The thesis committee for Eisuke Nakamura
certifies that this is the approved version of the following thesis:

Shear Database for Prestressed Concrete Members

APPROVED BY

SUPERVISING COMMITTEE:

____

Oguzhan Bayrak, Supervisor

____

James O. Jirsa

Shear Database for Prestressed Concrete Members

by

Eisuke Nakamura, B.E.; M.E.

Thesis

Presented to the Faculty of the Graduate School of

The University of Texas at Austin

in Partial Fulfillment

of the Requirements

for the Degree of

Master of Science in Engineering

The University of Texas at Austin

May 2011

DEDICATION

To researchers who spend their time on shear in structural concrete

ACKNOWLEDGEMENTS

This thesis was made possible with help from many individuals.

I would like to thank Dr. Oguzhan Bayrak for inviting me to the world of the shear database. Your enthusiasm towards research always encouraged me when I got lost in the world of the shear database. I would also like to thank Dr. James O. Jirsa for giving me meaningful comments to survive in the world of the shear database. It was my great privilege to learn from each of you.

I would like to show my appreciation to the prestressed concrete shear research team: Alejandro Avendaño, Catherine Hovell, and David Langefeld. It was a great pleasure to work with you in the world of the shear database. I would like to thank Chris Williams for sharing a lot of time in classroom. Your diligent attitude towards everything set me free from my laziness. I would like to appreciate tremendous support of the staff at the Phil M. Ferguson Structural Engineering Laboratory.

I would like to express my gratitude to the Government of Japan, especially to the staff at the Ministry of Land, Infrastructure, Transport and Tourism, the National Institute of Land and Infrastructure Management, the Public Works Research Institute, and the National Personnel Authority, for providing me an opportunity and financial support to study at the University of Texas at Austin.

Finally, I would like to thank my family Shiho, Eito, and Hana. You do not even know the presence of the world of the shear database, but I am very sure that you have always stood by me and kept me sane throughout our two years in Austin, Texas.

May 6, 2011

Shear Database for Prestressed Concrete Members

Eisuke Nakamura, M.S.E.

The University of Texas at Austin, 2011

SUPERVISOR: Oguzhan Bayrak

Development of shear databases attracted a great deal of attention in the shear research community within the last decade. Although a few shear databases have already been developed by several research groups, there is no comprehensive shear database that is focused on prestressed concrete members. This thesis aims to develop a shear database for prestressed concrete members with an intensive literature review. This literature review resulted in a database that contained a total of 1,696 tests reported in North America, Japan, and Europe from 1954 to 2010.

The database was used to evaluate shear design provisions available in North America, Japan, and Europe. The variations in measured versus calculated shear strength using twelve shear design equations were analyzed. The analysis results indicated that design expressions based on the Modified Compression Filed Theory (MCFT) produced the best performance to estimate the shear strength of prestressed concrete members with sufficient shear reinforcement. The MCFT-based design expressions, however, provided unconservative strength estimations for members that failed in shear but exhibited signs of horizontal shear damage and/or anchorage zone distress. The ACI 318-08 detailed

method was found to be less conservative than the MCFT-based design expressions. Additionally, on the basis of a careful examination of test results included in the database, a new limit for the minimum shear reinforcement was proposed.

The database was also used to investigate the shear behavior of prestressed concrete members. This investigation revealed that there was no evidence of size effect in the shear strength of prestressed concrete members with sufficient shear reinforcement. Additionally, it was found that prestress force and shear reinforcement increased the shear strength although there was an upper limit on the effectiveness of shear reinforcement.

TABLE OF CONTENTS

List of Tables

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The shear behavior of structural concrete has been one of the most important research issues for over 100 years. Throughout the history of shear research, researchers proposed substantially different shear design equations from empirical and/or theoretical aspects. There is, however, no internationally accepted method to estimate the shear strength of reinforced and/or prestressed concrete members. Existing design codes adopted various shear design equations based on different rationales. This process resulted in substantial differences in shear design provisions among design codes, organizations, and countries.

Recent research that is focused on the shear behavior of structural concrete can be divided into two general approaches: (1) experimental approach; and (2) database approach. The experimental approach involves conducting load tests and investigating the shear behavior on the basis of experimental evidences. The database approach, in contrast, involves assembling a database by collecting shear tests from previous studies. An advantage of the database approach is that it enables researchers to utilize a large number of results from previous shear tests. An analysis using numerous test results is indispensable for not only obtaining a better understanding of the shear behavior of structural concrete but also evaluating the existing shear design provisions.

Several research groups have already developed their own shear databases for reinforced concrete members (Reineck, et al. 2005, Brown, et al. 2006, Collins, et al. 2008). These databases that contain a large number of shear tests from around the world are useful in evaluating various shear design provisions. However, these databases have not exclusively focused on the shear behavior of prestressed concrete members. On the basis of this trend, it is reasonable to suppose that the next stage of the database approach is to develop another shear database that is focused on prestressed concrete members.

Researchers at the University of Texas at Austin initiated the development of a shear database for prestressed concrete members (Avendaño and Bayrak 2008). The University of Texas Prestressed Concrete Shear Database (UTPCSDB) contains 506 tests of prestressed concrete members mainly from U.S literature. This number is almost equivalent to that of Hawkins and Kuchma's database (Hawkins and Kuchma 2007). The UTPCSDB, however, has room for improvement in populating test results published outside of the U.S. This thesis aims to develop a more complete shear database for prestressed concrete members with a comprehensive literature review. This database is referred to as UTPCSDB-2011.

1.2 OBJECTIVES

The overall objectives of this thesis are (1) to develop a shear database for prestressed concrete members with a comprehensive literature survey and (2) to use this database to evaluate existing shear design provisions. In order to achieve these objectives, the following research work was conducted:

- Existing shear databases developed by other research groups were reviewed. Additionally, the current shear design provisions available in North America, Japan, and Europe were summarized, and their differences were identified.

- A shear database for prestressed concrete members, UTPCSDB-2011, was completed with a literature survey on papers and research reports published in North America, Japan, and Europe prior to December 2010.

- The shear behavior of prestressed concrete members was investigated with the UTPCSDB-2011.

- The shear design provisions for prestressed concrete members were evaluated by using the UTPCSDB-2011. This evaluation was focused on (1) the shear strength calculation, (2) the maximum shear strength limit, and (3) the minimum shear reinforcement design.

- Design recommendations were made on the basis of research findings.

1.3 CHAPTER OUTLINE

This thesis consists of eight chapters. Chapter 2 presents a review of existing shear databases developed by other research groups. The chapter also summarizes various types of shear design provisions that are available in North America, Japan, and Europe.

Chapter 3 shows the database development procedure in detail. The chapter includes a discussion on the assembly of the UTPCSDB-2011 and the filtering criteria that are used to reduce the collection database down to the filtered database and to the evaluation database.

Chapter 4 reveals the characteristics of the shear tests stored in the UTPCSDB-2011. The chapter illustrates the historical trends of shear tests conducted in the last sixty years and the distributions of key experimental variables.

Chapters 5 through 7 present the research findings derived from database analyses. Chapter 5 shows results of a careful examination of the shear behavior of prestressed concrete members with sufficient shear reinforcement. Chapter 6 presents results of an evaluation that is focused on the accuracy and conservativeness of existing shear design equations. Chapter 7 provides a discussion on the requirement for the minimum shear reinforcement.

Chapter 8 summarizes the conclusions derived in this thesis and provides recommendations for future work.

CHAPTER 2

LITERATURE REVIEW

2.1 OVERVIEW

This chapter contains two literature reviews conducted on (1) shear databases and (2) shear design provisions. The first literature review provides an outline of the shear databases assembled by other research groups. The second literature review summarizes the shear design provisions that are available in North America, Japan, and Europe. In the review of the shear design provisions, special attention is paid to terms related to the effect of prestress force and the requirement for the minimum shear reinforcement.

2.2 SHEAR DATABASES

2.2.1 Shear Database for Reinforced Concrete Members

Prior to the advent of the first version of the UTPCSDB (Avendaño and Bayrak 2008), several research groups have already developed shear databases for reinforced concrete members and proposed code revisions with their database analyses.

The earliest database approach dates back to the ACI-ASCE Committee 326 (1962). The committee developed a shear database of 194 tests of reinforced concrete beams without shear reinforcement. With a careful examination of this database, the committee proposed a shear design equation for reinforced concrete members without shear reinforcement. This equation is still available in the current ACI 318 code (ACI Committee 318 2008).

After the ACI-ASCE Committee 326 report, numerous shear tests were conducted around the world, and the number of available test results increased exponentially. In the last decade, the database approach attracted a great deal of attention in the shear research community. Several research groups (Reineck, et al. 2005, Brown, et al. 2006, Collins, et al. 2008) compiled results from existing shear tests of reinforced concrete members and proposed revisions to the current ACI 318 shear design provisions. Additionally, another shear database that was focused on deep beams was developed and utilized for evaluating design provisions of the strut-and-tie model (Birrcher, et al. 2009).

2.2.2 Shear Database for Prestressed Concrete Members

The basis of the current ACI 318 shear design equations for prestressed concrete members was also empirical, and these equations were the results of database approaches (MacGregor and Hanson 1969, ACI Committee 318 1965). In the last decade, numerous shear tests reported in the literature enabled the development of more extended shear database for prestressed concrete members.

Hawkins et al. (2005) assembled a shear database including results from 1,359 tests: 878 reinforced concrete members (718 without shear reinforcement and 160 with shear reinforcement) and 481 prestressed concrete members (321 without shear reinforcement and 160 with shear reinforcement). The researchers utilized this database to evaluate existing shear design provisions available around the world. Additionally, the researchers proposed revisions to the shear design provisions in the AASHTO LRFD Bridge Design Specifications and verified the applicability of their revisions by using a selected group of test data from their database.

In 2007, Hawkins and Kuchma expanded their database to 1,874 tests that consisted of 1287 reinforced concrete members and 587 prestressed concrete members (Hawkins and Kuchma 2007). On the basis of their database analysis, the researchers explained the influences of key experimental variables, such as the concrete strength, shear reinforcement ratio, and member depth, on the accuracy of the shear design provisions in the AASHTO LRFD Bridge Design Specifications. The researchers also identified gaps in our knowledge of the shear behavior of reinforced and prestressed concrete members and conducted load tests on full-scale prestressed concrete girders.

At the University of Texas at Austin, Avendaño and Bayrak (2008) developed the first version of the UTPCSDB, which was focused on prestressed concrete members and stored a total of 506 tests. The researchers evaluated the current U.S. shear design provisions and proposed a few revisions on the basis of their database analysis.

In summary, there are several shear databases that contain more than 400 tests on prestressed concrete members. These databases, however, consist mainly of test results reported in U.S. literature. Therefore, it is reasonable to suppose that the first version of the UTPCSDB should be updated with literature published outside of the U.S. The updated database, which is referred to as UTPCSDB-2011, allows further analyses of the shear behavior of prestressed concrete members.

2.2.3 Filtering Criteria of Database Analysis

As mentioned above, several research groups have already assembled the shear databases for reinforced and/or prestressed concrete members, and utilized the databases in their analyses. Obviously, these databases contain various types of tests. For instance, material strengths and member heights vary between the past and the present tests. Additionally, the loading type, shear span to depth ratio, and cross section type vary depending on research objectives.

For their database analyses, those research groups filtered out irrelevant test data from an original database, called the collection database, by employing several filtering criteria, and constructed a new database, called the evaluation database. Objectives and filtering criteria employed in each shear database are summarized in Table 2.1. According to this summary, although several databases use the same experimental variables, such as the concrete strength, member size, shear span to depth ratio, and amount of shear reinforcement, as filtering criteria, their threshold values vary from database to database. It should be emphasized that extracting appropriate data in accordance with research objectives is of great significance for the database analysis.

Table 2.1 Objectives and filtering criteria of existing shear databases

Database	***Main objectives***	***Primary filtering criteria***
Reineck, et al. (2005) 1,007 tests: RC	✓ Accuracy of the ACI 318 shear design provisions ✓ Influence of key variables	● Concrete strength (> 1.7 ksi (12 MPa)) ● Member width (> 2.0 in. (50 mm)) ● Member height (> 2.8 in. (70 mm)) ● Shear span to depth ratio (> 2.4, ≥ 2.9) ● Flexural reinforcement ratio (≤ 0.03)
Brown, et al. (2006) 1,200 tests: RC	✓ Revisions to ACI 318 ✓ Influence of key variables	● Loading type (concentrated, uniform) ● Minimum shear reinforcement
Collins, et al. (2008) 1,849 tests: RC	✓ Accuracy of shear design provisions in North America ✓ Requirements for shear reinforcement	● Shear span to depth ratio (arch action, beam action)
Birrcher (2009) 868 tests: RC (Deep Beam)	✓ Revisions to U.S. design provisions for the strut-and-tie model	● Concrete strength (> 2 ksi (14 MPa)) ● Member width, b_w (> 4.5 in. (114 mm)) ● Effective deph, d (> 12 in. (305 mm)) ● $b_w d$ (> 100 in.^{2} (64,516 mm^2))
Hawkins, et al. (2005) 878 tests: RC 481 tests: PC	✓ Revisions to the AASHTO LRFD specifications ✓ Accuracy of shear design provisions around the world	● Concrete strength (≥ 4 ksi (28 MPa)) ● Member depth (≥ 20 in. (508 mm)) ● Minimum shear reinforcement
Hawkins, Kuchma (2007) 1,287 tests: RC 587 tests: PC	✓ Revisions to the AASHTO LRFD specifications ✓ Influence of key variables	N/A
Avendaño, Bayrak (2008) 506 tests: PC	✓ Accuracy of U.S. shear design provisions ✓ Revisions to U.S. shear design provisions	● Member depth (≥ 12 in. (305 mm)) ● w/ or w/o shear reinforcement ● Shear force at diagonal crack

Note1: "RC": reinforced concrete member "PC": prestressed concrete member

Note2: All databases set "shear failure" as one of the primary filtering criteria.

Note3: The original papers present other filtering criteria in detail.

2.3 SHEAR DESIGN PROVISIONS: NORTH AMERICA

The shear design provisions for prestressed concrete members available in North America are reviewed in this section. These provisions are provided in the following four design specifications:

- ACI 318-08 Building Code Requirements for Structural Concrete (ACI Committee 318 2008);
- AASHTO LRFD Bridge Design Specifications 5th Edition (AASHTO 2010);
- AASHTO Standard Specifications for Highway Bridges 17th Edition (AASHTO 2002); and
- CSA A.23.3-04 Design of Concrete Structures (CSA 2010).

2.3.1 ACI 318-08 Building Code Requirements for Structural Concrete

The shear design equations in ACI 318-08 are based on the modified truss analogy. In other words, the nominal shear strength, V_n, is provided by the sum of the concrete contribution, V_c, and the shear reinforcement contribution, V_s, as follows:

$$V_n = V_c + V_s \qquad \textbf{\textit{Equation 2-1}}$$

The ACI 318-08 code specifies two methods to calculate the V_c term for prestressed concrete members, often called (1) the simplified method and (2) the detailed method. The details of these methods are summarized later in this section.

In terms of the V_s term, the ACI 318-08 code adopts the 45 degree truss model. Thus, V_s provided by shear reinforcement perpendicular to a longitudinal axis of a member is given in:

$$V_s = \frac{A_v f_{ft} d}{s} \qquad \text{(psi, in., lbs)} \qquad \textbf{\textit{Equation 2-2}}$$

where:

A_v = area of shear reinforcement spacing s

f_{yt} = specified yield strength of transverse reinforcement

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement

s = center-to-center spacing of transverse shear reinforcement

However, V_s shall not be taken greater than $8\lambda\sqrt{f'_c}b_w d$. This upper limit, which was originally proposed by ACI-ASCE Committee 326 (1962), was adopted to avoid web crushing failure prior to yielding of shear reinforcement. Note that the coefficient λ is the modification factor reflecting the reduced mechanical properties of lightweight concrete in ACI 318-08.

Additionally, it is worth noting the upper limits on material strengths and the definition of the critical section in ACI 318-08. For prestressed concrete members, f'_c greater than 10,000 psi (69 MPa) shall be allowed in the case of beams satisfying the specified minimum amount of shear reinforcement. The upper limits on the yield strength of shear reinforcement used in design are 60 ksi (414 MPa) for normal steel and 80 ksi (552 MPa) for welded deformed wire reinforcement. The critical section is permitted to be taken as $h/2$ from the internal face of the support since the effective depth varies frequently on prestressed concrete members with harped prestressing steel.

2.3.1.1 Simplified Method

The simplified method was empirically derived by MacGregor and Hanson (1969) with test results on prestressed concrete beams without shear reinforcement at the University of Illinois (Sozen, et al. 1959) and Lehigh University. One benefit of this method is to enable designers to avoid complex expressions and demanding calculation procedures. The simplified method is, however, only applicable to prestressed concrete members with effective prestress greater than 40% of the tensile strength of flexural reinforcement. According to this method, V_c is calculated by:

$$V_c = (0.6\lambda\sqrt{f'_c} + 700\frac{V_u d_p}{M_u})b_w d \quad \text{(psi, in., lbs)} \qquad \textbf{\textit{Equation 2-3}}$$

where:

λ	=	modification factor reflecting the reduced mechanical properties of lightweight concrete
f'_c	=	specified compressive strength of concrete
V_u	=	factored shear force at section
d_p	=	distance from extreme compression fiber to centroid of prestressing steel
M_u	=	factored moment at section
b_w	=	web width
d	=	distance from extreme compression fiber to centroid of prestressed and nonprestressed longitudinal tension reinforcement, but need not be taken less than 0.80h
h	=	overall height of member

However, V_c need not be taken less than $2\lambda\sqrt{f'_c}b_w d$ and greater than $5\lambda\sqrt{f'_c}b_w d$. This upper limit aims to serve as a restriction on the web shear crack.

2.3.1.2 Detailed Method

The detailed method assumes that two different types of inclined cracks appear in prestressed concrete members: (1) the web shear crack; and (2) the flexure shear crack, as illustrated in Figure 2.1 (ACI Committee 318 2008). On the basis of this assumption, the detailed method takes the V_c term as the lesser of two shear forces at the formation of the web shear crack and the flexure shear crack.

The origin of the detailed method dates back to the ACI 318 code published in 1963. In the light of a code review by Ramirez and Breen (1983a), this expression has remained almost the same for more than half a century except for minor changes on its basic assumptions. The original derivation of the detailed method is summarized in the Commentary on the ACI 318-63 code (ACI Committee 318 1965).

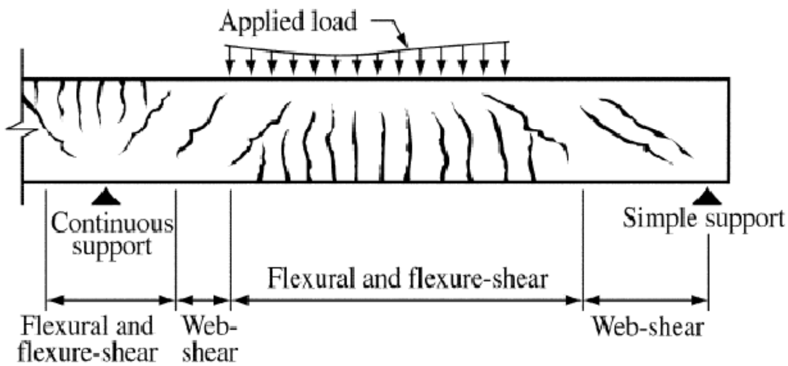


Figure 2.1 Typical types of crack in concrete beams

(Adopted from ACI 318-08 code (ACI Committee 318 2008))

The shear at the formation of the flexure shear crack, V_{ci}, is the sum of the shear to cause an initial flexural crack at the section under consideration and an additional increase of the shear required to turn the initial flexural crack into a flexure shear crack. V_{ci} is given in:

$$V_{ci} = 0.6\lambda\sqrt{f'_c}\,b_w d_p + V_d + \frac{V_i M_{cre}}{M_{\max}} \qquad \text{(psi, in., lbs)} \qquad \textbf{\textit{Equation 2-4}}$$

where:

λ = modification factor reflecting the reduced mechanical properties of lightweight concrete

f'_c = specified compressive strength of concrete

b_w = web width

d_p = distance from extreme compression fiber to centroid of prestressing steel, but need not be taken less than $0.8h$

V_d = shear force at section due to unfactored dead load

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}

M_{cre} = moment causing flexural cracking at section due to externally applied loads, calculated by:

$$M_{cre} = (I / y_t)\left(6\lambda\sqrt{f'_c} + f_{pe} - f_d\right) \qquad \textbf{\textit{Equation 2-5}}$$

M_{max} = maximum factored moment at section due to externally applied loads

h = overall height of member

I = moment of inertia of section about centroidal axis

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to tension face

f_{pe} = compressive stress in concrete due to effective prestress force only at extreme fiber of section where tensile stress in caused by externally applied loads

f_d = stress due to unfactored dead load at extreme fiber of section where tensile stress is caused by externally applied loads

However, V_{ci} need not be taken less than $1.7\lambda\sqrt{f'_c}b_w d$. The original form of Equation 2-4 was developed by Sozen and Hawkins (1962) on the basis of test results at the University of Illinois (Sozen, et al. 1959, MacGregor, et al. 1960a, MacGregor, et al. 1960b, Hawkins, et al. 1961). The first term on the right-hand side in Equation 2-4 stands for the additional shear to turn the initial flexural crack into the inclined crack, the second term accounts for the effect due to dead load, and the third term represents the shear corresponding to the formation of the initial flexural crack. The coefficient of the first term was empirically derived from the aforementioned test results.

The shear at the formation of the web shear crack, V_{cw}, is equivalent to the shear causing a diagonal crack in the web and is calculated by:

$$V_{cw} = \left(3.5\lambda\sqrt{f'_c} + 0.3 f_{pc}\right) b_w d_p + V_p \qquad \text{(psi, in., lbs)} \qquad \textbf{\textit{Equation 2-6}}$$

where:

λ = modification factor reflecting the reduced mechanical properties of lightweight concrete

f'_c = specified compressive strength of concrete

f_{pc} = compressive stress in concrete at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange

b_w = web width

d_p = distance from extreme compression fiber to centroid of prestressing steel, but need not be taken less than $0.8h$

V_p = vertical component of effective prestress force at section

h = overall height of member

Equation 2-6 was derived from an analysis using Mohr's circle. It was assumed that the web shear crack appeared once tensile stress at the centroidal axis of the section under consideration reached the tensile strength of concrete. The tensile strength of concrete was taken conservatively as $3.5\sqrt{f'_c}$ (ACI Committee 318 1965).

In terms of the ACI 318-08 shear design provisions, it should be stressed that both the simplified and detailed methods were derived from classic shear tests more than half a century ago. Most of those specimens had relatively small member heights (around 12 in. (305 mm)) and low concrete strengths (around 6,000 psi (41 MPa)) compared with today's typical bridge girders. As Avendaño and Bayrak (2008) also stated in their literature review, it is imperative to verify the applicability of the ACI 318-08 shear design equations to more realistic members, namely specimens with large member heights and high concrete strengths. It should be emphasized that the UTPCSDB-2011 that contains more recent test results will facilitate this evaluation.

2.3.1.3 Effect of Prestress

It is interesting to note that both the simplified and detailed methods incorporate the effect of prestress force in completely different manners. Although the simplified method has no direct expression to account for the effect of prestress force in Equation 2-3, its use is limited to members that have effective prestress in prestressing steel larger than 40% of its tensile strength. In contrast, the detailed method incorporates the effect of prestress force directly in Equations 2-4 through 2-6. The equations assume that the shear strength provided by concrete is increased due to the effect of prestress force since the prestress force contributes to delay the formation of the inclined crack in concrete.

2.3.1.4 Minimum Shear Reinforcement

The ACI 318-08 code employs two expressions for the minimum amount of shear reinforcement, $A_{v,min}$. The first expression is applicable to both reinforced and prestressed concrete members:

$$A_{v,\min} = 0.75\sqrt{f'_c}\,\frac{b_w s}{f_{yt}} \qquad \text{(psi, in.)} \qquad \textbf{\textit{Equation 2-7}}$$

where:

f'_c	=	specified compressive strength of concrete
b_w	=	web width
s	=	center-to-center spacing of transverse shear reinforcement
f_{yt}	=	specified yield strength of transverse reinforcement

However, $A_{v,min}$ shall not be less than $(50b_w s)/f_{yt}$. Equation 2-7 results in an increase in the minimum amount of shear reinforcement as the concrete strength increases. Equation

2-7 is based on the results of tests conducted by Roller and Russell (1990). The researchers revealed that an increasing amount of shear reinforcement is required to prevent sudden shear failures in high strength concrete members.

The second expression is valid only for prestressed concrete members with effective prestress in prestressing steel at least 40% of its tensile strength:

$$A_{v,\min} = \frac{A_{ps} f_{pu} s}{80 f_{yt} d} \sqrt{\frac{d}{b_w}} \qquad \text{(psi, in.)} \qquad \textbf{\textit{Equation 2-8}}$$

where:

A_{ps}	=	area of prestressing steel in flexural tension zone
f_{pu}	=	specified tensile strength of prestressing steel
s	=	center-to-center spacing of transverse shear reinforcement
f_{yt}	=	specified yield strength of transverse reinforcement
d	=	distance from extreme compression fiber to centroid of prestressed and nonprestressed longitudinal tension reinforcement
b_w	=	web width

Equation 2-8 was based on the results of shear tests at the University of Illinois (Olesen, et a. 1967). The ACI 318-08 code requires satisfying the lesser of two values calculated from Equations 2-7 and 2-8 to assure ductile behavior in prestressed concrete members.

2.3.2 AASHTO LRFD Bridge Design Specifications 5th Edition

The AASHTO LRFD Bridge Design Specifications 5th Edition (AASHTO 2010), which is referred to as AASHTO LRFD 2010, provide the nominal shear strength, V_n, as the lesser of two values calculated by:

$$V_n = V_c + V_s + V_p \qquad \textbf{\textit{Equation 2-9}}$$

$$V_n = 0.25 f'_c b_v d_v + V_p \qquad \text{(ksi, in., kips)} \qquad \textbf{\textit{Equation 2-10}}$$

where:

V_c	=	nominal shear resistance provided by tensile stresses in the concrete
V_s	=	shear resistance provided by shear reinforcement
V_p	=	component in the direction of the applied shear of the effective prestressing force, positive if resisting the applied shear
f'_c	=	specified compressive strength of concrete for use in design
b_v	=	effective web width taken as the minimum web width within d_v
d_v	=	effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; it need not be taken to be less than the greater of $0.9d_e$ or $0.72h$, or calculated as follows:

$$d_v = \frac{M_n}{A_s f_y + A_{ps} f_{ps}} \qquad \textbf{\textit{Equation 2-11}}$$

d_e	=	effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement, calculated as follows:

$$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y} \qquad \textbf{\textit{Equation 2-12}}$$

h	=	overall thickness or depth of a member
M_n	=	nominal flexural resistance
A_s	=	area of nonprestressed tension reinforcement
f_y	=	specified minimum yield strength of reinforcing bars
A_{ps}	=	area of prestressing steel
f_{ps}	=	average stress in prestressing steel at the time for which the nominal resistance of member is required
d_p	=	distance from extreme compression fiber to the centroid of the prestressing tendons
d_s	=	distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement

Except for segmental post-tensioned concrete box girder bridges, there are two procedures to determine the V_c and V_s terms for prestressed concrete members: (1) the general procedure; and (2) the simplified procedure. These two procedures have completely different technical bases. While the general procedure employs the Modified Compression Field Theory (MCFT, Vecchio and Collins 1986, Collins and Mitchell 1997), the simplified procedure is based on a revision to the ACI 318 detailed method (Hawkins, et al. 2005).

It should be noted that Equation 2-10 aims to avoid web crushing failure prior to yielding of shear reinforcement. This upper limit is substantially different from that of ACI 318-08. Hawkins and Kuchma (2007) recommended reducing $0.25f'_c$ in Equation 2-10 to $0.18f'_c$ on the basis of their experimental results. This recommendation is adopted to avoid a localized diagonal compression failure and horizontal shear failure in AASHTO LRFD 2010.

For segmental post-tensioned concrete box girder bridges, there is a different shear design procedure other than the general and simplified procedures. This procedure was originally employed in the AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges (AASHTO 1999, 2003). The AASHTO LRFD 2010 specifications also adopt this procedure as a shear design equation for segmental post-tensioned box girders.

Additionally, in AASHTO LRFD 2010, design concrete strengths above 10 ksi (69 MPa) are permitted only when specific articles and physical tests validate their usage. The design yield strength of reinforcement is limited to 75 ksi (517 MPa).

2.3.2.1 General Procedure

The first edition of the AASHTO LRFD Bridge Design Specifications, which adopted the MCFT-based design expressions, appeared in 1994. The MCFT is capable of predicting the response of diagonally cracked concrete subjected to shear and other forces in both prestressed and nonprestressed concrete members. One of the significant characteristics of the MCFT is to employ equations that represent equilibrium conditions, compatibility conditions, and stress-strain relationships. The MCFT makes it possible to account for the tensile stresses in concrete between diagonal cracks. While the introduction of the MCFT was appreciated as a meaningful step towards unified shear design, the method's complicated calculation procedure due to its iterative process caused concerns among engineers.

In order to reduce the MCFT's computational effort, simplified expressions were later derived by using three assumptions: (1) to consider one biaxial element within the web; (2) to assume a uniform shear stress distribution over the web depth; and (3) to define a direction of diagonal compressive stress as constant in the web. The V_c and V_s terms in the AASHTO LRFD 2010 specifications are given in:

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v \qquad \text{(ksi, in., kips)} \qquad \textbf{\textit{Equation 2-13}}$$

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \qquad \text{(ksi, in., kips)} \qquad \textbf{\textit{Equation 2-14}}$$

where:

β	=	factor relating effect of longitudinal strain on the shear capacity of concrete, as indicated by the ability of diagonally cracked concrete to transmit tension
f'_c	=	specified compressive strength of concrete for use in design
b_v	=	effective web width taken as the minimum web width within d_v
d_v	=	effective shear depth
A_v	=	area of shear reinforcement within a distance s
f_y	=	specified minimum yield strength of reinforcing bars
θ	=	angle of inclination of diagonal compressive stresses
α	=	angle of inclination of transverse reinforcement to longitudinal axis
s	=	spacing of transverse reinforcement measured in a direction parallel to the longitudinal reinforcement

These equations indicate that the V_c and V_s terms are functions of two parameters, β and θ. These parameters are calculated by the following equations:

For sections containing at least the minimum amount of shear reinforcement:

$$\beta = \frac{4.8}{(1+750\varepsilon_s)} \qquad \textbf{\textit{Equation 2-15}}$$

For sections containing less than the minimum amount of shear reinforcement:

$$\beta = \frac{4.8}{(1+750\varepsilon_s)}\frac{51}{(39+s_{xe})} \quad \text{(in.)} \qquad \textbf{\textit{Equation 2-16}}$$

For all sections:

$$\theta = 29 + 3500\varepsilon_s \quad \text{(degree)} \qquad \textbf{\textit{Equation 2-17}}$$

where:

ε_s = average tensile strain in cracked concrete in the direction of tension tie

s_{xe} = equivalent value of s_x, which allows for influence of aggregate size

s_x = crack spacing parameter, the lesser of either d_v or the maximum distance between layers of longitudinal crack control reinforcement, where the area of the reinforcement in each layer is not less than $0.003b_v s_x$.

d_v = effective shear depth

b_v = effective web width taken as the minimum web width within d_v

The parameters, β and θ, are functions of ε_s and s_{xe}. ε_s is calculated from a relationship of axial force, shear force, flexural moment, prestress force, and stiffness of flexural reinforcement as shown in Figure 2.2. ε_s is given in:

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}\right)}{E_s A_s + E_p A_{ps}} \quad \text{(ksi, in., kips)} \qquad \textbf{\textit{Equation 2-18}}$$

where:

M_u = factored moment at the section

d_v = effective shear depth

N_u = applied factored axial force, taken as positive if tensile

V_u = factored shear force at the section

V_p = component in the direction of the applied shear of the effective prestressing force, positive if resisting the applied shear

A_{ps} = area of prestressing steel

f_{po} = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in differene in strain between the prestressing tendons and the surrounding concrete

E_s = modulus of elasticity of reinforcing bars

A_s = area of nonprestressed tension reinforcement

E_p = modulus of elasticity of prestressing tendons

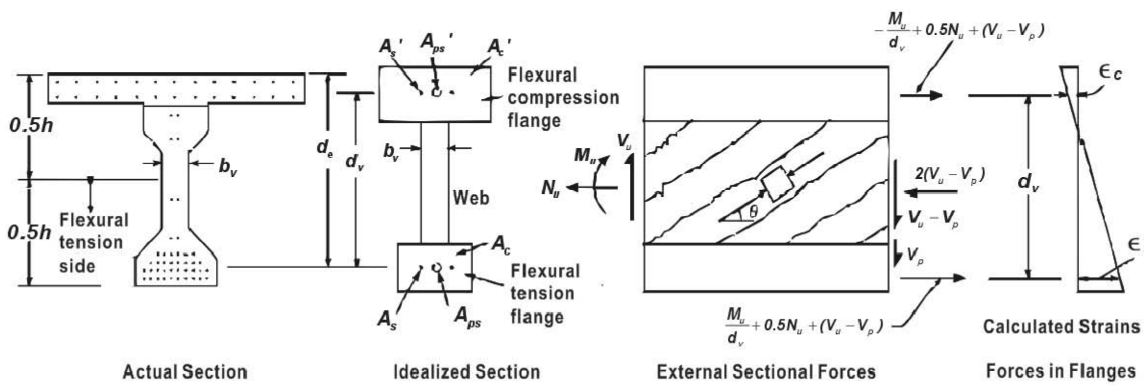


Figure 2.2 Calculation procedure for determining ε_s

(Adopted from AASHTO LRFD Bridge Design Specifications (AASHTO 2010))

The crack spacing parameter, s_x, accounts for the influence of size effect. Equations 2-15 and 2-16 give β depending on amounts of shear reinforcement because members having at least the minimum amount of shear reinforcement are assumed to reveal well distributed diagonal cracks. In contrast, for members having less than the minimum amount of shear reinforcement, an ability of diagonally cracked concrete to transmit shear stress is expected to be lower. Thus, the crack spacing parameter, s_x, affects β only in Equation 2-16 indirectly through the calculation of s_{xe} as follows:

$$s_{xe} = s_x \frac{1.38}{\left(a_g + 0.63\right)} \quad \text{(in.)} \qquad \textbf{\textit{Equation 2-19}}$$

where:

$12.0\ in. \leq s_{xe} \leq 80.0\ in.$

where:

s_x = crack spacing parameter, the lesser of either d_v or the maximum distance between layers of longitudinal crack control reinforcement, where the area of the reinforcement in each layer is not less than $0.003 b_v s_x$.

a_g = maximum aggregate size

This procedure, which was functionally equivalent to that of the CSA A23.3-04 code (CSA 2010), was first adopted into the 2008 interim revision of the AASHTO LRFD Bridge Design Specifications (AASHTO 2008). Prior to this revision, the engineers were forced to determine β and θ from iterative solutions using graphs or tables shown in Tables 2.2 and 2.3. On the basis of the studies that were focused on simplified expressions (Bentz, et al. 2006a, 2006b) and their applicability (Hawkins, et al. 2005), Equations 2-13 through 2-19 were first introduced to the 2008 interim revision of the AASHTO LRFD Bridge Design Specifications. These equations are also employed as the general procedure in AASHTO LRFD 2010.

Table 2.2 Values of θ and β for sections with transverse reinforcement

(Adopted from AASHTO LRFD Bridge Design Specifications (AASHTO 2010))

$\frac{v_u}{f'_c}$	$\varepsilon_x \times 1{,}000$								
	≤−0.20	≤−0.10	≤−0.05	≤0	≤0.125	≤0.25	≤0.50	≤0.75	≤1.00
≤0.075	22.3 6.32	20.4 4.75	21.0 4.10	21.8 3.75	24.3 3.24	26.6 2.94	30.5 2.59	33.7 2.38	36.4 2.23
≤0.100	18.1 3.79	20.4 3.38	21.4 3.24	22.5 3.14	24.9 2.91	27.1 2.75	30.8 2.50	34.0 2.32	36.7 2.18
≤0.125	19.9 3.18	21.9 2.99	22.8 2.94	23.7 2.87	25.9 2.74	27.9 2.62	31.4 2.42	34.4 2.26	37.0 2.13
≤0.150	21.6 2.88	23.3 2.79	24.2 2.78	25.0 2.72	26.9 2.60	28.8 2.52	32.1 2.36	34.9 2.21	37.3 2.08
≤0.175	23.2 2.73	24.7 2.66	25.5 2.65	26.2 2.60	28.0 2.52	29.7 2.44	32.7 2.28	35.2 2.14	36.8 1.96
≤0.200	24.7 2.63	26.1 2.59	26.7 2.52	27.4 2.51	29.0 2.43	30.6 2.37	32.8 2.14	34.5 1.94	36.1 1.79
≤0.225	26.1 2.53	27.3 2.45	27.9 2.42	28.5 2.40	30.0 2.34	30.8 2.14	32.3 1.86	34.0 1.73	35.7 1.64
≤0.250	27.5 2.39	28.6 2.39	29.1 2.33	29.7 2.33	30.6 2.12	31.3 1.93	32.8 1.70	34.3 1.58	35.8 1.50

Table 2.3 Values of θ and β for sections with less than minimum transverse reinforcement

(Adopted from AASHTO LRFD Bridge Design Specifications (AASHTO 2010))

s_{xe}, in.	$\varepsilon_x \times 1000$										
	≤−0.20	≤−0.10	≤−0.05	≤0	≤0.125	≤0.25	≤0.50	≤0.75	≤1.00	≤1.50	≤2.00
≤5	25.4 6.36	25.5 6.06	25.9 5.56	26.4 5.15	27.7 4.41	28.9 3.91	30.9 3.26	32.4 2.86	33.7 2.58	35.6 2.21	37.2 1.96
≤10	27.6 5.78	27.6 5.78	28.3 5.38	29.3 4.89	31.6 4.05	33.5 3.52	36.3 2.88	38.4 2.50	40.1 2.23	42.7 1.88	44.7 1.65
≤15	29.5 5.34	29.5 5.34	29.7 5.27	31.1 4.73	34.1 3.82	36.5 3.28	39.9 2.64	42.4 2.26	44.4 2.01	47.4 1.68	49.7 1.46
≤20	31.2 4.99	31.2 4.99	31.2 4.99	32.3 4.61	36.0 3.65	38.8 3.09	42.7 2.46	45.5 2.09	47.6 1.85	50.9 1.52	53.4 1.31
≤30	34.1 4.46	34.1 4.46	34.1 4.46	34.2 4.43	38.9 3.39	42.3 2.82	46.9 2.19	50.1 1.84	52.6 1.60	56.3 1.30	59.0 1.10
≤40	36.6 4.06	36.6 4.06	36.6 4.06	36.6 4.06	41.2 3.20	45.0 2.62	50.2 2.00	53.7 1.66	56.3 1.43	60.2 1.14	63.0 0.95
≤60	40.8 3.50	40.8 3.50	40.8 3.50	40.8 3.50	44.5 2.92	49.2 2.32	55.1 1.72	58.9 1.40	61.8 1.18	65.8 0.92	68.6 0.75
≤80	44.3 3.10	44.3 3.10	44.3 3.10	44.3 3.10	47.1 2.71	52.3 2.11	58.7 1.52	62.8 1.21	65.7 1.01	69.7 0.76	72.4 0.62

2.3.2.2 Simplified Procedure

The AASHTO LRFD Bridge Design Specifications 4th Edition (AASHTO 2007) adopted the simplified procedure, which was based on recommendations by Hawkins, et al. (2005). The basis of the simplified procedure is compatible with the ACI 318-08 detailed method, namely the nominal shear strength is given by adding the lesser of V_{ci} and V_{cw} to the V_s term. Although the ACI 318-08 detailed method is applicable only to prestressed concrete members, the researchers calibrated the original equations so as to make them compatible with both prestressed and nonprestressed concrete members. According to this procedure, the V_c term is taken as the lesser of V_{ci} and V_{cw}, which are given in:

$$V_{ci} = 0.02\sqrt{f'_c}\,b_v d_v + V_d + \frac{V_i M_{cre}}{M_{\max}} \geq 0.06\sqrt{f'_c}\,b_v d_v \quad \text{(ksi, in., kips)} \qquad \textbf{\textit{Equation 2-20}}$$

$$V_{cw} = \left(0.06\sqrt{f'_c} + 0.30 f_{pc}\right) b_v d_v + V_p \quad \text{(ksi, in., kips)} \qquad \textbf{\textit{Equation 2-21}}$$

where:

f'_c	=	specified compressive strength of concrete for use in design
b_v	=	effective web width taken as the minimum web width within d_v
d_v	=	effective shear depth
V_d	=	shear force at section due to unfactored dead load
V_i	=	factored shear force at section due to externally applied loads occurring simultaneously with M_{max}
M_{cre}	=	moment causing flexural cracking at section due to externally applied loads
M_{max}	=	maximum factored moment at section due to externally applied loads
f_{pc}	=	compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross section resisting live load or at the junction of the web and flange when the centroid lies in the flange
V_p	=	component in the direction of the applied shear of the effective prestressing force, positive if resisting the applied shear

It should be noted that stress terms in AASHTO LRFD 2010 are expressed in ksi. Once the unit is converted to psi, Equation 2-20 becomes equivalent to Equation 2-4. On the other hand, Equation 2-21 is not consistent with Equation 2-6 even after the unit conversion because Hawkins, et al. (2005) considered that the diagonal tensile strength of concrete decreases in nonprestressed members and prestressed members with low levels of prestress force.

Another difference from the ACI 318-08 detailed method is the adoption of the variable angle truss method for the V_s term. The simplified procedure employs Equation 2-14 and sets the angle of inclination of diagonal compressive stresses, θ, as follows:

when $V_{ci} < V_{cw}$: $\cot\theta = 1.0$ ***Equation 2-22***

when $V_{ci} > V_{cw}$: $\cot\theta = 1.0 + 3(\frac{f_{pc}}{\sqrt{f'_c}}) \leq 1.8$ (ksi) ***Equation 2-23***

where:

θ = angle of inclination of diagonal compressive stresses

f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross section resisting live load or at the junction of the web and flange when the centroid lies in the flange

f'_c = specified compressive strength of concrete for use in design

Equaiton 2-22, which is equivalent to the traditional 45 degree truss model, is used in the flexure shear cracking region, and Equation 2-23, which is derived from an analysis of Mohr's circle, is employed in the web shear cracking region. The lower limit of θ is taken as approximately 30 degree.

2.3.2.3 Procedure for Segmental Box Girder Bridges

The shear design procedure for segmental box girder bridges, which is referred to as the segmental procedure, employs a simple expression for the V_c term. The basis of this expression was derived by Ramirez and Breen (1983a, 1983b, 1983c, 1991).

According to this method, the nominal shear strength, V_n, is given in the lesser of following equations:

$$V_n = V_c + V_s \quad \text{(ksi, in., kips)} \qquad \textbf{\textit{Equation 2-24}}$$

$$V_n = 0.379\sqrt{f'_c}\,b_v d_v \quad \text{(ksi, in., kips)} \qquad \textbf{\textit{Equation 2-25}}$$

where:

f'_c = specified compressive strength of concrete for use in design

b_v = effective web width taken as the minimum web width within d_v

d_v = effective shear depth

Note that Equation 2-25 intends to avoid web crushing failure prior to yielding of shear reinforcement.

The V_c and V_s term are calicluated as follows:

$$V_c = 0.0632K\sqrt{f'_c}\,b_v d_v \quad \text{(ksi, in., kips)} \qquad \textbf{\textit{Equation 2-26}}$$

$$V_s = \frac{A_v f_y d_v}{s} \quad \text{(ksi, in., kips)} \qquad \textbf{\textit{Equation 2-27}}$$

where:

K = stress variable

f'_c = specified compressive strength of concrete for use in design

b_v = effective web width taken as the minimum web width within d_v

d_v = effective shear depth

A_v = area of shear reinforcement within a distance s

f_y = specified minimum yield strength of reinforcing bars

s = spacing of transverse reinforcement measured in a direction parallel to the longitudinal reinforcement

In Equation 2-26, the stress variable, K, is a factor that represents the increase of the concrete contribution due to prestress force. The equation of the K factor is derived from Mohr's circle on an element at the neutral axis of a prestressed concrete member before diagonal cracking. This derivation process is similar to that of V_{cw} in ACI 318-08. The K factor is given in:

$$K = \sqrt{1 + \frac{f_{pc}}{0.0632\sqrt{f'_c}}} \leq 2.0 \qquad \text{(ksi)} \qquad \textbf{\textit{Equation 2-28}}$$

where:

f_{pc}	=	compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross section resisting live load or at the junction of the web and flange when the centroid lies in the flange
f'_c	=	specified compressive strength of concrete for use in design

However, the K factor need not exceed 1.0 when the stress at the extreme tension fiber due to factored load and effective prestress force exceeds $0.19\sqrt{f'_c}$ in tension. Note that the gross section properties are used in this calculation. In the light of a database analysis by Avendaño and Bayrak (2008), the accuracy of this procedure is highly improved by removing this limit on the K factor and the upper limit on the concrete strength.

The V_s term is based on the 45 degree truss model. Equation 2-27 is the same as that in the ACI 318-08 code.

