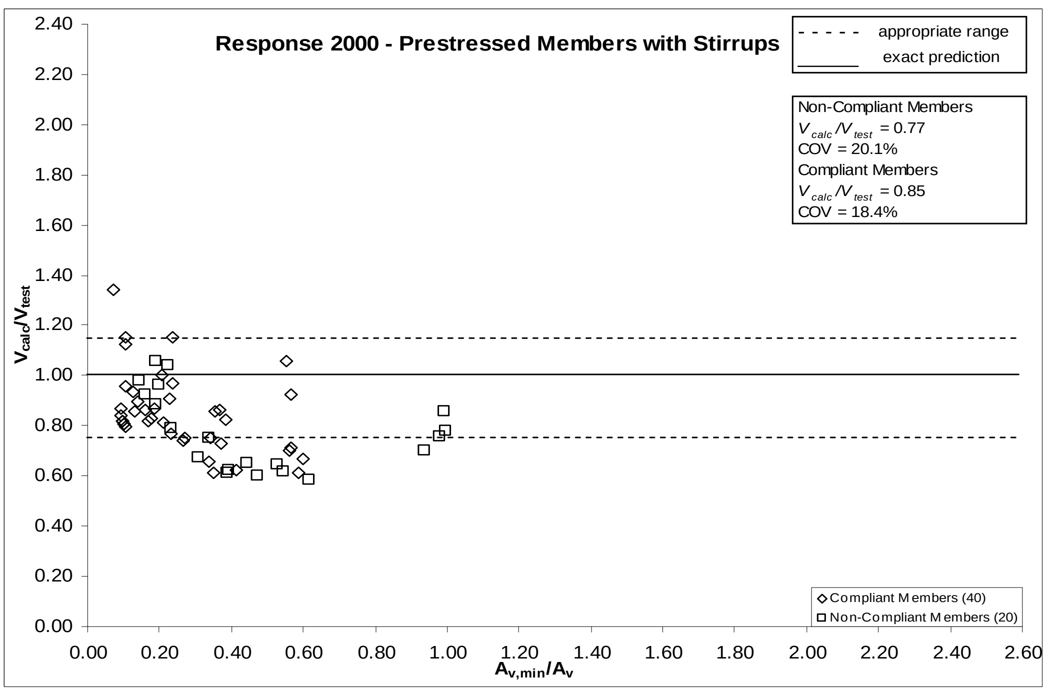



Figure 4.27 - $V_{calc}/V_{test}$ *vs.* $A_{v,min}/A_v$ for 60 Prestressed Members with Stirrups Evaluated using Response 2000

Table 4.17 – Response 2000 Prestressed Members - $V_{calc}/V_{test}$ Ranges and Demerit Points

| **Response 2000 – Prestressed Members with Stirrups** | **Full Data Set** | **Non-Compliant** | **Compliant** | **Demerits - Full Data Set** | **Demerits - Non-Compliant** | **Demerits - Compliant** |
|---|---|---|---|---|---|---|
| Total | 60 | 20 | 40 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 19<br>31.7% | 9<br>45.0% | 10<br>25.0% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 38<br>63.3% | 11<br>55.0% | 27<br>67.5% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 3<br>5.0% | 0<br>0.0% | 3<br>7.5% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 390 | 149 | 241 |
| Average Demerits/Members | | | | 6.50 | 7.46 | 6.01 |

Table 4.18 indicates that 55.0% of predictions for non-compliant prestressed members evaluated in this study using Response 2000 are in the range $V_{calc}/V_{test}$ range defined as 'appropriate' (see Table 4.1). The other 45.0% of non-compliant prestressed members evaluated in this study fall in the range deemed 'conservative'. None of the predictions of non-compliant prestressed members fall in the range defined as 'low safety' or more unsafe ranges. 67.5% of the compliant prestressed members evaluated in this study are calculated to have $V_{calc}/V_{test}$ ratios in the 'appropriate' range which is a considerable improvement over shear predictions of prestressed members using S6-06 (see Table 4.9) and AASHTO LRFD-05 (see Table 4.13), each of which present with 40% of predictions in the 'appropriate' range. Comparing compliant members in Table 4.16 and Table 4.17 indicates that the percentage of 'appropriate' shear capacity predictions for non-prestressed members does not compare well to the percentage of 'appropriate' shear capacity predictions for prestressed members (90.2% compared to 67.5%). This contrast of 'appropriate' predictions of shear capacity indicates that software Response 2000 has a tendency to provide more conservative predictions of shear capacity for prestressed members than for non-prestressed members. This observation is

consistent with results derived from Bentz (2000). Three compliant prestressed members are calculated to have shear capacity predictions in the 'low safety' range. One of these 'low safety' members appears to deviate from the rest of the compliant prestressed category and is noted later in this section.

Compliant prestressed members average 24.1% fewer demerit points per member (7.46 compared to 6.01) than non-compliant prestressed members. This difference in average demerit points per member is notably larger than for the non-prestressed members evaluated using software Response 2000 (Bentz, 2000). Unlike predictions using the shear provisions in S6-06 and AASHTO LRFD-05, this percent difference can not be explained by the crack spacing assumption since diagonal crack spacing is calculated by Response 2000 using Eqn (3.23) for all reinforced concrete members, regardless of the member's stirrup spacing or area details. The expression used by Response 2000 for calculating diagonal crack spacing is discussed in Section 3.4. The increased demerit points allotted to the non-compliant prestressed members, relative to the compliant prestressed members, is a result of the more conservative predictions of shear capacity calculated for the non-compliant members.

No deviation in $V_{calc}/V_{test}$ values for the non-compliant prestressed members evaluated in this study is shown in either Figure 4.26 or Figure 4.27 as the stirrup detail ratios $s/s_{m1}$ and $A_{v,min}/A_v$ vary, indicating that stirrup spacing and area details are appropriately accounted for by software Response 2000 (Bentz, 2000). The non-compliant data points shown in Figures 4.26 and 4.27 also do not exhibit any noticeable outliers.

Predictions of the forty compliant prestressed members displayed in Figures 4.26 and 4.27 are devoid of any indicative trends in $V_{calc}/V_{test}$ ratios with respect to $s/s_{m1}$ and $A_{v,min}/A_v$ ratios, further indicating that Response 2000 (Bentz, 2000) correctly accounts for stirrup spacing and area details. Evaluation of the forty compliant members exhibits one outlying prediction. This member (A1-00-1.5R_N) is discussed in Section 4.5.1.

### 4.7.2 Evaluation of Members without Transverse Reinforcement using Response 2000

Based on the criteria for member selection discussed in Section 4.3, thirty-three members without stirrups were identified for evaluation in this study. The results are used to assess the agreement between predicted and tested shear capacities calculated using software Response 2000 (Bentz, 2000) for members fabricated with no transverse reinforcement. Figure 4.28 shows the relationship $V_{calc}/V_{test}$ vs. $d$ for the thirty-three members. Table 4.18 provides the quantity and percentage of predictions falling in the various statistical ranges (see Table 4.1), as well as the average demerit points per member corresponding to predictions.

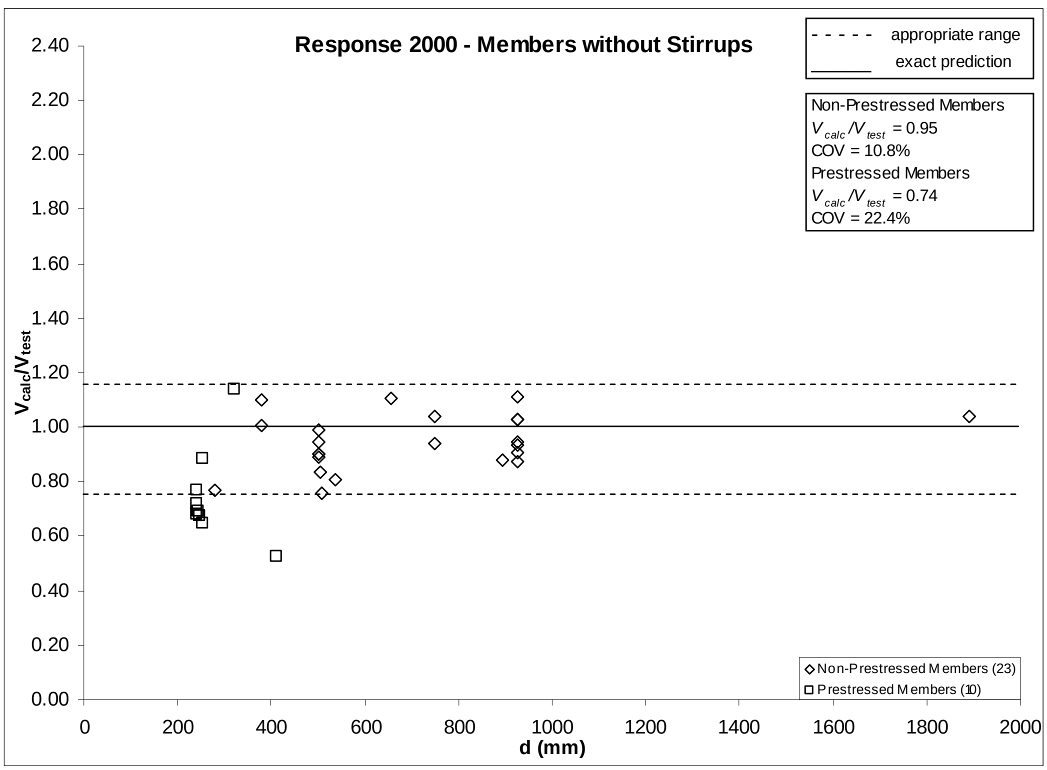



Figure 4.28 - $V_{calc}/V_{test}$ *vs.* $d$ for 33 Members without Shear Reinforcement Evaluated using Response 2000

Table 4.18 – Response 2000 Members without Stirrups- $V_{calc}/V_{test}$ Ranges and Demerit Points

| **Response 2000 - Members without Stirrups** | **Full Data Set** | **Non-Prestressed** | **Prestressed** | **Demerits - Full Data Set** | **Non-Prestressed** | **Prestressed** |
|---|---|---|---|---|---|---|
| Total | 33 | 23 | 10 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 7<br>21.2% | 0<br>0.0% | 7<br>70.0% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 26<br>78.8% | 23<br>100.0% | 3<br>30.0% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 143 | 42 | 101 |
| Average Demerits/Members | | | | 4.33 | 1.82 | 10.10 |

Table 4.18 indicates a better agreement between predicted and tested shear capacities calculated using software Response 2000 (Bentz, 2000) for non-prestressed members without stirrups than for prestressed members without stirrups. This is a result of the prestressed members demonstrating more conservative predictions of shear capacity than did the non-prestressed members. This is consistent with predictions of shear capacity for members without stirrups derived from Bentz (2000).

Non-prestressed members are allotted with an average of 1.82 demerit points per member, compared to prestressed members which are allotted 10.10 average demerit points per member. This increase is a result of the increased conservatism related to Response 2000 shear capacity predictions of prestressed members.

Figure 4.28 does not demonstrate any definable variations in $V_{calc}/V_{test}$ ratios as the section depth changes for the members in both the non-prestressed and prestressed categories, which indicates that this parameter is appropriately accounted for in software

Response 2000. This is consistent with observations by Bentz (2000). No $V_{calc}/V_{test}$ ratios for the non-prestressed members appear to deviate from the average $V_{calc}/V_{test}$ ratio for the non-prestressed in Figure 4.28. The small variation in section depth for the prestressed members does not permit for an examination in trends for these members.

### 4.7.3 Parametric Sensitivity Analysis of Shear Predictions using Response 2000

Section 4.7.3 assesses the influence that variations of the parameters discussed in Section 2.5 have on the agreement between predicted and tested shear capacities calculated using software Response 2000 (Bentz, 2000). As described in Section 2.5, member parameters such as the concrete strength $f'_c$, shear span to depth ratio $a/d$, longitudinal reinforcement ratio $\rho$, and member shape (flanged members vs. rectangular members) are known to affect the shear capacity of concrete members. These parameters are studied against the forty-nine non-compliant members evaluated in this study to assess whether they are appropriately accounted for by software Response 2000. The eighty-one compliant members and thirty-three members without stirrups are included to provide a comparison of $V_{calc}/V_{test}$ ratios with the non-compliant members.

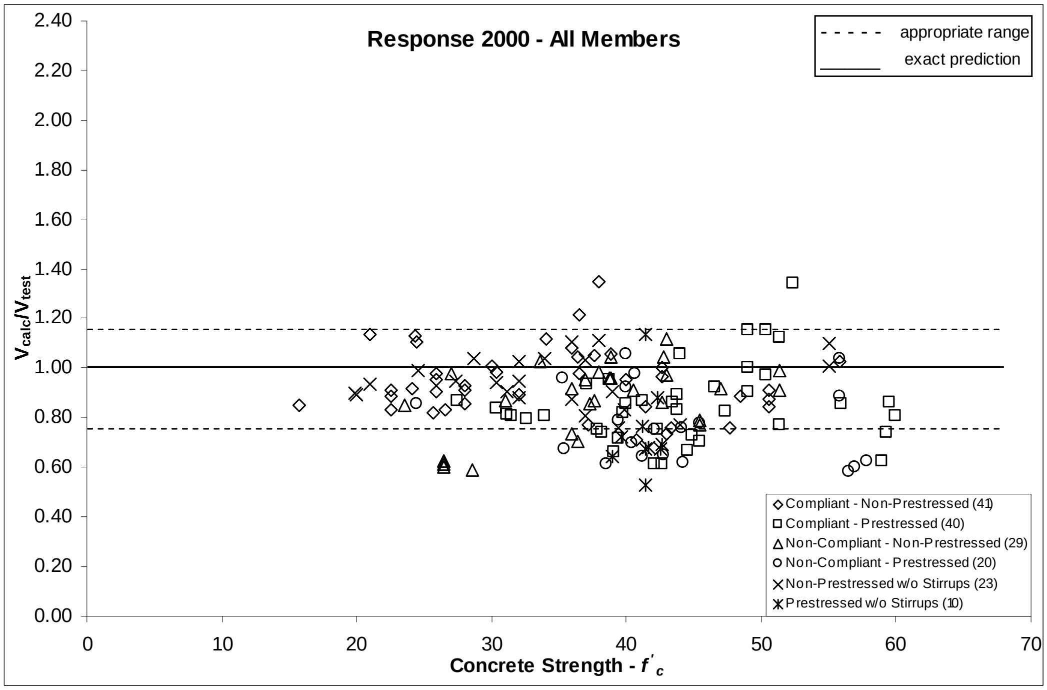



Figure 4.29 - $V_{calc}/V_{test}$ *vs.* $f'_c$ for 163 Members Evaluated using Response 2000

Figure 4.29 shows the relationship between the predicted-to-tested shear capacities and the specified concrete strength for the 163 members evaluated in this study. This figure is devoid of any specific skews in $V_{calc}/V_{test}$ ratios as the concrete strength varies for any of the data categories, indicating that software Response 2000 correctly accounts for concrete strength.

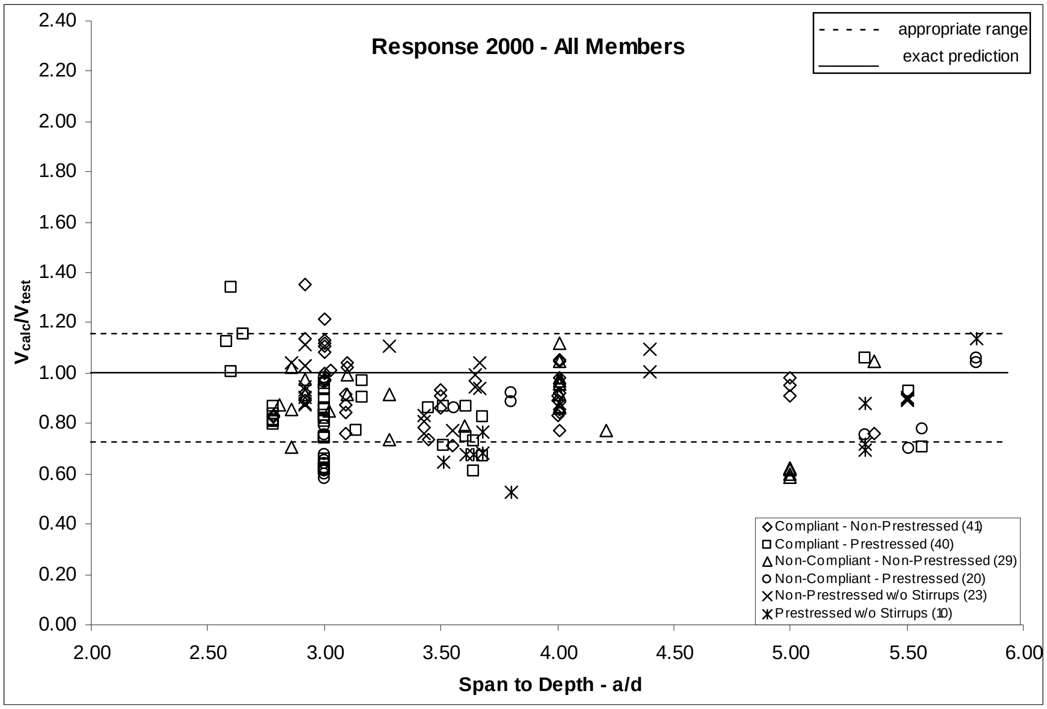



Figure 4.30 - $V_{calc}/V_{test}$ *vs. a/d* for 163 Members Evaluated using Response 2000

Figure 4.30 provides the relationship between predicted-to-tested shear capacities and the shear span-to-depth ratio *a/d* for the 163 members identified for evaluation in this study. The horizontal distribution of $V_{calc}/V_{test}$ ratios shown in Figure 4.30 indicates that software Response 2000 appropriately accounts for the *a/d* ratio.

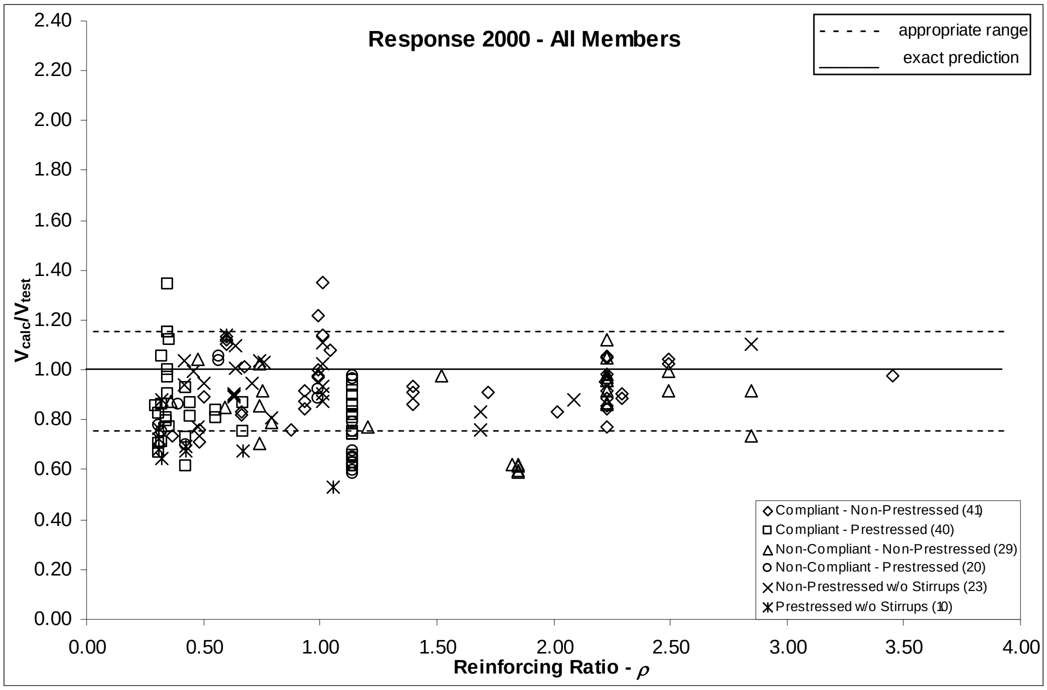



Figure 4.31 - $V_{calc}/V_{test}$ *vs.* $\rho$ for 163 Members Evaluated using Response 2000

Figure 4.31 provides the relationship between $V_{calc}/V_{test}$ ratios and the longitudinal reinforcement ratio for the 163 members evaluated in this study. This figure does not demonstrate any trend in $V_{calc}/V_{test}$ ratios for any of the data categories as the longitudinal reinforcement percentage changes indicating that software Response 2000 appropriately accounts for the quantity of longitudinal steel.

Results of shear capacity analysis in Bentz (2000) indicates that variations in concrete strength, shear span to depth ratio and longitudinal reinforcement ratio are appropriately accounted for by software Response 2000, which is consistent with results shown in Figures 4.29 through 4.31 respectively. During the literature review performed for the present study, Bentz (2000) was the only reference found that examined the influence of these parameters on the agreement between predicted and tested shear capacities when using Response 2000.

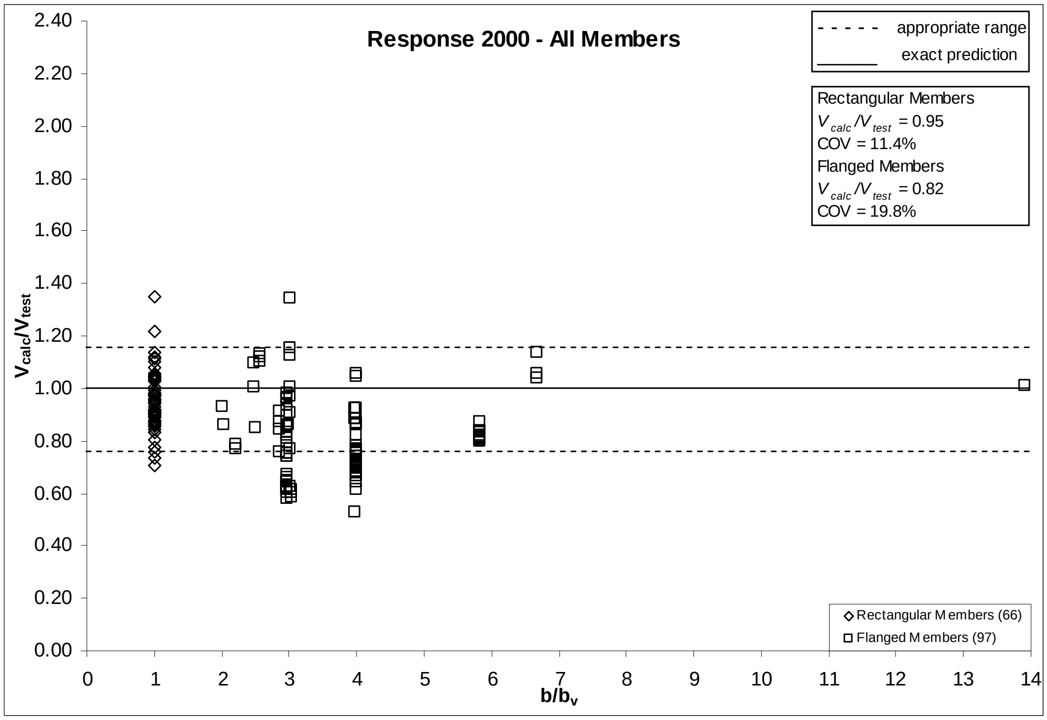



Figure 4.32 - $V_{calc}/V_{test}$ *vs.* $b/b_v$ for 163 Members Evaluated using Response 2000

Figure 4.32 provides the relationship between predicted-to-tested shear capacities, calculated using software Response 2000 (Bentz, 2000), and the member flange width to web width ratio $b/b_v$ for the 163 members identified for evaluation in this study. Similar to predictions of shear capacity calculated using provisions in S6-06 Section 8 (see Figure 4.14), Figure 4.32 indicates that typical $V_{calc}/V_{test}$ ratios are smaller for flanged members than for rectangular members. This increase in reserve capacity for members with compression flanges is similar to results in other studies (Placas and Regan, 1971; Giaccio et al, 2002). The statistical results from the entire dataset, summarized in Figure 4.32, compares well with the statistical results for the non-compliant members, which are calculated to have a mean $V_{calc}/V_{test}$ ratio for rectangular and flanged members of 0.94 and 0.75 respectively, with corresponding COV of 11.7% and 19.6%. The increased scatter at $b/b_v$ equal to 3.0 is a result of the majority of prestressed members having this ratio and of prestressed members being calculated to have more conservative predictions than did non-prestressed members. No well defined variations in $V_{calc}/V_{test}$ ratios are identified for flanged members as the $b/b_v$ ratio varies. As discussed in Section 4.5.3, member T1 had its effective flange width reduced for calculations purposes as per S6-06 Clause 5.8.2.1.

### 4.7.4 Summary of Shear Predictions of using Response 2000

Table 4.19 presents the results of statistical analyses and the average demerit points per member from evaluation of shear capacity using Response 2000 (Bentz, 2000) for the 163 members identified for this study. Although Kim (2004) did not use Response 2000 in his study, $V_{test}/V_{calc}$ values and corresponding COV have been included in Table 4.20 to allow convenient comparison to test results found in literature.

Table 4.19 - Evaluation Results using Response 2000

| Test Group (number) | $V_{calc}/V_{test}$ Mean | $V_{calc}/V_{test}$ C.O.V (%) | Demerit Points per Member | $V_{test}/V_{calc}$ Mean | $V_{test}/V_{calc}$ COV (%) |
|---|---|---|---|---|---|
| All Members (163) | 0.87 | 17.9 | 4.89 | 1.18 | 18.8 |
| Non-Compliant Non-Prestressed Members (29) | 0.86 | 17.4 | 4.12 | 1.20 | 19.9 |
| Compliant Non-Prestressed Members (41) | 0.95 | 14.2 | 3.55 | 1.08 | 13.5 |
| Non-Prestressed Members without Stirrups (23) | 0.96 | 10.8 | 1.82 | 1.07 | 13.5 |
| Non-Compliant Prestressed Members (20) | 0.77 | 20.1 | 7.46 | 1.34 | 19.0 |
| Compliant Prestressed Members (40) | 0.85 | 18.4 | 6.01 | 1.21 | 17.1 |
| Prestressed Members without Stirrups (10) | 0.74 | 22.4 | 10.10 | 1.40 | 19.0 |
| All Rectangular Members (66) | 0.95 | 11.4 | 2.52 | 1.06 | 11.2 |
| All Flanged Members (97) | 0.82 | 19.8 | 6.51 | 1.26 | 19.1 |

Consistent with the other shear methods studied, non-compliant non-prestressed members present with a more conservative average $V_{calc}/V_{test}$ ratio than do the compliant non-prestressed members (0.86 compared to 0.95). The statistical values calculated from the evaluation of the seventy non-prestressed members with stirrups are consistent with results derived from other research (Bentz, 2000; Lubell, 2006) as provided in Section 3.4. It should be noted that compliant and non-compliant members had similar ratios of flanged to rectangular members – thus this detail is not believed to contribute to the difference in $V_{calc}/V_{test}$ ratios. The good agreement between predicted and tested shear capacities, which is evident from the average $V_{calc}/V_{test}$ ratio being close to unity and corresponding low COV for the twenty-nine non-compliant non-prestressed members evaluated using Response 2000, is reflected in the low value of average demerit points per member (4.12). This value is well below the 'appropriate' limit for average demerit points recognized in Section 4.2, indicating that Response 2000 is expected to calculate $V_{calc}/V_{test}$ ratios which are close to unity for non-compliant non-prestressed members.

Consistent with the other shear methods used in this study, non-compliant prestressed members are found to demonstrate a more conservative mean $V_{calc}/V_{test}$ ratio than do the compliant prestressed members (0.77 compared to 0.85). The statistical values of the compliant prestressed category of members evaluated in this study are comparable to those derived from Bentz (2000) as discussed in Section 3.4. The results of the statistical analyses indicate that predictions of shear capacity for non-compliant prestressed members calculated using Response 2000 will be in good agreement with tested shear capacities. This good agreement is reflected in the observation that the average demerit points per non-compliant prestressed member falls below the 'appropriate' limit of 7.50 average demerit points, discussed in Section 4.2.

The statistical values for the non-prestressed members without stirrups evaluated in this study are similar to those for the non-prestressed members without stirrups evaluated in Bentz (2000) using Response 2000. However the average $V_{calc}/V_{test}$ ratio of the prestressed members without stirrups evaluated in this study is 22.9% smaller (0.74 compared to 0.96) and the COV is 72.3% larger (22.4% compared to 13.0%) than the corresponding values derived from Bentz (2000). This is likely due to the smaller data set of prestressed members examined in this study. Both studies suggest that evaluating the shear capacity of prestressed members will result in more conservative predictions than evaluating the shear capacity of non-prestressed members.

Results from the evaluation of the members identified for this study indicate that software Response 2000 is appropriate for predicting the shear capacity of members with non-compliant stirrup spacing and area details. The lack of trends in $V_{calc}/V_{test}$ ratios as the stirrup ratios $s/s_{m1}$ and $A_{v,min}/A_v$ vary in Figures 4.24 through 4.27 indicate that Response 2000 appropriately considers stirrup spacing and area details. Evaluation of shear capacity, using Response 2000, for non-compliant members with section heights greater than 800 mm is shown to provide better agreement between predicted and tested shear capacities than evaluations using the sectional shear method is S6-06 Section 8. This is believed to be in large part a result of the difference in the calculation of diagonal crack spacing used in Response 2000 and S6-06 Section 8. These crack spacing assumptions are discussed in Section 3.2 and Section 3.4 for S6-06 and Response 2000 respectively.

## 4.8 Building Code Requirements for Structural Concrete ACI 318-08

Section 4.8 assesses the agreement between predicted and tested shear capacities calculated using the sectional shear provisions in ACI 318-08 for members with non-compliant stirrup spacing and area details. The shear capacity of forty-nine test specimens with these non-compliant attributes was evaluated using provisions in ACI 318-08 Section 11 following the procedure discussed in Section 3.5 and the results are compared against shear capacity evaluations of eighty-one members complying with S6-06 minimum stirrup requirements and thirty-three members without stirrups. The $s/s_{max}$ and $A_{v,min}/A_v$ ratios shown for the evaluations in Section 4.8 were calculated based on the stirrup spacing and area requirements in ACI 318-08 (see Section 3.5). Variations in concrete strength $f'_c$, shear span-to-depth ratio $a/d$, longitudinal reinforcing ratio $\rho$ and member shape are also studied in this section to assess any influence they have on $V_{calc}/V_{test}$ ratios for concrete members calculated using the sectional shear provisions in ACI 318-08. The effect of these parameters on the shear strength of reinforced concrete members is discussed in Section 2.5.

### 4.8.1 Evaluation of Members with Stirrups not Complying with Minimum Transverse Reinforcement Requirements using ACI 318-08

Evaluation of Non-Prestressed Members not Complying with S6-06 Section 14 Stirrup Spacing and Area Details using ACI 318-08

Figure 4.33 and Figure 4.34 show the relationships $V_{calc}/V_{test}$ vs. $s/s_{max}$ and $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ respectively for the twenty-nine non-compliant non-prestressed members evaluated in this study using the sectional shear method in ACI 318-08 Section 11. Shear capacity evaluations of forty-one compliant non-prestressed members are included in these figures to facilitate a comparison of the agreement between predicted-to-tested shear capacities with the non-compliant members. As noted in Table 4.3, four of the non-prestressed members listed as compliant are actually non-compliant according to the stirrup spacing and area requirements in ACI 318-08. These test specimens have been left among the compliant category of members so that similar data sets can be compared for the four shear evaluation methods used in this study. The solid line in each of Figures 4.33 and 4.34 represents exact shear predictions, while the two dashed lines define the range of ‘appropriate predictions’ as given in Section 4.2. Members evaluated in this study are classified as non-compliant if either of the stirrup detail ratios, $s/s_{max}$ or $A_{v,min}/A_v$, are greater than 1.00. Table 4.20 distributes the member predictions into the ranges classified in Table 4.1, and provides the average demerit points per member for the full data set of non-prestressed members with stirrups, as well as the compliant and non-compliant data categories.

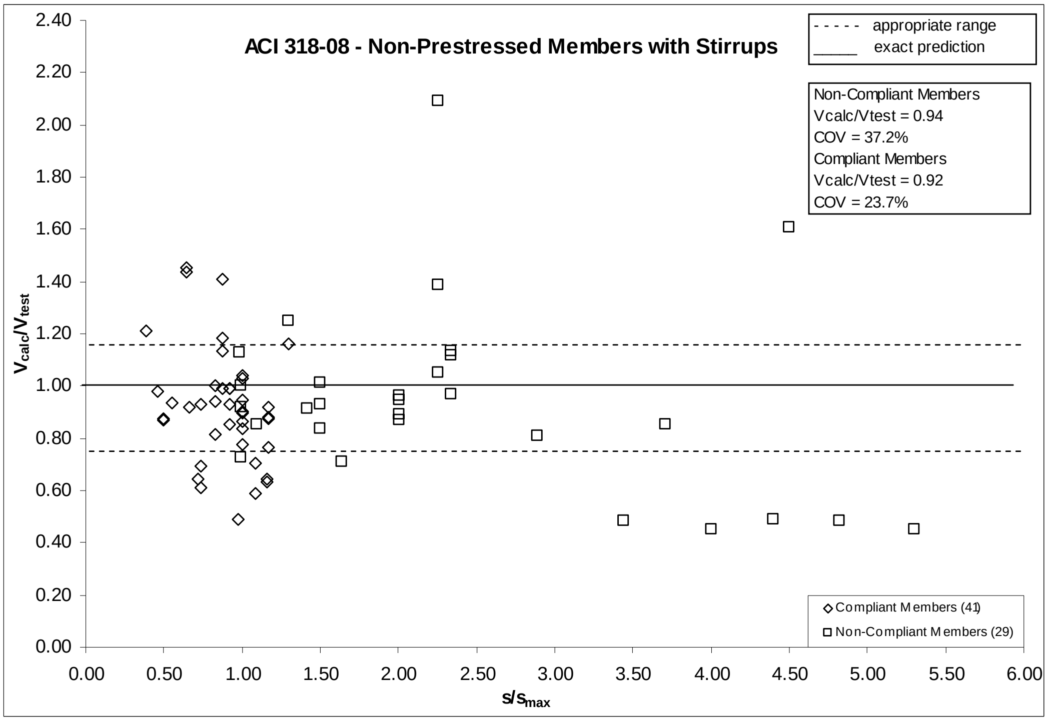



Figure 4.33 - $V_{calc}/V_{test}$ *vs.* $s/s_{max}$ for 70 Non-Prestressed Members with Stirrups Evaluated using ACI 318-08

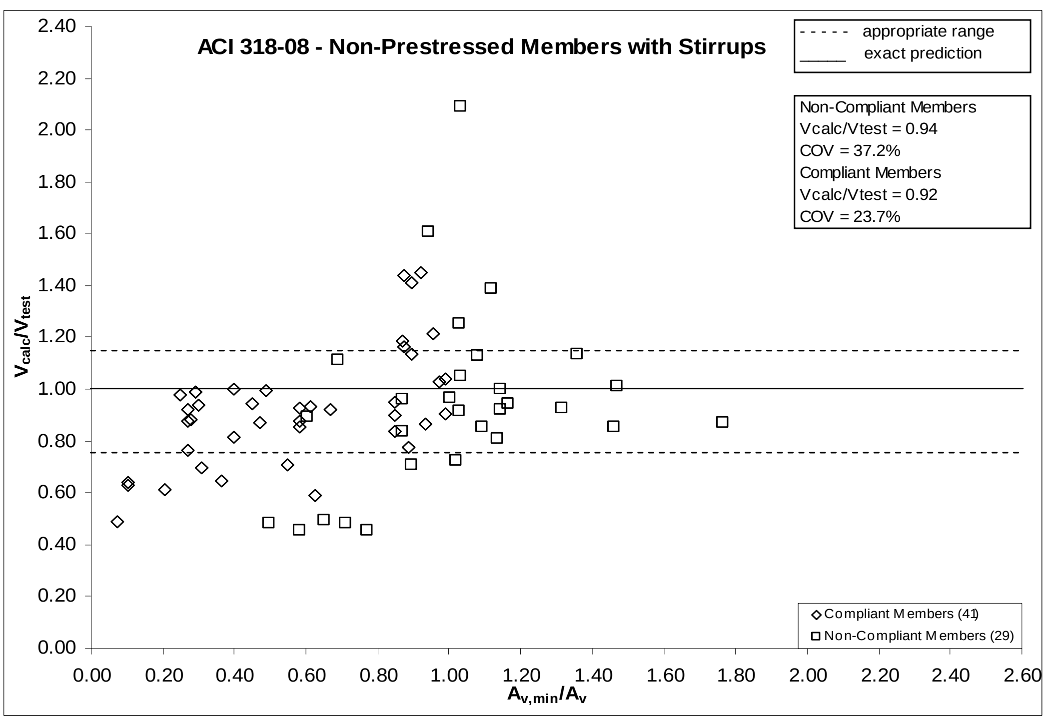



Figure 4.34 - $V_{calc}/V_{test}$ *vs.* $A_{v,min}/A_v$ for 70 Non-Prestressed Members with Stirrup Evaluated using ACI 318-08

Table 4.20 – ACI 318-08 Non-Prestressed Members - $V_{calc}/V_{test}$ Ranges and Demerit Points

| **ACI 318-08 - Non-Prestressed Members with Stirrups** | **Full Data Set** | **Non-Compliant** | **Compliant** | **Demerits - Full Data Set** | **Demerits - Non-Compliant** | **Demerits - Compliant** |
|---|---|---|---|---|---|---|
| Total | 70 | 29 | 41 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 6<br>8.6% | 5<br>17.2% | 1<br>2.4% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 9<br>12.9% | 2<br>6.9% | 7<br>17.1% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 45<br>64.3% | 18<br>62.1% | 27<br>65.9% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 8<br>11.4% | 2<br>6.9% | 6<br>14.6% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 1<br>1.4% | 1<br>3.4% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 1<br>1.4% | 1<br>3.4% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 701 | 389 | 312 |
| Average Demerits/Members | | | | 10.01 | 13.41 | 7.61 |

Bentz (2000) evaluated the shear capacity of 189 non-prestressed members with stirrups using the sectional shear method in ACI 318-99. 63.1% of shear capacity predictions derived from Bentz (2000) for these members result in $V_{calc}/V_{test}$ ratios in the 'appropriate' range, which compares well with the percentage of 'appropriate' predictions for the twenty-nine non-compliant non-prestressed members evaluated in this study, as shown in Table 4.20. A description of Bentz's members and the differences between the sectional shear methods in ACI 318-08 and ACI 318-99 are discussed in Section 3.5. Two members from the non-compliant non-prestressed category of test specimen evaluated in this study are calculated to have predictions of shear capacity in the 'conservative' range; one of these members (Specimen N1-N) had a rectangular section with a longitudinal reinforcement ratio of 2.85% while the other member (Specimen P21) was fabricated with a compression flange. Five of the non-compliant members were classified as 'very conservative'. These members all had small depths and compression flanges, details which are known to result in conservative predictions of shear capacity (Collins, 2000; Moayer and Regan, 1971; Giaccio et al, 2002). Two of the non-compliant non-prestressed members present with 'low safety' predictions, one member has a $V_{calc}/V_{test}$ ratio in the 'dangerous' range and one member has a $V_{calc}/V_{test}$ ratio in the 'extremely dangerous' range. These are all rectangular members with depths greater than 800 mm and with longitudinal reinforcement ratios less than 1.00%. Of the forty-one compliant non-prestressed members evaluated in this study using the sectional shear provisions in ACI 318-08 Section 11 65.9% are in the 'appropriate' range, which is consistent with predictions of shear capacity for the twenty-nine non-compliant non-prestressed members. Shear capacity evaluation of the forty-one compliant non-prestressed specimens examined in this study result in seven members having 'conservative' predictions and one member having a 'very conservative' prediction. These eight compliant non-prestressed members which have $V_{calc}/V_{test}$ ratios less than 0.75 are all sections with compression flanges. Five of the compliant non-prestressed members evaluated in this study are calculated to have predictions of shear capacity in the 'low safety' range while one compliant non-prestressed member has a $V_{calc}/V_{test}$ ratio in the 'dangerous' range. All compliant non-prestressed members presenting with unsafe predictions of shear capacity are rectangular sections with heights of 500 mm or greater and with longitudinal reinforcement ratios of 1.05% or less.

The non-prestressed members in the non-compliant category are allotted 76.2% more demerit points per member (13.41 compared to 7.61) than are the test specimens in the compliant category based on evaluation using the shear provisions in ACI 318-08. The increase in average demerit points per non-compliant member is a result of these specimens presenting with more overly safe and more unsafe predictions of shear capacity than do the compliant non-prestressed specimens. The average demerit points assigned to the compliant members in this study are consistent with other studies. The 106 non-prestressed members with stirrups evaluated by Lubell (2006) are allotted 8.03 average demerit points per member while the ninety-four non-prestressed members with stirrups evaluated by Collins (2001) are allotted 6.38 average demerit points per member. The demerit points for all studies were calculated using the method proposed in Section 4.2.

The considerable scatter resulting from shear capacity predictions of the non-compliant non-prestressed members shown in Figure 4.33 makes it difficult to discern a particular trend in $V_{calc}/V_{test}$ ratios with respect to variations in the $s/s_{max}$ ratio. Figure 4.34

demonstrates a more definable trend, with $V_{calc}/V_{test}$ ratios increasing as the $A_{v,min}/A_v$ ratio increases. This is consistent for both compliant and non-compliant members indicating that as members became less compliant with respect to stirrup area requirements, predictions of shear capacity become progressively more unconservative. Members evaluated to have the most unconservative predictions of shear capacity are large rectangular members with stirrups not meeting minimum area requirements and with low longitudinal reinforcement ratios (approximately 1.05% or less). The five specimens tested by Kani (1967) all present with $V_{calc}/V_{test}$ ratios less than 0.50 which deviates from the rest of the non-compliant non-prestressed category of members. These members were all small, flanged sections with heights of 305 mm and with longitudinal reinforcement ratios of 1.82%. Kani loaded his members with shear span to depth ratios of 5.00 and none failed at greater than 82% of their flexural capacities. Predictions of shear capacity for members YB2000/9 and SB2003/6 also appear to deviate from the rest of the non-compliant non-prestressed data category. Both members had heights of 2000 mm and longitudinal reinforcing ratios of 0.74% and 0.36% respectively. These members were loaded with shear span-to-depth ratios *a/d* of approximately 3.00 and neither failed at greater than 60% of their flexural capacity. $V_{calc}/V_{test}$ ratios as high as 2.09 were calculated for these members, indicating that ACI 318-08 can predict shear capacities for non-compliant non-prestressed members which are 'dangerously' unsafe .

Figure 4.33 does not exhibit any discernable trends for the non-prestressed compliant members, suggesting that stirrup spacing is adequately accounted for by the shear method in ACI 318-08. There is still considerable scatter in $V_{calc}/V_{test}$ ratios for the compliant non-prestressed members, although it is notably less than the scatter shown by the non-compliant non-prestressed category of test specimens. $V_{calc}/V_{test}$ ratios for three of the compliant non-prestressed members evaluated in this study are large enough to suggest that they deviate from the other compliant non-prestressed members evaluated using the shear method in ACI 318-08. Two of these members (DB140M and V1) are discussed in Section 4.5.1. The other member, DBO530M, was a rectangular member with a height of 1000 mm, and with a longitudinal reinforcement ratio of 0.50%. This member was loaded with an *a/d* ratio of 2.92 and failed at 73% of its flexural capacity. Similar to evaluation of non-compliant non-prestressed members, Figure 4.34 indicates that predictions of shear capacity for compliant non-prestressed members become more unsafe as the ratio $A_{v,min}/A_v$ increases.

Evaluation of Prestressed Members not Complying with Stirrup Spacing and Area Details using ACI 318-08 Shear Provisions

The twenty prestressed members not complying with S6-06 Section 14 stirrup spacing requirements were evaluated to assess the influence that variations in the stirrup detail ratios $s/s_{max}$ and $A_{v,min}/A_v$ have on the agreement between predicted and tested shear capacities calculated using ACI 318-08 Section 11 provisions. All prestressed members evaluated in this study complied with S6-06 Section 14 stirrup area requirements. Forty prestressed members complying with S6-06 Section 14 minimum stirrup requirements were evaluated to facilitate a comparison of the agreement between predicted-to-tested shear capacities with the non-compliant members. Figure 4.35 provides the relationship $V_{calc}/V_{test}$ vs. $s/s_{max}$ for the sixty prestressed members with stirrups, while Figure 4.36 provides the relationship $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for the same members. The solid line in these figures represent exact shear predictions, while the two dashed lines define the range of 'appropriate predictions' as given in Section 4.2. Table 4.21 distributes the member predictions into the ranges provided in Table 4.1, and provides the average demerit points per member for the full data set of prestressed members with stirrups, as well as the compliant and non-compliant member categories.

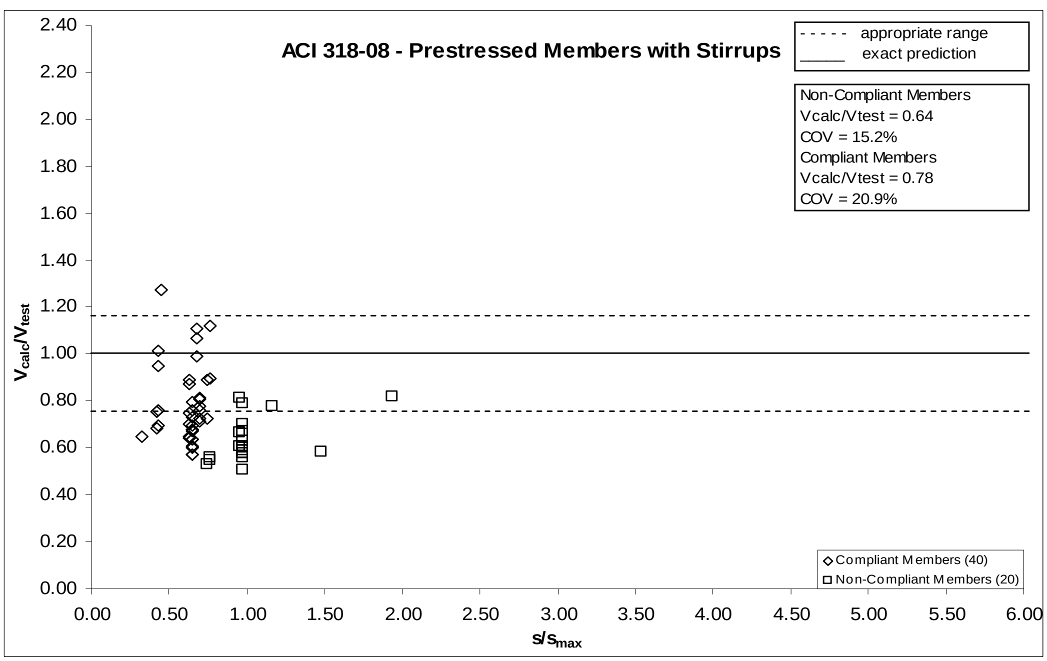



Figure 4.35 - $V_{calc}/V_{test}$ *vs.* $s/s_{max}$ for 60 Prestressed Members with Stirrups Evaluated using ACI 318-08

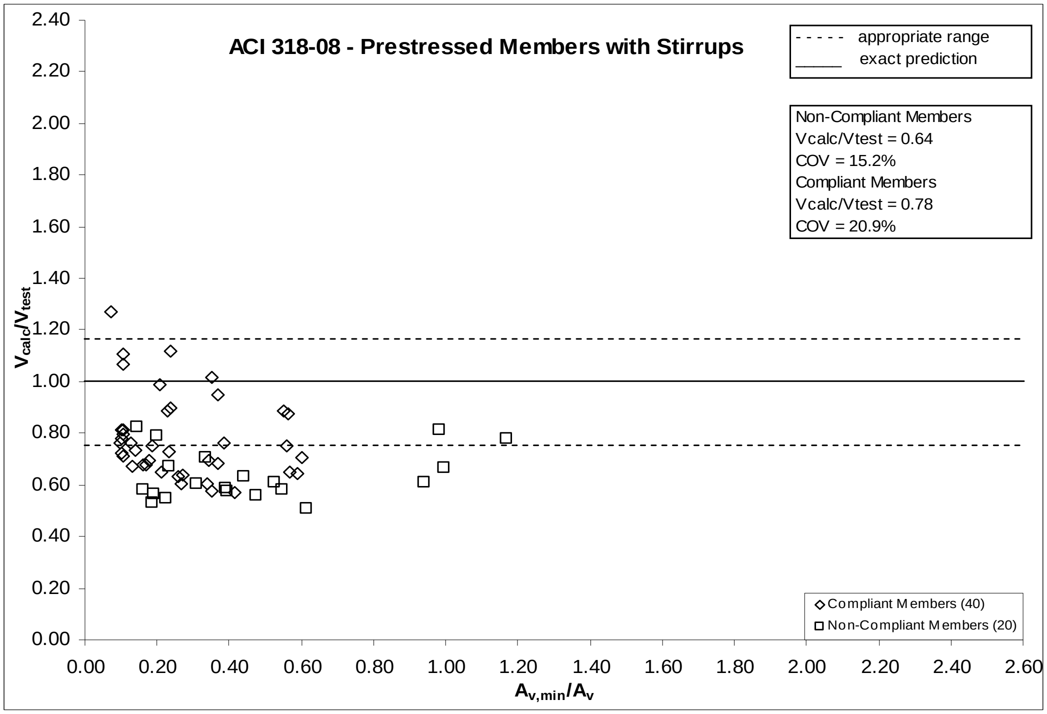



Figure 4.36 - $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for 60 Prestressed Members with Stirrups Evaluated using ACI 318-08

Table 4.21 - ACI 318-08 Prestressed Members - $V_{calc}/V_{test}$ Ranges and Demerit Points

| **ACI 318-08 - Prestressed Members with Stirrups** | **Full Data Set** | **Non-Compliant** | **Compliant** | **Demerits - Full Data Set** | **Demerits - Non-Compliant** | **Demerits - Compliant** |
|---|---|---|---|---|---|---|
| Total | 60 | 20 | 40 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 34<br>56.7% | 14<br>70.0% | 20<br>50.0% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 23<br>38.3% | 4<br>20.0% | 19<br>47.5% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 3<br>5.0% | 2<br>10.0% | 1<br>2.5% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 597 | 263 | 334 |
| Average Demerits/Members | | | | 9.95 | 13.16 | 8.35 |

Evaluation of forty-one prestressed members with stirrups by Collins (2001), using the sectional shear method in ACI 318-95, presented with 65.8% of predictions in the 'appropriate' range, which compares poorly with the percentage of 'appropriate' predictions for the twenty non-compliant prestressed members evaluated in this study, as shown in Table 4.21. It is not clear as to why the difference in 'appropriate' predictions between evaluation of shear capacity in this study and Collins' study exists, but it is believed to be a result of the inappropriately high scatter inherent to shear predictions calculated using the sectional shear method in ACI 318. This scatter is evident in this study and in work by others (Bentz, 2000; Kim, 2004). A description of Collins' members and the differences between the sectional shear methods in ACI 318-08 and ACI 318-95 are discussed in Section 3.5. 47.5% of shear capacity evaluations for the compliant prestressed members examined in this study using the sectional shear method in ACI 318-08 Section 11 are calculated to have $V_{calc}/V_{test}$ ratios in the 'appropriate' range. This percentage of members in the 'appropriate' range compares reasonably well with Collins' shear capacity evaluations of forty-one prestressed concrete members with stirrups, when typical scatter of ACI 318-08 in predictions for compliant prestressed members in the current study and Collins' study (20.9% and 23.0% respectively) is considered. Comparing the distribution of shear capacity predictions of prestressed members shown in Table 4.21 to the distribution of non-prestressed members shown in Table 4.20 suggests that evaluations of sectional shear capacity calculated using provisions in ACI 318-08 Section 11 are more conservative for prestressed members than for non-prestressed members. This is consistent with results in other studies (Collins, 2001; Kim, 2000) and is consistent with evaluation using the other sectional shear methods examined in the present study. It should however be noted that most of the prestressed members evaluated are small, flanged sections. These two details are known to result in lower $V_{calc}/V_{test}$ predictions (Collins, 2000; Moayer and Regan, 1971; Giaccio et al., 2002).