2.3.2.4 Effect of Prestress Force

Since the general and simplified procedures are based on completely different shear design philosophies, there are substantial differences in the way to take into account the effect of prestress force. The general procedure accounts for the effect of prestress force in the calculation of ε_s, which affects the values of β and θ. This implies that the effect of prestress force is involved indirectly in both the V_c and V_s terms. In contrast, the simplified procedure considers the effect of prestress force in V_{ci} and V_{cw} like the ACI 318-08 detailed method. The simplified procedure also takes into account the effect of prestress force by adjusting θ in the web shear region.

On the other hand, in the segmental procedure, prestress force affects only the V_c term because the V_s term is calculated by using the traditional 45 degree truss model.

2.3.2.5 Minimum Shear Reinforcement

In AASHTO LRFD 2010, there are two expressions of the minimum amount of shear reinforcement depending on girder types as follows:

Except for segmental post-tensioned concrete box girders:

$$A_v \geq 0.0316\sqrt{f'_c}\,\frac{b_v s}{f_y} \qquad \text{(ksi, in.)} \qquad \textbf{\textit{Equation 2-29}}$$

For segmental post-tensioned concrete box girders:

$$A_v \geq 0.05\frac{b_v s}{f_y} \qquad \text{(ksi, in.)} \qquad \textbf{\textit{Equation 2-30}}$$

where:

A_v = area of transverse reinforcement within a distance s

f'_c = specified compressive strength of concrete for use in design

b_v = width of web adjusted for the presence of ducts

s = spacing of transverse reinforcement

f_y = yield strength of transverse reinforcement

Again, note that stress terms in AASHTO LRFD 2010 are expressed in ksi. Once strengths of concrete and transverse reinforcement in Equation 2-29 are expressed in psi, the coefficient, 0.0316, turns into 1.0. The minimum shear reinforcement of the AASHTO LRFD 2010 specifications is larger than that of the ACI 318-08 code. In contrast, Equation 2-30 is equivalent to the lower limit given for Equation 2-7.

2.3.3 AASHTO Standard Specifications for Highway Bridges 17th Edition

The AASHTO Standard Specifications for Highway Bridges (AASHTO 2002), which is referred to as the AASHTO standard specifications, utilizes exactly the same procedure as the ACI 318-08 code. One negligible difference is found in the minimum shear reinforcement requirements. In the AASHTO standard specifications, the minimum amount of shear reinforcement is limited to $(50b_w s)/f_{yt}$, which is the same as the lower limit given for Equation 2.7. However, Equation 2.7 appeared in the ACI 318 code in 2002, and the latest edition of the AASHTO standard specifications was also published in 2002. Thus, we can conclude that there is no technical conflict on the shear design provision between the AASHTO standard specifications and the ACI 318-08 code.

2.3.4 CSA A23.3-04 Design of Concrete Structures

The CSA A23.3-04 Design of Concrete Structures (CSA 2010), which is referred to as CSA A23.3-04, contains two shear design methods based on the MCFT: (1) the simplified method; and (2) the general method. The simplified method consists of simplified expressions that are derived by making several assumptions on the general method. Since the CSA A23.3-04 code indicates that the general method is more accurate, the focus is placed on the general method in this thesis.

As mentioned previously, the shear design equations in CSA A23.3-04 are functionally equivalent to those of the AASHTO LRFD 2010 general procedure. Thus, the primary differences between the CSA A23.3-04 general method and the AASHTO LRFD 2010 general procedure are summarized in this section.

- Different definitions of the tensile strain are used to calculate the parameter β. While the net longitudinal tensile strain at the centroid of the tension reinforcement, ε_s, is used in the AASHTO LRFD 2010 general procedure, the longitudinal strain at mid-depth of the member, ε_x, is employed in the CSA

A.23.3-04 general method. This difference, however, vanishes through the calculation process.

- Different limits on the equivalent crack spacing parameter are adopted. In AASHTO LRFD 2010, s_{xe} ranges from 12 in. (305 mm) to 60 in. (1,524 mm). In contrast, s_{ze} in CSA A.23.3-04 is calculated by taking aggregate size, a_g, as zero when f'_c exceeds 10 ksi (70 MPa) and reducing a_g linearly to zero when f'_c increases from 8.7 ksi (60 MPa) to 10 ksi (70 MPa).

- The CSA A.23.3-04 code specifies the minimum shear reinforcement as shown in Equation 2-31. The coefficient, 0.06, turns into 0.72 once the U.S. customary units are used. This implies that the minimum shear reinforcement in CSA A.23.3-04 is lower than those in AASHTO LRFD 2010 and ACI 318-08.

$$A_v = 0.06\sqrt{f'_c}\frac{b_w s}{f_y} \qquad \text{(MPa, mm)} \qquad \textbf{\textit{Equation 2-31}}$$

where:

A_v	=	area of shear reinforcement within a distance s
f'_c	=	specified compressive strength of concrete
b_w	=	minimum effective web width
s	=	spacing of shear reinforcement measured parallel to the longitudinal axis of the member
f_y	=	specified yield strength of nonprestressed reinforcement

Additionally, the CSA A23.3-04 code sets upper limits on material strengths for design. The compressive strength of concrete is limited up to 12 ksi (80 MPa), and the yield strength of nonprestressed reinforcement is restricted up to 73 ksi (500 MPa).

2.4 Shear Design Provisions: Japan

The Japanese shear design provisions for prestressed concrete members are described in this section. More specifically, the relevant provisions in the following design specifications are summarized:

- JSCE Standard Specifications for Concrete Structures 2007 (Japan Society of Civil Engineers 2007); and
- JRA Specifications for Highway Bridges Part III Concrete Bridges (Japan Road Association 2002).

2.4.1 JSCE Standard Specifications for Concrete Structures 2007

The shear design equations in the JSCE Standard Specifications for Concrete Structures (Japan Society of Civil Engineers 2007), which is referred to as JSCE 2007, are based on the modified truss model. The shear strength, V_{yd}, is provided by the sum of the concrete contribution, V_{cd}, the shear reinforcement contribution, V_{sd}, and the vertical component of effective prestress force, V_{ped}, as follows:

$$V_{yd} = V_{cd} + V_{sd} + V_{ped} \qquad \textbf{\textit{Equation 2-32}}$$

In JSCE 2007, an upper limit on the shear strength is also established as the diagonal compression strength of the web, V_{wcd}, as follows:

$$V_{wcd} = f_{wcd} b_w d / \gamma_b \qquad \text{(MPa, mm, N)} \qquad \textbf{\textit{Equation 2-33}}$$

where:

f_{wcd} = $1.25\sqrt{f'_{cd}} \leq 7.8$

b_w = web width

d = effective depth

γ_b = member factor, shall be taken as 1.3 in general

f'_{cd} = design compressive strength of concrete, obtained by dividing design strength by material factor that is taken as 1.3 for ultimate limit state

It should be noted that there are two equations to calculate the shear strength depending on the shear span to depth ratio in JSCE 2007. The first equation, which is called the general equation in this thesis, is applicable to both reinforced and prestressed concrete members with relatively large shear span to depth ratios, namely ratios of greater than 2.0 in general. The second equation is used for members with shear span to depth ratios of smaller than 2.0, often called the deep beam equation.

Additionally, there are upper limits on the yield strength of shear reinforcement in design. The yield strength of shear reinforcement is limited up to 58 ksi (400 MPa). This limit is increased to 116 ksi (800 MPa) when the concrete strength is greater than 8.7 ksi (60 MPa) for normal concrete and 7.3 ksi (50 MPa) for self-consolidating concrete.

2.4.1.1 General Equation

The general equation provides the V_c term for reinforced and prestressed concrete members with shear span to depth ratios of greater than 2.0 as follows:

$$V_{cd} = \beta_d \beta_p \beta_n f_{vcd} b_w d / \gamma_b$$ (MPa, mm, N) ***Equation 2-34***

where:

β_d = $\sqrt[4]{1000/d} \leq 1.5$

β_p = $\sqrt[3]{100\, p_v} \leq 1.5$

β_n = $1 + 2\dfrac{M_0}{M_{ud}} \leq 2$ when $N'_d \geq 0$

= $1 + 4\dfrac{M_0}{M_{ud}} \geq 0$ when $N'_d < 0$

f_{vcd} = $0.20\sqrt[3]{f'_{cd}} \leq 0.72$

b_w = web width

d	=	effective depth
γ_b	=	factor, shall be taken as 1.3 in general
p_v	=	$\dfrac{A_s}{b_w d}$
M_0	=	decompression moment necessary to cancel stress due to axial force at extreme tension fiber
M_{ud}	=	design flexural moment capacity
N'_d	=	design axial compressive force
f'_{cd}	=	design compressive strength of concrete
A_s	=	area of tension reinforcement

The V_c term calculated by the general equation depends on four parameters: (1) compressive strength of concrete; (2) effective depth; (3) axial force; and (4) longitudinal tension reinforcement ratio. Figures 2.3 and 2.4 illustrate the influence of these four parameters. The upper limit on f_{vcd} intends to limit the concrete strength up to 8.7 ksi (60 MPa). In members with effective depths of larger than 39 in. (1,000 mm), the shear strength is reduced by β_d, which accounts for the influence of size effect. The amount of longitudinal tensile reinforcement is also assumed to affect the shear strength.

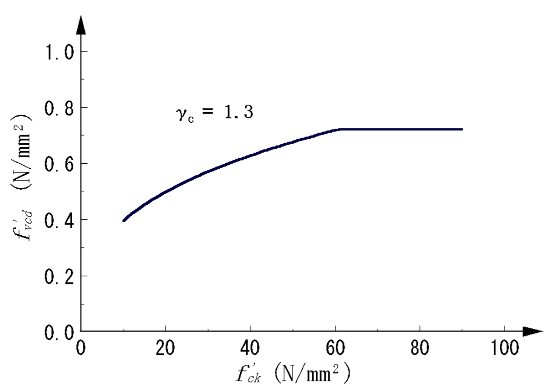


Figure 2.3 Relationship between characteristic concrete strength f'_{ck} and f'_{vcd}

(Adopted from Japan Society of Civil Engineers (2007))

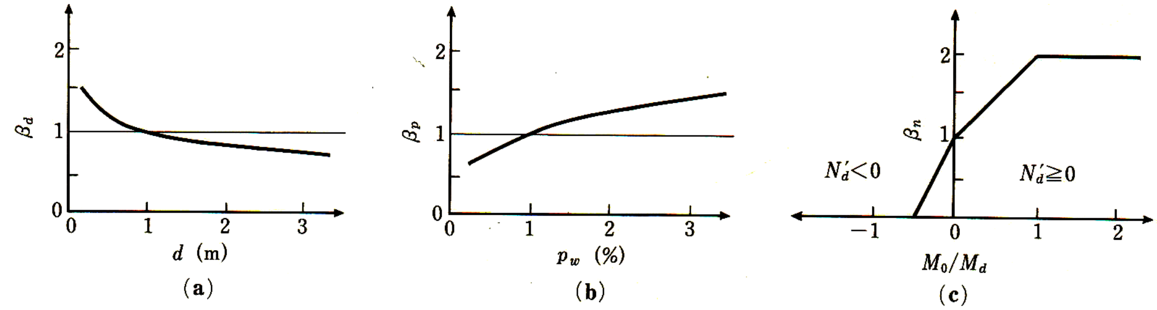


Figure 2.4 Influences of three parameters on shear strength

(Adopted from Japan Society of Civil Engineers (2007))

The influence of these three parameters except for the axial force is derived from an empirical equation proposed by Niwa, et al (1986). Niwa, et al (1986) modified an equation that was empirically developed by Okamura and Higai (1980) by incorporating results of shear tests on large reinforced concrete beams (Iguro, et al. 1984, Shioya 1988).

The V_s term is obtained from the 45 degree truss model that is the same as the ACI 318-08 code.

The effect of prestress force is discussed later in this section.

2.4.1.2 Deep Beam Equation

The deep beam equation provides the shear strength for reinforced and prestressed concrete members with shear span to depth ratios of less than 2.0 by calculating the shear compression strength, V_{dd}, as follows:

$$V_{dd} = (\beta_d \beta_n + \beta_w)\beta_p \beta_a f_{dd} b_w d / \gamma_b \quad \text{(MPa, mm, N)} \qquad \textbf{\textit{Equation 2-35}}$$

where:

$$\beta_d = \sqrt[4]{1000/d} \leq 1.5$$

$$\beta_n = 1 + 2\frac{M_0}{M_{ud}} \leq 2 \qquad \text{when } N'_d \geq 0$$

$$1+4\frac{M_0}{M_{ud}} \geq 0 \qquad \text{when } N'_d < 0$$

β_w	=	$4.2\sqrt[3]{100\,p_w}\left(a/d-0.75\right)/\sqrt{f'_{cd}} \geq 0$
β_p	=	$\dfrac{1+\sqrt{100\,p_v}}{2} \leq 1.5$
β_a	=	$\dfrac{5}{1+(a/d)^2}$
f_{dd}	=	$0.19\sqrt{f'_{cd}}$
b_w	=	web width
d	=	effective depth
γ_b	=	member factor, shall be taken as 1.2 in general
M_0	=	decompression moment necessary to cancel stress due to axial force at extreme tension fiber
M_{ud}	=	design flexural moment capacity
N'_d	=	design axial compressive force
p_w	=	shear reinforcement ratio
		$\dfrac{A_w}{b_w s_s}$ when $p_w \geq 0.002$
		0 when $p_w < 0.002$
a	=	shear span
f'_{cd}	=	design compressive strength of concrete
p_v	=	tensile reinforcement ratio
		$p_{v1}+p_{v2}d_2/d_1$
s_s	=	spacing of shear reinforcement perpendicular to longitudinal axis of member
p_{v1}	=	tensile reinforcement ratio except for horizontal shear reinforcement
p_{v2}	=	horizontal shear reinforcement ratio

d_2 = distance from extreme compression fiber to horizontal shear reinforcement

d_1 = distance from extreme compression fiber to tensile reinforcement except for horizontal shear reinforcement

The basis of the deep beam equation was originally derived from the analytical and empirical studies by Niwa (1983).

It should be noted that Equation 2-35 is able to provide the shear strength of members having shear reinforcement. In other words, there is no need to calculate the V_c and V_s terms separate since the equation adopts the shear reinforcement parameter, β_w, which represents the influence of shear reinforcement on the shear strength of deep beams. The expression of β_w was derived from an empirical evaluation of existing test results and first introduced to JSCE 2007.

Additionally, the adoption of the parameter β_p enables the deep beam equation to account for the effect of horizontal shear reinforcement on the shear strength. The deep beam equation also takes into account the effect of prestress force in the same manner as the general equation, namely the deep beam equation also adopts β_n.

2.4.1.3 Effect of Prestress Force

Both the general equation and the deep beam equation employ the parameter β_n to account for the effect of prestress force on the shear strength. The basic concept of β_n is that the shear strength of prestressed concrete members is enhanced by the amount of additional shear required to overcome the compressive stress due to prestress force at the extreme tension fiber. This concept assumes that prestress force contributes to delay the formation of a flexural crack and there is no difference in the shear strength between prestressed and nonprestressed members after the flexural crack appears. The increase of the shear strength due to prestress force is expressed by incorporating a ratio of the decompression moment to the flexural moment capacity, namely M_0/M_{ud}.

The JSCE specifications adopted this approach by reference to CEB-FIP Model Code 78 (CEB 1978), and its original derivation was explained by Losberg and Hedman (1978). Walraven (1987) investigated a relationship between the shear strength and the ratio of the decompression moment to the flexural moment at failure. On the basis of this investigation, Walraven (1987) recommended to use a different expression instead of the expression employing the decompression moment. Afterwards, this expression was replaced with different approaches in the later version of Model Code. CEB-FIP Model Code 1990 (CEB 1990) introduced the strut-and-tie model and another expression incorporating axial stress due to prestress force. The latest version of Model Code (fib 2010a, 2010b) adopted the MCFT.

In the JSCE specifications, the expression using M_0/M_{ud} has remained the same for more than 20 years since its first introduction. However, the original developer, CEB-FIP Model Code, has been revised several times and no longer employs this expression. Therefore, it is reasonable to suppose that the applicability of the parameter β_n should be revaluated with a database analysis. A clear understanding of the effect of prestress force is indispensable for estimating the shear strength of prestressed concrete members accurately.

2.4.1.4 Minimum Shear Reinforcement

In JSCE 2007, the minimum amount of shear reinforcement, $A_{w\min}$, is specified as follows:

$$A_{w\min} = 0.0015 b_w s \quad \text{(mm)} \qquad \textbf{\textit{Equation 2-36}}$$

where:

b_w = web width

s = spacing of shear reinforcement

It is interesting to note that the minimum amount of shear reinforcement in JSCE 2007 is not related to the compressive strength of concrete and the yield strength of shear reinforcement. This approach is completely different from those in North American and European shear design provisions.

2.4.2 JRA Specifications for Highway Bridges Part III Concrete Bridges

In Japan, highway bridges are designed and constructed in accordance with the JRA Specifications for Highway Bridges (Japan Road Association 2002), which is referred to as JRA 2002. The basis of the shear design provisions in JRA 2002 is similar to those in JSCE 2007. Therefore, this section provides a brief summary of primary differences between the JRA 2002 specifications and the JSCE 2007 specifications.

- The upper limit on the nominal shear strength in JRA 2002 is defined as the strength of the web concrete against the compression failure, S_{uc}, as follows:

  $$S_{uc} = \tau_{\max} b_w d + S_p \qquad \text{(MPa, mm, N)} \qquad \textbf{\textit{Equation 2-37}}$$

  where:

  τ_{max} = maximum value of average shear strength of concrete as shown in Table 2.4

  b_w = web width

  d = effective depth

  S_p = vertical component of effective prestress force

Table 2.4 Maximum value of average shear strength of concrete

Specified concrete strength (MPa)	21	24	27	30	40	50	60
τ_{max} (MPa)	2.8	3.2	3.6	4.0	5.3	6.0	6.0

- In JRA 2002, the nominal shear strength is calculated by adding the V_c, V_s, and V_p terms. The V_s and V_p terms are calculated by using the same equations as those in JSCE 2007. The V_c term is given as S_c by the following equation:

$$S_c = k\tau_c b_w d \qquad \text{(MPa, mm, N)} \qquad \textbf{\textit{Equation 2-38}}$$

where:

k = $1+\dfrac{M_0}{M_d} \le 2$

τ_c = average shear strength of concrete as shown in Table 2.5

b_w = web width

d = effective depth

M_0 = decompression moment necessary to cancel stress due to axial force at extreme tension fiber

M_d = flexural moment at section

Table 2.5 Average shear strength of concrete

Specified concrete strength (MPa)	21	24	27	30	40	50	60
τ_c (MPa)	0.36	0.39	0.42	0.45	0.55	0.65	0.70

It should be noted that there is a minor difference in expressions that account for the effect of prestress force between the two specifications. The flexural moment at section, M_d, is employed in the formula of the parameter k in the JRA specifications. The flexural moment capacity, M_{ud}, is used in the equation of the parameter β_n in the JSCE specifications. The latter equation is employed as an approximation of the former formula.

- In JRA 2002, the minimum amount of shear reinforcement, $A_{w\min}$, perpendicular to a longitudinal axis of a member is specified as follows:

For deformed reinforcement:

$$A_{w\min} = 0.002 b_w s \qquad \text{(mm)} \qquad \textbf{\textit{Equation 2-39}}$$

For plain reinforcement:

$$A_{w\min} = 0.003 b_w s \qquad \text{(mm)} \qquad \textbf{\textit{Equation 2-40}}$$

where:

b_w = web width

s = spacing of shear reinforcement

Note that these values are larger than that in JSCE 2007.

It should be noted that the origin of the maximum value of average shear strength of concrete in Table 2.4 and the average shear strength of concrete in Table 2.5 are based on CEB-FIP Model Code 78 (CEB 1978). However, these values are not employed in the latest Model Code (fib 2010a, 2010b).

Additionally, several upper limits on material strengths are provided in JRA 2002. The compressive strength of concrete is limited up to 8.7 ksi (60 MPa) for shear design, and the yield strength of nonprestressed reinforcement is restricted up to 50 ksi (345 MPa).

2.5 SHEAR DESIGN PROVISIONS: EUROPE

This section reviews the shear design provisions recently adopted in Europe:

- fib Model Code 2010 (fib 2010a, 2010b); and
- EN 1992 Eurocode 2: Design of Concrete Structures (European Committee for Standardization 2004).

2.5.1 fib Model Code 2010

As mentioned previously, CEB-FIP Model Code 78 (CEB 1978) adopted the shear design equation employing the decompression moment to account for the effect of prestress force. Model Code 1990 (CEB 1990) introduced two different approaches: (1) the strut-and-tie model; and (2) the equation incorporating the axial stress due to prestress force. Afterwards, Model Code 2010 (fib 2010a, 2010b) employs shear design equations based on the MCFT. The equations in Model Code 2010 are functionally equivalent to those in AASHTO LRFD 2010 and CSA A23.3-04. Therefore, this section summarizes primary differences in shear design provisions between Model Code 2010 and AASHTO LRFD 2010.

- Model Code 2010 incorporates three levels of calculation procedures depending on their complexity and accuracy. The highest level, Level III, employs almost the same equations as those in AASHTO LRFD 2010 and CSA A23.3-04. Other two levels adopt simple equations derived by making several assumptions on the equations of the Level III procedure.

- Model Code 2010 specifies an upper limit on the shear strength, $V_{Rd,max}$, as follows:

$$V_{Rd,\max} = k_c \frac{f_{ck}}{\gamma_c} b_w z \frac{\cot\theta + \cot\alpha}{1+\cot^2\theta} \qquad \text{(MPa, mm, N)} \qquad \textbf{\textit{Equation 2-41}}$$

where:

k_c = $0.55\left(\frac{30}{f_{ck}}\right)^{1/3} \le 0.55$

f_{ck} = characteristic value of cylinder compressive strength of concrete, f'_c, derived from strength test by the criterion that 5% of measurements for the specified concrete is assumed to be below the value f_{ck}

γ_c = partial safety factor for concrete material properties, shall be taken as 1.2 or 1.5

b_w = breadth of web

z = internal lever arm

θ = inclination of compression stresses

α = inclination of shear reinforcement relative to member axis

- Model Code 2010 calculates the longitudinal strain at the mid-depth of a member, ε_x, to estimate the V_c term. This procedure is same as that in CSA A23.3-04 but different from that in AASHTO LRFD 2010.

- The equivalent crack spacing parameter, k_{dg}, in Model Code 2010 is calculated by taking aggregate size, a_g, as zero when f'_c exceeds 10 ksi (70 MPa) and reducing a_g linearly to zero when f'_c increases from 9.3 ksi (64 MPa) to 10 ksi (70 MPa).

- Model Code 2010 specifies the minimum amount of shear reinforcement, $A_{sw,\min}$, as shown in Equation 2-42. The coefficient, 0.12, turns into 1.45 once material strengths are expressed in psi. The minimum shear reinforcement in Model Code 2010 is greater than those in ACI 318-08, AASHTO LRFD 2010, and CSA A23.3-04.

$$A_{sw,\min} = 0.12\sqrt{f_{ck}}\,\frac{b_w s_w}{f_{yk}} \qquad \text{(MPa, mm)} \qquad \textbf{\textit{Equation 2-42}}$$

where:

f_{ck} = characteristic value of cylinder compressive strength of concrete, f'_c, derived from strength test by the criterion that 5% of measurements for the specified concrete is assumed to be below the value f_{ck}

b_w = breadth of web

s_w = spacing of shear reinforcement

f_{yk} = characteristic value of yield strength of shear reinforcement

In Model Code 2010, the compressive strength of concrete is limited to 17 ksi (120 MPa) for normal concrete and 12 ksi (80MPa) for light weight concrete.

2.5.2 EN 1992 Eurocode 2: Design of Concrete Structures

The EN 1992 Eurocode 2: Design of Concrete Structures, which is referred to as EN 1992, specifies two methods to calculate the shear strength depending on the requirement for shear reinforcement. For members not requiring shear reinforcement, the shear strength is provided only by the V_c term. In contrast, for members requiring shear reinforcement, the shear strength consists only of the V_s term. It is important to appreciate the fact that the V_c term is ignored in members requiring shear reinforcement.

Cladera and Marí (2007) investigated the accuracy of the shear equations in EN 1992 by using existing test results. The researchers revealed that the shear strength calculated by EN 1992 is too conservative for prestressed concrete members, and concluded that the EN 1992 shear design equations seem to be an oversimplification.

Additionally, EN 1992 limits the compressive strength of concrete for design up to 13 ksi (90 MPa).

2.5.2.1 Members not requiring design shear reinforcement

In EN 1992, for members without shear reinforcement, the design value of the shear strength, $V_{Rd,c}$, is given in:

$$V_{Rd,c} = \left(C_{Rd,c} k \left(100 \rho_l f_{ck}\right)^{1/3} + k_1 \sigma_{cp}\right) b_w d \geq \left(v_{\min} + k_1 \sigma_{cp}\right) b_w d$$

(MPa, mm, N) ***Equation 2-43***

where:

$C_{Rd,c}$ = $0.18 / \gamma_c$

k	=	$1+\sqrt{\frac{200}{d}} \leq 2.0$
ρ_l	=	$\frac{A_{sl}}{b_w d} \leq 0.02$
f_{ck}	=	characteristic compressive cylinder strength of concrete at 28 days
k_1	=	0.15
σ_{cp}	=	compressive stress in concrete from axial force or prestress force
		$\frac{N_{Ed}}{A_c} \leq 0.2 f_{cd}$
b_w	=	the smallest width of cross section in tensile area
d	=	effective depth of a cross section
v_{min}	=	$0.035k^{3/2}f_{ck}^{1/2}$
γ_c	=	partial factor for concrete, shall be taken as 1.2 or 1.5
A_{sl}	=	area of tensile reinforcement
N_{Ed}	=	axial force in cross section due to loading or prestress force
A_c	=	area of concrete cross section
f_{cd}	=	design value of concrete compressive strength

Additionally, in prestressed single span members without shear reinforcement, the shear strength of regions uncracked in bending is calculated by:

$$V_{Rd,c} = \frac{Ib_w}{S}\sqrt{\left(f_{ctd}\right)^2 + \alpha_l \sigma_{cp} f_{ctd}}$$ (MPa, mm, N) ***Equation 2-44***

where:

I	=	second moment of area	
b_w	=	web width	
S	=	first moment of area above and about the centroidal axis	
f_{ctd} =	=	design tensile strength of concrete	
α_l	=	$l_x / l_{pt2} \leq 1.0$	for pretensioned members
	=	1.0	for other types of prestressing

σ_{cp} = compressive stress in concrete from axial force or prestress force

l_x = distance of section considered from the starting point of the transmission length

l_{pt2} = upper bound value of the transmission length of prestressing element

Both Equations 2-43 and 2-44 provide the concrete contribution to the shear strength. The former equation predicts the shear strength at the formation of the flexure shear crack, and the latter serves as a limit on the web shear crack in regions without the flexural crack.

It should be noted that EN 1992 is intended to be used in European countries. Because different countries use different shear design provisions, EN 1992 includes "National Annex", where each country has a right to set own values depending on their own decision. In Equations 2-43 and 2-44, the values of $C_{Rd,c}$, k_1, and v_{min} are regarded as the National Annex.

2.5.2.2 Members requiring design shear reinforcement

For members having shear reinforcement, the design value of the shear strength, V_{Rd}, is the lesser of the shear strength provided by shear reinforcement, $V_{Rd,s}$, and the maximum shear strength limited by crushing of the compression strut, $V_{Rd,max}$. These two shear strengths are given in:

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot\theta \qquad \text{(MPa, mm, N)} \qquad \textbf{\textit{Equation 2-45}}$$

$$V_{Rd,\max} = \frac{\alpha_{cw} b_w z v_1 f_{cd}}{\cot\theta + \tan\theta} \qquad \text{(MPa, mm, N)} \qquad \textbf{\textit{Equation 2-46}}$$

where:

$1 \leq \cot\theta \leq 2.5$

where:

A_{sw} = area of shear reinforcement

s	=	spacing of shear reinforcement
z	=	inner lever arm, for a member with constant depth, corresponding to bending moment in the element under consideration, $0.9d$ for shear analysis of reinforced concrete without axial force
f_{ywd}	=	design yield strength of shear reinforcement
θ	=	angle between concrete compression strut and beam axis perpendicular to shear force
α_{cw}	=	coefficient taking account of the state of stress in compression chord

1	for nonprestressed members
$1 + \sigma_{cp}/f_{cd}$	for $0 < \sigma_{cp} \leq 0.25f_{cd}$
1.25	for $0.25f_{cd} < \sigma_{cp} \leq 0.5f_{cd}$
$2.5(1 - \sigma_{cp}/f_{cd})$	for $0.5f_{cd} < \sigma_{cp} \leq f_{cd}$

b_w	=	minimum width between tension and compression chords
v_1	=	strength reduction factor for concrete cracked in shear

0.6	when design stress of shear reinforcement is below 80% of characteristic yield strength and $f_{ck} \leq 60$ MPa
$0.6\left(1-\dfrac{f_{ck}}{250}\right)$	recommended value, and when design stress of shear reinforcement is below 80% of characteristic yield strength and $f_{ck} \geq 60$ MPa

f_{cd}	=	design value of concrete compressive strength
d	=	effective depth of cross section
σ_{cp}	=	compression stress in concrete from axial force or prestress force
f_{ck}	=	characteristic compressive cylinder strength of concrete at 28 days

When there is a vertical component of prestress force, it is added to $V_{Rd,s}$ and supposed to contribute to the shear strength. The values of v_1 and α_{cw} are regarded as the National Annex.

It should be emphasized that in cases when $V_{Rd,s}$ is lower than $V_{Rd,max}$, namely the web crushing failure is not assumed to happen, the shear strength is provided only by the shear reinforcement contribution. Additionally, an angle of inclination of the compressive strut to the member axis, θ, is arbitrarily decided by the engineers between 22 degree ($\cot\theta = 1.0$) and 45 degree ($\cot\theta = 2.5$).

2.5.2.3 Effect of Prestress Force

In the members having no shear reinforcement, the effect of prestress force is incorporated directly in both Equations 2-43 and 2-44. This implies that the concrete contribution is supposed to be increased by prestress force regardless of the type of inclined cracks. In contrast, in the members having shear reinforcement, only the vertical component of prestress force is supposed to contribute to the shear strength. The shear reinforcement contribution calculated by Equation 2-45 is not a function of the level of prestress force and the angle of the compressive strut is arbitrarily decided. It seems reasonable to think that Equation 2-45 provides significantly conservative estimations of the shear strength in prestressed concrete members with sufficient shear reinforcement.

2.5.2.4 Minimum shear reinforcement

EN 1992 specifies the minimum amount of shear reinforcement as shown in Equation 2-47. The coefficient, 0.08, is to be replaced with 0.96 if the material strengths are expressed in psi. The minimum shear reinforcement in EN 1992 is smaller than that in Model Code 2010.

$$A_{sw,\min} = 0.08\sqrt{f_{ck}}\,\frac{b_w s}{f_{yk}} \qquad \text{(MPa, mm)} \qquad \textbf{\textit{Equation 2-47}}$$

where:

f_{ck} = characteristic compressive cylinder strength of concrete at 28 days

b_w = web width

s = spacing of shear reinforcement

f_{yk} = characteristic value of yield strength of shear reinforcement

2.6 SUMMARY OF SEAR DESIGN PROVISIONS

An examination of the existing shear design provisions is given in this chapter. This critical review reveals that there are substantial differences in the shear design provisions among design codes, organizations, and countries. A brief summary of characteristics of the shear design provisions that are available in North America, Japan, and Europe is presented in Table 2.6. The primary differences among these shear design provisions are summarized as follows:

- The design codes in North America, Japan, and Europe adopt different shear design equations that are based on various shear design philosophies, such as the modified truss model, the MCFT, and the variable angle truss method.

- The existing shear design provisions incorporate the effect of prestress force in completely different manners. The MCFT-based shear design equations use the initial strain of prestressing steel in the calculation of the longitudinal tensile strain of members. The value of the longitudinal tensile affects both the V_c and V_s terms. Several North American shear design equations consider the effect of prestress force only in the V_c term. These equations' V_c terms are based on Mohr's circle analyses or empirical derivations. Japanese shear design equations take into account the effect of prestress force by using the decompression moment in the V_c term.

- The existing shear design provisions employ different expressions to limit the maximum shear strength even though all of these expressions are aimed at preventing the same failure mode; crushing of the web prior to yielding of shear reinforcement.

- The existing shear design provisions adopt different minimum shear reinforcement requirements. Although most of the shear design provisions define the minimum amount of shear reinforcement as functions of the compressive strength of concrete and/or the yield strength of shear reinforcement, their governing coefficients are completely different.

2.7 RESEARCH SIGNIFICANCE

Although development of a shear database for prestressed concrete members has already been started by several research groups, existing databases still have room for improvement in the collection of test results. Additionally, because of the complex nature of the shear behavior of structural concrete, there is no internationally accepted method to estimate the shear strength of reinforced and/or prestressed concrete members. As a result, substantially different shear design provisions are adopted in various design codes around the world.

As mentioned earlier, this thesis aims to produce a more complete shear database for prestressed concrete members by including test results from literature published in North America, Japan, and Europe. This new database, which is referred to as UTPCSDB-2011, is subsequently used to perform a critical evaluation of the existing shear design provisions for prestressed concrete members. This database is also utilized to obtain a clear understanding of the shear behavior of prestressed concrete members with sufficient shear reinforcement.

Table 2.6 Characteristics of shear design provisions (1/2)

Shear design equation	*Design philosophy*	*Effect of prestress froce*	*Minimum shear reinforcement (psi, in.)*
ACI 318-08 *Simplified Method*	✓ Modified truss model V_c: empirical equation V_s: 45 degree truss model ✓ Upper limit: $V_c + 8\sqrt{f'_c}b_w d$	● Applicable when $f_{se} \geq 0.4 f_{pu}$ (Not incorporated directly in V_c)	The lesser of $0.75\sqrt{f'_c}\dfrac{b_w s}{f_{yt}}$, $\dfrac{50 b_w s}{f_{yt}}$, and $\dfrac{A_{ps} f_{pu} s}{80 f_{yt} d}\sqrt{\dfrac{d}{b_w}}$ when $f_{se} \geq 0.4 f_{pu}$
ACI 318-08 *Detailed Method*	✓ Modified truss model V_c: the lesser of V_{ci} and V_{cw} V_s: 45 degree truss model ✓ Upper limit: $V_c + 8\sqrt{f'_c}b_w d$	● Incorporated in V_{ci} and V_{cw}	
AASHTO LRFD 2010 *General Procedure*	✓ MCFT ✓ Upper limit: $0.25 f'_c b_v d_v + V_p$	● Incorporated in ε_x	$\sqrt{f'_c}\dfrac{b_v s}{f_y}$
AASHTO LRFD 2010 *Simplified Procedure*	✓ Modified truss model V_c: the lesser of V_{ci} and V_{cw} V_s: variable angle truss method ✓ Upper limit: $0.25 f'_c b_v d_v + V_p$	● Incorporated in V_{ci}, V_{cw}, and V_s	
AASHTO LRFD 2010 *Segmental Procedure*	✓ Modified truss model V_c: empirical equation (based on Mohr's circle) V_s: 45 degree truss model ✓ Upper limit: $0.379\sqrt{f'_c}b_v d_v$	● Incorporated in V_c	$\dfrac{50 b_v s}{f_y}$
AASHTO Standard Specifications 2002	✓ Same as the ACI 318-08 Detailed Method		$\dfrac{50 b_w s}{f_y}$

Table 2.6 Characteristics of shear design provisions (2/2)

Shear design equation	*Design philosophy*	*Effect of prestress force*	*Minimum shear reinforcement (psi, in.)*
CSA A23.3-04	✓ MCFT ✓ Upper limit: $0.25f'_c b_w d_v + V_p$	● Incorporated in ε_s	$0.72\sqrt{f'_c}\dfrac{b_w s}{f_y}$
JSCE 2007	✓ Modified truss model V_c: empirical equation (slender beam, deep beam) V_s: 45 degree truss model ✓ Upper limit: $1.25\sqrt{f'_{cd}}b_w d \le 7.8 b_w d$	● Expressed by M_0/M_u	$0.0015 b_w s$
JRA 2002	✓ Modified truss model V_c: empirical equation V_s: 45 degree truss model ✓ Upper limit: $\tau_{\max} b_w d + S_p$	● Expressed by M_0/M_d	Deformed steel: $0.002 b_w s$ Plain steel: $0.003 b_w s$
fib Model Code 2010	✓ MCFT ✓ Upper limit: $k_c \dfrac{f_{ck}}{\gamma_c} b_w z \dfrac{\cot\theta + \cot\alpha}{1+\cot^2\theta}$	● Incorporated in ε_s	$1.45\sqrt{f'_{ck}}\dfrac{b_w s_w}{f_{yk}}$
EN 1992	✓ w/o min. shear reinforcement V_c: empirical equation ✓ w/ min. shear reinforcement V_s: variable angle truss method ✓ Upper limit: $\dfrac{\alpha_{cw} b_w z v_1 f_{cd}}{\cot\theta + \tan\theta}$	● Incorporated in V_c (w/o min. shear reinforcement) ● Not incorporated (w/ min. shear reinforcement)	$0.96\sqrt{f'_{ck}}\dfrac{b_w s}{f_{yk}}$

CHAPTER 3

DATABASE DEVELOPMENT

3.1 OVERVIEW

The development procedure of the UTPCSDB-2011 is presented in this chapter. The comprehensive literature survey that was conducted to develop the UTPCSDB-2011 is explained in detail. A list of investigated journals and reports, the collection criteria, and the format of the UTPCSDB-2011 are discussed.

As a result of the comprehensive literature survey, a total of 1,696 tests from 99 references were compiled in the collection database. The filtering criteria that were used to construct the filtered database and the evaluation database are also explained later in this chapter.

3.2 THE UNIVERSITY OF TEXAS PRESTRESSED CONCRETE SHEAR DATABASE

The first version of the University of Texas Prestressed Concrete Shear Database (UTPCSDB) was developed by Avendaño and Bayrak (2008). This first version of the UTPCSDB contains 506 tests reported in 30 references from 1954 to 2008. 367 of the 506 specimens failed in shear, and 153 of those 367 specimens had member heights of larger than 12 in. and contained shear reinforcement. Most of the references used to assemble the first version of the UTPCSDB were from U.S. literature.

The first version of the UTPCSDB was utilized to evaluate the accuracy of the current U.S. shear design provisions. Avendaño and Bayrak (2008) proposed revisions to the current provisions for the minimum shear reinforcement and the maximum shear strength on the basis of their database analysis.

3.3 COLLECTION OF SHEAR TEST RESULTS

3.3.1 Literature Survey

Since most of the references in the first version of the UTPCSDB are from U.S. literature, this thesis aims to expand the database by including test results published in North America, Japan, and Europe. Table 3.1 presents a list of investigated journals and reports. In addition to these documents, student theses and dissertations, textbooks, proceedings of international symposiums, and research reports published by universities, research institutes, and U.S. Departments of Transportation were examined. This literature survey was focused on documents published prior to December 2010.

3.3.2 Collection Criteria

It is important to note that no document was intentionally ignored in this literature survey. The collection of papers and research reports was conducted in accordance with the following criteria:

- Any types of cross section, concrete, and prestressing;
- Both specimens with and without shear reinforcement;
- Specimens with fiber reinforced concrete were not included;
- Specimens with FRP reinforcement were not included;
- No limitation on support and loading conditions; and
- No limitation on the size and geometry of test specimens.

The collection database is defined as a database that contains all tests reported in the collected documents. It should be noted that some documents report tests that are not relative to the primary objectives of this thesis, such as nonprestressed members, failure modes other than shear failure, and tests missing crucial information for the database analysis. These irrelevant tests were filtered out by using filtering criteria.

Table 3.1 List of investigated journals and reports

No.	***Title of journals and reports***	***Organization, country***
1	ACI Journal Proceedings	American Concrete Institute, U.S.
2	ACI Structural Journal	American Concrete Institute, U.S.
3	ACI Special/Symposium Publication	American Concrete Institute, U.S.
4	Journal of Japan Society of Civil Engineers	Japan Society of Civil Engineers, Japan
5	Journal of Prestressed Concrete	Japan Prestressed Concrete Engineering Association, Japan
6	Magazine of Concrete Research	Institution of Civil Engineers, U.K.
7	Materials and Structures	RILEM, France
8	NCHRP Reports	Transportation Research Board of the National Academies, U.S.
9	PCA research reports	Portland Cement Association, U.S.
10	PCI Journal	Precast/Prestressed Concrete Institute, U.S.
11	Proceedings of Annual Conference of the Japan Society of Civil Engineers	Japan Society of Civil Engineers, Japan
12	Proceedings of Concrete Research and Technology	Japan Concrete Institute, Japan
13	Proceedings of the Concrete Structure Scenarios	The Society of Materials Science, Japan
14	Proceedings of the Institution of Civil Engineers	Institution of Civil Engineers, U.K.
15	Proceedings of the Japan Prestressed Concrete Engineering Association	Japan Prestressed Concrete Engineering Association, Japan
16	Transactions of the Japan Cement Association	Japan Cement Association, Japan
17	Transactions of the Japan Concrete Institute	Japan Concrete Institute, Japan
18	Transactions of the Japan Prestressed Concrete Engineering Association	Japan Prestressed Concrete Engineering Association, Japan

Note: Student theses and dissertations, textbooks, proceedings of international symposiums, and research reports published by universities, research institutes, and U.S. Departments of Transportation were also reviewed.

3.3.3 Database Format

The format of the UTPCSDB-2011 is based on that of the first version of the UTPCSDB. The content of the UTPCSDB-2011 is presented in Table 3.2. Note that all units were converted to the U.S. customary units. The information shown in Table 3.2 was extracted from the original references. Other important data, such as the cross sectional area, moment of inertia, reinforcement ratio, shear span to depth ratio, and so on, were calculated by using reported values shown in Table 3.2.

There were some documents that failed to report sufficient information on material properties and specimen geometries. In the case of these documents, the values that were used to populate into the UTPCSDB-2011 were calculated as follows:

- If the compressive strength of concrete was determined by testing cubes, the cylinder strength of concrete was taken as 80% of that of concrete cubes.

- If the concrete strength tested at the test day was not available, the concrete strength tested at 28 days was used to populate into the UTPCSDB-2011. If the concrete strength tested at 28 days was not reported, the design strength was used in the UTPCSDB-2011.

- If the yield and/or tensile strengths of prestressing steel were not tested, specified strengths were used. If the specified strengths were not available, other documents published by the same research group and/or organization were used. The following relationships were also used to obtain some of the calculated values (Collins and Mitchell 1997).

Low-relaxation strand	: f_{py}/f_{pu} =	0.90
Stress-relieved strand	: f_{py}/f_{pu} =	0.85
Plain prestressing bars	: f_{py}/f_{pu} =	0.85
Deformed prestressing bars	: f_{py}/f_{pu} =	0.80

- If the yield strength of reinforcement was not reported, specified strengths were used to populate into the UTPCSDB-2011.

- If there was no information on effective prestress force, prestress losses were estimated to determine effective prestress force at the test day.

- If the maximum aggregate size was not reported, documents published by the same research group and/or organization were used. If it was still difficult to decide on the assumed values, the maximum aggregate size was taken as 0.75 in. (19 mm). This assumption agrees with that in the shear database for reinforced concrete members (Collins, et al. 2008).

- If there was no sufficient information on the size and geometry of specimens, figures in the original documents were utilized to determine the geometry.

Table 3.2 List of primary contents in database (1/2)

Contents	Parameters	Descriptions
Geometry	Cross section type	Box, I, rectangular, T or U shaped section
	Height, h (in.)	
	Web width, b_w (in.)	
	Effective depth, d (in.)	
	Flange width, b_f (in.)	Top and bottom flanges
	Flange thickness, t_f (in.)	Top and bottom flanges
	Shear span, a (in.)	
	Clear span, l_s (in.)	
	Deck width, b_{deck} (in.)	Composite members
	Deck thickness, t_{deck} (in.)	Composite members
Concrete (Girder, Deck)	Compressive strength, f'_c (psi)	Cylinder strength tested at the test day
	Tensile strength, f_t (psi)	Tested at the test day
	Concrete type	Normal, self-consolidating, lightweight, etc.
	Aggregate size, a_g (in.)	
	Aggregate type	
Prestressing steel (Bottom, Top)	Area, A_p (in.^{2})	Effect of debonded and/or draped strand
	Depth, d_p (in.)	Effect of debonded and/or draped strand
	Diameter (in.)	
	Tensile strength, f_{pu} (ksi)	
	Yield strength, f_{py} (ksi)	
	Draped strand fraction	
	Draped strand angle, α (degree)	
	Effective prestress, f_{se} (ksi)	
	Initial prestress, f_{po} (ksi)	
	Prestress loss (%)	
Reinforcing bar (Bottom, Top)	Area, A_s (in.^{2})	
	Depth, d_s (in.)	
	Diameter (in.)	
	Yield strength, f_y (ksi)	

Table 3.2 List of primary contents in database (2/2)

Contents	*Parameters*	*Descriptions*
Shear Reinforcement (Vertical, Horizontal)	Area, A_v (in.^{2})	
	Spacing, s (in.)	
	Yield strength, f_y (ksi)	
	Type	Plain steel, deformed steel, WWF, etc.
Concrete Stress	Due to effective prestress (psi)	Top, centroid, and bottom of cross section
Load	Shear at shear crack (kips)	
	Shear at flexural crack (kips)	
	Shear at failure (kips)	
Failure mode	Shear, flexure, etc.	Failure mode reported by authors in the original references
Loading Type	Concentrated or uniform load	
Support Condition	Simple or continuous	
	Plate dimension (in.)	Loading plate, bearing plate
End Condition	w/ or w/o end block	
Prestressing type	Pretensioned, post-tensioned, externally post-tensioned	
Specimen Type	Segmental or non-segmental	
Calculated Capacity	Calculated shear strength	Calculated capacity reported by authors in the original references

Note: unit conversion 1 in. = 25.4 mm
1 kips = 4.448 kN
1 ksi = 6.895 MPa

3.4 COLLECTION DATABASE

A total of 1,696 tests reported in 99 references from 1954 to 2010 were stored in the collection database. A list of all collected references in the UTPCSDB-2011 is presented in Table 3.3. This table contains the number of tests and descriptions of specimens reported in each reference. Again, it should be noted that the collection database contains all test results reported in these references. The collection database was filtered out to construct the filtered database and the evaluation databases by using several filtering criteria.

Table 3.3 List of references in UTPCSDB-2011 (1/5)

No.	***Authors***	***Year***	***Number of tests***	***Special descriptions***
1	Alshegeir and Ramirez	1992	3 (3)	AASHTO type I and II girder
2	Arthur	1965	55 (39)	
3	Arthur, et al.	1973	19 (19)	Uniform load
4	Avendaño and Bayrak	2008	4 (4)	TxDOT girder, Composite section
5	Bennett and Balasooriya	1971	26 (20)	
6	Bennett and Debaiky	1974	33 (22)	
7	Bruce	1962	24 (20)	Composite section
8	Bruggeling, et al.	1978	9 (7)	
9	Burgueño and Bendert	2007	4 (1)	Box beam, Self consolidating concrete
10	Cederwall, et al.	1974	33 (26)	
11	Choulli, et al.	2008	12 (12)	Self-compacting concrete
12	Cumming, et al.	1998	4 (3)	MnDOT girder, Composite section
13	De Silva, et al.	2006	7 (4)	
14	Dill	2000	8 (3)	AASHTO type II girder, Composite section, High performance concrete
15	Durrani and Robertson	1987	13 (10)	Unbond strand, wwf
16	Dymond	2007	2 (1)	AASHTO/PCI BT-53 girder, Composite section, Lightweight concrete Self-consolidating concrete,
17	Elzanaty, et al.	1987	34 (34)	
18	Evans and Schumacher	1963	54 (40)	
19	Funakoshi and Okamoto	1979	8 (8)	
20	Funakoshi and Okamoto	1981	20 (16)	
21	Funakoshi, et al.	1982	40 (28)	
22	Funakoshi, et al.	1984	17 (10)	
23	Gregor and Collins	1995	6 (2)	Uniform load, Continuous beam

Table 3.3 List of references in UTPCSDB-2011 (2/5)

No.	*Authors*	*Year*	*Number of tests*	*Special descriptions*
24	Haines	2005	3 (1)	AASHTO/PCI BT-56 girder, Composite section, High performance concrete
25	Hamada, et al.	1999	6 (5)	Lightweight concrete
26	Hamilton III, et al.	2009	11 (2)	AASHTO type III and IV girder, Composite section
27	Hanson and Hulsbos	1964	35 (35)	Uniform load
28	Hanson and Hulsbos	1969	9 (6)	Box beam
29	Hartman, et al.	1988	10 (10)	Composite section
30	Hawkins and Kuchma	2007	20 (17)	Composite section, Uniform load
31	Hawkins, et al.	1961	24 (11)	Continuous beam
32	Heckmann	2008	18 (18)	TxDOT girder
33	Hernandez	1958	38 (11)	Composite section
34	Hicks	1958	21 (18)	
35	Horibe and Ueda	1986	9 (7)	
36	Hosoda, et al.	2002	15 (12)	Externally post-tensioned
37	Hovell, et al.	2010	32 (27)	Box beam, U beam, wwf
38	Imano, et al.	2001	6 (2)	
39	Ito, et al.	1996	6 (4)	Lightweight concrete
40	Ito, et al.	1997	12 (6)	Externally post-tensioned, Segmental beam
41	Kang, et al.	1989	42 (26)	
42	Kar	1969	47 (44)	Uniform load
43	Kaufman and Ramirez	1987	6 (4)	AASHTO type I and II girder
44	Kobayashi and Nieda	1991	15 (6)	
45	Kondo, et al.	1994	4 (1)	Externally post-tensioned
46	Kuroda, et al.	2001	5 (4)	Blast load

Table 3.3 List of references in UTPCSDB-2011 (3/5)

No.	Authors	Year	Number of tests	Special descriptions
47	Labonte and Hamilton III	2005	12 (5)	AASHTO type II girder, Self-consolidating concrete
48	Laskar, et al.	2007	7 (5)	TxDOT girder
49	Lee, et al.	2010	11 (7)	
50	Lin	1955	4 (2)	Continuous beam
51	Lyngberg	1976	9 (6)	
52	Ma, et al.	2000	5 (4)	Composite section, wwf
53	MacGregor, et al.	1960a	22 (17)	
54	MacGregor, et al.	1960b	67 (18)	Composite section, Moving load
55	Magnel	1954	1 (1)	Continuous beam
56	Mahgoub	1975	25 (14)	
57	Maruyama and Rizkalla	1988	11 (9)	wwf
58	Mattock and Kaar	1961	15 (14)	Composite section, Continuous beam
59	Meyer	2002	18 (6)	AASHTO type II girder, Lightweight concrete
60	Mikata, et al.	2001	24 (16)	
61	Mitamura, et al.	2001	3 (2)	Lightweight concrete, Externally post-tensioned
62	Moayer and Regan	1974	34 (16)	
63	Morice and Lewis	1955	28 (1)	Continuous beam
64	Muguruma, et al.	1983	8 (3)	
65	Naito, et al.	2005	6 (2)	PennDOT girder, Self consolidating concrete
66	Nakamura, et al.	2009	2 (1)	Corroded beam
67	Nguyen, et al.	2010	7 (7)	Externally post-tensioned, Segmental beam
68	Niitsu, et al.	1999	4 (3)	Externally post-tensioned
69	Nunnally	2005	3 (2)	Self-consolidating concrete
70	Okada and Toyofuku	1982	28 (14)	

Table 3.3 List of references in UTPCSDB-2011 (4/5)

No.	***Authors***	***Year***	***Number of tests***	***Special descriptions***
71	Okada, et al.	1980	18 (13)	
72	Olesen, et al.	1967	20 (3)	
73	Public Works Research Institute	1995	25 (12)	
74	Ramirez and Aguilar	2005	4 (2)	AASHTO type I girder
75	Ramirez, et al.	2000	4 (4)	AASHTO type I girder, Lightweight concrete
76	Rangan	1991	16 (12)	
77	Raymond, et al.	2005	6 (4)	Composite section, wwf
78	Runzell, et al.	2007	2 (2)	MnDOT girder, Composite section
79	Sakurada, et al.	2001	5 (3)	Lightweight concrete
80	Saqan and Frosch	2009	9 (7)	
81	Sato, et al.	1987	51 (38)	
82	Sethunarayanan	1960	32 (30)	
83	Shahawy and Batchelor	1996	40 (24)	AASHTO type II girder, Composite section
84	Sivaleepucth, et al.	2009	4 (4)	Externally post-tensioned, Segmental beam
85	Sivaleepunth, et al.	2007	4 (4)	Externally post-tensioned
86	Sozen, et al.	1959	99 (76)	
87	Takagi, et al.	2000	6 (2)	Externally post-tensioned
88	Tamura, et al.	2001	4 (4)	Lightweight concrete
89	Tan and Mansur	1992	8 (3)	
90	Tan and Ng	1998	7 (2)	Externally post-tensioned
91	Tan, et al.	1999	12 (8)	
92	Tawfiq	1995	12 (12)	AASHTO type II girder, Composite section
93	Teng, et al.	1998	21 (17)	
94	Teoh, et al.	2002	10 (6)	
95	Toyofuku	1984	30 (24)	
96	Watanabe, et al.	2003	12 (5)	Lightweight concrete

Table 3.3 List of references in UTPCSDB-2011 (5/5)

No.	***Authors***	***Year***	***Number of tests***	***Special descriptions***
97	Xuan, et al.	1998	6 (6)	wwf
98	Zekaria	1958	12 (6)	Continuous beam
99	Zwoyer and Siess	1954	34 (29)	

Note1: Collection database: 1,696 tests
Filtered database: 1,146 tests

Note2: The number of tests in the collection database is shown in the column "Number of tests". The number of tests in the filtered database is shown in parentheses in the column "Number of tests".

Note3: Some tests in No.7, 53, 54, and 72 overlapped. The column "Number of tests" shows the number of tests after resolving duplication of tests. Therefore, the number of tests of these references shown in the column "Number of tests" is different from that reported in the original references.

3.5 FILTERED DATABASE

The filtered database is defined as a database that consists of the shear tests conducted on prestressed concrete members. Thus, as a part of the filtering process, tests that were irrelevant to the shear behavior of prestressed concrete members were removed.

The filtering criteria shown in Table 3.4 were employed to construct the filtered database. According to the definition of the filtered database, 417 specimens whose failure modes were other than shear failure and 156 specimens without prestress force were removed. The removed failure modes were flexural failure, bond failure, and bearing failure. Six specimens missing applied failure load, four specimens with initial defects, and seven specimens subjected to moving loads were also filtered out since these test results were deemed to be not applicable to the evaluation of the shear design provisions.

Table 3.4 Filtering criteria for filtered database

Collection Database		***1,696 tests***
Filtering Criteria	- failure modes other than shear failure (flexural failure, bond failure, bearing failure)	- 417 tests
	- nonprestressed member	- 156 tests
	- missing applied load at failure	- 6 tests
	- with initial defects	- 4 tests
	- subjected to moving loads	- 7 tests
Filtered Database		***1,146 tests***

After this filtering process, a total of 1,146 tests were stored in the filtered database. The number of tests from each reference in the filtered database is shown in parentheses in the column "Number of tests" in Table 3.3. Technically, the filtered database is suitable for the evaluation of the shear design provisions. There is, however, still a wide range of experimental parameters in the filtered database. In other words, the filtered database still contains inappropriate tests for the evaluation of the shear design provisions. Therefore, test results in the filtered database were carefully-screened, and the evaluation database was constructed by using more rigorous filtering criteria.

3.6 EVALUATION DATABASE-LEVEL I

The evaluation database consists only of shear tests deemed useful for the evaluation of the shear design provisions. The filtering criteria shown in Table 3.5 were adopted to construct the evaluation database-level I.

Table 3.5 Filtering criteria for evaluation database-level I

Filtered Database		***1,146 tests***
Filtering Criteria	- concrete strength < 4,000 psi (27.6 MPa)	- 162 tests
	- concrete types other than normal concrete	- 59 tests
	- member height < 12.0 in. (305 mm)	- 337 tests
	- shear span to depth ratio < 2.0 (if member was subjected to concentrated load)	- 119 tests
	- insufficient amount of shear reinforcement (less than the minimum shear reinforcement of the AIC 318-08 code)	- 644 tests
	- insufficient amount of shear reinforcement (less than the minimum shear reinforcement of the AASHTO LRFD 2010 specifications)	- 631 tests
	- continuous beams	- 37 tests
	- segmental specimens	- 18 tests
	- externally post-tensioned specimens	- 35 tests
Evaluation Database-Level I		***223 tests***

With the filtering criteria shown in Table 3.5, 162 specimens with concrete strengths of lower than 4,000 psi (27.6 MPa) and 337 specimens with member heights of less than 12.0 in. (305 mm) were filtered out. Additionally, specimens having less than the minimum amount of shear reinforcement required in ACI 318-08 and AASHTO LRFD 2010 were removed. These specimens are dissimilar to today's typical prestressed concrete beams used in bridges and buildings, which are built with relatively high strength concrete, have large member heights, and satisfy the minimum shear reinforcement requirements. 119 specimens with shear span to depth ratios lower than 2.0 were also removed because the evaluation of strut-and-tie provisions is beyond the scope of this thesis. Moreover, unique tests that employed continuous beams, segmental specimens, and externally post-tensioned specimens were excluded from the filtered database.

In summary, the evaluation database level-I contains data from 223 tests. The results of the tests conducted on these 223 test specimens are supposed to be informative in examining the accuracy and conservativeness of various shear design provisions. The detailed information of the 223 tests is tabulated in Appendix A.

3.7 EVALUATION DATABASE-LEVEL II

In the evaluation database-level I, all specimens were reported to have failed in shear. However, it should be emphasized that the description of "shear failure" is not adequate to explain how test specimens resist the shear force and lose their shear-carrying capacities. The reason for this is that several different types of shear failure modes were observed in the previous shear tests performed on prestressed concrete members. It is of great significance to focus on differences in shear failure modes to evaluate the shear design provisions. Thus, the UTPCSDB-2011 stores the shear failure modes that were reported in the original references. It is important to note that assumptions were not made in storing the shear failure modes. The shear failure modes in the UTPCSDB-2011 are categorized into the following seven groups:

- Shear failure

  There were test specimens that were difficult to be categorized into the following six shear failure modes since some references provided insufficient information to specify shear failure modes of test specimens. For instance, the use of "shear failure" was not viewed to be sufficient to describe actual shear failure modes. These tests were put into the group of "shear failure" without making any assumptions.

- Flexural shear failure (Figure 3.1 (a))

  Specimens lose their shear-carrying capacities after widening of a flexural shear crack.

- Web crushing failure (Figure 3.1 (b))

  This failure mode is usually observed in specimens with one or two thin webs, such as Box-, I-, T-, and U-beams. These specimens lose their shear-carrying capacities after crushing of the compressive concrete strut in the web.

- Shear compression failure (Figure 3.1 (c))

  Specimens lose their shear-carrying capacities due to crushing of concrete not only in the web but also other parts, such as the top flange.

- Shear tension failure (Figure 3.1 (d))

  Specimens lose their shear-carrying capacities after yielding or subsequent rupture of shear reinforcement.

- Shear failure with signs of horizontal shear damage

  The horizontal sliding of the interface between the web and the bottom flange was reported in full-scale specimens as shown in Figure 3.1 (e) (Hawkins and Kuchma 2007, Avendaño and Bayrak 2008, Hovell, et al. 2010). This group includes two types of specimens: (1) specimens that failed in horizontal shear failure; and (2) specimens that failed in one of other shear failure modes but also showed some signs of horizontal shear damage. For instance, some of those full-scale specimens were reported to have lost their shear carrying-capacities due to both crushing of the web concrete and horizontal shear damage.

- Shear failure with signs of anchorage zone distress

  This category includes specimens that failed in shear but also showed some damages in anchorage regions. These damages involve strand slip and breakdown of bond between strands and concrete. The readers are encouraged to note that specimens that failed in anchorage failure were removed in constructing the filtered database as shown in Table 3.4. In the case of specimens that showed both signs of horizontal shear damage and anchorage zone distress, more critical damage was used to categorize those specimens.

(a) Flexural shear failure (Adopted from Hovell, et al. 2010)

Figure 3.1 Example of shear failure mode (1/2)

(b)Web crushing failure (Adopted from Hovell, et al. 2010)

(c) Shear compression failure

(d) Shear tension failure

(Adopted from Heckmann 2008)

(e) Horizontal shear failure (Adopted from Hovell, et al. 2010)

Figure 3.1 Example of shear failure mode (2/2)

It is reasonable to suppose that the shear design provisions reviewed in Chapter 2 are used to estimate the shear-carrying capacity corresponding to the following five traditional shear failure modes: (1) shear failure; (2) flexural shear failure; (3) web crushing failure; (4) shear compression failure; and (5) shear tension failure. In this thesis, these five shear failure modes are referred to as typical shear failure. On the other hand, it is not fully understood that those shear design provisions are also applicable to test specimens that failed in (1) shear failure where signs of horizontal shear damage appeared and (2) shear failure where signs of anchorage zone distress appeared. The evaluation database-level I includes test specimens that failed in one of these seven shear failure modes, and stores the shear failure mode of each test specimen. This information is supposed to be useful to investigate the influence of shear failure modes on the accuracy and conservativeness of the existing shear design provisions.

Additionally, in order to clarify the accuracy and conservativeness of the shear strength estimations derived from various shear design provisions, the evaluation database-level II was constructed by using filtering criteria shown in Table 3.6. The evaluation database-level II includes only specimens that failed in typical shear failure. 30 specimens that failed in shear failure with signs of horizontal shear damage and 22 specimens that failed in shear failure with signs of anchorage zone distress were removed. As a result, a total of 171 tests were included in the evaluation database-level II. The readers are encouraged to note that those 171 tests included in the evaluation database-level II are more suitable to evaluate the accuracy and conservativeness of the existing shear design provisions.

Table 3.6 Filtering criteria for evaluation database-level II

Evaluation Database-Level I		***223 tests***
Filtering Criteria	- shear failure with signs of horizontal shear damage	- 30 tests
	- shear failure with signs of anchorage zone distress	- 22 tests
Evaluation Database-Level II		***171 tests***

Figure 3.2 summarizes the relationship between the database development flow and the failure modes of test specimens included in each database.

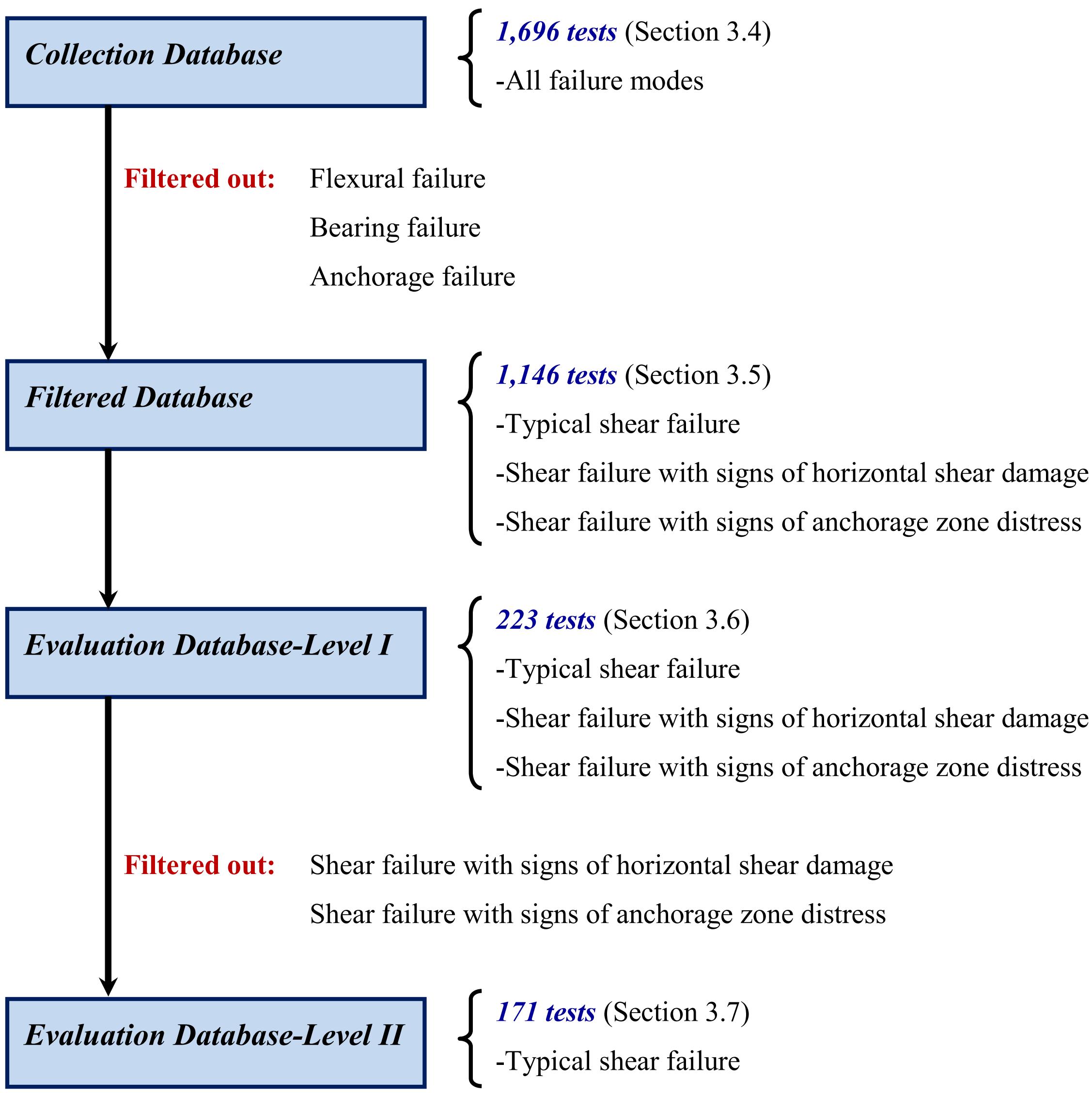


Figure 3.2 Database development flow and failure mode

3.8 EVALUATION DATABASE-TYPE MSR

In addition to the evaluation of the accuracy and conservativeness of the existing shear design provisions, the UTPCSDB-2011 is also utilized to evaluate the suitability of various design provisions for the minimum shear reinforcement. In order to conduct this evaluation, the evaluation database-type MSR is constructed by using filtering criteria shown in Table 3.7.

Table 3.7 Filtering criteria for evaluation database-type MSR

Filtered Database		***1,146 tests***
Filtering Criteria	- concrete strength < 4.0 ksi (27.6 MPa)	- 162 tests
	- concrete types other than normal concrete	- 59 tests
	- member height < 12.0 in. (305 mm)	- 337 tests
	- shear span to depth ratio < 2.0 (if member subjected to concentrated load)	- 119 tests
	- without shear reinforcement	- 428 tests
	- continuous beam	- 37 tests
	- segmental specimen	- 18 tests
	- externally post-tensioned specimen	- 35 tests
	- tests where shear cracking load is not reported	- 551 tests
	- specimens with poor transverse reinforcement anchorage details	- 3 tests
Evaluation Database-Type MSR		***171 tests***

The evaluation database-type MSR stores a total of 171 tests. Although the number of tests in the evaluation database-type MSR is haphazardly the same as that of tests in the evaluation database-level II, these two evaluation databases include different test results. As shown in Table 3.7, the evaluation database-type MSR excludes test results that are not relevant to the analysis of the minimum shear reinforcement design by adding the following two new filtering criteria: (1) test specimens that contain no shear reinforcement are excluded; and (2) tests that fail to report the applied shear force at the formation of inclined cracks are excluded. The latter criterion is significant to evaluate the suitability of the design provisions for the minimum shear reinforcement since this analysis adopts the reserve shear strength as an indicator. The definition of the reserve shear strength is explained in Chapter 7.

In addition to those filtering criteria, proper detailing of shear reinforcement is of great significance to ensure that shear reinforcement provided in test specimens behave efficiently. In the light of the study by Avendaño and Bayrak (2008), several specimens tested by Kaufman and Ramirez (1988) showed no additional shear strength after the formation of inclined cracks even though these specimens contain adequate amounts of shear reinforcement. Avendaño and Bayrak (2008) indicated that the reason for this is that those specimens had improper detailing of shear reinforcement; shear reinforcement in those specimens had no 90 degree hook in the bottom flange. Therefore, three tests by Kaufman and Ramirez (1988) were removed.

After all filtering processes, the evaluation database-type MSR, which includes a total of 171 test results, is constructed. The application of the evaluation database-type MSR is discussed in Chapter 7.

CHAPTER 4

DATABASE CHARACTERISTICS

4.1 OVERVIEW

The characteristics of the shear tests that are stored in the UTPCSDB-2011 are presented in this chapter. The existing shear tests on prestressed concrete members are presented in a historical context in order to appreciate the research conducted over the last sixty years. The primary variables that influence the shear behavior of prestressed concrete members are also studied by using the UTPCSDB-2011. These variables are important for the purpose of understanding the basis of various shear design provisions.

Additionally, the characteristics of shear tests conducted on prestressed and nonprestressed members are compared by using the other shear database. The database developed by Collins, et al. (2008) is employed for this purpose since their database contains the largest number of shear tests conducted on reinforced concrete members at the moment.

4.2 PRESTRESSED CONCRETE SHEAR TESTS: A HISTORICAL PERSPECTIVE

Figure 4.1 was prepared to provide an overview on the number of shear tests conducted on prestressed concrete members since 1954. In this figure, 1,696 tests were arranged in chronological order and divided into two groups according to overall member heights. The figure also illustrates five primary developments on shear design in North America.

An examination of Figure 4.1 reveals that results of shear tests conducted on prestressed concrete members have been continuously reported from 1954 to 2010 except for 1956 and 1957. It is interesting to observe that most of the test specimens after 1986 have overall member heights of greater than 12 in. (305 mm). This trend is supposed to contribute to develop a clear understanding of the shear behavior of prestressed concrete members by eliminating size or scaling effects since most of the prestressed concrete beams used in bridges and buildings are deeper than 12 in. (305 mm).

As mentioned in Chapter 2, the ACI 318 shear design equations have remained almost the same after their first introduction in 1960s. On the basis of Figure 4.1, the empirical derivation of the ACI 318 shear design equations was primarily derived from test results obtained by using specimens with overall member heights of less than or equal to 12 in. (305 mm). Therefore, it is important to verify the applicability of the ACI 318 shear design equations to more recent tests conducted on realistically sized members.

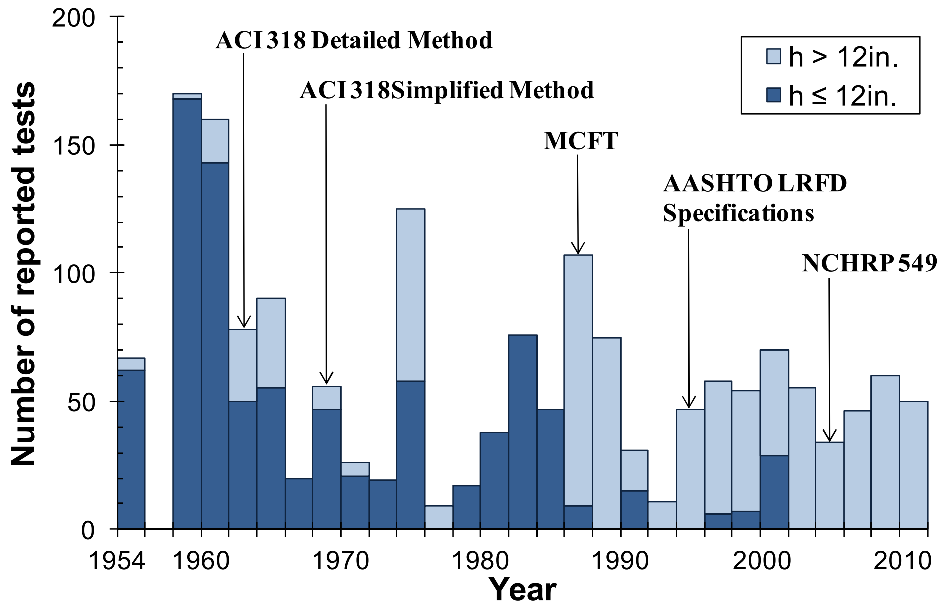


Figure 4.1 Number of reported tests of prestressed concrete members

In Figures 4.2 through 4.4, the historical trends of the specimen size, concrete strength, and shear reinforcement index of the 1,696 tests in the UTPSCDB-2011 are presented. The historical trends are given for both the maximum and mean values of each valuable in these figures. The specimen size is calculated as the product of the web width,

b_w, and the effective depth, d. The shear reinforcement index is defined as the product of the shear reinforcement ratio, ρ_v, and the yield strength of shear reinforcement, f_y.

An examination of Figure 4.2 reveals that there was a growing interest in testing large scale test specimens after 1990. This trend is similar to that shown in Figure 4.1. The main reason for this increase in the specimen size is that most of the references published after 1990 reported results from shear tests that employed full-scale specimens with practical cross sections, such as I-, T-, U-, and Box-beams. The largest specimen in the sixty-year history of prestressed concrete shear tests is a full-scale U-beam tested at the University of Texas at Austin (Hovell, et al 2010). Additionally, because of load tests of full-scale I- and Box-beams reported by Hanson and Hulsbos (1969), the maximum specimen size in 1968 and 1969 is notable. It is safe to say that substantial interest in full-scale testing in the last two decades resulted in important contributions to the UTPCSDB-2011.

In Figure 4.3, a similar trend also appears in the compressive strength of concrete that was used to fabricate test specimens over the years. While the maximum value of the compressive strength of concrete remained below 9,000 psi (62 MPa) before 1980, it increased to larger than 10,000 psi (69 MPa) after 1980 except for 1990 and 1991. The average value of the compressive strength of concrete also increased slightly after 1980. These trends imply that the use of high strength concrete attracted a great deal of attention in the prestressed concrete shear tests after 1980.

In Figure 4.4, the shear reinforcement index of specimens tested after 1988 was larger than that of specimens tested before 1988. Most of the specimens tested before 1980 had no shear reinforcement. Therefore, the average value of the shear reinforcement index was lower for those years except for 1970 and 1971.

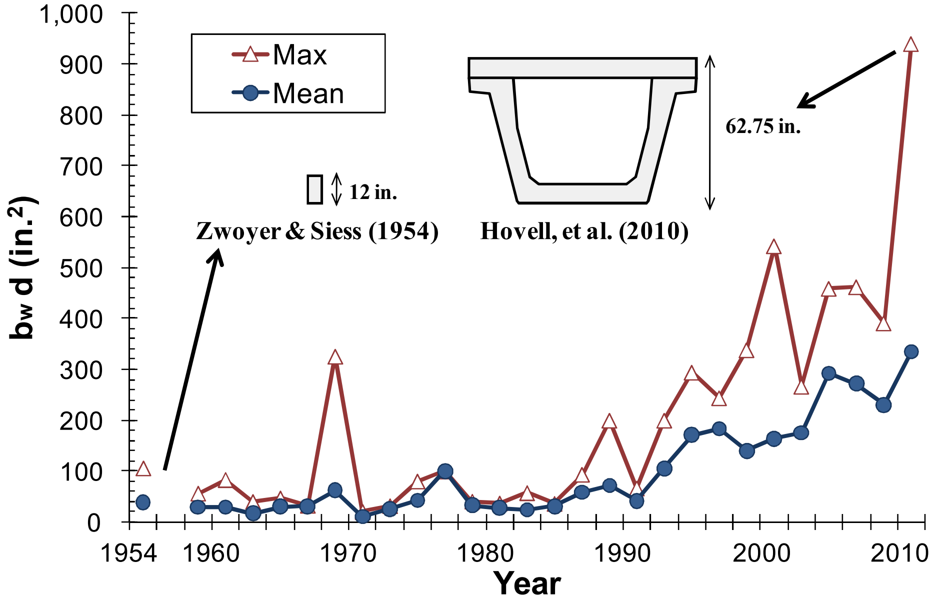


Figure 4.2 Specimen size: a historical perspective

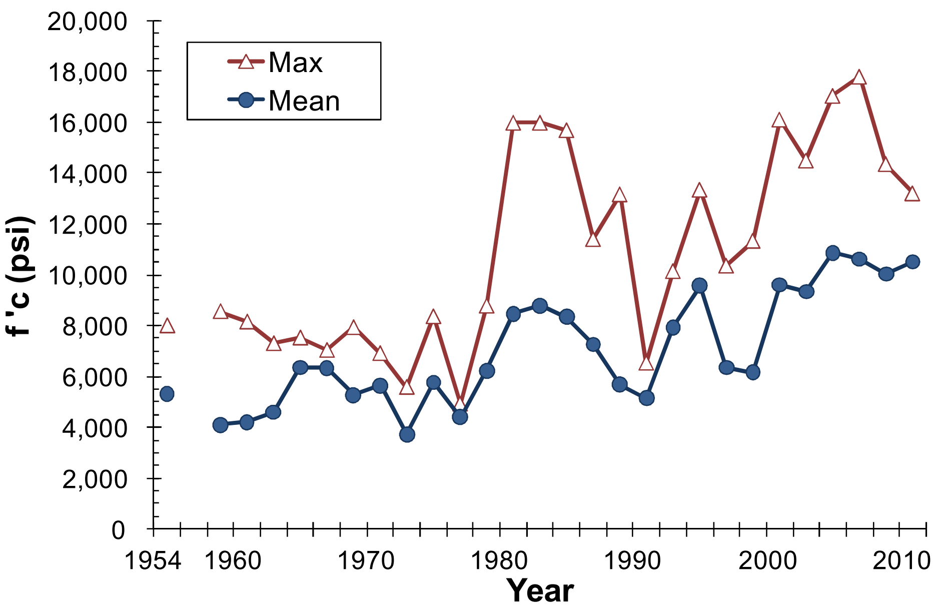


Figure 4.3 Concrete compressive strength: a historical perspective

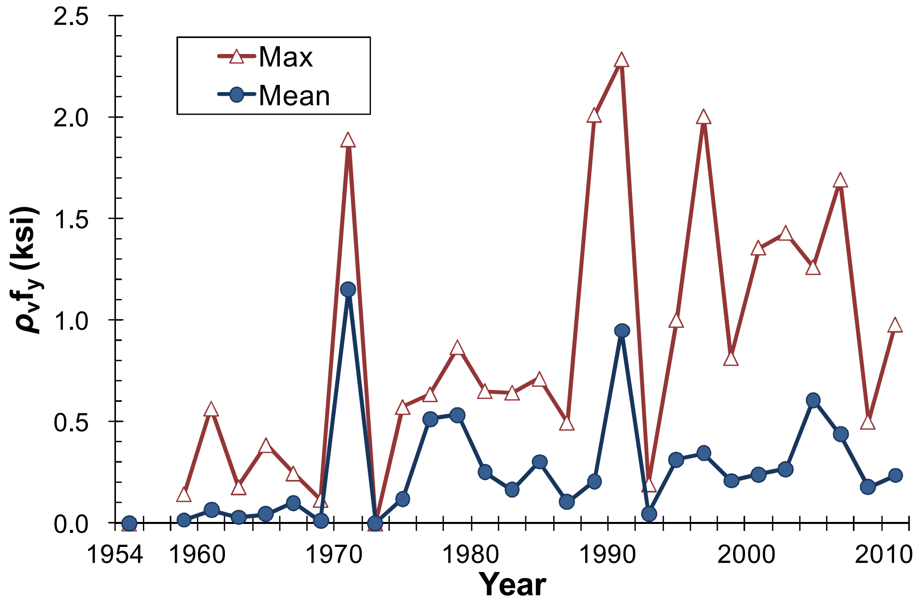


Figure 4.4 Shear reinforcement index: a historical perspective

4.3 DISTRIBUTION OF EXPERIMENTAL VARIABLES: FILTERED DATABASE

An overview of six primary variables of the 1,146 specimens included in the filtered database is provided in Figure 4.5. The observations on the six pie charts in Figure 4.5 are summarized as follows:

- Cross section

  59% of specimens included in the filtered database are I-beams, which account for the highest percentage among the five types of cross section. 26% are rectangular beams, and 12% are T-beams. 3% are U- and Box-beams, which have two separated webs. Additionally, 82% (557 specimens) of those I-beams have no deck, and 18% (120 specimens) have a reinforced concrete deck on top.

- Prestressing type

  57% of specimens included in the filtered database are pretensioned members. 40% are post-tensioned members, and 3% are externally post-tensioned members.

- Concrete type

  95% of specimens included in the filtered database were fabricated with normal concrete. 5% were built with special types of concrete, such as lightweight concrete, self-consolidating concrete, or both of them.

- Loading type

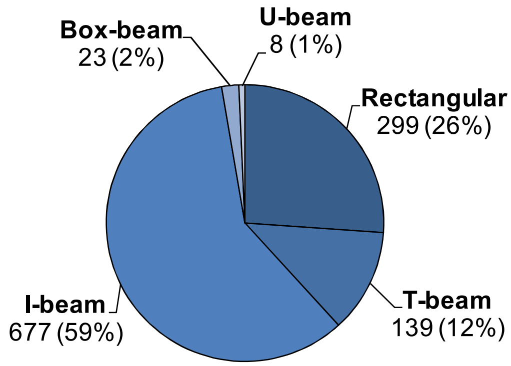
  96% of specimens included in the filtered database were subjected to concentrated loads, and 4% were tested under uniform loads. The number of the concentrated loads applied on test specimens varied on testing procedures.

- Support type

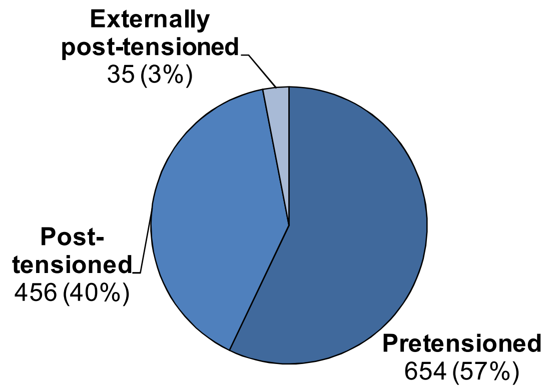
  97% of specimens included in the filtered database are simply supported beams, and 3% are continuous beams.

- Specimen type

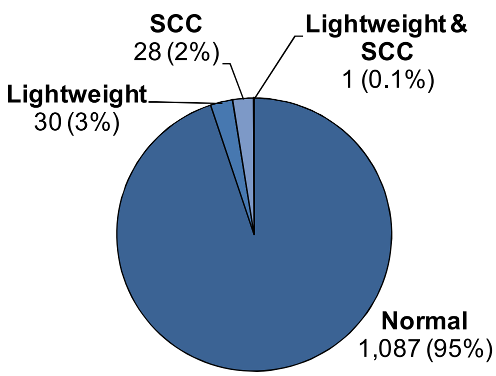
  98% of specimens included in the filtered database are normal specimens, and 2% are segmental specimens.



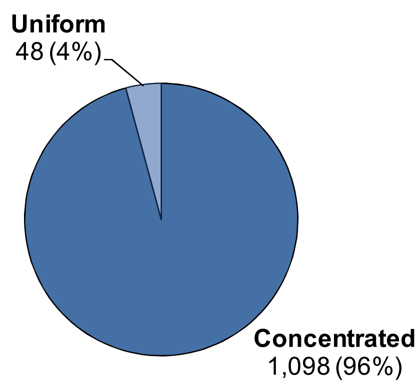
(a) Cross section



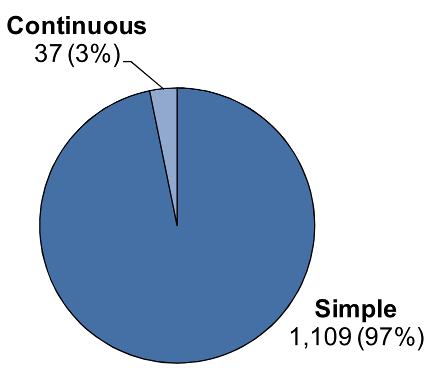
(b) Prestressing type



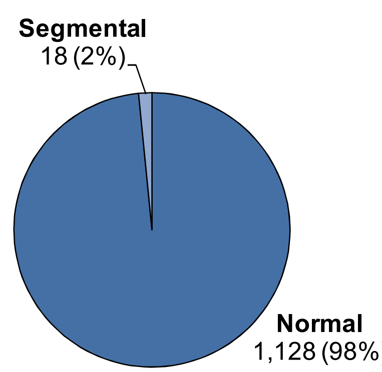
(c) Concrete type



(d) Loading type



(e) Support type



(f) Specimen type

Figure 4.5 Characteristics of specimens in filtered database

The distributions of six variables of the 1,146 shear tests in the filtered database are shown in Figures 4.6 through 4.11. Their characteristics are summarized as follows:

- Concrete compressive strength, f'_c

  Approximately one half of specimens included in the filtered database were fabricated with concrete that had compressive strengths of at least 6 ksi (41 MPa), while the other half were built with concrete that had compressive strengths of lower than 6 ksi (41 MPa). 17% of the specimens included in the filtered database were made with high strength concrete with compressive strengths above 10 ksi (69 MPa). 14% had compressive strengths below 4 ksi (28 MPa).

- Overall member height, h

  71% of the specimens included in the filtered database have overall member heights of greater than or equal to 12 in. (305 mm), and the rest have overall member heights of less than 12 in. (305 mm). 22% have member heights of greater than or equal to 24 in. (610 mm).

- Shear span to depth ratio, a/d

  90% of the specimens included in the filtered database have shear span to depth ratios of at least 2.0. 10% have shear span to depths ratios of less than 2.0, which are deep beams. Note that the behavior of deep beams cannot be explained by sectional design methods. Only 4% were subjected to uniform loads.

- Shear reinforcement index, $\rho_v f_y$

  Approximately one half of the specimens included in the filtered database have no shear reinforcement, while the other half contain shear reinforcement.

- <u>Ratio of effective prestress in concrete, f_{pc}/f'_c</u>

  90% of the specimens included in the filtered database have ratios of effective prestress in concrete, f_{pc}/f'_c, of lower than 20%. Note that f_{pc} is compressive stress in concrete at the centroid of the cross section due to prestress force or axial loads after all prestress losses.

- <u>Ratio of effective prestress in strand, f_{se}/f_{pu}</u>

  71% of the specimens included in the filtered database have ratios of effective prestress in prestressing steel, f_{se}/f_{pu}, of at least 40%. The simplified method in the ACI 318-08 code is applicable to members with f_{se}/f_{pu} of higher than 40%. Note that f_{se} is effective prestress in prestressing steel after all prestress losses.