Non-compliant prestressed members are allotted an average of 63.0% more demerit points per member (13.16 compared to 8.35) than are the compliant prestressed members. The increase in demerit points for the non-compliant prestressed members is due to the fact that they present with more conservative predictions of shear capacity than do the compliant prestressed members. The average demerit points per member allotted to the compliant prestressed members in this study are similar to Collins (2001) which are allotted with an average of 7.30 demerit points per member. Average demerit points per member were allotted to Collins' (2001) predictions using the model described in Section 4.2.

The vertical clustering of the data points at an $s/s_{max}$ value equal to 1.00 in Figure 4.35 indicates that the majority of prestressed members containing stirrups evaluated in this study were constructed with the maximum spacing permitted by ACI 318-08. Thus no specific trends in $V_{calc}/V_{test}$ ratios are apparent in Figure 4.35. Figure 4.36 does not exhibit any defined trends in $V_{calc}/V_{test}$ ratios for the non-compliant prestressed members evaluated in this study as the stirrup area ratio $A_{v,min}/A_v$ varies, which suggests that the sectional shear method in ACI 318-08 appropriately accounts for stirrup area in prestressed members. This lack of a defined trend in $V_{calc}/V_{test}$ ratios as the ratio $A_{v,min}/A_v$ varies deviates from behavior shown in Figure 4.34 for non-prestressed members, and is based on a small data set. None of the $V_{calc}/V_{test}$ ratios for the non-compliant prestressed specimens deviate from the rest of that member category (eg. no outliers).

Due to the large scatter of predictions for test results shown in Figures 4.35 and 4.36 no trends in $V_{calc}/V_{test}$ ratios corresponding to variations in the stirrup detail ratios $s/s_{max}$ and $A_{v,min}/A_v$ are apparent for the compliant prestressed category of members. One member prediction appears to deviate from the rest of the prestressed compliant data category. This member (A1-00-1.5R_N) was 1118 mm deep and had 0.35% longitudinal reinforcement. Test specimen A1-00-1.5R_N was loaded with an *a/d* ratio of 2.60 and failed at approximately 62% of its flexural capacity.

### 4.8.2 Evaluation of Members without Shear Reinforcement using ACI 318-08 Shear Provisions

Based on the criteria for member selection discussed in Section 4.3, thirty-three members without stirrups were identified for evaluation in this study. The results are used to assess the agreement between predicted and tested shear capacities, calculated using the sectional shear provisions in ACI 318-08, for members fabricated with no transverse reinforcement. Figure 4.37 shows the relationship $V_{calc}/V_{test}$ vs. $d$ for the thirty-three members. Table 4.22 provides the quantity and percentage of predictions falling in the various statistical ranges (see Table 4.1), as well as the average demerit points per member corresponding to predictions.

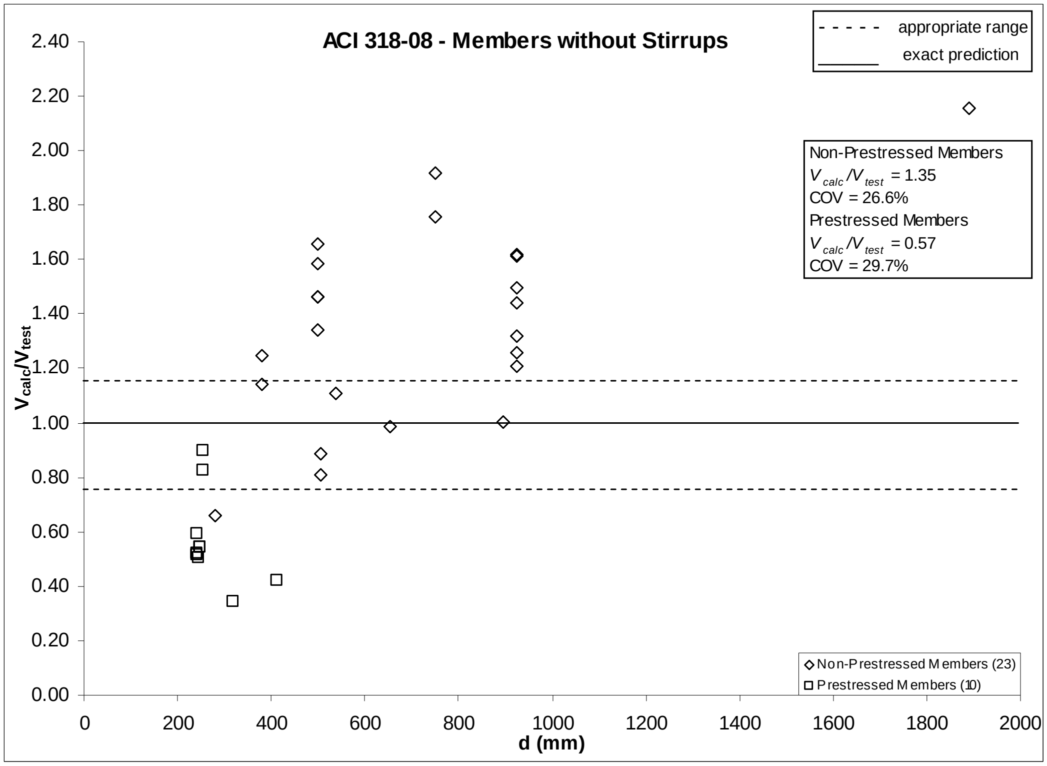



Figure 4.37 - $V_{calc}/V_{test}$ *vs. d* for 33 Identified Members without Shear Reinforcement Evaluated using ACI 318-08

Table 4.22 – ACI 318-08 – Members without Stirrups - $V_{calc}/V_{test}$ Ranges and Demerit Points

| ACI 318-08 - Members without Stirrups | Full Data Set | Non-Prestressed | Prestressed | Demerits - Full Data Set | Non-Prestressed | Prestressed |
|---|---|---|---|---|---|---|
| Total | 33 | 23 | 10 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 2<br>6.1% | 0<br>0.0% | 2<br>20.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 7<br>21.2% | 1<br>4.3% | 6<br>60.0% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 8<br>24.2% | 6<br>26.1% | 2<br>20.0% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 9<br>27.3% | 9<br>39.1% | 0<br>0.0% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 6<br>18.2% | 6<br>26.1% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 1<br>3.0% | 1<br>4.3% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 999 | 834 | 165 |
| Average Demerits/Members | | | | 30.27 | 36.27 | 16.45 |

Table 4.22 indicates that of the twenty-three non-prestressed members without stirrups evaluated in this study, only six present with $V_{calc}/V_{test}$ ratios in the range defined as 'appropriate'. Of the remaining seventeen members, sixteen fall in ranges deemed unsafe in this study (see Table 4.1). The primary parameter influencing predicted-to-tested shear capacity ratios is the overall section height, as members without shear reinforcement are not able to control crack spacing adequately (Collins and Kuchma, 1999). The shear method in ACI 318-08 does not address this issue (known as 'size effect in shear') and as such its predictions are considerably skewed by variations in member height as is shown in Figure 4.37. Prestressed members present with considerably more conservative predictions of shear capacity although these members all had depths less than 500 mm. Six of the ten prestressed members had predictions in the `conservative` range, while two had predictions in the `very conservative` range. This further indicates that ACI 318-08 shear provisions calculate more conservative predictions for prestressed members than for non-prestressed members. This is consistent with predictions of shear capacity for members without stirrups by Kim (2004).

Non-prestressed members without stirrups evaluated in this study are allotted an average of 120.5% more demerit points per member (36.27 compared to 16.45) than are the prestressed members without stirrups. This is in contrast to the other shear methods used in this study, where non-prestressed members are alloted considerably fewer average demerit points per member. The higher number of average demerit points per member attributed to the predictions of non-prestressed members without stirrups than to the prestressed members without stirrups, evaluated using ACI 318-08 shear provisions, is a result of these provisions not properly accounting for overall section height.

Figure 4.37 demonstrates a clear trend of fewer safe shear predictions among non-prestressed members as their depth increases. This behavior is noted in other studies (Kani, 1967; Collins and Kuchma, 1999; Kim, 2004). The highest $V_{calc}/V_{test}$ ratio calculated for the non-prestressed members is 2.16, which falls in the range deemed 'extremely dangerous' in this study (see Table 4.1). This prediction is for a girder with a height of 2000 mm (Member YB2000/0). No trends are discernable in Figure 4.39 for the ten prestressed members without stirrups evaluated in this study. These ten members had small depths, which limited the range of useful data. The predicted shear capacity of two prestressed members not containing stirrups present in the 'very conservative' range, which deviates from the rest of this test category of members. These test specimens, designated CI8 and CW8 and tested by Elzanaty et al. (1986), had heights of 356 mm and 457 mm respectively with corresponding longitudinal reinforcement ratios of 0.60% and 1.05%. The two deviating members had shear span to depth ratios *a/d* ranging from 3.80 to 5.80 and neither member failed at greater than 40% of the specimen's corresponding flexural capacity. The significant trend of increasing $V_{calc}/V_{test}$ ratios as the member depth increases indicates that the shear method in ACI 318-08 is not appropriate for predicting the shear capacity of members without stirrups.

### 4.8.3 Parametric Sensitivity Analysis of Shear Predictions Made using the Shear Provisions in ACI 318-08

Section 4.8.3 assesses the influence that variations of the parameters discussed in Section 2.5 have on the agreement between predicted to tested shear capacities calculated using the sectional shear method in ACI 318-08. As described in Section 2.5, variations in concrete strength $f'_c$, shear span to depth ratio $a/d$, longitudinal reinforcement ratio $\rho$, and member shape (flanged members vs. rectangular members) are known to affect the shear capacity of concrete members. These parameters are studied against the forty-nine non-compliant members evaluated in this study to assess whether they are appropriately accounted for by the sectional shear method in ACI 318-08 Section 11. The eighty-one compliant members and thirty-three members without stirrups are included to provide a comparison of $V_{calc}/V_{test}$ ratios with the non-compliant members.

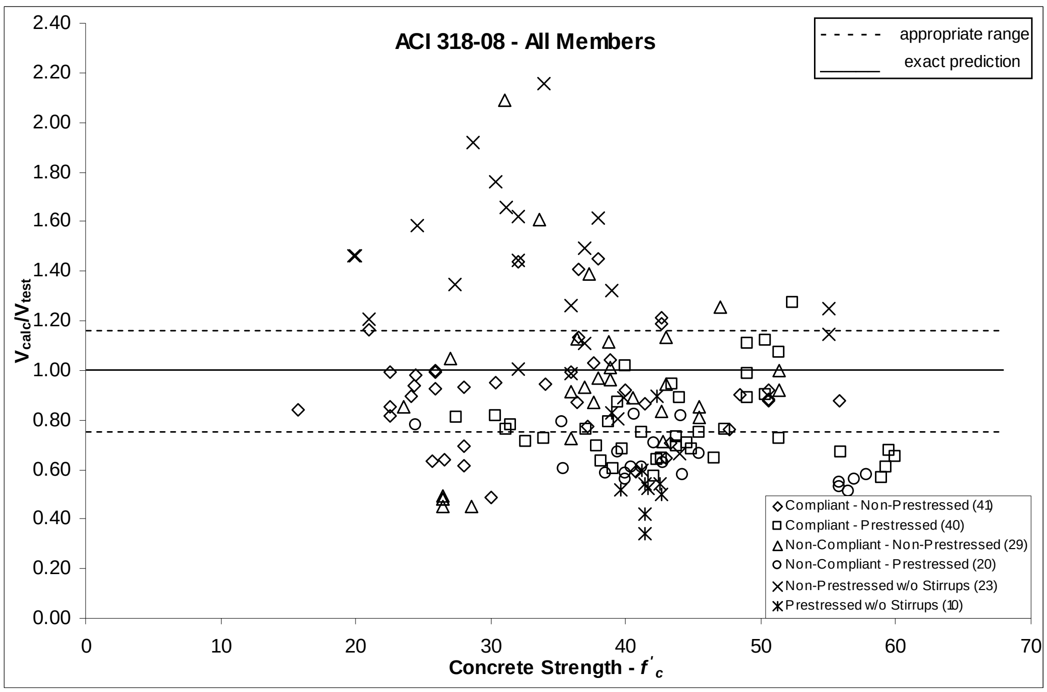



Figure 4.38 - $V_{calc}/V_{test}$ *vs.* $f'_c$ for 163 Members Evaluated using ACI 318-08

Figure 4.38 shows the relationship between predicted-to-tested shear capacities and the specified concrete strength for the 163 members evaluated in this study. Due to the large scatter in predicted to tested shear capacity ratios calculated using the sectional shear method in ACI 318-08, this figure is inclusive for any definable trends in $V_{calc}/V_{test}$ ratios as the concrete strength varies for any of the data categories. This is similar to behavior seen in Kim (2004).

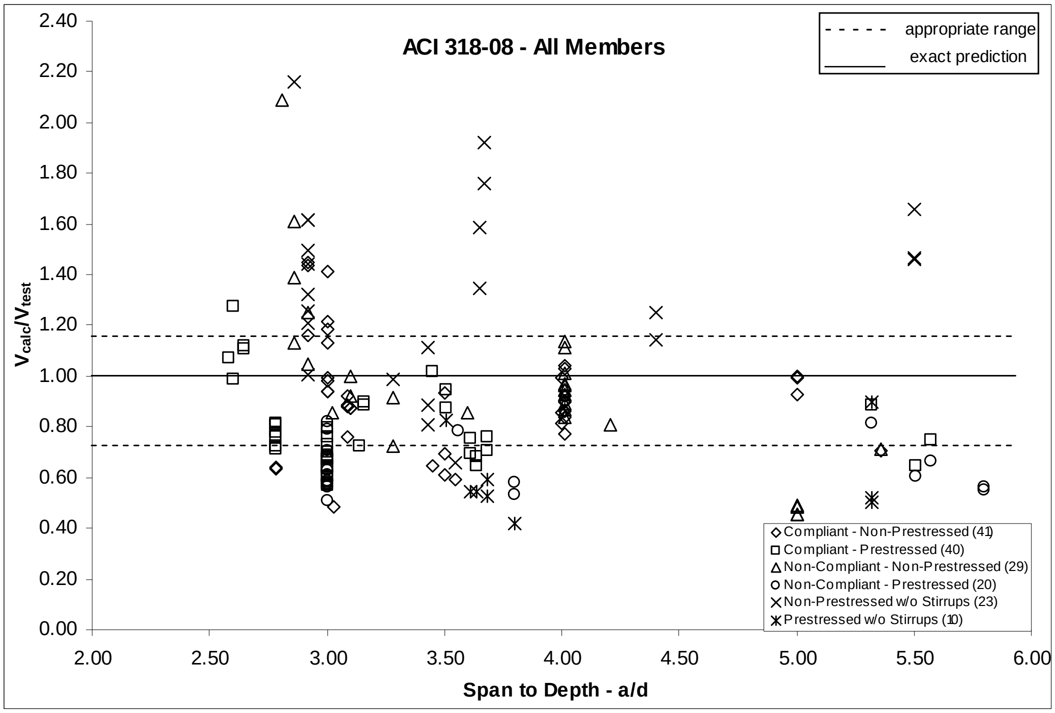


Figure 4.39 - $V_{calc}/V_{test}$ *vs. a/d* for 163 Members Evaluated using ACI 318-08

Figure 4.39 provides the relationship between predicted-to-tested shear capacities and the shear span-to-depth ratio *a/d* for the 163 members identified for evaluation in this study. The horizontal distribution of $V_{calc}/V_{test}$ ratios plotted in Figure 4.39 shows considerable scatter, which makes determining whether variations in *a/d* ratios are appropriate accounted for by the ACI 318-08 shear method inconclusive. This is similar to observations in other research (Kim, 2004).

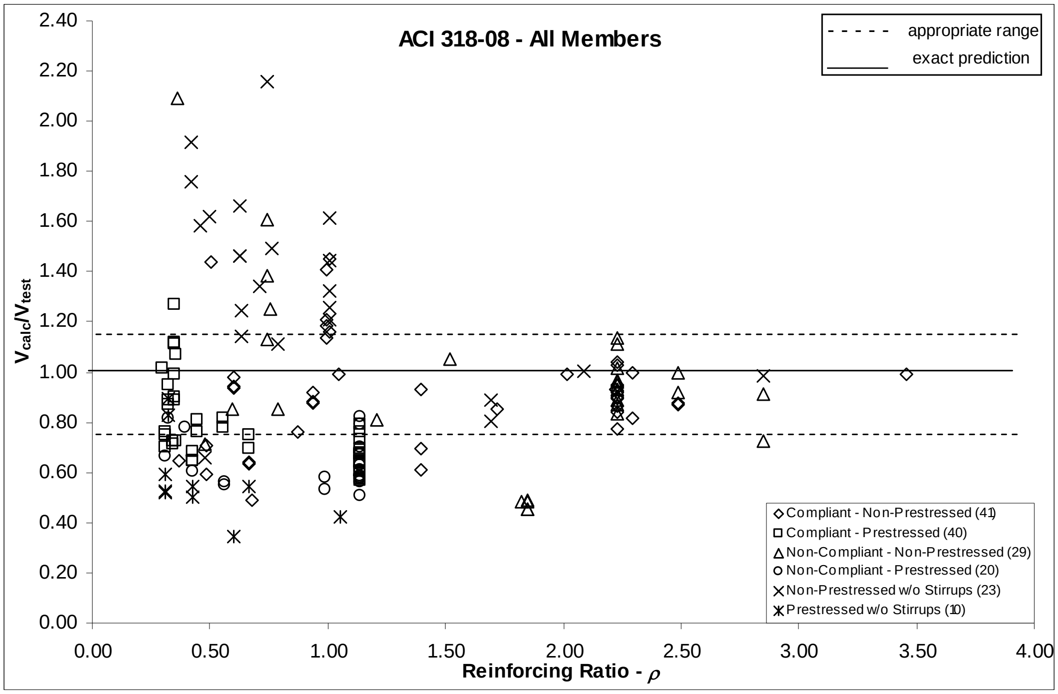



Figure 4.40 - $V_{calc}/V_{test}$ *vs.* $\rho$ for 163 Members Evaluated using ACI 318-08

Figure 4.40 provides the relationship between $V_{calc}/V_{test}$ ratios and the longitudinal reinforcement ratio for the 163 members evaluated in this study. $V_{calc}/V_{test}$ ratios in this figure show a propensity to increase as the longitudinal reinforcement ratio decreases, indicating that the ACI 318-08 shear method can be unconservative for members with low longitudinal reinforcement ratios. This is noted in Section 4.8.1 and is consistent with research by Kim (2004).

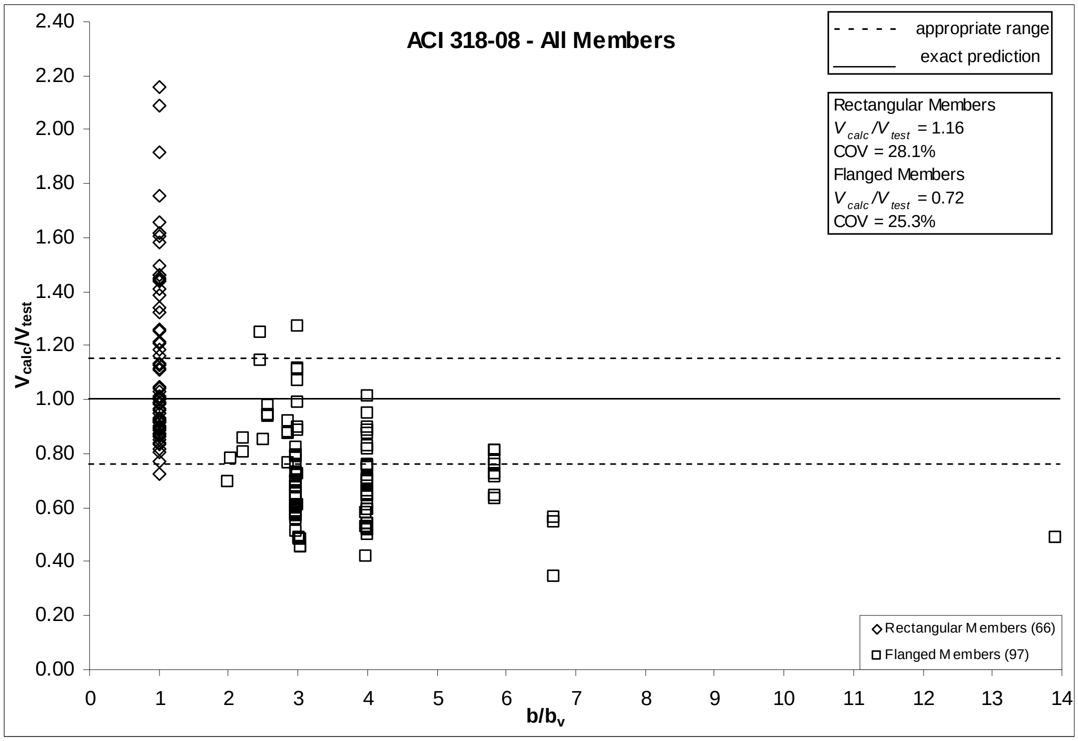



Figure 4.41 - $V_{calc}/V_{test}$ *vs.* $b/b_v$ for 163 Members Evaluated using ACI 318-08

Figure 4.41 provides the relationship between predicted-to-tested shear capacities calculated using the sectional shear method in ACI 318-08 and the member flange width to web width ratio $b/b_v$ for the 163 members identified for evaluation in this study. Similar to predictions using the Canadian Highway Bridge Design Code S6-06 (see Figure 4.14), Figure 4.41 indicates that typical $V_{calc}/V_{test}$ ratios are smaller for flanged members than for rectangular members. This increase in reserve capacity for members with compression flanges is similar to results in other studies (Placas and Regan, 1971; Giaccio et al, 2002). The rectangular non-compliant members are calculated to have a mean $V_{calc}/V_{test}$ ratio of 1.25 and a COV of 31.1%, while non-compliant flanged members are calculated to have a mean $V_{calc}/V_{test}$ ratio of 0.63 and a COV of 20.2%. No well defined variations in $V_{calc}/V_{test}$ ratios are identified for flanged members as the $b/b_v$ ratio varies. As discussed in Section 4.5.3, member T1 had its effective flange width reduced as per S6-06 Clause 5.8.2.1.

### 4.8.4 Summary of Shear Predictions using the ACI 318-08 Shear Method

Table 4.23 provides results from statistical analyses and the average demerit points per member from the sectional shear capacity evaluation of the 163 members evaluated in this study using the provisions in ACI 318-08 Section 11.

Table 4.23 - Results using of ACI 318-08 Sectional Shear Provisions

| Test Group (number) | $V_{calc}/V_{test}$ Mean | $V_{calc}/V_{test}$ C.O.V (%) | Average Demerit Points / Member | $V_{test}/V_{calc}$ Mean | $V_{test}/V_{calc}$ COV |
|---|---|---|---|---|---|
| All Members (163) | 0.90 | 36.7 | 14.19 | 1.25 | 33.3 |
| Non-Compliant Non-Prestressed Members (29) | 0.94 | 37.2 | 13.41 | 1.21 | 38.7 |
| Compliant Non-Prestressed Members (41) | 0.92 | 23.7 | 7.61 | 1.15 | 25.0 |
| Non-Prestressed Members without Stirrups (23) | 1.35 | 26.6 | 36.27 | 0.80 | 31.0 |
| Non-Compliant Prestressed Members (20) | 0.64 | 15.2 | 13.16 | 1.60 | 13.9 |
| Compliant Prestressed Members (40) | 0.78 | 20.9 | 8.35 | 1.32 | 18.1 |
| Prestressed Members without Stirrups (10) | 0.57 | 29.7 | 16.45 | 1.88 | 27.5 |
| All Rectangular Members (66) | 1.16 | 28.1 | 19.41 | 0.92 | 23.6 |
| All Flanged Members (97) | 0.72 | 25.3 | 10.16 | 1.48 | 24.9 |

NOTE: Vtest/Vcalc values for the data set evaluated in this study were included to allow direct comparison to results by Kim (2004) in Section 3.5.

The statistical values for the non-compliant non-prestressed members presented in this study, calculated based on evaluations using ACI 318-08 shear provisions and shown in Table 4.23, are consistent with statistical results of non-prestressed members with stirrups derived from other studies and provided in Section 3.5, although the COV falls at the higher range of these statistical results. While the non-compliant non-prestressed members evaluated in this study present with an average $V_{calc}/V_{test}$ ratio appropriately close to unity, the corresponding COV value is inappropriately high. This considerable scatter of $V_{calc}/V_{test}$ ratios is reflected in the high average demerit points allotted to the non-compliant non-prestressed members, which exceeds the 'appropriate' limit discussed in Section 4.2 by 78.8% (13.41 compared to 7.50).

The twenty non-compliant prestressed members evaluated in this study using the sectional shear provisions in ACI 318-08 Section 11 present with a mean $V_{calc}/V_{test}$ ratio of 0.64 and a coefficient of variation of 15.2%. This mean $V_{calc}/V_{test}$ ratio is considerably lower than the corresponding ratio for the forty compliant (0.78). The $V_{calc}/V_{test}$ ratio from the non-compliant prestressed members is smaller than the $V_{calc}/V_{test}$ ratio calculated for this data category by the other methods used for evaluations in this study. The small $V_{calc}/V_{test}$ ratio for the non-compliant prestressed members is reflected by the large value of average demerit points allotted to these specimens; the non-compliant prestressed members exceed the 'appropriate' limit of average demerit points per member by 75.5% (13.16 compared to 7.50).

The twenty-three non-prestressed members without stirrups evaluated using ACI 318-08 shear provisions calculated a mean $V_{calc}/V_{test}$ ratio of 1.35 and a coefficient of variation of 26.6%. These values are within the ranges of statistical values derived from other studies and discussed in Section 3.5.

Predictions of shear capacity calculated using ACI 318-08 shear provisions show poor agreement with tested shear capacities for members with non-compliant stirrup spacing and area details. This is primarily due to the fact that ACI 318-08 does not account for diagonal crack spacing for members loaded critically in shear. Figure 4.34 indicates that the influence of varying stirrup area is not appropriately accounted for by the sectional shear method in ACI 318-08, and that predictions of shear capacity become more unsafe as the $A_{v,min}/A_v$ ratios increases. Figure 4.40 indicates that provisions for shear in ACI 318-08 Section 11 do not adequately account for the longitudinal reinforcement ratio, as members having percentages of longitudinal steel less than approximately 1.00% are more likely to present with unconservative predictions of shear capacity. All member categories evaluated using the sectional shear method in ACI 318-08 are allotted with greater than 7.50 average demerit points per member, which is the upper limit recommended as 'appropriate' in Section 4.2. ACI 318-08 is the only method used in this study which consistently exceeds this limit. Based on these findings this study does not recommend using the sectional shear method in ACI 318-08 for predicting the shear capacity of members with non-compliant stirrup spacing and area details or for members without stirrups. ACI 318-08 is able to typically give 'appropriate' predictions of sectional shear capacity for members with stirrups which comply with the minimum transverse reinforcement requirements in ACI 318-08 Section 11. These predictions however are consistently not in as good agreement with actual shear capacities as are predictions calculated using sectional shear methods which are based on the Modified Compression Field Theory (Vecchio and Collins, 1986)

## 4.9 Comparison of Methods

A detailed discussion of the agreement between predicted and tested shear capacities, calculated using the four sectional shear evaluation methods discussed in Chapter 3, has been provided in Sections 4.5 through 4.8. Section 4.9 provides a summary of the mean $V_{calc}/V_{test}$ ratios, COV and average demerit points per member, calculated using the four sectional shear methods, for the 163 members identified for this study. This section also assesses the four sectional shear evaluation methods against the criteria identified in Section 2.7 to determine which method is the most appropriate for predicting the shear capacity of members with non-compliant stirrup spacing and area details.

Table 4.24 summarizes the mean $V_{calc}/V_{test}$ ratios, COV and average demerit points per member presented in Sections 4.5 through 4.8 for the six member categories provided in Figure 4.2. Tables 4.25, 4.26 and 4.27 distribute shear capacity predictions of the forty-nine non-compliant members, eighty-one compliant members and thirty-three members without stirrups respectively, calculated using the four sectional shear methods discussed in Chapter 4, into the classifications provided in Table 4.1.

Table 4.24 – Comparison of Sectional Shear Evaluation Methods – 163 Members

| Member Type | | NP | NP | NP | P | P | P |
|---|---|---|---|---|---|---|---|
| | | $A_{v,C}$ | $A_{v,NC}$ | No $A_v$ | $A_{v,C}$ | $A_{v,NC}$ | No $A_v$ |
| Number of Members | | 41 | 29 | 23 | 40 | 20 | 10 |
| CSA S6-06 | Mean $V_{calc}/V_{test}$ | 0.93 | 0.80 | 0.96 | 0.78 | 0.80 | 0.66 |
| | C.O.V (%) | 16.3 | 20.8 | 13.0 | 16.6 | 14.7 | 11.9 |
| | DPm | 3.91 | 6.84 | 2.96 | 7.49 | 6.02 | 11.84 |
| AASHTO LRFD-05 | Mean $V_{calc}/V_{test}$ | 0.98 | 0.71 | 0.90 | 0.78 | 0.77 | 0.62 |
| | C.O.V (%) | 17.7 | 19.0 | 14.2 | 17.2 | 17.4 | 10.1 |
| | DPm | 5.68 | 9.53 | 2.89 | 7.61 | 7.06 | 13.80 |
| Response 2000 | Mean $V_{calc}/V_{test}$ | 0.95 | 0.86 | 0.95 | 0.85 | 0.77 | 0.74 |
| | C.O.V (%) | 14.2 | 17.4 | 10.8 | 18.4 | 20.1 | 22.4 |
| | DPm | 3.55 | 4.12 | 1.82 | 6.01 | 7.46 | 10.10 |
| ACI 318-08 | Mean $V_{calc}/V_{test}$ | 0.92 | 0.94 | 1.35 | 0.78 | 0.64 | 0.57 |
| | C.O.V (%) | 23.7 | 37.2 | 26.6 | 20.9 | 15.2 | 29.7 |
| | DPm | 7.61 | 13.41 | 36.27 | 8.35 | 13.16 | 16.45 |

*NOTE –$A_v$ indicates data set contains members with stirrups and No $A_v$ indicates data set contains only members without stirrups. The subscripts NC and C designate non-compliant members and compliant members respectively. DPm represents the average demerit points per member. NP designates non-prestressed members while P designates prestressed members.*

Table 4.25 – Distribution of Shear Predictions (%) – 49 Non-Compliant Members

| $V_{calc}/V_{test}$ | <0.50 | 0.5~0.75 | 0.75~1.15 | 1.15~1.50 | 1.50~2.00 | > 2.00 | Average Demerit Points per Members |
|---|---|---|---|---|---|---|---|
| Classification | Extremely Conservative | Conservative | Appropriate | Low Safety | Dangerous | Extremely Dangerous | |
| S6-06 | 2.0 | 42.9 | 55.1 | 0.0 | 0.0 | 0.0 | 6.51 |
| AASHTO | 2.0 | 57.1 | 40.8 | 0.0 | 0.0 | 0.0 | 8.52 |
| R2K | 0.0 | 32.7 | 67.3 | 0.0 | 0.0 | 0.0 | 5.49 |
| ACI | 10.2 | 32.7 | 44.9 | 8.2 | 2.0 | 2.0 | 13.31 |

Table 4.26 – Distribution of Shear Predictions (%) – 81 Compliant Members

| $V_{calc}/V_{test}$ | <0.50 | 0.5~0.75 | 0.75~1.15 | 1.15~1.50 | 1.50~2.00 | > 2.00 | Average Demerit Points per Members |
|---|---|---|---|---|---|---|---|
| Classification | Extremely Conservative | Conservative | Appropriate | Low Safety | Dangerous | Extremely Dangerous | |
| S6-06 | 0.0 | 32.1 | 63.0 | 4.9 | 0.0 | 0.0 | 5.68 |
| AASHTO | 0.0 | 30.9 | 59.3 | 9.9 | 0.0 | 0.0 | 6.63 |
| R2K | 0.0 | 14.8 | 79.0 | 6.2 | 0.0 | 0.0 | 4.77 |
| ACI | 1.2 | 33.3 | 56.8 | 8.6 | 0.0 | 0.0 | 7.98 |

Table 4.27 – Distribution of Shear Predictions (%) – 33 Members without Stirrups

| $V_{calc}/V_{test}$ | <0.50 | 0.5~0.75 | 0.75~1.15 | 1.15~1.50 | 1.50~2.00 | > 2.00 | Average Demerit Points per Members |
|---|---|---|---|---|---|---|---|
| Classification | Extremely Conservative | Conservative | Appropriate | Low Safety | Dangerous | Extremely Dangerous | |
| S6-06 | 0.0 | 27.3 | 72.7 | 0.0 | 0.0 | 0.0 | 5.65 |
| AASHTO | 0.0 | 42.4 | 57.6 | 0.0 | 0.0 | 0.0 | 6.19 |
| R2K | 0.0 | 21.2 | 78.8 | 0.0 | 0.0 | 0.0 | 4.33 |
| ACI | 6.1 | 21.2 | 24.2 | 27.3 | 18.2 | 3.0 | 30.27 |

As discussed in Section 2.7, this study identifies the following criteria as being critical for a method to address in order to declare that the method is suitable for predicting sectional shear capacity of concrete members. The paragraphs to follow are organized to discuss each criterion.

- Ability to calculate shear capacity quickly and efficiently at various vertical cross sections along the length of a member;
- Predicted-to-test capacity ratios appropriately close to 1.00 and with a low coefficient of variation (COV);
- Low value of assigned demerit points per member calculated using the model proposed in Section 4.2
- No influence on $V_{calc}/V_{test}$ ratios due to variations of the stirrup detail ratios $s/s_{m1}$ and $A_{v,min}/A_v$; and

- No influence on $V_{calc}/V_{test}$ ratios due to variations in concrete strength $f_c'$, shear span-to-depth ratio $a/d$, longitudinal reinforcement ratio $\rho$ and the cross-sectional geometry of the section (rectangular or flanged members). These four parameters are discussed in Section 2.5.

Load evaluation of concrete members requires the shear capacity to be checked at critical sections along a member, especially for members subject to moving loads or with changes in geometric and reinforcing details. It is therefore desirable that any sectional shear evaluation method be easily incorporated into spreadsheets or other software. The sectional design method for shear in S6-06 and ACI 318-08 are both easy to implement into spreadsheets. The General Method for shear in AASHTO LRFD-05 can be implemented into spreadsheet form, but this requires considerably more complex calculations to interpolate values for the shear parameters $\beta$ and $\theta$, as discussed in Section 3.3. Response 2000 is not able to provide shear predictions at various locations along the length of the member in an efficient manner, but is able to quickly provide a prediction of shear capacity at a single critical section.

A comparison of $V_{calc}/V_{test}$ ratios in Table 4.25 indicates that the sectional shear methods in S6-06 Section 8 and AASHTO LRFD-05 predict notably more conservative average $V_{calc}/V_{test}$ ratios for non-prestressed members with non-compliant stirrup spacing and area details than for non-prestressed members with stirrups that comply with S6-06 Section 14 minimum requirements. As discussed in Sections 4.5.1 and 4.6.1, these conservative predictions are a result of the diagonal crack spacing assumptions used in S6-06 and AASHTO LRFD-05. Predictions of shear capacity for the forty-one non-prestressed members and forty prestressed members complying with S6-06 Section 14 minimum stirrup requirements, calculated by applying the four sectional shear methods used in this study, present with average $V_{calc}/V_{test}$ ratios which are in good agreement with $V_{calc}/V_{test}$ ratios derived from other studies as discussed in Chapter 3. Coefficients of variation (COV) for all members evaluated in this study are typically in good agreement with COV values derived from other studies and provided in Chapter 3. Table 4.24 shows that COV values resulting from evaluations of shear capacity using the provisions in ACI 318-08 Section 11 are typically higher than the COV values determined from evaluations using the shear methods in S6-06 Section 8, AASHTO LRFD-05 Section 5 and software Response 2000 (Bentz, 2000). As shown in Table 4.25, ACI 318-08 is the only sectional shear method to present with unsafe predictions for non-compliant members. ACI 318-08 shear provisions typically also calculate unsafe predictions of shear capacity for members without stirrups as discussed in Section 4.8.2. This study does not recommend using the sectional shear method in ACI 318-08 Section 11 for calculating the shear capacity of these non-compliant members and members without stirrups. As discussed in Section 4.8.4, ACI 318-08 is able to typically give 'appropriate' predictions of sectional shear capacity for members with stirrups which comply with the minimum transverse reinforcement requirements in ACI 318-08 Section 11. The good agreement between predicted and tested shear capacities indicates that the sectional shear methods in S6-06, AASHTO LRFD-05 and software Response 2000 are appropriate for predicting the shear capacity of concrete members not complying with minimum transverse reinforcement requirements, as well as members complying with minimum stirrup requirements and members without stirrups.

The sectional shear provisions in S6-06, AASHTO LRFD-05 and ACI 318-08 are all allotted with more average demerit points per member, calculated using the demerit point model proposed in Section 4.2, for non-compliant members than for members complying with S6-06 Section 14 minimum stirrup requirements. The forty-nine non-compliant members evaluated using the sectional shear method in S6-06 present with 14.6% more average demerit points per member (6.51 compared to 5.68) than did the eighty-one compliant members. This compares favorably to the corresponding percent difference of average demerit points per member calculated based on evaluation results using AASHTO LRFD-05 and ACI 318-08, which are 28.5% (8.52 compared to 6.63) and 66.8% (13.31 compared to 7.98) respectively. Sectional shear capacity evaluation of the forty-nine non-compliant members using S6-06 presents with 30.9% (8.52 compared to 6.51) and 104.5% (13.31 compared to 6.51) fewer average demerit points per member than do AASHTO LRFD-05 and ACI 318-08 respectively, indicating that S6-06 typically provides $V_{calc}/V_{test}$ ratios closer to 1.00 than do the other two shear methods discussed. Average demerit points per member allotted to predictions of shear capacity calculated using Response 2000 are consistently smaller than are average demerit points allotted to the other sectional methods examined in this study. Average demerit points per member allotted to predictions of shear capacity determined using ACI 318-08 for members without stirrups are considerably higher than for any other member category using any alternative method. This is due to ACI 318-08 shear provisions not appropriately accounting for the depth of members without stirrups, which results in the unsafe predictions of shear capacity. The Demerit Points model, proposed in Section 4.2, further supports this study's assertion that the shear methods in S6-06, AASHTO LRFD-05 and Response 2000 are appropriate for evaluation of members with non-compliant stirrup spacing and area details, whereas use of the ACI 318-08 shear method is not recommended.

The sectional shear methods in S6-06, AASHTO LRFD-05 and computer program Response 2000 (Bentz, 2000) appear to account appropriately for stirrup spacing and area details. Variations of $V_{calc}/V_{test}$ ratios with respect to the average $V_{calc}/V_{test}$ ratio are believed to be the result of other factors, such as the diagonal crack spacing assumption employed by the sectional shear provisions in S6-06 and AASHTO LRFD-05 and the influence of member shape (rectangular shape vs. flanges members). These factors are examined further in Chapter 5. The lack of trends in $V_{calc}/V_{test}$ values as the stirrup detail ratios vary further indicates that the sectional shear methods used in S6-06, AASHTO LRFD-05 and computer program Response 2000 (Bentz, 2000) are appropriate for predicting the shear capacity of members with non-compliant stirrup spacing and area details. The shear method in ACI 318-08 appears to account correctly for variations in stirrup spacing, but evaluations of shear capacity in this study indicate that as a member becomes more non-compliant with respect to stirrup area, the resulting ratios of predicted to tested shear capacities increase. Over-predicting the shear capacity of concrete members can lead to unsafe decisions made by engineers as discussed in Section 1.1. This further demonstrates that the sectional shear method in ACI 318-08 Section 11 is not appropriate for predicting the shear capacity of concrete members not complying with minimum transverse reinforcement requirements.

Consistent with other studies (Angelakos, 1999; Kim, 2004; Bentz, 2000), evaluation of the 163 members identified for this study indicates that changes in concrete strength $f_c'$, shear span to depth ratio $a/d$, and the longitudinal reinforcement ratio $\rho$ are correctly accounted for by the sectional shear methods in S6-06, AASHTO LRFD-05 and software Response 2000. As such variations in these parameters will not have a notable influence on the agreement between predicted and tested shear capacities of members not complying with S6-06 Section 14 minimum stirrup requirements calculated using methods derived from the Modified Compression Field Theory (Vecchio and Collins, 1986). Similar to other studies (Moayer and Regan, 1971; Giaccio et al, 2002), members with compression flanges are found to present with lower $V_{calc}/V_{test}$ ratios than do rectangular members. Conservative predictions of shear capacity for flanged members are typical to all four sectional shear methods used in this study and are not a safety issue, although neglecting shear capacity can lead engineers to make uneconomical decisions in practice. ACI 318-08 predicts with more unsafe shear capacities for members with low longitudinal reinforcing ratios which is consistent with results in Kim (2004) and is typical for the compliant members, non-compliant members and members without stirrups evaluated in this study.

Based on the criteria discussed in Section 2.7 and reiterated earlier in this Section, this study recommends that the sectional shear method employed by S6-06 Section 8 is the most appropriate method for one-way shear capacity evaluation of concrete girders with non-compliant stirrup spacing and area details. Based on the shear capacity evaluation of 1363 non-prestressed and prestressed concrete members, Kim (2004) also concluded that the A23.3-04 shear method, which is similar to that used in S6-06, was preferred over those in AASHTO LRFD and ACI 318. Software Response 2000 was not used in Kim's study. Evaluation in this study indicates that the shear method in S6-06 provides appropriate and safe predictions for the forty-nine non-compliant members evaluated, as well as for the eighty-one compliant members and thirty-three members without shear reinforcement that were evaluated.

## 4.10 Possible Modifications to the Sectional Shear Method in S6-06 Section 8

Evaluation of sectional shear capacity for non-compliant members using the provisions in S6-06 Section 8 demonstrates two systematic deficiencies which warrant further investigation in this study. The first deficiency noted is the conservatism of predictions for non-compliant members with heights greater than 800 mm. The sectional shear method in S6-06 assumes that, for evaluation of existing structures, the longitudinal spacing of the diagonal cracks is equal to the shear depth $d_v$ for members not complying with S6-06 Section 14 stirrup spacing and area details. As member depth continues to increase, this assumption has a tendency to become overly punitive. The crack spacing assumption also leads to the issue of non-convergence in the predictions of shear capacity discussed in Section 3.2. The problem of non-convergence is more significant as member depth increases, as the diagonal crack spacing assumption for non-compliant members in S6-06 Section has a larger impact on predicted shear capacities of members with greater section height. The second deficiency is the prevalence of conservative predictions of shear capacity for flanged members, calculated when using the shear method in S6-06. As shown in Figure 4.14, flanged members evaluated in this study are calculated to have an average $V_{calc}/V_{test}$ ratio which is 17.9% lower (0.78 compared to 0.95) than for rectangular members.

Chapter 5 proposes modifications to the sectional shear method in S6-06 Section 8 with the intent of mitigating these two deficiencies. The first modified shear method proposes to fix the issues related to the diagonal crack spacing term by assuming that the longitudinal spacing of the diagonal cracks $s_z$ is equal to the longitudinal spacing of the stirrups. As discussed in Section 2.4.2.i this assumption of diagonal crack spacing was proposed by Lubell (2006) for members complying with minimum stirrup requirements. The second modified shear method proposes to address the reserve shear capacity exhibited by flanged members' by including a portion of the flange area in the calculation of the concrete's contribution to shear capacity $V_c$. Assumptions such as this have been used by others (Tureyan et al., 2006; Zsutty as cited in ASCE-ACI 426, 1973).

# Chapter 5
# Modified Sectional Shear Provisions Based on S6-06

## 5.1 Introduction

As discussed in Section 4.10, evaluations of shear capacity calculated using the sectional shear method in S6-06 Section 8 for the forty-nine members with non-compliant stirrup spacing and area details identified for this study demonstrated deficiencies related to 1) the assumed diagonal crack spacing and 2) the cross-sectional geometry of the member (rectangular vs. flanged members). To address these two deficiencies, two modified methods for predicting one-way shear capacity, based on the sectional provisions in S6-06 Section 8, have been developed in this study. The first modified method, named S6-06 M, assumes that the diagonal crack spacing of members meeting S6-06 minimum stirrup requirements and of members not complying with S6-06 Section 14 stirrup spacing and area requirements equals the longitudinal spacing of the stirrups. The second modified method, called S6-06 F, incorporates both the crack spacing assumption in S6-06 M and a revised concrete area which includes a portion of the compression flange in the concrete contribution to shear capacity term $V_c$. Chapter 5 presents these modified sectional shear evaluation methods. These proposed methods are validated through predictions of shear capacity for the 163 members identified in this study.

## 5.2 S6-06 M – Proposed Method

The modified sectional shear method S6-06 M makes the assumption that the spacing of diagonal cracks will be equal to the longitudinal spacing of the stirrups. This assumption is applied consistently for both compliant and non-compliant members. Other than this assumption for diagonal crack spacing, the modified shear method S6-06 M is the same as the sectional shear method in S6-06 Section 8. As such S6-06 M can also be considered a Modified Compression Field Theory (Vecchio and Collins, 1986) based approach. Figure 5.1 provides the diagonal crack spacing assumptions used by the shear method in S6-06 and the modified shear method S6-06 M.