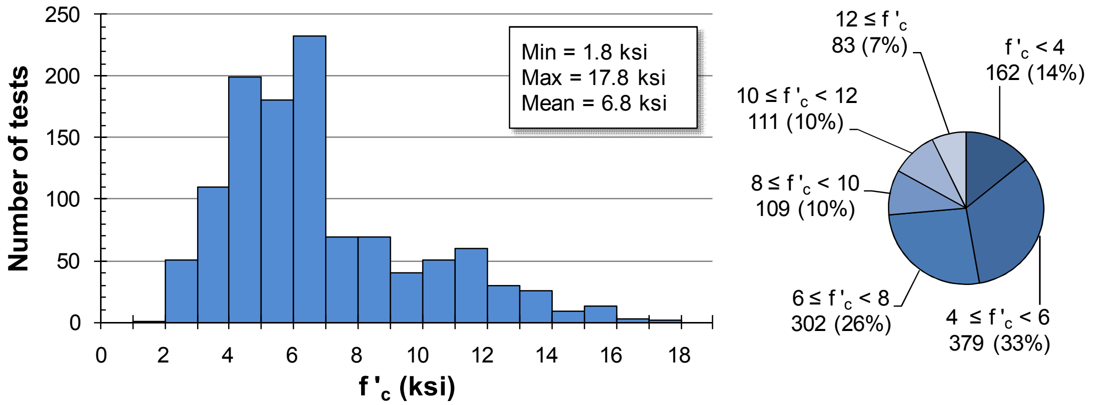


Figure 4.6 Concrete compressive strength in filtered database

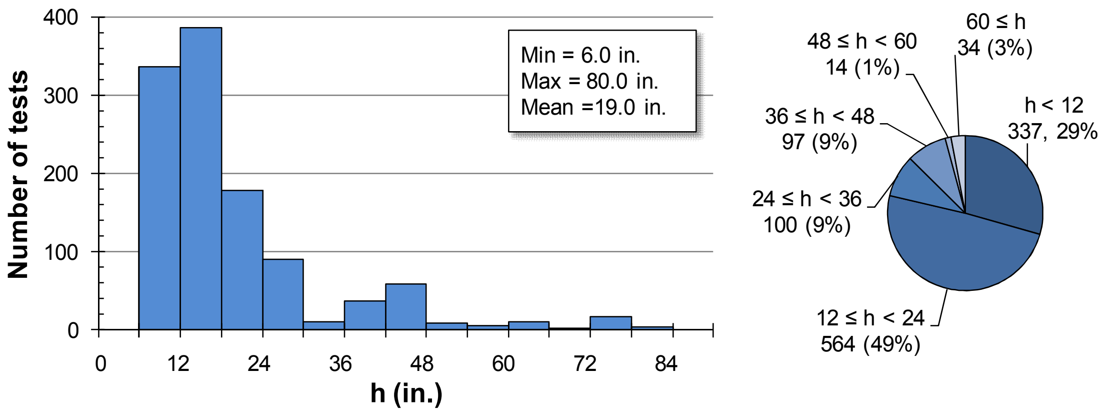


Figure 4.7 Overall member height in filtered database

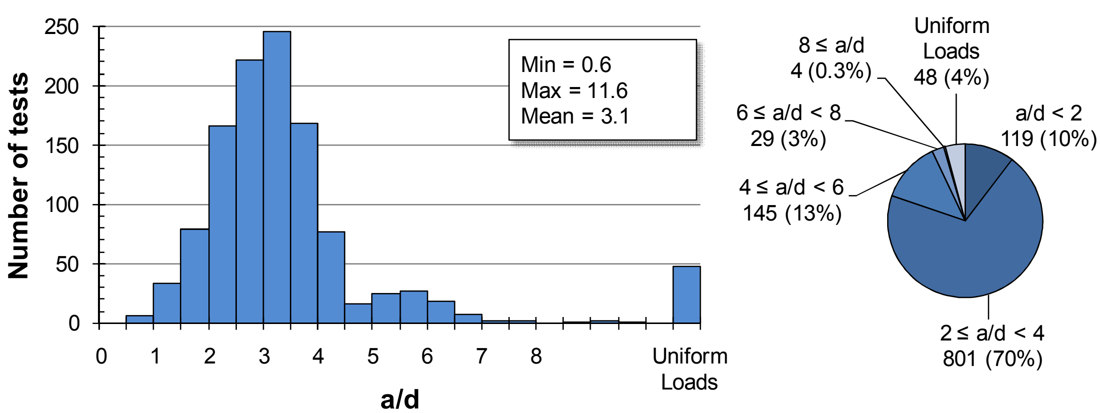


Figure 4.8 Shear span to depth ratio in filtered database

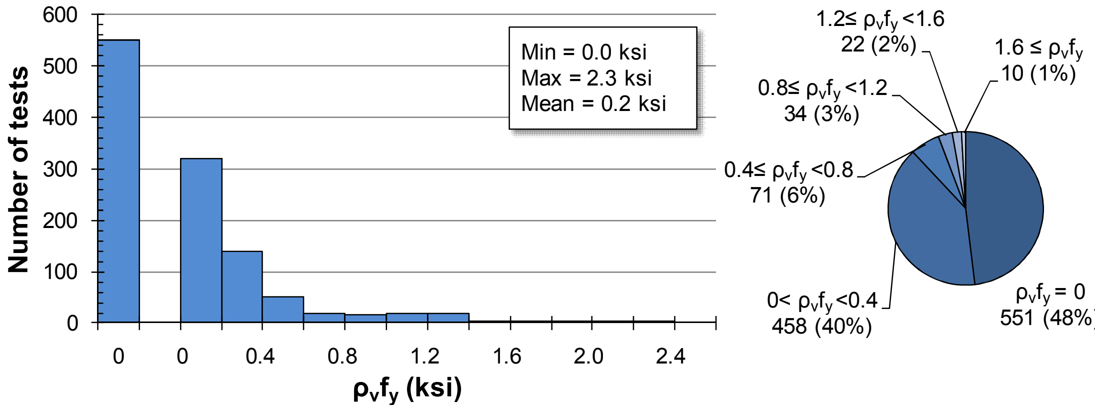


Figure 4.9 Shear reinforcement index in filtered database

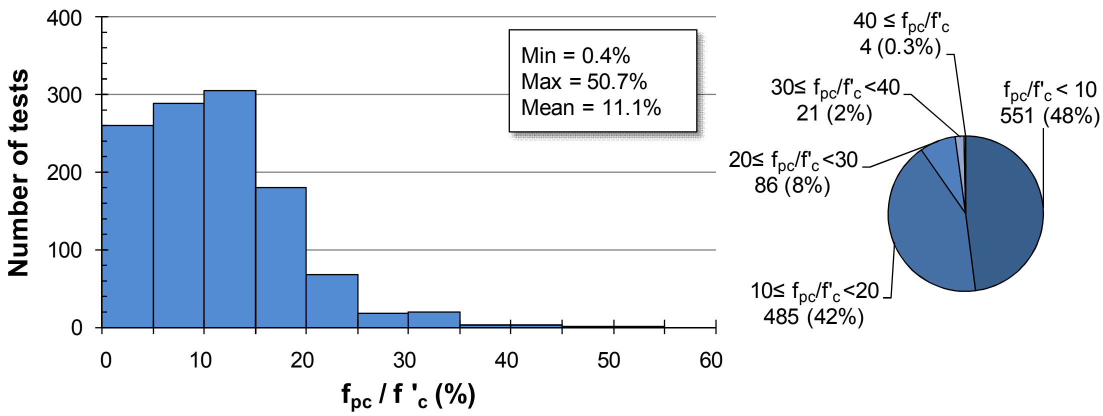


Figure 4.10 Effective prestress in concrete in filtered database

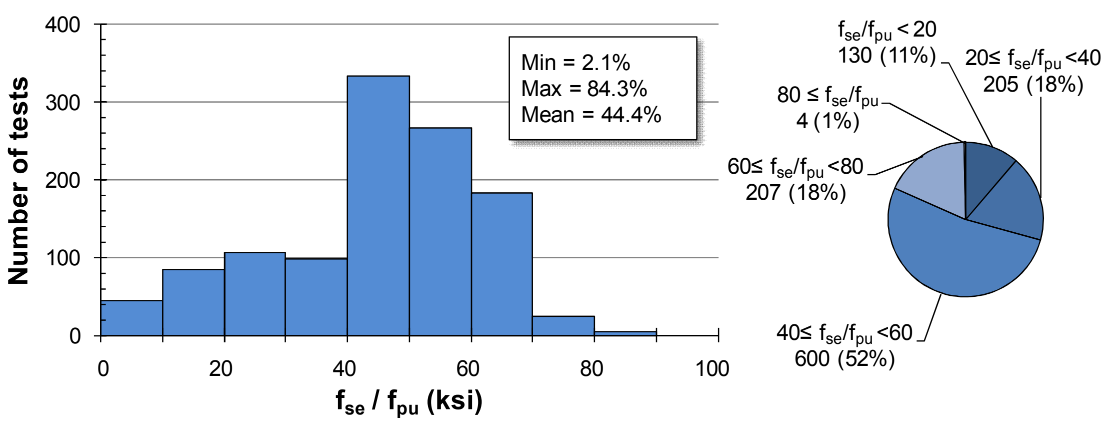


Figure 4.11 Effective prestress in prestressing steel in filtered database

It is interesting to note that the distribution pattern of effective prestress is completely different between concrete (f_{pc}/f'_c) and prestressing steel (f_{se}/f_{pu}). On the basis of an examination of histograms in Figures 4.10 and 4.11, 71% of specimens included in the filtered database have f_{se}/f_{pu} of at least 40%, while only 10% have f_{pc}/f'_c of greater than or equal to 20%. The relationship between f_{se}/f_{pu} and f_{pc}/f'_c is presented in Figure 4.12. These data points are divided into two groups according to the type of specimens: (1) full-scale specimens; and (2) non-full-scale specimens. The full-scale specimen is defined as a specimen that has the same cross sectional properties as standard bridge girders specified by the AASHTO, PCI, and U.S. Departments of Transportation, and the non-full-scale specimens are fabricated in accordance with in-house designs at various laboratories.

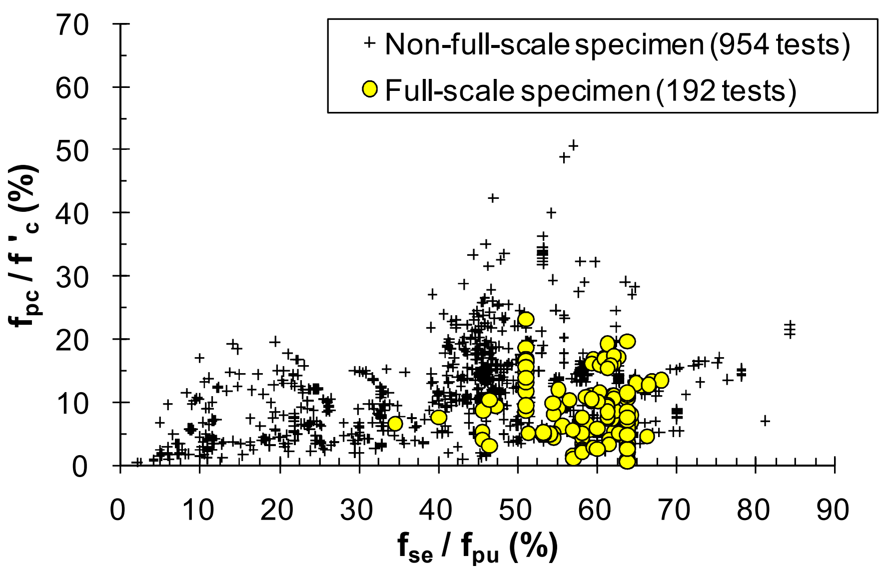


Figure 4.12 Effective prestress in concrete and prestressing steel

The reason of the discrepancy between the two expressions that are used to qualify effective prestress (i.e., f_{pc}/f'_c, f_{se}/f_{pu}) is explained by investigating characteristics of those two types of test specimens. The values of f_{se}/f_{pu} of the full-scale specimens are between 30% and 70% in Figure 4.12 since bridge girders in practice are designed to maximize a use of material strengths without exceeding the permissible stress of materials. On the other hand, the non-full-scale specimens employ an excessive amount of prestressing steel to induce the shear failure prior to the flexural failure in general. The excessive amount of prestressing steel prevents researchers from utilizing high stress levels of prestressing steel since too much prestress force would damage concrete at prestress transfer. Most of the researchers who used the non-full-scale specimens kept the stress level of prestressing steel low but used a large quantity of prestressing steel. Thus, some non-full-scale specimens have f_{se}/f_{pu} of lower than 30% even though their values of f_{pc}/f'_c are comparable with those of the full-scale specimens.

It is also possible that the different expressions used to express effective prestress (i.e., f_{pc}/f'_c, f_{se}/f_{pu}) affect the accuracy of the shear design equations. The simplified method and one of the requirements for the minimum shear reinforcement in the ACI 318-08 code are applicable to members with f_{se}/f_{pu} of larger than 40%. In contrast, the V_c terms in several design codes are affected by the amount of effective prestress in concrete. Again, it should be emphasized that a better understanding of the effect of prestress force on the shear behavior is indispensable for understanding the rationale used in various shear design provisions.

4.4 DISTRIBUTION OF EXPERIMENTAL VARIABLES: EVALUATION DATABASE

An overview of three primary variables of the 223 specimens included in the evaluation database-level I is given in Figure 4.13. The observations on the three pie charts in Figure 4.13 are summarized as follows:

- <u>Cross section</u>

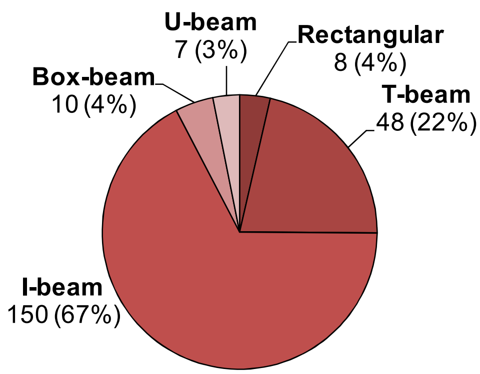
  67% of the specimens included in the evaluation database-level I are I-beams, 22% are T-beams, and 4% are rectangular beams. 7% are U- and Box-beams, which have two separated webs. 67% (100 specimens) of those I-beams have no deck, and 33% (50 specimens) have a reinforced concrete deck on top.

- <u>Prestressing type</u>

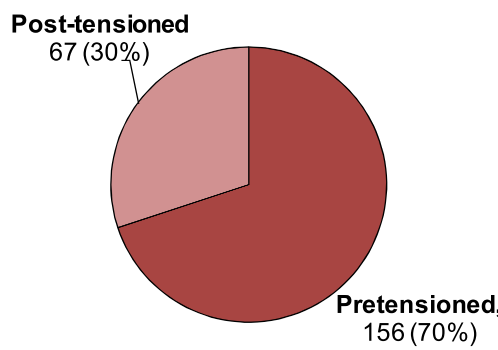
  70% of the specimens included in the evaluation database-level I are pretensioned members, and 30% are post-tensioned members.

- <u>Loading type</u>

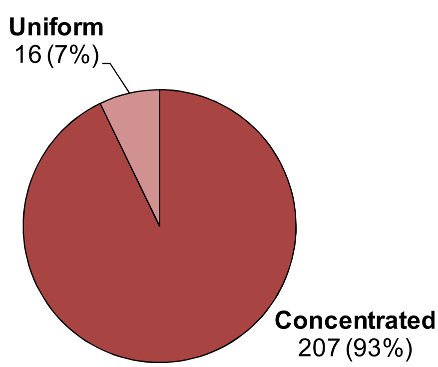
  93% of the specimens included in the evaluation database-level I were subjected to concentrated loads, and 7% were tested under uniform loads.



(a) Cross section



(b) Prestressing type



(c) Loading type

Figure 4.13 Characteristics of specimens in evaluation database-level I

The distributions of six variables of the 223 shear tests included in the evaluation database-level I are shown in Figures 4.14 through 4.19. Their characteristics are summarized as follows:

- Concrete compressive strength, f'_c

  The minimum concrete strength of the specimens included in the evaluation database-level I is 4 ksi (28 MPa). 64% of the specimens included in the evaluation database-level I were built with concrete of compressive strengths of lower than 10 ksi (69 MPa), and 36% were fabricated with concrete of compressive strength of at least 10 ksi (69 MPa). The ACI 318-08 code and the AASHTO LRFD 2010 specifications set 10 ksi (69 MPa) as an upper limit of the design concrete compressive strength if the requirement for the minimum shear reinforcement is not satisfied.

- Overall member height, h

  All specimens in the evaluation database-level I have overall member heights of at least 12 in. (305 mm). 47% of the specimens included in the evaluation database-level I have overall member heights of greater than or equal to 24 in. (610 mm), and the rest have overall member heights of less than 24 in. (610 mm).

- Shear span to depth ratio, a/d

  In the evaluation database-level I, all specimens subjected to concentrated loads have shear span to depth ratios of at least 2.0. 90% of the specimens included in the evaluation database-level I have shear span to depth ratios of between 2.0 and 4.0. It is important to note that the shear span to depth ratio is not defined for specimens tested under uniform loads.

- <u>Shear reinforcement index, $\rho_v f_y$</u>

  All specimens in the evaluation database-level I satisfy the requirements for the minimum shear reinforcement specified in both the ACI 318-08 code and the AASHTO LRFD 2010 specifications. 73% of the specimens included in the evaluation database have shear reinforcement indices of lower than 0.4 ksi (2.8 MPa).

- <u>Ratio of effective prestress in concrete, f_{pc}/f'_c</u>

  93% of the specimens included in the evaluation database-level I have ratios of effective prestress in concrete, f_{pc}/f'_c, of less than or equal to 20%.

- <u>Ratio of effective prestress in strand, f_{se}/f_{pu}</u>

  85% of the specimens included in the evaluation database-level I have ratios of effective prestress in prestressing steel, f_{se}/f_{pu}, of at least 40%.

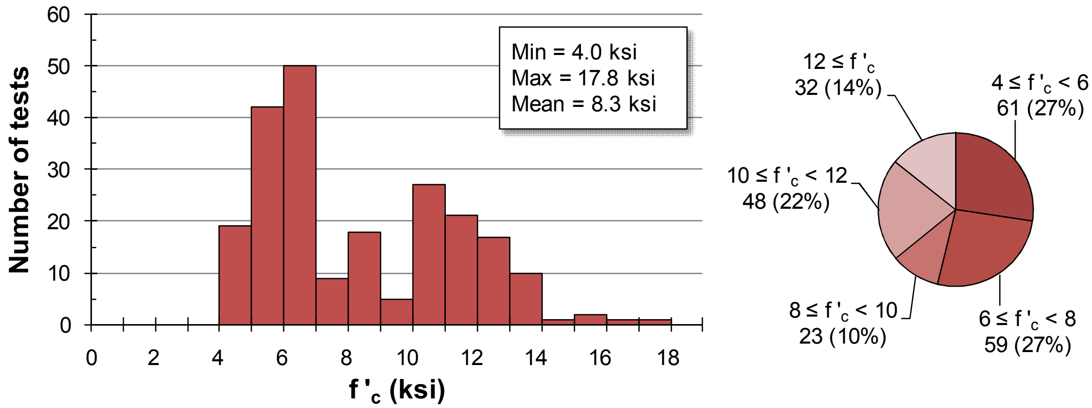


Figure 4.14 Concrete compressive strength in evaluation database-level I

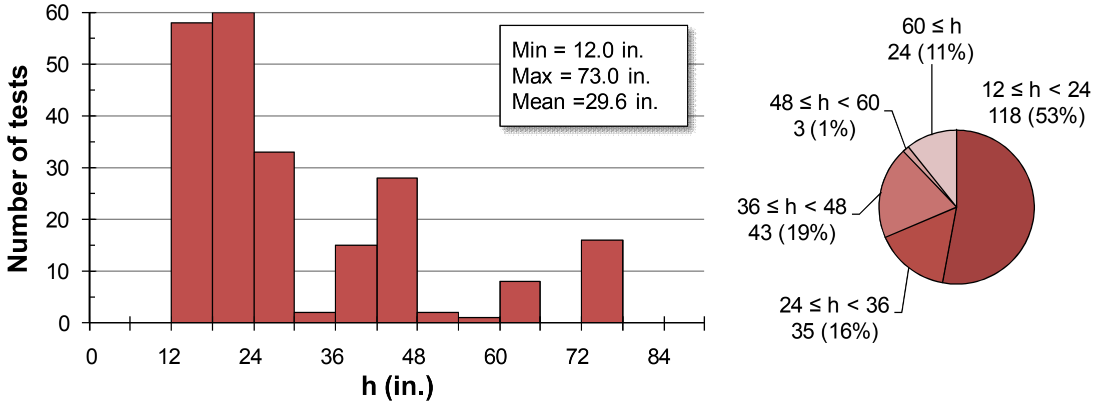


Figure 4.15 Overall member height in evaluation database-level I

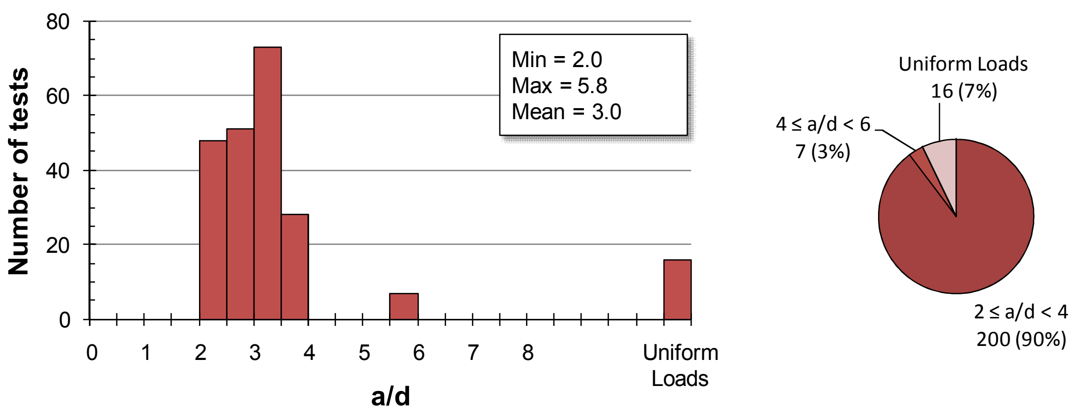


Figure 4.16 Shear span to depth ratio in evaluation database-level I

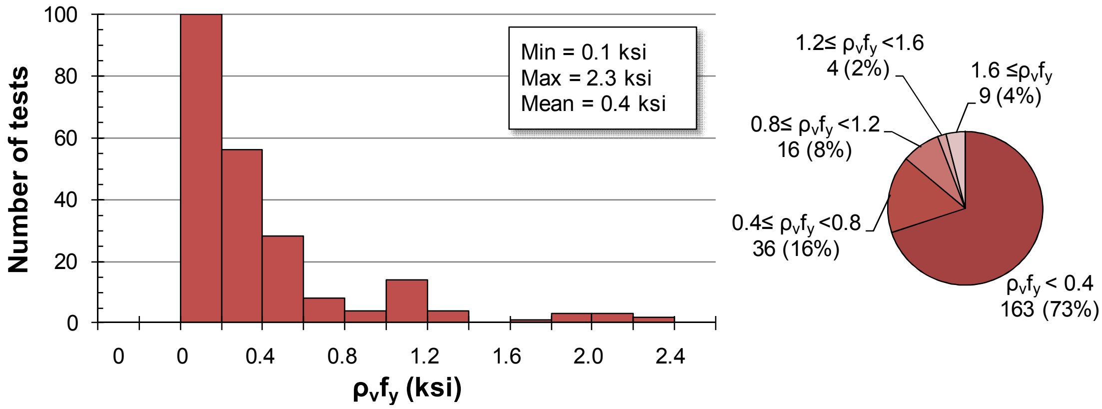


Figure 4.17 Shear reinforcement index in evaluation database-level I

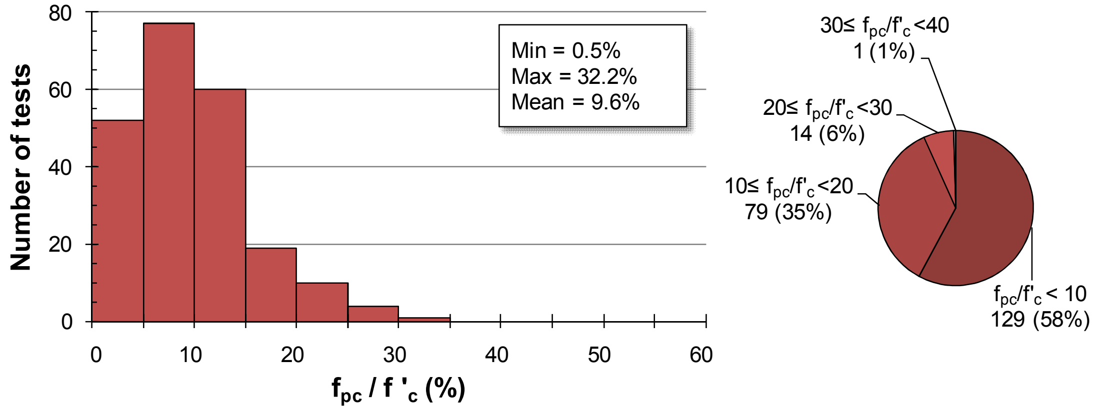


Figure 4.18 Effective prestress in concrete in evaluation database-level I

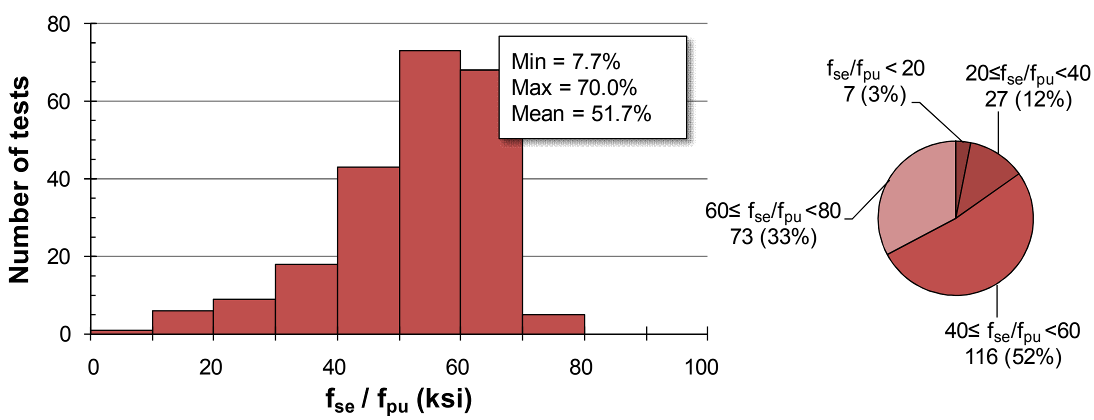


Figure 4.19 Effective prestress in prestressing steel in evaluation database-level I

On the basis of the seven groups of the shear failure modes defined in previous chapter, the number and percentage of each shear failure mode in the evaluation database-level I is shown in Figure 4.20. An examination of Figure 4.20 reveals that 171 of the 223 specimens (77% of total) were reported to have failed in one of the typical shear failures: (1) shear failure; (2) flexural shear failure; (3) web crushing failure; (4) shear compression failure; and (5) shear tension failure. The readers are encouraged to note that these 171 specimens are included in the evaluation database-level II.

Additionally, 30 specimens (13% of total) were reported to have failed in shear failure with signs of horizontal shear damage, and 22 specimens (10% of total) were reported to have failed shear failure with signs of anchorage zone distress. These 52 specimens are not included in the evaluation database-level II. Again, it should be emphasized that the evaluation database-level II is supposed to be more appropriate to evaluate the accuracy and conservativeness of the existing shear design provisions. The evaluation database-level I is utilized to make an observation on the performance of the existing shear design provisions on specimens that failed in shear failure with sings of horizontal shear damage and/or anchorage zone distress.

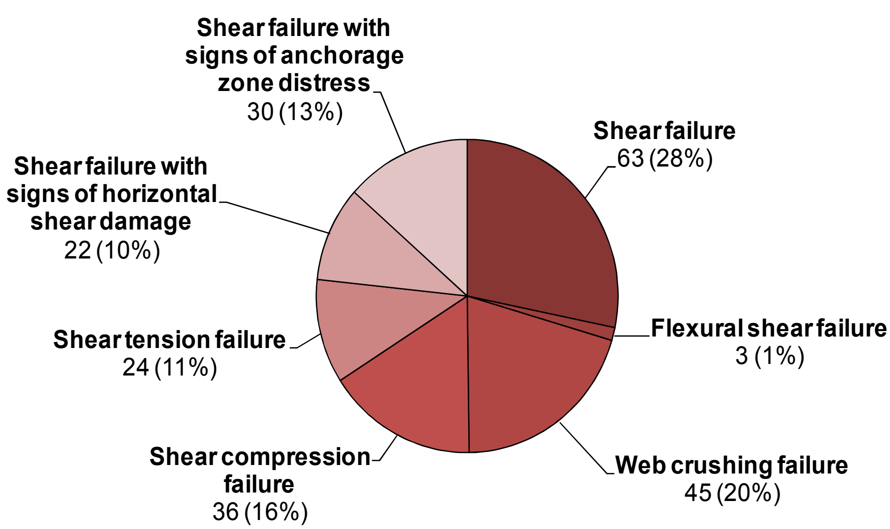


Figure 4.20 Shear failure mode in evaluation database-level I

4.5 COMPARISON WITH REINFORCED CONCRETE SHEAR TESTS

This section aims to clarify differences between the UTPCSDB-2011 and existing shear databases for reinforced concrete members. The characteristics of primary variables in the UTPCSDB-2011 are compared with those of the existing shear databases for reinforced concrete members. The database developed by Collins, et al. (2008), which is referred to as RC database, is utilized for this comparison because this database contains the largest number of shear tests conducted on reinforced concrete members. The RC database has 1,849 tests conducted on reinforced concrete members without shear reinforcement. After specimens that failed in flexural failure were filtered out from the original database, the number of tests in the RC database was reduced to 1,696.

The distributions of three variables of 1,696 tests in the RC database are presented in Figures 4.21 through 4.23. These distributions are compared with those of the filtered database in the UTPCSDB-2011 in Figures 4.6 through 4.8. The differences between two databases are summarized as follows:

- Number of available shear tests

  The number of available shear tests in the UTPCSDB-2011 is smaller than that in the RC database. The UTPCSDB-2011 stores a total of 1,696 tests. 1,146 of the 1,696 specimens were prestressed concrete members that failed in shear. Moreover, 551 of those 1,146 tests employed specimens without shear reinforcement, and the rest used specimens with shear reinforcement. In contrast, the RC database contains a total of 1,849 tests. 1,696 of the 1,849 specimens failed in shear. All specimens in the RC database had no shear reinforcement.

- Concrete compressive strength f'_c

  The UTPCSDB-2011 has a higher percentage of specimens built with high strength concrete than that of the RC database. The percentages of specimens with concrete strength of at least 10 ksi (69 MPa) are 17% in the UTPCSDB-2011 and 7% in the RC database. The high strength concrete is more likely to be used in prestressed concrete members.

- Overall member height *h*

  The UTPCSDB-2011 has a lower percentage of specimens with relatively small overall member heights than that of the RC database. The percentages of specimens with overall member heights of less than 12 in. (305 mm) are 29% in the UTPCSDB-2011 and 33% in the RC database. The RC database, however, stores deeper specimens than those in the UTPCSDB-2011. The maximum overall member heights are 80 in. (2,032 mm) in the UTPCSDB-2011 and 123.6 in. (3,139 mm) in the RC database. The RC database includes large reinforced concrete specimens tested to investigate the influence of size effect on the shear strength.

- Shear span to depth ratio *a/d*

  The specimens in both databases have similar distributions of the shear span to depth ratio.

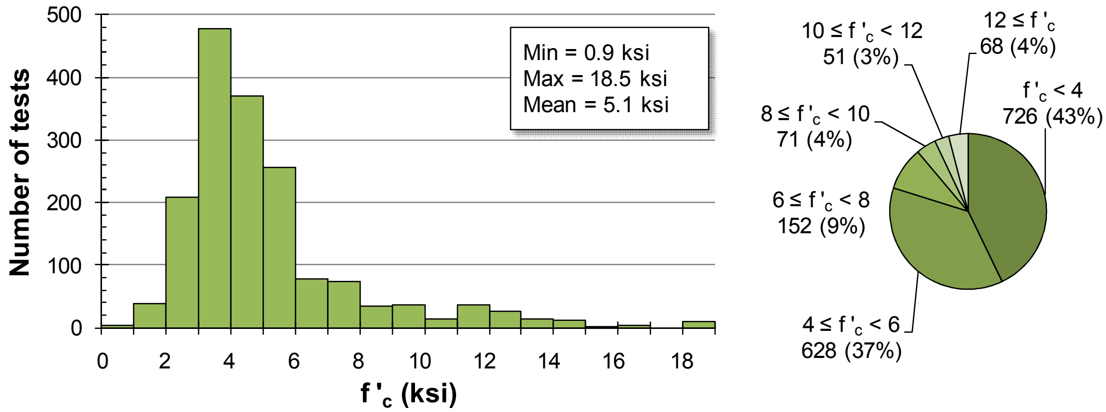


Figure 4.21 Concrete compressive strength in RC database

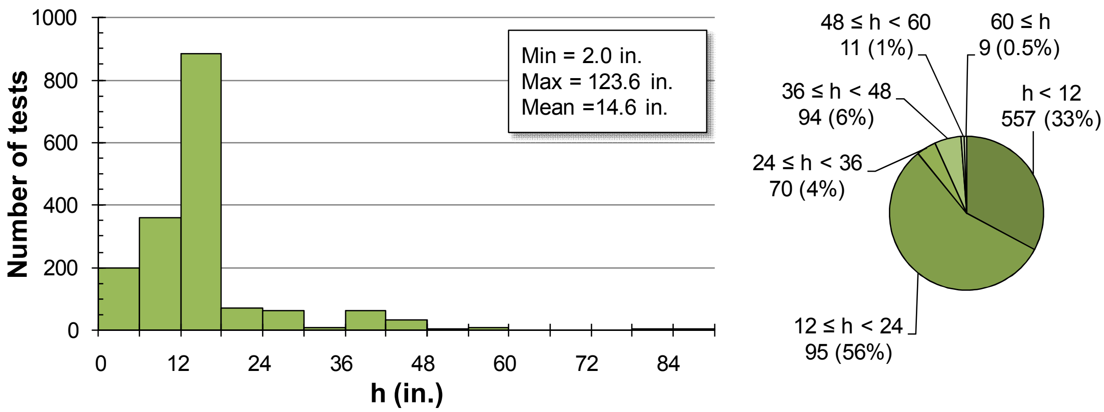


Figure 4.22 Overall member height in RC database

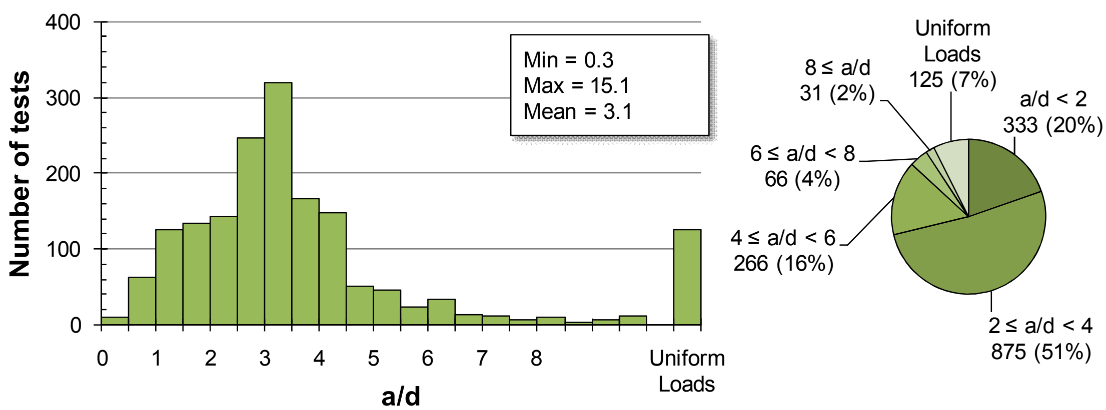


Figure 4.23 Shear span to depth ratio in RC database

CHAPTER 5

DATABASE ANALYSIS: ULTIMATE SHEAR STRESS

5.1 OVERVIEW

The research findings on the ultimate shear stress that can be carried by prestressed concrete beams are presented in this chapter. The UTPCSDB-2011 is fully utilized to clarify the influence of primary experimental variables on the ultimate shear stress of prestressed concrete members. Additionally, the relationship between the ultimate shear stress and the shear failure mode is investigated.

In this chapter, the ultimate shear stress is represented with two terms: (1) shear stress at failure, $V_{test}/b_w d$; and (2) normalized shear stress at failure, $V_{test}/\sqrt{f'_c}\, b_w d$.

5.2 INFLUENCE OF EXPERIMENTAL VARIABLES

The influence of experimental variables on the ultimate shear stress of prestressed concrete members is discussed in this section. The relationships between the ultimate shear stress and each experimental variable are presented in Figures 5.1 through 5.6. In these figures, 223 tests included in the evaluation database-level I are differentiated from other tests included in the filtered database by using different symbols. Again, it is important to appreciate that these 223 tests included in the evaluation database-level I are more suitable for the evaluation of the existing shear design provisions than other tests included in the filtered database.

5.2.1 Influence of Concrete Strength on Ultimate Shear Stress

In Figure 5.1(a), the shear stress at failure increases as the concrete strength increases. In contrast, in Figure 5.1(b) there is no obvious sign of an increase in the normalized shear stress at failure depending on the concrete strength. The normalized shear stress at failure was calculated by dividing the shear stress at failure by $\sqrt{f'_c}$. It is reasonable to suppose that the shear strength of prestressed concrete members with sufficient shear reinforcement is likely to be proportional to $\sqrt{f'_c}$. As mentioned in Chapter 2, most of the shear design equations adopt $\sqrt{f'_c}$ in their V_c terms. On the basis of the aforementioned observations, the adoption of $\sqrt{f'_c}$ in the V_c term is appropriate to estimate the shear strength of prestressed concrete members.

5.2.2 Influence of Effective Depth on Ultimate Shear Stress

An examination of Figure 5.2 indicates that there is no strong relationship between the ultimate shear stress and the effective depth. In other words, no clear sign of size effect appears on the shear strength of prestressed concrete members with adequate amounts of shear reinforcement. Although some of the normalized shear stresses of specimens with effective depths of between 15 in. and 20 in. are notable, these high values of the shear stress can be attributed to large amounts of shear reinforcement used in those test specimens. The influence of the shear reinforcement index is discussed later with Figure 5.4.

As mentioned in Chapter 2, the design expressions based on the MCFT assume that members having at least the minimum amount of shear reinforcement are free from the influence of size effect. The ACI 318-08 code, the AASHTO LRFD 2010 segmental procedure, and the JRA 2002 specifications adopt no expression to account for size effect in their V_c terms. On the basis of Figure 5.2, these assumptions are reasonable for prestressed concrete members with sufficient shear reinforcement. On the other hand, the V_c term in the JSCE 2007 specifications incorporates the influence of size effect regardless of the amount of shear reinforcement. Although EN 1992 also employs the V_c terms that account for size effect, the shear strength is provided only by the V_s terms in members with adequate shear reinforcement.

5.2.3 Influence of Shear Span to Depth Ratio on Ultimate Shear Stress

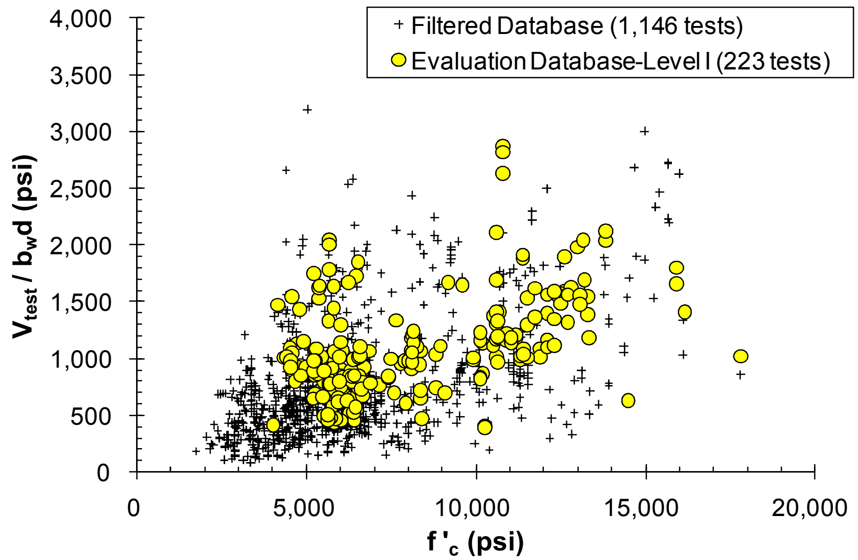
An examination of Figure 5.3 indicates that the shear strength of prestressed concrete members is strongly affected by the shear span to depth ratio. The ultimate shear stress increases as the shear span to depth ratio decreases. The reason for this increase in the ultimate shear stress is that members with small shear span to depth ratios resist the shear force mainly by the arch action, i.e., a large portion of the shear force is transmitted directly to the support of the members. For members with shear span to depth ratios of greater than 2.5, the lower bound of the ultimate shear stress is not influenced by increasing shear span to depth ratios. Therefore, it is reasonable to suppose that the shear design equation based on the sectional design method may not need to be a function of the shear span to depth ratio.

5.2.4 Influence of Shear Reinforcement Index on Ultimate Shear Stress

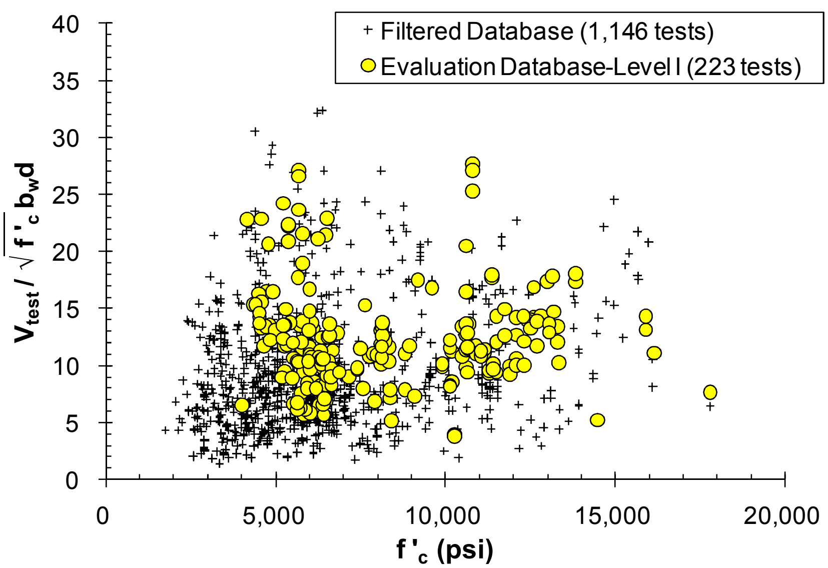
In Figure 5.4, the shear reinforcement index also possesses a strong relationship with the ultimate shear stress. The ultimate shear stress increases as the shear reinforcement index increases. The reason for this increase in the ultimate shear stress is that the shear reinforcement contribution is enhanced in specimens with large amounts of shear reinforcement. The figure, however, implies that there is an upper limit on the maximum shear strength depending on the shear reinforcement index. As reviewed in Chapter 2, all shear design provisions adopt the upper limit on the maximum shear strength, which is provided to avoid web crushing failure prior to yielding of shear reinforcement. With the trend shown in Figure 5.4, it is safe to say that the adoption of the upper limit on the maximum shear strength is reasonable especially for members with large amounts of shear reinforcement.

5.2.5 Influence of Effective Prestress on Ultimate Shear Stress

In Figures 5.5 and 5.6, the ultimate shear stress increases as effective prestress increases. In particular, effective prestress in concrete is more related to the increase in the shear strength of prestressed concrete members. Therefore, it is reasonable to rationalize that prestress force enhances the shear strength of prestressed concrete members by delaying the formation of inclined cracks in concrete and improving aggregate interlock after the formation of the inclined cracks.

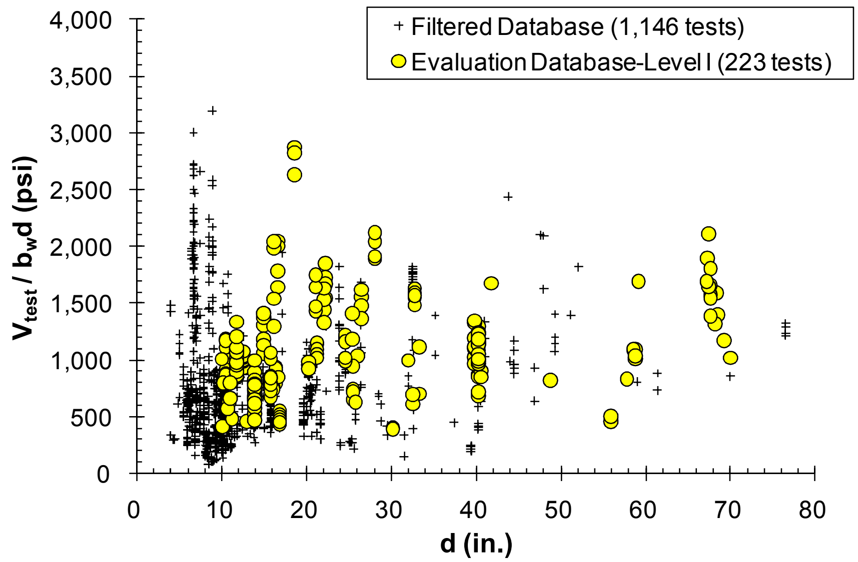


(a) Shear stress at failure

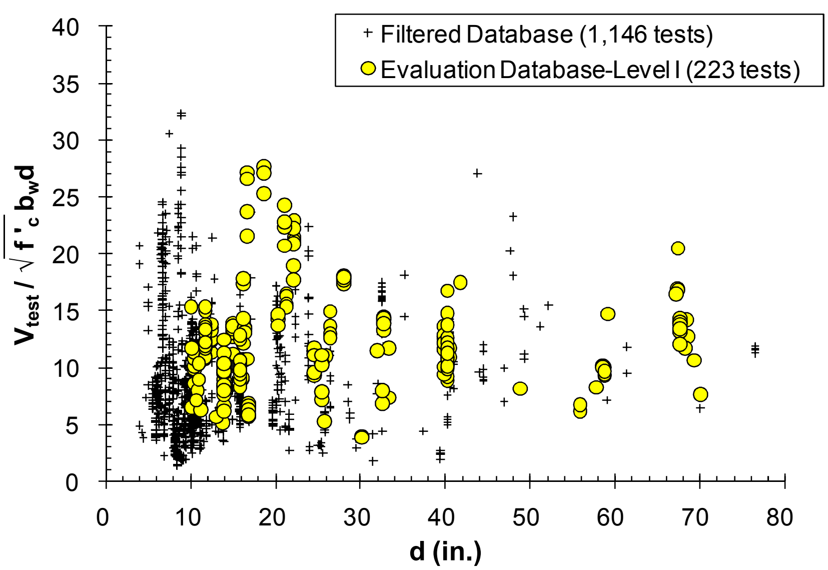


(b) Normalized shear stress at failure

Figure 5.1 Ultimate shear stress versus concrete compressive strength

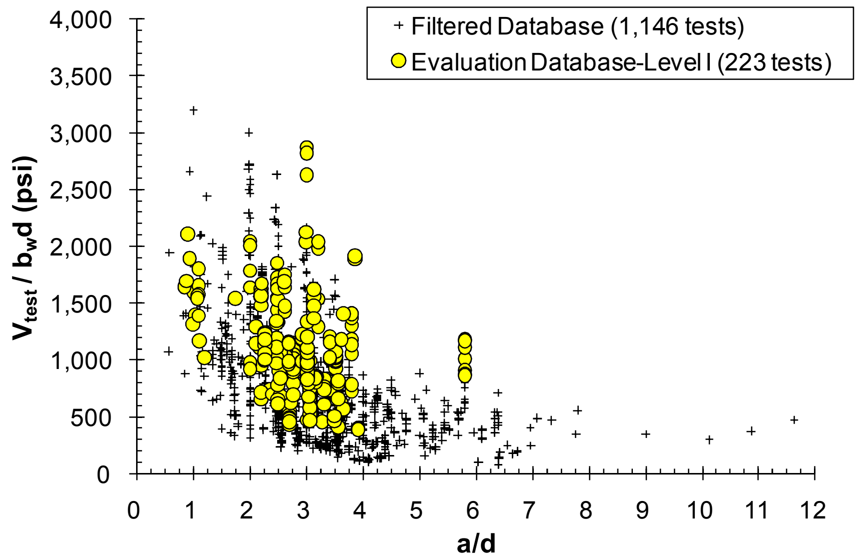


(a) Shear stress at failure

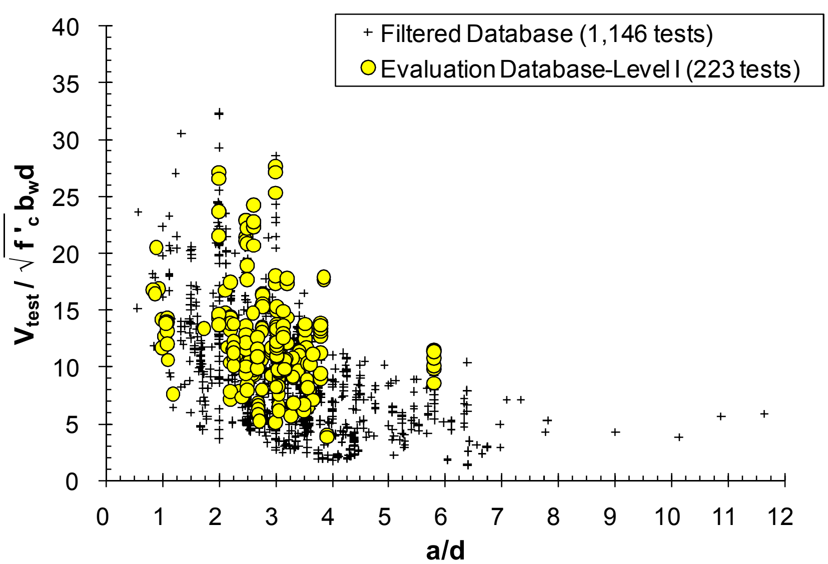


(b) Normalized shear stress at failure

Figure 5.2 Ultimate shear stress versus effective depth

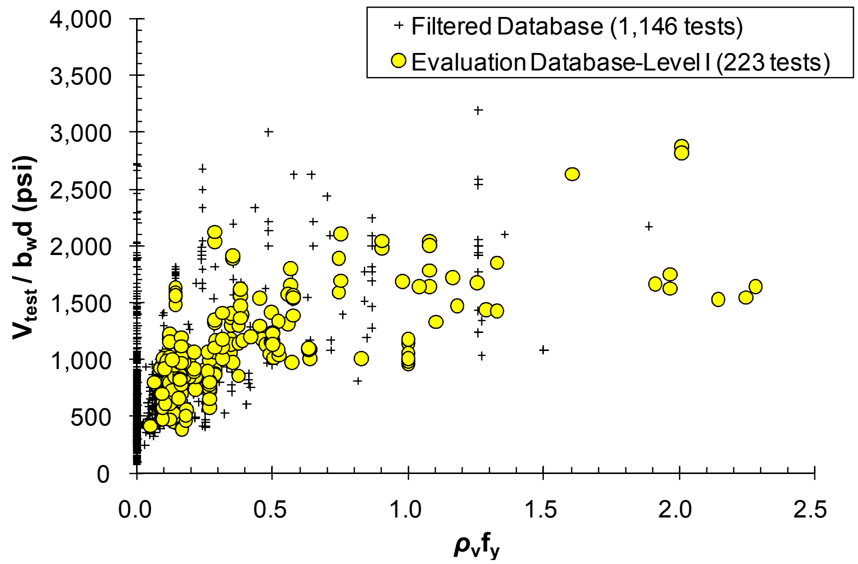


(a) Shear stress at failure

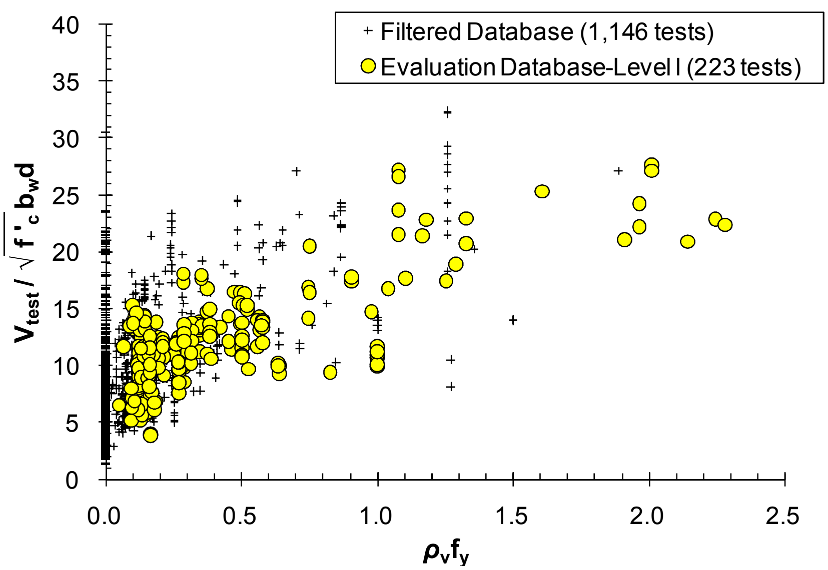


(b) Normalized shear stress at failure

Figure 5.3 Ultimate shear stress versus shear span to depth ratio

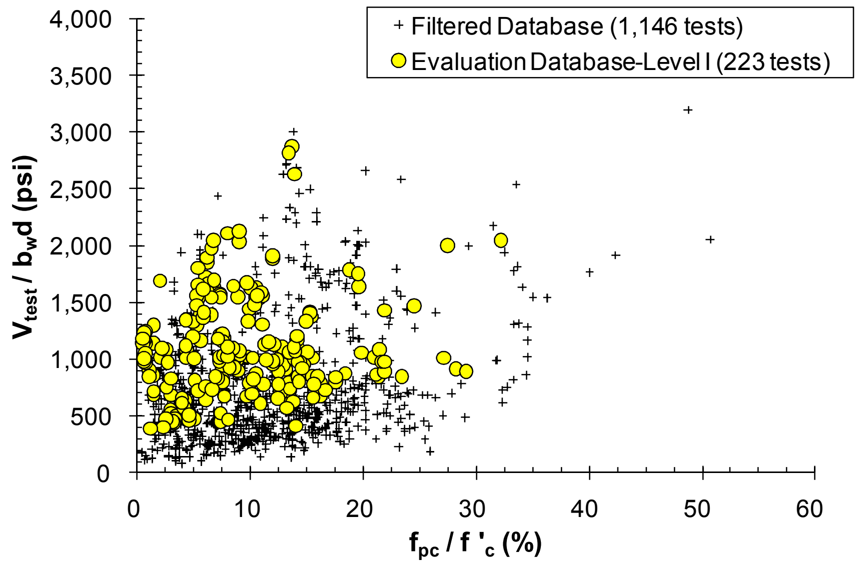


(a) Shear stress at failure

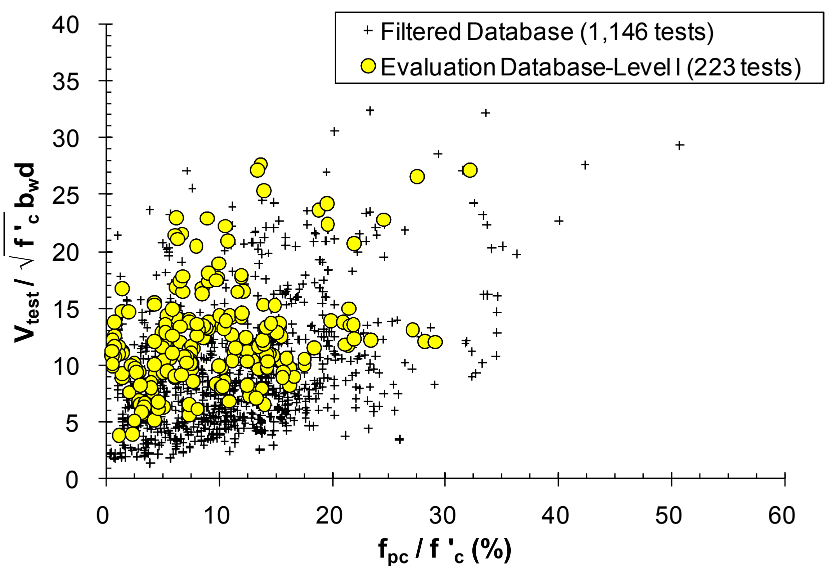


(b) Normalized shear stress at failure

Figure 5.4 Ultimate shear stress versus shear reinforcement index

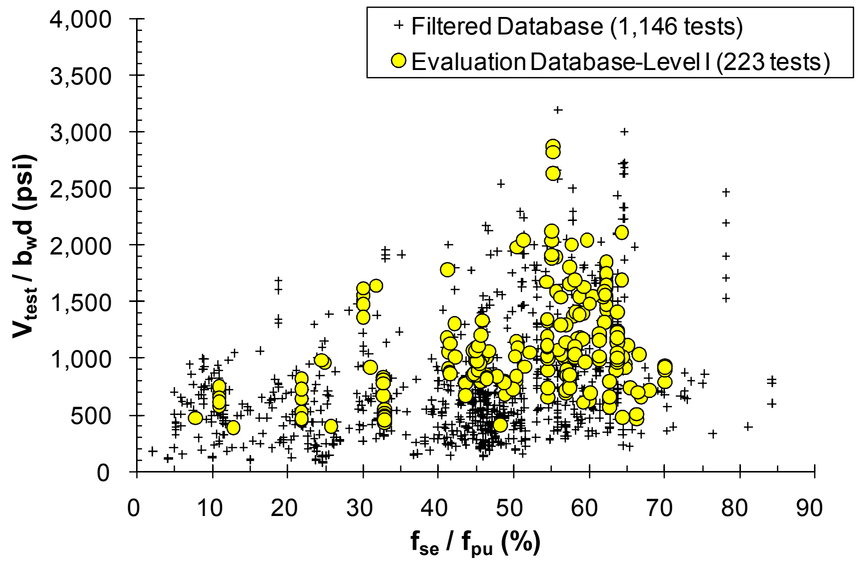


(a) Shear stress at failure

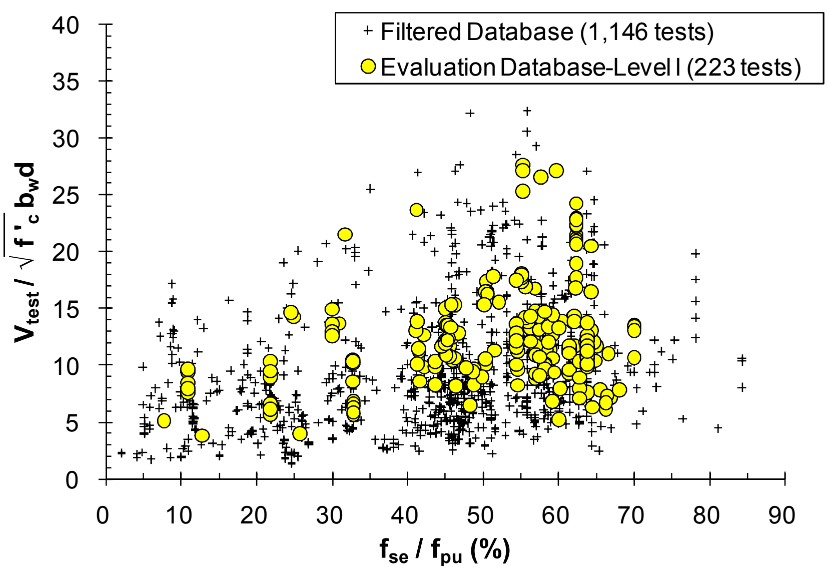


(b) Normalized shear stress at failure

Figure 5.5 Ultimate shear stress versus effective prestress in concrete



(a) Shear stress at failure



(b) Normalized shear stress at failure

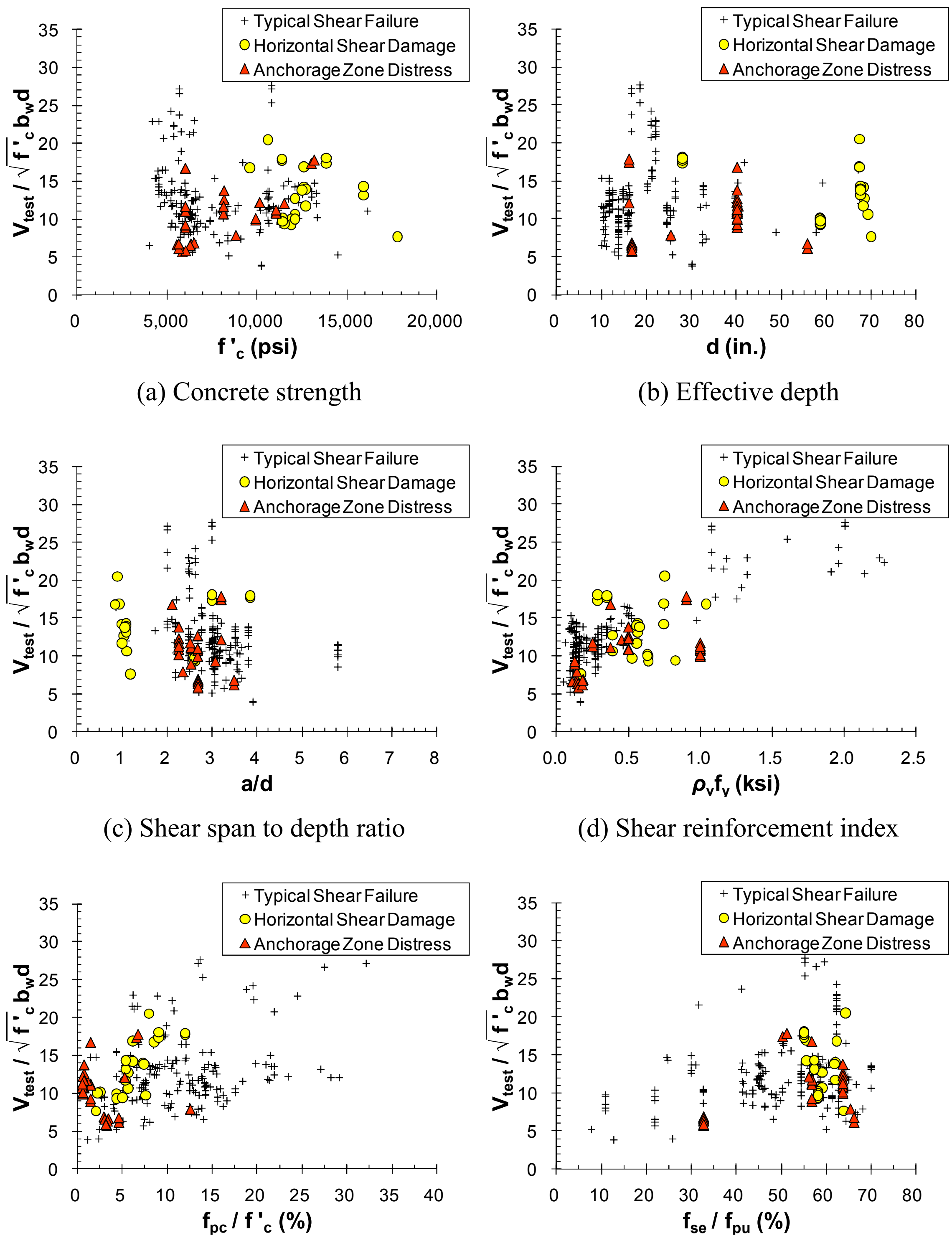
Figure 5.6 Ultimate shear stress versus effective prestress in prestressing steel

5.3 INFLUENCE OF SHEAR FAILURE MODES

The influence of shear failure modes and key experimental variables on the normalized shear stress at failure is shown in Figure 5.7. The data points in Figure 5.7 were obtained from results of the 223 tests included in the evaluation database-level I. Three types of shear failure modes are differentiated in this figure: (1) typical shear failure; (2) shear failure with signs of horizontal shear damage; and (3) shear failure with signs of anchorage zone distress. Similarly, in Figure 5.8, the influence of shear failure modes and experimental variables on the normalized shear stress at failure in the 171 tests included in the evaluation database-level II is presented. In this figure, five shear failure modes are differentiated: (1) shear failure; (2) flexural shear failure; (3) web crushing failure; (4) shear compression failure; and (5) shear tension failure. Again, the readers are encouraged to note that tests without clear descriptions of the shear failure mode are categorized into “shear” in Figure 5.8.

An examination of Figure 5.8 reveals that specimens that failed in web crushing failure carried high shear stresses at failure. Most of these specimens contain large amounts of shear reinforcement as shown in Figure 5.8(d). This implies that too much shear reinforcement results in web crushing failure with or without yielding of shear reinforcement. The adoption of the upper limit on the maximum shear strength seems appropriate to avoid this brittle failure mode. However, it is difficult to specify if web crushing failure occurs before or after yielding of shear reinforcement because of limited information reported in the original references.

Meanwhile, there is no clear relationship between the normalized shear stress at failure and other shear failure modes except for web shear failure. The signs of horizontal shear damage are likely to be observed in specimens with relatively high concrete strengths, large effective depths, and high levels of effective prestress in prestressing steel as shown in Figure 5.7(a), 5.7(b), and 5.7(f). One of the reasons of these trends is that the signs of horizontal shear damage appeared only in full-scale specimens, such as Box-, I- and U-beams. These test specimens were fabricated with high strength concrete, had large overall member heights, and had high levels of effective prestress in prestressing steel. However, it is beyond the scope of this thesis to discuss a detailed mechanics of horizontal shear failure. A further experimental investigation is required to clarify the influence of these experimental variables on horizontal shear failure. The University of Texas at Austin is conducting a research project dealing with horizontal shear failure at the moment (Hovell, et al. 2010).



(a) Concrete strength

(b) Effective depth

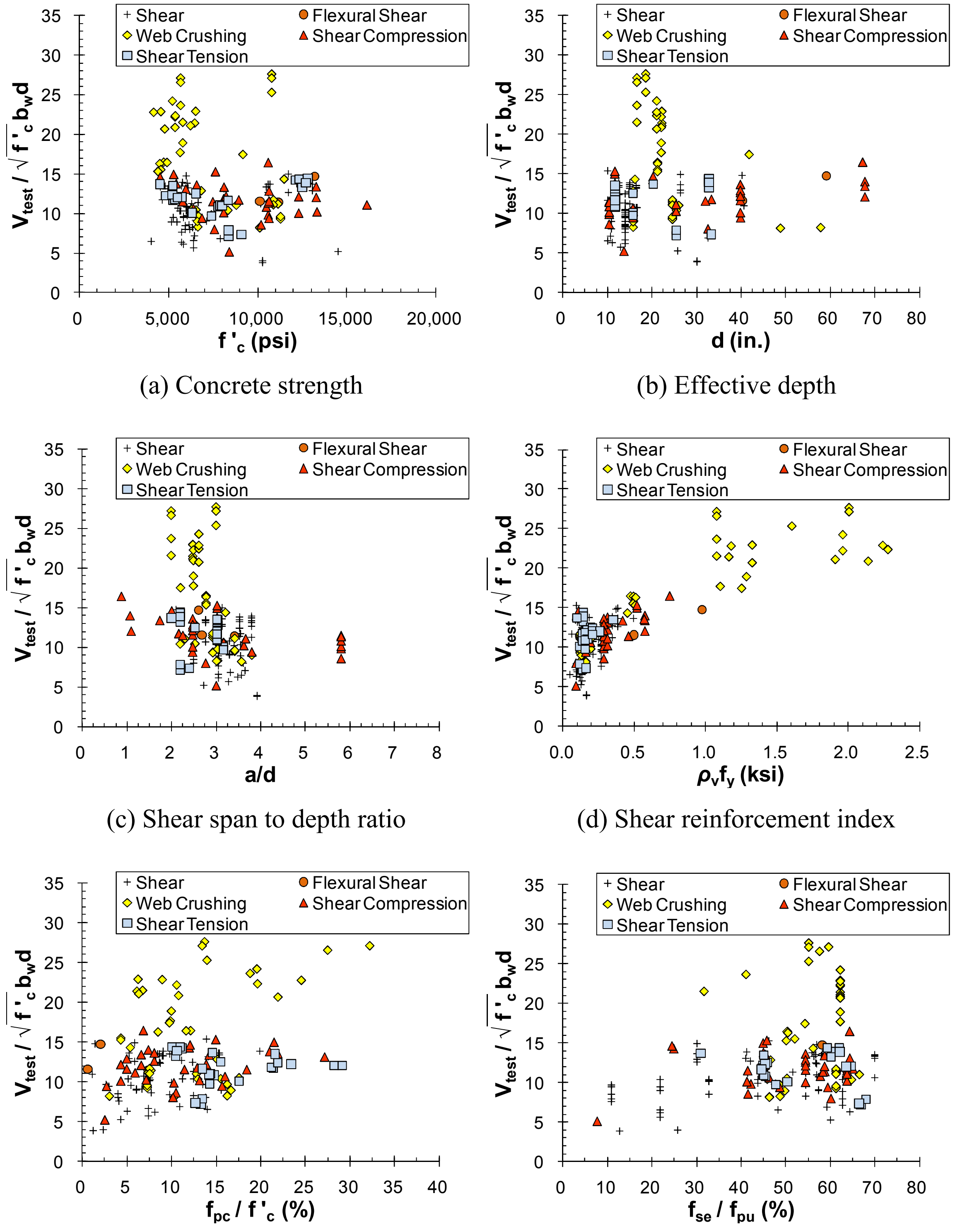
(c) Shear span to depth ratio

(d) Shear reinforcement index

(e) Effective prestress in concrete

(f) Effective prestress in prestressing steel

Figure 5.7 Influence of shear failure modes and experimental variables



(a) Concrete strength (b) Effective depth

(c) Shear span to depth ratio (d) Shear reinforcement index

(e) Effective prestress in concrete (f) Effective prestress in prestressing steel

Figure 5.8 Influence of typical shear failure modes and experimental variables

CHAPTER 6

DATABASE ANALYSIS: SHEAR DESIGN

6.1 OVERVIEW

In this chapter, the UTPCSDB-2011 is used to study shear design provisions in various codes and specifications. First, the accuracy and conservativeness of twelve shear design equations are investigated. Next, the influence of primary experimental variables and shear failure modes on the accuracy of the shear design equations is discussed. In the last part of the chapter, the upper limit on the maximum shear strength permitted in the U.S. design specifications is analyzed by using the UTPCSDB-2011.

6.2 ACCURACY AND CONSERVATIVENESS OF SHEAR DESIGN EQUATIONS

6.2.1 Shear Design Equations

The shear design equations that are evaluated in this chapter are:

- ACI 318-08 simplified method (ACI Committee 318 2008);
- ACI 318-08 detailed method (ACI Committee 318 2008);
- AASHTO LRFD 2007 general procedure (AASHTO 2007);
- AASHTO LRFD 2010 general procedure (AASHTO 2010);
- AASHTO LRFD 2010 simplified procedure (AASHTO 2010);
- AASHTO LRFD 2010 segmental procedure with the limit on the K factor (AASHTO 2010);
- AASHTO LRFD 2010 segmental procedure without the limit on the K factor (AASHTO 2010);
- CSA A23.3-04 general method (CSA 2010);

- JSCE 2007 (Japan Society of Civil Engineers 2007);
- JRA 2002 (Japan Road Association 2002);
- fib Model Code 2010 (fib 2010a, 2010b); and
- EN 1992 (European Committee for Standardization 2004).

Two different methods of the AASHTO LRFD general procedure are separately evaluated. One method employs Tables 2.2 and 2.3 to calculate β and θ (AASHTO 2007), and the other method is based on the simplified equations presented as Equations 2-15 through 2-19 (AASHTO 2010).

The AASHTO LRFD 2010 segmental procedure with and without the limit on the K factor are also evaluated. This analysis is presented by taking into account Avendaño and Bayrak's recommendation (Avendaño and Bayrak 2008). These researchers indicated that the accuracy of the AASHTO LRFD segmental procedure is highly improved by removing the limits on the K factor and the concrete strength on the basis of their database analysis with the first version of the UTPCSDB.

6.2.2 Evaluation Procedure

The shear strength of each test specimen was calculated with those twelve shear design equations in accordance with the following general rules:

- <u>All strength reduction factors are taken as 1.0.</u>
  Since different design specifications adopt completely different approaches on strength reduction factors, all strength reduction factors are set to be equal to 1.0. It is beyond the scope of this evaluation to discuss an appropriate application of the strength reduction factors.

- <u>Upper and lower limits on the material strengths are not used.</u>
  Although design specifications set different upper and lower limits on the material strengths, no limit is imposed on the calculation procedure of the twelve shear design equations.

- <u>Limits on the shear design equations follow the procedures of the design codes in which they appear.</u>
  As mentioned in Chapter 2, there are significant differences in not only shear design equations themselves but also provisions on the minimum shear reinforcement and the maximum shear strength. In this evaluation, each shear design equation follows provisions specified in its own design specifications.

(1) filtered database (1,146 tests); (2) evaluation database-level I (223 tests); and (3) evaluation database-level II (171 tests) were used for this evaluation.

The variations in measured versus computed shear strength with the twelve shear design equations were mainly evaluated by using statistics of the shear strength ratio, which was calculated by dividing the shear force carried by test specimens, V_{test}, by the shear strength estimated with the shear design equations, V_{calc}.

6.2.3 Statistics of Shear Strength Ratio

The accuracy and conservativeness of the twelve shear design equations were compared in terms of three statistical parameters of the shear strength ratio: (1) mean; (2) COV: coefficient of variation; and (3) percentage of unconservative cases. A desirable shear design equation is deemed to have the following statistical characteristics:

- Mean

  The mean value of the desirable shear design equation should be close to, but greater than, 1.0 for accuracy and to avoid unconservative strength estimations.

- COV: coefficient of variation

  The desirable shear design equation shows low COV. This implies that the equation has low variation in its shear strength estimations, i.e. the scatter is minimized.

- Percentage of unconservative cases

  Although the percentage of unconservative cases needs to be low for safe shear design, overly conservative estimations of the shear strength are also undesirable. Therefore, we should focus on both values of the mean and percentage of unconservative cases at the same time.

The statistics of the shear strength ratio of each shear design equation in the three types of the UTPCSDB-2011 are summarized in Tables 6.1 through 6.3. The top five preferable values of the mean, COV, and percentage of unconservative cases are highlighted in the tables. Additionally, the mean, COV, and percentage of unconservative cases among the twelve shear design equations in the three databases are compared in Figures 6.1 through 6.3.

6.2.3.1 Shear strength ratio: Mean

In Figure 6.1, the mean values of all shear design equations are larger than 1.0 in all databases. All shear design equations provide conservative estimations of the shear strength. Additionally, the mean values obtained by using the evaluation database-level I and II are likely to be lower than those seen in the analysis results obtained by using the filtered database. This implies that the accuracy of the shear design equations is highly improved by removing irrelevant tests from the filtered database.

In Tables 6.2 and 6.3, the mean values of the following six equations are lower than or equal to 1.50 in the evaluation database-level I and II: (1) ACI 318-08 detailed method; (2) AASHTO LRFD 2007 general procedure; (3) AASHTO LRFD 2010 general procedure; (4) AASHTO LRFD 2010 simplified procedure; (5) CSA A23.3-04; and (6) Model Code 2010. These six shear design equations provide more accurate strength estimations than other shear design equations in terms of the mean shear strength ratio.

6.2.3.2 Shear strength ratio: Coefficient of variation

An examination of Figure 6.2 indicates that the COV values are greatly reduced if results from the evaluation database-level II is used in the analysis. All twelve shear design equations share this trend. The distributions of the shear strength ratio of the twelve shear design equations in each database are shown in Figures 6.4 through 6.6. These figures reveal that the distributions of the shear strength ratio are highly related to the COV; the equations that have low COV values show dense histograms. It should be noted that the distributions of the shear strength ratio is dense in the following order: (1) the evaluation database-level II (Figure 6.6); (2) the evaluation database-level I (Figure 6.5); and (3) the filtered database (Figure 6.4). In summary, it can be concluded that the twelve shear design equations succeed to provide less scattered estimations to the tests that displayed the typical shear failure modes since the COV in the evaluation database-level II is lower than those in other two databases.

In Tables 6.2 and 6.3, the COV values of the following six equations are lower than 0.25 in both the evaluation database-level I and II: (1) ACI 318-08 detailed method; (2) AASHTO LRFD 2007 general procedure; (3) AASHTO LRFD 2010 general procedure; (4) AASHTO LRFD 2010 segmental procedure without the limit on the K factor; (5) CSA A23.3-04; and (6) Model Code 2010. In terms of the COV, these six equations provide shear strength estimations with the least scatter.

6.2.3.3 Percentage of unconservative cases

In Figure 6.3, most of the shear design equations have the lowest percentage of unconservative cases in the evaluation database-level II. Again, it is important to note that both the mean and percentage of unconservative cases should be considered at the same time to avoid overly conservative estimations. In addition to the previous criterion on the mean of up to 1.50, another criterion that limits the percentage of unconservative cases up to 5.0% is employed. In Tables 6.3, the following four shear design equations satisfy both criteria: (1) AASHTO LRFD 2007 general procedure; (2) AASHTO LRFD 2010 general procedure; (3) CSA A.23.3-04; and (4) Model Code 2010. It is interesting to stress that these three equations are based on the MCFT.

6.2.4 Comparison of Shear Design Equations

With a careful examination on the statistics shown in Tables 6.1 through 6.3, the twelve shear design equations are categorized into four approaches on the basis of the statistical characteristics of the shear strength ratio and those equations' basic concepts. These four approaches are as follows: (1) MCFT-based design expressions; (2) design expressions based on the ACI detailed approach (V_{ci}, V_{cw}); (3) design expressions based on the 45 degree truss model; and (4) design expressions based on the variable angle truss model. The characteristics of the four approaches are summarized as follows.

6.2.4.1 MCFT-based design expressions

The MCFT-based design expressions are adopted in the following four shear design provisions: (1) AASHTO LRFD 2007 general procedure; (2) AASHTO LRFD 2010 general procedure; (3) CSA A23.3-04; and (4) Model Code 2010. The shear design equations in the AASHTO LRFD 2007 general procedure calculate β and θ with the given tables, and other three expressions use the simplified formulas.

As shown in Tables 6.1 through 6.3, those four shear design equations provide almost similar statistics of the shear strength ratio. On the basis of the statistics in the evaluation database-level I and II, the MCFT-based design expressions produce the best performance to estimate the shear strength on prestressed concrete members with sufficient shear reinforcement. These shear design provisions result in low values of the mean, COV, and percentage of unconservative cases.

As mentioned in Chapter 2, all MCFT-based design expressions are functionally equivalent despite the several minor differences in their expressions. Those differences are negligible because the statistical parameters obtained by using all of these expressions are very similar.

6.2.4.2 Design expressions based on ACI detailed approach (V_{ci}, V_{cw})

The ACI 318-08 detailed method suggests calculating the V_c term by taking the lesser of V_{ci} and V_{cw}. The following two design methods are categorized as the design expressions based on the ACI detailed approach: (1) ACI 318-08 detailed method; and (2) AASHTO LRFD 2010 simplified procedure. The latter design equation is modified from the former by revising the equation of V_{cw} and the V_s term.

In Tables 6.2 and 6.3, the mean and COV of the design expressions based on the ACI detailed approach are comparable with those of the MCFT-based design expressions. The design expressions based on the ACI detailed approach, however, has slightly higher percentages of unconservative cases than those of the MCFT-based design expressions. Therefore, it is reasonable to conclude that the design expressions based on the ACI detailed approach provide the second best estimation of the shear strength of prestressed concrete members with sufficient shear reinforcement.

Additionally, the AASHTO LRFD 2010 simplified procedure produces slightly more conservative estimations than those of the ACI 318-08 detailed method in terms of the mean shear strength ratio. The primary reasons for this conservativeness are that (1) the AASHTO LRFD 2010 simplified procedure was revised in order to make this method applicable to both reinforced and prestressed concrete members and (2) the UTPCSDB-2011 consists only of shear tests conducted on prestressed concrete members.

6.2.4.3 Design expressions based on 45 degree truss model

This approach is based on a combination of the empirically-derived V_c term and the 45 degree truss model. The following five shear design equations are included in this approach: (1) ACI 318-08 simplified method; (2) AASHTO LRFD 2010 segmental procedure with the limit on the K factor; (3) AASHTO LRFD 2010 segmental procedure without the limit on the K factor; (4) JSCE 2007; and (5) JRA 2002.

In tables 6.1 through 6.3, these five shear design equations show similar trends of the shear strength ratio; although the mean and COV are higher than those of the design expressions based on the MCFT and ACI detailed approach, the percentage of unconservative cases is lower. In other words, the design expressions based on the 45 degree truss model provide more conservative strength estimations than design expressions based on the MCFT and ACI detailed approach.

As for the AASHTO LRFD 2010 segmental procedure, the statistical parameters listed in Tables 6.2 and 6.3 indicate that the mean and COV are decreased by removing the limit on the K factor. Despite a slight increase in the percentage of unconservative cases, the AASHTO LRFD 2010 segmental procedure still provides more conservative strength estimations than those of the design expressions based on the MCFT and ACI detailed approach. This evidence implies that the removal of the limit on the K factor improves the accuracy of the AASHTO LRFD 2010 segmental procedure.

6.2.4.4 Design expressions based on variable angle truss model

The EN 1992 shear design equations estimate the shear strength of members with sufficient shear reinforcement by calculating only the V_s term. Note that the V_s term in EN 1992 is based on the variable angle truss model. In tables 6.1 through 6.3, although EN1992 provides conservative estimations, the COV is larger than 0.50 in all databases.

It should also be recognized that the strength estimations obtained by using the design provisions of EN 1992 are highly scattered.

Table 6.1 Statistics of shear strength ratio in filtered database (N = 1,146 tests)

	ACI 318-08		*AASHTO LRFD*					*CSA A23.3 -04*	*JSCE 2007*	*JRA 2002*	*fib Model Code 2010*	*EN 1992*
	Simplified Method	*Detailed Method*	*General Procedure*		*Simplified Procedure 2010*	*Segmental Procedure 2010*						
			2007	*2010*		*w/ K limit*	*w/o K limit*					
Min	0.46	0.51	0.58	0.59	0.52	0.60	0.41	0.55	0.66	0.81	0.62	0.54
Max	7.52	5.30	7.04	7.37	9.49	11.82	9.40	6.55	5.94	22.03	6.10	16.17
Mean	1.98	**1.54**	**1.65**	**1.67**	1.75	2.79	1.85	**1.62**	1.92	3.66	**1.56**	2.36
COV	0.46	**0.35**	**0.39**	**0.40**	0.44	0.56	0.48	**0.40**	**0.35**	0.61	**0.37**	0.68
Unconservative case	90	89	67	69	67	39	121	73	43	3	86	56
Unconservative %	7.9	7.8	**5.8**	6.0	**5.8**	**3.4**	10.6	6.4	**3.8**	**0.3**	7.5	**4.9**

Table 6.2 Statistics of shear strength ratio in evaluation database-level I (N = 223 tests)

	ACI 318-08		*AASHTO LRFD*					*CSA A23.3 -04*	*JSCE 2007*	*JRA 2002*	*fib Model Code 2010*	*EN 1992*
	Simplified Method	*Detailed Method*	*General Procedure*		*Simplified Procedure 2010*	*Segmental Procedure 2010*						
			2007	*2010*		*w/ K limit*	*w/o K limit*					
Min	0.86	0.73	0.62	0.62	0.69	0.81	0.81	0.61	0.79	1.11	0.62	0.59
Max	3.55	2.32	2.27	2.07	2.55	4.41	2.73	2.09	3.48	4.91	2.32	6.21
Mean	1.79	**1.35**	**1.35**	**1.36**	**1.42**	2.21	1.68	**1.36**	1.86	2.59	**1.42**	2.20
COV	0.29	**0.22**	**0.23**	**0.22**	0.27	0.33	0.24	**0.22**	0.31	0.31	**0.23**	0.54
Unconservative case	8	26	24	19	26	8	10	18	11	0	15	17
Unconservative %	**3.6**	11.7	10.8	8.5	11.7	**3.6**	**4.5**	8.1	**4.9**	**0.0**	6.7	7.6

Table 6.3 Statistics of shear strength ratio in evaluation database-level II (N = 171 tests)

	ACI 318-08		*AASHTO LRFD*					*CSA A23.3 -04*	*JSCE 2007*	*JRA 2002*	*fib Model Code 2010*	*EN 1992*
	Simplified Method	*Detailed Method*	*General Procedure*		*Simplified Procedure 2010*	*Segmental Procedure 2010*						
			2007	*2010*		*w/ K limit*	*w/o K limit*					
Min	0.98	0.82	0.91	0.94	0.81	1.05	0.86	0.95	0.96	1.17	0.99	0.88
Max	3.11	2.32	2.27	2.07	2.39	4.41	2.73	2.08	3.48	4.91	2.32	6.21
Mean	1.90	**1.39**	**1.43**	**1.43**	**1.43**	2.38	1.73	**1.43**	1.95	2.74	1.50	2.32
COV	0.23	**0.21**	**0.19**	**0.18**	0.25	0.29	**0.21**	**0.19**	0.28	0.28	**0.20**	0.52
Unconservative case	1	11	4	1	15	0	2	2	4	0	1	5
Unconservative %	**0.6**	6.4	2.3	**0.6**	8.8	**0.0**	1.2	1.2	2.3	**0.0**	**0.6**	2.9

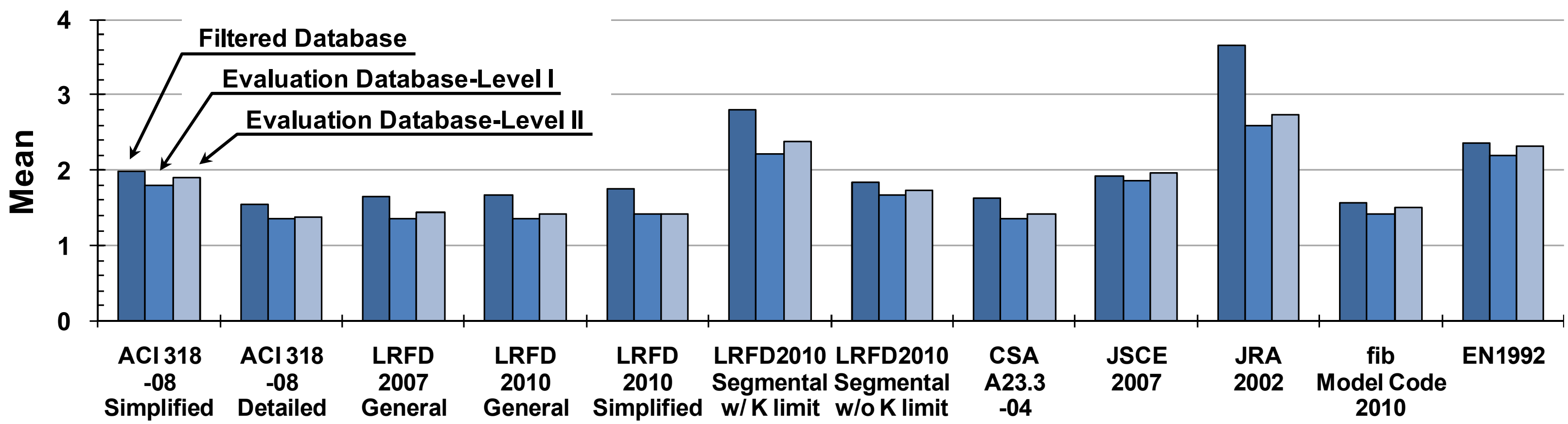


Figure 6.1 Comparison of mean calculated for shear strength ratio

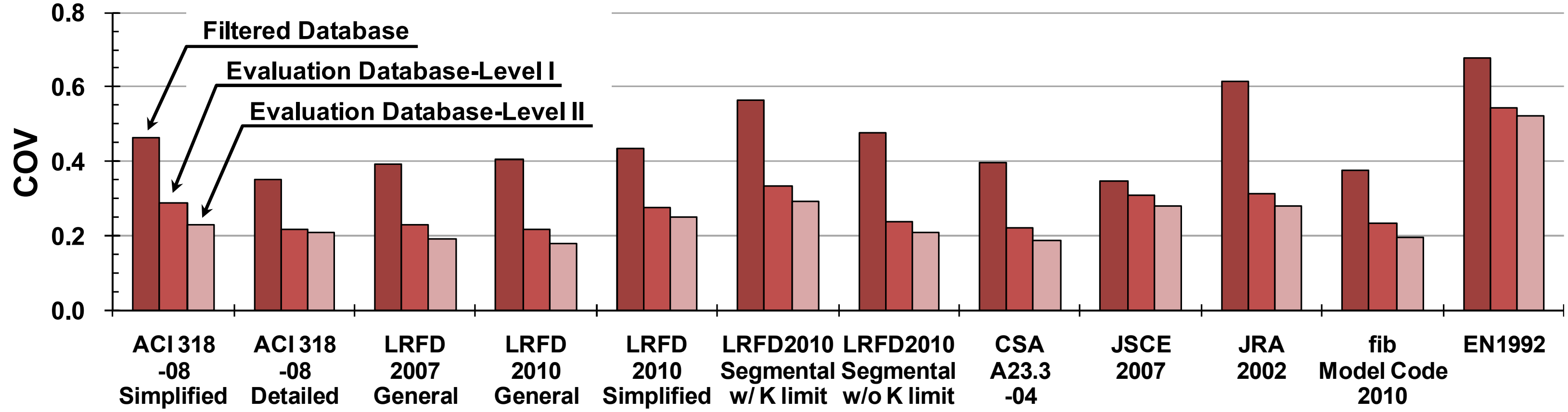


Figure 6.2 Comparison of COV calculated for shear strength ratio

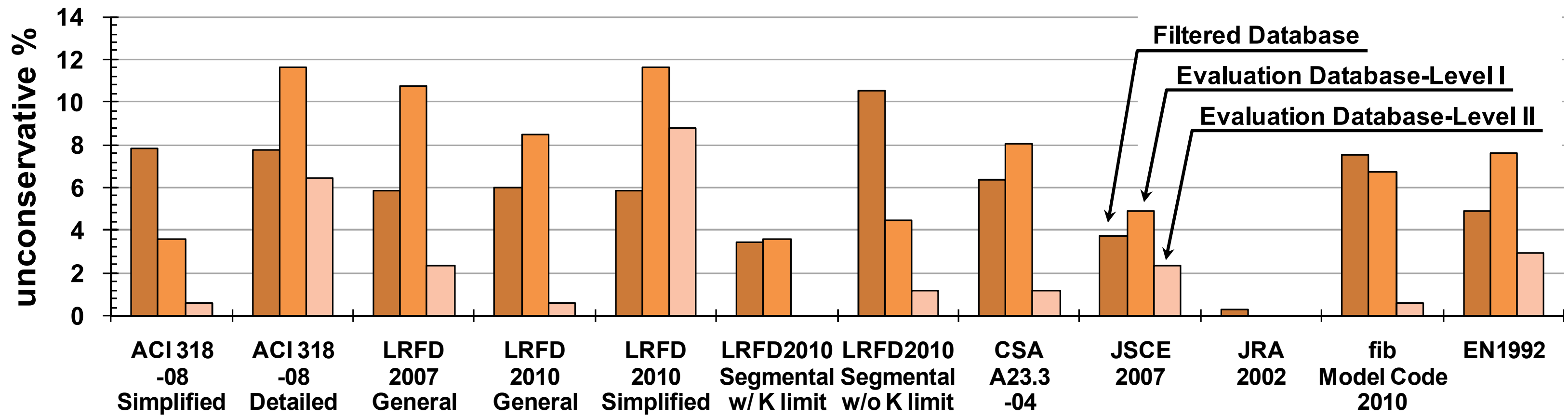
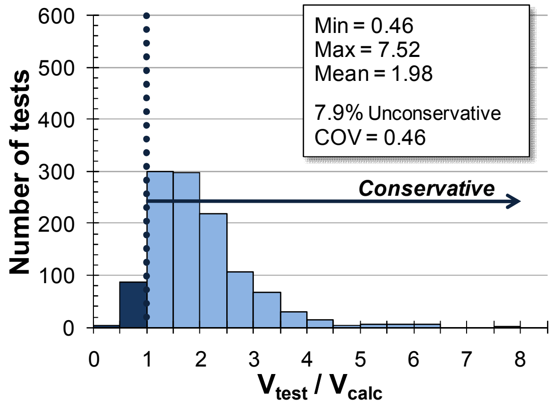
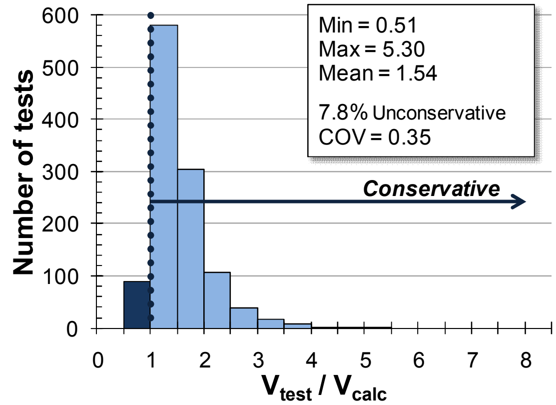