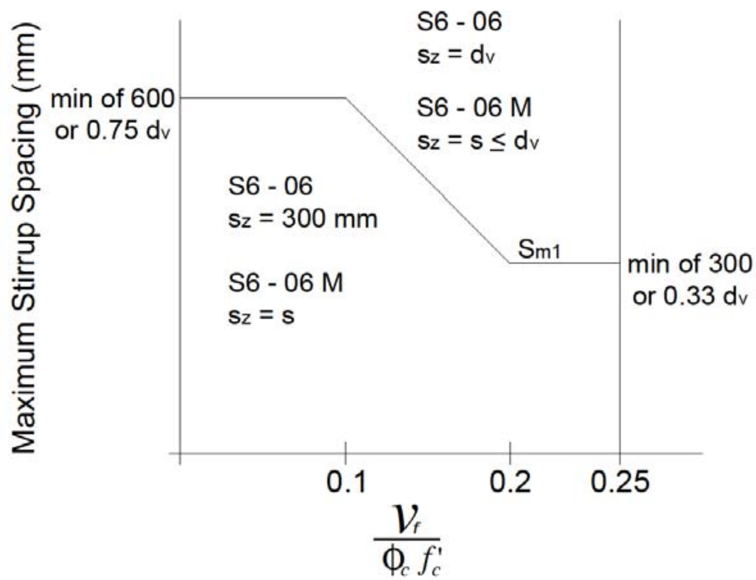



Figure 5.1 – S6-06 and S6-06 M Crack Spacing Assumptions for Members with Stirrups

As illustrated in Figure 5.1, S6-06 M assumes that the crack spacing term $s_z$ is equal to the longitudinal spacing of the stirrups but not more than the shear depth $d_v$. It should be noted from Figure 5.1 that the diagonal crack spacing assumed by S6-06 M is independent of how the longitudinal stirrup spacing compares to the maximum stirrup spacing limit $s_{m1}$. Thus the limit $s_{m1}$ does not impact calculations using the proposed method S6-06 M in this study because the focus is entirely based on evaluating the shear strength of a member. However, when considering both strength and ductility, it is important to compare the actual stirrup spacing to a maximum spacing limit. Members with properly detailed stirrups ($s/s_{m1}$ less than 1.00) are found to demonstrate considerably more ductile behavior than do members with excessive stirrup spacing and deficient stirrup area (DeGeer and Stephens, 1993; Bentz, 2005). S6-06 Clause 14.12 accounts for ductility by assigning a higher reliability index $\beta_r$ to members with $s/s_{m1}$ ratios greater than 1.00 than for members with $s/s_{m1}$ ratios less than 1.00. This is consistent with work in MacGregor (1976). The assumed reliability index then impacts the applied load factors ($\alpha_{LF}$) and the reliability factor ($U$) at a section. Although issues relating to ductility (and as such $\beta_r$, $\alpha_{LF}$ and $U$ terms) are outside the scope of this study, the $s_{m1}$ stirrup spacing limit is still included for analysis using S6-06 M. This is both to facilitate comparisons of trends to analysis using S6-06, and because the $s_{m1}$ limit will be required when considering member ductility. S6-06 M determines the stirrup spacing limit $s_{m1}$ using the same process employed by S6-06 (see Figure 2.1). Full stirrup contribution is assumed by the modified shear method S6-06 M regardless of stirrup spacing. S6-06 M uses Eqn. (3.12) for determining the effective stirrup contribution to shear capacity based on actual stirrup area. Eqn. (3.12) is discussed in Section 3.2. Other than the crack spacing assumption, the proposed shear method S6-06 M uses the same process described in the flow chart provided in Section 3.2. Because the modification in the proposed shear method S6-06 M affects only members with stirrups, discussion of members without stirrups is not included in Section 5.2.

### 5.2.1 S6-06 M – Background

As discussed in Section 2.4.2.i, results in literature (Dilger and Divakar, 1987; Angelakos, 1999; Yoshida, 2000) suggest that a correlation exists between the longitudinal spacing of stirrups and the longitudinal spacing of diagonal cracks. Lubell (2006) proposed a modification to the shear method in A23.3-04 (CSA, 2004), termed CSA-M, where the spacing of diagonal cracks in members complying with minimum stirrup requirements were assumed to be equal to the longitudinal spacing of the stirrups, rather than 300 mm. CSA-M limited the maximum diagonal crack spacing to a member's shear depth $d_v$. This method is discussed in Section 2.4.2.i. Table 5.1 provides a summary of shear capacity predictions derived in this study from results in Lubell (2006). This study evaluated 106 non-prestressed members with stirrups typically complying with minimum transverse reinforcement requirements using the sectional shear method in A23.3-04 and the shear method CSA-M. The results indicate that the crack spacing assumption proposed by Lubell (2006) provided improvements in both accuracy and consistency of shear capacity predictions.

Table 5.1 – Results of 106 Members Evaluated by Lubell (2006)

| | CSA A23.3 | CSA-M |
|---|---|---|
| $\frac{V_{calc}}{V_{test}}$ | 0.87 | 0.94 |
| COV (%) | 16.5 | 15.3 |

For sectional shear strength calculations, S6-06 M is very similar to Lubell's (2006) shear method CSA-M. The differences between the two modified shear methods are:

- S6-06 M assumes that the diagonal crack spacing is equal to the longitudinal stirrup spacing, but limited to $d_v$, for members not complying with minimum stirrup spacing and area requirements, while CSA-M assumes that the diagonal crack spacing of these members is equal to the shear depth $d_v$.
- S6-06 M uses Eqn (3.7) to calculate the diagonal compression field angle while CSA-M uses Eqn (3.6). These two expressions are discussed in Section 3.2.
- S6-06 M determines maximum permissible stirrup spacing $s_{m1}$ using Figure 5.1, while CSA-M determines maximum permissible stirrup spacing as the lesser of $0.7 \cdot d_v$ or 600 mm in accordance with A23.3-04 Clause 11.3.8.1. This maximum permissible stirrup spacing is reduced by a factor of 2 if the normalized shear stress $\frac{v_f}{\phi_c \cdot f_c'}$ exceeds 0.125 in accordance with A23.3-04 Clause 11.3.8.3.

Issues relating to load and resistance factors differ between these two modified shear methods but such issues are outside of the scope set for this study.

### 5.2.2 S6-06 M - Results from Shear Capacity Evaluation of Members with Stirrups and Comparison to S6-06 Evaluation

This section discusses the results of the shear capacity evaluation of the 130 members with stirrups analyzed in this study using the modified shear method S6-06 M and provides comparison to the results of the similar analysis in Section 4.5 pertaining to evaluation using S6-06. These comparisons are used to justify the modification to diagonal crack spacing proposed in S6-06 M. The focus of Section 5.2.2 is divided into four parts: i) tables providing the results from evaluation using the modified shear method S6-06 M, ii) comparison of the mean statistical data between predictions using S6-06 M and S6-06 Section 8, iii) allocation of members into the prediction classifications given in Table 4.1 and average demerit points per member and iv) trends in the $V_{calc}/V_{test}$ ratios due to variations in the stirrup detail ratios $s/s_{m1}$ and $A_{v,min}/A_v$.

### 5.2.2.i Statistical Data and Average Demerit Points per Member

Table 5.2 provides the $V_{calc}/V_{test}$, $s/s_{m1}$ and $A_{v,min}/A_v$ ratios for the 130 members with stirrups identified for this study calculated using the modified shear method S6-06 M and includes $V_{calc}/V_{test}$ ratios from evaluation using the sectional shear method in S6-06 Section 8. A summary of the statistical results and the average demerit points per member for these 130 members is provided in Table 5.3. $V_{calc}/V_{test}$ ratios calculated using S6-06 M vary with respect to the corresponding $V_{calc}/V_{test}$ ratios calculated using S6-06 Section 8 provisions for 61.2% of the non-compliant members and 97.5% of the compliant members. This indicates that variations in the statistical data between predicted and tested shear capacities calculated using S6-06 M and S6-06 are representative of the entire dataset. The percent differences in $V_{calc}/V_{test}$ ratios between evaluation using S6-06 M and evaluation using the sectional shear method in S6-06 Section are included in Table 5.2. Positive percent differences signify that the $V_{calc}/V_{test}$ ratios calculated using S6-06 M have increased with respect to the corresponding shear prediction calculated using S6-06, indicating that S6-06 M attributes greater sectional shear capacity to a member than does S6-06.

Table 5.2 – Results of Evaluation using S6-06 M – 130 Members with Stirrups

| Member | S6-06 $V_{calc}/V_{test}$ | S6-06 M $V_{calc}/V_{test}$ | S6-06 M – S6-06 Percent Difference (%) | $s/s_{m1}$ | $A_{v,min}/A_v$ |
|---|---|---|---|---|---|
| Non-Compliant Non-Prestressed | | | | | |
| YB2000/9 | 0.70 | 0.70 | 0.0 | 4.50 | 0.94 |
| YB2000/6 | 0.58 | 0.66 | 13.8 | 2.25 | 1.12 |
| YB2000/4 | 0.48 | 0.76 | 58.3 | 0.98 | 1.08 |
| 5084 | 0.65 | 0.65 | 0.0 | 2.55 | 0.44 |
| 5053 | 0.67 | 0.67 | 0.0 | 3.26 | 0.57 |
| 5052 | 0.66 | 0.66 | 0.0 | 3.57 | 0.63 |
| 5051 | 0.63 | 0.63 | 0.0 | 3.93 | 0.68 |
| 5063 | 0.61 | 0.61 | 0.0 | 2.96 | 0.54 |
| N2-S | 0.98 | 1.04 | 6.1 | 1.05 | 1.03 |
| N1-N | 0.78 | 0.89 | 14.1 | 0.74 | 1.02 |
| P21 | 0.76 | 0.77 | 1.3 | 1.22 | 0.90 |
| Ss2-321-3 | 1.08 | 1.08 | 0.0 | 1.73 | 1.36 |
| Ss2-318-3 | 0.92 | 0.92 | 0.0 | 1.49 | 1.17 |
| Ss2-313.5-3 | 0.84 | 0.87 | 3.6 | 1.11 | 0.87 |
| Ss2-321-2 | 1.00 | 1.00 | 0.0 | 1.73 | 1.00 |
| Ss2-318-2 | 0.98 | 0.98 | 0.0 | 1.49 | 0.87 |
| Ss2-321-1 | 1.13 | 1.13 | 0.0 | 1.73 | 0.69 |
| Ss2-318-1 | 0.89 | 0.89 | 0.0 | 1.49 | 0.60 |
| Ss2-218a-2 | 0.83 | 0.83 | 0.0 | 1.49 | 1.77 |
| Ss2-213.5-2 | 0.91 | 0.94 | 3.3 | 1.11 | 1.32 |
| Ss2-213.5-1 | 0.97 | 1.00 | 3.1 | 1.11 | 1.47 |

| Table 5.2 continued | | | | | |
|---|---|---|---|---|---|
| Member | S6-06 $V_{calc}/V_{test}$ | S6-06 M $V_{calc}/V_{test}$ | S6-06 M – S6-06 Percent Difference (%) | $s/s_{m1}$ | $A_{v,min}/A_v$ |
| Non-Compliant Non-Prestressed | | | | | |
| J & R - 7 | 0.95 | 1.05 | 10.5 | 0.73 | 1.14 |
| J & R - 8 | 1.03 | 1.14 | 10.7 | 0.73 | 1.14 |
| BM100 | 0.73 | 0.82 | 12.3 | 1.00 | 1.03 |
| SB 2012/6 | 0.58 | 0.66 | 13.8 | 2.25 | 0.92 |
| SB 2003/6 | 0.72 | 0.83 | 15.3 | 2.25 | 0.99 |
| 10T24 | 0.71 | 0.82 | 15.5 | 1.02 | 0.91 |
| PE1 | 0.78 | 0.78 | 0.0 | 2.14 | 1.14 |
| PE2 | 0.74 | 0.74 | 0.0 | 2.74 | 1.46 |
| Non-Compliant Prestressed | | | | | |
| CH-6-240 | 0.73 | 0.74 | 1.4 | 1.19 | 0.39 |
| CM-6-240 | 0.71 | 0.72 | 1.4 | 1.19 | 0.47 |
| CL-6-240 | 0.64 | 0.65 | 1.6 | 1.19 | 0.62 |
| PH-6-240 | 0.86 | 0.87 | 1.2 | 1.37 | 0.34 |
| PM-6-240 | 0.77 | 0.78 | 1.3 | 1.28 | 0.44 |
| PL-6-240 | 0.70 | 0.71 | 1.4 | 1.23 | 0.55 |
| NM-10-240 | 0.96 | 0.97 | 1.0 | 1.93 | 0.14 |
| NL-10-240 | 0.83 | 0.84 | 1.2 | 1.61 | 0.23 |
| NM-8-240 | 0.99 | 1.00 | 1.0 | 1.88 | 0.20 |
| NH-6-240 | 0.76 | 0.77 | 1.3 | 1.55 | 0.31 |
| NM-6-240 | 0.73 | 0.74 | 1.4 | 1.39 | 0.39 |
| NL-6-240 | 0.75 | 0.76 | 1.3 | 1.27 | 0.53 |
| CW12 | 0.83 | 0.87 | 4.8 | 1.90 | 0.16 |
| CW11 | 0.78 | 0.82 | 5.1 | 1.30 | 0.19 |
| CI12 | 1.00 | 1.03 | 3.0 | 1.62 | 0.19 |
| CI11 | 1.01 | 1.05 | 4.0 | 1.21 | 0.22 |
| P9 | 0.69 | 0.69 | 0.0 | 1.25 | 0.94 |
| P14 | 0.69 | 0.69 | 0.0 | 1.20 | 0.98 |
| P19 | 0.76 | 0.76 | 0.0 | 1.26 | 1.00 |
| BW.14.34 | 0.87 | 0.87 | 0.0 | 1.68 | 0.99 |
| Compliant Non-Prestressed | | | | | |
| V18-2 | 0.90 | 0.95 | 5.6 | 0.65 | 0.49 |
| V36-3 | 1.05 | 1.02 | -2.9 | 0.65 | 0.87 |
| V36-2 | 1.08 | 1.14 | 5.6 | 0.30 | 0.99 |
| V1 | 1.28 | 1.24 | -3.1 | 0.65 | 0.90 |
| V2 | 1.03 | 1.00 | -2.9 | 0.65 | 0.90 |
| A1 | 0.83 | 0.86 | 3.6 | 0.68 | 0.39 |
| A2 | 0.89 | 0.93 | 4.5 | 0.68 | 0.42 |
| B1 | 0.84 | 0.89 | 6.0 | 0.62 | 0.27 |
| B2 | 0.96 | 1.01 | 5.2 | 0.62 | 0.29 |
| C1 | 1.05 | 1.09 | 3.8 | 0.69 | 0.20 |
| C2 | 0.98 | 1.02 | 4.1 | 0.68 | 0.21 |
| P5 | 0.59 | 0.66 | 11.9 | 0.53 | 0.37 |
| P20 | 0.61 | 0.68 | 11.5 | 0.81 | 0.58 |
| P22 | 0.63 | 0.67 | 6.4 | 0.81 | 0.55 |
| DBO530M | 1.01 | 1.01 | 0.0 | 0.50 | 0.85 |
| DB120M | 1.10 | 0.93 | -15.5 | 1.00 | 0.69 |
| DM140M | 1.26 | 1.26 | 0.0 | 0.50 | 0.92 |
| Ss2-29g-2 | 0.99 | 1.03 | 4.0 | 0.74 | 0.58 |
| Ss2-29e-2 | 0.93 | 0.96 | 3.2 | 0.74 | 0.99 |
| Ss2-29d-2 | 1.04 | 1.08 | 3.9 | 0.74 | 0.80 |
| Ss2-29c-2 | 1.01 | 1.05 | 4.0 | 0.74 | 0.71 |
| Ss2-29b-2 | 0.91 | 0.94 | 3.3 | 0.74 | 0.94 |
| Ss2-29a-2 | 0.83 | 0.86 | 3.6 | 0.74 | 0.89 |
| Ss2-29b-1 | 1.10 | 1.14 | 3.6 | 0.74 | 0.97 |

| Table 5.2 continued | | | | | |
|---|---|---|---|---|---|
| Member | S6-06 $V_{calc}/V_{test}$ | S6-06 M $V_{calc}/V_{test}$ | S6-06 M – S6-06 Percent Difference (%) | $s/s_{m1}$ | $A_{v,min}/A_v$ |
| Compliant Non-Prestressed | | | | | |
| Ss2-29a-1 | 1.11 | 1.15 | 3.6 | 0.74 | 1.00 |
| Ss2-26-1 | 0.96 | 1.03 | 7.3 | 0.49 | 0.67 |
| 1 | 1.01 | 1.09 | 7.9 | 0.37 | 0.48 |
| 5 | 0.96 | 1.04 | 8.3 | 0.37 | 0.59 |
| 5A-0 | 0.77 | 0.82 | 6.5 | 0.98 | 0.09 |
| 5B-0 | 0.75 | 0.80 | 6.7 | 0.98 | 0.09 |
| Test 1.1 | 0.82 | 0.83 | 1.2 | 0.87 | 0.27 |
| Test 2.1 | 0.95 | 0.97 | 2.1 | 0.87 | 0.27 |
| Test 2.2 | 0.99 | 1.01 | 2.0 | 0.87 | 0.27 |
| Test 2.3 | 0.96 | 0.98 | 2.1 | 0.87 | 0.28 |
| T1 | 0.72 | 0.72 | 0.0 | 0.82 | 0.07 |
| ET1 | 0.94 | 1.02 | 8.5 | 0.54 | 0.56 |
| ET2 | 0.81 | 0.88 | 8.6 | 0.55 | 0.28 |
| ET3 | 0.76 | 0.82 | 7.9 | 0.69 | 0.19 |
| 2T10 | 0.98 | 1.00 | 2.0 | 0.43 | 0.21 |
| 2T12 | 0.96 | 0.96 | 0.0 | 0.51 | 0.25 |
| 1T18 | 0.96 | 0.90 | -6.2 | 0.76 | 0.45 |
| Prestressed Compliant | | | | | |
| PL-6-160 | 0.69 | 0.74 | 7.2 | 0.93 | 0.35 |
| NL-6-80 | 0.81 | 0.87 | 7.4 | 0.65 | 0.17 |
| NH-6-80 | 0.89 | 0.96 | 7.9 | 0.90 | 0.11 |
| NM-6-80 | 0.88 | 0.95 | 8.0 | 0.89 | 0.13 |
| NL-6-160 | 0.73 | 0.78 | 6.8 | 0.98 | 0.34 |
| NM-6-160 | 0.78 | 0.83 | 6.4 | 1.10 | 0.26 |
| CH-6-80 | 0.75 | 0.80 | 6.7 | 0.55 | 0.13 |
| CM-6-80 | 0.77 | 0.83 | 7.8 | 0.47 | 0.16 |
| CL-6-80 | 0.77 | 0.83 | 7.8 | 0.44 | 0.21 |
| CH-6-160 | 0.75 | 0.78 | 4.0 | 0.83 | 0.27 |
| CL-6-160 | 0.71 | 0.76 | 7.0 | 0.80 | 0.42 |
| PM-6-80 | 0.83 | 0.89 | 7.2 | 0.65 | 0.14 |
| PM-6-160 | 0.77 | 0.82 | 6.5 | 1.00 | 0.27 |
| PL-6-80 | 0.82 | 0.88 | 7.3 | 0.57 | 0.18 |
| A1-00-1.5R-N | 1.17 | 1.22 | 4.3 | 0.32 | 0.07 |
| B0-00-R-S | 0.72 | 0.70 | 2.0 | 0.74 | 0.23 |
| B0-00-R-N | 0.96 | 0.98 | -2.3 | 0.42 | 0.11 |
| A1-00-M-N | 1.18 | 1.16 | -1.9 | 0.78 | 0.24 |
| A1-00-R_N | 0.99 | 1.01 | -3.4 | 0.43 | 0.10 |
| A1-00-M-S | 0.88 | 0.86 | 5.5 | 0.76 | 0.24 |
| A1-00-0.5R-N | 1.03 | 1.01 | 4.1 | 0.72 | 0.21 |
| A1-00-0.5R-S | 0.87 | 0.84 | 5.5 | 0.74 | 0.23 |
| 2A-3 | 0.73 | 0.77 | 5.0 | 0.98 | 0.11 |
| 2B-3 | 0.74 | 0.77 | 5.6 | 0.98 | 0.10 |
| 3A-2 | 0.73 | 0.77 | 6.7 | 0.98 | 0.10 |
| 3B-2 | 0.80 | 0.84 | 9.1 | 0.98 | 0.09 |
| 4A-1 | 0.72 | 0.76 | 4.8 | 0.91 | 0.10 |
| 4B-1 | 0.75 | 0.80 | 4.8 | 0.98 | 0.09 |
| P4 | 0.66 | 0.72 | 4.4 | 0.53 | 0.35 |
| P8 | 0.63 | 0.66 | 7.4 | 0.86 | 0.59 |
| P13 | 0.62 | 0.65 | 7.0 | 0.80 | 0.57 |
| P18 | 0.68 | 0.71 | 5.7 | 0.84 | 0.60 |
| P24 | 0.68 | 0.73 | 2.7 | 0.54 | 0.37 |
| P25 | 0.71 | 0.76 | 7.8 | 0.80 | 0.55 |
| P26 | 0.70 | 0.74 | 7.2 | 0.56 | 0.38 |
| P27 | 0.75 | 0.77 | 7.4 | 0.84 | 0.56 |

| Table 5.2 continued | | | | | |
|---|---|---|---|---|---|
| Member | S6-06 $V_{calc}/V_{test}$ | S6-06 M $V_{calc}/V_{test}$ | S6-06 M – S6-06 Percent Difference (%) | $s/s_{m1}$ | $A_{v,min}/A_v$ |
| Prestressed Compliant | | | | | |
| P28 | 0.64 | 0.69 | 7.9 | 0.55 | 0.37 |
| P29 | 0.68 | 0.72 | 5.9 | 0.83 | 0.57 |
| P49 | 0.68 | 0.73 | 7.4 | 0.55 | 0.34 |
| P50 | 0.72 | 0.77 | 6.9 | 0.60 | 0.19 |

Table 5.3 – S6-06 M and S6-06 Shear Capacity Evaluation Comparison – Members with Stirrups

| Test Group (number) | Mean $V_{calc}/V_{test}$ Ratio S6-06M | $V_{calc}/V_{test}$ C.O.V (%) S6-06 M | Average Demerit Points / Member S6-06 M | Mean $V_{calc}/V_{test}$ Ratio S6-06 | $V_{calc}/V_{test}$ C.O.V (%) S6-06 | Average Demerit Points / Member S6-06 |
|---|---|---|---|---|---|---|
| All Members with Stirrups (130) | 0.87 | 17.2 | 5.03 | 0.84 | 18.8 | 5.99 |
| Non-Compliant Non-Prestressed Members (29) | 0.85 | 18.8 | 5.64 | 0.80 | 20.8 | 6.84 |
| Compliant Non-Prestressed Members (41) | 0.96 | 14.6 | 3.61 | 0.93 | 16.3 | 3.91 |
| Non-Compliant Prestressed Members (20) | 0.82 | 14.6 | 5.36 | 0.80 | 14.7 | 6.02 |
| Compliant Prestressed Members (40) | 0.82 | 15.0 | 5.87 | 0.78 | 16.6 | 7.49 |

### 5.2.2.ii S6-06 M – Comparison of Statistical Results with S6-06 Section 8

It is observed from Table 5.3 that the agreement between predicted and tested shear capacities, calculated using the modified shear method S6-06 M, is improved for both the twenty-nine non-prestressed members with non-compliant stirrup spacing and area details and the forty-one non-prestressed members meeting minimum stirrup requirements relative to predictions using the shear provisions in S6-06 Section 8. The twenty-nine non-prestressed non-compliant members and forty-one non-prestressed compliant members show improvements of 6.3% (0.85 compared to 0.80) and 3.2% (0.96

compared to 0.93) in their mean $V_{calc}/V_{test}$ ratio respectively. Improvements in $V_{calc}/V_{test}$ ratios are defined in this study as having these ratios closer to unity, relative to predictions of capacity using another sectional shear method. The COV of the twenty-nine non-prestressed non-compliant members and forty-one non-prestressed compliant members, calculated from predictions of shear capacity using S6-06 M, decrease by 9.6% (18.8% compared to 20.8%) and 10.4% (14.6% compared to 16.3%) respectively compared to predictions using the shear method in S6-06. The forty-one compliant non-prestressed members evaluated using the modified shear method S6-06 M present with an average $V_{calc}/V_{test}$ ratio and COV consistent with values derived from Lubell's (2006) study using the method CSA-M (see Tables 5.1 and 5.3). Table 5.3 shows that the forty-one compliant members are calculated to have a larger average $V_{calc}/V_{test}$ ratio than do the twenty-nine non-compliant members, indicating that the modified shear method S6-06 M may present with more conservative predictions of shear capacity for non-compliant members than for compliant members. This is consistent with shear capacity evaluations calculated using the sectional shear method in S6-06 Section 8 as discussed in Section 4.5.4. Compliant and non-compliant members had similar ratios of the number of flanged to rectangular members – thus this detail would not likely affect the difference in average $V_{calc}/V_{test}$ ratios.

As inferred from Table 5.3 the agreement between predicted and tested shear capacities, calculated using the modified shear method S6-06 M, is also improved for the prestressed categories of members evaluated in this study relative to predictions of shear capacity using the provisions in S6-06 Section 8. Evaluations of shear capacity using S6-06 M for the twenty non-compliant prestressed members and forty compliant prestressed members show improvements of 2.6% (0.82 compared to 0.80) and 5.1% (0.82 compared to 0.78) respectively in their mean $V_{calc}/V_{test}$ ratios and corresponding decreases in their COV of 0.7% (14.6% compared to 14.7%) and 9.6% (15.0% compared to 16.6%) respectively, relative to shear predictions using S6-06 Section 8. Discussion of shear capacity predictions calculated using the sectional shear provisions in S6-06 is provided in Section 4.5.1.

### 5.2.2.iii S6-06 M - Prediction Classifications and Average Demerit Points per Member

Based on evaluation using the modified shear method S6-06 M, Table 5.4 distributes the shear capacity predictions of the seventy non-prestressed members with stirrups evaluated in this study into the ranges provided in Table 4.1, and provides the average demerit points per member for the full data set of non-prestressed members with stirrups, as well as for the compliant and non-compliant data categories. Table 5.5 provides the same information for the sixty prestressed members with stirrups identified for evaluation in this study.

Table 5.4 - S6-06 M – Non-Prestressed Members - $V_{calc}/V_{test}$ Ranges and Demerit Points

| **S6-06 M - Non-Prestressed Members with Stirrups** | **Full Data Set** | **Non-Compliant** | **Compliant** | **Demerits - Full Data Set** | **Demerits - Non-Compliant** | **Demerits - Compliant** |
|---|---|---|---|---|---|---|
| Total | 70 | 29 | 41 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 13<br>18.6% | 9<br>31.0% | 4<br>9.8% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 55<br>78.6% | 20<br>69.0% | 35<br>85.4% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 2<br>2.9% | 0<br>0.0% | 2<br>4.9% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 312 | 164 | 148 |
| Average Demerits/Members | | | | 4.45 | 5.64 | 3.61 |

Table 5.4 indicates that for the twenty-nine non-compliant non-prestressed members with stirrups evaluated in this study, twenty members present with predictions in the range considered to be 'appropriate' (see Table 4.1). This is an improvement over the predictions using the shear provisions in S6-06, which resulted in sixteen members

having $V_{calc}/V_{test}$ ratios in the range considered to be 'appropriate' (see Table 4.8). Due to the diagonal crack spacing assumption used by S6-06 M, members with heights greater than 800 mm and with stirrups which do not comply with S6-06 Section 14 spacing and area requirements show a considerable improvement in the agreement between predicted and tested shear capacities. Evaluation using S6-06 M results with three fewer predictions in the 'conservative' range relative to S6-06 and no predictions in the 'very conservative' range. The members with 'conservative' predictions are mostly flanged members, a detail which is not addressed by S6-06 M. The conservative nature of shear capacity predictions for members with compression flanges is discussed in Section 2.5.4 and is examined further in Section 5.3. Evaluation of non-compliant non-prestressed members using S6-06 M does not present with any $V_{calc}/V_{test}$ ratios in the 'low safety' or more unsafe ranges, suggesting that the crack spacing assumption used by S6-06 M will not produce unsafe predictions of shear capacity (see Table 4.1). A comparison of Tables 4.8 and 5.4 indicates that evaluation of the forty-one compliant non-prestressed members using the modified shear method S6-06 M results with the same distribution of members into the ranges provided in Table 4.1 as does evaluation using the sectional shear method in S6-06 Section 8. S6-06 M did calculate a $V_{calc}/V_{test}$ ratio closer to unity for one of the 'low safety' members (Specimen V1), although this change in $V_{calc}/V_{test}$ was not enough for this member to move into the 'appropriate' classification. Specimen V1 is discussed further in Section 4.5.1. S6-06 M does not improve the other 'low safety' prediction (Specimen DB140M) as this member's poor prediction of shear capacity was influenced by a stirrup failure during testing.

S6-06 M is allotted 17.5% (5.64 compared to 6.84) and 7.7% (3.61 compared to 3.91%) fewer average demerit points per non-prestressed non-compliant and compliant member respectively than is S6-06 (see Table 5.3), determined using the demerit point model proposed in Section 4.2. This decrease in average Demerit Points per member indicates that S6-06 M will typically calculate $V_{calc}/V_{test}$ ratios closer to unity than will the sectional shear method in S6-06 Section 8. The non-prestressed members in the non-compliant data category are allotted an average of 56.2% more demerit points per member (5.64 compared to 3.61) than are the non-prestressed members in the compliant category when using the modified shear method S6-06 M. This reduction in the difference of demerit points compares favorably to predictions using S6-06, which has a 74.9% difference between non-prestressed non-compliant and compliant members as discussed in Section 4.5.1. The reduction in average demerit points allotted to members evaluated using the modified shear method S6-06 M, relative to evaluation using S6-06, is primarily a result of the improved agreement between predicted and tested shear capacities for non-compliant members with heights greater than 800 mm.

Table 5.5 – S6-06 M –Prestressed Members - $V_{calc}/V_{test}$ Ranges and Demerit Points

| **S6-06 M – Prestressed Members with Stirrups** | **Full Data Set** | **Non-Compliant** | **Compliant** | **Demerits - Full Data Set** | **Demerits - Non-Compliant** | **Demerits - Compliant** |
|---|---|---|---|---|---|---|
| Total | 60 | 20 | 40 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 18<br>30.0% | 8<br>40.0% | 10<br>25.0% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 40<br>66.7% | 12<br>60.0% | 28<br>70.0% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 2<br>3.3% | 0<br>0.0% | 2<br>5.0% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 342 | 107 | 235 |
| Average Demerits/Members | | | | 5.70 | 5.36 | 5.87 |

Table 5.5 indicates that for the twenty non-compliant prestressed members evaluated in this study using S6-06 M, twelve members are calculated to have $V_{calc}/V_{test}$ ratios in the 'appropriate' range. Evaluations of one-way shear capacity using S6-06 M show improvements over S6-06 sectional shear provisions, which present with eleven of the twenty non-compliant prestressed members having $V_{calc}/V_{test}$ ratios in the 'appropriate' range (see Table 4.9). This improvement in the agreement of predicted to tested shear capacities over S6-06 is a result of the crack spacing assumption in S6-06 M, which allows closer spacing of diagonal cracks for members with stirrups than does the sectional shear provisions in S6-06 Section 8. This reduced crack spacing in turn predicts the ability for larger shear forces to be transferred across a cracked surface by aggregate

interlock, as discussed in Section 2.4.2ii. Based on the prediction classification provided in Table 4.1 none of the predictions of shear capacity using S6-06 M are unsafe for non-compliant prestressed members. Of the forty compliant prestressed members evaluated in this study, twenty-eight are in the 'appropriate' range. This is an improvement compared to the sixteen compliant prestressed members evaluated as 'appropriate' using the shear method in S6-06 (see Table 4.9).

The non-compliant prestressed members evaluated in this study using S6-06 M are allotted 8.7% fewer average demerit points per member than are the compliant prestressed members (5.36 compared to 5.87). This difference in average demerit points per member compares favorably to the corresponding difference from calculations using S6-06 Section 8, where non-compliant members are allotted with 19.6% fewer average demerit points per member than are compliant members. Predictions of shear capacity calculated using the modified shear method S6-06 M are allotted 11.0% (5.36 compared to 6.02) and 21.6% (5.87 compared to 7.49) fewer average demerit points per prestressed non-compliant and compliant member respectively than evaluation using S6-06 shear provisions. The lower value of average demerit points allotted to S6-06 M indicates that this modified shear method will typically present with $V_{calc}/V_{test}$ ratios closer to 1.00 than will the sectional shear method in S6-06.

### 5.2.2.iv S6-06 M - Relationship Between $V_{calc}/V_{test}$ Ratios and Stirrup Detail Ratios $s/s_{m1}$ and $A_{v,min}/A_v$

Evaluation of Shear Capacity for Non-Prestressed Members with Stirrups using S6-06 M

Figure 5.2 provides the relationship $V_{calc}/V_{test}$ vs. $s/s_{m1}$ for the seventy non-prestressed members with stirrups, while Figure 5.3 provides the relationship $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for the same members. These figures are used to study the influence that variations in stirrup spacing and area have on the agreement between predicted and tested shear capacities calculated using the modified shear method S6-06 M. The solid line in each of the following figures represent exact shear predictions, while the two dashed lines define the range of 'appropriate predictions' given in Section 4.2 (see Table 4.1). Members are classified as non-compliant if either of the stirrup detail ratios, $s/s_{m1}$ or

$A_{v,min}/A_v$, are greater than 1.00. Summary statistical data for the member categories is provided in Figures 5.2 and 5.3.

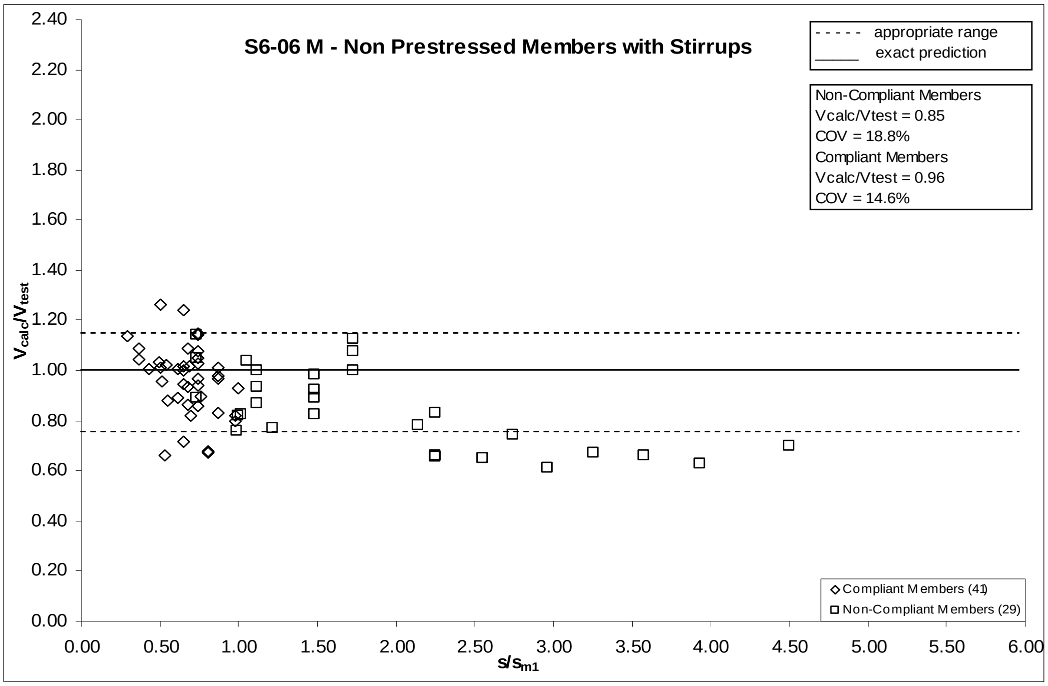



Figure 5.2 - $V_{calc}/V_{test}$ vs. $s/s_{m1}$ for 70 Non-Prestressed Members with Stirrups Evaluated using S6-06 M

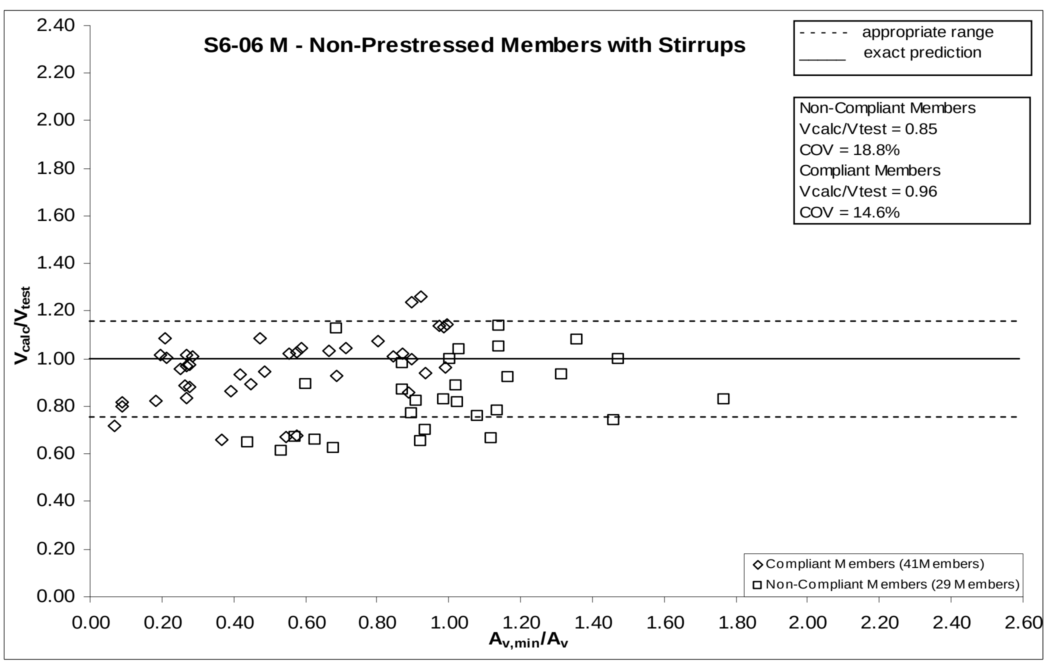



Figure 5.3 - $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for 70 Non-Prestressed Members with Stirrups Evaluated using S6-06 M

The lack of defined trends in $V_{calc}/V_{test}$ ratios shown in Figures 5.2 and 5.3 indicates that the agreement between predicted and tested shear capacities for the twenty-nine non-prestressed members not complying with minimum transverse reinforcement requirements is not influenced by variations in the stirrup spacing and area ratios $s/s_{m1}$ and $A_{v,min}/A_v$. This lack of trends in $V_{calc}/V_{test}$ ratios is similar to behavior noted from shear capacity evaluation using S6-06 (see Figures 4.6 and 4.7) and suggests that the modified shear method S6-06 M appropriately accounts for variations in stirrup spacing and area details. Non-compliant members presenting as possible outliers are typically flanged members, a detail which is not accounted for by the proposed shear method S6-06 M. Shear capacity of flanged members is discussed in Section 2.5.4 and examined further in Section 5.3. In order to further quantify improvements in shear predictions calculated using S6-06 M, members with improvements in percent difference of $V_{calc}/V_{test}$ ratios greater than 10% with respect to S6-06 predictions are identified. As discussed in Section 5.2.2.ii, improvement in $V_{calc}/V_{test}$ ratios is defined in this study as having $V_{calc}/V_{test}$ ratios closer to unity, relative to evaluation using another sectional shear method. Seven data points from the non-compliant non-prestressed data category show improvements in $V_{calc}/V_{test}$ ratios greater than 10% when compared to S6-06 shear predictions. These members are rectangular sections with heights ranging from 1000 mm to 2000 mm. It should be noted that the outlier point YB2000/4 identified from the evaluation using shear provisions in S6-06 no longer appears to deviate from the non-compliant non-prestressed category of members using the modified shear method S6-06 M.

Neither Figure 5.2 nor Figure 5.3 demonstrate any discernable trends in $V_{calc}/V_{test}$ ratios due to variations in the stirrup detail ratios $s/s_{m1}$ and $A_{v,min}/A_v$ for compliant non-prestressed members, indicating that i) the diagonal crack spacing assumption used by S6-06 M will not adversely affect shear predictions of compliant non-prestressed members and ii) variations in stirrup spacing and area details do no influence the agreement between predicted and tested shear capacities calculated using S6-06 M. The $V_{calc}/V_{test}$ ratios for test specimens V1 and DB140M appear to deviate from the rest of the compliant non-prestressed category of members – these members are discussed in Section 4.5.1. Three predictions of non-prestressed compliant members show improvements in percent difference of $V_{calc}/V_{test}$ ratios greater than 10%. Two specimen (P5 and P20) show increases in their corresponding $V_{calc}/V_{test}$ ratios while specimen

DB120M shows a decrease in its $V_{calc}/V_{test}$ ratio (see Table 5.2). This indicates that both conservative and unsafe predictions can benefit from the crack spacing assumption used in the modified shear method S6-06 M.

<u>Evaluation of Prestressed Members with Stirrups using S6-06 M</u>

Figure 5.4 provides the relationship $V_{calc}/V_{test}$ vs. $s/s_{m1}$ for the sixty prestressed members with stirrups, while Figure 5.5 provides the relationship $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for the same members. These figures are used to study the influence that variations in stirrup spacing and area have on the agreement between predicted and tested shear capacities for prestressed members. The solid line represents the condition in which the tested shear capacity is equal to the predicted shear capacity. The upper and lower dashed lines define the boundaries that are considered 'appropriate' predictions in this study.

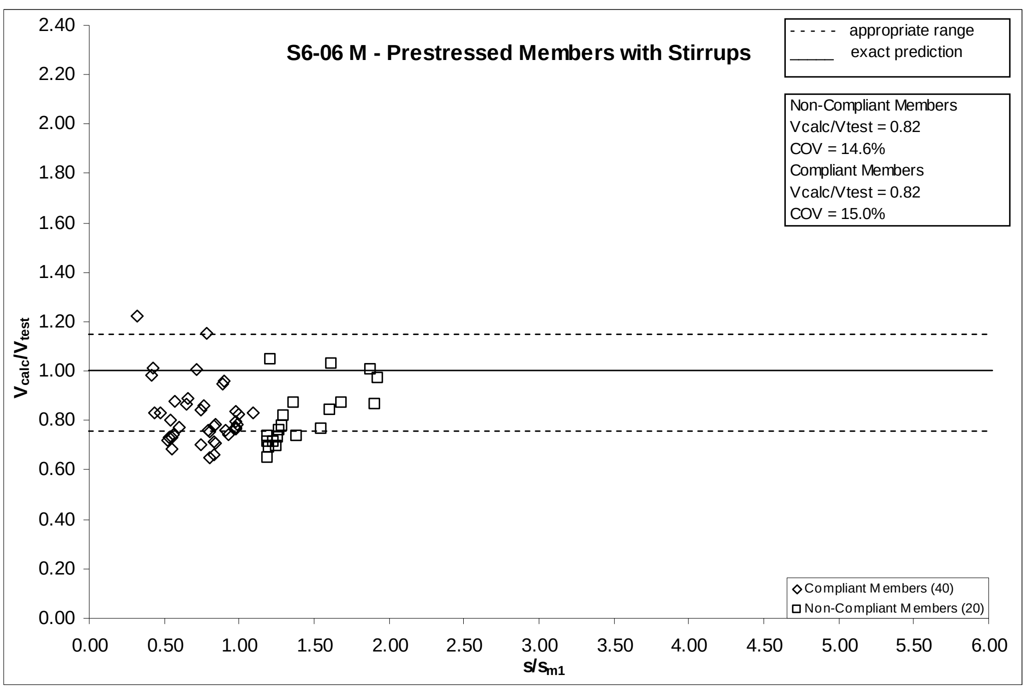



Figure 5.4 - $V_{calc}/V_{test}$ vs. $s/s_{m1}$ for 60 Prestressed Members with Stirrups Evaluated using S6-06 M

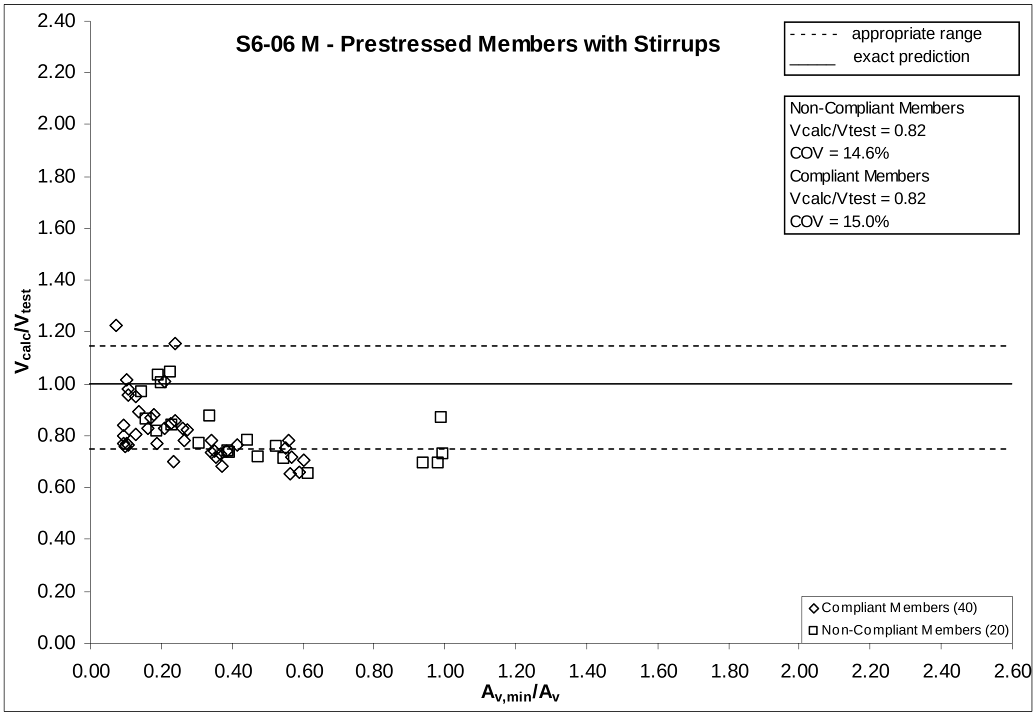



Figure 5.5 - $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for 60 Prestressed Members with Stirrups Evaluated using S6-06 M

As inferred from Figure 4.8 regarding the sectional shear provisions in S6-06 Section 8, Figure 5.4 demonstrates an increase in $V_{calc}/V_{test}$ ratios for non-compliant prestressed members as the $s/s_{ml}$ ratio increases from 1.00 to 2.00. However, this noted trend is based on a small data set and is not concerning as predictions do not go into the 'low safety' or more unsafe classification ranges. None of the non-compliant prestressed member predictions calculated using S6-06 M vary significantly from the rest of that data category (eg. no outliers). Figure 5.5 demonstrates a decrease in $V_{calc}/V_{test}$ ratios for the non-compliant prestressed members as their $A_{v,min}/A_v$ ratio increases. This is similar to results seen in Figure 4.9 from the evaluation using the shear provisions in S6-06. Behavior exhibited by the modified shear method S6-06 M is relevant to behavior exhibited by the sectional shear method in S6-06 because both shear methods are derived based on simplifications to the Modified Compression Field Theory (Vecchio and Collins, 1986). All prestressed members with non-compliant stirrup details evaluated in this study calculate shear capacities which are less than their corresponding actual shear capacities, which indicates that S6-06 M is capable of producing typically safe shear predictions for prestressed members with stirrup spacing details not complying with S6-06 Section 14 requirements.

Figure 5.4 does not exhibit any discernable trends in $V_{calc}/V_{test}$ ratios for compliant prestressed members as the stirrup spacing ratio $s/s_{m1}$ varies, indicating that stirrup spacing is appropriately accounted for by the proposed shear method S6-06 M. Figure 5.5 demonstrates a decrease in $V_{calc}/V_{test}$ ratios for compliant members as the stirrup area ratio $A_{v,min}/A_v$ increases. This trend in $V_{calc}/V_{test}$ ratios suggests that shear capacity predictions of prestressed members complying with stirrup spacing and area requirements may have a tendency to ignore a greater portion of their shear capacity as the actual stirrup area decreases with respect to the minimum required stirrup area. This noted behavior is consistent with evaluation of the forty compliant members using S6-06 (see Figure 4.9) and is discussed in Section 4.5.1. This trend in Figure 5.6 suggests that variations in stirrup area will not have a significant influence on predicted-to-tested shear capacities ratio which indicates that the modified shear method S6-06 M appropriately accounts for stirrup area details.

### 5.2.3 Parametric Sensitivity Analysis of Shear Predictions using the Proposed Shear Method S6-06 M

Section 5.2.3 assesses the influence that variations of the parameters discussed in Section 2.5 have on the agreement between predicted to tested shear capacities calculated using the modified shear method in S6-06 M. As described in Section 2.5, variations in concrete strength $f'_c$, shear span to depth ratio $a/d$, longitudinal reinforcement ratio $\rho$, and member shape (flanged members vs. rectangular members) are known to affect the shear capacity of concrete members. These details are studied against the 130 members with stirrups evaluated in this study to assess whether they are appropriately accounted for by S6-06 M.

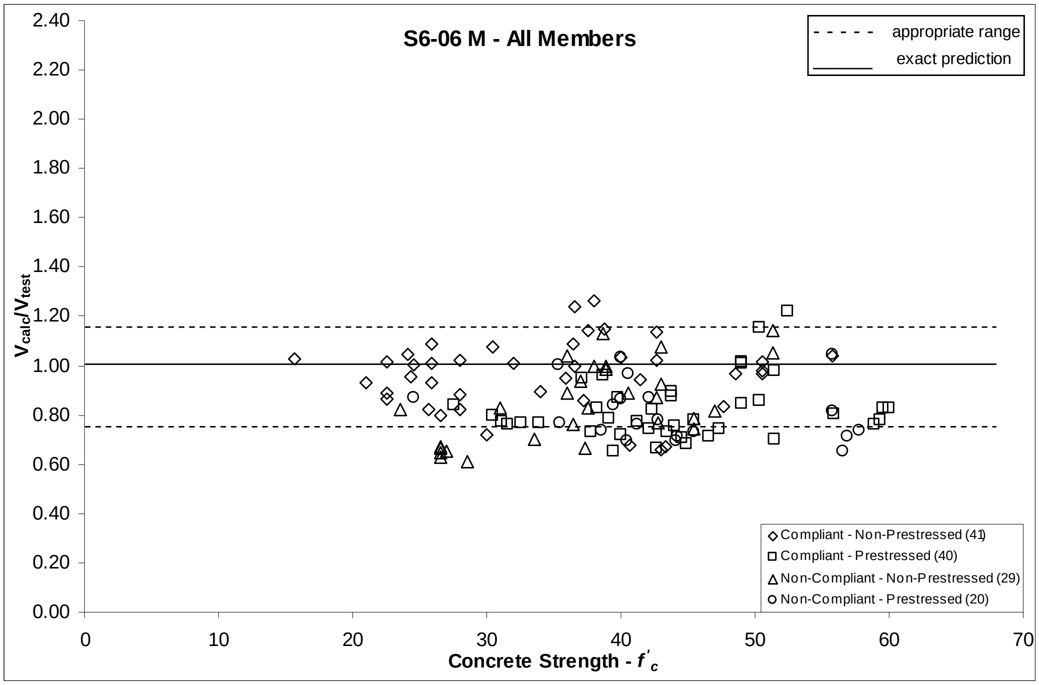



Figure 5.6 - $V_{calc}/V_{test}$ *vs.* $f_c^{'}$ for 130 Members Evaluated using S6-06 M

Figure 5.6 shows the relationship between the predicted-to-tested shear capacities and the specified concrete strength for the 130 members with stirrups evaluated in this study. This figure is devoid of any specific trend in $V_{calc}/V_{test}$ ratios as the concrete strength varies for any of the data categories, indicating that the modified shear method S6-06 M correctly accounts for concrete strength. This is consistent with results using the sectional shear provisions in S6-06 (see Figure 4.11).

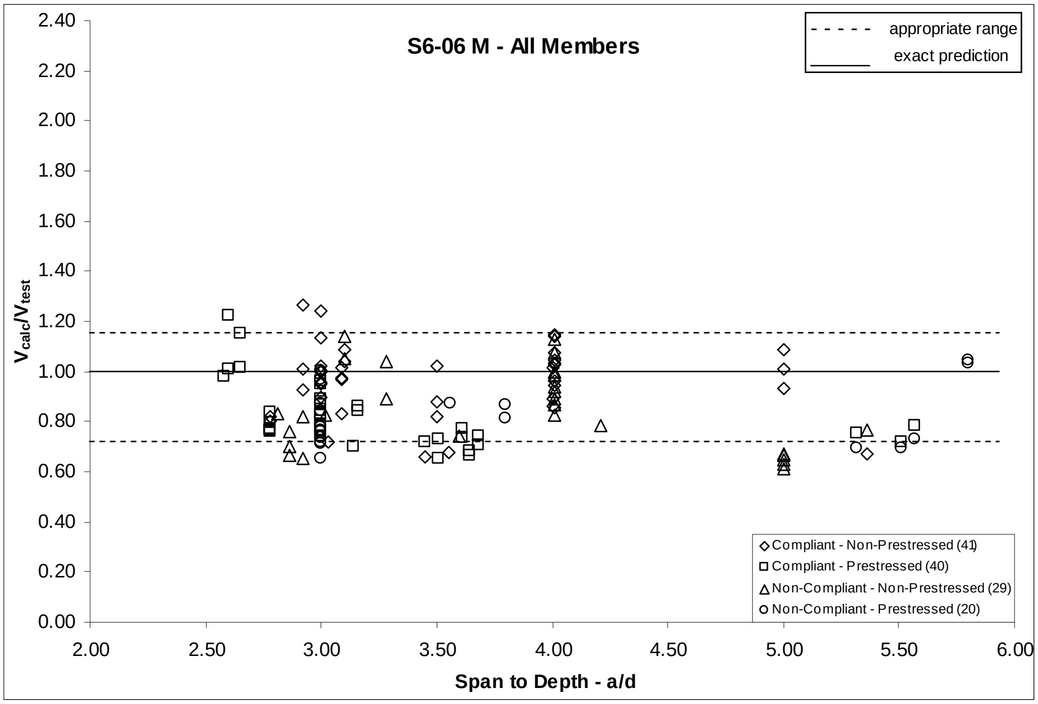



Figure 5.7 - $V_{calc}/V_{test}$ *vs. a/d* for 130 Members with Stirrups Evaluated using S6-06 M

Figure 5.7 provides the relationship between predicted-to-tested shear capacities and the shear span-to-depth ratio *a/d* for the 130 members with stirrups identified for evaluation in this study. This figure does not exhibit any defined changes in $V_{calc}/V_{test}$ ratios as the shear span-to-depth ratio changes for any of the member categories, indicating that the modified shear method in S6-06 M appropriately accounts for the *a/d* ratio. This is consistent with the evaluation using the shear provisions in S6-06 (see Figure 4.12).

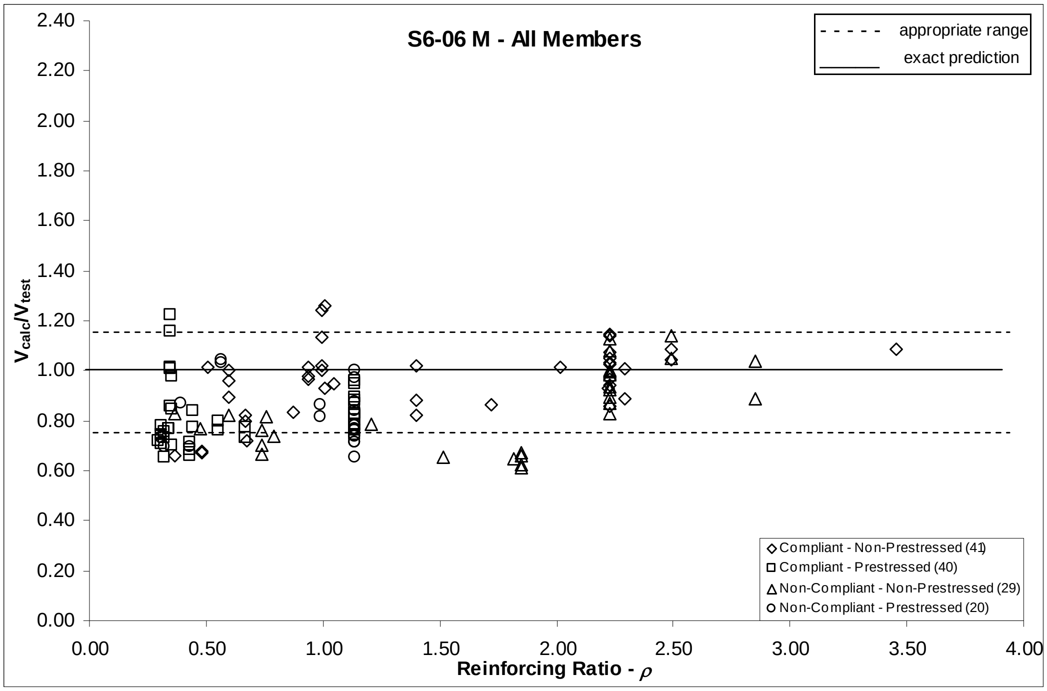



Figure 5.8 - $V_{calc}/V_{test}$ *vs.* $\rho$ for 130 Members with Stirrups Evaluated using S6-06 M

Figure 5.8 provides the relationship between $V_{calc}/V_{test}$ ratios and the longitudinal reinforcement ratio for the 130 members with stirrups evaluated in this study. As inferred from Figure 4.13 for the provisions in S6-06, the horizontal distribution of data points in Figure 5.8 indicates that the shear method in S6-06 M appropriately accounts for the quantity of longitudinal steel.

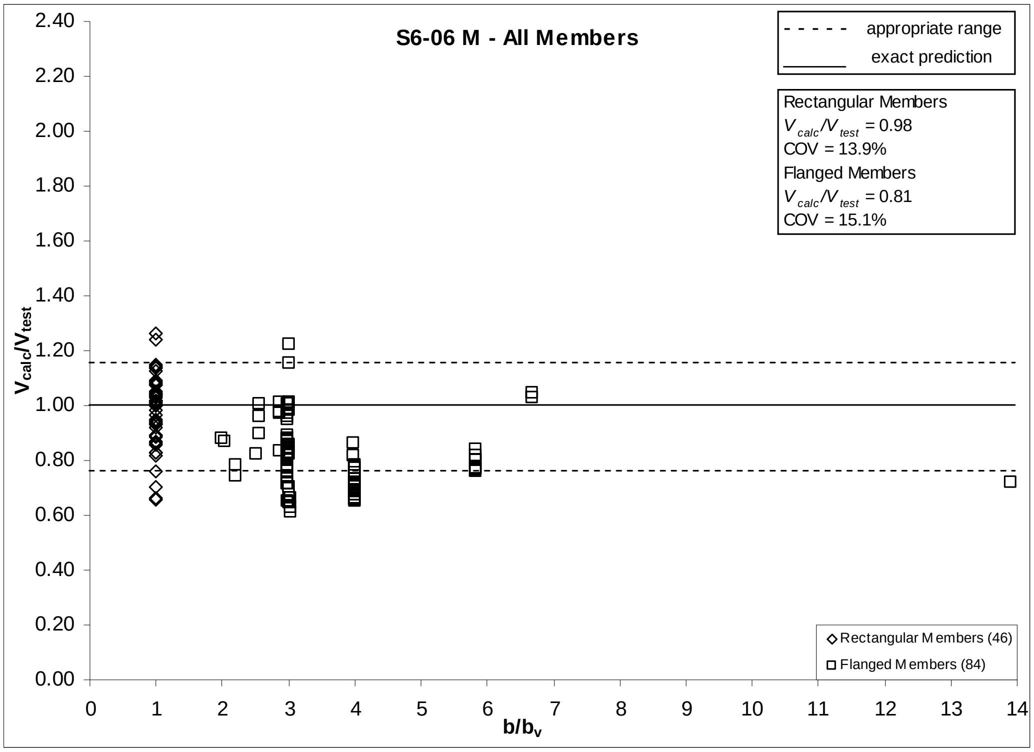



Figure 5.9 - $V_{calc}/V_{test}$ *vs.* $b/b_v$ for 130 Members with Stirrups Evaluated using S6-06 M

Figure 5.9 provides the relationship between predicted-to-tested shear capacities and the member flange width to web width ratio $b/b_v$ for the 130 members with stirrups identified for evaluation in this study. This figure indicates that typical $V_{calc}/V_{test}$ ratios are smaller for flanged members than for rectangular members. The increased conservatism of shear capacity predictions for flanged members is similar to results found in other studies (Placas and Regan, 1971; Giaccio et al, 2002) and that evaluation in this study indicates that this behavior is independent of the sectional shear method used for analysis. Summary statistical results for the entire dataset are provided in Figure 5.9. The non-compliant members alone have a mean $V_{calc}/V_{test}$ ratio for rectangular and flanged members of 0.91 and 0.78 respectively with corresponding COV of 15.2% and 15.4%. No well defined variations in $V_{calc}/V_{test}$ ratios are identified for flanged members as the $b/b_v$ ratio varies. Figure 5.9 indicates that one member (T1) has a $b/b_v$ ratio of 13.9. This value represents the effective flange to web width ratio, which has been reduced for calculation purposes from the actual $b/b_v$ value of 15 in accordance with S6-06 Clause 5.8.2.1.

### 5.2.4 S6-06 M - Crack Spacing Assumption leading to Shear Prediction Non-Convergence

As discussed in Section 3.2, evaluating the shear capacity of concrete members using the sectional method in S6-06 Section 8 and the evaluation provisions in Section 14 have an inherent tendency to produce a situation whereby iterative shear predictions will not reach a converged capacity. This is primarily a result of the change in assumed diagonal crack spacing $s_z$ as a member transitions from having compliant to non-compliant stirrup spacing.

To demonstrate this issue, a case study showing the evaluation of shear capacity for an Alberta bridge is provided in Appendix C and a brief summary is provided here. The bridge consists of four lines of 45 m simple span 'PO' girders spaced transversely at 2.74 m, with a shear connected 165 mm thick deck cast on top. This is shown in Figure C.1. Type 'O' Girders are large concrete 'I' sections used for the superstructure of some Alberta bridges; the prefix 'P' indicates that it is prestressed. The geometry of the 'PO' girders is provided in Figure C.2.

The sectional shear provisions for concrete members in S6-06 Section 8 combined with minimum stirrup requirements in Section 14 were used to evaluate the bridge chosen in the case study. Based on a moving load analysis the critical section for shear was identified at 0.09·L from the end of the girders. At this location the shear demand arising from iterations of the sectional forces caused the member to transition from being compliant to non-compliant with respect to the maximum allowable stirrup spacing. In this case study iteration of shear capacity was accomplished by varying the applied moments and shears at the critical section resulting from the moving load analysis. The moment-to-shear ratio at the critical section did not change with successive iterations. At the cusp of the transition in member compliancy S6-06 Section 8 sectional shear provisions abruptly predict the crack spacing $s_z$ to change from 300 mm to 2204 mm (the shear depth of the section). This causes the $\beta$ value to decrease immediately from 0.442 to 0.179, the $\theta$ value to increase from 28.6$^\circ$ to 50.3$^\circ$ and the corresponding predicted shear capacity at the critical section to change from 2153 kN to 1036 kN. As indicated in Table C.1, iterative predictions of shear capacity $V_r$ for the 'PO' girder do not converge, which causes the decision concerning the predicted shear capacity to be an ambiguous one. Early iterations in Table C.1 predict that the girders will have ample shear capacity to carry the proposed load, while the final iteration suggests that the girders are deficient.

Evaluation of the ‘PO’ girder using the modified shear method S6-06 M does not demonstrate the issue of non-convergent shear capacity displayed using S6-06 shear provisions, as demonstrated in Table C.2. This is due to the uniform diagonal crack spacing assumption, regardless of whether or not the member complies with stirrup spacing and area details at a section. The critical position for shear capacity evaluation remains at 0.09·L from the girder’s end using S6-06 M. As expected, the converged predicted capacity when using S6-06 M is between the two non-converged predictions made according to S6-06 Section 14. At convergence the $\beta$ and $\theta$ values are 0.400 and 30.3$^{\circ}$ respectively, which produces a predicted shear capacity of 2036 kN. The converged predicted shear capacity indicates that the bridge is able to carry the proposed load. Having a converged shear prediction removes the ambiguity demonstrated by using the sectional shear method in S6-06 as shown in this case study. Although the actual capacity of the girder is not known, the advantage of eliminating the discontinuity in evaluated shear capacity is the benefit of interest in this case study.

### 5.2.5 S6-06 M – Summary

The modified shear method S6-06 M is able to improve on S6-06 shear predictions by calculating average $V_{calc}/V_{test}$ ratios closer to 1.00 and a decreased correpsonding COV. These improvements are shown for both non-compliant and compliant members and are reflected in the low average demerit points per member (see Table 5.3). The average demerit points allotted to all member categories evaluated using S6-06 M is fewer than 7.50, which is the ‘appropriate’ upper limit recommended in Section 4.2. This indicates that the modified shear method S6-06 M will consistently present with predicted-to-tested shear capacity ratios ‘appropriately’ close to unity for both non-prestressed and prestressed members with stirrups. S6-06 M is also able to eliminate the issue of non-convergence that can result during sectional shear calculations, which is a significant advantage for shear capacity evaluations.

## 5.3 S6-06 F – Proposed Method

As discussed in Section 4.10, shear capacity evaluations using the provisions in S6-06 Section 8 for flanged members provide $V_{calc}/V_{test}$ ratios which are on average 17.9% lower (0.78 compared to 0.95) than the $V_{calc}/V_{test}$ ratios for the rectangular members. The conservative predictions of shear capacity calculated using S6-06 for members with compression flanges demonstrated in Section 4.5.3 are consistent with research by others (Moayer and Regan, 1971; Giaccio et al., 2002; Zsutty as cited in ACI-ASCE 426, 1973) using other shear methods. Based on these observations, a modified shear method based on S6-06 has been developed in this study which improves shear capacity predictions of members with compression flanges by including a portion of the flange in the concrete area term used to calculate the concrete contribution to shear capacity $V_c$. This modified shear method is termed S6-06 F.

The modified shear method S6-06 F uses the same process for predicting the sectional shear capacity as that discussed in Section 3.2 for S6-06, with the following modifications: 1) S6-06 F incorporates the crack spacing assumption used in the modified shear method S6-06 M and discussed in Section 5.2 and 2) S6-06 F incorporates a portion of the compression flange in the sectional shear capacity attributed to the concrete $V_c$. Figure 5.10 illustrates the geometry used in S6-06 F. The geometry used by the modified shear method S6-06 F is similar to that used by Zsutty (as cited in ACI-ASCE 426, 1973), the main difference being that the sectional geometry utilized by Zsutty used the flexural depth $d$ (see Section 2.5.4) while S6-06 F uses the shear depth $d_v$. The modified shear method S6-06 F determines the shear depth $d_v$ as the larger of $0.9 \cdot d$ and $0.72 \cdot h$; this is consistent with S6-06 Clause 8.9.1.5. S6-06 F includes checks to ensure that the portion of the compression flange assumed to contribute towards shear capacity does not extend beyond the actual section.

The geometry of the concrete assumed by S6-06 F to contribute toward sectional shear capacity varies considerably from that used by Tureyan et al. (2006) as shown in Figure 2.10. The geometry used by S6-06 F is consistent with that used by the sectional shear provisions in S6-06 Section 8 (see Section 3.2) which is the area of concrete that is effective for transferring shear stress (Bentz and Collins, 2006). The geometry used by Tureyan et al. (2006) is based solely on the compression zone contribution to shear capacity, which is inconsistent with shear provisions based on the Modified Compression Field Theory (Vecchio and Collins, 1986), as discussed in Section 2.4.2.

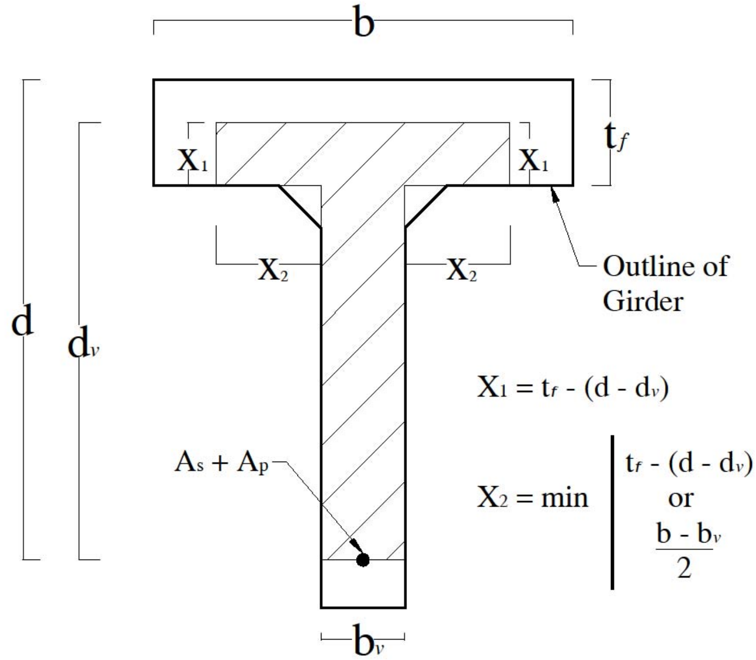



Figure 5.10 – Assumed Shape of Shear Area for Flanged Members using S6-06 F

The area of concrete assumed by S6-06 F to contribute to shear capacity is determined according to Eqn (5.1). This formulation is the same for members with and without stirrups.

$$A_{cv} = A_{web} + A_{flange} \qquad (\text{mm}^2) \qquad \text{Eqn. (5.1)}$$

The web contribution $A_{web}$ is calculated as the product of the shear depth $d_v$ and the shear width $b_v$, which is the same area used by the sectional shear method in S6-06 and the modified shear method S6-06 M.

The height of the flange contribution $X_1$ is consistently equal to $t_f - (d - d_v)$. For cases in which $t_f$ is less than $d$ - $d_v$ the concrete flange contribution term $A_{flange}$ is taken as zero. The flange width $X_2$ is limited to the lesser of the following:

- $t_f - (d - d_v)$. This flange width contribution is consistent with Zsutty (as cited in ASCE-ACI 426, 1973) as discussed in Section 2.5.4, the only difference being the use of $d_v$ as opposed to $d$ as discussed at the beginning of this section.

- $\frac{b-b_v}{2}$. This limit ensures that the area of the flange assumed to contribute to the sectional shear capacity attributed to the concrete $V_c$ does not extend horizontally beyond the width of the compression flange.

For T-sections and I-sections Eqn. (5.2) provides the expression used by the modified shear method S6-06 F to determine the area of the compression flange contributing to sectional shear capacity. For members with an L-shaped sectional geometry Eqn. (5.2) is divided by 2. As shown in Figure 5.10, in order to simplify determining the area of the flange assumed to contribute toward sectional shear capacity, S6-06 F ignores any chamfers between the compression flange and the web. Many other sectional geometries could be considered, but these are outside the scope of this study.

$$A_{flange} = 2 \cdot \left(t_f - \left(d - d_v\right)\right) \cdot \min \begin{cases} t_f - (d - d_v) \\ or \\ \frac{b-b_v}{2} \end{cases} \quad (\text{mm}^2) \qquad \text{Eqn. (5.2)}$$

Eqn. (5.3) is then used to determine the sectional shear capacity using the modified shear method S6-06 F. Eqn. (5.3) includes Eqn. (3.12), the interpolation for stirrup area $\gamma$ proposed in this study, to accommodate the stirrup area provisions in S6-06 Clause 14.14.1.6.2. Eqn. (3.12) is discussed in Section 3.2.

$$V_r = 2.5 \cdot \phi_c \cdot \beta \cdot f_{cr} \cdot A_{cv} + \gamma \cdot \frac{\phi_s \cdot A_v \cdot f_y \cdot d_v}{s \cdot \tan\theta} + \phi_p \cdot V_p \qquad (\text{kN}) \qquad \text{Eqn. (5.3)}$$

For shear capacity analysis in this study, the resistance factors $\phi_c$, $\phi_s$, and $\phi_p$ were taken at unity.

### 5.3.1 S6-06 F – Results from Shear Capacity Evaluation of Members with Stirrups and Comparison to S6-06 Evaluation

This section discusses the results of the shear capacity evaluation of the 130 members with stirrups analyzed in this study using the modified shear method S6-06 F and provides comparison to the results of the similar analysis in Section 4.5.1 pertaining to evaluation using S6-06 and S6-06 M. These comparisons are used to justify the proposed modifications in S6-06 F. The focus of this section is divided into four parts: i) tables providing the results from evaluation of members with stirrups using the modified shear method S6-06 F, ii) comparison of the mean statistical data between predictions using S6-06 F and S6-06 Section 8, iii) allocation of members into the prediction classifications given in Table 4.1 and average demerit points per member and iv) trends in the $V_{calc}/V_{test}$ ratios due to variations in the stirrup detail ratios $s/s_{m1}$ and $A_{v,min}/A_v$.

#### 5.3.1.i Statistical Data and Average Demerit Points per Member

Table 5.6 provides the $V_{calc}/V_{test}$, $s/s_{m1}$, $A_{v,min}/A_v$ and $b/b_v$ ratios, calculated using the modified shear method S6-06 F, for the 130 members with stirrups identified for this study. For comparison this table includes $V_{calc}/V_{test}$ ratios from evaluation using the sectional shear method in S6-06 Section 8 and the modified shear method S6-06 M. $V_{calc}/V_{test}$ ratios calculated using S6-06 F vary with respect to the corresponding $V_{calc}/V_{test}$ ratios calculated using S6-06 Section 8 provisions for 79.6% of non-compliant members and 97.5% of compliant members. This indicates that variations in the statistical data between predicted and tested shear capacities calculated using S6-06 F and S6-06 are representative of the entire dataset. The percent differences in $V_{calc}/V_{test}$ ratios between evaluation using S6-06 F and the shear methods S6-06 and S6-06 M are also included in Table 5.6. Positive percent differences signify that the $V_{calc}/V_{test}$ ratio calculated using S6-06 F has increased with respect to the corresponding shear prediction calculated using S6-06 Section 8 or S6-06 M, which indicates that S6-06 F attributes greater sectional shear capacity to a member than does the shear method to which S6-06 F is being compared. A summary of the average $V_{calc}/V_{test}$ ratios, COV and average demerit points per member for the 130 members with stirrups is provided in Table 5.7.

Table 5.6 – Results of Evaluation using S6-06 F – 130 Members with Stirrups

| Member | S6-06 F $V_{calc}/V_{test}$ | S6-06 $V_{calc}/V_{test}$ | S6-06 M $V_{calc}/V_{test}$ | $V_{calc}/V_{test}$ Percent Difference (%) | | $s/s_{m1}$ | $A_{v,min}/A_v$ | $b/b_v$ |
|---|---|---|---|---|---|---|---|---|
| | | | | S6-06 | S6-06 M | | | |
| Non-Prestressed Non-Compliant | | | | | | | | |
| YB2000/9 | 0.70 | 0.70 | 0.70 | 0.0 | 0.00 | 4.50 | 0.94 | 1.00 |
| YB2000/6 | 0.66 | 0.58 | 0.66 | 13.8 | 0.00 | 2.25 | 1.12 | 1.00 |
| YB2000/4 | 0.76 | 0.48 | 0.76 | 58.3 | 0.00 | 0.98 | 1.08 | 1.00 |
| 5084 | 0.72 | 0.65 | 0.65 | 10.8 | 10.77 | 2.55 | 0.44 | 3.02 |
| 5053 | 0.76 | 0.67 | 0.67 | 13.4 | 13.43 | 3.26 | 0.57 | 3.02 |
| 5052 | 0.75 | 0.66 | 0.66 | 13.6 | 13.64 | 3.57 | 0.63 | 3.03 |
| 5051 | 0.71 | 0.63 | 0.63 | 12.7 | 12.70 | 3.93 | 0.68 | 3.03 |
| 5063 | 0.69 | 0.61 | 0.61 | 13.1 | 13.11 | 2.96 | 0.54 | 3.03 |
| N2-S | 1.04 | 0.98 | 1.04 | 6.1 | 0.00 | 1.05 | 1.03 | 1.00 |
| N1-N | 0.89 | 0.78 | 0.89 | 14.1 | 0.00 | 0.74 | 1.02 | 1.00 |
| P21 | 0.82 | 0.76 | 0.77 | 7.9 | 6.49 | 1.22 | 0.90 | 4.00 |
| Ss2-321-3 | 1.08 | 1.08 | 1.08 | 0.0 | 0.00 | 1.73 | 1.36 | 1.00 |
| Ss2-318-3 | 0.92 | 0.92 | 0.92 | 0.0 | 0.00 | 1.49 | 1.17 | 1.00 |
| Ss2-313.5-3 | 0.87 | 0.84 | 0.87 | 3.6 | 0.00 | 1.11 | 0.87 | 1.00 |
| Ss2-321-2 | 1.00 | 1.00 | 1.00 | 0.0 | 0.00 | 1.73 | 1.00 | 1.00 |
| Ss2-318-2 | 0.98 | 0.98 | 0.98 | 0.0 | 0.00 | 1.49 | 0.87 | 1.00 |
| Ss2-321-1 | 1.13 | 1.13 | 1.13 | 0.0 | 0.00 | 1.73 | 0.69 | 1.00 |
| Ss2-318-1 | 0.89 | 0.89 | 0.89 | 0.0 | 0.00 | 1.49 | 0.60 | 1.00 |
| Ss2-218a-2 | 0.83 | 0.83 | 0.83 | 0.0 | 0.00 | 1.49 | 1.77 | 1.00 |
| Ss2-213.5-2 | 0.94 | 0.91 | 0.94 | 3.3 | 0.00 | 1.11 | 1.32 | 1.00 |
| Ss2-213.5-1 | 1.00 | 0.97 | 1.00 | 3.1 | 0.00 | 1.11 | 1.47 | 1.00 |
| J & R - 7 | 1.05 | 0.95 | 1.05 | 10.5 | 0.00 | 0.74 | 1.14 | 1.00 |
| J & R - 8 | 1.14 | 1.03 | 1.14 | 10.7 | 0.00 | 0.73 | 1.14 | 1.00 |
| BM100 | 0.82 | 0.73 | 0.82 | 12.3 | 0.00 | 1.00 | 1.03 | 1.00 |
| SB 2012/6 | 0.66 | 0.58 | 0.66 | 13.8 | 0.00 | 2.25 | 0.92 | 1.00 |
| SB 2003/6 | 0.83 | 0.72 | 0.83 | 15.3 | 0.00 | 2.25 | 0.99 | 1.00 |
| 10T24 | 0.83 | 0.71 | 0.82 | 16.9 | 1.22 | 1.02 | 0.91 | 3.00 |
| PE1 | 0.81 | 0.78 | 0.78 | 3.8 | 3.85 | 2.14 | 1.14 | 2.20 |
| PE2 | 0.76 | 0.74 | 0.74 | 2.7 | 2.70 | 2.74 | 1.46 | 2.20 |
| Prestressed Non-Compliant | | | | | | | | |
| CH-6-240 | 0.77 | 0.73 | 0.74 | 5.5 | 4.05 | 1.19 | 0.39 | 2.98 |
| CM-6-240 | 0.75 | 0.71 | 0.72 | 5.6 | 4.17 | 1.19 | 0.47 | 2.98 |
| CL-6-240 | 0.69 | 0.64 | 0.65 | 7.8 | 6.15 | 1.19 | 0.62 | 2.98 |
| PH-6-240 | 0.91 | 0.86 | 0.87 | 5.8 | 4.60 | 1.42 | 0.34 | 2.98 |
| PM-6-240 | 0.82 | 0.77 | 0.78 | 6.5 | 5.13 | 1.33 | 0.44 | 2.98 |
| PL-6-240 | 0.75 | 0.70 | 0.71 | 7.1 | 5.63 | 1.27 | 0.55 | 2.98 |
| NM-10-240 | 0.99 | 0.96 | 0.97 | 3.1 | 2.06 | 1.98 | 0.14 | 2.98 |
| NL-10-240 | 0.87 | 0.83 | 0.84 | 4.8 | 3.57 | 1.68 | 0.23 | 2.98 |
| NM-8-240 | 1.04 | 0.99 | 1.00 | 5.1 | 4.00 | 1.99 | 0.20 | 2.98 |
| NH-6-240 | 0.80 | 0.76 | 0.77 | 5.3 | 3.90 | 1.63 | 0.31 | 2.98 |
| NM-6-240 | 0.77 | 0.73 | 0.74 | 5.5 | 4.05 | 1.44 | 0.39 | 2.98 |
| NL-6-240 | 0.80 | 0.75 | 0.76 | 6.7 | 5.26 | 1.32 | 0.53 | 2.98 |
| CW12 | 1.01 | 0.83 | 0.87 | 21.7 | 16.09 | 2.34 | 0.16 | 3.98 |
| CW11 | 0.96 | 0.78 | 0.82 | 23.1 | 17.07 | 1.57 | 0.19 | 3.98 |
| CI12 | 1.03 | 1.00 | 1.03 | 3.0 | 0.00 | 1.62 | 0.19 | 6.98 |
| CI11 | 1.05 | 1.01 | 1.05 | 4.0 | 0.00 | 1.21 | 0.22 | 6.98 |
| P9 | 0.73 | 0.69 | 0.69 | 5.8 | 5.80 | 1.25 | 0.94 | 4.00 |
| P14 | 0.73 | 0.69 | 0.69 | 5.8 | 5.80 | 1.20 | 0.98 | 4.00 |
| P19 | 0.77 | 0.76 | 0.76 | 1.3 | 1.32 | 1.26 | 1.00 | 4.00 |
| BW.14.34 | 1.00 | 0.87 | 0.87 | 14.9 | 14.94 | 1.88 | 0.99 | 2.03 |
| Non-Prestressed Compliant | | | | | | | | |
| V18-2 | 0.95 | 0.90 | 0.95 | 5.6 | 0.00 | 0.65 | 0.50 | 1.00 |
| V36-3 | 1.02 | 1.05 | 1.02 | -2.9 | 0.00 | 0.65 | 0.87 | 1.00 |

| Table 5.6 continued | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Member | S6-06 F $V_{calc}/V_{test}$ | S6-06 $V_{calc}/V_{test}$ | S6-06 M $V_{calc}/V_{test}$ | $V_{calc}/V_{test}$ Percent Difference (%) | | $s/s_{m1}$ | $A_{v,min}/A_v$ | $b/b_v$ |
| | | | | S6-06 | S6-06 M | | | |
| Non-Prestressed Compliant | | | | | | | | |
| V36-2 | 1.14 | 1.08 | 1.14 | 5.6 | 0.00 | 0.29 | 0.99 | 1.00 |
| V1 | 1.24 | 1.28 | 1.24 | -3.1 | 0.00 | 0.65 | 0.90 | 1.00 |
| V2 | 1.00 | 1.03 | 1.00 | -2.9 | 0.00 | 0.65 | 0.90 | 1.00 |
| A1 | 0.86 | 0.83 | 0.86 | 3.6 | 0.00 | 0.68 | 0.39 | 1.00 |
| A2 | 0.93 | 0.89 | 0.93 | 4.5 | 0.00 | 0.68 | 0.42 | 1.00 |
| B1 | 0.89 | 0.84 | 0.89 | 6.0 | 0.00 | 0.62 | 0.27 | 1.00 |
| B2 | 1.01 | 0.96 | 1.01 | 5.2 | 0.00 | 0.62 | 0.29 | 1.00 |
| C1 | 1.09 | 1.05 | 1.09 | 3.8 | 0.00 | 0.68 | 0.22 | 1.00 |
| C2 | 1.02 | 0.98 | 1.02 | 4.1 | 0.00 | 0.69 | 0.21 | 1.00 |
| P5 | 0.68 | 0.59 | 0.66 | 15.3 | 3.03 | 0.53 | 0.37 | 4.00 |
| P20 | 0.71 | 0.61 | 0.68 | 16.4 | 4.41 | 0.81 | 0.57 | 4.00 |
| P22 | 0.71 | 0.63 | 0.67 | 12.7 | 5.97 | 0.81 | 0.55 | 4.00 |
| DBO530M | 1.01 | 1.01 | 1.01 | 0.0 | 0.00 | 0.50 | 0.85 | 1.00 |
| DB120M | 0.93 | 1.10 | 0.93 | -15.5 | 0.00 | 1.00 | 0.69 | 1.00 |
| DM140M | 1.26 | 1.26 | 1.26 | 0.0 | 0.00 | 0.50 | 0.92 | 1.00 |
| Ss2-29g-2 | 1.03 | 0.99 | 1.03 | 4.0 | 0.00 | 0.74 | 0.58 | 1.00 |
| Ss2-29e-2 | 0.97 | 0.93 | 0.96 | 4.3 | 1.04 | 0.74 | 0.99 | 1.00 |
| Ss2-29d-2 | 1.08 | 1.04 | 1.08 | 3.8 | 0.00 | 0.74 | 0.80 | 1.00 |
| Ss2-29c-2 | 1.05 | 1.01 | 1.05 | 4.0 | 0.00 | 0.74 | 0.71 | 1.00 |
| Ss2-29b-2 | 0.94 | 0.91 | 0.94 | 3.3 | 0.00 | 0.74 | 0.94 | 1.00 |
| Ss2-29a-2 | 0.86 | 0.83 | 0.86 | 3.6 | 0.00 | 0.74 | 0.89 | 1.00 |
| Ss2-29b-1 | 1.14 | 1.10 | 1.14 | 3.6 | 0.00 | 0.74 | 0.97 | 1.00 |
| Ss2-29a-1 | 1.15 | 1.11 | 1.15 | 3.6 | 0.00 | 0.74 | 1.00 | 1.00 |
| Ss2-26-1 | 1.03 | 0.96 | 1.03 | 7.3 | 0.00 | 0.49 | 0.67 | 1.00 |
| 1 | 1.09 | 1.01 | 1.09 | 7.9 | 0.00 | 0.38 | 0.48 | 1.00 |
| 5 | 1.04 | 0.96 | 1.04 | 8.3 | 0.00 | 0.38 | 0.59 | 1.00 |
| 5A-0 | 0.83 | 0.77 | 0.82 | 7.8 | 1.22 | 0.98 | 0.09 | 6.00 |
| 5B-0 | 0.81 | 0.75 | 0.80 | 8.0 | 1.25 | 0.98 | 0.09 | 6.00 |
| Test 1.1 | 0.84 | 0.82 | 0.83 | 2.4 | 1.20 | 0.87 | 0.27 | 2.86 |
| Test 2.1 | 0.99 | 0.95 | 0.97 | 4.2 | 2.06 | 0.87 | 0.27 | 3.00 |
| Test 2.2 | 1.04 | 0.99 | 1.01 | 5.1 | 2.97 | 0.87 | 0.27 | 3.00 |
| Test 2.3 | 0.99 | 0.96 | 0.98 | 3.1 | 1.02 | 0.87 | 0.28 | 3.00 |
| T1 | 0.72 | 0.72 | 0.72 | 0.0 | 0.00 | 0.82 | 0.07 | 15.00 |
| ET1 | 1.02 | 0.94 | 1.02 | 8.5 | 0.00 | 0.54 | 0.56 | 1.00 |
| ET2 | 0.94 | 0.81 | 0.88 | 16.0 | 6.82 | 0.57 | 0.28 | 2.00 |
| ET3 | 0.88 | 0.76 | 0.82 | 15.8 | 7.32 | 0.75 | 0.19 | 3.00 |
| 2T10 | 1.01 | 0.98 | 1.00 | 3.1 | 1.00 | 0.43 | 0.21 | 3.00 |
| 2T12 | 0.96 | 0.96 | 0.96 | 0.0 | 0.00 | 0.51 | 0.25 | 3.00 |
| 1T18 | 0.90 | 0.96 | 0.90 | -6.2 | 0.00 | 0.76 | 0.45 | 3.00 |
| Prestressed Compliant | | | | | | | | |
| PL-6-160 | 0.77 | 0.69 | 0.74 | 11.6 | 4.05 | 0.96 | 0.35 | 3.00 |
| NL-6-80 | 0.89 | 0.81 | 0.87 | 9.9 | 2.30 | 0.67 | 0.17 | 3.00 |
| NH-6-80 | 0.98 | 0.89 | 0.96 | 10.1 | 2.08 | 0.90 | 0.11 | 3.00 |
| NM-6-80 | 0.97 | 0.88 | 0.95 | 10.2 | 2.11 | 0.90 | 0.13 | 3.00 |
| NL-6-160 | 0.81 | 0.73 | 0.78 | 11.0 | 3.85 | 1.01 | 0.34 | 3.00 |
| NM-6-160 | 0.86 | 0.78 | 0.83 | 10.3 | 3.61 | 1.14 | 0.26 | 3.00 |
| CH-6-80 | 0.81 | 0.75 | 0.80 | 8.0 | 1.25 | 0.55 | 0.13 | 3.00 |
| CM-6-80 | 0.84 | 0.77 | 0.83 | 9.1 | 1.20 | 0.48 | 0.16 | 3.00 |
| CL-6-80 | 0.85 | 0.77 | 0.83 | 10.4 | 2.41 | 0.45 | 0.21 | 3.00 |
| CH-6-160 | 0.80 | 0.75 | 0.78 | 6.7 | 2.56 | 0.84 | 0.27 | 3.00 |
| CL-6-160 | 0.80 | 0.71 | 0.76 | 12.7 | 5.26 | 0.80 | 0.42 | 3.00 |
| PM-6-80 | 0.91 | 0.83 | 0.89 | 9.6 | 2.25 | 0.68 | 0.14 | 3.00 |
| PM-6-160 | 0.86 | 0.77 | 0.82 | 11.7 | 4.88 | 1.04 | 0.27 | 3.00 |
| PL-6-80 | 0.90 | 0.82 | 0.88 | 9.8 | 2.27 | 0.59 | 0.18 | 3.00 |

| Table 5.6 continued | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Member | S6-06 F $V_{calc}/V_{test}$ | S6-06 $V_{calc}/V_{test}$ | S6-06 M $V_{calc}/V_{test}$ | $V_{calc}/V_{test}$ Percent Difference (%) | | $s/s_{m1}$ | $A_{v,min}/A_v$ | $b/b_v$ |
| | | | | S6-06 | S6-06 M | | | |
| Prestressed Compliant | | | | | | | | |
| A1-00-1.5R-N | 1.23 | 1.17 | 1.22 | 5.1 | 0.82 | 0.32 | 0.07 | 3.00 |
| B0-00-R-S | 0.71 | 0.72 | 0.70 | -1.4 | 1.43 | 0.74 | 0.23 | 3.00 |
| B0-00-R-N | 0.99 | 0.96 | 0.98 | 3.1 | 1.02 | 0.42 | 0.11 | 3.00 |
| A1-00-M-N | 1.17 | 1.18 | 1.16 | -0.8 | 0.86 | 0.76 | 0.24 | 3.00 |
| A1-00-R_N | 1.02 | 0.99 | 1.01 | 3.0 | 0.99 | 0.43 | 0.10 | 3.00 |
| A1-00-M-S | 0.87 | 0.88 | 0.86 | -1.1 | 1.16 | 0.76 | 0.24 | 3.00 |
| A1-00-0.5R-N | 1.02 | 1.03 | 1.01 | -1.0 | 0.99 | 0.72 | 0.21 | 3.00 |
| A1-00-0.5R-S | 0.86 | 0.87 | 0.84 | -1.1 | 2.38 | 0.74 | 0.23 | 3.00 |
| 2A-3 | 0.77 | 0.73 | 0.77 | 5.5 | 0.00 | 0.98 | 0.10 | 6.00 |
| 2B-3 | 0.77 | 0.74 | 0.77 | 4.1 | 0.00 | 0.99 | 0.10 | 6.00 |
| 3A-2 | 0.78 | 0.73 | 0.77 | 6.8 | 1.30 | 0.98 | 0.10 | 6.00 |
| 3B-2 | 0.85 | 0.80 | 0.84 | 6.2 | 1.19 | 0.98 | 0.09 | 6.00 |
| 4A-1 | 0.77 | 0.72 | 0.76 | 6.9 | 1.32 | 0.92 | 0.10 | 6.00 |
| 4B-1 | 0.80 | 0.75 | 0.80 | 6.7 | 0.00 | 0.98 | 0.09 | 6.00 |
| P4 | 0.74 | 0.66 | 0.72 | 12.1 | 2.78 | 0.53 | 0.35 | 4.00 |
| P8 | 0.69 | 0.63 | 0.66 | 9.5 | 4.55 | 0.83 | 0.59 | 4.00 |
| P13 | 0.68 | 0.62 | 0.65 | 9.7 | 4.62 | 0.80 | 0.57 | 4.00 |
| P18 | 0.74 | 0.68 | 0.71 | 8.8 | 4.23 | 0.84 | 0.60 | 4.00 |
| P24 | 0.76 | 0.68 | 0.73 | 11.8 | 4.11 | 0.54 | 0.37 | 4.00 |
| P25 | 0.79 | 0.71 | 0.76 | 11.3 | 3.95 | 0.80 | 0.55 | 4.00 |
| P26 | 0.77 | 0.70 | 0.74 | 10.0 | 4.05 | 0.56 | 0.38 | 4.00 |
| P27 | 0.81 | 0.75 | 0.77 | 8.0 | 5.19 | 0.84 | 0.56 | 4.00 |
| P28 | 0.71 | 0.64 | 0.69 | 10.9 | 2.90 | 0.55 | 0.37 | 4.00 |
| P29 | 0.75 | 0.68 | 0.72 | 10.3 | 4.17 | 0.82 | 0.57 | 4.00 |
| P49 | 0.77 | 0.68 | 0.73 | 13.2 | 5.48 | 0.56 | 0.34 | 4.00 |
| P50 | 0.79 | 0.72 | 0.77 | 9.7 | 2.60 | 0.61 | 0.19 | 4.00 |

Table 5.7 - Summary of Results Comparing S6-06 F and S6-06 Sectional Shear Provisions – Members with Stirrups

| Test Group (number) | Mean $V_{calc}/V_{test}$ Ratio S6-06 F | $V_{calc}/V_{test}$ C.O.V (%) S6-06 F | Average Demerit Points / Member S6-06 F | Mean $V_{calc}/V_{test}$ Ratio S6-06 | $V_{calc}/V_{test}$ C.O.V (%) S6-06 | Average Demerit Points / Member S6-06 |
|---|---|---|---|---|---|---|
| All Members (163) | 0.89 | 16.3 | 4.47 | 0.84 | 19.2 | 5.92 |
| All Members with Stirrups (130) | 0.89 | 15.7 | 4.30 | 0.84 | 18.8 | 5.99 |
| Non-Compliant Non-Prestressed Members (29) | 0.86 | 16.4 | 4.79 | 0.80 | 20.8 | 6.84 |
| Compliant Non-Prestressed Members (41) | 0.97 | 13.8 | 3.36 | 0.93 | 16.3 | 3.91 |
| Non-Compliant Prestressed Members (20) | 0.86 | 14.3 | 3.89 | 0.80 | 14.7 | 6.02 |
| Compliant Prestressed Members (40) | 0.84 | 14.2 | 5.12 | 0.78 | 16.6 | 7.49 |

### 5.3.1.ii S6-06 F – Comparison of Statistical Results with S6-06 Section 8

As shown in Table 5.7 the agreement between predicted and tested shear capacities calculated using the modified shear method S6-06 F is improved for both the twenty-nine non-prestressed members with non-compliant stirrup spacing and area details and the forty-one non-prestressed members meeting minimum stirrup requirements relative to predictions using the shear provisions in S6-06 Section 8. The twenty-nine non-compliant non-prestressed members and forty-one compliant non-prestressed members show improvements of 7.5% (0.86 compared to 0.80) and 4.3% (0.97 compared to 0.93) in the mean $V_{calc}/V_{test}$ ratio respectively. As discussed in Section 5.2.2.ii, improvements in $V_{calc}/V_{test}$ ratios are defined in this study as having $V_{calc}/V_{test}$ ratios closer to unity relative to predictions of capacity using another sectional shear method. The COV of the twenty-nine non-compliant non-prestressed members and forty-one compliant non-prestressed members, calculated from predictions of shear capacity using S6-06 F, decrease by 21.2% (16.4% compared to 20.8%) and 15.3% (13.8% compared to 16.3%) respectively, compared to predictions using the shear method in S6-06.

As inferred from Table 5.7 the agreement between predicted and tested shear capacities, calculated using the modified shear method S6-06 F, is improved for the prestressed categories of members evaluated in this study relative to predictions of shear capacity using the provisions in S6-06 Section 8. The twenty prestressed members with non-compliant stirrup spacing details show a 7.5% improvement (0.86 compared to 0.80) in their average $V_{calc}/V_{test}$ ratios and a decrease of 2.7% (14.3% compared to 14.7%) in their corresponding COV compared to predictions using the sectional shear method in S6-06. Evaluations of shear capacity using S6-06 F for the forty compliant prestressed members show an improvement of 7.7% (0.84 compared to 0.78) in their mean $V_{calc}/V_{test}$ ratio and a decrease in the corresponding COV of 14.5% (14.2% compared to 16.6%).

Table 5.7 shows that the compliant members for both non-prestressed and prestressed categories are calculated to have a larger average $V_{calc}/V_{test}$ ratio than are the non-compliant members, indicating that the modified shear method S6-06 F may result in more conservative predictions of shear capacity for non-compliant members than for compliant members. This is consistent with shear capacity evaluations calculated using the sectional shear method in S6-06 Section 8, as discussed in Section 4.5.4. Compliant and non-compliant members have similar ratios of the number of flanged to rectangular members – thus the difference in average $V_{calc}/V_{test}$ ratios is not affected by the shape of the cross section.

### 5.3.1.iii S6-06 F - Prediction Classifications and Average Demerit Points per Member – Members with Stirrups

Table 5.8 distributes the shear capacity predictions of the seventy non-prestressed members with stirrups evaluated in this study into the classification ranges given in Table 4.1, and provides the average demerit points per member for the full dataset of non-prestressed members with stirrups, as well as for the compliant and non-compliant member categories. Table 5.9 provides the same information for the sixty prestressed members with stirrups identified for evaluation in this study.