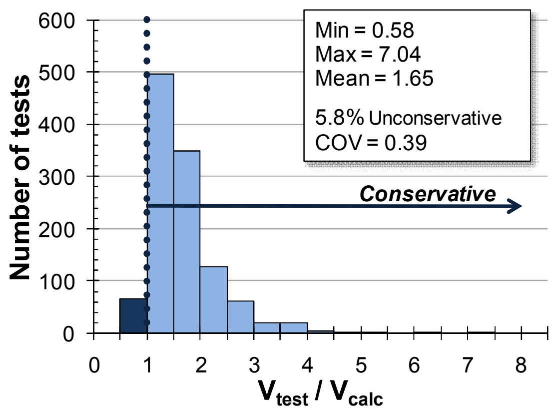
Figure 6.3 Comparison of unconservative % calculated for shear strength ratio



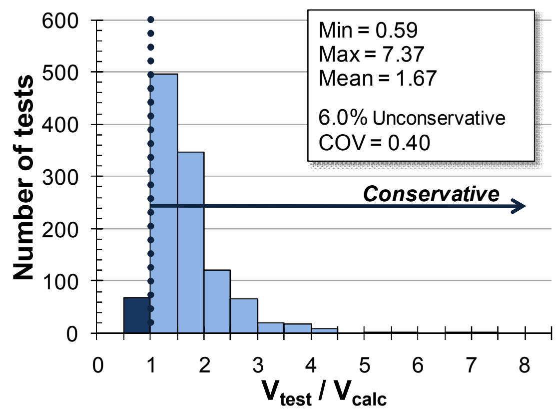
(a) ACI 318-08 Simplified Method



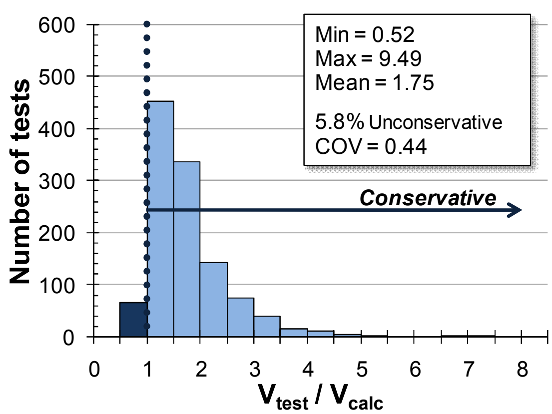
(b) ACI 318-08 Detailed Method



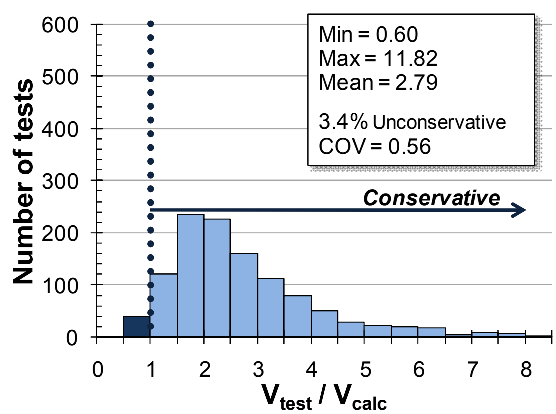
(c) LRFD 2007 General Procedure



(d) LRFD 2010 General Procedure

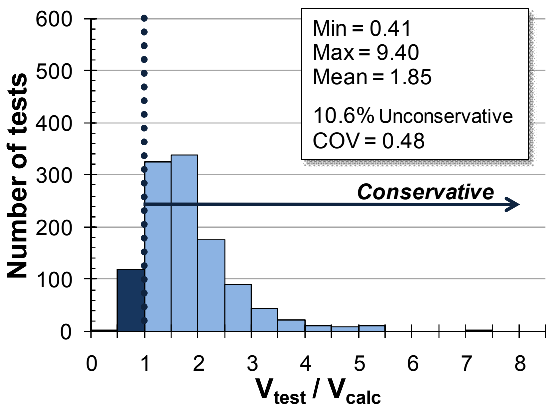


(e) LRFD 2010 Simplified Procedure

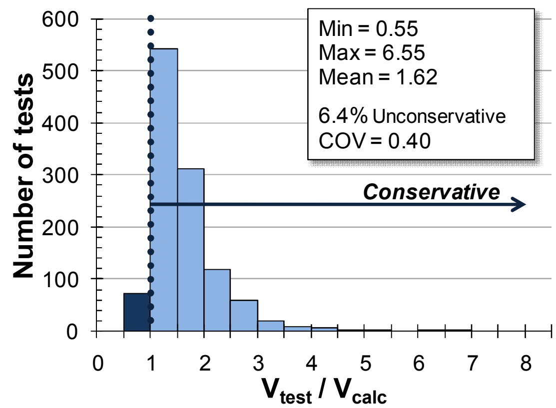


(f) LRFD 2010 Segmental w/ K limit

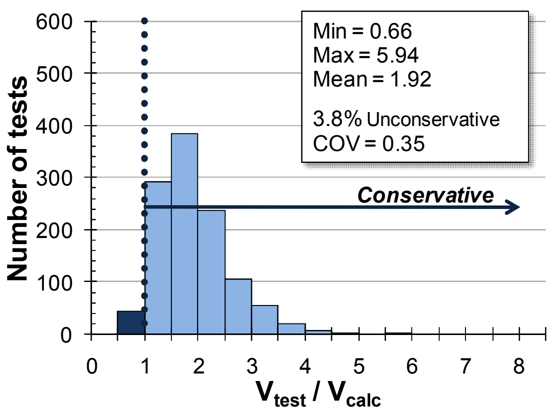
Figure 6.4 Comparison of shear design ratio in filtered database (1/2)



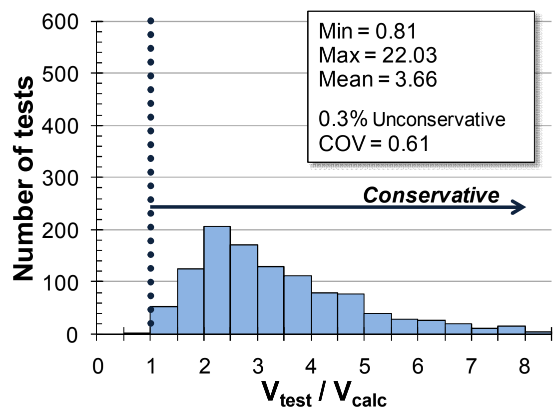
(g) LRFD 2010 Segmental w/o K limit



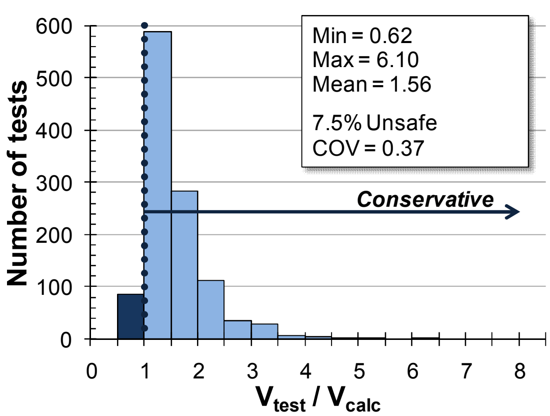
(h) CSA.A23.03-04



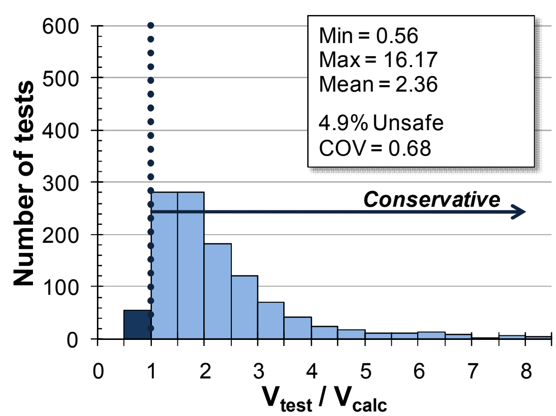
(i) JSCE 2007



(j) JRA 2002

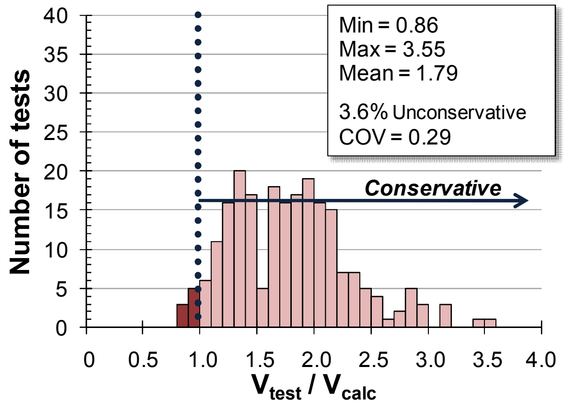


(k) fib Model Code 2010

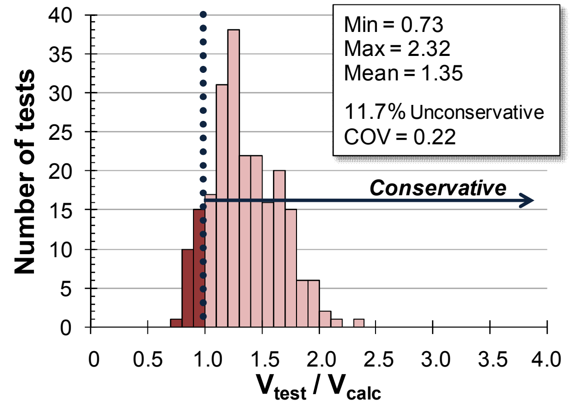


(l) EN 1992

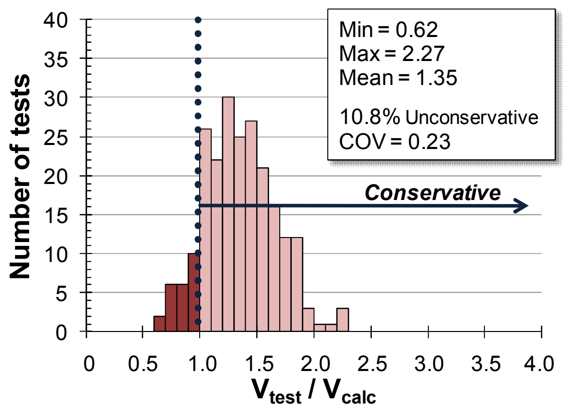
Figure 6.4 Comparison of shear design ratio in filtered database (2/2)



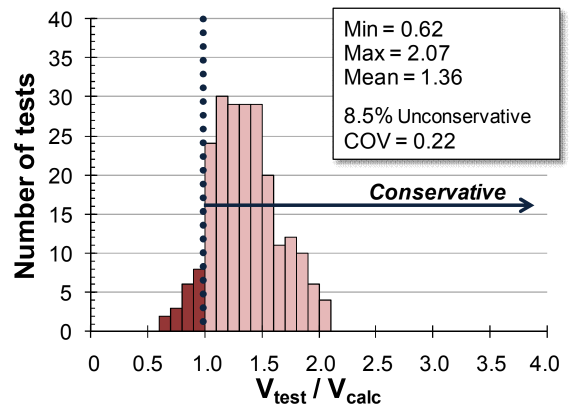
(a) ACI 318-08 Simplified Method



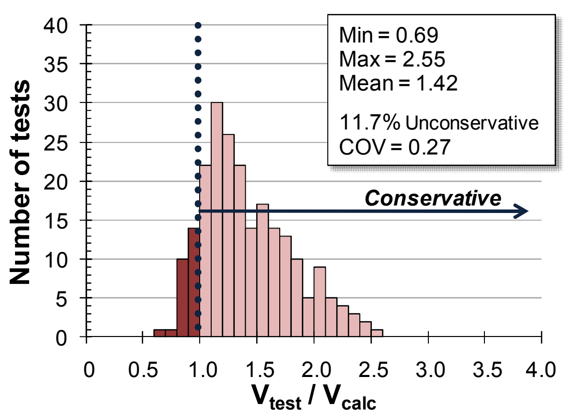
(b) ACI 318-08 Detailed Method



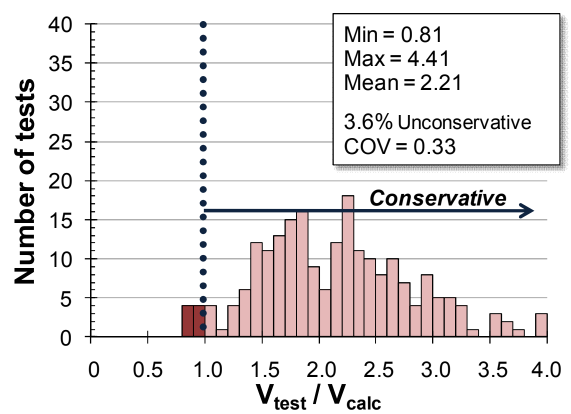
(c) LRFD 2007 General Procedure



(d) LRFD 2010 General Procedure



(e) LRFD 2010 Simplified Procedure



(f) LRFD 2010 Segmental w/ K limit

Figure 6.5 Comparison of shear design ratio in evaluation database-level I (1/2)

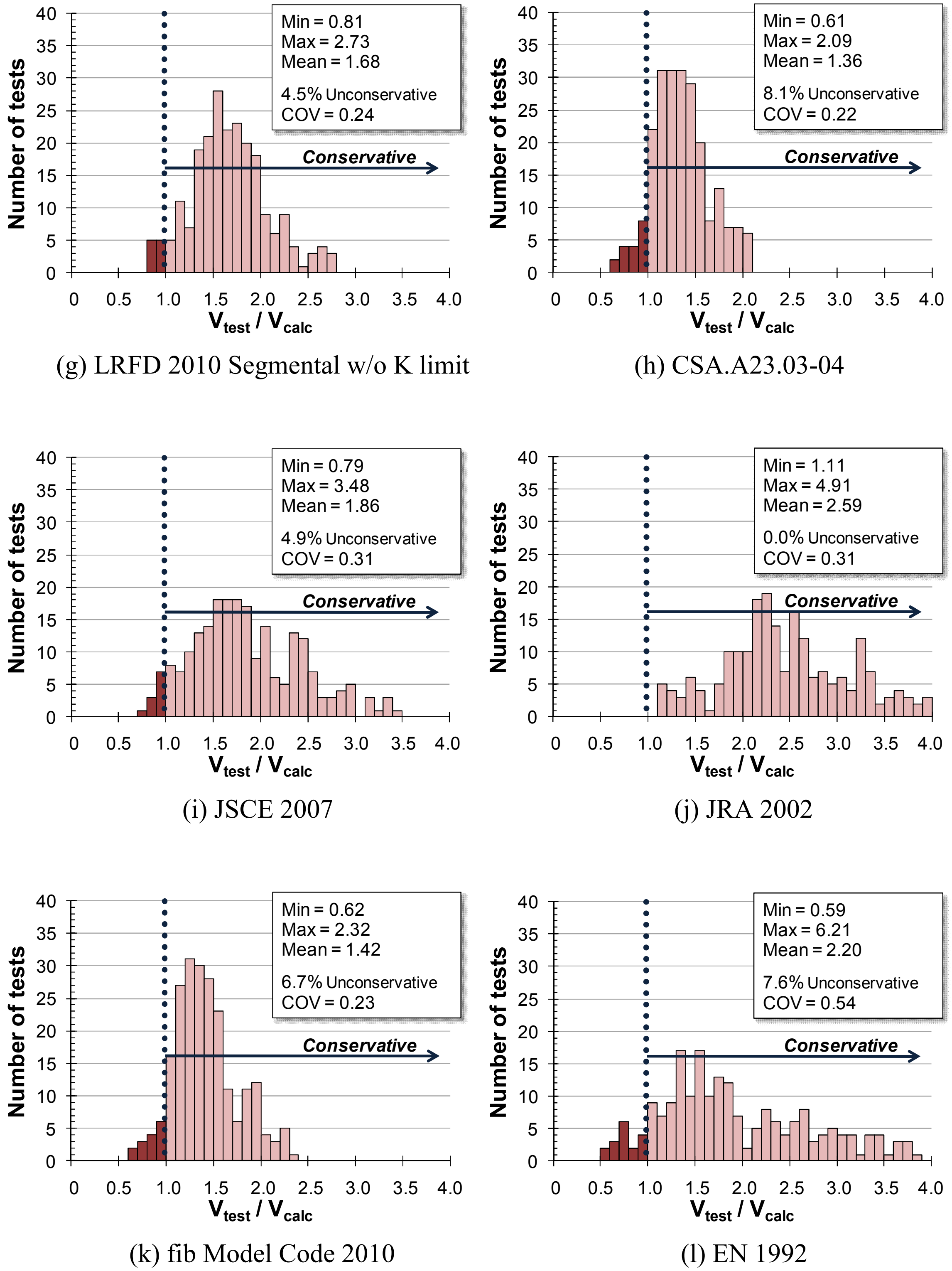
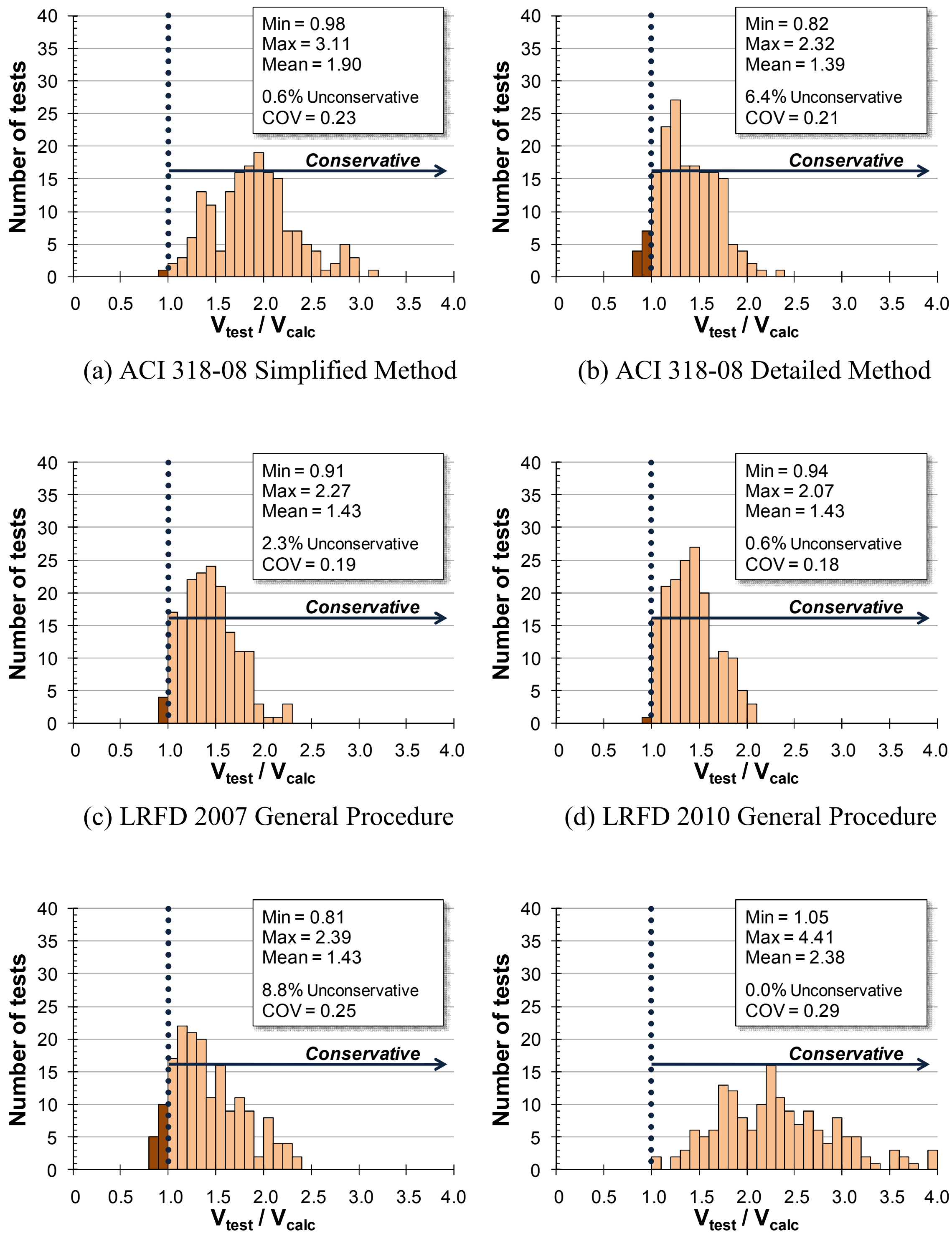


Figure 6.5 Comparison of shear design ratio in evaluation database-level I (2/2)



(a) ACI 318-08 Simplified Method

(b) ACI 318-08 Detailed Method

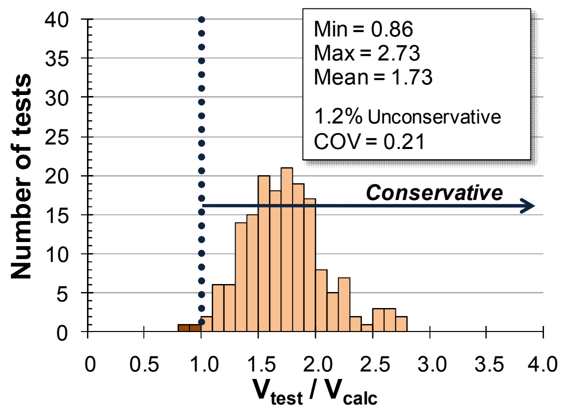
(c) LRFD 2007 General Procedure

(d) LRFD 2010 General Procedure

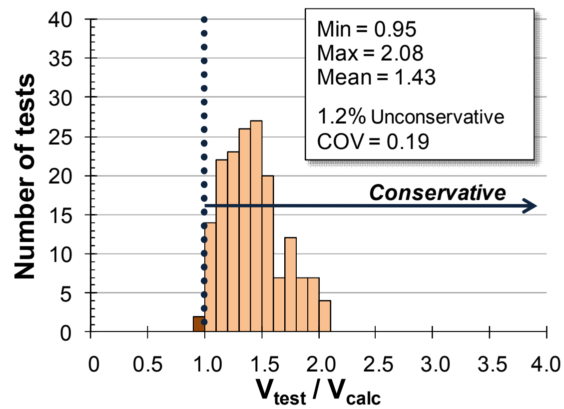
(e) LRFD 2010 Simplified Procedure

(f) LRFD 2010 Segmental w/ K limit

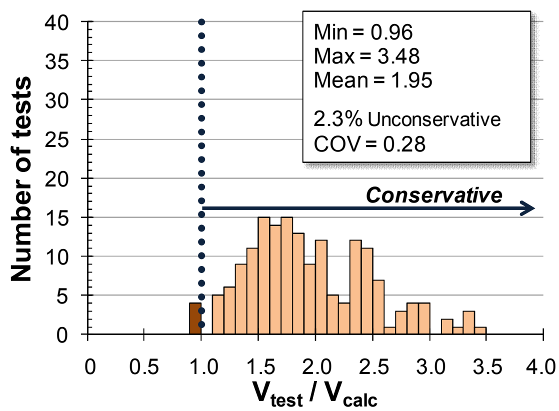
Figure 6.6 Comparison of shear design ratio in evaluation database-level II (1/2)



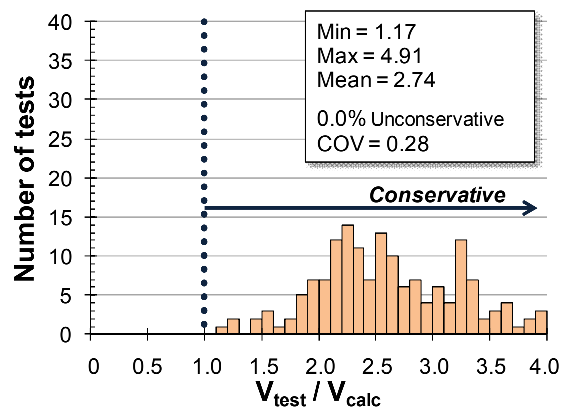
(g) LRFD 2010 Segmental w/o K limit



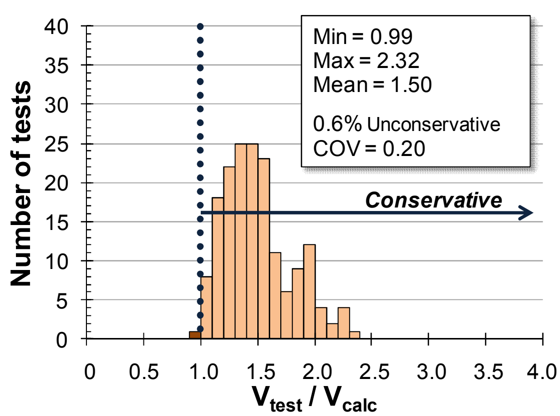
(h) CSA.A23.03-04



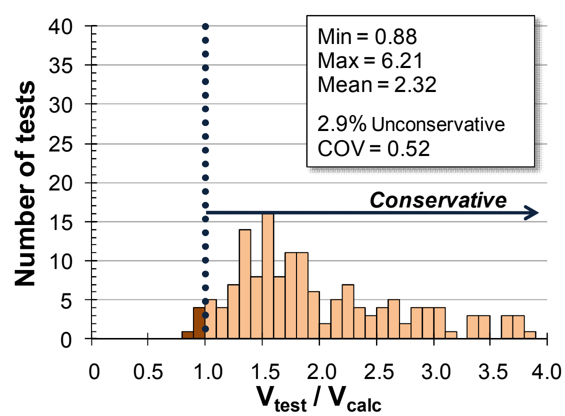
(i) JSCE 2007



(j) JRA 2002



(k) fib Model Code 2010



(l) EN 1992

Figure 6.6 Comparison of shear design ratio in evaluation database-level II (2/2)

6.2.5 Influence of Experimental Variables and Shear Failure Modes

In this section, an analysis is conducted to clarify the influence of experimental variables and shear failure modes on the accuracy and conservativeness of the twelve shear design equations. In Figures 6.7 through 6.18, the relationships among a given experimental variable, shear failure mode, and shear strength ratio for the twelve shear design equations are shown. Three types of shear failure modes are differentiated in these figures: (1) typical shear failure; (2) shear failure with signs of horizontal shear damage; and (3) shear failure with signs of anchorage zone distress. The findings of this analysis are summarized by using the four types of design expressions that were defined in the previous section.

6.2.5.1 MCFT-based design expressions

The shear strength ratios obtained from the MCFT-based design expressions are affected by the type of shear failure modes. An examination of Figures 6.9, 6.10, 6.14, and 6.17 reveals that most of the unconservative estimations consist of tests in which shear failure with signs of horizontal shear damage and/or anchorage zone distress ware observed. On the other hand, the MCFT-based design expressions provide conservative strength estimations to most of the specimens that experienced typical shear failure. This trend enables the MCFT-based design expressions to produce the best performance to estimate the shear strength for specimens that failed in shear without signs of horizontal shear damage and/or anchorage zone distress, i.e., typical shear failure, as shown in Table 6.3.

In Figures 6.9, 6.10, 6.14, and 6.17, the shear strength ratios calculated by using the MCFT-based design expressions decrease in specimens with effective depths of greater than 45 in. One of the reasons for this decrease in the shear strength ratio is that most of the specimens with effective depths of greater than 45 in. exhibited signs of horizontal shear damage and/or anchorage zone distress. Additional data on large test specimens are required to clarify the influence of the effective depth on the accuracy of the MCFT-based design expressions. The MCFT-based design expressions show no bias on other experimental variables.

Again, the readers are encouraged to note that it is beyond of the scope of this thesis to discuss the mechanics of horizontal shear failure. A clear understanding of horizontal shear failure is indispensable for estimating the shear strength accurately and conservatively by using the MCFT-based design expressions.

6.2.5.2 Design expressions based on ACI detailed approach (V_{ci}, V_{cw})

Unlike the MCFT-based design expressions, an examination of Figures 6.8 and 6.11 reveals that the design expressions based on the ACI detailed approach has no strong relationship between the shear strength ratio and the shear failure mode. This discrepancy seems to be caused by different definitions of the V_c and V_s terms in these two approaches. The MCFT-based design expressions define the V_c term as the tensile strength of diagonally cracked concrete, and calculates the V_s term by using the angle of the diagonal compressive strut, which is lower than 45 degrees in general. On the other hand, the design expressions based on the ACI detailed approach assumes the V_c term as the shear at the formation of inclined cracks (i.e., the lesser of V_{ci} and V_{cw}), and gives the V_s term with the traditional 45 degree truss model except for the web shear region in the AASHTO LRFD 2010 simplified procedure. It should be noted that horizontal shear damage and anchorage zone distress typically occur after the formation of inclined cracks but prior to other typical shear failures in most test specimens. Moreover, an occurrence of those two damages is unrelated to yielding of shear reinforcement. This implies that the lower angle of the diagonal compressive strut in the MCFT-based design expressions is likely to result in an overestimation of the V_s term, and hence leads to unconservative estimations of the shear strength for test specimens in which horizontal shear damage and/or anchorage zone distress were observed. In contrast, the empirical assumptions of the design expressions based on the ACI detailed approach contribute to provide conservative estimations in cases where signs of those two damages were observed prior to typical shear failure.

Additionally, in Figure 6.8, the shear strength ratios calculated by using the ACI 318-08 detailed method show no bias on the experimental variables. On the other hand, the AASHTO LRFD 2010 simplified procedure gives overly conservative strength estimations for test specimens with low shear reinforcement indices and low levels of effective presstress in concrete in Figure 6.11.

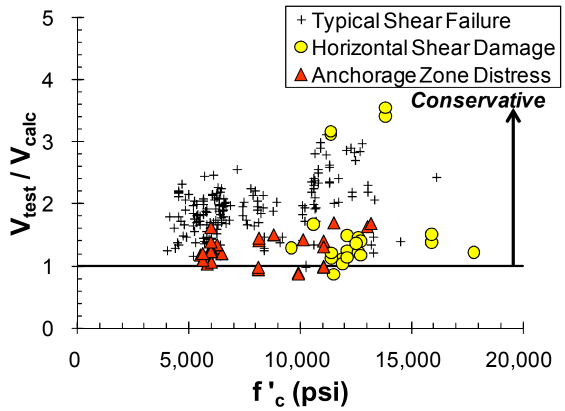
6.2.5.3 Design expressions based on 45 degree truss model

The ACI 318-08 simplified method, the AASHTO LRFD 2010 segmental procedure with and without the limit on the K factor, the JSCE 2007 specifications, and the JRA 2002 specifications present similar trends in the relationship between the shear strength ratio and each of the key experimental valuables as shown in Figures 6.7, 6.12, 6.13, 6.15, and 6.16. These five shear design equations provide overly conservative strength estimations for specimens with low shear reinforcement indices. There are two reasons to explain these conservative strength estimations. One reason is that design expressions in this category make two conservative assumptions on (1) the empirically-derived V_c term and (2) the 45 degree truss model. The other reason is that the shear strengths of specimens with high shear reinforcement indices are restricted by the limit on the maximum shear strength. In other words, the shear strength estimations for specimens with low shear reinforcement indices are controlled by those two conservative assumptions because the shear strength calculations for these specimens are not influenced by the limit on the maximum shear strength.

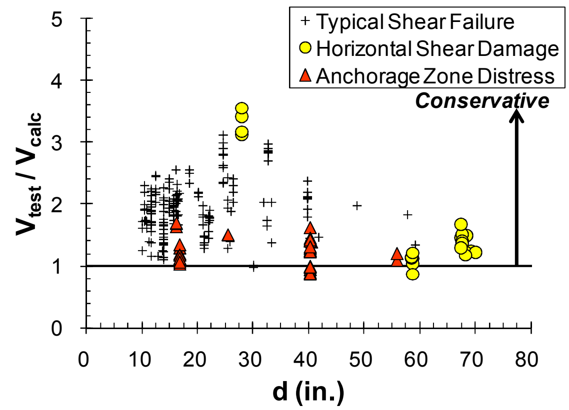
Once again, the readers are encouraged to note that the accuracy of the AASHTO LRFD 2010 segmental procedure is highly improved by removing the limit on the K factor as shown in Figures 6.12 and 6.13. Since the K factor restricts an increase in the V_c term due to prestress force, the improvement of the accuracy is notable in specimens with high levels of effective prestress in concrete. At the same time, this improvement is limited for specimens with low shear reinforcement indices because the shear strength of these specimens is not controlled by the limit on the maximum shear strength.

6.2.5.4 Design expressions based on variable angle truss model

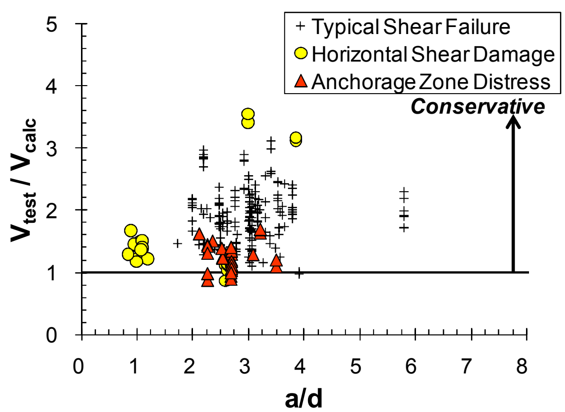
In Figure 6.18, the EN 1992 shear design equations provide overly conservative strength estimations for specimens with low shear reinforcement indices and those with high levels of effective prestress. There are two reasons to explain this unnecessary level of conservativeness. One reason relates to the fact that the V_c term is neglected. Although the variable angle truss model in EN 1992 enables the engineers to select the angle of the diagonal compressive strut other than 45 degrees, the exclusion of the V_c term seems to contribute to overly conservative strength estimations especially for specimens with high levels of effective prestress. The other reason is that the limit on the maximum shear strength is not applied to specimens with low shear reinforcement indices.



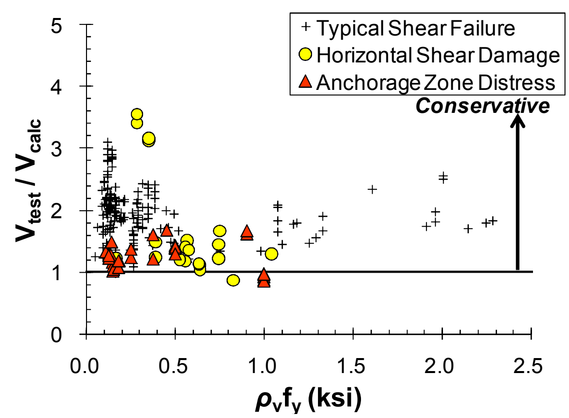
(a) Concrete strength



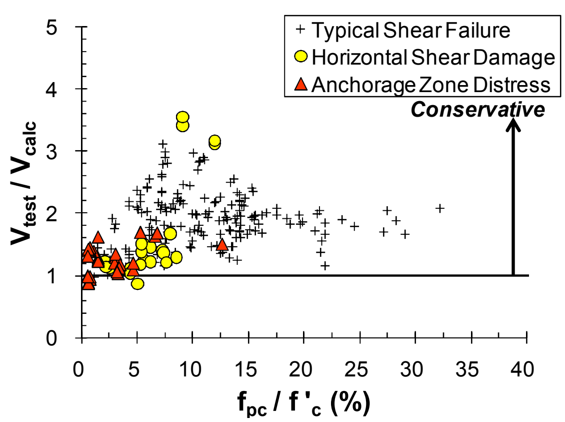
(b) Effective depth



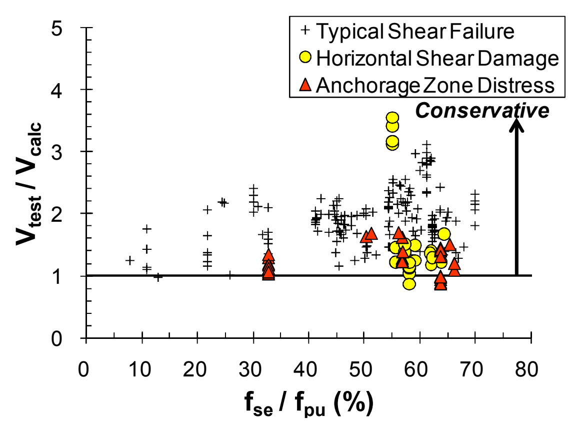
(c) Shear span to depth ratio



(d) Shear reinforcement index

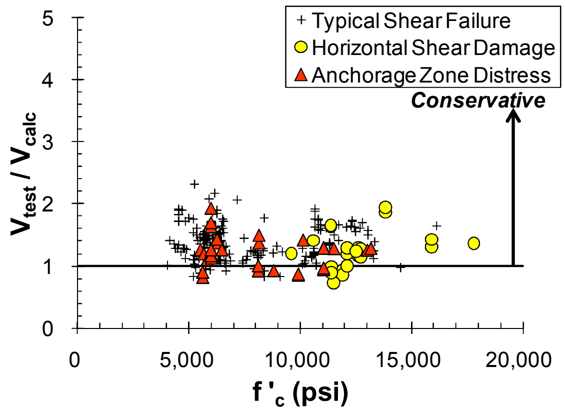


(e) Effective prestress in concrete

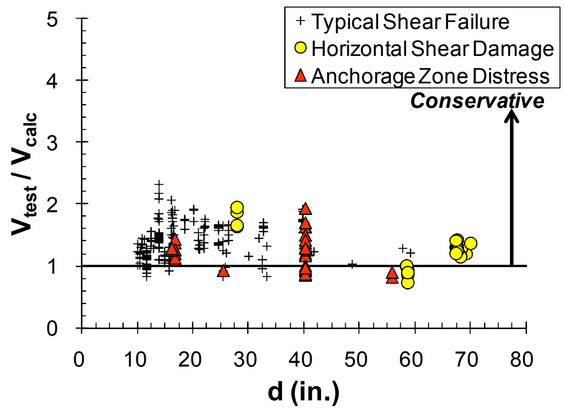


(f) Effective prestress in prestressing steel

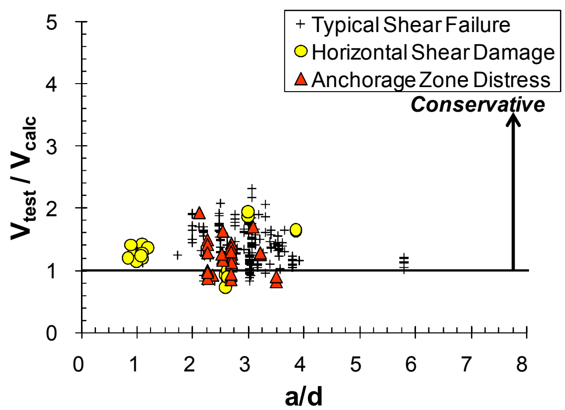
Figure 6.7 Shear strength ratio, experimental variable, and shear failure mode (ACI 318-08 Simplified Method)



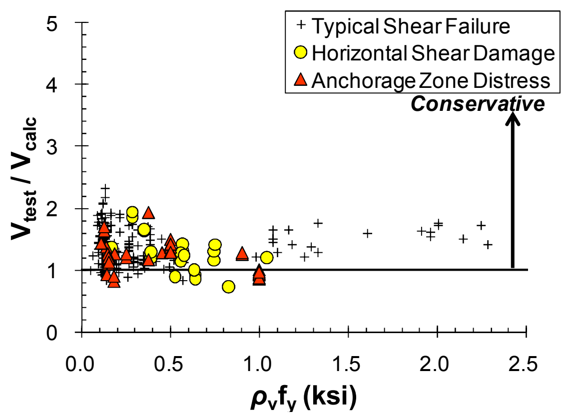
(a) Concrete strength



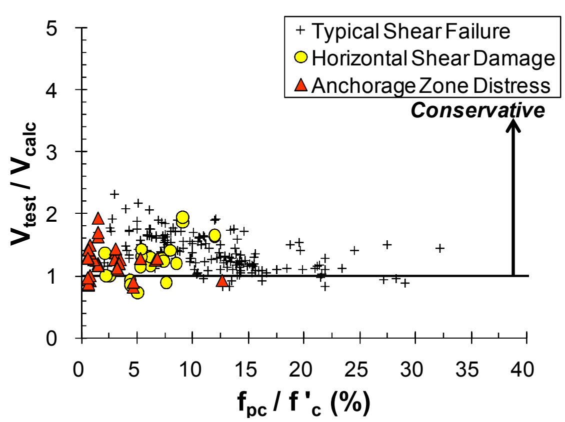
(b) Effective depth



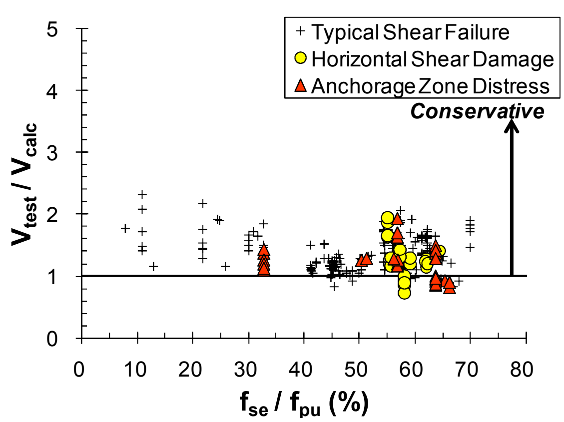
(c) Shear span to depth ratio



(d) Shear reinforcement index

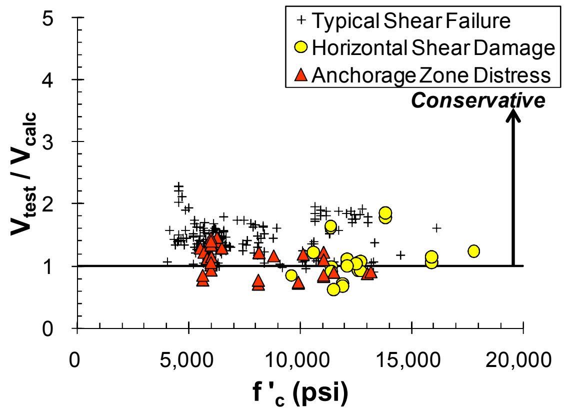


(e) Effective prestress in concrete

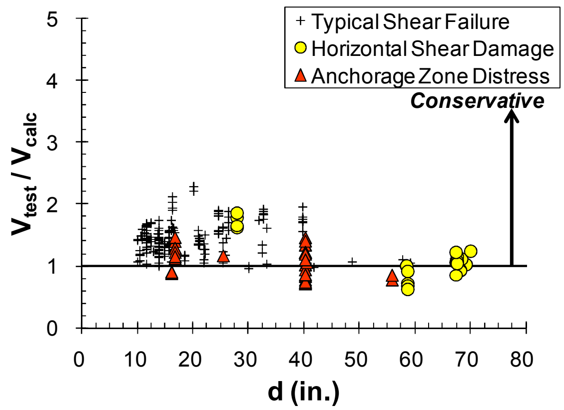


(f) Effective prestress in prestressing steel

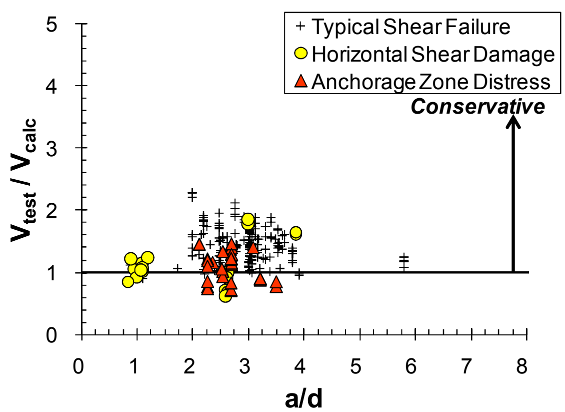
Figure 6.8 Shear strength ratio, experimental variable, and shear failure mode (ACI 318-08 Detailed Method)