Table 5.8 - S6-06 F –Non-prestressed Members - $V_{calc}/V_{test}$ Ranges and Demerit Points

| **S6-06 F - Non-Prestressed Members with Stirrups** | **Full Data Set** | **Non-Compliant** | **Compliant** | **Demerits - Full Data Set** | **Demerits - Non-Compliant** | **Demerits - Compliant** |
|---|---|---|---|---|---|---|
| Total | 70 | 29 | 41 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 11<br>15.7% | 7<br>24.1% | 4<br>9.8% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 57<br>81.4% | 22<br>75.9% | 35<br>85.4% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 2<br>2.9% | 0<br>0.0% | 2<br>4.9% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 277 | 139 | 138 |
| Average Demerits/Members | | | | 3.95 | 4.79 | 3.36 |

Table 5.8 indicates that for the twenty-nine non-compliant non-prestressed members with stirrups evaluated in this study, twenty-two members result in predictions in the range considered to be 'appropriate' (see Table 4.1). This is an improvement over the predictions made using the shear provisions in S6-06 and the modified shear method S6-06 M, which have sixteen and twenty members respectively in the range considered to be 'appropriate' (see Tables 4.8 and 5.4). Thus it can be seen that, relative to predictions of shear capacity calculated using S6-06 Section 8, S6-06 F presents with only two extra $V_{calc}/V_{test}$ ratios in the 'appropriate' range compared to evaluation using S6-06 M. This comparison indicates that the diagonal crack spacing assumption used by the two modified shear methods will have a more beneficial effect on the agreement between predicted and tested shear capacities than will the sectional geometry used by S6-06 F for

calculating the portion of the compression flange attributed to sectional shear capacity. Of the forty-one compliant non-prestressed members evaluated in this study using the sectional shear provisions in S6-06 F, thirty-five are in the 'appropriate' range. This is the same distribution as is determined from evaluation using the sectional shear method in S6-06 Section 8 (see Table 4.8). The majority of members evaluated as 'conservative' using the modified shear S6-06 F are flanged members, indicating that the increased concrete area assumed does not completely account for the shear capacity exhibited by flanged members. Evaluation of shear capacities using S6-06 F does not present with any $V_{calc}/V_{test}$ ratios in the 'low safety' or more unsafe ranges (see Table 4.1) for non-compliant members, which suggests that the concrete area modification will not produce unsafe predictions.

Predictions of shear capacity calculated using S6-06 F are allotted 30.0% (4.79 compared to 6.84) and 14.1% (3.36 compared to 3.91) fewer average demerit points per non-prestressed, non-compliant and compliant member respectively than from evaluation using S6-06 (see Table 5.7), using the demerit point model proposed in Section 4.2. This suggests that S6-06 F will consistently determine predicted-to-tested shear capacity ratios closer to unity than will the sectional shear method in S6-06 Section 8. This improvement is primarily a result of the diagonal crack spacing assumption used by the modified shear methods and discussed in Section 5.2. However S6-06 F is allotted 15.1% (4.79 compared to 5.64) and 6.9% (3.36 compared to 3.61) fewer average demerit points per non-prestressed, non-compliant and compliant member respectively relative to evaluation using S6-06 M, indicating that shear predictions for both categories of members benefit from the concrete area attributed to shear capacity by S6-06 F.

Table 5.9 – S6-06 F : Prestressed Members - $V_{calc}/V_{test}$ Ranges and Demerit Points

| **S6-06 F – Prestressed Members with Stirrups** | **Full Data Set** | **Non-Compliant** | **Compliant** | **Demerits - Full Data Set** | **Demerits - Non-Compliant** | **Demerits - Compliant** |
|---|---|---|---|---|---|---|
| Total | 60 | 20 | 40 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range = 0.50 - 0.75<br>Percent of Total | 14<br>23.3% | 7<br>35.0% | 7<br>17.5% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range = 0.75 - 1.15<br>Percent of Total | 44<br>73.3% | 13<br>65.0% | 31<br>77.5% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range = 1.15 - 1.50<br>Percent of Total | 2<br>3.3% | 0<br>0.0% | 2<br>5.0% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range = 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 282 | 78 | 205 |
| Average Demerits/Members | | | | 4.71 | 3.89 | 5.12 |

As indicated by Table 5.9, $V_{calc}/V_{test}$ ratios for thirteen of the twenty non-compliant members evaluated using the modified shear method S6-06 F are determined to be in the 'appropriate' range. This compares well to predictions using the shear provisions in S6-06 and the modified shear method S6-06 M, where ten and twelve members respectively are in the 'appropriate' range. Of the forty compliant prestressed members evaluated in this study using S6-06 F, thirty-one are calculated to have $V_{calc}/V_{test}$ ratios in the 'appropriate' range. This is a considerable improvement over the shear predictions made in this study using S6-06 Section 8 provisions, where sixteen members are determined to be in the 'appropriate' range (see Table 4.9). As discussed in Section 5.2.2.iii, twenty-eight of the forty compliant prestressed members evaluated using the modified shear method S6-06 M are determined to have $V_{calc}/V_{test}$ ratios in the 'appropriate' range (see Table 5.5). The fact that the diagonal crack spacing assumption used by S6-06 M is responsible for twelve of these improved $V_{calc}/V_{test}$ ratios, relative to evaluation using S6-06, indicates that the diagonal crack spacing assumption has a larger influence on improving the agreement between predicted and tested shear capacities than does the concrete geometry modification used solely by S6-06 F. None of the predictions of shear capacity calculated using S6-06 F were found to be unsafe (see Table 4.1) for prestressed non-compliant members.

The average demerit points allotted to the non-compliant members is 24.0% fewer than the average demerit point per compliant member (3.89 compared to 5.12) based on evaluation using the proposed shear method S6-06 F. This difference is considerably larger than for predictions calculated using S6-06 M, which are discussed in Section 5.2.2.iii; however this difference is not influenced by a large value of demerit points being allotted to any single compliant prestressed member. This is believed to be a result of the non-compliant members being smaller sections, which cause the flange area contribution to constitute a larger portion of the predicted shear capacity. However, S6-06 F is allotted fewer average demerit points per member than S6-06 (see Table 4.9) or S6-06 M (see Table 5.5) for both compliant and non-compliant prestressed members, indicating that S6-06 F will typically predict shear capacities closer to the actual capacity than will the other two sectional shear methods.

### 5.3.1.iv S6-06 F - Relationship Between $V_{calc}/V_{test}$ Ratios and Stirrup Detail Ratios $s/s_{m1}$ and $A_{v,min}/A_v$

Evaluation of Non-Prestressed Members with Stirrups using S6-06 F

Figure 5.11 provides the relationship $V_{calc}/V_{test}$ vs. $s/s_{m1}$ for the seventy non-prestressed members with stirrups, while Figure 5.12 provides the relationship $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for the same members. These figures are used to study the influence that variations in stirrup spacing and area have on the agreement between predicted and tested shear capacities calculated using the modified shear method S6-06 F. The solid line in each of the following figures represents exact shear predictions, while the two dashed lines define the range of 'appropriate predictions' as defined in Section 4.2 (see Table 4.1). Members evaluated in this study are classified as non-compliant if either of the stirrup detail ratios, $s/s_{m1}$ or $A_{v,min}/A_v$, is greater than 1.00. Summary statistical data for the member categories is provided in Figures 5.11 and 5.12.

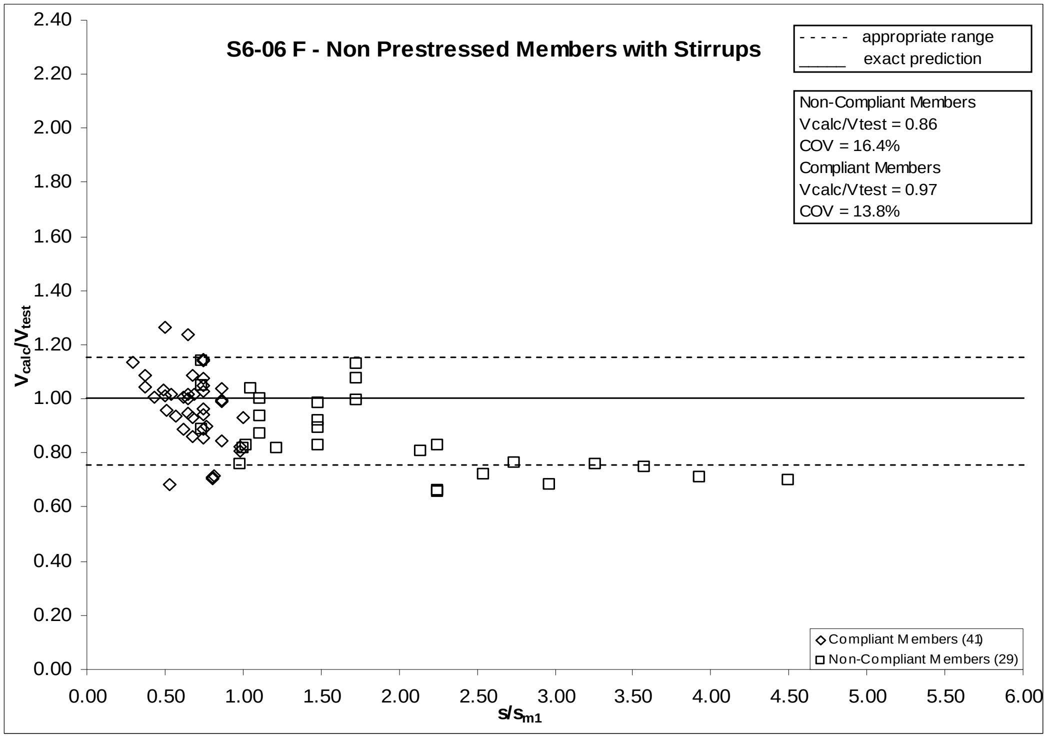



Figure 5.11 - $V_{calc}/V_{test}$ vs. $s/s_{m1}$ for 70 Non-Prestressed Members with Stirrups Evaluated using S6-06 F

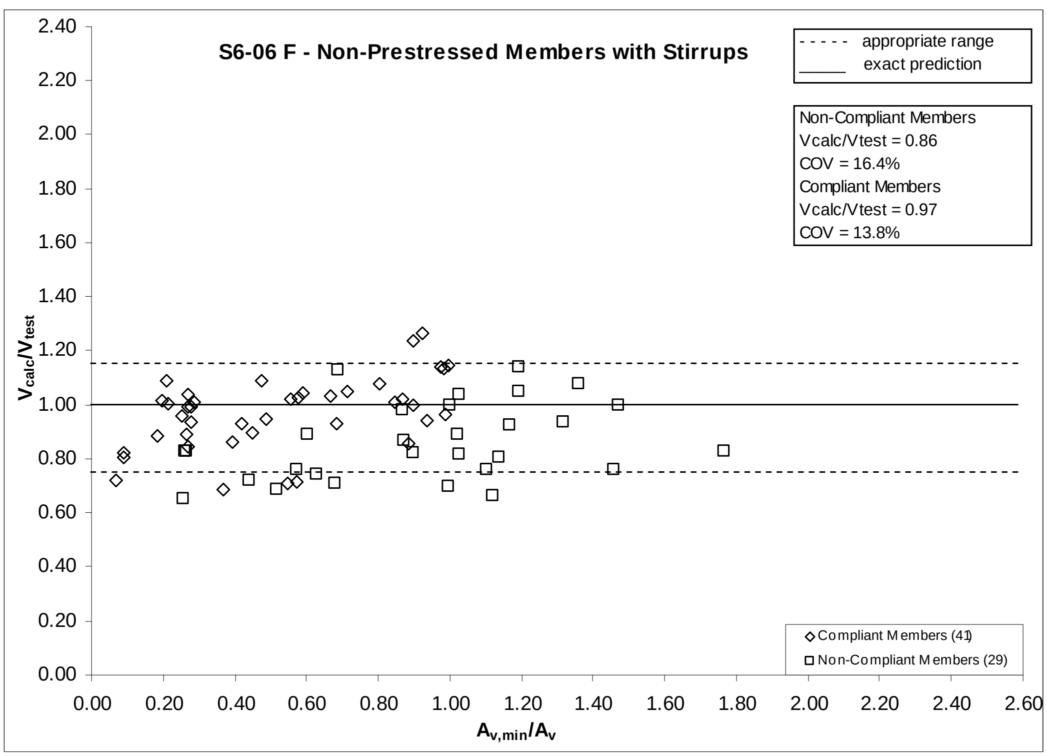



Figure 5.12 - $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for 70 Non-Prestressed Members with Stirrups Evaluated using S6-06 F

The lack of defined trends in $V_{calc}/V_{test}$ ratios shown in Figures 5.11 and 5.12 indicates that the agreement between predicted and tested shear capacities for the twenty-nine non-prestressed members not complying with minimum transverse reinforcement requirements is not influenced by variations in the stirrup spacing and area ratios $s/s_{m1}$ and $A_{v,min}/A_v$. This lack of trends in $V_{calc}/V_{test}$ ratios is similar to behavior demonstrated from shear capacity evaluation using S6-06 (see Figures 4.6 and 4.7). The modified area term for flanged members used in S6-06 F appears to reduce the occurrence of apparent outlier predictions compared to evaluation using S6-06 as discussed in Section 4.5.1. This is demonstrated in Figure 5.11 by the positive vertical shift of the $V_{calc}/V_{test}$ ratios for the flanged members having $s/s_{m1}$ ratios greater than 2.00, relative to evaluation using S6-06 Section 8 (see Figure 4.6). In order to further quantify improvements of shear predictions calculated using S6-06 F, members with improvements in percent difference of $V_{calc}/V_{test}$ ratios greater than 10% with respect to S6-06 predictions have been identified. As discussed earlier in Section 5.2.2.iii, improvement in $V_{calc}/V_{test}$ ratios is defined in this study as having $V_{calc}/V_{test}$ ratios closer to unity, relative to predictions of capacity using another sectional shear method. Twelve data points from the non-compliant non-prestressed member category show improvements greater than 10% in $V_{calc}/V_{test}$ ratios when compared to S6-06 shear predictions. Seven of these improved predictions are for the same members improved by the modified shear method S6-06 M as discussed in Section 5.2.2.iv, while the other five are flanged member. The improvement in $V_{calc}/V_{test}$ ratios for members with compression flanges indicates that the concrete area assumed to contribute to shear capacity by the modified shear method S6-06 F will allow engineers to make more economical decisions regarding the shear capacity of flanged members.

Neither Figure 5.11 nor Figure 5.12 exhibit any discernable trends in $V_{calc}/V_{test}$ due to changes in $s/s_{m1}$ and $A_{v,min}/A_v$ ratios for the compliant non-prestressed category of members, further indicating that S6-06 F appropriately accounts for variations in stirrup spacing and area details. The modifications in S6-06 F cause $V_{calc}/V_{test}$ ratios of six compliant non-prestressed members to have improvements greater than 10% compared to evaluations of shear capacity using the provisions in S6-06 Section 8. Five of these are increased $V_{calc}/V_{test}$ ratios. The improved prediction which presents with a decreased $V_{calc}/V_{test}$ ratio is discussed in Section 5.2.2.iv.

Evaluation of Prestressed Members with Stirrups using S6-06 F

Figure 5.13 provides the relationship $V_{calc}/V_{test}$ vs. $s/s_{m1}$ for the sixty prestressed members with stirrups, while Figure 5.14 provides the relationship $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for the same members. These figures are used to study the influence that variations in stirrup spacing and area have on the agreement between predicted and tested shear capacities for prestressed members, calculated using the modified shear method S6-06 F. The solid line represents the condition where the tested shear capacity is equal to the predicted shear capacity. The upper and lower dashed lines define the boundaries that are considered 'appropriate' predictions in this study.

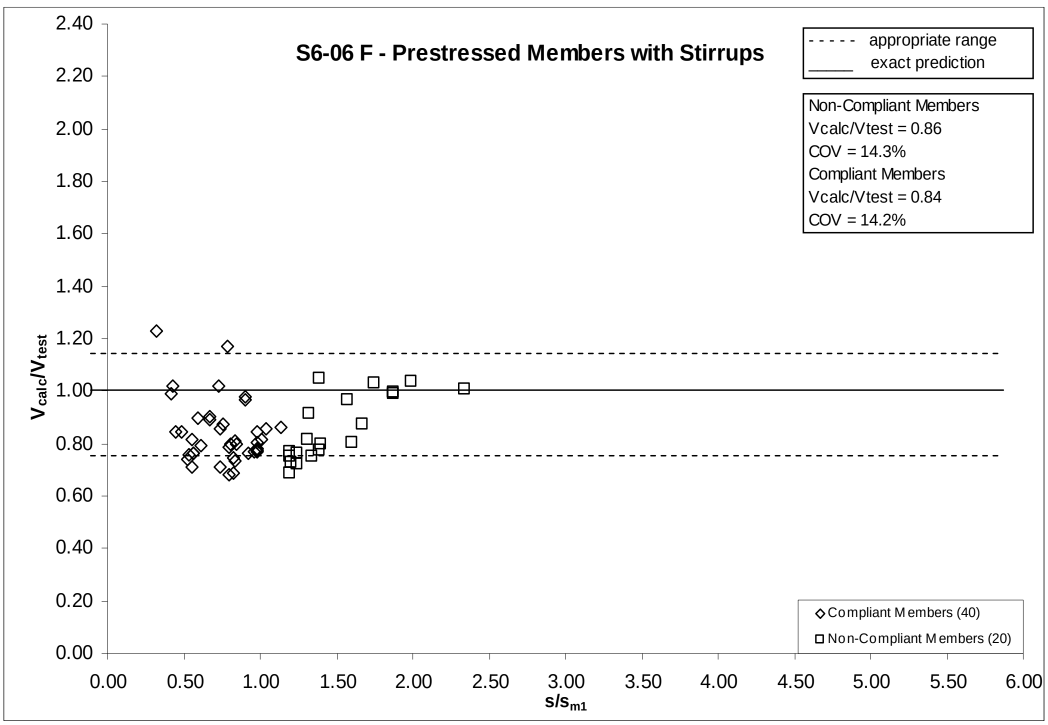



Figure 5.13 - $V_{calc}/V_{test}$ vs. $s/s_{m1}$ for 60 Prestressed Members with Stirrups Evaluated using S6-06 F

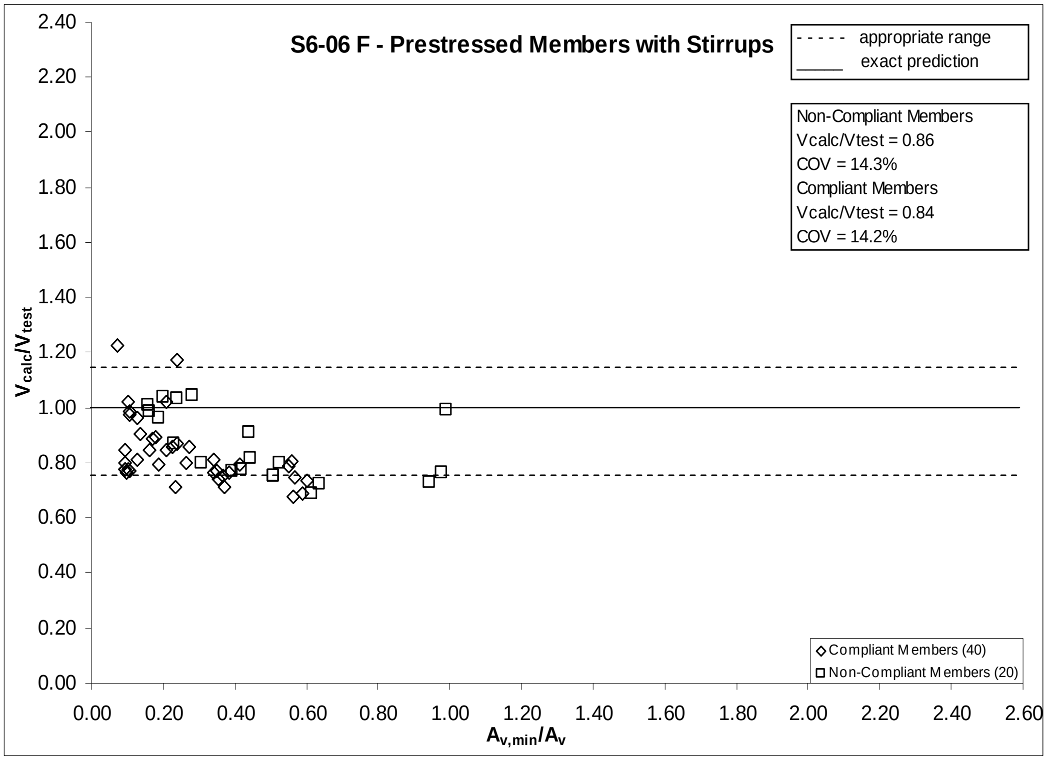



Figure 5.14 - $V_{calc}/V_{test}$ vs. $A_{v,min}/A_v$ for 60 Prestressed Members with Stirrups Evaluated using S6-06 F

Figure 5.13 shows an increase in $V_{calc}/V_{test}$ ratios for non-compliant members as the $s/s_{m1}$ ratio increases from 1.00 to 2.00. This is consistent with evaluation using the provisions in S6-06 and is discussed in Section 4.5.1. None of the non-compliant prestressed member predictions varied significantly from the rest of that data category (eg. no outliers). Figure 5.14 demonstrates a decrease in $V_{calc}/V_{test}$ ratios for the non-compliant prestressed members as their $A_{v,min}/A_v$ ratio increases. This is similar to results seen in Figure 4.9 from evaluation using the shear provisions in S6-06. The modifications in S6-06 F cause $V_{calc}/V_{test}$ ratios to have improvements greater than 10% for three non-compliant prestressed members, compared to evaluations of shear capacity using S6-06. Members determined have the most conservative predictions are small members with compression flanges, details which are known to perform better in shear than predicted using the sectional shear evaluation methods in this study (Moayer and Regan, 1971; Giaccio et al, 2002). This indicates that, although the concrete contribution to shear capacity modification in S6-06 F leads to improved agreement between predicted and tested shear capacities for members with compression flanges, the entirety of shear capacity exhibited by flanged members is not fully accounted for by S6-06 F.

Figure 5.13 does not provide any discernable trends in $V_{calc}/V_{test}$ ratios for compliant members as the stirrup spacing ratio $s/s_{m1}$ varies, indicating that stirrup spacing is appropriately accounted for by the proposed shear method S6-06 F. Figure 5.14 demonstrates a decrease in $V_{calc}/V_{test}$ ratios for compliant members as the stirrup area ratio $A_{v,min}/A_v$ increases. This behavior is consistent with evaluation using shear provisions in S6-06 and is discussed in Section 4.5.1. It should be noted from Figure 5.14 that the increase in $V_{calc}/V_{test}$ ratios as the stirrup ratio $A_{v,min}/A_v$ decreases does not lead to unsafe predictions of shear capacity, defined in this study as $V_{calc}/V_{test}$ ratios greater than 1.15. The two test specimens (A1-00-1.5R_N and A1-00-M_N) with $V_{calc}/V_{test}$ ratios greater than 1.15 deviate from the rest of the compliant prestressed category of members, and are discussed in Section 4.5.1. Fifteen compliant prestressed members evaluated using S6-06 F show improvements greater than 10% in their $V_{calc}/V_{test}$ ratios compared to evaluations of shear capacity using S6-06 Section 8. Nine of these members transitioned from the ‘conservative’ range to the ‘appropriate’ range.

### 5.3.2 S6-06 F - Evaluation of Members without Stirrups and Comparison to S6-06 Evaluation

This section discusses the results of the shear capacity evaluation for the thirty-three members without stirrups analyzed in this study using the modified shear method S6-06 F and provides comparison to the results of the similar analysis in Section 4.5.2 pertaining to evaluation using S6-06. These comparisons are used to justify the proposed modification to the concrete area contributing to shear capacity in S6-06 F. As discussed in Section 5.2.1, the diagonal crack spacing assumption used by S6-06 M and S6-06 F does not affect shear capacity predictions of members without stirrups. The focus of Section 5.3.3 is divided into four parts: i) tables providing the results from evaluation of members without stirrups using the modified shear method S6-06 F, ii) comparison of the mean statistical data between predictions using S6-06 F and S6-06 Section 8, iii) allocation of members into the prediction classifications given in Table 4.1 and average demerit points per member and iv) trends in the $V_{calc}/V_{test}$ ratios resulting from variations in section depth *d.*

## 5.3.2.i Statistical Data and Average Demerit Points per Member

Table 5.10 provides the $V_{calc}/V_{test}$, $d$ and $b/b_v$ values, calculated using the modified shear method S6-06 F, for the thirty-three members without stirrups identified for this study. For comparison this table includes $V_{calc}/V_{test}$ ratios from evaluation using the sectional shear method in S6-06 Section. The percent differences in $V_{calc}/V_{test}$ ratios between evaluation using S6-06 F and S6-06 Section 8 are also included in Table 5.10, while a summary of the significance of changes in percent difference are discussed in Section 5.3.2.i.

Table 5.10 - Results of Evaluation using S6-06 F – 33 Members without Stirrups

| Member | S6-06 F $V_{calc}/V_{test}$ | S6-06 $V_{calc}/V_{test}$ | S6-06 F – S6-06 Percent Difference (%) | $d$ | $b/b_v$ |
|---|---|---|---|---|---|
| Non-prestressed Members without Stirrups | | | | | |
| YB2000/0 | 1.01 | 1.01 | 0.0 | 1890 | 1.00 |
| N1-S | 1.12 | 1.12 | 0.0 | 655 | 1.00 |
| P41 | 0.82 | 0.75 | 9.3 | 279 | 4.00 |
| SD – 1 | 1.10 | 1.03 | 6.8 | 381 | 2.47 |
| SD – 2 | 1.20 | 1.12 | 7.1 | 381 | 2.47 |
| A & S – 8 | 1.04 | 1.04 | 0.0 | 500 | 1.00 |
| A & S – 9 | 1.01 | 1.01 | 0.0 | 500 | 1.00 |
| A & S – 10 | 1.00 | 1.00 | 0.0 | 500 | 1.00 |
| A & S – 11 | 1.09 | 1.09 | 0.0 | 500 | 1.00 |
| A & S – 12 | 1.07 | 1.07 | 0.0 | 500 | 1.00 |
| A & S – 16 | 1.03 | 1.03 | 0.0 | 750 | 1.00 |
| A & S – 17 | 1.14 | 1.14 | 0.0 | 750 | 1.00 |
| DB120 | 0.89 | 0.89 | 0.0 | 925 | 1.00 |
| DB130 | 1.00 | 1.00 | 0.0 | 925 | 1.00 |
| DB140 | 1.09 | 1.09 | 0.0 | 925 | 1.00 |
| DB230 | 0.86 | 0.86 | 0.0 | 895 | 1.00 |
| DB0.530 | 0.89 | 0.89 | 0.0 | 925 | 1.00 |
| B100 | 0.86 | 0.86 | 0.0 | 925 | 1.00 |
| B100L | 0.89 | 0.89 | 0.0 | 925 | 1.00 |
| BN100 | 0.92 | 0.92 | 0.0 | 925 | 1.00 |
| AW1 | 0.79 | 0.79 | 0.0 | 538 | 1.00 |
| AW4 | 0.80 | 0.80 | 0.0 | 506 | 1.00 |
| AW8 | 0.73 | 0.73 | 0.0 | 507 | 1.00 |
| Prestressed Members without Stirrups | | | | | |
| CI8 | 0.81 | 0.81 | 0.0 | 254 | 6.98 |
| CW8 | 0.99 | 0.63 | 57.1 | 363 | 3.98 |
| P12 | 0.85 | 0.80 | 6.2 | 282 | 4.00 |
| P16 | 0.69 | 0.66 | 4.5 | 272 | 4.00 |
| P17 | 0.69 | 0.66 | 4.5 | 269 | 4.00 |
| P10 | 0.62 | 0.59 | 5.1 | 269 | 4.00 |
| P11 | 0.64 | 0.60 | 6.7 | 282 | 4.00 |
| P15 | 0.63 | 0.60 | 5.0 | 272 | 4.00 |
| P47 | 0.65 | 0.61 | 6.6 | 274 | 4.00 |
| P48 | 0.70 | 0.67 | 4.5 | 269 | 4.00 |

A summary of the average $V_{calc}/V_{test}$ ratios, COV and the average demerit points per member for the thirty-three members without stirrups is provided in Table 5.11.

Table 5.11 - Summary of Results Comparing S6-06 F and S6-06 Sectional Shear Provisions

| Test Group (number) | Mean $V_{calc}/V_{test}$ Ratio S6-06 F | $V_{calc}/V_{test}$ C.O.V (%) S6-06 F | Average Demerit Points / Member S6-06 F | Mean $V_{calc}/V_{test}$ Ratio S6-06 | $V_{calc}/V_{test}$ C.O.V (%) S6-06 | Average Demerit Points / Member S6-06 |
|---|---|---|---|---|---|---|
| Non-Prestressed Members Without Stirrups (23) | 0.97 | 13.2 | 3.37 | 0.96 | 13.0 | 2.96 |
| Prestressed Members Without Stirrups (10) | 0.73 | 16.4 | 9.12 | 0.66 | 11.9 | 11.84 |

### 5.3.2.ii S6-06 F – Mean Statistical Results - Members without Stirrups

Evaluation of the twenty-three non-prestressed members without stirrups, using the modified shear method S6-06 F, shows little change from shear capacity predictions using the provisions in S6-06, a result of these members typically having rectangular cross-sections. Using S6-06 F, evaluation of the ten prestressed members without stirrups shows a more significant variation in $V_{calc}/V_{test}$ ratios and COV. Evaluation using S6-06 F improved the average $V_{calc}/V_{test}$ ratio of the non-prestressed members by 10.6% (0.73 compared to 0.66) but increased the COV by 37.8% (16.4% compared to 11.9%) compared to evaluation using S6-06 shear provisions.

### 5.3.2.iii S6-06 F – Prediction Classifications and Average Demerit Points per Member – Members without Stirrups

Table 5.12 provides the quantity and percentage of predictions falling in the various statistical ranges provided in Table 4.1, as well as the average demerit points per member corresponding to predictions.

Table 5.12 – S6-06 F – Members without Stirrups - $V_{calc}/V_{test}$ Ranges and Demerit Points

| **S6-06 F - Members without Stirrups** | **Full Data Set** | **Non-Prestressed** | **Prestressed** | **Demerits - Full Data Set** | **Non-Prestressed** | **Prestressed** |
|---|---|---|---|---|---|---|
| Total | 33 | 23 | 10 | | | |
| **Very Conservative**<br>Less than $V_{calc}/V_{test}$ = 0.5<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Conservative**<br>$V_{calc}/V_{test}$ Range - 0.50 - 0.75<br>Percent of Total | 8<br>24.2% | 1<br>4.3% | 7<br>70.0% | | | |
| **Appropriate**<br>$V_{calc}/V_{test}$ Range - 0.75 - 1.15<br>Percent of Total | 24<br>72.7% | 21<br>91.3% | 3<br>30.0% | | | |
| **Low Safety**<br>$V_{calc}/V_{test}$ Range - 1.15 - 1.50<br>Percent of Total | 1<br>3.0% | 1<br>4.3% | 0<br>0.0% | | | |
| **Dangerous**<br>$V_{calc}/V_{test}$ Range - 1.50 - 2.00<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| **Very Dangerous**<br>Greater than $V_{calc}/V_{test}$ = 2.0<br>Percent of Total | 0<br>0.0% | 0<br>0.0% | 0<br>0.0% | | | |
| Sum Total | 100.0% | 100.0% | 100.0% | 169 | 78 | 91 |
| Average Demerits/Members | | | | 5.11 | 3.37 | 9.12 |

As discussed in Section 4.5.2, evaluation of shear capacity using the provisions in S6-06 Section 8 resulted in twenty-one non-prestressed members without stirrups having $V_{calc}/V_{test}$ ratios in the 'appropriate' range and two predictions in the 'conservative' range (see Table 4.10). As shown in Table 5.12, evaluations of shear capacity for non-prestressed members using S6-06 F also result in twenty-one having $V_{calc}/V_{test}$ ratios in the appropriate range, but present as well with one member having a 'low safety' prediction. From the prestressed category of members without stirrups S6-06 F calculated with one extra prediction of shear capacity in the 'appropriate' range, compared to evaluation using S6-06.

The non-prestressed members without stirrups evaluated using S6-06 F are allotted an average of 25.3% more demerit points per member than from evaluation using S6-06 (3.37 compared to 2.69), while prestressed members evaluated using S6-06 F are allotted an average of 23.0% fewer demerit points per member than are members evaluated using S6-06 (9.12 compared to 11.84). In both cases the deviations in demerit points between shear predictions calculated using S6-06 F and S6-06 are a result of S6-06 F calculating larger $V_{calc}/V_{test}$ ratios (see Table 5.10).

### 5.3.2.iv S6-06 F – Relationship Between $V_{calc}/V_{test}$ Ratios and $d$

Figure 5.15 shows the relationship $V_{calc}/V_{test}$ vs. $d$ for the thirty-three members without stirrups identified for this study. The modified shear method S6-06 F assumes that members without stirrups present with diagonal cracks spaced equal to the shear depth $d_v$ of the member, as recommended by Bentz and Collins (2006). This crack spacing assumption is identical to that used by the shear method in S6-06 Section 8 as discussed in Section 3.2. The solid line represents the condition in which the predicted shear capacity is equal to the tested shear capacity, while the upper and lower dashed lines define the boundaries that are considered 'appropriate' predictions in this study.

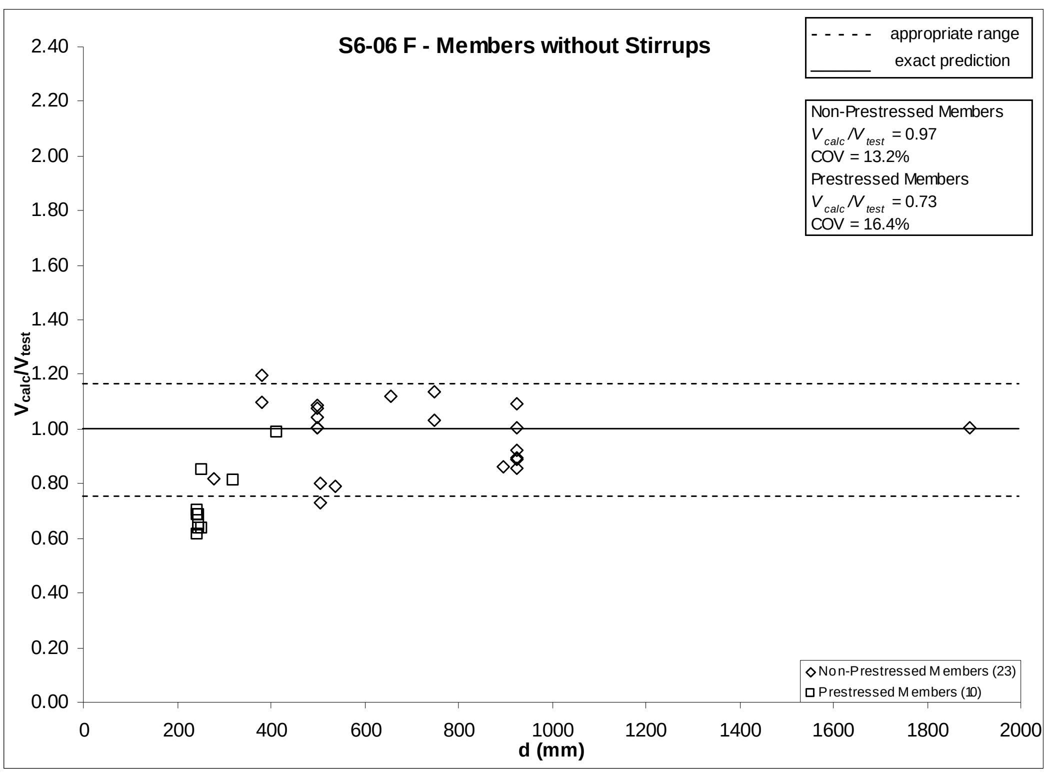



Figure 5.15 - $V_{calc}/V_{test}$ *vs.* $d_v$ for 33 Members without Stirrups using S6-06 F

The absence of any specific trend of $V_{calc}/V_{test}$ ratios with respect to varying depth shown in Figure 5.15 indicates that sectional height is appropriately accounted for by the modified shear method S6-06 F. The highest $V_{calc}/V_{test}$ ratio calculated for non-prestressed members is 1.20, which is at the low end of the range deemed 'low safety' in this study (see Table 4.1). This prediction is for an inverted channel section (SD – 1) without transverse reinforcement, which is not typical of members in service. No trends of $V_{calc}/V_{test}$ ratios as the depth varies are discernable in Figure 5.15 for the ten prestressed members without stirrups evaluated in this study. It should be noted that these ten members have small depths, which limits the range of useful information for prestressed members derived from Figure 5.15.

### 5.3.3 S6-06 F - Parametric Sensitivity Analysis of Shear Predictions

Section 5.3.4 assesses the influence that variations of the parameters discussed in Section 2.5 have on the agreement between predicted and tested shear capacities calculated using the modified shear method S6-06 F. As described in Section 2.5, variations in concrete strength $f'_c$, shear span to depth ratio $a/d$, longitudinal reinforcement ratio $\rho$, and member shape (flanged members vs. rectangular members) are known to affect the shear capacity of concrete members. These parameters are studied against the 163 members evaluated in this study to assess whether they are appropriately accounted for by the modified shear method S6-06 F.

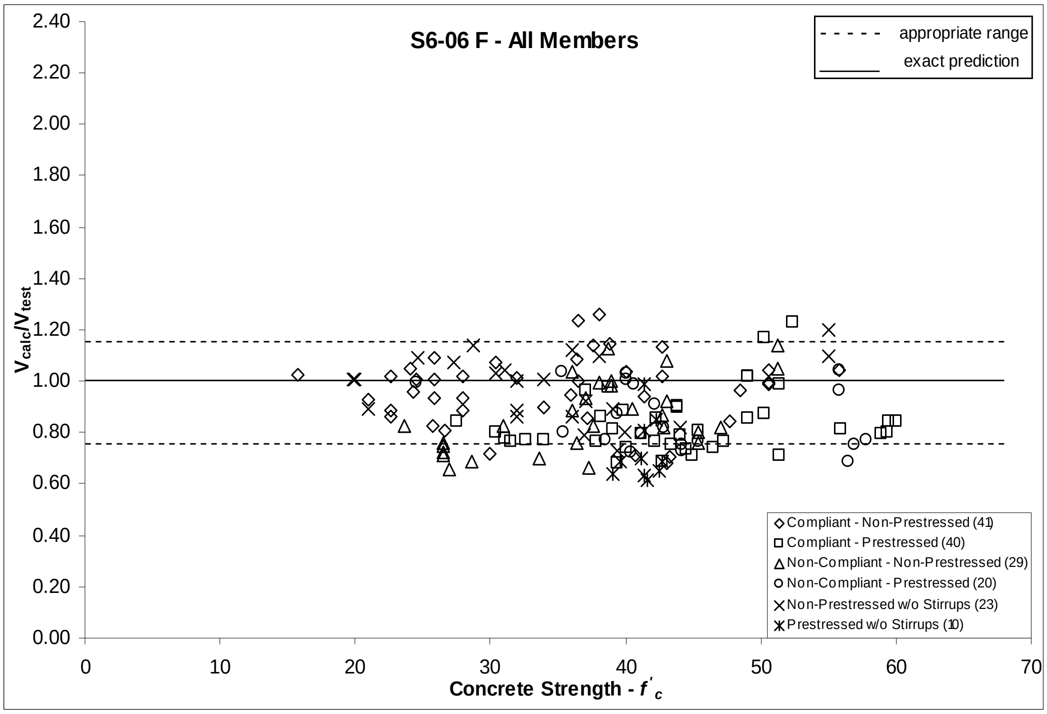



Figure 5.16 - $V_{calc}/V_{test}$ *vs.* $f'_c$ for 163 Members Evaluated using S6-06 F

Figure 5.16 shows the relationship between the predicted-to-tested shear capacity ratios and the specified concrete strength for the 163 members evaluated in this study. This figure does not show any specific trends in $V_{calc}/V_{test}$ ratios as the concrete strength varies for any of the data categories, indicating that the modified shear method in S6-06 F correctly accounts for concrete strength. This was consistent with results using the shear provisions in S6-06 (see Figure 4.11).

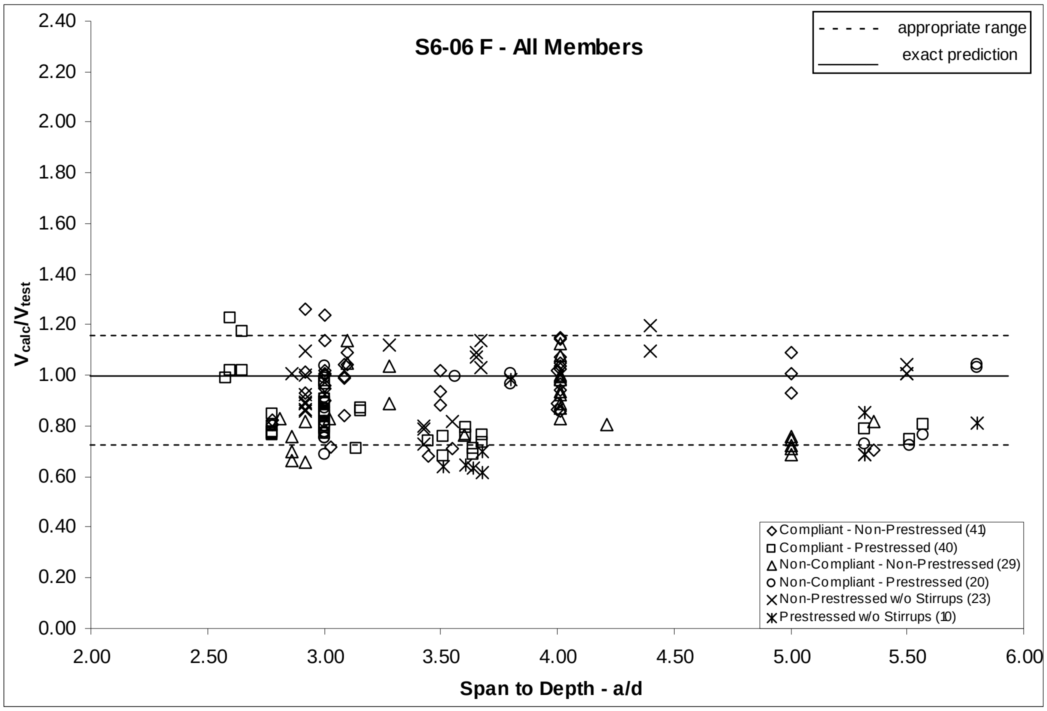



Figure 5.17 - $V_{calc}/V_{test}$ *vs. a/d* for 163 Members Evaluated using S6-06 F

Figure 5.17 provides the relationship between predicted and tested shear capacities and the shear span-to-depth ratio *a/d* for the 163 members identified for evaluation in this study. As inferred from Figure 4.12 for the provisions in S6-06 Section 8, the horizontal distribution of data points in Figure 5.17 indicates that the modified shear method S6-06 F appropriately accounts for the shear span to depth ratio.

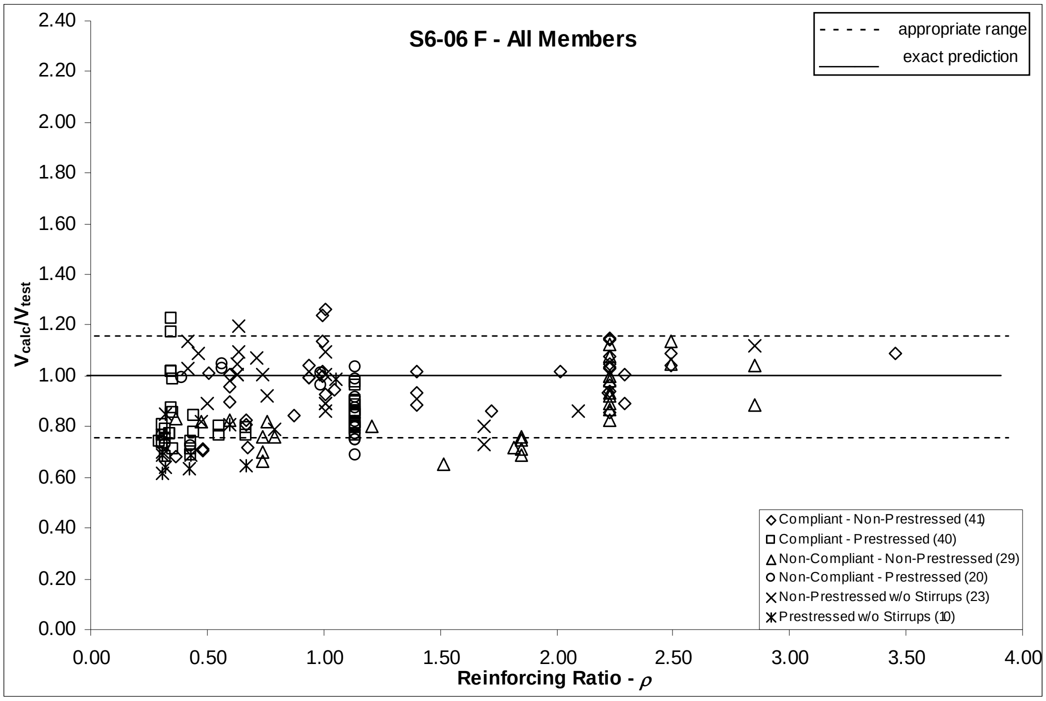



Figure 5.18 - $V_{calc}/V_{test}$ *vs.* $\rho$ for 163 Members Evaluated using S6-06 F

Figure 5.18 provides the relationship between $V_{calc}/V_{test}$ ratios and the longitudinal reinforcement ratio for the 163 members evaluated in this study. This figure does not exhibit any skews in $V_{calc}/V_{test}$ ratios with respect to the average $V_{calc}/V_{test}$ ratio for any of the data categories as the longitudinal reinforcement percentage changes. Similar to evaluation using the shear provisions in S6-06 (see Figure 4.13), this suggests that the modified shear method S6-06 F appropriately accounts for the quantity of longitudinal steel.

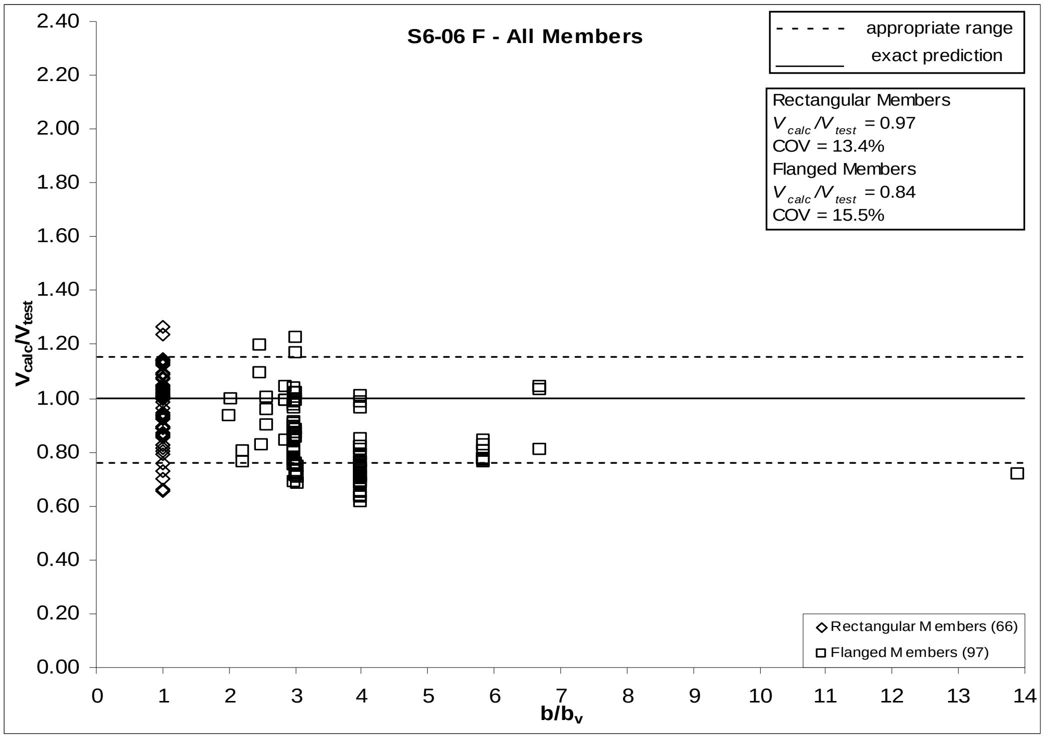



Figure 5.19 - $V_{calc}/V_{test}$ *vs.* $b/b_v$ for 163 Members Evaluated using S6-06 F

Figure 5.19 provides the relationship between predicted-to-tested shear capacities and the member flange width to web width ratio $b/b_v$ for the 163 members identified for evaluation in this study. Similar to evaluation using S6-06 (see Figure 4.14) this figure indicates that the ratio $V_{calc}/V_{test}$ is smaller for flanged members than for rectangular members. The difference between $V_{calc}/V_{test}$ ratios for flanged members and rectangular members indicates that the modified concrete contribution area used by S6-06 F does not account for the entirety of the reserve shear capacity for flanged members. Comparing predictions of the ninety-seven flanged members, S6-06 F shows a 7.7% improvement (0.84 compared to 0.78) in the average $V_{calc}/V_{test}$ ratio, and a 9.9% decrease (15.5% compared to 17.2%) in the COV compared to predictions using S6-06 sectional shear provisions. Thus, although S6-06 F does not account for all the reserve shear capacity noted for flanged members, it does improve the agreement between predicted and tested shear capacities relative to evaluation using the sectional shear method in S6-06. No well defined variations in $V_{calc}/V_{test}$ ratios are identified for flanged members as the $b/b_v$ ratio varies. Figure 5.19 indicates that one member (T1) had a $b/b_v$ ratio of 13.9. This value represents the effective flange to web width ratio, which has been reduced for calculation purposes from the actual $b/b_v$ value of 15.0 in accordance with S6-06 Clause 5.8.2.1.

### 5.3.4 S6-06 F – Summary

S6-06 F uses the diagonal crack spacing assumption employed by the modified shear method S6-06 M, and includes a further modification to the concrete area considered for members with compression flanges (see Figure 5.11). Thus S6-06 F has the advantages inherent to S6-06 M as discussed in Section 5.2, and accounts for a portion of the increased shear capacity exhibited by members with compression flanges. Although S6-06 F accounts for increased shear capacity of flanged members relative to predictions using S6-06 or S6-06 M, none of the calculated shear capacities for the 163 members evaluated in this study using S6-06 F fall in the 'dangerous' or 'very dangerous' ranges (see Table 4.1). S6-06 F typically shows a modest improvement in average $V_{calc}/V_{test}$ ratios, COV and average demerit points per member relative to predictions using the sectional shear method in S6-06 for all member categories evaluated in this study (see Tables 5.7 and 5.11). Evaluation using S6-06 F consistently results with fewer than 7.50 demerit points being allotted to each member, which, as discussed in Section 4.2, indicates that this modified shear method will typically determine 'appropriate' predictions of shear capacity. Thus S6-06 F is appropriate for evaluating the shear capacity of concrete members with non-compliant stirrup spacing and area details as well as compliant members and members without stirrups.

## 5.4 Comparison of Modified Shear Methods to S6-06 Sectional Shear Model – Classification Distributions

Table 5.13 provides the distribution of the twenty-nine non-prestressed members with non-compliant stirrup spacing and area details into the categories provided in Table 4.1, while Table 5.14 provides the same distribution of the forty-one compliant non-prestressed members. Table 5.16 provides the distribution of the twenty prestressed members with non-compliant stirrup spacing details into the categories provided in Table 4.1, while Table 5.16 provides the same distribution of the forty compliant prestressed members. Table 5.17 provides the distributions (Table 4.1) for the thirty-three members without stirrups evaluated by S6-06 and the modified shear method S6-06 F.

Table 5.13 – Classification Distribution (%) – 29 Non-Compliant Non-Prestressed Members

| $V_{calc}/V_{test}$ | <0.50 | 0.5~0.75 | 0.75~1.15 | 1.15~1.50 | 1.50~2.00 | > 2.00 | |
|---|---|---|---|---|---|---|---|
| Classification | Extremely Conservative | Conservative | Appropriate | Low Safety | Dangerous | Extremely Dangerous | Demerit Points per Members |
| S6-06 | 3.4 | 41.4 | 55.2 | 0.0 | 0.0 | 0.0 | 6.84 |
| S6-06 M | 0.0 | 31.0 | 69.0 | 0.0 | 0.0 | 0.0 | 5.64 |
| S6-06 F | 0.0 | 24.1 | 75.9 | 0.0 | 0.0 | 0.0 | 4.79 |

Table 5.14 – Classification Distribution (%) – 41 Compliant Non-Prestressed Members

| $V_{calc}/V_{test}$ | <0.50 | 0.5~0.75 | 0.75~1.15 | 1.15~1.50 | 1.50~2.00 | > 2.00 | |
|---|---|---|---|---|---|---|---|
| Classification | Extremely Conservative | Conservative | Appropriate | Low Safety | Dangerous | Extremely Dangerous | Demerit Points per Members |
| S6-06 | 0.0 | 9.8 | 85.4 | 4.9 | 0.0 | 0.0 | 3.91 |
| S6-06 M | 0.0 | 9.8 | 85.4 | 4.9 | 0.0 | 0.0 | 3.61 |
| S6-06 F | 0.0 | 9.8 | 85.4 | 4.9 | 0.0 | 0.0 | 3.36 |

Table 5.15 – Classification Distribution (%) – 20 Non-Compliant Prestressed Members

| $V_{calc}/V_{test}$ | <0.50 | 0.5~0.75 | 0.75~1.15 | 1.15~1.50 | 1.50~2.00 | > 2.00 | |
|---|---|---|---|---|---|---|---|
| Classification | Extremely Conservative | Conservative | Appropriate | Low Safety | Dangerous | Extremely Dangerous | Demerit Points per Members |
| S6-06 | 0.0 | 45.0 | 55.0 | 0.0 | 0.0 | 0.0 | 6.02 |
| S6-06 M | 0.0 | 40.0 | 60.0 | 0.0 | 0.0 | 0.0 | 5.36 |
| S6-06 F | 0.0 | 35.0 | 65.0 | 0.0 | 0.0 | 0.0 | 3.89 |

Table 5.16 – Classification Distribution (%) – 40 Compliant Prestressed Members

| $V_{calc}/V_{test}$ | <0.50 | 0.5~0.75 | 0.75~1.15 | 1.15~1.50 | 1.50~2.00 | > 2.00 | |
|---|---|---|---|---|---|---|---|
| Classification | Extremely Conservative | Conservative | Appropriate | Low Safety | Dangerous | Extremely Dangerous | Demerit Points per Members |
| S6-06 | 0.0 | 55.0 | 40.0 | 5.0 | 0.0 | 0.0 | 7.49 |
| S6-06 M | 0.0 | 25.0 | 70.0 | 5.0 | 0.0 | 0.0 | 5.87 |
| S6-06 F | 0.0 | 17.5 | 77.5 | 5.0 | 0.0 | 0.0 | 5.12 |

Table 5.17 – Classification Distribution (%) – 33 Members without Stirrups

| $V_{calc}/V_{test}$ | <0.50 | 0.5~0.75 | 0.75~1.15 | 1.15~1.50 | 1.50~2.00 | > 2.00 | |
|---|---|---|---|---|---|---|---|
| Classification | Extremely Conservative | Conservative | Appropriate | Low Safety | Dangerous | Extremely Dangerous | Demerit Points per Members |
| S6-06 | 0.0 | 27.3 | 72.7 | 0.0 | 0.0 | 0.0 | 5.65 |
| S6-06 F | 0.0 | 24.2 | 72.7 | 3.0 | 0.0 | 0.0 | 5.11 |

The modified shear methods S6-06 M and S6-06 F are consistently allotted fewer average demerit points per member than are the shear provisions in S6-06, for a given member category. This indicates that $V_{calc}/V_{test}$ ratios determined using S6-06 M and S6-06 F will on average be closer to unity than will $V_{calc}/V_{test}$ ratios calculated using the sectional shear method in S6-06 Section 8. Predictions using S6-06 M and S6-06 F also show no considerable increase in unsafe predictions compared to S6-06, further indicating that these modified shear methods are appropriate for predicting the shear capacity of members not complying with stirrup spacing and area requirements, as well as compliant members and members without shear reinforcement.

As discussed in Section 5.2.3, the diagonal crack spacing assumption used by the modified shear methods S6-06 M and S6-06 F is able to eliminate the non-convergent shear prediction issue inherent to evaluation using the sectional shear method in S6-06 Section 8. This reduces the ambiguity that can result during shear capacity evaluations.

Tables 5.13 through 5.16 indicate that the modified shear methods S6-06 M and S6-06 F consistently present with an equal or greater percentage of shear capacity predictions in the 'appropriate' range (see Table 4.1) than do predictions calculated using the sectional shear method in S6-06 Section 8. Consistent with discussion throughout this study, Tables 5.13 through 5.16 further show that members with stirrups not meeting minimum shear reinforcement requirements typically provide more conservative predictions of shear capacity than do compliant members. The conservative nature of shear predictions for non-compliant members is less pronounced for the modified shear methods S6-06 M and S6-06 F relative to evaluations using S6-06. As discussed in Section 5.2, this is due to the improvement in shear capacity predictions resulting from the modification to diagonal spacing assumed by the modified shear methods. Although the flange area assumption contributing to shear capacity used by S6-06 F is able to further improve shear predictions, this study recommends neglecting the increased shear capacity exhibited by members with compression flanges. This is consistent with recommendations by others (Tureyen et al, 2006; ASCE-ACI 426, 1973). Insufficient test results of concrete members with heights greater than 700 mm and with compression flanges exist in literature. Assessing the agreement between predicted and tested shear capacities of a larger test group of such members would be required in order to be able to recommend the modified shear method S6-06 F for use in practical situations.

## 5.5 Recommended Method for Sectional Shear Capacity Evaluation of Concrete Members

The following flowchart provides the method recommended by this study for calculating the sectional shear capacity of concrete members. Section 5.2 indicates that the modified shear method S6-06 M is appropriate for predicting the shear capacity of members both compliant and non-compliant with respect to S6-06 Section 14 stirrup spacing and area requirements. S6-06 M does not modify shear capacity predictions of members without stirrups relative to predictions using the sectional shear method in S6-06. The modified shear method S6-06 M is able to eliminate the issue of discontinuity in shear capacity predictions, which can occur when using the provisions in S6-06 Section 8, as discussed in Section 5.2.3. S6-06 M is also able to improve the agreement between predicted and tested shear capacities for all member categories with stirrups evaluated in this study (see Table 5.3), particularly for members which do not comply with minimum stirrup spacing and area requirements and which have overall member heights greater than 800 mm.

The layout of the following flowchart is similar to the S6-06 sectional shear method flowchart, given in Section 3.2. The notable variations in the recommended flowchart with respect to the flowchart in Section 3.2 are highlighted in bold and italics. In order for the flow chart at the end of this Section to be applicable for members subjected to a uniformly distributed load, Step 3 needs to be revised. Instead of using an assumed critical section at a distance $d_v$ away from the applied load, the shear capacity should be checked at numerous sections along the member length. The section which produces the highest $V_{calc}/V_{test}$ ratio should be selected as the governing section.

## Recommended Method for Sectional Shear Capacity Evaluation

Step 1: Determine geometry of section, longitudinal reinforcement layout, prestressed reinforcement layout, and stirrup details along member.

↓

Step 2: Determine material properties for concrete, longitudinal reinforcement, prestressing reinforcement, and stirrups.

↓

Step 3: Calculate Forces at a Section of Interest

Calculate dead load moments and shears at the section of interests. These forces include, but are not limited to, girder self-weight, deck weight, wearing surfaces, curbs, rails, etc.

Calculate live load moments and shears at the section of interest.

↓

Step 4: Determine if Section Complies with S6-06 Section 14 Stirrup Spacing and Area Requirements

Members complying with S6-06 Section 14 stirrup spacing and area requirements use different load factors than do members not complying with stirrup spacing and area requirements. In this study, all load and resistance factors were taken as 1.0.

***Stirrup Area Detail***

***Section 3.2 proposed an expression (γ) to interpolate between the stirrup area requirements in S6-06 Section 14.***

$$\gamma = 10 \cdot \frac{A_v \cdot f_v}{b_v \cdot s \cdot f_{cr}} - 0.5 \quad \textbf{\textit{where}} \quad 0 \le \gamma \le 1.0 \qquad \textbf{\textit{Eqn (1)}}$$

***γ values greater than 1.00 indicate that the member complies with stirrup area requirements.***

↓

Step 4: cont

Maximum Permissible Stirrup Spacing Detail

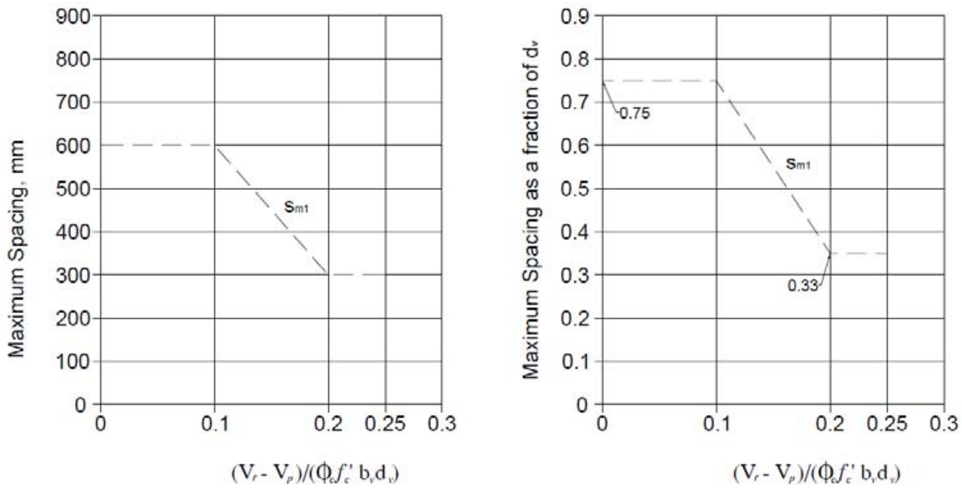



Figure 1 – S6-06 Section 14 Stirrup Spacing Limits

Step 5: Calculate the Crack Spacing $s_z$ and Effective Crack Spacing $s_{ze}$

***The assumed inclined crack spacing $s_z$ is taken as the lesser of the longitudinal spacing of the transverse reinforcement s and the shear depth $d_v$.***

Sufficient longitudinal reinforcement distributed over the depth of the member has been suggested to be adequate to control crack spacing (Collins et al., 1996). Collins et al. recommend a minimum area of $0.003 \cdot b_v \cdot s_x$ to control crack spacing. If this condition is met, the crack spacing term $s_z$ is taken as the vertical spacing of the longitudinal reinforcement.

The effective crack spacing term is calculated in the same manner as in S6-06, using Eqn. (2):

$$s_{ze} = s_z \frac{35}{15 + a_g} \leq 0.85 \cdot s_z \qquad \text{(mm)} \qquad \text{Eqn. (2)}$$

Step 6: Calculate the longitudinal strain at mid-depth $\varepsilon_x$

The longitudinal strain is determined using the shears, moments, axial loads and section information at the critical section, and is calculated using Eqn. (3):

$$\varepsilon_x = \frac{\frac{M_f}{d_v} + V_f - V_p + 0.5N_f - A_p f_{po}}{2 \cdot (A_s f_y + A_p E_p + A_{ct} E_c)} \text{ (mm/mm)} \qquad \text{Eqn. (3)}$$

Note: The value of $\varepsilon_x$ is limited to a minimum of -0.0002 and a maximum of 0.003. The term $A_{ct}E_c$ is only used when the longitudinal strain at mid-depth is calculated to be in compression ($\varepsilon_x$ calculated to be negative).

Step 7: Calculate the Shear Term $\beta$ and $\theta$

The $\beta$ factor is a term that indicates the ability of the concrete to transfer tensile forces across a cracked surface, and is calculated using Eqn. (4):

$$\beta = \frac{0.4}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} \qquad \text{Eqn. (4)}$$

The $\theta$ value is a term that gives an indication of the average angle of inclination of the compressive stresses. It is calculated using Eqn. (5):

$$\theta = (29^\circ + 7000 \cdot \varepsilon_x) \cdot \left(0.88 + \frac{s_{ze}}{2500}\right) \quad (^\circ) \qquad \text{Eqn. (5)}$$

Step 8: Calculate Factored Shear Resistance $V_r$

Concrete Shear Resistance

The concrete contribution to shear resistance $V_c$ is calculated using Eqn. (6)

$$V_c = 2.5 \cdot \phi_c \cdot \beta \cdot f_{cr} \cdot d_v \cdot b_v \qquad \text{(N)} \qquad \text{Eqn. (6)}$$

Stirrup Shear Resistance

The stirrup contribution is calculated using Eqn. (7)

$$V_s = \gamma \cdot \frac{\phi_s \cdot A_v \cdot f_v \cdot d_v}{s \cdot \tan\theta} \qquad \text{(N)} \qquad \text{Eqn. (7)}$$

Member Shear Resistance

The member's sectional shear capacity is calculated using Eqn. (8)

$$V_r = V_c + V_s + \phi_p \cdot V_p \qquad \text{(N)} \qquad \text{Eqn. (8)}$$

The predicted shear capacity is obtained when the calculated shear capacity $V_r$ in iteration $n$ equals the shear capacity from iteration $n$-$1$. Iterations are conducted by varying the externally applied moments and shears at the section of interest.

# Chapter 6
# Conclusions and Future Work

## 6.1 Introduction

In order to accomplish the objectives discussed in Section 1.2, forty-nine concrete members with stirrups not compliant with S6-06 Section 14 spacing and area requirements were evaluated in this study using the sectional shear methods in S6-06, AASHTO LRFD-05, ACI 318-08 and software Response 2000 (Bentz, 2000). Eighty-one compliant members and thirty-three members without stirrups were also evaluated for the purpose of providing a comparison to the results of the non-compliant members. The results of these evaluations, discussed in Chapter 4, were used to indicate whether the four sectional shear methods could calculate predicted shear capacities which were in appropriate agreement with tested shear capacities for non-prestressed and prestressed concrete girders not meeting minimum transverse reinforcement requirements. As discussed in Section 1.2, although ductility of the failure mode is an important issue, it was considered outside the scope of this study. Variations in parameters known to affect the shear resistance of concrete members (concrete strength $f_c'$, shear span to depth ratio $a/d$, longitudinal reinforcing ratio $\rho$, and member shape) were also studied to examine whether predictions of shear capacity, calculated using the four sectional methods, could account for these variations. Based on deficiencies noted from evaluation using the sectional shear method in S6-06, as discussed in Section 4.10, modifications to the S6-06 diagonal crack spacing assumption and to the concrete contribution to shear resistance term for members with compression flanges were then studied in Chapter 5 to determine whether they could improve the shear capacity predictions. Chapter 6 considers the principal conclusions from this study of sectional shear evaluation of concrete members with non-compliant stirrup spacing and area details, and recommends future research which could develop from this study.

## 6.2 Conclusions

The following points summarize the significant observations noted during this study.

1. **Predictions of shear capacity calculated using sectional methods based on the Modified Compression Field Theory (Vecchio and Collins, 1986) are in good agreement with test results for members with non-compliant stirrup spacing and area details.**

Average $V_{calc}/V_{test}$ ratios and corresponding COV for the forty-nine members not complying with S6-06 Section 14 minimum transverse reinforcement requirements and evaluated in this study using the sectional shear methods in S6-06, AASHTO LRFD-05 and Response 2000, are in good agreement with shear prediction results of compliant member from other studies, discussed in Chapter 3. These shear capacity predictions are also in good agreement with predictions of the compliant members evaluated in this study. Results from evaluation using S6-06, AASHTO LRFD-05 and software Response 2000 are discussed in Sections 4.5 through 4.7. The average demerit points per non-compliant member, calculated using the Demerit Point model proposed in Section 4.2, are similar to those of the compliant members evaluated in this study. This agreement in average demerit points per member further demonstrates that predictions of shear capacity for non-compliant members are in good agreement with those of the compliant members. None of the shear predictions for non-compliant members, calculated in this study using the three methods derived from the Modified Compression Field Theory (Vecchio and Collins, 1986), are considered unsafe (see Table 4.1). Parametric sensitivity analyses assessing the effect of variations in concrete strength $f_c^{'}$, shear span to depth ratio $a/d$ and longitudinal reinforcing ratio $\rho$ on predicted-to-tested shear capacities found these parameters to be well accounted for by the Modified Compression Field Theory-based shear methods used in this study.

2. **The sectional design model for shear (Bentz and Collins, 2006) in the Canadian Highway Bridge Design Code S6-06 Section 8 and the in lieu stirrup spacing and area provisions in Section 14 provided the best standard method for evaluating members with non-compliant stirrup spacing and area details.**

Based on the good agreement between predicted and tested shear capacities and on the ease of use, the sectional design method for shear (Bentz and Collins, 2006) in S6-06 Section 8 is considered to be the most appropriate of the standard methods assessed in this study for predicting shear capacity. Kim (2004) provided the same conclusion based on evaluation using the sectional shear method in A23.3-04. As discussed in Section 3.2, the sectional shear methods in S6-06 and A23.3-04 are very similar. The shear method in S6-06 Section 8 is considerably easier to use than the General Method for shear (Collins et al., 1996) in AASHTO LRFD-05 because no interpolation is required to obtain the shear terms $\beta$ and $\theta$. Response 2000 consistently calculates average $V_{calc}/V_{test}$ ratios closer to 1.00, a lower corresponding COV, and fewer average demerit points per member, but is not as convenient for predicting the shear capacity at various cross sections along a member's length.

3. **The shear capacity method for beams in ACI 318-08 is deficient for predicting the shear capacity of members with non-compliant stirrup spacing and area details.**

ACI 318-08 predictions of shear capacity for concrete members with stirrups not complying with spacing and area requirements do not agree well with tested capacities. Of the twenty-nine non-compliant non-prestressed members evaluated in this study, 13.8% of member predictions are in the 'low safety' range, 3.4% of predictions are in the 'dangerous' range and 3.4% of predictions are in the 'extremely dangerous' range (see Table 4.1). The COV corresponding to the $V_{calc}/V_{test}$ ratios calculated by ACI 318-08 are significantly greater than the other methods used in this study.

The quality of agreement between predicted and tested shear capacities becomes poorer as the actual area of transverse reinforcement decreases with respect to the minimum required stirrup area (see Figure 4.34). As discussed in Section 4.8.1, this indicates that the sectional shear method in ACI 318-08 does not appropriately account for variations in stirrup area.

All member categories evaluated using the sectional shear method in ACI 318-08 are allotted greater than 7.50 average demerit points per member. The Demerit Point model presented in this study proposes that shear evaluation methods determined to have greater than 7.50 average demerit points per member would not consistently be expected to calculate 'appropriate' predictions of shear capacity. This study recommends against using ACI 318-08 to evaluate members not meeting minimum stirrup requirements (see Section 4.8).

4. **Predictions of shear capacity determined using the sectional shear methods in S6-06, AASHTO LRFD-05 and software Response 2000 (Bentz, 2000) are in good agreement with tested shear capacities for members complying with S6-06 Section 14 stirrup spacing and area requirements and for members without stirrups.**

As discussed in Sections 4.5 through 4.7 the majority of $V_{calc}/V_{test}$ ratios for compliant members and members without stirrups, evaluated in this study using the three sectional shear methods derived from the Modified Compression Field Theory (Vecchio and Collins, 1986), are in the 'appropriate' and 'conservative' ranges (see Table 4.1). MCFT-based shear methods present with low average demerit points per member, which indicates that these methods typically determine $V_{calc}/V_{test}$ ratios close to unity. No predicted-to-tested shear capacity ratios are in the 'dangerous' or 'very dangerous' ranges. This study recommends using MCFT-based provisions for calculating the shear capacity of members compliant with respect to S6-06 Section 14 stirrup spacing and area requirements and members without stirrups.

5. **While the ACI 318-08 shear method appropriately predicts the shear capacity of members complying with minimum stirrup requirements, it is unable to appropriately predict the shear capacity of members without stirrups.**

ACI 318-08 typically provides 'appropriate' predictions (see Table 4.26) of shear capacity for members with stirrups complying with spacing and area requirements. However, ACI 318-08 predictions of shear capacity for members without stirrups are in poor agreement with tested shear capacities. Of the twenty-three non-prestressed members without stirrups evaluated in this study, 39.1% are determine to have $V_{calc}/V_{test}$ ratios in the 'low safety' range, 26.1% are determine to have $V_{calc}/V_{test}$ ratios in the 'dangerous' range, and 4.3% are determine to have 'extremely dangerous' predictions (see Table 4.1). As discussed in Section 4.8.2, shear capacity predictions of members without stirrups using the shear provisions in ACI 318-08 become more unsafe as the depth of the member increases. This study recommends against using the sectional shear method in ACI 318-08 to predict the shear capacity of members without stirrups.

6. **Adhering to the stirrup spacing requirement $s_{m2}$ in S6-06 Section 14 can cause predictions of shear capacity to compare poorly to their corresponding tested shear capacities.**

The non-compliant members considered in this study were evaluated using the shear method in S6-06 Section 8 and by adhering to the stirrup spacing requirements in S6-06 Clause 14.14.1.6.2, including the stirrup spacing limit $s_{m2}$ (see Section 4.5.1). Results from the analysis indicates that adhering to the $s_{m2}$ limit is inappropriate for evaluating the shear capacity of concrete members. A significant quantity of tested shear resistance is ignored when the $s_{m2}$ limit is followed, which results in higher average demerit points assigned to predictions. The scatter in predicted-to-tested shear capacity ratios is also considerably larger when the $s_{m2}$ limit is considered. It should be noted that disregarding the $s_{m2}$ stirrup spacing limit has no affect on load ($\alpha$), resistance ($\phi$) or reliability factors ($U$) determined using provisions in S6-06 Section 14.

7. **In this study a modified shear method is proposed which assumes that the spacing of diagonal cracks is equal to the longitudinal spacing of stirrups. This modified method, titled S6-06 M and presented in Section 5.2, improves the agreement between predicted and tested shear capacities relative to evaluation using the sectional shear method in S6-06 Section 8. As discussed in Section 5.2, the crack spacing assumption used by S6-06 M is similar to the crack spacing assumption used by the shear method CSA-M, proposed by Lubell (2006).**

Results from the evaluation of the forty-nine members not complying with minimum transverse reinforcement requirements, using S6-06 M, have average $V_{calc}/V_{test}$ ratios closer to unity and have lower corresponding COV than results from evaluations using the sectional shear provisions in S6-06. Evaluation of the eighty-one compliant members also shows improved $V_{calc}/V_{test}$ ratios (closer to 1.00) with lower corresponding COV, indicating that the assumption that diagonal crack spacing is equal to the longitudinal spacing of stirrups is appropriate for all members with shear reinforcement. Evaluation using S6-06 M decreases the average demerit point/member assigned to both compliant and non-compliant members relative to evaluation using S6-06 shear provisions, thereby demonstrating that the diagonal crack spacing assumption used by S6-06 M consistently results with $V_{calc}/V_{test}$ ratios closer to unity. Evaluation of shear capacity using the modified method S6-06 M also presents with a larger percentage of members in the range deemed 'appropriate' in this study (see Tables 4.1, 5.13 through 5.16), relative to evaluation using S6-06.

8. **In this study a modified shear method is proposed which incorporates the same diagonal crack spacing assumed by the modified method S6-06 M and includes a portion of the flange area in the calculated concrete contribution to shear capacity. This method, termed S6-06 F and discussed in Section 5.3, typically results with improved agreement between predicted and tested shear capacities relative to evaluation using S6-06 and S6-06 M. As discussed in Section 5.3, the concrete area assumed to contribute to shear capacity by S6-06 F is similar to the concrete area used by Zsutty (as cited in ASCE-ACI 426, 1973).**

In addition to the advantages of using the proposed diagonal crack spacing assumption discussed in Section 5.2, the increased concrete area term for members with compression flanges employed by S6-06 F results in $V_{calc}/V_{test}$ ratios closer to 1.00 and decreased COV and average demerit points per member compared to evaluation using either S6-06 or S6-06 M. These improvements are typical of both compliant and non-compliant members. Members with compression flanges identified for evaluation in this study still present with smaller average $V_{calc}/V_{test}$ ratios than did rectangular sections. Although predictions of shear capacity determined using S6-06 F are in good agreement with tested shear capacities this study recommends neglecting the increased shear capacity exhibited by members with compression flanges. This recommendation is based on the lack of documented test results for members with heights greater than 700 mm and with compression flanges loaded critically in shear. Ignoring the increased shear capacity exhibited by flanged members is consistent with recommendations in other studies (Tureyan et al, 2006; ASCE-ACI 426, 1973).

## 6.3 Recommendations

1. **Based on the evaluations of shear capacity performed in this study, it is this study's recommendation that the modified shear method S6-06 M be used to evaluate the shear capacity of compliant members, non-compliant members and members without stirrups.**

As discussed in Section 5.2, evaluation using the modified shear method S6-06 M typically results in 'appropriate' predictions of shear for both non-prestressed and prestressed members with stirrups. These predictions of shear capacity are also closer to unity and exhibit less scatter than predictions of shear capacity calculated using the sectional shear method in S6-06 Section 8. Predictions of shear capacity for members without stirrups calculated using S6-06 M are the same as predictions calculated using S6-06 Section 8. As discussed in Section 4.5.2, predictions of shear capacity for members without stirrup calculated using S6-06 Section 8 are in good agreement with their corresponding tested capacities.

The diagonal crack spacing assumption employed in the modified shear method S6-06 M is able to eliminate the issue of non-convergence of predicted shear capacity which can result from using the diagonal crack spacing assumption employed by the sectional shear method in S6-06. Eliminating this issue of non-convergence allows engineers to make unambiguous predictions of shear capacity, as demonstrated in Appendix C.

2. **This study recommends that the stirrup spacing limit $s_{m2}$ should be disregarded when evaluating the sectional shear capacity of concrete members.**

As discussed in Section 4.5.1, adhering to the $s_{m2}$ stirrup spacing limit can causes predicted shear capacities to compare very conservatively to a member's corresponding tested capacity, relative to evaluation ignoring the $s_{m2}$ stirrup spacing limit. Due to the sudden discontinuity in predicted shear capacity at the $s_{m2}$ limit, adhering to this stirrup spacing limit can introduce a source for encountering non-convergent shear capacity predictions. This cause of non-convergent shear capacity predictions is not eliminated by the diagonal crack spacing assumption used by S6-06 M; it can only be eliminated by ignoring the $s_{m2}$ stirrup spacing limit.

## 6.4 Future Work

Research of the following issues will further build on the conclusions of this study.

1. **Does diagonal crack spacing continue to correlate with longitudinal stirrup spacing as member depth increases?**

Diagonal crack spacing diagrams presented in Angelakos (1999) and Yoshida (2000) suggest that the spacing of inclined cracks corresponds well with longitudinal stirrup spacing. This observation is consistent with work by Dilger and Divakar (1987), although work by Dilger and Divakar is limited to members with heights of 300 mm and stirrups spaced from 130 mm to 150 mm. In order to further validate the diagonal crack spacing assumption used by the modified shear methods S6-06 M and S6-06 F, a test program consisting of members having greater member height and larger variability in longitudinal stirrup spacing would be useful. As the diagonal crack spacing assumption used by S6-06 M and S6-06 F is applied to both compliant and non-compliant members, the test program should examine longitudinal stirrup spacing ranging from approximately 150 mm up to one-and-a-half times the shear depth $d_v$ of the member. In order to constitute 'very valuable test' results (Leonhardt as cited in Collins, 2001) which will be representative of members encountered in service, test members should have a minimum height of 700 mm and a maximum longitudinal reinforcing ratio of 1.5%. The flexural capacity of the members evaluated in this test program should exceed their corresponding shear capacity by a reasonably low margin to ensure the members in this test program are representative of members encountered in service. When determining the design flexural capacity of the members, it is important to consider the typical COV noted for flexural failures in concrete members as well as the COV for shear failures. Both non-prestressed and prestressed members should be included in this test program. Properly scaled photographs taken perpendicular to the side of the concrete members, and showing the diagonal crack spacing, will make for a useful component in future studies as this will assist in assessing the influence of longitudinal stirrup spacing on diagonal crack spacing. Such photographs were typically unavailable for the members evaluated in this study.

2. **Additional test data of members with transverse reinforcement details not complying with S6-06 Section 14 stirrup spacing and area requirements will help validate the finding from this study.**

The results of this study indicate that sectional shear provisions derived from simplifications to the Modified Compression Field Theory (Vecchio and Collins, 1986) are able to appropriately predict the shear capacity of members with non-compliant stirrup spacing and area details. However test results and studies of non-prestressed and prestressed concrete members with stirrup spacing and area details non-compliant with S6-06 Section 14 requirements are rare. To further validate these results, future testing programs with a larger data set of non-compliant members should be studied, both to assess the agreement between predicted and tested shear capacities calculated using various shear methods, and to examine the influence that varying stirrup spacing and area details have on $V_{calc}/V_{test}$ ratios. To provide 'very valuable test' results members should have minimum heights of 700 mm, a maximum longitudinal reinforcement ratio of 1.50% and stirrups spaced longitudinally as far apart as 1.5 times the shear depth of the member. Both non-prestressed and prestressed members should be tested. The flexural capacity of the members evaluated in this test program should exceed their corresponding shear capacity by a reasonably low margin to ensure the members in this test program are representative of members encountered in service.

3. **Future test programs of members with non-compliant stirrup spacing and area details should include load-deflection behavior so that the ductility of these members can be studied.**

A useful behavior to study would be the load-deflection response of non-compliant members. This information was not provided in the literature describing the test results of most of the members evaluated in this study and, as a result, an investigation into the relative ductility of compliant to non-compliant members is not provided herein. The non-compliant members evaluated in this study which provided load-deflection figures suggest that members with deficient shear reinforcement show less ductility than do members with stirrups which comply with S6-06 Section 14 stirrup spacing and area requirements. This observation is consistent with discussion in DeGeer and Stephens (1993). Studies examining member ductility at different stirrup spacing and area ratios ($s/s_{m1}$ and $A_{v,min}/A_v$) could allow for more efficient detailing of stirrups in concrete members as the results could provide insight on minimum transverse reinforcement details required to ensure ductile failures of members loaded critically in shear.

# Chapter 7
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# Appendix A

## Dataset Section Properties

## Table Assumptions

The following assumptions were used to determine the values provided in Tables A1 and A2.

-Sectional geometry was taken as the member cross section at the critical shear location. It should be noted that the geometry of the specimens evaluated in this study did not vary along the member's length.

-the $b/b_v$ ratios given in Table A1 represent the effective flange geometries, and have been checked as per S6-06 Clause 5.8.2.1.

-As discussed in Section 3.2 the stirrup area $A_v$ is taken in this study as the cross sectional area per stirrup. To determine longitudinal stirrup spacing, this study calculates the ratio $A_v/s$ for a distance $d_v$ from the applied load in the direction of the nearest support. The longitudinal stirrup spacing $s$ is then determined by dividing the cross sectional area per stirrup $A_v$ by the ratio $A_v/s$.

-Concrete compressive strength f'c represents the tested strength of concrete cylinders. The compressive strengths of concrete cubes was converted to an equivalent cylinder strength using the conversation $f_{cylinder} = 0.85 \cdot f_{cube}$ (Kurian, 2005).

-Steel properties $f_y$, $f_v$, $f_{pu}$, and $f_{pe}$ are taken as reported in the published test results.

-The tested shear capacity $V_{test}$ represents the shear at failure at a distance $d_v$ away from the applied load and includes both the shear resulting from the externally applied load and the shear from self-weight.



Table A1 – Member Geometric Properties

| Specification | Tested by | $a_g$ (mm) | h (mm) | d (mm) | $b_v$ (mm) | $b_{top}$ (mm) | $b_{bot}$ (mm) | $t_{top}$ (mm) | $t_{bot}$ (mm) | $b/b_v$ | L (mm) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Non-Compliant Non-Prestressed Members | | | | | | | | | | | |
| YB 2000/9 | Yoshida | 10 | 2000 | 1890 | 300 | - | - | - | - | 1.00 | 10800 |
| YB 2000/6 | Yoshida | 10 | 2000 | 1890 | 300 | - | - | - | - | 1.00 | 10800 |
| YB 2000/4 | Yoshida | 10 | 2000 | 1890 | 300 | - | - | - | - | 1.00 | 10800 |
| 5084 | Kani | 20 | 305 | 271 | 152 | 459 | - | 100 | - | 3.02 | 3634 |
| 5053 | Kani | 20 | 305 | 271 | 152 | 460 | - | 100 | - | 3.02 | 3634 |
| 5052 | Kani | 20 | 305 | 275 | 152 | 461 | - | 100 | - | 3.03 | 3634 |
| 5051 | Kani | 20 | 305 | 273 | 152 | 459 | - | 100 | - | 3.03 | 3634 |
| 5063 | Kani | 20 | 305 | 272 | 152 | 461 | - | 100 | - | 3.03 | 3634 |
| N2-S | Yoon et al. | 20 | 750 | 655 | 375 | - | - | - | - | 1.00 | 4300 |
| N1-N | Yoon et al. | 20 | 750 | 655 | 375 | - | - | - | - | 1.00 | 4300 |
| P21 | Moayer and Regan | 20 | 320 | 279 | 150 | 600 | - | 80 | - | 4.00 | 3000 |
| Ss2-321-3 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3658 |
| Ss2-318-3 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3658 |

Table A1 continued

| Specification | Tested by | $a_g$ (mm) | h (mm) | d (mm) | $b_v$ (mm) | $b_{top}$ (mm) | $b_{bot}$ (mm) | $t_{top}$ (mm) | $t_{bot}$ (mm) | $b/b_v$ | L (mm) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Non-Compliant Non-Prestressed Members | | | | | | | | | | | |
| Ss2-313.5-3 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3658 |
| Ss2-321-2 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3658 |
| Ss2-318-2 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3658 |
| Ss2-321-1 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3658 |
| Ss2-318-1 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3658 |
| Ss2-218a-2 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3658 |
| Ss2-213.5a-2 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3658 |
| Ss2-213.5-1 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3658 |
| 7 | Johnson and Ramirez | 20 | 610 | 539 | 305 | - | - | - | - | 1.00 | 4255 |
| 8 | Johnson and Ramirez | 20 | 610 | 539 | 305 | - | - | - | - | 1.00 | 4255 |
| BM100 | Collins and Kuchma | 10 | 1000 | 925 | 300 | - | - | - | - | 1.00 | 5400 |
| SB 2012/6 | ShenCao | 10 | 2000 | 1845 | 300 | - | - | - | - | 1.00 | 10800 |
| SB 2012/6 | ShenCao | 10 | 2000 | 1925 | 300 | - | - | - | - | 1.00 | 10800 |
| 10T24 | Higgins et al. | 20 | 1219 | 1111 | 365 | 914 | - | 152 | - | 2.50 | 7315 |
| PE 1 | Stephens and Degeers | 20 | 610 | 528 | 207 | 456 | - | 102 | - | 2.20 | 10663 |
| PE 2 | Stephens and Degeers | 20 | 610 | 528 | 207 | 456 | - | 102 | - | 2.20 | 8261 |
| Non-Compliant Prestressed Members | | | | | | | | | | | |
| CH-6-240 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 2.98 | 3660 |
| CM-6-240 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 2.98 | 3660 |
| CL-6-240 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 2.98 | 3660 |
| PH-6-240 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 2.98 | 3660 |

Table A1 continued

| Specification | Tested by | $a_g$ (mm) | h (mm) | d (mm) | $b_v$ (mm) | $b_{top}$ (mm) | $b_{bot}$ (mm) | $t_{top}$ (mm) | $t_{bot}$ (mm) | $b/b_v$ | L (mm) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Non-Compliant Prestressed Members | | | | | | | | | | | |
| PM-6-240 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 2.98 | 3660 |
| PL-6-240 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 2.98 | 3660 |
| NM-10-240 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 2.98 | 3660 |
| NL-10-240 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 2.98 | 3660 |
| NM-8-240 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 2.98 | 3660 |
| NH-6-240 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 2.98 | 3660 |
| NM-6-240 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 2.98 | 3660 |
| NL-6-240 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 2.98 | 3660 |
| CW12 | Elzanaty et al. | 13 | 457 | 363 | 51 | 203 | 203 | 102 | 102 | 3.98 | 3810 |
| CW11 | Elzanaty et al. | 13 | 457 | 363 | 51 | 203 | 203 | 102 | 102 | 3.98 | 3810 |
| CI12 | Elzanaty et al. | 13 | 356 | 254 | 76 | 508 | 152 | 51 | 127 | 6.68 | 3810 |
| CI11 | Elzanaty et al. | 13 | 356 | 254 | 76 | 508 | 152 | 51 | 127 | 6.68 | 3810 |
| P9 | Moayer and Regan | 20 | 320 | 272 | 150 | 600 | - | 80 | - | 4.00 | 3000 |
| P14 | Moayer and Regan | 20 | 320 | 282 | 150 | 600 | - | 80 | - | 4.00 | 3000 |
| P19 | Moayer and Regan | 20 | 320 | 269 | 150 | 600 | - | 80 | - | 4.00 | 3000 |
| BW.14.34 | MacGregor | 20 | 305 | 257 | 74 | 150 | 150 | 74 | 74 | 2.03 | 2743 |
| Compliant Non-Prestressed Members | | | | | | | | | | | |
| V18-2 | Tompos and Frosch | 20 | 486 | 425 | 229 | - | - | - | - | 1.00 | 2550 |
| V36-3 | Tompos and Frosch | 20 | 914 | 851 | 457 | - | - | - | - | 1.00 | 5100 |
| V36-2 | Tompos and Frosch | 20 | 914 | 851 | 457 | - | - | - | - | 1.00 | 5100 |
| V1 | Frosch | 20 | 914 | 851 | 457 | - | - | - | - | 1.00 | 5100 |
| V2 | Frosch | 20 | 914 | 851 | 457 | - | - | - | - | 1.00 | 5100 |
| A1 | Vecchio and Shim | 20 | 552 | 457 | 305 | - | - | - | - | 1.00 | 3660 |

Table A1 continued

| Specification | Tested by | $a_g$ (mm) | h (mm) | d (mm) | $b_v$ (mm) | $b_{top}$ (mm) | $b_{bot}$ (mm) | $t_{top}$ (mm) | $t_{bot}$ (mm) | $b/b_v$ | L (mm) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Compliant Non-Prestressed Members | | | | | | | | | | | |
| A2 | Vecchio and Shim | 20 | 552 | 457 | 305 | - | - | - | - | 1.00 | 4570 |
| B1 | Vecchio and Shim | 20 | 552 | 457 | 229 | - | - | - | - | 1.00 | 3660 |
| B2 | Vecchio and Shim | 20 | 552 | 457 | 229 | - | - | - | - | 1.00 | 4570 |
| C1 | Vecchio and Shim | 20 | 552 | 457 | 152 | - | - | - | - | 1.00 | 3660 |
| C2 | Vecchio and Shim | 20 | 552 | 457 | 152 | - | - | - | - | 1.00 | 4570 |
| P5 | Moayer and Regan | 20 | 320 | 285 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| P20 | Moayer and Regan | 20 | 320 | 279 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| P22 | Moayer and Regan | 20 | 320 | 279 | 150 | 600 | - | 80 | - | 4.00 | 3000 |
| DBO530M | Angelakos | 10 | 1000 | 925 | 300 | - | - | - | - | 1.00 | 5410 |
| DB120M | Angelakos | 10 | 1000 | 925 | 300 | - | - | - | - | 1.00 | 5410 |
| DM140M | Angelakos | 10 | 1000 | 925 | 300 | - | - | - | - | 1.00 | 5410 |
| Ss2-29g-2 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3578 |
| Ss2-29e-2 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3578 |
| Ss2-29d-2 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3578 |
| Ss2-29c-2 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3578 |
| Ss2-29b-2 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3578 |
| Ss2-29a-2 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3578 |
| Ss2-29b-1 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3578 |
| Ss2-29a-1 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3578 |
| Ss2-26-1 | Krefeld and Thurston | 20 | 508 | 456 | 254 | - | - | - | - | 1.00 | 3578 |
| 1 | Johnston and Ramirez | 20 | 610 | 539 | 305 | - | - | - | - | 1.00 | 4255 |

Table A1 continued

| Specification | Tested by | $a_g$ (mm) | h (mm) | d (mm) | $b_v$ (mm) | $b_{top}$ (mm) | $b_{bot}$ (mm) | $t_{top}$ (mm) | $t_{bot}$ (mm) | $b/b_v$ | L (mm) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Compliant Non-Prestressed Members | | | | | | | | | | | |
| 5 | Johnston and Ramirez | 20 | 610 | 539 | 305 | - | - | - | - | 1.00 | 4255 |
| 5A-0 | Lyngberg | 20 | 600 | 540 | 120 | 700 | 380 | 95 | 125 | 6.00 | 5000 |
| 5B-0 | Lyngberg | 20 | 600 | 540 | 120 | 700 | 380 | 95 | 125 | 6.00 | 5000 |
| Test 1.1 | Mao et al. | 20 | 508 | 434 | 318 | 911 | - | 100 | - | 2.86 | 9838 |
| Test 2.1 | Mao et al. | 20 | 508 | 434 | 159 | 455 | - | 100 | - | 2.86 | 9838 |
| Test 2.2 | Mao et al. | 20 | 508 | 434 | 159 | 455 | - | 100 | - | 2.86 | 9838 |
| Test 2.3 | Mao et al. | 20 | 508 | 434 | 318 | 911 | - | 100 | - | 2.86 | 9838 |
| T1 | Leonhardt and Walther | 20 | 900 | 825 | 100 | 1500 | 500 | 150 | 100 | 13.90 | 6000 |
| ET1 | Leonhardt and Walther | 30 | 350 | 300 | 300 | 300 | - | - | - | 1.00 | 3000 |
| ET2 | Leonhardt and Walther | 30 | 350 | 300 | 150 | 300 | - | 100 | - | 2.00 | 3000 |
| ET3 | Leonhardt and Walther | 30 | 350 | 300 | 100 | 300 | - | 100 | - | 3.00 | 3000 |
| 2T10 | Higgins et al. | 20 | 1219 | 1103 | 356 | 914 | - | 152 | - | 3.00 | 7315 |
| 2T12 | Higgins et al. | 20 | 1219 | 1103 | 356 | 914 | - | 152 | - | 3.00 | 7315 |
| 1T18 | Higgins et al. | 20 | 1219 | 1103 | 356 | 914 | - | 152 | - | 3.00 | 7315 |
| Compliant Prestressed Members | | | | | | | | | | | |
| PL-6-160 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |
| NL-6-80 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |
| NH-6-80 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |
| NM-6-80 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |
| NL-6-160 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |
| NM-6-160 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |
| CH-6-80 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |
| CM-6-80 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |
| CL-6-80 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |

Table A1 continued

| Specification | Tested by | $a_g$ (mm) | h (mm) | d (mm) | $b_v$ (mm) | $b_{top}$ (mm) | $b_{bot}$ (mm) | $t_{top}$ (mm) | $t_{bot}$ (mm) | $b/b_v$ | L (mm) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Compliant Prestressed Members | | | | | | | | | | | |
| CH-6-160 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |
| CL-6-160 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |
| PM-6-80 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |
| PM-6-160 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |
| PL-6-80 | Bennett and Debaiky | 20 | 330 | 298 | 51 | 152 | 152 | 57 | 57 | 3.00 | 3660 |
| A1-00-1.5R-N | Shahaway and Batchelor | 20 | 1118 | 997 | 152 | 457 | 305 | 191 | 229 | 3.00 | 12192 |
| B0-00-R-S | Shahaway and Batchelor | 20 | 1118 | 997 | 152 | 457 | 305 | 191 | 229 | 3.00 | 9601 |
| B0-00-R-N | Shahaway and Batchelor | 20 | 1118 | 997 | 152 | 457 | 305 | 191 | 229 | 3.00 | 12192 |
| A1-00-M-N | Shahaway and Batchelor | 20 | 1118 | 997 | 152 | 457 | 305 | 191 | 229 | 3.00 | 12192 |
| A1-00-R-N | Shahaway and Batchelor | 20 | 1118 | 997 | 152 | 457 | 305 | 191 | 229 | 3.00 | 12192 |
| A1-00-M-S | Shahaway and Batchelor | 20 | 1118 | 997 | 152 | 457 | 305 | 191 | 229 | 3.00 | 9601 |
| A1-00-0.5R-N | Shahaway and Batchelor | 20 | 1118 | 997 | 152 | 457 | 305 | 191 | 229 | 3.00 | 12192 |
| A1-00-0.5R-S | Shahaway and Batchelor | 20 | 1118 | 997 | 152 | 457 | 305 | 191 | 229 | 3.00 | 9601 |
| 2A-3 | Lyngberg | 20 | 600 | 540 | 120 | 700 | 380 | 95 | 125 | 6.00 | 5000 |
| 2B-3 | Lyngberg | 20 | 600 | 540 | 120 | 700 | 380 | 95 | 125 | 6.00 | 5000 |
| 3A-2 | Lyngberg | 20 | 600 | 540 | 120 | 700 | 380 | 95 | 125 | 6.00 | 5000 |
| 3B-2 | Lyngberg | 20 | 600 | 540 | 120 | 700 | 380 | 95 | 125 | 6.00 | 5000 |
| 4A-1 | Lyngberg | 20 | 600 | 540 | 120 | 700 | 380 | 95 | 125 | 6.00 | 5000 |
| 4B-1 | Lyngberg | 20 | 600 | 540 | 120 | 700 | 380 | 95 | 125 | 6.00 | 5000 |
| P4 | Moayer and Regan | 20 | 320 | 287 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| P8 | Moayer and Regan | 20 | 320 | 272 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| P13 | Moayer and Regan | 20 | 320 | 282 | 150 | 600 | - | 80 | - | 4.00 | 2000 |