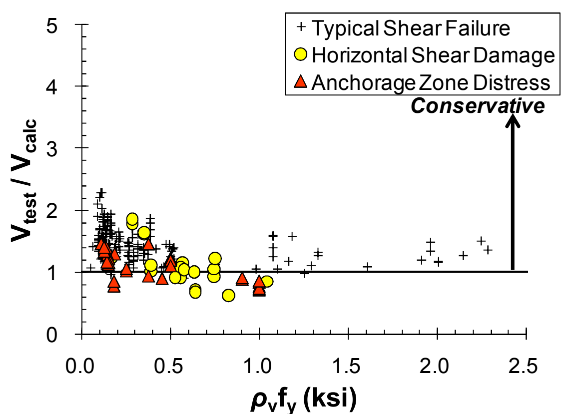
(a) Concrete strength



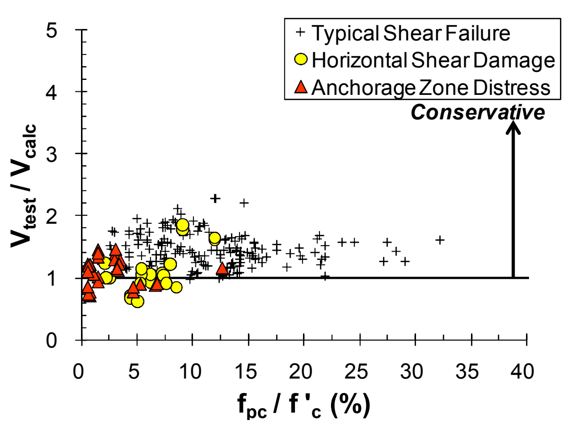
(b) Effective depth



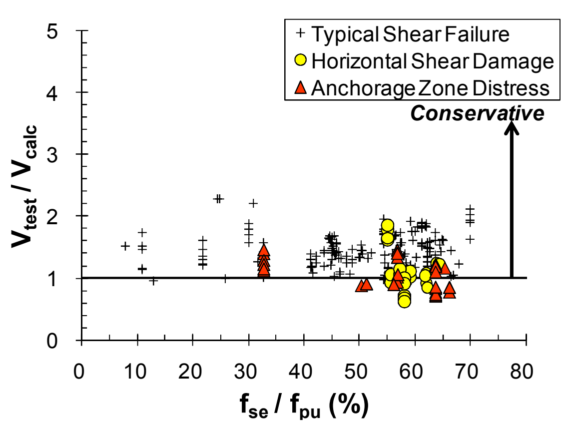
(c) Shear span to depth ratio



(d) Shear reinforcement index

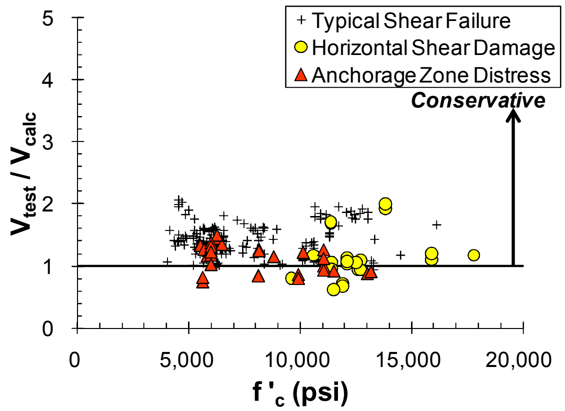


(e) Effective prestress in concrete

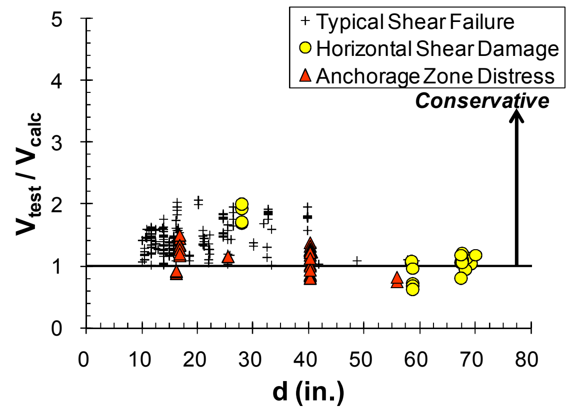


(f) Effective prestress in prestressing steel

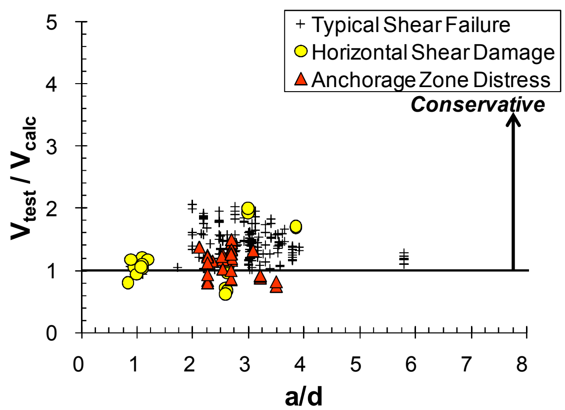
Figure 6.9 Shear strength ratio, experimental variable, and shear failure mode (AASHTO LRFD 2007 General Procedure)



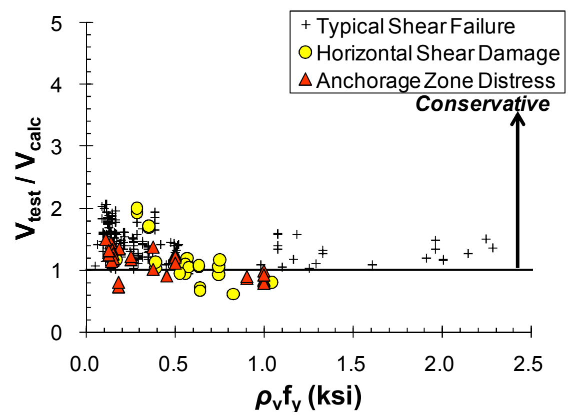
(a) Concrete strength



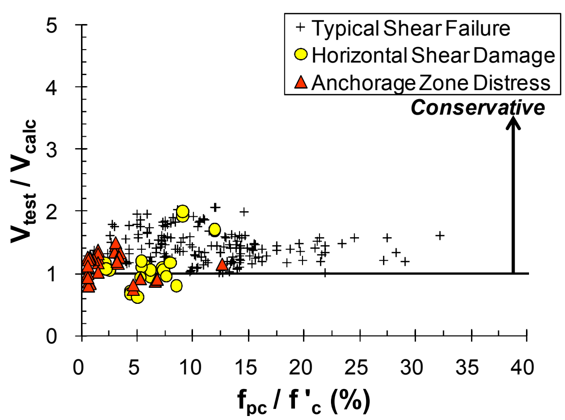
(b) Effective depth



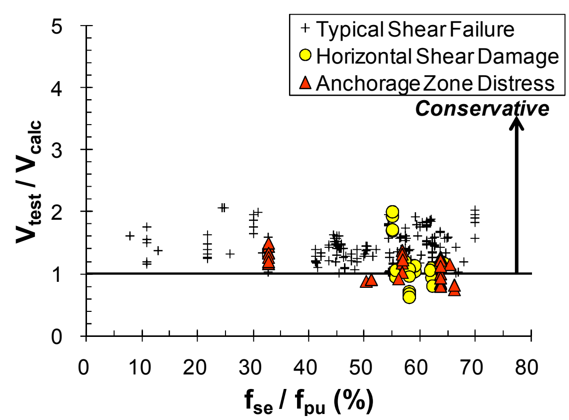
(c) Shear span to depth ratio



(d) Shear reinforcement index

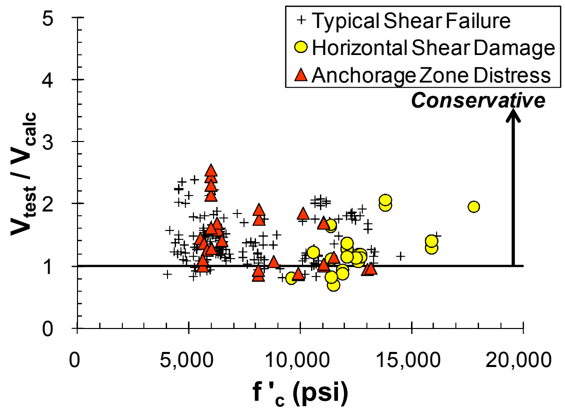


(e) Effective prestress in concrete

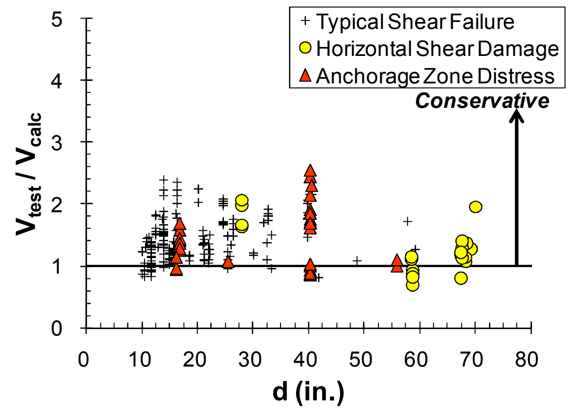


(f) Effective prestress in prestressing steel

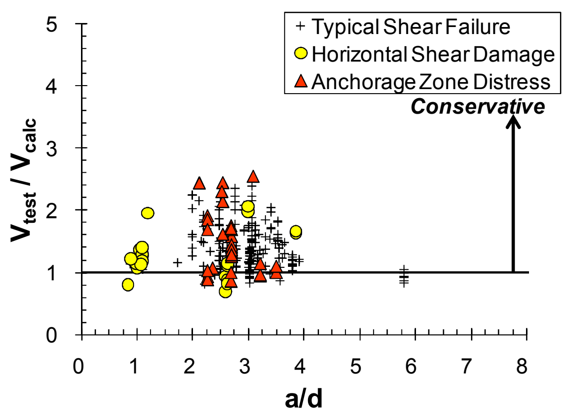
Figure 6.10 Shear strength ratio, experimental variable, and shear failure mode (AASHTO LRFD 2010 General Procedure)



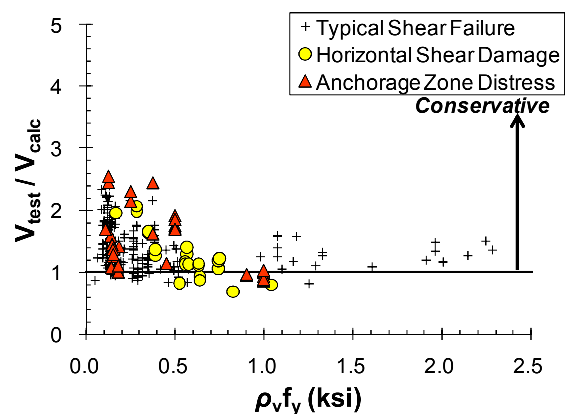
(a) Concrete strength



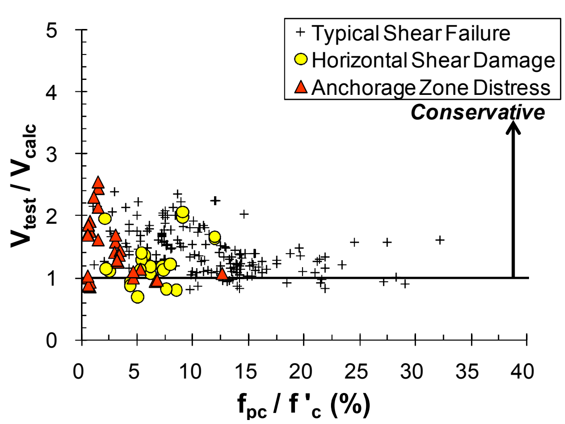
(b) Effective depth



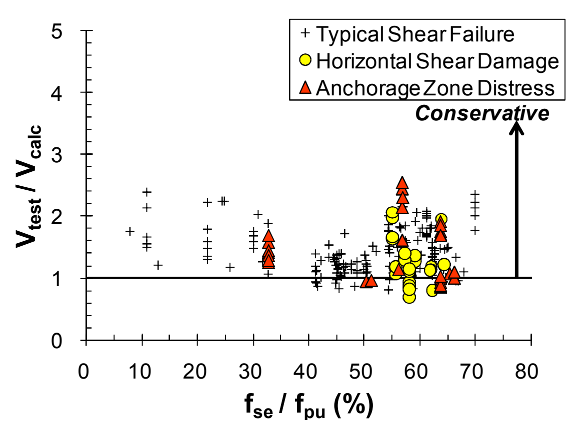
(c) Shear span to depth ratio



(d) Shear reinforcement index

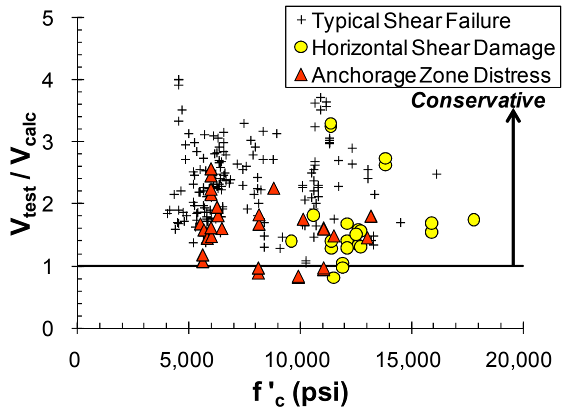


(e) Effective prestress in concrete

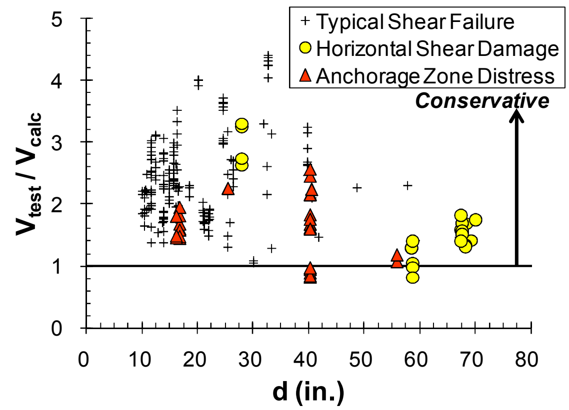


(f) Effective prestress in prestressing steel

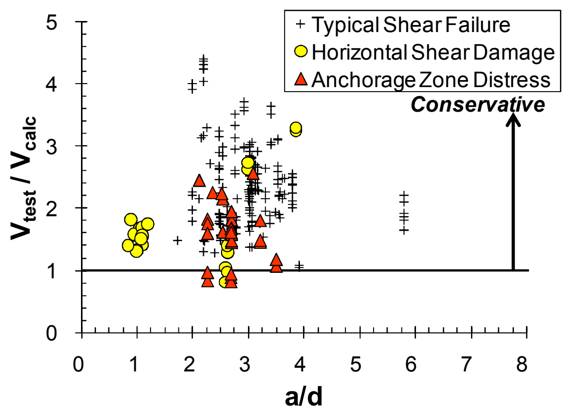
Figure 6.11 Shear strength ratio, experimental variable, and shear failure mode (AASHTO LRFD 2010 Simplified Procedure)



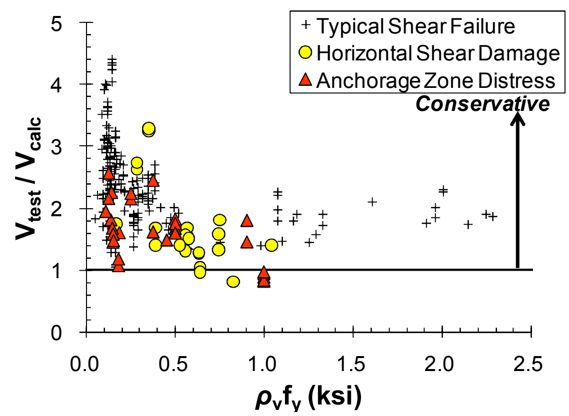
(a) Concrete strength



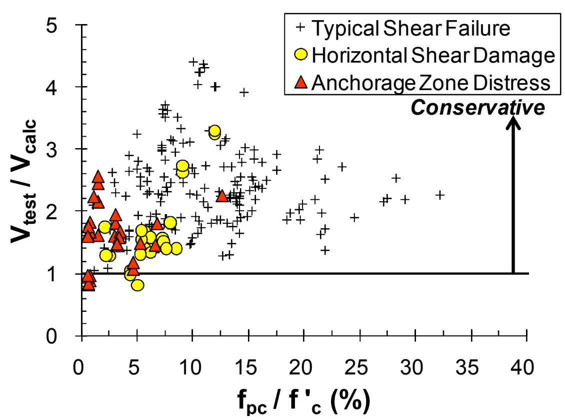
(b) Effective depth



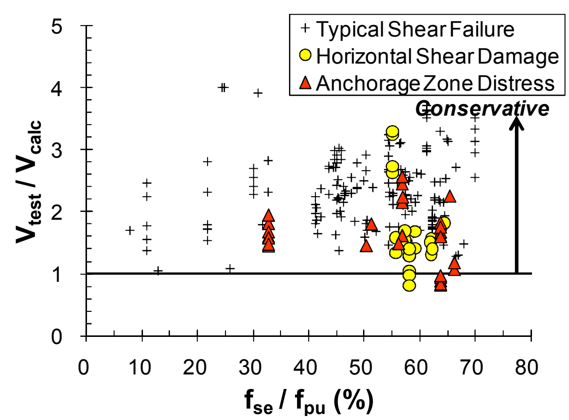
(c) Shear span to depth ratio



(d) Shear reinforcement index

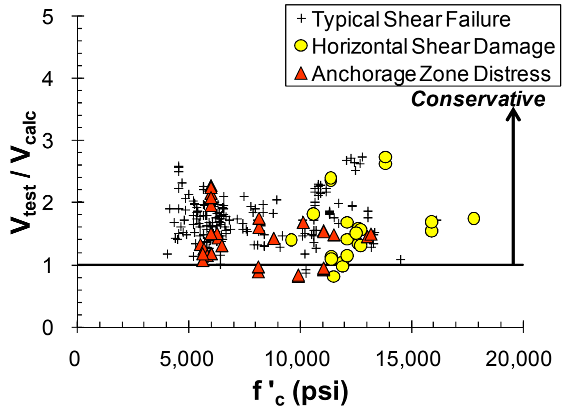


(e) Effective prestress in concrete

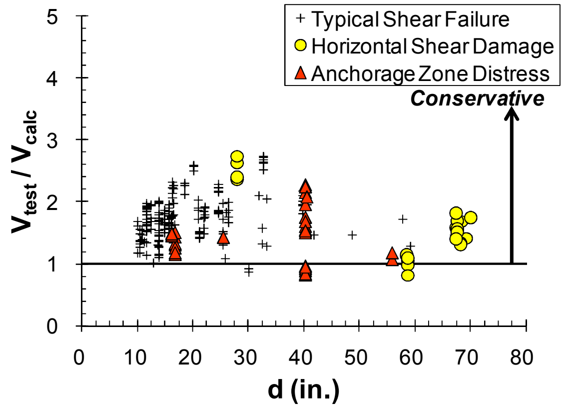


(f) Effective prestress in prestressing steel

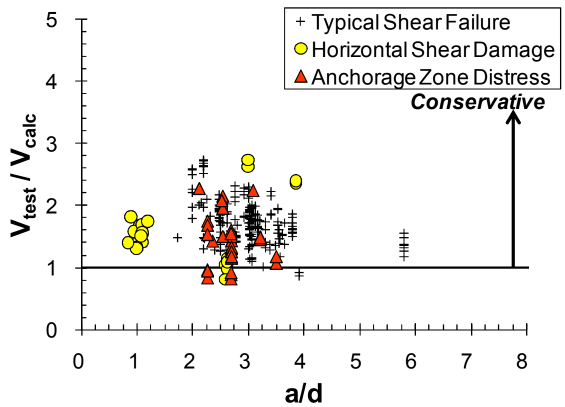
Figure 6.12 Shear strength ratio, experimental variable, and shear failure mode (AASHTO LRFD 2010 Segmental Procedure with limit on K factor)



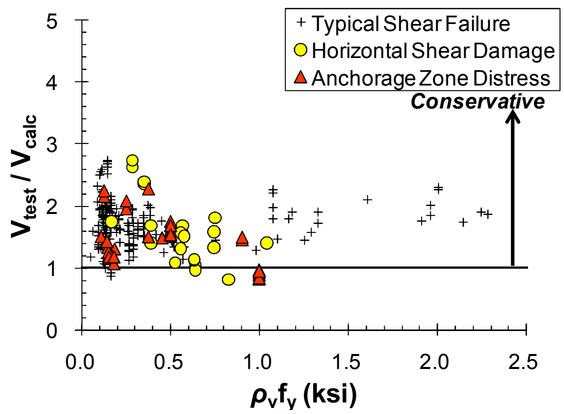
(a) Concrete strength



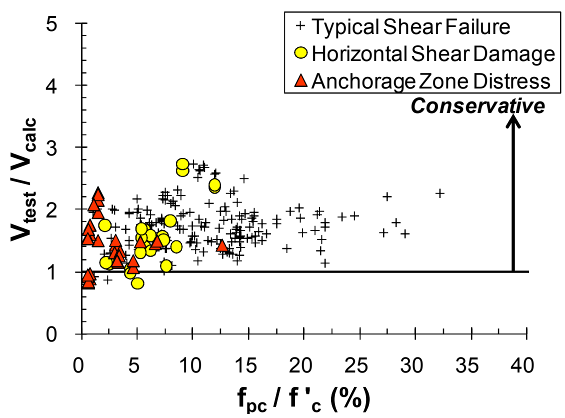
(b) Effective depth



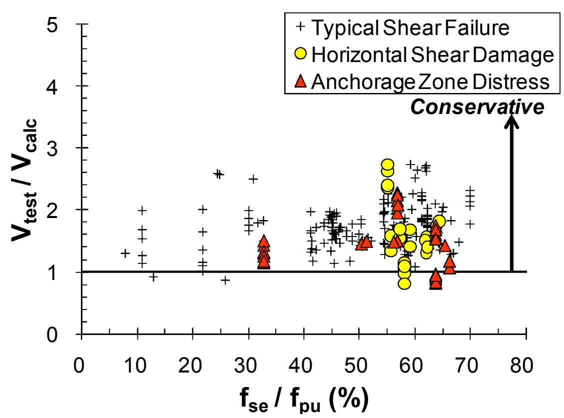
(c) Shear span to depth ratio



(d) Shear reinforcement index

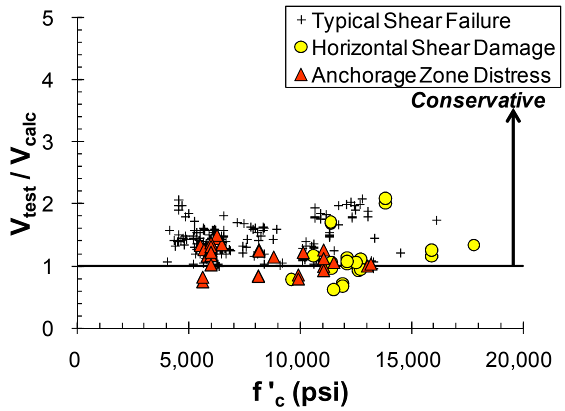


(e) Effective prestress in concrete

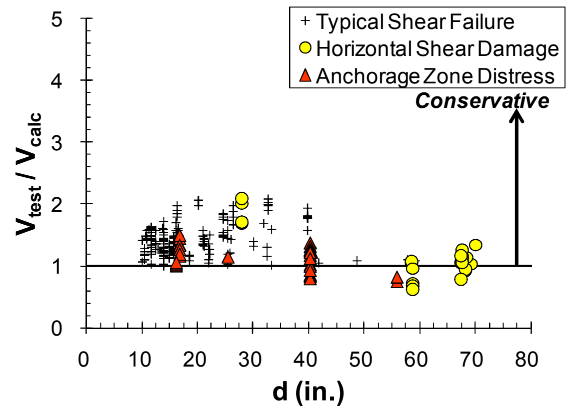


(f) Effective prestress in prestressing steel

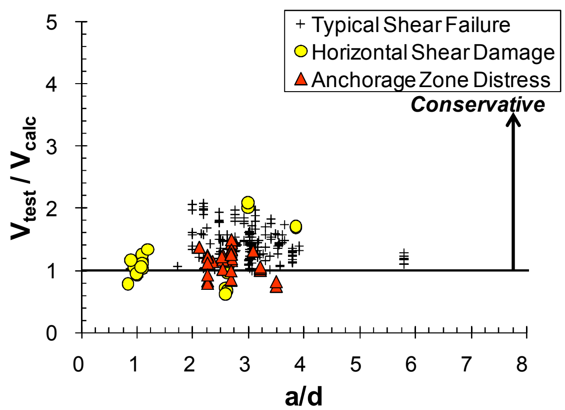
Figure 6.13 Shear strength ratio, experimental variable, and shear failure mode (AASHTO LRFD 2010 Segmental Procedure without limit on K factor)



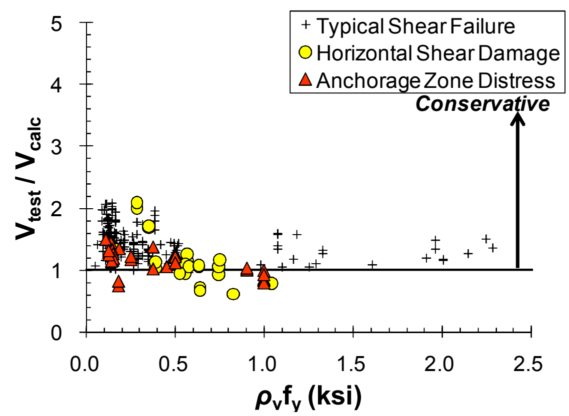
(a) Concrete strength



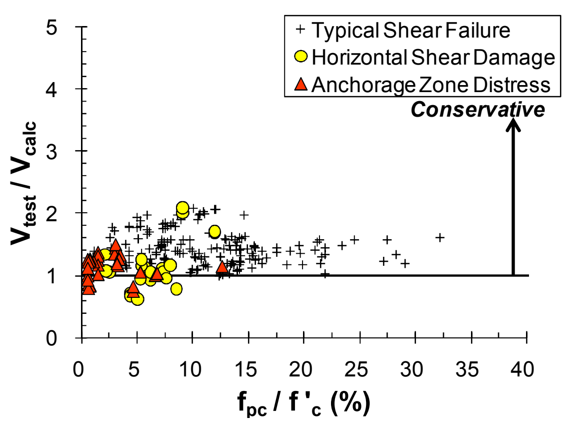
(b) Effective depth



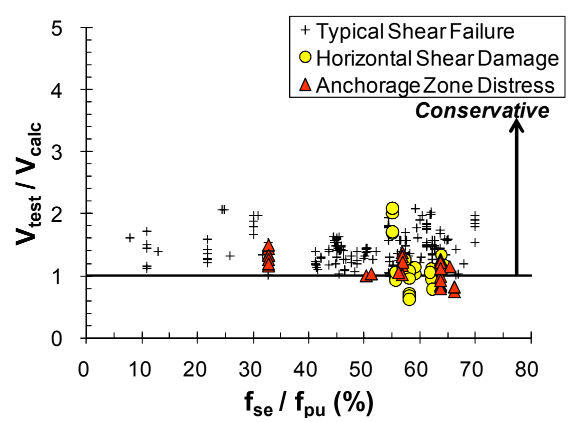
(c) Shear span to depth ratio



(d) Shear reinforcement index

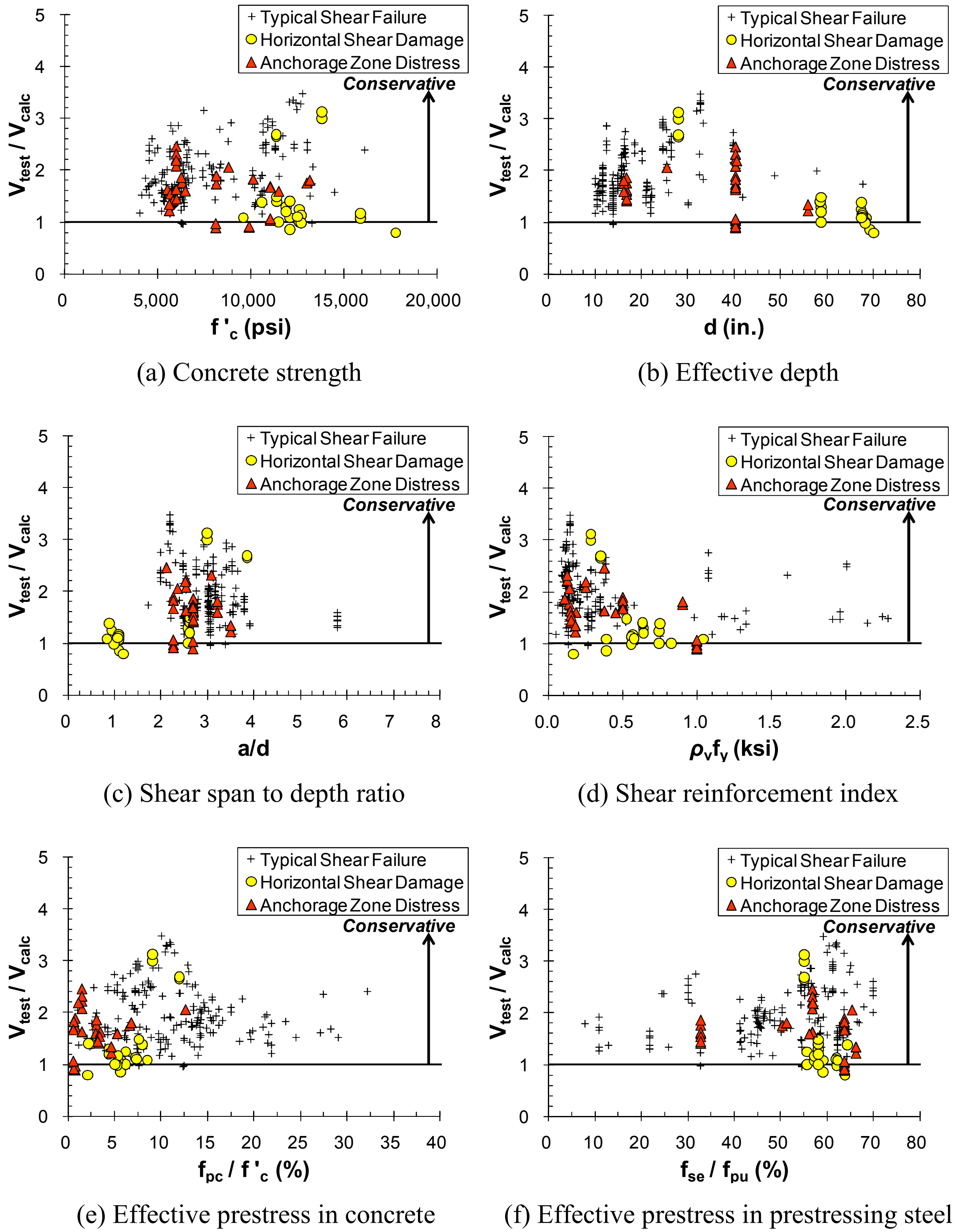


(e) Effective prestress in concrete



(f) Effective prestress in prestressing steel

Figure 6.14 Shear strength ratio, experimental variable, and shear failure mode (CSA A23.3-04)



(a) Concrete strength

(b) Effective depth

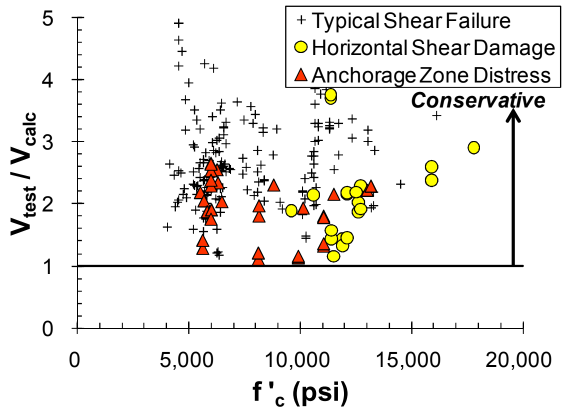
(c) Shear span to depth ratio

(d) Shear reinforcement index

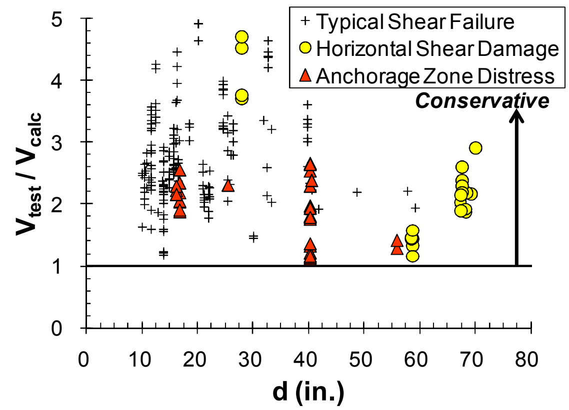
(e) Effective prestress in concrete

(f) Effective prestress in prestressing steel

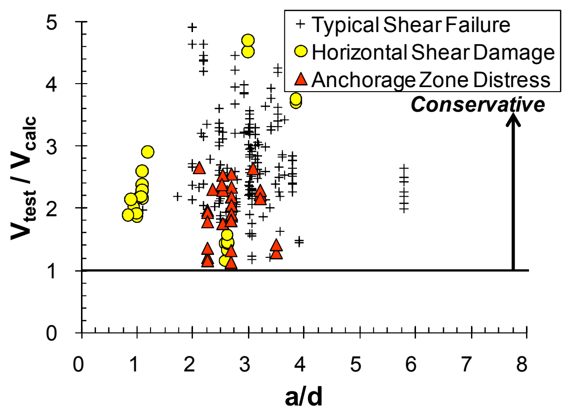
Figure 6.15 Shear strength ratio, experimental variable, and shear failure mode (JSCE 2007)



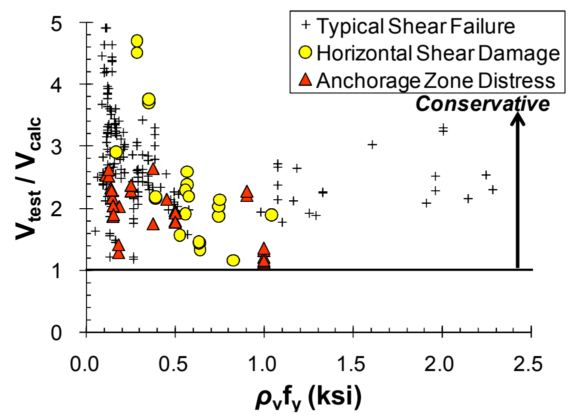
(a) Concrete strength



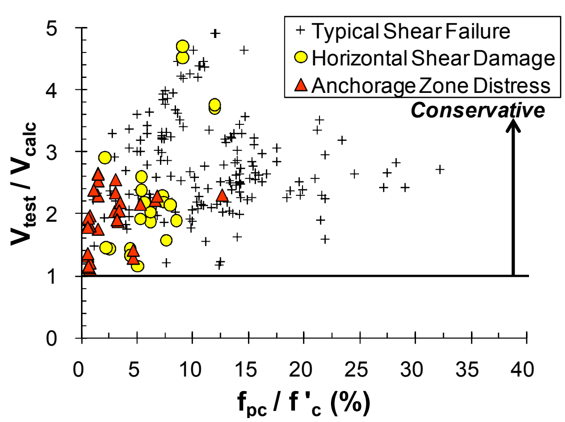
(b) Effective depth



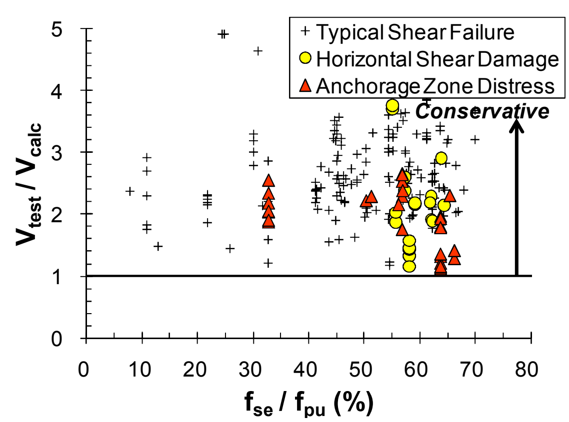
(c) Shear span to depth ratio



(d) Shear reinforcement index

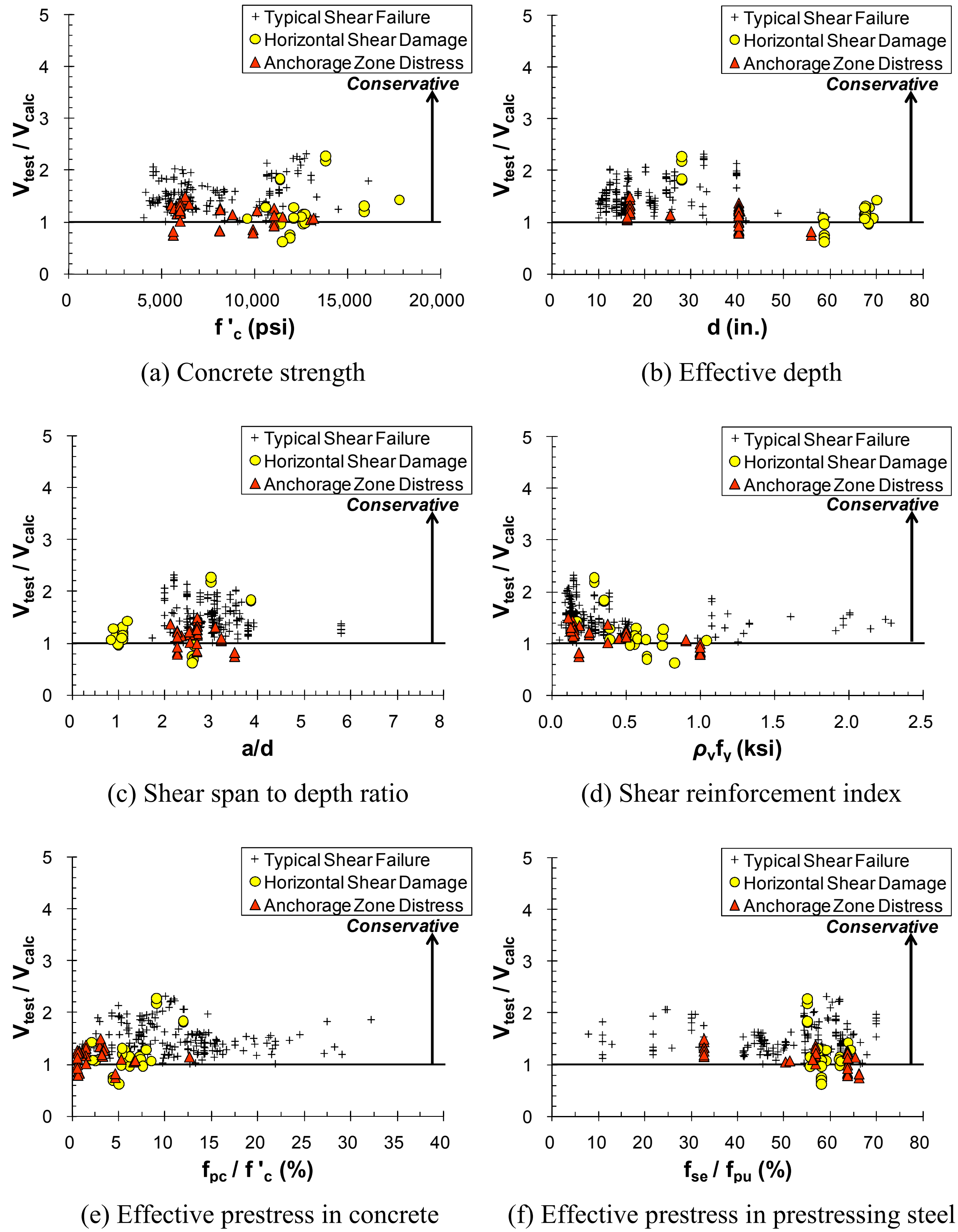


(e) Effective prestress in concrete



(f) Effective prestress in prestressing steel

Figure 6.16 Shear strength ratio, experimental variable, and shear failure mode (JRA 2002)



(a) Concrete strength

(b) Effective depth

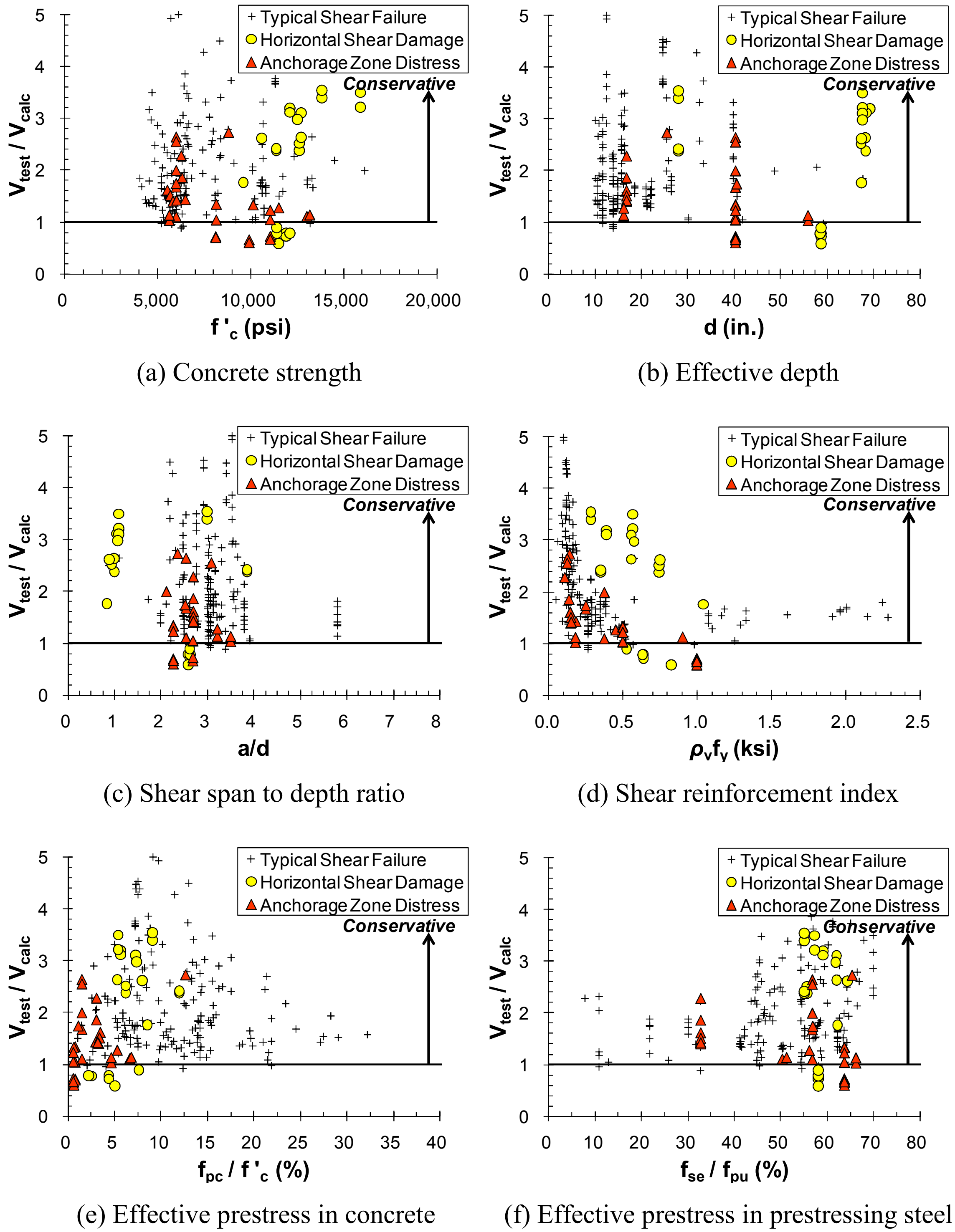
(c) Shear span to depth ratio

(d) Shear reinforcement index

(e) Effective prestress in concrete

(f) Effective prestress in prestressing steel

Figure 6.17 Shear strength ratio, experimental variable, and shear failure mode (fib Model Code 2010)



(a) Concrete strength

(b) Effective depth

(c) Shear span to depth ratio

(d) Shear reinforcement index

(e) Effective prestress in concrete

(f) Effective prestress in prestressing steel

Figure 6.18 Shear strength ratio, experimental variable, and shear failure mode (EN 1992)

6.3 LIMIT IMPOSED ON CALCULATED MAXIMUM SHEAR STRENGTH

In this section, the influence of design provisions for the maximum shear strength on the accuracy and conservativeness of shear design equations is investigated by using the evaluation database-level I. This investigation is focused on four U.S. shear design equations: (1) ACI 318-08 detailed method; (2) AASHTO LRFD 2010 general procedure; (3) AASHTO LRFD 2010 simplified procedure; and (4) AASHTO LRFD 2010 segmental procedure without the limit on the K factor.

6.3.1 Design Provisions for Maximum Shear Strength

The four shear design equations included in ACI 318-08 and AASHTO LRFD 2010 adopt different upper limits on the maximum shear strength as shown in Table 6.4.

Table 6.4 Design provisions for maximum shear strength

Design code	***Limit on maximum shear strength***	***Unit***
ACI 318-08 Detailed Method	$V_c + V_s \leq V_c + 8\sqrt{f'_c} b_w d$ The V_c term is the lesser of V_{ci} and V_{cw}	lbs, psi
AASHTO LRFD 2010 General Procedure	$V_c + V_s \leq 0.25 f'_c b_v d_v$	kips, ksi
AASHTO LRFD 2010 Simplified Procedure	$V_c + V_s \leq 0.25 f'_c b_v d_v$ The V_c term is the lesser of V_{ci} and V_{cw}	kips, ksi
AASHTO LRFD 2010 Segmental Procedure	$V_c + V_s \leq 0.379\sqrt{f'_c} b_v d_v$	kips, ksi
AASHTO LRFD 2010	$0.18 f'_c$ Upper limit on shear stress in beam-type element that is not built integrally with support	kips, ksi

The ACI 318-08 detailed method sets the upper limit on the V_s term as $8\sqrt{f'_c}b_w d$, and defines the V_c term as the lesser of V_{ci} and V_{cw}. In AASHTO LRFD 2010, the general and simplified procedures provide the upper limit on the shear strength as $0.25f'_c\, b_v d_v$, and the segmental procedure limits the maximum shear strength up to $0.379\sqrt{f'_c}b_v d_v$. Although all of these limits share the same purpose, i.e., to avoid web crushing failure prior to yielding of shear reinforcement, there are substantial differences in the upper limits on the maximum shear strength among these four U.S. shear design equations.

Additionally, in AASHTO LRFD 2010, the shear stress in the beam-type element that is not built integrally with the support is limited to $0.18f'_c$ in order to avoid a local diagonal compression failure and horizontal shear failure. In the case where the shear stress exceeds $0.18f'_c$, the end region of the element is recommended to be designed by using the strut-and-tie model.