Table A1 continued

| Specification | Tested by | $a_g$ (mm) | h (mm) | d (mm) | $b_v$ (mm) | $b_{top}$ (mm) | $b_{bot}$ (mm) | $t_{top}$ (mm) | $t_{bot}$ (mm) | $b/b_v$ | L (mm) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Compliant Prestressed Members | | | | | | | | | | | |
| P18 | Moayer and Regan | 20 | 320 | 269 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| P24 | Moayer and Regan | 20 | 320 | 282 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| P25 | Moayer and Regan | 20 | 320 | 282 | 150 | 600 | - | 80 | - | 4.00 | 3000 |
| P26 | Moayer and Regan | 20 | 320 | 269 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| P27 | Moayer and Regan | 20 | 320 | 269 | 150 | 600 | - | 80 | - | 4.00 | 3000 |
| P28 | Moayer and Regan | 20 | 320 | 272 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| P29 | Moayer and Regan | 20 | 320 | 272 | 150 | 600 | - | 80 | - | 4.00 | 3000 |
| P49 | Moayer and Regan | 20 | 320 | 274 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| P50 | Moayer and Regan | 20 | 320 | 274 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| Non-Prestressed Members without Stirrups | | | | | | | | | | | |
| YB 2000/0 | Yoshida | 10 | 2000 | 1890 | 300 | - | - | - | - | 1.00 | 10800 |
| N1-S | Yoon et al. | 20 | 750 | 655 | 375 | - | - | - | - | 1.00 | 4300 |
| P41 | Moayer and Regan | 20 | 320 | 279 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| 1 | Durham | 20 | 432 | 381 | 432 | 1067 | - | 127 | - | 2.47 | 5791 |
| 2 | Durham | 20 | 432 | 381 | 432 | 1067 | - | 127 | - | 2.47 | 5791 |
| 8 | Aster and Koch | 30 | 544 | 500 | 1000 | - | - | - | - | 1.00 | 5500 |
| 9 | Aster and Koch | 30 | 544 | 500 | 1000 | - | - | - | - | 1.00 | 5500 |
| 10 | Aster and Koch | 30 | 544 | 500 | 1000 | - | - | - | - | 1.00 | 5500 |
| 11 | Aster and Koch | 30 | 539 | 500 | 1000 | - | - | - | - | 1.00 | 3658 |
| 12 | Aster and Koch | 30 | 544 | 500 | 1000 | - | - | - | - | 1.00 | 3658 |
| 16 | Aster and Koch | 30 | 794 | 750 | 1000 | - | - | - | - | 1.00 | 5500 |

Table A1 continued

| Specification | Tested by | $a_g$ (mm) | h (mm) | d (mm) | $b_v$ (mm) | $b_{top}$ (mm) | $b_{bot}$ (mm) | $t_{top}$ (mm) | $t_{bot}$ (mm) | $b/b_v$ | L (mm) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Non-Prestressed Members without Stirrups | | | | | | | | | | | |
| 17 | Aster and Koch | 30 | 794 | 750 | 1000 | - | - | - | - | 1.00 | 5500 |
| AW1 | Lubell | 10 | 590 | 538 | 1170 | - | - | - | - | 1.00 | 3700 |
| AW4 | Lubell | 10 | 590 | 506 | 1168 | - | - | - | - | 1.00 | 3700 |
| AW8 | Lubell | 10 | 590 | 507 | 1168 | - | - | - | - | 1.00 | 3700 |
| DB120 | Angelakos | 10 | 1000 | 925 | 300 | - | - | - | - | 1.00 | 5400 |
| DB130 | Angelakos | 10 | 1000 | 925 | 300 | - | - | - | - | 1.00 | 5400 |
| DB140 | Angelakos | 10 | 1000 | 925 | 300 | - | - | - | - | 1.00 | 5400 |
| DB230 | Angelakos | 10 | 1000 | 895 | 300 | - | - | - | - | 1.00 | 5400 |
| DB0.530 | Angelakos | 10 | 1000 | 925 | 300 | - | - | - | - | 1.00 | 5400 |
| B100 | Angelakos | 10 | 1000 | 925 | 300 | - | - | - | - | 1.00 | 5400 |
| B100L | Angelakos | 10 | 1000 | 925 | 300 | - | - | - | - | 1.00 | 5400 |
| BN100 | Angelakos | 10 | 1000 | 925 | 300 | - | - | - | - | 1.00 | 5400 |
| Prestressed Members without Stirrups | | | | | | | | | | | |
| CI8 | Elzanaty et al. | 13 | 356 | 254 | 76 | 508 | 152 | 51 | 127 | 6.68 | 3810 |
| CW8 | Elzanaty et al | 13 | 457 | 362 | 51 | 203 | 203 | 102 | 102 | 3.98 | 3810 |
| P12 | Moayer and Regan | 20 | 320 | 282 | 150 | 600 | - | 80 | - | 4.00 | 3000 |
| P16 | Moayer and Regan | 20 | 320 | 272 | 150 | 600 | - | 80 | - | 4.00 | 3000 |
| P17 | Moayer and Regan | 20 | 320 | 269 | 150 | 600 | - | 80 | - | 4.00 | 3000 |
| P10 | Moayer and Regan | 20 | 320 | 269 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| P11 | Moayer and Regan | 20 | 320 | 282 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| P15 | Moayer and Regan | 20 | 320 | 272 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| P47 | Moayer and Regan | 20 | 320 | 274 | 150 | 600 | - | 80 | - | 4.00 | 2000 |
| P48 | Moayer and Regan | 20 | 320 | 269 | 150 | 600 | - | 80 | - | 4.00 | 2000 |

Table A2 – Member Properties

| Specification | Loading | $f_c'$ (MPa) | $A_s$ ($mm^2$) | $f_y$ (MPa) | $A_v$ ($mm^2$) | $f_v$ (MPa) | s (mm) | $A_p$ ($mm^2$) | $f_{pu}$ (MPa) | $f_{pe}/f_{pu}$ | $\rho$ (%) | a/d | $V_{test}$ (kN) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Non-Compliant Non-Prestressed Members | | | | | | | | | | | | | |
| YB 2000/9 | 1 Point | 33.6 | 4200 | 447 | 641 | 470 | 2700 | - | - | - | 0.74 | 2.86 | 472 |
| YB 2000/6 | 1 Point | 37.3 | 4200 | 447 | 285 | 465 | 1350 | - | - | - | 0.74 | 2.86 | 550 |
| YB 2000/4 | 1 Point | 36.4 | 4200 | 447 | 127 | 468 | 590 | - | - | - | 0.74 | 2.86 | 674 |
| 5084 | 2 Point | 26.5 | 2308 | 350 | 142 | 350 | 466 | - | - | - | 1.86 | 5.00 | 133 |
| 5053 | 2 Point | 26.5 | 2308 | 344 | 142 | 344 | 596 | - | - | - | 1.85 | 5.00 | 117 |
| 5052 | 2 Point | 26.5 | 2308 | 349 | 142 | 349 | 633 | - | - | - | 1.82 | 5.00 | 120 |
| 5051 | 2 Point | 26.5 | 2308 | 352 | 142 | 352 | 724 | - | - | - | 1.84 | 5.00 | 120 |
| 5063 | 2 Point | 28.6 | 2323 | 352 | 142 | 349 | 544 | - | - | - | 1.85 | 5.00 | 136 |
| N2-S | 1 Point | 36.0 | 7000 | 400 | 142 | 430 | 465 | - | - | - | 2.85 | 3.28 | 363 |
| N1-N | 1 Point | 36.0 | 7000 | 400 | 100 | 430 | 325 | - | - | - | 2.85 | 3.28 | 457 |
| P21 | 1 Point | 42.8 | 800 | 641 | 48.5 | 310 | 229 | - | - | - | 0.48 | 5.36 | 90 |
| Ss2-321-3 | 1 Point | 43.0 | 2580 | 386 | 142 | 276 | 533 | - | - | - | 2.23 | 4.01 | 141 |
| Ss2-318-3 | 1 Point | 43.0 | 2580 | 386 | 142 | 276 | 457 | - | - | - | 2.23 | 4.01 | 175 |
| Ss2-313.5-3 | 1 Point | 42.7 | 2580 | 386 | 142 | 276 | 343 | - | - | - | 2.23 | 4.01 | 213 |
| Ss2-321-2 | 1 Point | 38.0 | 2580 | 386 | 142 | 352 | 533 | - | - | - | 2.23 | 4.01 | 167 |
| Ss2-318-2 | 1 Point | 38.9 | 2580 | 386 | 142 | 352 | 457 | - | - | - | 2.23 | 4.01 | 177 |
| Ss2-321-1 | 1 Point | 38.7 | 2580 | 386 | 142 | 517 | 533 | - | - | - | 2.23 | 4.01 | 164 |
| Ss2-318-1 | 1 Point | 40.5 | 2580 | 386 | 142 | 517 | 457 | - | - | - | 2.23 | 4.01 | 220 |
| Ss2-218a-2 | 1 Point | 37.6 | 2580 | 386 | 65 | 372 | 457 | - | - | - | 2.23 | 4.01 | 164 |
| Ss2-213.5a-2 | 1 Point | 37.0 | 2580 | 386 | 65 | 372 | 343 | - | - | - | 2.23 | 4.01 | 161 |
| Ss2-213.5-1 | 1 Point | 38.9 | 2580 | 386 | 65 | 341 | 343 | - | - | - | 2.23 | 4.01 | 148 |
| 7 | 2 Point | 51.3 | 4095 | 524 | 64 | 479 | 267 | - | - | - | 2.49 | 3.10 | 280 |
| 8 | 2 Point | 51.3 | 4095 | 524 | 64 | 479 | 267 | - | - | - | 2.49 | 3.10 | 258 |
| BM100 | 1 Point | 47.0 | 2100 | 483 | 142 | 508 | 600 | - | - | - | 0.76 | 2.92 | 342 |
| SB 2012/6 | 1 Point | 27.0 | 8400 | 436 | 284 | 483 | 1350 | - | - | - | 1.52 | 2.92 | 635 |
| SB 2003/6 | 1 Point | 31.0 | 2100 | 436 | 284 | 483 | 1350 | - | - | - | 0.36 | 2.81 | 350 |
| 10T24-B4 | 2 Point | 23.6 | 6036 | 478 | 258 | 276 | 610 | - | - | - | 0.59 | 3.02 | 537 |
| PE 1 | 1 Point | 45.4 | 2904 / leg | 311 | 142 / leg | 395 | 762 | - | - | - | 1.21 | 4.21 | 200 |
| PE 2 | 1 Point | 45.4 | 1902 / leg | 344 | 142 / leg | 395 | 978 | - | - | - | 0.79 | 3.60 | 179 |
| Non-Compliant Prestressed Members | | | | | | | | | | | | | |
| CH-6-240 | 2 Point | 57.8 | 284 | 410 | 26 | 545 | 240 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 103 |
| CM-6-240 | 2 Point | 56.9 | 284 | 410 | 28 | 418 | 240 | 231 | 1720 | 0.46 | 1.14 | 3.00 | 100 |
| CL-6-240 | 2 Point | 56.5 | 284 | 410 | 32 | 280 | 240 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 103 |
| PH-6-240 | 2 Point | 42.1 | 284 | 410 | 26 | 545 | 240 | 231 | 1720 | 0.63 | 1.14 | 3.00 | 81 |
| PM-6-240 | 2 Point | 42.8 | 284 | 410 | 28 | 418 | 240 | 231 | 1720 | 0.64 | 1.14 | 3.00 | 85 |
| PL-6-240 | 2 Point | 44.2 | 284 | 410 | 32 | 280 | 240 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 89 |
| NM-10-240 | 2 Point | 40.6 | 284 | 410 | 79 | 410 | 240 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 96 |

Table A2 continued

| Specification | Loading | $f_c$' (MPa) | $A_s$ ($mm^2$) | $f_y$ (MPa) | $A_v$ ($mm^2$) | $f_v$ (MPa) | s (mm) | $A_p$ ($mm^2$) | $f_{pu}$ (MPa) | $f_{pe}/f_{pu}$ | $\rho$ (%) | a/d | $V_{test}$ (kN) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Non-Compliant Prestressed Members | | | | | | | | | | | | | |
| NL-10-240 | 2 Point | 39.4 | 284 | 410 | 71 | 280 | 240 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 94 |
| NM-8-240 | 2 Point | 35.3 | 284 | 410 | 52 | 420 | 240 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 80 |
| NH-6-240 | 2 Point | 35.4 | 284 | 410 | 26 | 545 | 240 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 89 |
| NM-6-240 | 2 Point | 38.5 | 284 | 410 | 28 | 418 | 240 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 89 |
| NL-6-240 | 2 Point | 41.2 | 284 | 410 | 32 | 280 | 240 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 82 |
| CW12 | 2 Point | 40.0 | 213 | 434 | 71 | 434 | 254 | 560 | 1860 | 0.45 | 0.99 | 3.80 | 141 |
| CW11 | 2 Point | 55.8 | 213 | 434 | 71 | 434 | 254 | 560 | 1860 | 0.45 | 0.99 | 3.80 | 157 |
| CI12 | 2 Point | 40.0 | 213 | 434 | 71 | 434 | 203 | 560 | 1860 | 0.45 | 0.56 | 5.80 | 122 |
| CI11 | 2 Point | 55.8 | 213 | 434 | 71 | 434 | 203 | 560 | 1860 | 0.45 | 0.56 | 5.80 | 127 |
| P9 | 1 Point | 40.4 | 400 | 641 | 45 | 310 | 229 | 297 | 1882 | 0.59 | 0.42 | 5.51 | 120 |
| P14 | 1 Point | 44.1 | 400 | 641 | 45 | 310 | 229 | 148 | 1882 | 0.61 | 0.32 | 5.32 | 97 |
| P19 | 1 Point | 45.4 | 200 | 641 | 45 | 310 | 229 | 297 | 1882 | 0.59 | 0.31 | 5.57 | 109 |
| BW.14.34 | 2 Point | 18.6 | - | - | 26 | 234 | 267 | 155 | 1779 | 0.48 | 0.40 | 3.56 | 57 |
| Compliant Non-Prestressed Members | | | | | | | | | | | | | |
| V18-2 | 1 Point | 35.9 | 1018 | 910 | 64 | 483 | 186 | - | - | - | 1.05 | 3.00 | 172 |
| V36-3 | 1 Point | 42.7 | 3870 | 483 | 142 | 538 | 371 | - | - | - | 1.00 | 3.00 | 512 |
| V36-2 | 1 Point | 42.7 | 3870 | 483 | 64 | 483 | 165 | - | - | - | 1.00 | 3.00 | 488 |
| V1 | 1 Point | 36.5 | 3870 | 476 | 142 | 483 | 371 | - | - | - | 1.00 | 3.00 | 395 |
| V2 | 1 Point | 36.5 | 3870 | 476 | 142 | 483 | 371 | - | - | - | 1.00 | 3.00 | 492 |
| A1 | 1 Point | 22.6 | 2400 | 440 | 64 | 600 | 210 | - | - | - | 1.72 | 4.00 | 230 |
| A2 | 1 Point | 25.9 | 3100 | 440 | 64 | 600 | 210 | - | - | - | 2.22 | 5.00 | 220 |
| B1 | 1 Point | 22.6 | 2400 | 440 | 64 | 600 | 190 | - | - | - | 2.29 | 4.00 | 217 |
| B2 | 1 Point | 25.9 | 2400 | 440 | 64 | 600 | 190 | - | - | - | 2.29 | 5.00 | 183 |
| C1 | 1 Point | 22.6 | 1400 | 440 | 64 | 600 | 210 | - | - | - | 2.02 | 4.00 | 141 |
| C2 | 1 Point | 25.9 | 2400 | 440 | 64 | 600 | 210 | - | - | - | 3.46 | 5.00 | 145 |
| P5 | 1 Point | 43.0 | 626 | 641 | 64.5 | 255 | 102 | - | - | - | 0.37 | 3.45 | 145 |
| P20 | 1 Point | 40.7 | 806 | 641 | 45 | 310 | 152 | - | - | - | 0.48 | 3.55 | 120 |
| P22 | 1 Point | 43.3 | 806 | 641 | 64.5 | 225 | 152 | - | - | - | 0.48 | 5.36 | 109 |
| DBO530M | 1 Point | 32.0 | 1400 | 550 | 71 | 508 | 300 | - | - | - | 0.50 | 2.92 | 263 |
| DB120M | 1 Point | 21.0 | 2800 | 550 | 142 | 508 | 600 | - | - | - | 1.01 | 2.92 | 282 |
| DM140M | 1 Point | 38.0 | 2800 | 550 | 71 | 508 | 300 | - | - | - | 1.01 | 2.92 | 277 |
| Ss2-29g-2 | 1 Point | 15.7 | 2580 | 386 | 64.5 | 372 | 229 | - | - | - | 2.23 | 4.01 | 150 |
| Ss2-29e-2 | 1 Point | 48.5 | 2580 | 386 | 64.5 | 372 | 229 | - | - | - | 2.23 | 4.01 | 206 |
| Ss2-29d-2 | 1 Point | 30.4 | 2580 | 386 | 64.5 | 372 | 229 | - | - | - | 2.23 | 4.01 | 165 |
| Ss2-29c-2 | 1 Point | 24.1 | 2580 | 386 | 64.5 | 372 | 229 | - | - | - | 2.23 | 4.01 | 161 |
| Ss2-29b-2 | 1 Point | 41.4 | 2580 | 386 | 64.5 | 372 | 229 | - | - | - | 2.23 | 4.01 | 202 |
| Ss2-29a-2 | 1 Point | 37.2 | 2580 | 386 | 64.5 | 372 | 229 | - | - | - | 2.23 | 4.01 | 217 |
| Ss2-29b-1 | 1 Point | 37.6 | 2580 | 386 | 64.5 | 341 | 229 | - | - | - | 2.23 | 4.01 | 160 |
| Ss2-29a-1 | 1 Point | 38.8 | 2580 | 386 | 64.5 | 341 | 229 | - | - | - | 2.23 | 4.01 | 160 |

Table A2 continued

| Specification | Loading | $f_c'$ (MPa) | $A_s$ (mm$^2$) | $f_y$ (MPa) | $A_v$ (mm$^2$) | $f_v$ (MPa) | s (mm) | $A_p$ (mm$^2$) | $f_{pu}$ (MPa) | $f_{pe}/f_{pu}$ | $\rho$ (%) | a/d | $V_{test}$ (kN) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Compliant Non-Prestressed Members | | | | | | | | | | | | | |
| Ss2-26-1 | 1 Point | 40.0 | 2580 | 386 | 64.5 | 341 | 150 | - | - | - | 2.23 | 4.01 | 207 |
| 1 | 2 Point | 36.4 | 4095 | 525 | 65 | 479 | 133 | - | - | - | 2.49 | 3.10 | 338 |
| 5 | 2 Point | 55.8 | 4095 | 525 | 65 | 479 | 133 | - | - | - | 2.49 | 3.10 | 383 |
| 5A-0 | 2 Point | 25.7 | 2515 | 640 | 100 | 674 | 157 | - | - | - | 0.66 | 2.78 | 435 |
| 5B-0 | 2 Point | 26.6 | 2515 | 640 | 100 | 647 | 157 | - | - | - | 0.66 | 2.78 | 435 |
| Test 1.1 | 1 Point | 47.7 | 3456 | 350 | 284 | 435 | 254 | - | - | - | 0.87 | 3.09 | 498 |
| Test 2.1 | 1 Point | 50.6 | 1851 | 314 | 142 | 448 | 254 | - | - | - | 0.93 | 3.09 | 219 |
| Test 2.2 | 1 Point | 50.6 | 1851 | 314 | 142 | 448 | 254 | - | - | - | 0.93 | 3.09 | 209 |
| Test 2.3 | 1 Point | 50.6 | 3702 | 314 | 284 | 448 | 254 | - | - | - | 0.94 | 3.09 | 428 |
| T1 | 2 Point | 30.0 | 8464 | 465 | 232 | 427 | 80 | - | - | - | 0.68 | 3.03 | 785 |
| ET1 | 2 Point | 28.0 | 1257 | 420 | 60 | 314 | 110 | - | - | - | 1.40 | 3.50 | 142 |
| ET2 | 2 Point | 28.0 | 1257 | 420 | 60 | 314 | 110 | - | - | - | 1.40 | 3.50 | 132 |
| ET3 | 2 Point | 28.0 | 1257 | 420 | 60 | 314 | 110 | - | - | - | 1.40 | 3.50 | 128 |
| 2T10 | 2 Point | 24.5 | 6036 | 523 | 258 | 491 | 254 | - | - | - | 0.59 | 3.04 | 898 |
| 2T12 | 2 Point | 24.3 | 6036 | 523 | 258 | 491 | 305 | - | - | - | 0.59 | 3.04 | 840 |
| 1T18 | 2 Point | 34.0 | 6036 | 463 | 258 | 491 | 457 | - | - | - | 0.59 | 3.04 | 738 |
| Compliant Prestressed Members | | | | | | | | | | | | | |
| PL-6-160 | 2 Point | 42.1 | 284 | 410 | 32 | 280 | 160 | 231 | 1720 | 0.65 | 1.14 | 3.00 | 98 |
| NL-6-80 | 2 Point | 39.8 | 284 | 410 | 32 | 280 | 80 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 106 |
| NH-6-80 | 2 Point | 38.7 | 284 | 410 | 26 | 545 | 80 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 114 |
| NM-6-80 | 2 Point | 37.1 | 284 | 410 | 28 | 418 | 80 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 106 |
| NL-6-160 | 2 Point | 39.1 | 284 | 410 | 32 | 280 | 160 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 91 |
| NM-6-160 | 2 Point | 38.2 | 284 | 410 | 28 | 418 | 160 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 94 |
| CH-6-80 | 2 Point | 55.9 | 284 | 410 | 26 | 545 | 80 | 231 | 1720 | 0.46 | 1.14 | 3.00 | 140 |
| CM-6-80 | 2 Point | 59.5 | 284 | 410 | 28 | 418 | 80 | 231 | 1720 | 0.46 | 1.14 | 3.00 | 126 |
| CL-6-80 | 2 Point | 60.0 | 284 | 410 | 32 | 280 | 80 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 116 |
| CH-6-160 | 2 Point | 59.3 | 284 | 410 | 26 | 545 | 160 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 112 |
| CL-6-160 | 2 Point | 58.9 | 284 | 410 | 32 | 280 | 160 | 231 | 1720 | 0.45 | 1.14 | 3.00 | 103 |
| PM-6-80 | 2 Point | 43.9 | 284 | 410 | 28 | 418 | 80 | 231 | 1720 | 0.62 | 1.14 | 3.00 | 115 |
| PM-6-160 | 2 Point | 42.3 | 284 | 410 | 28 | 418 | 160 | 231 | 1720 | 0.65 | 1.14 | 3.00 | 96 |
| PL-6-80 | 2 Point | 43.8 | 284 | 410 | 32 | 280 | 80 | 231 | 1720 | 0.64 | 1.14 | 3.00 | 106 |
| A1-00-1.5R-N | 1 Point | 52.4 | - | - | 258 | 483 | 135 | 1580 | 1860 | 0.57 | 0.35 | 2.60 | 920 |
| B0-00-R-S | 1 Point | 51.4 | - | - | 258 | 483 | 445 | 1619 | 1860 | 0.57 | 0.36 | 3.14 | 916 |
| B0-00-R-N | 1 Point | 51.4 | - | - | 258 | 483 | 203 | 1619 | 1860 | 0.57 | 0.36 | 2.58 | 979 |
| A1-00-M-N | 1 Point | 50.3 | - | - | 258 | 483 | 457 | 1580 | 1860 | 0.57 | 0.35 | 2.60 | 627 |
| A1-00-R-N | 1 Point | 49.0 | - | - | 258 | 483 | 203 | 1580 | 1860 | 0.57 | 0.35 | 2.60 | 934 |
| A1-00-M-S | 1 Point | 50.3 | - | - | 258 | 483 | 457 | 1580 | 1860 | 0.57 | 0.35 | 3.16 | 747 |
| A1-00-0.5R-N | 1 Point | 49.0 | - | - | 258 | 483 | 406 | 1580 | 1860 | 0.57 | 0.35 | 2.60 | 738 |
| A1-00-0.5R-S | 1 Point | 49.0 | - | - | 258 | 483 | 445 | 1580 | 1860 | 0.57 | 0.35 | 3.16 | 770 |

Table A2 continued

| Specification | Loading | $f_c'$ (MPa) | $A_s$ ($mm^2$) | $f_y$ (MPa) | $A_v$ ($mm^2$) | $f_v$ (MPa) | s (mm) | $A_p$ ($mm^2$) | $f_{pu}$ (MPa) | $f_{pe}/f_{pu}$ | $\rho$ (%) | a/d | $V_{test}$ (kN) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Compliant Prestressed Members | | | | | | | | | | | | | |
| 2A-3 | 2 Point | 32.6 | 616 | 442 | 100 | 616 | 157 | 678 | 1843 | 0.51 | 0.34 | 2.78 | 506 |
| 2B-3 | 2 Point | 33.9 | 616 | 441 | 100 | 643 | 157 | 678 | 1843 | 0.50 | 0.34 | 2.78 | 515 |
| 3A-2 | 2 Point | 31.1 | 1232 | 445 | 100 | 663 | 157 | 452 | 1843 | 0.51 | 0.45 | 2.78 | 489 |
| 3B-2 | 2 Point | 27.5 | 1232 | 446 | 100 | 625 | 157 | 452 | 1843 | 0.51 | 0.45 | 2.78 | 433 |
| 4A-1 | 2 Point | 31.5 | 1873 | 481 | 100 | 639 | 157 | 226 | 1843 | 0.52 | 0.55 | 2.78 | 469 |
| 4B-1 | 2 Point | 30.4 | 1873 | 481 | 100 | 658 | 157 | 226 | 1843 | 0.50 | 0.55 | 2.78 | 454 |
| P4 | 1 Point | 40.0 | 400 | 641 | 65 | 255 | 102 | 110 | 1882 | 0.62 | 0.30 | 3.45 | 142 |
| P8 | 1 Point | 42.7 | 400 | 641 | 49 | 310 | 152 | 297 | 1882 | 0.59 | 0.43 | 3.64 | 178 |
| P13 | 1 Point | 39.4 | 400 | 641 | 49 | 310 | 152 | 148 | 1882 | 0.61 | 0.32 | 3.51 | 140 |
| P18 | 1 Point | 44.5 | 200 | 641 | 49 | 310 | 152 | 297 | 1882 | 0.59 | 0.31 | 3.68 | 160 |
| P24 | 1 Point | 43.4 | 400 | 641 | 65 | 255 | 102 | 148 | 1882 | 0.61 | 0.32 | 3.51 | 148 |
| P25 | 1 Point | 44.0 | 400 | 641 | 65 | 255 | 152 | 148 | 1882 | 0.61 | 0.32 | 5.32 | 104 |
| P26 | 1 Point | 47.3 | 200 | 641 | 65 | 255 | 102 | 297 | 1882 | 0.59 | 0.31 | 3.68 | 170 |
| P27 | 1 Point | 45.4 | 200 | 641 | 65 | 255 | 152 | 297 | 1882 | 0.59 | 0.31 | 5.57 | 115 |
| P28 | 1 Point | 44.9 | 400 | 641 | 65 | 255 | 102 | 297 | 1882 | 0.59 | 0.43 | 3.64 | 194 |
| P29 | 1 Point | 46.5 | 400 | 641 | 65 | 225 | 152 | 297 | 1882 | 0.59 | 0.43 | 5.51 | 135 |
| P49 | 1 Point | 37.8 | 800 | 641 | 65 | 255 | 102 | 297 | 1882 | 0.59 | 0.67 | 3.61 | 190 |
| P50 | 1 Point | 41.2 | 800 | 641 | 110 | 283 | 102 | 297 | 1882 | 0.45 | 0.67 | 3.61 | 230 |
| Non-Prestressed Members without Stirrups | | | | | | | | | | | | | |
| YB 2000/0 | 1 Point | 33.6 | 4200 | 457 | - | - | - | - | - | - | 0.74 | 2.86 | 255 |
| N1-S | 1 Point | 36.0 | 7000 | 400 | - | - | - | - | - | - | 2.85 | 3.28 | 249 |
| P41 | 1 Point | 44.0 | 800 | 641 | - | - | - | - | - | - | 0.48 | 3.55 | 70 |
| 1 | 2 Point | 55.0 | 2580 | 414 | - | - | - | - | - | - | 0.63 | 4.40 | 178 |
| 2 | 2 Point | 55.0 | 2580 | 414 | - | - | - | - | - | - | 0.63 | 4.40 | 169 |
| 8 | 1 Point | 31.1 | 3150 | 536 | - | - | - | - | - | - | 0.63 | 5.50 | 280 |
| 9 | 1 Point | 19.9 | 3150 | 536 | - | - | - | - | - | - | 0.63 | 5.50 | 254 |
| 10 | 1 Point | 20.0 | 3150 | 536 | - | - | - | - | - | - | 0.63 | 5.50 | 255 |
| 11 | 1 Point | 24.6 | 2300 | 535 | - | - | - | - | - | - | 0.46 | 3.65 | 261 |
| 12 | 1 Point | 27.3 | 3250 | 535 | - | - | - | - | - | - | 0.71 | 3.65 | 324 |
| 16 | 1 Point | 30.4 | 3150 | 536 | - | - | - | - | - | - | 0.42 | 3.67 | 392 |
| 17 | 1 Point | 28.7 | 3150 | 536 | - | - | - | - | - | - | 0.42 | 3.67 | 349 |
| AW1 | 1 Point | 36.9 | 5000 | 457 | - | - | - | - | - | - | 0.79 | 3.43 | 585 |
| AW4 | 1 Point | 39.9 | 10000 | 457 | - | - | - | - | - | - | 1.69 | 3.43 | 716 |
| AW8 | 1 Point | 39.4 | 10000 | 457 | - | - | - | - | - | - | 1.69 | 3.43 | 783 |
| DB120 | 1 Point | 21.0 | 2800 | 550 | - | - | - | - | - | - | 1.01 | 2.92 | 179 |
| DB130 | 1 Point | 32.0 | 2800 | 550 | - | - | - | - | - | - | 1.01 | 2.92 | 185 |
| DB140 | 1 Point | 38.0 | 2800 | 550 | - | - | - | - | - | - | 1.01 | 2.92 | 180 |
| DB230 | 1 Point | 32.0 | 5600 | 550 | - | - | - | - | - | - | 2.09 | 2.92 | 257 |
| DB0.530 | 1 Point | 32.0 | 1400 | 550 | - | - | - | - | - | - | 0.50 | 2.92 | 165 |

Table A2 continued

| Specification | Loading | $f_c'$ (MPa) | $A_s$ ($mm^2$) | $f_y$ (MPa) | $A_v$ ($mm^2$) | $f_v$ (MPa) | s (mm) | $A_p$ ($mm^2$) | $f_{pu}$ (MPa) | $f_{pe}/f_{pu}$ | $\rho$ (%) | a/d | $V_{test}$ (kN) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Non-Prestressed Members without Stirrups | | | | | | | | | | | | | |
| B100 | 1 Point | 36.0 | 2800 | 550 | - | - | - | - | - | - | 1.01 | 2.92 | 225 |
| B100L | 1 Point | 39.0 | 2800 | 483 | - | - | - | - | - | - | 1.01 | 2.92 | 223 |
| BN100 | 1 Point | 37.0 | 2100 | 550 | - | - | - | - | - | - | 0.76 | 2.92 | 192 |
| Prestressed Members without Stirrups | | | | | | | | | | | | | |
| CI8 | 2 Point | 41.4 | 213 | 434 | - | - | - | 561 | 1860 | 0.47 | 0.60 | 5.80 | 87 |
| CW8 | 2 Point | 41.4 | 213 | 434 | - | - | - | 561 | 1860 | 0.47 | 1.05 | 3.80 | 90 |
| P12 | 1 Point | 42.3 | 400 | 641 | - | - | - | 148 | 1882 | 0.61 | 0.32 | 5.32 | 68 |
| P16 | 1 Point | 42.7 | 400 | 641 | - | - | - | 297 | 1882 | 0.59 | 0.43 | 5.51 | 113 |
| P17 | 1 Point | 39.6 | 200 | 641 | - | - | - | 297 | 1882 | 0.59 | 0.31 | 5.57 | 106 |
| P10 | 1 Point | 41.6 | 200 | 641 | - | - | - | 297 | 1882 | 0.59 | 0.31 | 3.68 | 160 |
| P11 | 1 Point | 39.0 | 400 | 641 | - | - | - | 148 | 1882 | 0.61 | 0.32 | 3.51 | 110 |
| P15 | 1 Point | 44.1 | 400 | 641 | - | - | - | 297 | 1882 | 0.59 | 0.43 | 3.64 | 160 |
| P47 | 1 Point | 42.5 | 800 | 641 | - | - | - | 297 | 1882 | 0.59 | 0.67 | 3.61 | 160 |
| P48 | 1 Point | 41.2 | 200 | 641 | - | - | - | 297 | 1882 | 0.59 | 0.31 | 3.68 | 141 |

# Appendix B

# Sample Calculations

**Sample Calculations 1 – Non-Prestressed Non-Compliant Member– PE 1 (DeGeer and Stephens, 1993)**

| | | |
|---|---|---|
| $f_c' = 45.4$ MPa | $A_s = 2904$ mm$^2$ | $f_y = 311$ MPa |
| $d = 528$ mm | $\rho = 0.0136$ | $f_v = 395$ MPa |
| $b = 456$ mm | $a_g = 20$ mm | $A_v = 142$ mm$^2$ |
| $b_v = 207$ mm | $d_v = 475$ mm | $a/d = 4.21$ |
| $s = 762$ mm | $(M/V) = 1.75$ | $V_{test} = 200$ kN |
| $L = 10663$ mm | $w_D = 3.81$ kN/m | $M_{test} = 350$ kN-m |
| $t_f = 102$ mm | $f_{cr} = 2.70$ MPa | $l_{bearing} = 200$ mm |
| | | $l_d = 474$ mm |

Load and resistance factors are taken as 1.0 in the following example and thus are not shown in the calculations.

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**CSA S6-06 Sectional Shear Method**

Iteration 1 (point of interest taken at $d_v$ away from the applied load). Iteration is accomplished in this study by varying the *externally applied* shears and moments. The subscript *applied* represents the externally applied loads.

$V_{applied} = 188$ kN

$M_{applied} = 329$ kN-m

$V_D = 12$ kN

$M_D = 36$ kN-m

Determine S6-06 Section 14 minimum stirrup spacing and area requirements.

$$A_{v,\min} = 0.15 \cdot f_{cr} \cdot \frac{b_v \cdot s}{f_v} \qquad \text{Eqn. (2.1)}$$

$$= 0.15 \cdot 2.70MPa \cdot \frac{207mm \cdot 762mm}{395MPa}$$

$=161$ mm$^2$

Use shear area interpolation expression (Eqn. 3.12) proposed in Section 3.2 to interpolate for stirrup area deficiency.

$$\gamma = 10 \cdot \frac{A_v \cdot f_v}{f_{cr} \cdot b_v \cdot s} - 0.5 \text{ where } 0 \leq \gamma \leq 1. \qquad \text{Eqn. (3.12)}$$

$$= 10 \cdot \frac{142mm^2 \cdot 395MPa}{2.70MPa \cdot 207mm \cdot 762mm} - 0.50$$

=0.819

A value less than 1.00 indicates that a member does not comply with S6-06 Section 14 stirrup area requirements.

Determine the normalized shear demand. As discussed in Section 3.2 normalized shear demand is used to determine the maximum permissible stirrup spacing $s_{m1}$. This test specimen is not prestressed so $V_p$ is zero.

$$\frac{V_f}{f_c^{'} \cdot b_v \cdot d_v} = \frac{200000N}{45.4MPa \cdot 207mm \cdot 475mm} \qquad \text{Eqn (3.8)}$$

=0.045

Based on the normalized shear stress and the use of Figure 2.1, the maximum allowable stirrup spacing $s_{m1}$ is determined. As discussed in Section 4.5.1 this study recommends neglecting the $s_{m2}$ stirrup spacing limit.

$s_{m1}$ = 356 mm

Because $s > s_{m1}$ and $A_{v,min} > A_v$ S6-06 Section 8 requires that the assumed inclined crack spacing be take equal to the shear depth.

$s_z = d_v$ = 475 mm

The effective crack spacing $s_{ze}$ is determined using Eqn. (3.11):

$$s_{ze} = 35 \cdot \frac{s_z}{15 + a_g} = 35 \cdot \frac{475mm}{15 + 20mm} \qquad \text{Eqn. (3.11)}$$

= 475 mm

The longitudinal strain at mid-depth is determined using Eqn. (3.10). The term $A_{ct} \cdot E_c$ is taken as zero because the longitudinal strain term is positive. There is no applied tensile loads or prestressing, therefore $N_f$ and $A_p$ are also taken as zero.

$$\varepsilon_x = \frac{\frac{M_f}{d_v} + V_f + N_f - A_p \cdot f_{po}}{2 \cdot \left(E_s \cdot A_s + E_p \cdot A_p + A_{ct} \cdot E_c\right)} = \frac{\frac{365 \times 10^6 \, N \cdot mm}{475mm} + 200 \times 10^3 \, N}{2 \cdot \left(200000 MPa \cdot 2904 mm^2\right)} \qquad \text{Eqn (3.10)}$$

=0.000 832

The shear term $\beta$, which provides an indication of the ability of a member to resist shear by aggregate interlock, is determined using Eqn. (3.5).

$$\beta = \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = \frac{0.40}{1 + 1500 \cdot 0.000832} \cdot \frac{1300}{1000 + 475mm} \qquad \text{Eqn. (3.5)}$$

=0.157

The shear term $\theta$, which calculates the predicted angle of the compression field, is calculated using Eqn. (3.7).

$$\theta = \left(29 + 7000 \cdot \varepsilon_x\right) \cdot \left(0.88 + \frac{s_{ze}}{2500}\right) \qquad \text{Eqn. (3.7)}$$

$$= \left(29 + 7000 \cdot 0.000832\right) \cdot \left(0.88 + \frac{475mm}{2500}\right)$$

=37.3$^{\circ}$

The concrete contribution to shear capacity is calculated using Eqn. (3.2). The stirrup contribution to shear capacity, modified using Eqn (3.12), is determined using Eqn. (3.3). The concrete cracking strength $f_{cr}$ is determined in accordance with S6-06 Clause 8.4.1.8.1 as discussed in Section 3.2.

$$V_c = 2.5 \cdot \beta \cdot f_{cr} \cdot d_v \cdot b_v = 2.5 \cdot 0.157 \cdot 2.70 MPa \cdot 475mm \cdot 207mm \qquad \text{Eqn. (3.2)}$$

=104 kN

$$V_s = \gamma \cdot \frac{A_v \cdot f_v \cdot d_v}{s \cdot \tan\theta} = 0.819 \cdot \frac{142mm^2 \cdot 395MPa \cdot 475mm}{762mm \cdot \tan(37.3^\circ)} \qquad \text{Eqn. (3.3)}$$

=38 kN

The sum of the concrete and stirrup contributions to shear capacity is used to determine the total shear capacity of test specimen PE1.

$V_r = V_c + V_c$ = 104 kN + 38 kN Eqn. (3.1)

=142 kN

Because the first iteration of shear demand $V_f$ = 200 kN does not equal $V_r$ the predicted shear capacity has not been properly converged on. The next iteration is taken as the average of the $V_f$ and $V_r$ from the previous iteration. Therefore

$V_{r,n+1}$ is taken as $\frac{200kN + 142kN}{2} = 171kN$.

Table B.1 provides the iterations of applied loads until a converged shear capacity is achieved. The term $V_f$ represents the shear demand resulting from both the externally applied load and the self-weight at a section $d_v$ away from the externally applied load.

Table B.1 – Evaluation of PE1 – S6-06

| Iteration | $V_f$ (kN) | $s_{ze}$ (mm) | $\frac{v}{\phi_c \cdot f'_c}$ | $\varepsilon_x$ x $10^6$ | $V_c$ (kN) | $V_s$ (kN) | $V_r$ (kN) |
|---|---|---|---|---|---|---|---|
| 1 | 200 | 475 | 0.045 | 832 | 104 | 38 | 142 |
| 2 | 171 | 475 | 0.038 | 714 | 113 | 39 | 152 |
| 3 | 162 | 475 | 0.036 | 679 | 116 | 39 | 155 |
| n | 157 | 475 | 0.035 | 660 | 117 | 39 | 157 |

The converged shear capacity of Specimen PE 1 calculated using the sectional method in S6-06 Section 8 is 157 kN. This compares appropriately well to the tested shear capacity of 200 kN (see Table 4.1). The normalized shear demand at iteration $n$ is less than 0.25

which indicates that the member is not expected to fail due to web crushing prior to beam shear failure.

**Check Moment Capacity at Final Iteration of Load**

$$\alpha = 0.85 - 0.0015 \cdot f_c^{'} = 0.85 - 0.0015 \cdot 45.4MPa$$

=0.78

$$M_r = A_s \cdot f_y \cdot \left( d - \frac{A_s \cdot f_y}{2 \cdot \alpha \cdot f_c^{'} \cdot b} \right)$$
$$= 2904mm^2 \cdot 311MPa \cdot \left( 528mm - \frac{2904mm^2 \cdot 311MPa}{2 \cdot 0.78 \cdot 45.4MPa \cdot 456mm} \right)$$
$$= 451kN \cdot m$$

$M_f$ = 12 kN·m + 254 kN·m

=266 kN·m < $M_r$. Therefore Specimen PE 1 is not expected to fail in flexural.

**Check anchorage of the longitudinal reinforcement in accordance with S6-06 Clause 8.9.3.14.**

$$T_r = A_s \cdot f_y \cdot \frac{l_{bearing}}{l_d} = 2 \cdot 819mm^2 \cdot \frac{200mm}{474mm}$$
$$= 215kN$$

$$V_{anchorage} = T_r \cdot \tan\theta + 0.50 \cdot \min(V_s, V_f)$$
$$= 215kN \cdot \tan(36) + 0.50 \cdot \min(39kN, 157kN)$$
$$= 176kN$$

$V_{anchorage}$ is greater than $V_f$ therefore we do not expect specimen PE 1 to be governed by anchorage capacity of the longitudinal reinforcement.

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**ACI 318-08 Sectional Shear Method**

Determine ACI 318-08 maximum stirrup spacing as discussed in Section 3.5.

$$s_{\max} = 0.5 \cdot d = 0.5 \cdot 528mm \leq 600 \text{ mm}$$

= 264 mm

Determine ACI 318-08 minimum permissible stirrup area.

$$A_v = 0.06 \cdot \sqrt{f_c^{'}} \cdot \frac{b_v \cdot s}{f_v} = 0.06 \cdot \sqrt{45.4}MPa \cdot \frac{207mm \cdot 762mm}{395MPa} \qquad \text{Eqn. (2.6)}$$

= 161 mm$^2$

The concrete contribution to shear capacity for non-prestressed members is calculated using Eqn. (3.27). This expression is not dependent on sectional forces, therefore iteration is not required for calculating shear capacity. The stirrup contribution to shear capacity is calculated using Eqn. (3.29). These expressions are discussed in Section 3.5.

$$V_c = \frac{1}{6} \cdot \sqrt{f_c^{'}} \cdot b_v \cdot d = 0.17 \cdot \sqrt{45.4}MPa \cdot 207mm \cdot 528kN \qquad \text{Eqn. (3.27)}$$

=125 kN

$$V_s = \frac{A_v \cdot f_v \cdot d}{s} = \frac{142mm^2 \cdot 395MPa \cdot 528mm}{762mm} \qquad \text{Eqn. (3.29)}$$

= 39 kN

ACI 318-08 calculates the sectional shear capacity using Eqn. (3.25)

$$V_r = V_c + V_s = 125 \text{ kN} + 39 \text{ kN} \qquad \text{Eqn. (3.25)}$$

= 164 kN

ACI 318-08 predicts the shear capacity of Specimen PE 1 to be 164 kN which compares appropriately well to the tested shear capacity of 200 kN (see Table 4.1).

**S6-06 M Sectional Shear Evaluation Method same as CSA S6-06 Sectional Shear Method – $s > d_v$ therefore $s_z = d_v$**

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**S6-06 F Sectional Shear Evaluation Method**

Iteration 1 (point of interest taken at $d_v$ away from the applied load). It should be noted that iteration is accomplished in this study by varying the *externally applied* shears and moments.

$V_{applied}$= 188 kN

$M_{applied}$ = 329 kN-m

$V_D$ = 12 kN

$M_D$ = 36 kN-m

$$A_v = 0.15 \cdot f_{cr} \cdot \frac{b_v \cdot s}{f_v} = 0.15 \cdot 2.70MPa \cdot \frac{207mm \cdot 762mm}{395MPa}$$ Eqn. (2.1)

=161 mm$^2$

$$\gamma = 10 \cdot \frac{A_v \cdot f_v}{f_{cr} \cdot b_v \cdot s} - 0.5$$ where $0 \le \gamma \le 1.0$ Eqn. (3.12)

$$= 10 \cdot \frac{142mm^2 \cdot 395MPa}{2.70MPa \cdot 207mm \cdot 762mm} - 0.50$$

=0.819

$$\frac{V_f}{f_c^{'} \cdot b_v \cdot d_v} = \frac{200000N}{45.4MPa \cdot 207mm \cdot 475mm}$$ Eqn. (3.8)

=0.045

Based on the normalized shear stress and the use of Figure 2.1, the maximum allowable stirrup spacing $s_{m1}$ is determined. As discussed in Section 4.5.1 this study recommends neglecting the $s_{m2}$ stirrup spacing limit.

$s_{m1}$ = 356 mm

Because $s > s_{m1}$ and $A_{v,min} > A_v$ S6-06 Clause 8.9.3.6 requires that the assumed inclined crack spacing be take equal to the shear depth.

$s_z = d_v$ = 475 mm

$$s_{ze} = 35 \cdot \frac{s_z}{15 + a_g} = 35 \cdot \frac{475mm}{15 + 20mm} \qquad \text{Eqn. (3.11)}$$

= 475 mm

$$\varepsilon_x = \frac{\frac{M_f}{d_v} + V_f - V_p + 0.5 \cdot N_f - A_p \cdot f_{po}}{2 \cdot \left(E_s \cdot A_s + A_p \cdot E_p + E_c \cdot A_{ct}\right)} \qquad \text{Eqn. (3.10)}$$

$$= \frac{\frac{365 \times 10^6 N \cdot mm}{475mm} + 200 \times 10^3 N}{2 \cdot \left(200000MPa \cdot 2904mm^2\right)}$$

=0.00832

$$\beta = \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = \frac{0.40}{1 + 1500 \cdot 0.000832} \cdot \frac{1300}{1000 + 475mm} \qquad \text{Eqn. (3.5)}$$

=0.157

$$\theta = \left(29 + 7000 \cdot \varepsilon_x\right) \cdot \left(0.88 + \frac{s_{ze}}{2500}\right) \qquad \text{Eqn. (3.7)}$$

$$= (29 + 7000 \cdot 0.00832) \cdot \left(0.88 + \frac{475mm}{2500}\right)$$

$=37.3^{\circ}$

The concrete contribution to shear capacity used by the modified shear method S6-06 F is equal to the sum of the web contribution and the flange contribution. These components of the concrete shear area are discussed in Section 5.3.1.

$$A_{web} = b_v \cdot d_v = 207mm \cdot 475mm$$

$=98\ 366\ mm^2$

$$A_{flange} = (t_f - (d - d_v)) \cdot \min\begin{Bmatrix} t_f - (d - d_v) \\ or \\ \dfrac{b - b_v}{2} \end{Bmatrix} \qquad \text{Eqn. (5.2)}$$

$$= (102mm - (528mm - 475mm)) \cdot \min\begin{Bmatrix} 102mm - (528mm - 475mm) \\ or \\ \dfrac{456mm - 207mm}{2} \end{Bmatrix}$$

$=2\ 421\ mm^2$

$A_{cv} = A_{web} + A_{flange} = 98\ 325\ mm^2 + 2\ 401\ mm^2$ Eqn. (5.1)

$=100\ 787\ mm^2$

The sectional shear capacity calculated using the modified shear method S6-06 F is then determined using Eqn (5.3).

$$V_r = 2.5 \cdot \beta \cdot f_{cr} \cdot A_{cv} + \gamma \cdot \frac{A_v \cdot f_v \cdot d_v}{s \cdot \tan\theta} \qquad \text{Eqn. (5.3)}$$

$$= 2.5 \cdot 0.157 \cdot 2.70MPa \cdot 100787mm^2 + 0.819 \cdot \frac{142mm^2 \cdot 395MPa \cdot 475mm}{762mm \cdot \tan(37.3^{\circ})}$$

$= 106\ kN + 38\ kN$

=144 kN

Because the first iteration of shear demand $V_f$ = 200 kN does not equal $V_r$ the predicted shear capacity has not been properly converged on. The next iteration is taken as the average of the $V_f$ and $V_r$ from the previous iteration. Therefore

$V_{r,n+1}$ is taken as $\frac{214kN + 143kN}{2} = 179kN$ .

Table B.2 provides the iterations of applied loads until a converged shear capacity is achieved. The term $V_f$ represents the shear demand resulting from both the externally applied load and the self-weight at a section $d_v$ away from the externally applied load.

Table B.2 – Evaluation of PE1 – S6-06 F

| Iteration | $V_f$ (kN) | $s_{ze}$ (mm) | $\frac{v}{\phi_c \cdot f_c'}$ | $\varepsilon_x$ x $10^6$ | $V_c$ (kN) | $V_s$ (kN) | $V_r$ (kN) |
|---|---|---|---|---|---|---|---|
| 1 | 200 | 475 | 0.045 | 832 | 106 | 38 | 144 |
| 2 | 172 | 475 | 0.039 | 721 | 115 | 39 | 154 |
| 3 | 163 | 475 | 0.036 | 683 | 118 | 39 | 158 |
| n | 159 | 475 | 0.036 | 667 | 120 | 39 | 159 |

The converged shear capacity of Specimen PE 1 calculated using the modified method in S6-06 F is 159 kN. This compares appropriately well to the tested shear capacity of 200 kN (see Table 4.1).

**Sample Calculations 2 – Non-Compliant Non-Prestressed Member – YB 2000/4 (Angelakos, 1999)**

| | | |
|---|---|---|
| $f_c^{'}$ = 36.4 MPa | $A_s$ = 4200 mm$^2$ | $f_y$ = 447 MPa |
| $d$ = 1980 mm | $\rho$ = 0.0074 | $f_v$ = 468 MPa |
| $b$ = 300 mm | $a_g$ = 10 mm | $A_v$ = 127 mm$^2$ |
| $b_v$ = 300 mm | $d_v$ = 1701 mm | $a/d$ = 2.86 |
| $s$ = 590 mm | $(M/V)$ = 3.70 | $V_{test}$ = 674 kN |
| $L$ = 10800 mm | $w_D$ = 14.4 kN/m | $f_{cr}$ = 2.41 MPa |

All load and resistance factors are taken as unity and thus are not shown in the calculations.

**CSA S6-06 Sectional Shear Method**

Iteration 1 – Loads are taken at a distance $d_v$ away from the externally applied load.

$V_{applied}$ = 500 kN

$M_{applied}$= 1850 kN-m

$V_D$ = 24 kN

$M_D$ = 189 kN-m

Determine S6-06 Section 14 minimum stirrup spacing and area requirements.

$$A_v = 0.15 \cdot f_{cr} \cdot \frac{b_v \cdot s}{f_v} = 0.15 \cdot 2.41MPa \cdot \frac{300mm \cdot 590mm}{468MPa} \qquad \text{Eqn. (2.1)}$$

=137 mm$^2$

$$\gamma = 10 \cdot \frac{A_v \cdot f_v}{f_{cr} \cdot b_v \cdot s} - 0.5 \qquad \text{where } 0 \le \gamma \le 1.0 \qquad \text{Eqn. (3.12)}$$

$$= 10 \cdot \frac{127mm^2 \cdot 468MPa}{2.41MPa \cdot 300mm \cdot 590mm} - 0.50$$

=0.891

$$\frac{V_f}{f_c^{'} \cdot b_v \cdot d_v} = \frac{524000N}{37.7MPa \cdot 300mm \cdot 1701mm} \qquad \text{Eqn. (3.8)}$$

=0.027

Based on the normalized shear stress and the use of Figure 2.1, the maximum allowable stirrup spacing $s_{m1}$ is determined. As discussed in Section 4.5.1 this study recommends neglecting the $s_{m2}$ stirrup spacing limit.

$s_{m1}$ = 600 mm

Because $s > s_{m1}$ and $A_{v,min} > A_v$ S6-06 Section 8 requires that the assumed inclined crack spacing be take equal to the shear depth.

$s_z = d_v$ = 1701 mm

$$s_{ze} = 35 \cdot \frac{s_z}{15 + a_g} = 35 \cdot \frac{1701mm}{15 + 10mm} \qquad \text{Eqn. (3.11)}$$

= 2381 mm

There is no prestressing component and no applied tensile loading so the $A_{ct}$, $A_p$ and $N_f$ terms in Eqn. (3.10) are all taken as zero.

$$\varepsilon_x = \frac{\frac{M_f}{d_v} + V_f - V_p + 0.5 \cdot N_f - A_p \cdot f_{po}}{2 \cdot \left(A_s \cdot E_s + A_p \cdot E_p + A_{ct} \cdot E_c\right)} \qquad \text{Eqn. (3.10)}$$

$$= \frac{\frac{2039 \times 10^6 N \cdot mm}{1701mm} + 524 \times 10^3 N}{2 \cdot \left(200000MPa \cdot 4200mm^2\right)}$$

=0.001026

$$\beta = \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = \frac{0.40}{1 + 1500 \cdot 0.001026} \cdot \frac{1300}{1000 + 2381mm} \qquad \text{Eqn. (3.5)}$$

=0.0606

$$\theta = (29 + 7000 \cdot \varepsilon_x) \cdot \left(0.88 + \frac{s_{ze}}{2500}\right) \qquad \text{Eqn. (3.7)}$$

$$= (29 + 7000 \cdot 0.001026) \cdot \left(0.88 + \frac{2381mm}{2500}\right)$$

$=66.3^{\circ}$

$$V_c = 2.5 \cdot \beta \cdot f_{cr} \cdot b_v \cdot d_v = 2.5 \cdot 0.0606 \cdot 2.41MPa \cdot 300mm \cdot 1701mm \qquad \text{Eqn. (3.2)}$$

= 187 kN

$$V_s = \gamma \cdot \frac{A_v \cdot f_v \cdot d_v}{s \cdot \tan\theta} = 0.891 \cdot \frac{127mm^2 \cdot 468MPa \cdot 1701mm}{590mm \cdot \tan(66.3^{\circ})} \qquad \text{Eqn. (3.3)}$$

=67 kN

$$V_r = V_c + V_s = 187kN + 67kN \qquad \text{Eqn. (3.1)}$$

=254 kN

Because the first iteration of shear demand $V_f$ = 524 kN does not equal $V_r$ the predicted shear capacity has not been properly converged on. The next iteration is taken as the average of the $V_f$ and $V_r$ from the previous iteration. Therefore

$V_f$ is taken as $\dfrac{524kN + 254kN}{2} = 389kN$

Table B.3 provides the iterations of applied loads until a converged shear capacity is achieved. The term $V_f$ represents the shear demand resulting from both the externally applied load and the self-weight at a section $d_v$ away from the externally applied load.

Table B.3 – Evaluation of Y2000/4 – S6-06

| Iteration | $V_f$ (kN) | $s_{ze}$ (mm) | $\frac{v}{\phi_c \cdot f_c'}$ | $\varepsilon_x$ x $10^6$ | $V_c$ (kN) | $V_s$ (kN) | $V_r$ (kN) |
|---|---|---|---|---|---|---|---|
| 1 | 524 | 2381 | 0.028 | 1026 | 187 | 67 | 254 |
| 2 | 389 | 2381 | 0.021 | 771 | 220 | 78 | 297 |
| 3 | 343 | 2381 | 0.018 | 684 | 234 | 82 | 315 |
| n | 323 | 2381 | 0.017 | 648 | 240 | 83 | 323 |

The converged shear capacity of Specimen Y2000/4 calculated using the sectional method in S6-06 Section 8 is 323 kN. This compares extremely conservatively to the tested shear capacity of 674 kN (see Table 4.1).

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**ACI 318-08 Sectional Shear Method**

$$s_{\max} = 0.5 \cdot d = 0.5 \cdot 1890mm \leq 600mm$$

= 600 mm

$$A_v = 0.06 \cdot \sqrt{f_c'} \cdot \frac{b_v \cdot s}{f_v} = 0.06 \cdot \sqrt{36.4}MPa \cdot \frac{300mm \cdot 590mm}{468MPa} \qquad \text{Eqn. (2.5)}$$

= 137 mm$^2$

$$V_c = \frac{1}{6} \cdot \sqrt{f_c'} \cdot b_v \cdot d = \frac{1}{6} \cdot \sqrt{36.4}MPa \cdot 300mm \cdot 1890mm \qquad \text{Eqn. (3.27)}$$

=570 kN

$$V_s = \frac{A_v \cdot f_v \cdot d}{s} = \frac{127mm^2 \cdot 468MPa \cdot 1890mm}{590mm} \qquad \text{Eqn. (3.29)}$$

= 190 kN

$$V_r = V_c + V_s = 592kN + 190kN \qquad \text{Eqn. (3.25)}$$

= 761 kN

ACI 318-08 predicts the shear capacity of Specimen YB2000/4 to be 761 kN which compares appropriately well to the tested shear capacity of 674 kN (see Table 4.1).

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**S6-06 M Sectional Shear Method**

Iteration 1 – Loads taken at $d_v$ away from the externally applied load.

$V_{applied}$ = 550 kN

$M_{applied}$ = 2035 kN-m

$V_D$ = 24 kN

$M_D$ = 189 kN-m

$$A_v = 0.15 \cdot f_{cr} \cdot \frac{b_v \cdot s}{f_v} = 0.15 \cdot 2.41MPa \cdot \frac{300mm \cdot 590mm}{468MPa} \qquad \text{Eqn (2.1)}$$

=137 mm$^2$

$$\gamma = 10 \cdot \frac{A_v \cdot f_v}{f_{cr} \cdot b_v \cdot s} - 0.50 \qquad \text{where } 0 \le \gamma \le 1.0 \qquad \text{Eqn. (3.12)}$$

$$= 10 \cdot \frac{127mm^2 \cdot 468MPa}{2.41MPa \cdot 300mm \cdot 590mm} - 0.50$$

=0.891

The modified shear method S6-06 M uses Eqn. (3.8) to determine normalized shear demand. For evaluation of specimen YB 2000/4 there was no prestressing and therefore the term $V_p$ was taken as zero.

$$\frac{V_f}{f_c^{'} \cdot b_v \cdot d_v} = \frac{574000N}{36.4MPa \cdot 300mm \cdot 1701mm} \qquad \text{Eqn. (3.8)}$$

=0.031

The modified shear method S6-06 M uses Figure 5.1 to determine maximum permissible stirrup spacing in the longitudinal direction. Based on the sectional geometry and the normalized shear demand the maximum stirrup spacing $s_{m1}$ is:

$s_{m1}$ = 600 mm

S6-06 M makes the assumptions that diagonal crack spacing $s_z$ is equal to the longitudinal spacing of stirrups therefore:

$s_z$ = s = 590 mm

$$s_{ze} = 35 \cdot \frac{s_z}{15 + a_g} \geq 0.85 \cdot s_z = 35 \cdot \frac{590mm}{15 + 10mm} \geq 0.85 \cdot 590mm \qquad \text{Eqn. (3.11)}$$

= 826 mm

Consistent with evaluation using the sectional shear method in S6-06 Section 8, the modified shear method S6-06 M determines the longitudinal strain at mid-depth using Eqn. (3.10). Specimen YB 2000/4 was not prestressed and had no applied axial tension, therefore the terms $V_p$, $A_p{\cdot}f_{po}$, $N_f$, and $A_{ct}$ were taken as zero.

$$\varepsilon_x = \frac{\frac{M_f}{d_v} + \left(V_f - V_p\right) + N_f - A_p \cdot f_{po}}{2 \cdot \left(A_s \cdot E_s + A_p \cdot E_p + A_{ct} \cdot E_c\right)} = \frac{\frac{2224 \times 10^6 N \cdot mm}{1701mm} + 574 \times 10^3 N}{2 \cdot \left(200000MPa \cdot 4200mm^2\right)} \qquad \text{Eqn. (3.10)}$$

=0.00112

The shear terms $\beta$ and $\theta$ were calculated using Eqn. (3.5) and Eqn (3.7) respectively.

$$\beta = \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = \frac{0.40}{1 + 1500 \cdot 0.00112} \cdot \frac{1300}{1000 + 826mm} \qquad \text{Eqn (3.5)}$$

=0.106

$$\theta = (29 + 7000 \cdot \varepsilon_x) \cdot \left(0.88 + \frac{s_{ze}}{2500}\right) \qquad \text{Eqn. (3.7)}$$

$$= (29 + 7000 \cdot 0.00112) \cdot \left(0.88 + \frac{826mm}{2500}\right)$$

$$=44.6^{\circ}$$

S6-06 M calculates the shear capacity attributed to the concrete and the stirrups using Eqn. (3.2) and Eqn. (3.3) respectively. This study used the proposed Eqn. (3.12) to accommodate the interpolation of effective stirrup area required by S6-06 Clause 14.14.1.6.2.

$$V_c = 2.5 \cdot \beta \cdot f_{cr} \cdot b_v \cdot d_v = 2.5 \cdot 0.106 \cdot 2.41MPa \cdot 300mm \cdot 1701mm \qquad \text{Eqn. (3.2)}$$

= 327 kN

$$V_s = \gamma \cdot \frac{A_v \cdot f_v \cdot d_v}{s \cdot \tan\theta} = 0.891 \cdot \frac{127mm^2 \cdot 468MPa \cdot 1701mm}{590mm \cdot \tan(44.6^{\circ})} \qquad \text{Eqn. (3.3)}$$

= 155 kN

S6-06 M determines the sectional shear capacity using Eqn. (3.1).

$$V_r = V_c + V_r + V_p = 327kN + 155kN \qquad \text{Eqn. (3.1)}$$

= 482 kN

Because the first iteration of shear demand $V_f$ = 574 kN does not equal $V_r$ the predicted shear capacity has not been properly converged on. The next iteration is taken as the average of the $V_f$ and $V_r$ from the previous iteration. Therefore

$V_{f,n+1}$ is taken as $\dfrac{574kN + 482kN}{2} = 528kN$.

Table B.4 provides the iterations of applied loads until a converged shear capacity is achieved. The term $V_f$ represents the shear demand resulting from both the externally applied load and the self-weight at a section $d_v$ away from the externally applied load.

Table B.4 – Evaluation of YB2000/4 – S6-06 M

| Iteration | $V_f$ (kN) | $s_{ze}$ (mm) | $\frac{v_f}{\phi_c \cdot f_c^{'}}$ | $\varepsilon_x$ x $10^6$ | $V_c$ (kN) | $V_s$ (kN) | $V_r$ (kN) |
|---|---|---|---|---|---|---|---|
| 1 | 574 | 590 | 0.031 | 1120 | 327 | 155 | 482 |
| 2 | 528 | 590 | 0.029 | 1033 | 344 | 159 | 503 |
| 3 | 516 | 590 | 0.028 | 1009 | 349 | 160 | 509 |
| n | 511 | 590 | 0.028 | 1001 | 351 | 161 | 511 |

The converged shear capacity of Specimen YB2000/4 calculated using the modified method in S6-06 F is 511 kN. This compares appropriately well to the tested shear capacity of 674 kN (see Table 4.1).

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**S6-06 F Sectional Shear Analysis is the same as S6-06 M Sectional Shear Analysis because the member is a rectangular section.**

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**Sample Calculations 3 – Non-Compliant Prestressed Member**

**NL – 10 - 240 (Bennett and Debaiky, 1974)**

$f_c^{'} = 39.4$ MPa
$d = 298$ mm
$\rho = 0.0136$
$b = 152$ mm
$b_v = 51$ mm
$s = 240$ mm
$L = 3660$ mm
$t_{bot} = 57$ mm
h = 330 mm

$A_s = 284$ mm$^2$
$A_p = 231$ mm$^2$
$f_v = 280$ MPa
$a_g = 20$ mm
$d_v = 268$ mm
$(M/V) = 0.632$
$w_D = 0.68$ kN/m
$f_{cr} = 2.51$ MPa

$f_y = 410$ MPa
$A_{ct} = 8\ 400$ mm$^2$
$E_c = 27\ 427$ MPa
$A_v = 71$ mm$^2$
$a/d = 3.00$
$V_{test} = 94$ kN
$f_{pu} = 1720$ MPa
$f_{pe} = 774$ MPa

All load and resistance factors are taken as unity and thus are not shown in the calculations.

---

**CSA S6-06 Sectional Shear Method**

Iteration 1 – Loads taken at $d_v$ away from the externally applied load.

$V_{applied}$ = 70 kN

$M_{applied}$ = 44 kN-m

$V_D$ = 1 kN

$M_D$ = 1 kN-m

$$A_v = 0.15 \cdot f_{cr} \cdot \frac{b_v \cdot s}{f_v} = 0.15 \cdot 2.51 MPa \cdot \frac{51 mm \cdot 240 mm}{280 MPa} \qquad \text{Eqn.(2.1)}$$

=16 mm$^2$

$$\gamma = 10 \cdot \frac{A_v \cdot f_v}{f_{cr} \cdot b_v \cdot s} - 0.5 \qquad \text{where } 0 \le \gamma \le 1.0 \qquad \text{Eqn. (3.12)}$$

$$10 \cdot \frac{71 mm^2 \cdot 280 MPa}{2.51 MPa \cdot 51 mm \cdot 240 mm} - 0.50$$

= 1.00

The normalized shear demand is determined using Eqn. (3.8). As discussed in Section 3.2 normalized shear demand is used to determine the maximum permissible stirrup spacing $s_{m1}$.

$$\frac{V_f}{f_c^{'} \cdot b_v \cdot d_v} = \frac{71000 N}{39.4 MPa \cdot 51 mm \cdot 268 mm} \qquad \text{Eqn. (3.8)}$$

=0.131

Based on the sectional geometry and the normalized shear demand the maximum stirrup spacing $s_{m1}$ is determined using Figure 2.1:

$s_{m1}$= 166 mm

Because $s > s_{m1}$ S6-06 Clause 8.9.3.6 requires that the assumed diagonal crack spacing term be take equal to the shear depth.

$s_z = d_v$ = 268 mm

The effective crack spacing $s_{ze}$ is determined using Eqn. (3.11):

$$s_{ze} = 35 \cdot \frac{s_z}{15 + a_g} \geq 0.85 \cdot s_z = 35 \cdot \frac{268mm}{15 + 20mm} \geq 0.85 \cdot 268mm \qquad \text{Eqn. (3.11)}$$

= 268 mm

The longitudinal strain at mid-depth is determined using Eqn. (3.10). The prestressing used in specimen NL–10–240 was not harped, so the $V_p$ term was taken as zero. As discussed in Section 3.2, $f_{po}$ is calculated as $0.70 \cdot f_{pu}$ in accordance with S6-06 Clause 8.9.3.8 (d).

$$\varepsilon_x = \frac{\frac{M_f}{d_v} + V_f + N_f - A_p \cdot f_{po}}{2 \cdot \left(E_s \cdot A_s + E_p \cdot A_p + A_{ct} \cdot E_c\right)} \qquad \text{Eqn. (3.10)}$$

$$\varepsilon_x = \frac{\frac{45 \times 10^6 N \cdot mm}{268mm} + 71 \times 10^3 N - 291mm^2 \cdot 0.7 \cdot 1720MPa}{2 \cdot \left(200000MPa \cdot 284mm^2 + 200000MPa \cdot 231mm^2 + 8400mm^2 \cdot 27427MPa\right)}$$

= -0.000 060

The shear terms $\beta$ and $\theta$ were calculated using Eqn. (3.5) and Eqn (3.7) respectively.

$$\beta = \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = \frac{0.40}{1 + 1500 \cdot -0.000060} \cdot \frac{1300}{1000 + 268mm} \qquad \text{Eqn. (3.5)}$$

=0.451

$$\theta = \left(29 + 7000 \cdot \varepsilon_x\right) \cdot \left(0.88 + \frac{s_{ze}}{2500}\right) \qquad \text{Eqn. (3.7)}$$

$$= \left(29 + 7000 \cdot -0.000060\right) \cdot \left(0.88 + \frac{268mm}{2500}\right)$$

$= 28.2^{\circ}$

S6-06 Section 8 calculates the shear capacity attributed to the concrete and the stirrups using Eqn. (3.2) and Eqn. (3.3) respectively. This study uses the proposed Eqn. (3.12) to accommodate the interpolation of effective stirrup area required by S6-06 Clause 14.14.1.6.2.

$$V_c = 2.5 \cdot \beta \cdot f_{cr} \cdot b_v \cdot d_v = 2.5 \cdot 0.451 \cdot 2.51 MPa \cdot 51 mm \cdot 268 mm \qquad \text{Eqn. (3.2)}$$

= 39 kN

$$V_s = \gamma \cdot \frac{A_v \cdot f_v \cdot d_v}{s \cdot \tan\theta} = 1.00 \cdot \frac{71 mm^2 \cdot 280 MPa \cdot 268 mm}{240 mm \cdot \tan(28.2^\circ)} \qquad \text{Eqn. (3.3)}$$

= 41 kN

S6-06 determines the sectional shear capacity using Eqn. (3.1).

$$V_r = V_c + V_s + V_p = 39 kN + 41 kN \qquad \text{Eqn. (3.1)}$$

= 80 kN

Because the first iteration of shear demand $V_f$ = 71 kN does not equal $V_r$ the predicted shear capacity has not been properly converged on. The next iteration is taken as the average of the $V_f$ and $V_r$ from the previous iteration. Therefore

$V_{f,n+1}$ is taken as $\frac{71kN + 80kN}{2} = 76kN$

Table B.5 provides the iterations of applied loads until a converged shear capacity is achieved. The term $V_f$ represents the shear demand resulting from both the externally applied load and the self-weight at a section $d_v$ away from the externally applied load.

Table B.5 – Evaluation of NL–10–240 – S6-06

| Iteration | $V_f$ (kN) | $s_{ze}$ (mm) | $\frac{v}{\phi_c \cdot f_c'}$ | $\varepsilon_x$ x $10^6$ | $V_c$ (kN) | $V_s$ (kN) | $V_r$ (kN) |
|---|---|---|---|---|---|---|---|
| 1 | 71 | 268 | 0.131 | -60 | 39 | 41 | 80 |
| 2 | 76 | 268 | 0.141 | -35 | 37 | 41 | 78 |
| n | 78 | 268 | 0.144 | -26 | 37 | 41 | 78 |

The converged shear capacity of Specimen NL-10-240 calculated using the sectional method in S6-06 Section 8 is 78 kN. This compares appropriately well to the tested shear capacity of 94 kN (see Table 4.1).

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**ACI 318-08 Sectional Shear Method**

Iteration 1 – Sectional forces taken at $d_v$ away from the externally applied load.

$V_D$ = 1 kN
$M_D$ = 1 kN-m
$V_{applied}$ = 80 kN
$M_{applied}$ = 24 kN-m

$$s_{\max} = 0.75 \cdot h = 0.75 \cdot 300mm \leq 600mm$$
= 248 mm

$$A_v = 0.06 \cdot \sqrt{f_c^{'}} \cdot \frac{b_v \cdot s}{f_v} = 0.06 \cdot \sqrt{39.4}MPa \cdot \frac{51mm \cdot 240mm}{280MPa} \qquad \text{Eqn. (2.5)}$$
= 16 mm$^2$

For prestressing members the sectional shear method in ACI 318-08 determines the shear capacity attributed to the concrete using Eqn. (3.28). It should be noted that specimen NL–10–240 has an effective prestressing force after losses which was greater than $0.40 \cdot f_{pu}$. The sectional shear capacity attributed to the stirrups is calculated using Eqn. (3.29).

$$V_c = \left(0.05 \cdot \sqrt{f_c^{'}} + 5 \cdot \frac{V_f \cdot d}{M_f}\right) \cdot b_v \cdot d \leq 0.40 \cdot \sqrt{f_c^{'}} \cdot b_v \cdot d \qquad \text{Eqn. (3.28)}$$

$$V_c = \left(0.05 \cdot \sqrt{39.4}MPa + 5 \cdot \frac{81 \times 10^3 N \cdot 298mm}{51 \times 10^6 N \cdot mm}\right) \cdot 51mm \cdot 298mm$$
$$\leq 0.4 \cdot \sqrt{39.4}MPa \cdot 51mm \cdot 298mm$$
=41 kN > 38 kN

$$V_s = \frac{A_v \cdot f_v \cdot d}{s} = \frac{71mm^2 \cdot 280MPa \cdot 298mm}{240mm} \qquad \text{Eqn. (3.29)}$$

= 25 kN

ACI 318-08 determines the sectional shear capacity using Eqn. (3.25).

$$V_r = V_c + V_s = 38kN + 25kN \qquad \text{Eqn. (3.25)}$$

= 63 kN

Table B.6 provides the iterations of applied loads until a converged shear capacity is achieved. The term $V_f$ represents the shear demand resulting from both the externally applied load and the self-weight at a section $d_v$ away from the externally applied load.

Table B.6 – Evaluation of NL–10–240 – ACI 318-08

| Iteration | $V_{r,n-1}$ (kN) | $V_c$ (kN) | $V_s$ (kN) | $V_r$ (kN) |
|---|---|---|---|---|
| 1 | 81 | 38 | 25 | 63 |
| 2 | 72 | 38 | 25 | 63 |
| n | 63 | 38 | 25 | 63 |

ACI 318-08 predicts the shear capacity of Specimen NL-10-240 1 to be 63 kN which compares conservatively to the tested shear capacity of 94 kN (see Table 4.1).

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**S6-06 M Sectional Shear Method**

Iteration 1 – Sectional forces taken at $d_v$ away from the externally applied load.

$V_D$ = 1 kN

$M_D$ = 1 kN-m

$V_{applied}$ = 70 kN

$M_{applied}$ = 44 kN-m

$$A_v = 0.15 \cdot f_{cr} \cdot \frac{b_v \cdot s}{f_v} = 0.15 \cdot 2.51MPa \cdot \frac{51mm \cdot 240mm}{280MPa} \qquad \text{Eqn. (2.1)}$$

= 16 mm$^2$

$$\gamma = 10 \cdot \frac{A_v \cdot f_v}{f_{cr} \cdot b_v \cdot s} - 0.5 \qquad \text{where } 0 \leq \gamma \leq 1.0 \qquad \text{Eqn. (3.12)}$$

$$= 10 \cdot \frac{71mm^2 \cdot 280MPa}{2.51MPa \cdot 51mm \cdot 240mm} - 0.50$$

= 1.00

The normalized shear demand is determined using Eqn. (3.8). As discussed in Section 3.2 normalized shear demand is used to determine the maximum permissible stirrup spacing $s_{m1}$.

$$\frac{V_f}{f_c^{'} \cdot b_v \cdot d_v} = \frac{71000N}{39.4MPa \cdot 51mm \cdot 268MPa} \qquad \text{Eqn. (3.8)}$$

=0.131

Based sectional geometry and the normalized shear demand the maximum stirrup spacing $s_{m1}$ is determined using Figure 2.1:

$s_{m1}$=166 mm

S6-06 M makes the assumption that diagonal crack spacing $s_z$ is equal to the longitudinal spacing of stirrups therefore:

$s_z$ = s = 240 mm

$$s_{ze} = 35 \cdot \frac{s_z}{15 + a_g} \geq 0.85 \cdot s_z = 35 \cdot \frac{240mm}{15 + 20mm} \geq 0.85 \cdot 240mm \qquad \text{Eqn. (3.11)}$$

= 240 mm

The longitudinal strain at mid-depth is determined using Eqn. (3.10). The prestressing used in specimen NL–10–240 was not harped, so the $V_p$ term was taken as zero.

$$\varepsilon_x = \frac{\frac{M_f}{d_v} + V_f + N_f - A_p \cdot f_{po}}{2 \cdot \left( E_s \cdot A_s + E_p \cdot A_p + A_{ct} \cdot E_c \right)} \qquad \text{Eqn. (3.10)}$$

$$\varepsilon_x = \frac{\frac{45 \times 10^6 N \cdot mm}{268mm} + 71 \times 10^3 N - 231mm^2 \cdot 0.7 \cdot 1720MPa}{2 \cdot \left( 200000MPa \cdot 284mm^2 + 200000MPa \cdot 231mm^2 + 8400mm^2 \cdot 27428MPa \right)}$$

= -0.000 060

S6-06 M determines the shear terms $\beta$ and $\theta$ using Eqn. (3.5) and Eqn. (3.7) respectively.

$$\beta = \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = \frac{0.40}{1 + 1500 \cdot -0.000060} \cdot \frac{1300}{1000 + 240mm} \qquad \text{Eqn. (3.5)}$$

= 0.461

$$\theta = \left( 29 + 7000 \cdot \varepsilon_x \right) \cdot \left( 0.88 + \frac{s_{ze}}{2500} \right) \qquad \text{Eqn. (3.7)}$$

$$= \left( 29 + 7000 \cdot -0.000060 \right) \cdot \left( 0.88 + \frac{240mm}{2500} \right)$$

= 27.9°

S6-06 M calculates the shear capacity attributed to the concrete and the stirrups using Eqn. (3.2) and Eqn. (3.3) respectively. This study used the proposed Eqn. (3.12) to accommodate the interpolation of effective stirrup area required by S6-06 Clause 14.14.1.6.2.

$$V_c = 2.5 \cdot \beta \cdot f_{cr} \cdot b_v \cdot d_v = 2.5 \cdot 0.461 \cdot 2.51MPa \cdot 51mm \cdot 268mm \qquad \text{Eqn. (3.2)}$$

= 40 kN

$$V_s = \gamma \cdot \frac{A_v \cdot f_v \cdot d_v}{s \cdot \tan\theta} = 1.00 \cdot \frac{71mm^2 \cdot 280MPa \cdot 268mm}{240mm \cdot \tan(27.9^{\circ})} \qquad \text{Eqn. (3.3)}$$

= 42 kN

S6-06 determines the sectional shear capacity using Eqn. (3.1).

$$V_r = V_c + V_s = 40kN + 42kN \qquad \text{Eqn. (3.1)}$$

= 82 kN

Because the first iteration of shear demand $V_f$ = 71 kN does not equal $V_r$ the predicted shear capacity has not been properly converged on. The next iteration is taken as the average of the $V_f$ and $V_r$ from the previous iteration. Therefore

$V_{r,n-1}$ is taken as $\dfrac{71kN + 82kN}{2} = 77kN$

Table B.7 provides the iterations of applied loads until a converged shear capacity is achieved. The term $V_f$ represents the shear demand resulting from both the externally applied load and the self-weight at a section $d_v$ away from the externally applied load.

Table B.7 – Evaluation of NL – 10 – 240 – S6-06 M

| Iteration | $V_{r,n-1}$ (kN) | $s_{ze}$ (mm) | $\dfrac{v}{\phi_c \cdot f_c^{'}}$ | $\varepsilon_x$ x $10^6$ | $V_c$ (kN) | $V_s$ (kN) | $V_{r,n}$ (kN) |
|---|---|---|---|---|---|---|---|
| 1 | 71 | 240 | 0.131 | -60 | 40 | 42 | 82 |
| 2 | 77 | 240 | 0.143 | -30 | 38 | 42 | 79 |
| n | 79 | 240 | 0.144 | -25 | 37 | 42 | 79 |

The converged shear capacity of Specimen NL-10-240 calculated using the modified method in S6-06 Section 8 is 79 kN. This compares appropriately well to the tested shear capacity of 94 kN (see Table 4.1).

---

**S6-06 F Sectional Shear Method**

Iteration 1 – Sectional forces taken at $d_v$ away from the externally applied load.

$V_D = 1$ kN

$M_D = 1$ kN-m

$V_{applied} = 70$ kN

$M_{applied} = 44$ kN-m

$$A_v = 0.15 \cdot f_{cr} \cdot \frac{b_v \cdot s}{f_v} = 0.15 \cdot 2.51 MPa \cdot \frac{51mm \cdot 240mm}{280MPa} \qquad \text{Eqn. (2.1)}$$

$= 16$ mm$^2$

$$\gamma = 10 \cdot \frac{A_v \cdot f_v}{f_{cr} \cdot b_v \cdot s} - 0.5 \qquad \text{where } 0 \leq \gamma \leq 1.0 \qquad \text{Eqn. (3.12)}$$

$$= 10 \cdot \frac{71mm^2 \cdot 280MPa}{2.51MPa \cdot 51mm \cdot 240mm} - 0.50$$

$= 1.00$

The normalized shear demand is determined using Eqn. (3.8). As discussed in Section 3.2 normalized shear demand is used to determine the maximum permissible stirrup spacing $s_{m1}$.

$$\frac{V_f}{f_c' \cdot b_v \cdot d_v} = \frac{71000N}{39.4MPa \cdot 51mm \cdot 268mm} \qquad \text{Eqn. (3.8)}$$

$=0.131$

Based sectional geometry and the normalized shear demand the maximum stirrup spacing $s_{m1}$ is determined using Figure 2.1:

$s_{m1}$=166 mm

S6-06 F makes the assumption that diagonal crack spacing $s_z$ is equal to the longitudinal spacing of stirrups therefore:

$s_z$ = s = 240 mm

$$s_{ze} = 35 \cdot \frac{s_z}{15 + a_g} \geq 0.85 \cdot s_z = 35 \cdot \frac{240mm}{15 + 20mm} \geq 0.85 \cdot 240mm \qquad \text{Eqn. (3.11)}$$

= 240 mm

The longitudinal strain at mid-depth is determined using Eqn. (3.10). The prestressing used in specimen NL–10–240 was not harped, so the $V_p$ term was taken as zero.

$$\varepsilon_x = \frac{\frac{M_f}{d_v} + V_f + N_f - A_p \cdot f_{po}}{2 \cdot \left(E_s \cdot A_s + E_p \cdot A_p + A_{ct} \cdot E_c\right)} \qquad \text{Eqn. (3.10)}$$

$$\varepsilon_x = \frac{\frac{45 \times 10^6 N \cdot mm}{268mm} + 70.8 \times 10^3 N - 231mm^2 \cdot 0.7 \cdot 1720MPa}{2 \cdot \left(200000MPa \cdot 284mm^2 + 200000MPa \cdot 231mm^2 + 8400mm^2 \cdot 27428MPa\right)}$$

= -0.000 060

S6-06 F determines the shear terms $\beta$ and $\theta$ using Eqn. (3.5) and Eqn. (3.7) respectively.

$$\beta = \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = \frac{0.40}{1 + 1500 \cdot -0.000060} \cdot \frac{1300}{1000 + 240mm} \qquad \text{Eqn. (3.5)}$$

= 0.461

$$\theta = \left(29 + 7000 \cdot \varepsilon_x\right) \cdot \left(0.88 + \frac{s_{ze}}{2500}\right) \qquad \text{Eqn. (3.7)}$$

$$= \left(29 + 7000 \cdot -0.000060\right) \cdot \left(0.88 + \frac{240mm}{2500}\right)$$

= 27.9°

The concrete contribution to shear capacity used by the modified shear method S6-06 F is equal to the sum of the web contribution and the flange contribution. These components of the concrete shear area are discussed in Section 5.3.1.

$$A_{web} = b_v \cdot d_v = 51mm \cdot 268mm$$

=13678 mm²

$$A_{flange} = 2 \cdot \left(t_f - (d - d_v)\right) \cdot \min \begin{Bmatrix} t_f - (d - d_v) \\ or \\ \dfrac{b - b_v}{2} \end{Bmatrix}$$ Eqn. (5.2)

$$= 2 \cdot \left(57mm - (298mm - 268mm)\right) \cdot \min \begin{Bmatrix} 57mm - (298mm - 268mm) \\ or \\ \dfrac{152mm - 51mm}{2} \end{Bmatrix}$$

=1480 mm$^2$

$A_{cv} = A_{web} + A_{flange} = 13678\ mm^2 + 1480\ mm^2$ Eqn. (5.1)

=15158 mm$^2$

S6-06 F calculates the shear capacity attributed to the concrete and the stirrups using Eqn. (5.3). This study used the proposed Eqn. (3.12) to accommodate the interpolation of effective stirrup area required by S6-06 Clause 14.14.1.6.2.

$$V_r = 2.5 \cdot \beta \cdot f_{cr} \cdot A_{cv} + \gamma \cdot \frac{A_v \cdot f_v \cdot d_v}{s \cdot \tan\theta}$$ Eqn. (5.3)

$$= 2.5 \cdot 0.461 \cdot 2.51 MPa \cdot 15158 mm^2 + 1.00 \cdot \frac{71 mm^2 \cdot 280 MPa \cdot 268 mm}{240 mm \cdot \tan 27.9^\circ}$$

= 86 kN

Because the first iteration of shear demand $V_f$ = 71 kN does not equal $V_r$ the predicted shear capacity has not been properly converged on. The next iteration is taken as the average of the $V_f$ and $V_r$ from the previous iteration. Therefore

$V_{f,n+1}$ is taken as $\dfrac{71kN + 86kN}{2} = 79kN$

Table B.8 provides the iterations of applied loads until a converged shear capacity is achieved. The term $V_f$ represents the shear demand resulting from both the externally applied load and the self-weight at a section $d_v$ away from the externally applied load.

Table B.8 – Evaluation of NL – 10 – 240 – S6-06 F

| Iteration | $V_{r,n-1}$ (kN) | $s_{ze}$ (mm) | $\frac{v}{\phi_c \cdot f_c'}$ | $\varepsilon_x$ x $10^6$ | $V_c$ (kN) | $V_s$ (kN) | $V_{r,n}$ (kN) |
|---|---|---|---|---|---|---|---|
| 1 | 71 | 240 | 0.131 | -60 | 44 | 42 | 86 |
| 2 | 77 | 240 | 0.144 | -25 | 41 | 42 | 83 |
| n | 82 | 240 | 0.152 | -5 | 40 | 42 | 82 |

The converged shear capacity of Specimen NL-10-240 calculated using the modified method in S6-06 F is 82 kN. This compares appropriately well to the tested shear capacity of 94 kN (see Table 4.1).

# Appendix C

## Shear Capacity Convergence Case Study

This case study presents the sectional shear evaluation of a 45 m simple span bridge. The superstructure consists of 4 type 'PO' girders spaced at 2.74 m on center, and constructed compositely with a 165 mm deck. Type 'PO' girder are prestressed I-shape girders. Figure C.1 shows a simplified cross section through the superstructure while Figure C.2 shows the dimensions of the 'PO' girders used in this bridge.

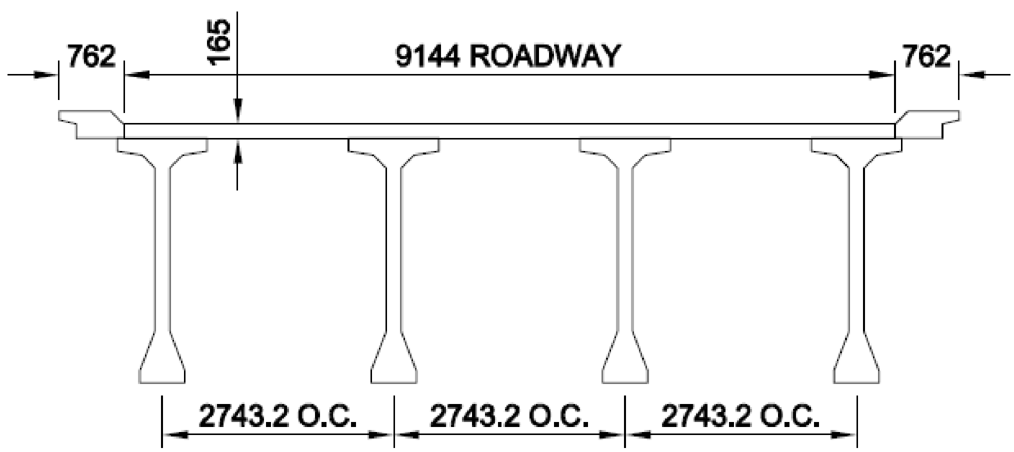


Figure C.1 – Cross section of Superstructure

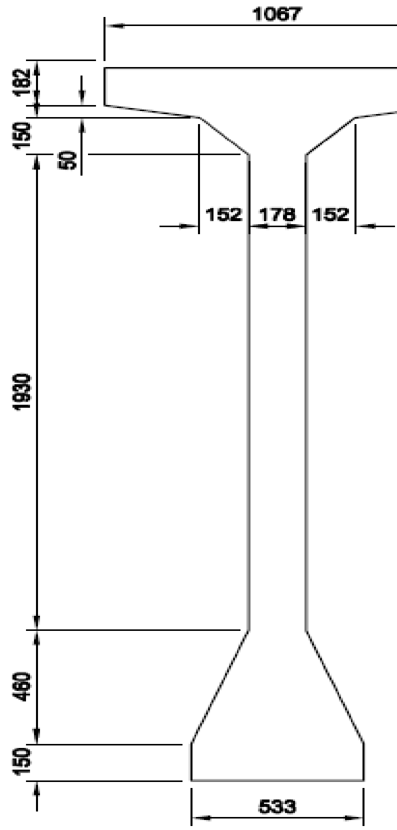


Figure C.2 – Cross Section of ‘PO’ Girder

**Loading**

Due to the fact that the deck was built compositely with the girders and the bridge had no skew, SECAN (Mufti et al., 1992) is a suitable program for determining the transverse distribution of loads among the girders. SECAN uses a semi-continuum method for determining the lateral distribution of load on each girder. Based on allowing the load to be offset by a maximum of 600 mm from the center of the bridge width, the maximum distribution for shear was calculated to be 44% of the loading configuration on a single girder.

Member Properties at Critical Section. The critical section of the girder was determined to be at x/L = 0.09 of the end of the span.

| | | |
|---|---|---|
| $f_c' = 34.7$ MPa | $f_{cr} = 2.36$ MPa | $w_D = 20.3$ kN/m |
| $d = 2436$ mm | $A_p = 6482$ mm$^2$ | $A_{ct} = 407\,518$ mm$^2$ |
| $L = 45\,000$ mm | $f_v = 276$ MPa | $E_c = 26\,192$ MPa |
| $b = 2743$ mm | $a_g = 20$ mm | $A_v = 400$ mm$^2$ |
| $b_v = 178$ mm | $d_v = 2204$ mm | $f_{pu} = 1720$ MPa |
| $s = 480$ mm | $t_{bot} = 165$ mm | $f_{pe} = 775$ MPa |
| h = 3061 mm | | Strand slope = 0.0400 |

Load and resistance factors are taken at unity and are not shown in the following calculations.

S6-06 Section 8 Sectional Shear Method – Iteration 1

$V_D = 596$ kN

$M_D = 2655$ kN-m

$V_{applied} = 509$ kN

$M_{applied} = 2034$ kN-m

Iterations in this case study are accomplished by varying the *externally applied* moments and shears at the critical section. It is important to note that the moment-to-shear ratio of the applied loads does not vary from iteration to iteration. $F$ is the live load capacity factor and is calculated as

$F = \dfrac{V_r - V_D}{V_{applied}}$ where $V_{applied}$ is the shear which results from the externally applied load.

$$V_f = V_D + F \cdot V_{applied} = 596kN + 1.00 \cdot 509kN$$

= 1105 kN

$$M_f = M_D + F \cdot M_{applied} = 2655kN \cdot m + 1.00 \cdot 2034kN \cdot m$$

=4689 kN-m

$$A_{v,\min} = 0.15 \cdot f_{cr} \cdot \frac{b_v \cdot s}{f_y} = 0.15 \cdot 2.36MPa \cdot \frac{178mm \cdot 480mm}{276MPa}$$ Eqn. (2.1)

= 110 mm$^2$

$$\gamma = 10 \cdot \frac{f_v \cdot A_v}{f_{cr} \cdot b_v \cdot s} - 0.5 = 10 \cdot \frac{276MPa \cdot 400mm^2}{2.36MPa \cdot 178mm \cdot 457mm} - 0.5$$ where $0 \leq \gamma \leq 1.0$

Eqn. (3.12)

= 1.00 This indicates that the 'PO' girders comply with S6-06 Section 14 stirrup area requirements.

Determine the normalized shear demand. As discussed in Section 3.2 normalized shear demand is used to determine the maximum permissible stirrup spacing $s_{m1}$.

$$\frac{V_f - V_p}{f_c^{'} \cdot b_v \cdot d_v} = \frac{1105000N - 201000N}{34.7MPa \cdot 178mm \cdot 2204mm}$$ Eqn. (3.8)

= 0.066

Based on the normalized shear stress and the use of Figure 2.1, the maximum allowable stirrup spacing $s_{m1}$ is determined. As discussed in Section 4.5.1 this study recommends neglecting the $s_{m2}$ stirrup spacing limit.

$$s_{m1} = 600mm$$

Because $s < s_{m1}$ and $A_{v,min} < A_v$ S6-06 Clause 8.9.3.7 requires that the assumed diagonal crack spacing term $s_z$ be take equal to 300 mm.

$s_z$ = 300 mm

The effective crack spacing $s_{ze}$ is determined using Eqn. (3.11):

$$s_{ze} = 35 \cdot \frac{s_z}{15 + a_g} \geq 0.85 \cdot s_z = 35 \cdot \frac{300mm}{15 + 20mm} > 0.85 \cdot 300mm \qquad \text{Eqn. (3.11)}$$

= 300 mm

The longitudinal strain at mid-depth is determined using Eqn. (3.10). It should be noted that the passive reinforcing steel was not considered at the section of interest.

$$\varepsilon_x = \frac{\frac{M_f}{d_v} + V_f + N_f - A_p \cdot f_{po}}{2 \cdot \left(E_s \cdot A_s + E_p \cdot A_p + A_{ct} \cdot E_c\right)} \qquad \text{Eqn. (3.10)}$$

$$\varepsilon_x = \frac{\frac{4689 \times 10^6 N \cdot mm}{2204mm} + (1105 - 201) \times 10^3 N - 0.7 \cdot 1620MPa \cdot 6482mm^2}{2 \cdot \left(200000MPa \cdot 6482mm^2 + 407518mm^2 \cdot 26192MPa\right)}$$

= -0.000180

The shear term $\beta$, which provides an indication of the ability of a member to resist shear by aggregate interlock, is determined using Eqn. (3.5).

$$\beta = \frac{0.40}{1 + 1500 \cdot \varepsilon_x} \cdot \frac{1300}{1000 + s_{ze}} = \frac{0.4}{1 + 1500 \cdot -0.000180} \cdot \frac{1300}{1000 + 300mm} \qquad \text{Eqn. (3.5)}$$

= 0.548

The shear term $\theta$, which calculates the predicted angle of the compression field, is calculated using Eqn. (3.7).

$$\theta = \left(29 + 7000 \cdot \varepsilon_x\right) \cdot \left(0.88 + \frac{s_{ze}}{2500}\right) \qquad \text{Eqn. (3.7)}$$

$$= \left(29 + 7000 \cdot -0.000180\right) \cdot \left(0.88 + \frac{300mm}{2500}\right)$$

= 27.7°

S6-06 Section 8 calculates the shear capacity attributed to the concrete and the stirrups using Eqn. (3.2) and Eqn. (3.3) respectively. This study used the proposed Eqn. (3.12) to accommodate the interpolation of effective stirrup area required by S6-06 Clause 14.14.1.6.2.

$$V_c = 2.5 \cdot \beta \cdot f_{cr} \cdot b_v \cdot d_v = 2.5 \cdot 0.548 \cdot 2.36MPa \cdot 178mm \cdot 2204mm \qquad \text{Eqn. (3.2)}$$

= 1267 kN

$$V_s = \gamma \cdot \frac{A_v \cdot f_v \cdot d_v}{s \cdot \tan\theta} = 1.00 \cdot \frac{400mm^2 \cdot 276MPa \cdot 2204mm}{480mm \cdot \tan(27.7^\circ)} \qquad \text{Eqn. (3.3)}$$

= 964 kN

The vertical component of the prestressing is calculated as the product of the slope of the strands and the force in the prestressing strands.

$$V_p = 0.04 \cdot A_{ps} \cdot f_{pe} = 0.04 \cdot 6482mm^2 \cdot 775MPa$$

=201 kN

S6-06 determines the sectional shear capacity using Eqn. (3.1).

$$V_r = V_c + V_s + V_p = 1267kN + 964kN + 201kN \qquad \text{Eqn. (3.1)}$$

= 2432 kN

$$F = \frac{V_r - V_D}{V_{applied}} = \frac{2432kN - 596kN}{509kN}$$

= 3.74

Because the first iteration of shear demand $V_f$ = 1105 kN does not equal $V_r$ the predicted shear capacity has not been properly converged on. The next iteration is taken as the average of the $V_f$ and $V_r$ from the previous iteration. Therefore

$V_{f,n+1}$ is taken as $\frac{1105kN + 2432kN}{2} = 1769kN$. The externally applied load $V_{applied}$ is varied as shown in Table C1 in order to accomplish the required iterations of shear demand.

Table C.1 – Iterations of Sectional Shear Capacity using S6-06 (CSA, 2006)

| Iteration | $V_f$ (kN) | $V_{applied}$ (kN) | $s_{ze}$ (mm) | $\frac{v}{f_c'}$ | $s$ mm | $s_{m1}$ | $\beta$ | $\theta$ | $V_r$ (kN) | $F$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1105 | 509 | 300 | 0.066 | 480 | 600 | 0.548 | 27.7 | 2432 | 3.74 |
| 2 | 1769 | 1173 | 300 | 0.115 | 480 | 554 | 0.473 | 28.3 | 2235 | 3.22 |
| n-1 | 2104 | 1508 | 300 | 0.140 | 480 | 481 | 0.442 | 28.6 | 2153 | 3.06 |
| n | 2109 | 1513 | 2204 | 0.140 | 480 | 479 | 0.179 | 50.3 | 1036 | 0.86 |

Between iteration n-1 and iteration n, the section reached a discontinuity in the predicted shear capacity of the member. The actual stirrup spacing and maximum permissible spacing converge at this point, causing the suggested crack spacing to change from 300 mm to $d_v = 2204$ mm.

S6-06 M assumes that diagonal crack spacing is equal to the longitudinal stirrup spacing as long as this spacing does not exceed the shear depth $d_v$. Table C.2 provides the sectional shear evaluation of the case study bridge using S6-06 M.

Table C.2 – Iterations of Sectional Shear Capacity using S6-06 M

| Iteration | $V_f$ (kN) | $V_{applied}$ (kN) | $s_{ze}$ (mm) | $\frac{v}{f_c'}$ | $\beta$ | $\theta$ | $V_r$ (kN) | $F$ |
|---|---|---|---|---|---|---|---|---|
| 1 | 1106 | 510 | 480 | 0.066 | 0.482 | 29.7 | 2202 | 3.15 |
| 2 | 1654 | 1058 | 480 | 0.107 | 0.425 | 30.2 | 2054 | 2.86 |
| 3 | 1854 | 1258 | 480 | 0.121 | 0.408 | 30.4 | 2008 | 2.77 |
| n | 1980 | 1384 | 480 | 0.131 | 0.398 | 30.5 | 1980 | 2.72 |

Table C.2 indicated how the diagonal crack spacing assumption in the modified shear method S6-06 M can eliminate the discontinuity in shear capacity issue inherent to the sectional shear evaluation procedure in S6-06 Section 8.