6.3.2 Evaluation of Design Provisions for Maximum Shear Strength

In Table 6.5, statistical parameters of the shear strength ratio of both calculations that were controlled by the upper limit and those that were not controlled by the upper limit are presented for the four shear design equations. The relationship between the shear strength ratio and the upper limit on the maximum shear strength in the four shear design provisions is shown in Figures 6.19 through 6.22. The characteristics of the upper limit of each shear design equation are summarized as follows.

6.3.2.1 ACI 318-08 detailed method

The relationship between the upper limit on the shear strength and the shear strength ratio of the ACI 318-08 detailed method is shown in Figure 6.19. Data points in this figure are divided into three groups: (1) tests that are controlled by V_{ci}; (2) tests that are controlled by V_{cw}; (3) tests that are controlled by both V_{cw} and the upper limit on the

V_s term. Note that there is no test that is controlled by both V_{ci} and the upper limit on the V_s term.

In Figure 6.19 and Table 6.5, most of the unconservative estimations, i.e., tests that had shear strength ratios of lower than 1.0, were controlled by V_{cw} or by both V_{cw} and the upper limit on the V_s term. These unconservative estimations show no bias on the key experimental variables as shown in all graphs included in Figure 6.19. As presented in the previous sections, the ACI 318-08 detailed method provides a higher percentage of unconservative cases than the MCFT-based design expressions. It is interesting to note that the cause of the high percentage of unconservative strength estimations is likely to be rooted in the V_{cw} calculation.

As can be observed in Figure 6.19(d), the calculations performed for specimens with high shear reinforcement indices were controlled by both V_{cw} and the upper limit on the V_s term. Most of these shear strengths are conservatively estimated except for six tests that showed the signs of the anchorage zone distress as presented in Figure 6.8(d).

6.3.2.2 AASHTO LRFD 2010 general procedure

The relationship between the upper limit on the maximum shear strength and the shear strength ratio calculated by using the AASHTO LRFD 2010 general procedure is presented in Figure 6.20. Similar to the ACI 318-08 detailed method, test specimens with high shear reinforcement indices were controlled by the upper limit on the maximum shear strength as shown in Figure 6.20(d). The upper limit of the AASHTO LRFD 2010 general procedure provides no unconservative estimation for 18 tests that were controlled by this upper limit. As shown in the previous section, most of the unconservative strength estimations obtained by using the MCFT-based design expressions consist of tests that exhibited signs of horizontal shear damage and/or anchorage zone distress. This implies that these tests were not controlled by the upper limit on the maximum shear strength. It is obvious to think that the current upper limit on the maximum shear strength in the

AASHTO LRFD 2010 is not applicable to cases in which the horizontal shear damage and anchorage zone distress are likely to occur.

As shown in Table 6.4, AASHTO LRFD 2010 recommends to design the end region of simply supported beams with the strut-and-tie model in the case where the shear stress exceeds $0.18f'_c$. In order to evaluate the applicability of this upper limit on the shear stress to test specimens that showed signs of the horizontal shear damage, the shear strength ratio was also calculated by using an upper limit of $0.18f'_c b_v d_v$ instead of $0.25f'_c b_v d_v$. The relationship between the shear failure mode and the shear strength ratio is shown in Figure 6.23, and the relationship between the upper limit on the maximum shear strength and the shear strength ratio is presented in Figure 6.24. Note that the upper limit on the maximum shear strength was taken as $0.18f'_c b_v d_v$ in these two figures.

On the basis of Figure 6.24 (d), the reduced upper limit controlled the shear strength of several specimens with low shear reinforcement indices since these specimens were fabricated with concrete that had relatively low compressive strength as shown in Figure 6.24(a). The reduced upper limit controlled the shear strength of these specimens. Additionally, the reduced upper limit resulted in more conservative strength estimations for specimens with high shear reinforcement indices.

Moreover, all unconservative strength estimations for specimens that were reported to have failed in shear with signs of horizontal shear damage and/or anchorage zone distress were not controlled by the reduced upper limit. In Figure 6.10(d), there were seven unconservative strength estimations for specimens that showed horizontal shear damage. An examination of Figure 6.23(d) shows that only one of these seven unconservative strength estimations was controlled by the reduced upper limit and turned to be conservative. Five of these seven unconservative strength estimations were obtained from tests conducted on full-scale U-beams at the University of Texas at Austin (Hovell, et al. 2010). On the basis of these evidences, it is not clear that the upper limit of $0.18f'_c$ is applicable to prevent unconservative strength estimations for specimens that failed in shear with signs of horizontal shear damage. Once again, the mechanics of horizontal shear failure is beyond the scope of this thesis. More comprehensive research is required to use the MCFT-based design expressions without any concern about unconservative strength estimations for members that are likely to be damaged due to horizontal shear.

6.3.2.3 AASHTO LRFD 2010 simplified procedure

The relationship between the upper limit on the maximum shear strength and the shear strength ratio calculated by using the AASHTO LRFD 2010 simplified procedure is shown in Figure 6.21. On the basis of Table 6.5, over 80% of the 223 strength estimations were controlled by the V_{cw} calculation, and these estimations accounted for a large part of unconservative estimations.

Unlike the AASHTO LRFD 2010 general procedure in Figure 6.20, the simplified procedure produced three unconservative estimations that were controlled by the upper limit on the maximum shear strength. The reason of this discrepancy is that there were disagreements on the tests that were controlled by the upper limit between the two procedures. Even though these two procedures adopt the same upper limit, there are differences in the V_c and V_s terms. In Table 6.5, the upper limit controlled 18 tests for the AASHTO LRFD 2010 general procedure, and the shear strength of all these tests were estimated conservatively. Meanwhile, the upper limit controlled 22 tests for the AASHTO LRFD 2010 simplified procedure, and 3 of the 22 tests were estimated unconservatively.

6.3.2.4 AASHTO LRFD 2010 segmental procedure without limit on K factor

The relationship between the upper limit on the maximum shear strength and the shear strength ratio calculated by using the AASHTO LRFD 2010 segmental procedure is shown in Figure 6.22. The limit on the K factor was not used in this calculation. Similar to aforementioned three equations, the shear strength calculated for specimens with high shear reinforcement indices are controlled by the upper limit on the maximum shear strength. On the basis of Table 6.5, the upper limit in the AASHTO LRFD 2010 segmental procedure produced unconservative estimations for four tests. These four test specimens exhibited signs of anchorage zone distress as shown in Figure 6.13(d).

In Table 6.5, the mean of the shear strength ratio for tests that were controlled by the upper limit in the AASHTO LRFD 2010 segmental procedure is higher than those of other three upper limits. This implies that the upper limit of the AASHTO LRFD 2010 segmental procedure is more conservative than other upper limits.

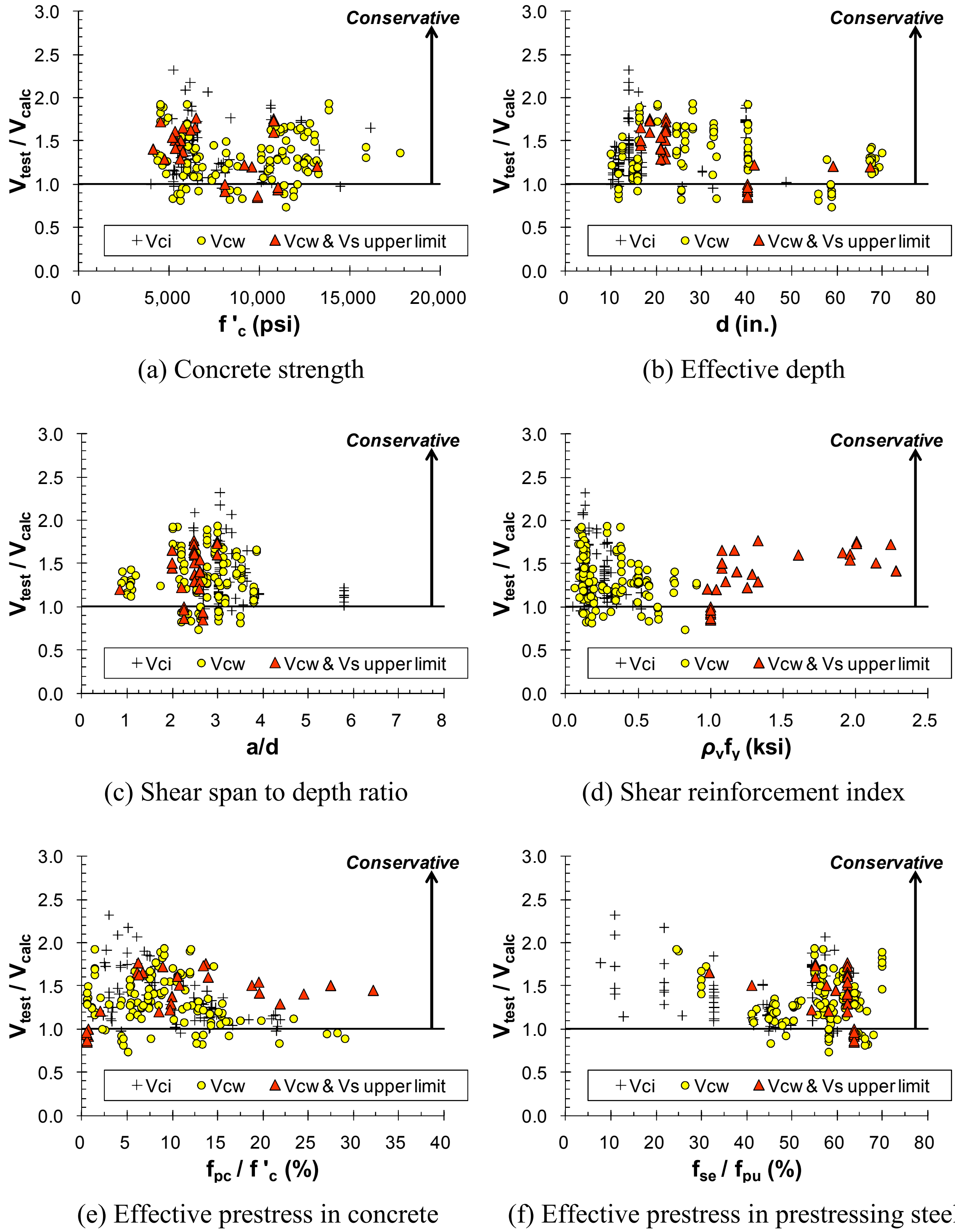
6.3.2.5 Summary of evaluation of design provisions for maximum shear strength

In the light of the database analysis performed on the code provisions for the maximum shear strength, those four design provisions for the maximum shear strength were found to produce different levels of variations in measured versus calculated shear strength. A discussion on the limit imposed on the maximum shear strength is meaningful if the interplay between the V_c and V_s terms are fully understood because the intent and calculation of the V_c and V_s terms are substantially different depending on the design codes.

Additionally, all design provisions that were discussed in this section contributed to provide conservative strength estimations especially for specimens with high shear reinforcement indices. On the other hand, these code provisions for the maximum shear strength were not appropriate to avoid shear failure accompanied by signs of horizontal shear damage and/or anchorage zone distress. These types of damages should be prevented to provide accurate and conservative shear strength estimations with those shear design equations.

Table 6.5 Statistics of shear strength ratio and limit on maximum shear strength (N = 223 tests)

	ACI 318-08 Detailed Method			***AASHTO LRFD 2010***						
				General Procedure		***Simplified Procedure***			***Segmental Procedure w/o limit on K factor***	
	V_{ci}	V_{cw}	***V_{cw} and upper lmit on V_s***	***Not controlled by upper limit***	***Controlled by upper limit***	V_{ci}	V_{cw}	***Controlled by upper limit***	***Not controlled by upper limit***	***Controlled by upper limit***
Min	0.95	0.73	0.85	0.62	1.05	1.18	0.69	0.81	0.81	0.82
Max	2.32	1.94	1.76	2.07	1.60	2.39	2.55	1.60	2.73	2.30
Mean	1.39	1.32	1.37	1.36	1.31	1.59	1.42	1.25	1.67	1.71
COV	0.23	0.21	0.21	0.22	0.14	0.20	0.28	0.18	0.24	0.26
Unsafe Case	3	17	6	19	0	0	23	3	6	4
Unsafe %	3.8	14.5	21.4	9.3	0.0	0.0	12.8	13.6	3.0	16.0
Number of tests	78 (35.0%)	117 (52.5%)	28 (12.6%)	205 (91.9%)	18 (8.1%)	22 (9.9%)	179 (80.3%)	22 (9.9%)	198 (88.8%)	25 (11.2%)
Total	223			223		223			223	



(a) Concrete strength

(b) Effective depth

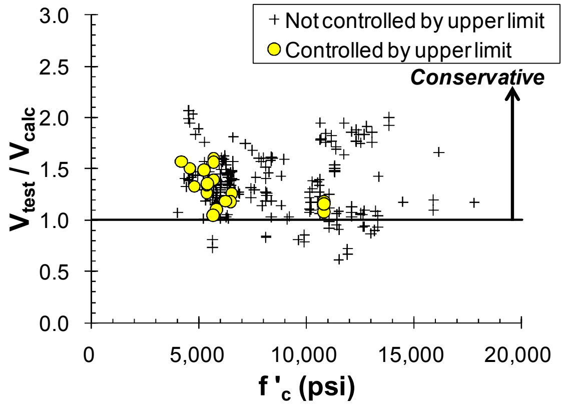
(c) Shear span to depth ratio

(d) Shear reinforcement index

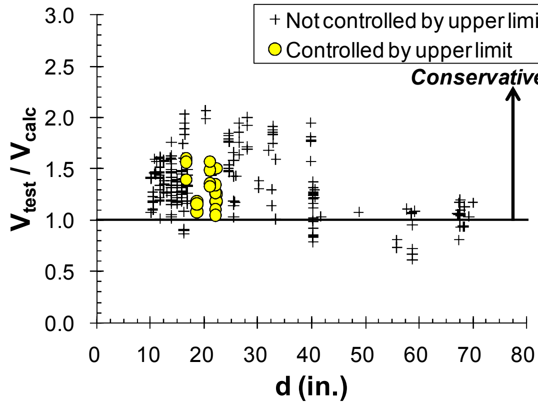
(e) Effective prestress in concrete

(f) Effective prestress in prestressing steel

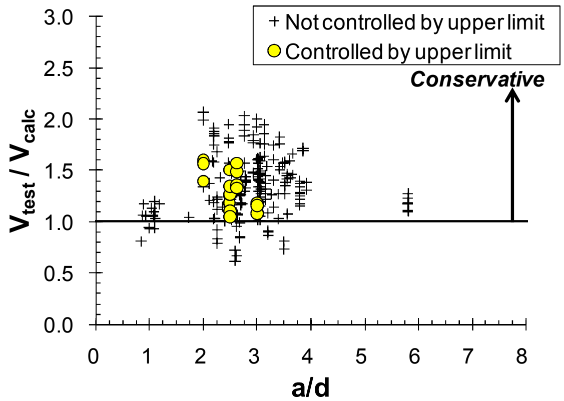
Figure 6.19 Upper limit on maximum shear strength and shear strength ratio (ACI 318-08 Detailed Method)



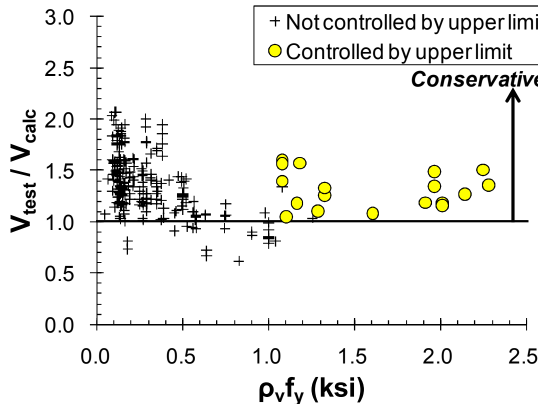
(a) Concrete strength



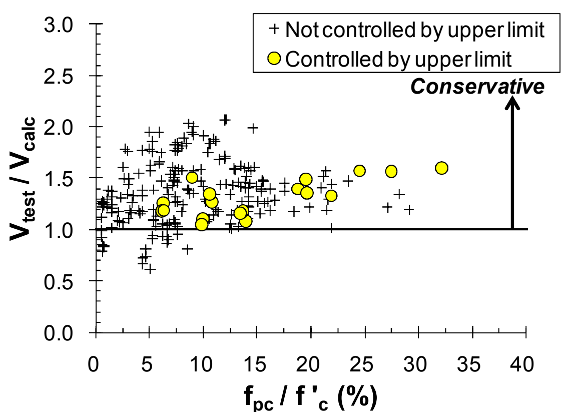
(b) Effective depth



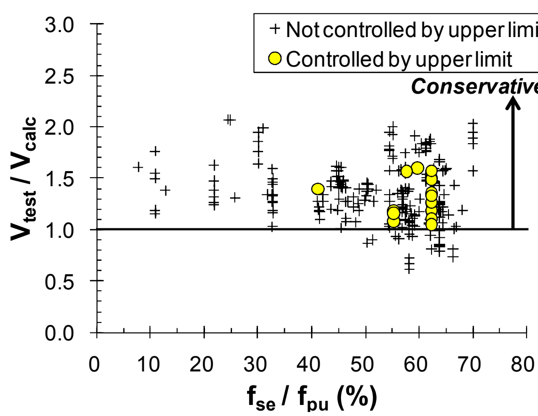
(c) Shear span to depth ratio



(d) Shear reinforcement index

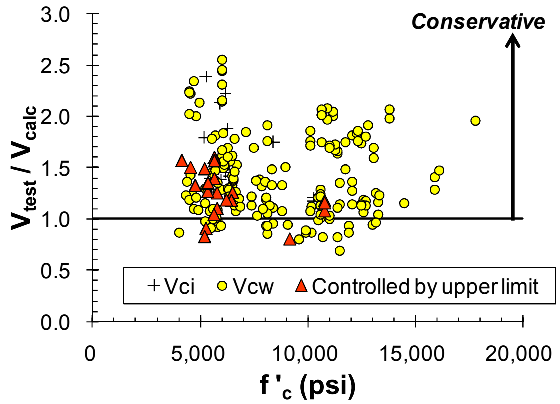


(e) Effective prestress in concrete

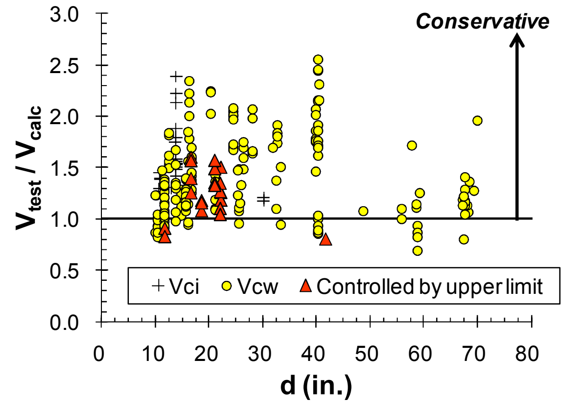


(f) Effective prestress in prestressing steel

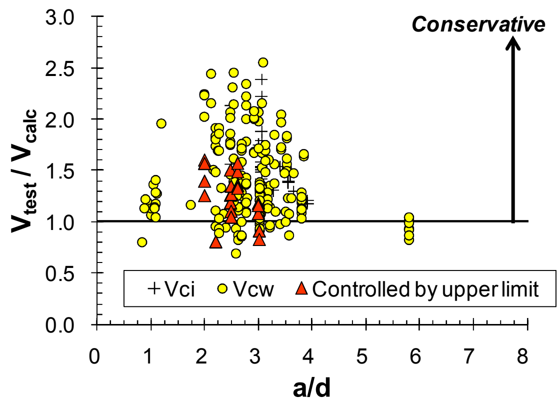
Figure 6.20 Upper limit on maximum shear strength and shear strength ratio (AASHTO LRFD 2010 General Procedure)



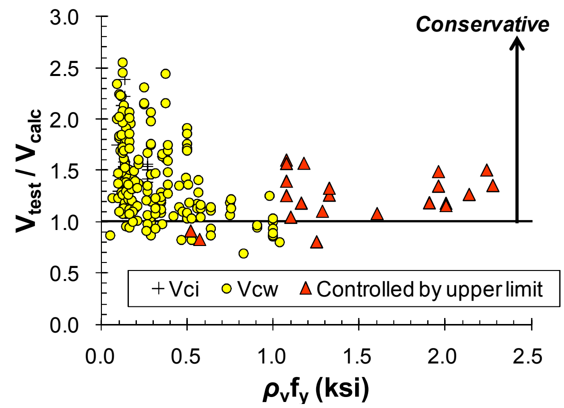
(a) Concrete strength



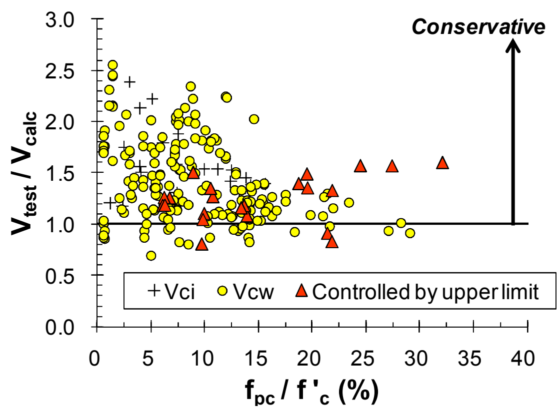
(b) Effective depth



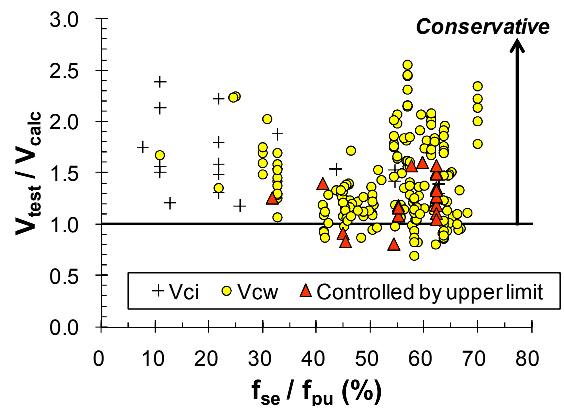
(c) Shear span to depth ratio



(d) Shear reinforcement index

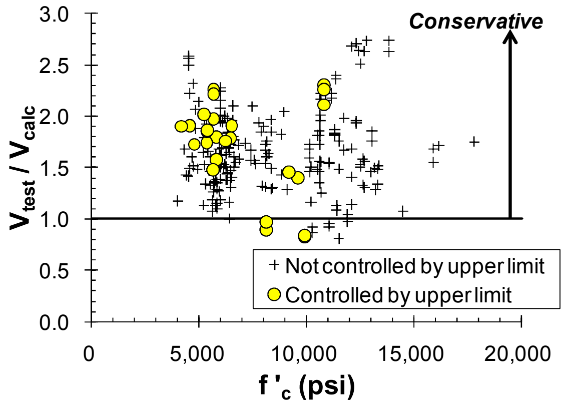


(e) Effective prestress in concrete

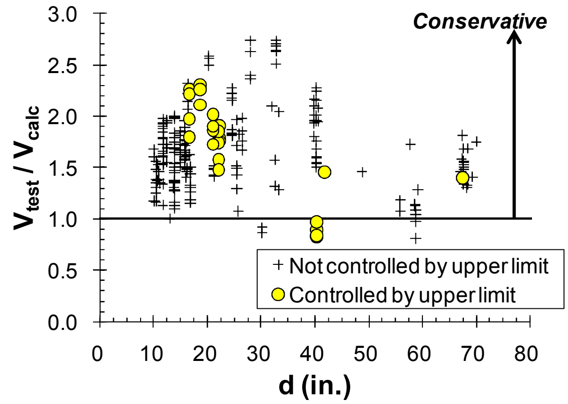


(f) Effective prestress in prestressing steel

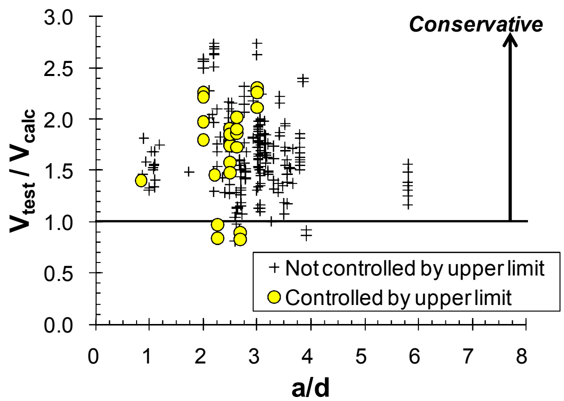
Figure 6.21 Upper limit on maximum shear strength and shear strength ratio (AASHTO LRFD 2010 Simplified Procedure)



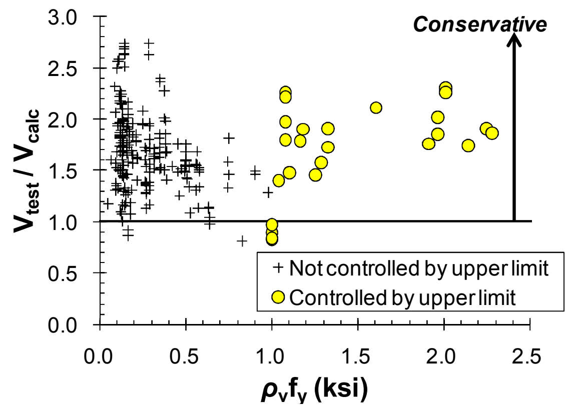
(a) Concrete strength



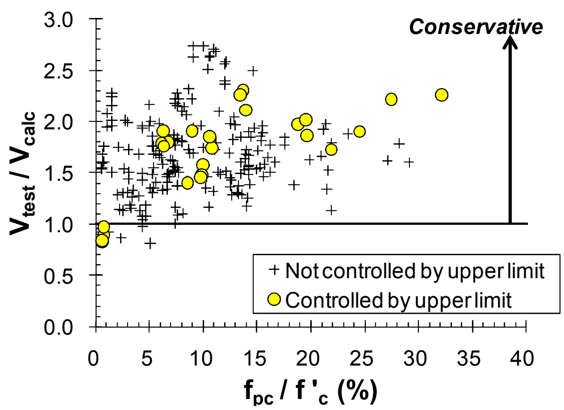
(b) Effective depth



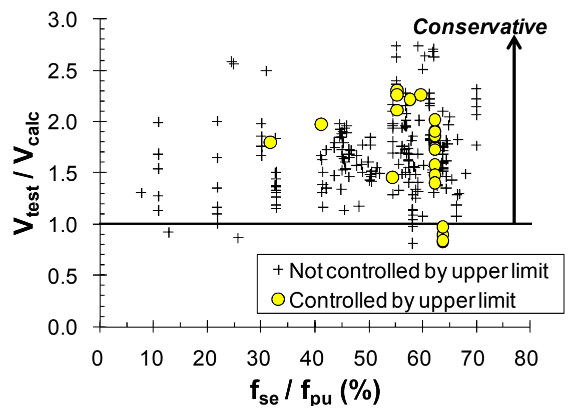
(c) Shear span to depth ratio



(d) Shear reinforcement index

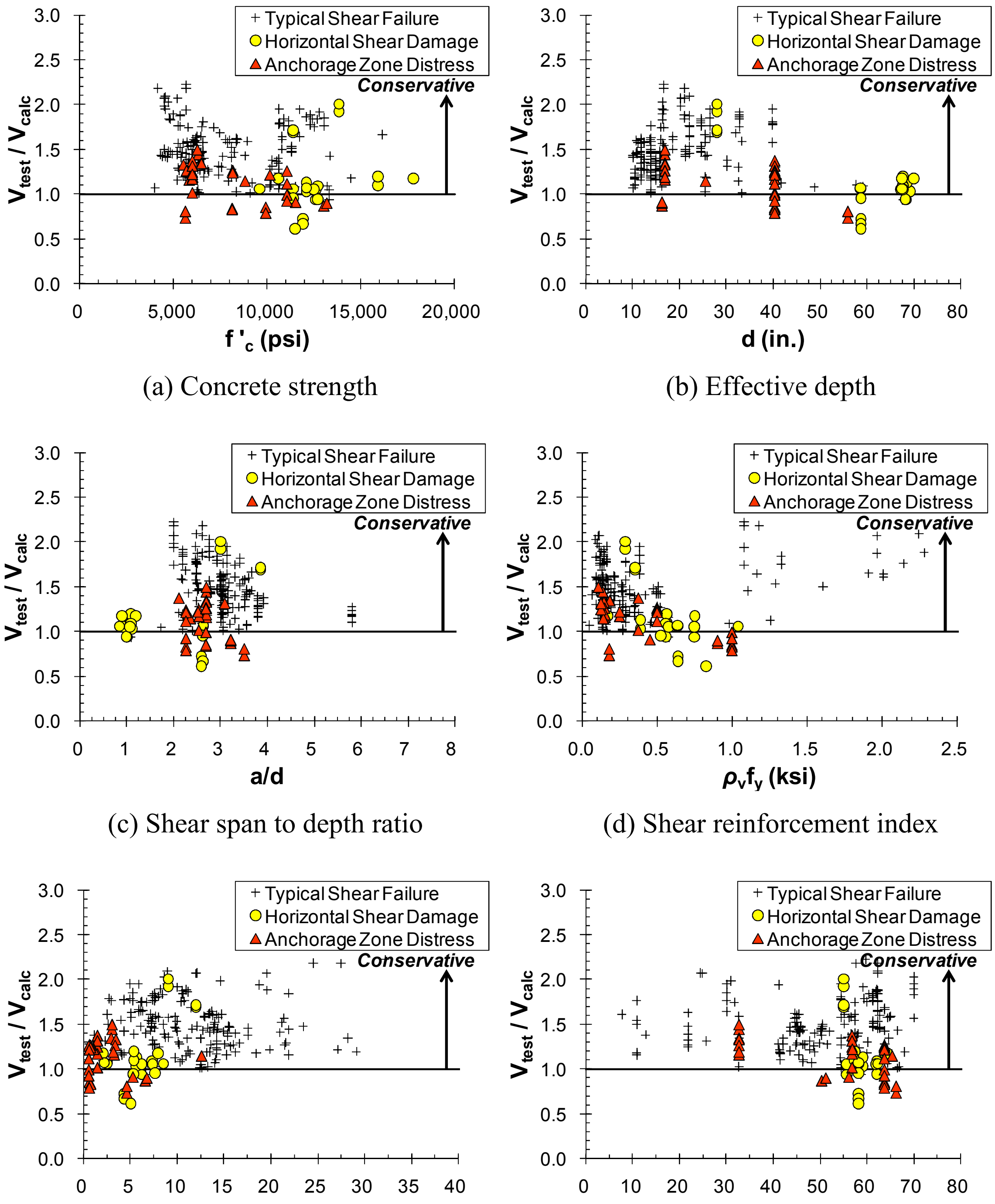


(e) Effective prestress in concrete



(f) Effective prestress in prestressing steel

Figure 6.22 Upper limit on maximum shear strength and shear strength ratio (AASHTO LRFD 2010 Segmental Procedure without limit on K factor)



(a) Concrete strength

(b) Effective depth

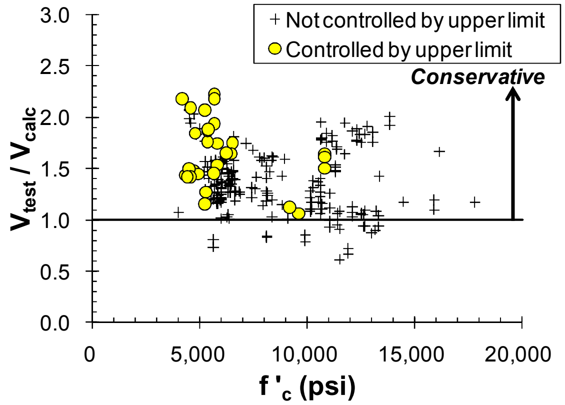
(c) Shear span to depth ratio

(d) Shear reinforcement index

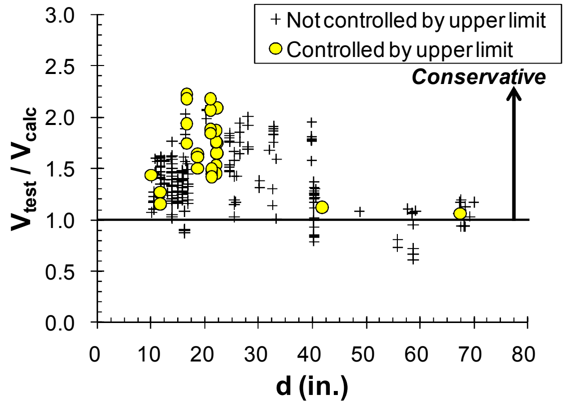
(e) Effective prestress in concrete

(f) Effective prestress in prestressing steel

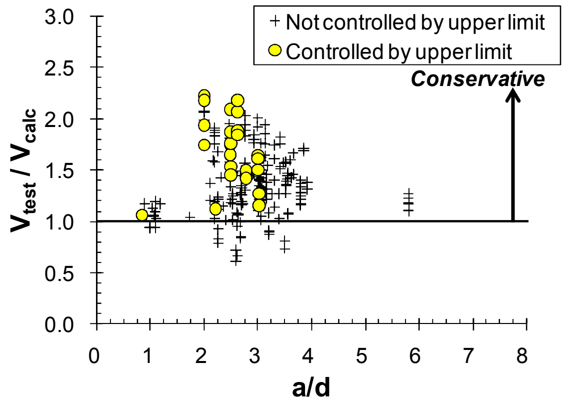
Figure 6.23 Shear strength ratio, experimental variable, and shear failure mode (AASHTO LRFD 2010 General Procedure with upper limit of 0.18f'c)



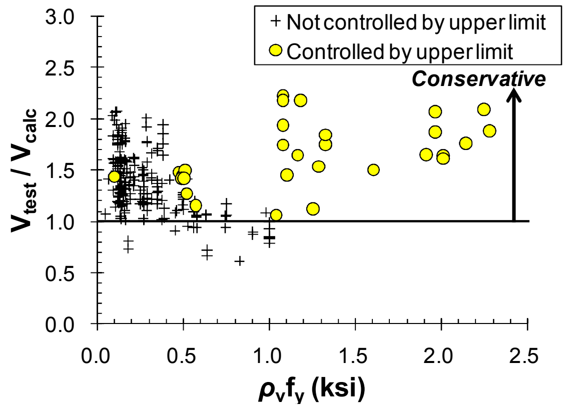
(a) Concrete strength



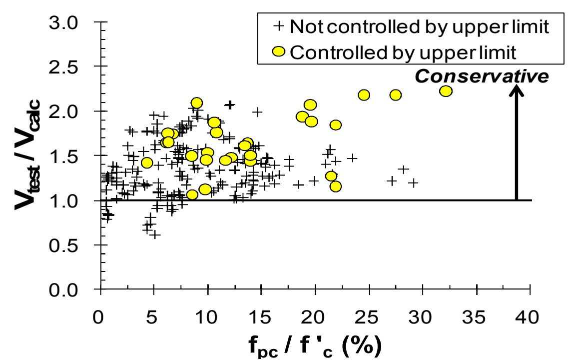
(b) Effective depth



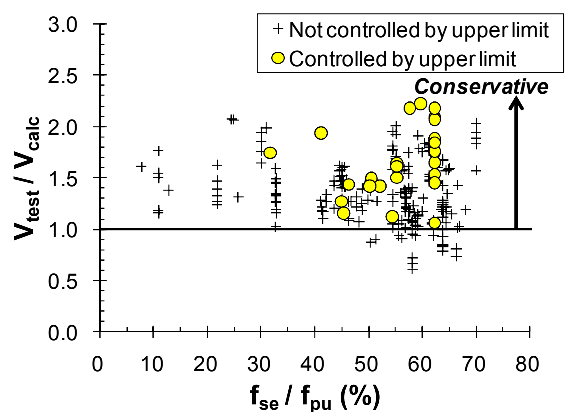
(c) Shear span to depth ratio



(d) Shear reinforcement index



(e) Effective prestress in concrete



(f) Effective prestress in prestressing steel

Figure 6.24 Upper limit on maximum shear strength and shear strength ratio (AASHTO LRFD 2010 General Procedure with upper limit of $0.18f'_c$)

CHAPTER 7

DATABASE ANALYSIS: MINIMUM SHEAR REINFORCEMENT

7.1 OVERVIEW

As mentioned in Chapter 2, there are substantial differences in various design provisions for the minimum shear reinforcement. The existing design provisions for the minimum shear reinforcement are analyzed with the UTPCSDB-2011 in this chapter. On the basis of results of this analysis, a new limit on the minimum amount of shear reinforcement is proposed.

7.2 DESIGN PROVISIONS FOR MINIMUM SHEAR REINFORCEMENT

In general, there are three primary reasons why the minimum shear reinforcement is specified in the design codes: (1) to avoid sudden shear failure right after the formation of inclined cracks; (2) to restrain growth of inclined cracks; and (3) to sustain the concrete contribution to the shear strength prior to yielding of shear reinforcement. Although the existing shear design provisions seem to share these three purposes, it is interesting to note that the minimum shear reinforcement provisions are expressed in different manners.

The current code provisions for the minimum shear reinforcement design are grouped into four different categories. These four categories are presented in Table 7.1.

The ACI 318-08 code adopts three different expressions of the minimum shear reinforcement. The first expression is a function of both of the compressive strength of concrete and the yield strength of shear reinforcement. According to this expression, the minimum amount of shear reinforcement, $A_{v,min}$, is presented in Equation 7-1:

$$A_{v,\min} = K\sqrt{f'_c}\,\frac{b_w s}{f_y} \qquad \text{(psi, in.)} \qquad \textbf{\textit{Equation 7-1}}$$

where:

K = coefficient

f'_c = specified compressive strength of concrete

b_w = web width

s = center-to-center spacing of transverse shear reinforcement

f_y = specified yield strength of transverse reinforcement

Equation 7-1 was first introduced to the ACI 318 code in 2002. According to Table 7.1, other design codes also employ similar expressions. However, there is a disagreement on the coefficient K. The smallest value of the coefficient K is 0.72 in CSA A.23.3-04, and the largest is 1.45 in Model Code 2010, which is twice as large as that specified in the CSA A.23.3-04 code. Despite the disagreement on the coefficient K, it is clear that Equation 7-1 is the most predominant expression for the minimum shear reinforcement design since five design codes adopt this expression.

As mentioned in Chapter 2, the original concept of this expression is based on the experimental results reported by Roller and Russell (1990). These test results indicate that the amount of the minimum shear reinforcement should be increased as the concrete strength increases.

The second expression of the minimum shear reinforcement is presented in Equation 7-2:

$$A_{v,\min} = \frac{50 b_w s}{f_y} \qquad \text{(psi, in.)} \qquad \textbf{\textit{Equation 7-2}}$$

where:

b_w = web width

s = center-to-center spacing of transverse shear reinforcement

f_y = specified yield strength of transverse reinforcement

Equation 7-2 is a function of the yield strength of shear reinforcement but not a function of the compressive strength of concrete. This expression has been utilized in the ACI 318 code for a much longer period of time. The ACI 318-08 code adopts Equation 7-2 as a lower limit of Equation 7-1. At the same time, Equation 7-2 is used for the segmental post-tensioned box girders in AASHTO LRFD 2010.

The third expression is applicable to members with effective prestress in prestressing steel of larger than 40% of its tensile strength. This expression is given in:

$$A_{v,\min} = \frac{A_{ps} f_{pu} s}{80 f_y d} \sqrt{\frac{d}{b_w}} \qquad \text{(psi, in.)} \qquad \textbf{\textit{Equation 7-3}}$$

where:

A_{ps} = area of prestressing steel in flexural tension zone

f_{pu} = specified tensile strength of prestressing steel

s = center-to-center spacing of transverse shear reinforcement

f_y = specified yield strength of transverse reinforcement

d = distance from extreme compression fiber to centroid of prestressed and nonprestressed longitudinal tension reinforcement

b_w = web width

In contrast, Japanese design specifications specify the provisions for the minimum shear reinforcement design as a function of the web width and the spacing of shear reinforcement. It is interesting to note that the minimum shear reinforcement requirements in the Japanese design specifications are not related to both the compressive strength of concrete and the yield strength of shear reinforcement.

Table 7.1 Design provisions for minimum shear reinforcement

Expression	Design Specifications
$K\sqrt{f'_c}\dfrac{b_w s}{f_y}$	- ACI 318-08: K = 0.75 - AASHTO LRFD 2010 (except for segmental post-tensioned box girders): K = 1.00 - CSA A23.3-04: K = 0.72 - Model Code 2010: K = 1.45 - EN 1992: K = 0.96
$\dfrac{50 b_w s}{f_y}$	- ACI 318-08 - AASHTO LRFD 2010 (for segmental post-tensioned box girders)
$\dfrac{A_{ps} f_{pu} s}{80 f_y d}\sqrt{\dfrac{d}{b_w}}$	- ACI 318-08 (applicable when $f_{se} \geq 0.4 f_{pu}$)
$K b_w s$	- JSCE specifications 2007: K = 0.0015 - JRA specifications 2002: K = 0.002 (Deformed steel), 0.003 (Mild steel)

7.3 Reserve Shear Strength

The evaluation of the design provisions for the minimum shear reinforcement is conducted by using the reserve shear strength. The reserve shear strength, which is referred to as RSS, is defined as a ratio of the shear force at failure to that at the formation of inclined cracks. The RSS is given in:

$$RSS = \frac{V_{test}}{V_{c,test}} \qquad \textit{Equation 7-4}$$

where:

V_{test} : shear force at failure

$V_{c,test}$: shear force at formation of inclined cracks

The concept of the RSS has been already employed for the analysis of the minimum shear reinforcement design by several research groups, such as Johnson and Ramirez (1989), Ozcebe, et al. (1999), and Teoh, et al. (2002). In particular, Ozcebe, et al. (1999) and Teoh, et al. (2002) assumed that 1.3 was a safe limit on the RSS to ensure ductile behavior of reinforced and prestressed concrete beams. This limit was derived from experimental results of reinforced concrete beams tested by Ozcebe, et al. (1999). Similarly, a RSS of 1.3 is taken as the safe limit of the minimum amount of shear reinforcement in this evaluation.

7.4 EVALUATION OF DESIGN PROVISIONS FOR MINIMUM SHEAR REINFORCEMENT

Because the safe limit of the RSS is taken as 1.3 in this evaluation, specimens that possess an RSS of at least 1.3 are assumed to demonstrate desirable behavior to resist the shear force. A reasonable limit of the minimum shear reinforcement is expected to be capable of distinguishing specimens that reveal the desirable and undesirable behavior depending on the amount of shear reinforcement. Therefore, this evaluation pays special attention to the number of tests categorized in the following two groups:

- Group I: Test specimens have RSS values of lower than 1.3 but satisfy the minimum shear reinforcement requirement. These test specimens showed less ductile behavior even though the test specimens have at least the minimum amount of shear reinforcement. It is safe to say that a reasonable limit on the minimum shear reinforcement should be able to minimize the number of these test specimens to avoid sudden shear failure in prestressed concrete members after the formation of inclined cracks.

- Group II: Test specimens have RSS values of at least 1.3 but fail to satisfy the minimum shear reinforcement requirement. These test specimens showed the desirable behavior, but contain insufficient amounts of shear reinforcement.

A reasonable limit on the minimum shear reinforcement should be able to minimize the number of these test specimens to avoid providing excessive amounts of shear reinforcement.

7.4.1 Database Analysis of Design Provisions for Minimum Shear Reinforcement

The evaluation database-type MSR, which contains a total of 171 test results, is utilized for the analysis of the design provisions for the minimum shear reinforcement. Since those 171 tests in the evaluation database-type MSR have the reported shear force at the formation of inclined cracks, it is possible to utilize 171 data points of the RSS. The relationship between the RSS and the three expressions of the minimum shear reinforcement except for those in the Japanese specifications is shown in Figure 7.1. All three graphs in Figure 7.1 reveal that if attention is paid on the data points that have RSS values of lower than 1.3, the RSS values are likely to increase as values on the x-axis increase. This trend implies that the number of the test specimens that have RSS values of lower than 1.3 is decreased by increasing the lower limit for the minimum shear reinforcement (i.e., the value for the K factor). However, it should be noted that an increase of the lower limit also increase the amount of shear reinforcement provided in members. Therefore, as mentioned previously, special attention should be paid on the number of tests in the aforementioned two groups simultaneously.

The number of tests categorized with respect to RSS values and the amount of shear reinforcement is presented in Table 7.2. An examination of Table 7.2 shows that 41 of the 171 tests have RSS values of lower than 1.3, and 130 have RSS values of at least 1.3. For Equation 7-1, an increase in the coefficient K contributes to a decrease in the number of tests in group I but an increase in the number of tests in group II. For instance, the limit in the ACI 318-08 code provides 13 tests in group I and 31 tests in group II, while the limit in Model Code 2010 gives two tests in group I and 96 tests in group II. An increase in the coefficient K leads to a significant increase in the number of tests in group II. Based on the evidence presented here, it is safe to say that the coefficient K in the ACI

318-08 code is reasonable to ensure ductile behavior without providing excessive amounts of shear reinforcement.

7.4.2 Proposed Limit on Minimum Shear Reinforcement

A careful examination of Figure 7.1(a) provides another reasonable selection of the coefficient K. In Figure 7.1(a), there is a difference in the number of tests included in group I if the value of K is to be set equal to 0.85. On the basis of this observation, the proposed lower limit for the minimum amount of shear reinforcement is given as follows:

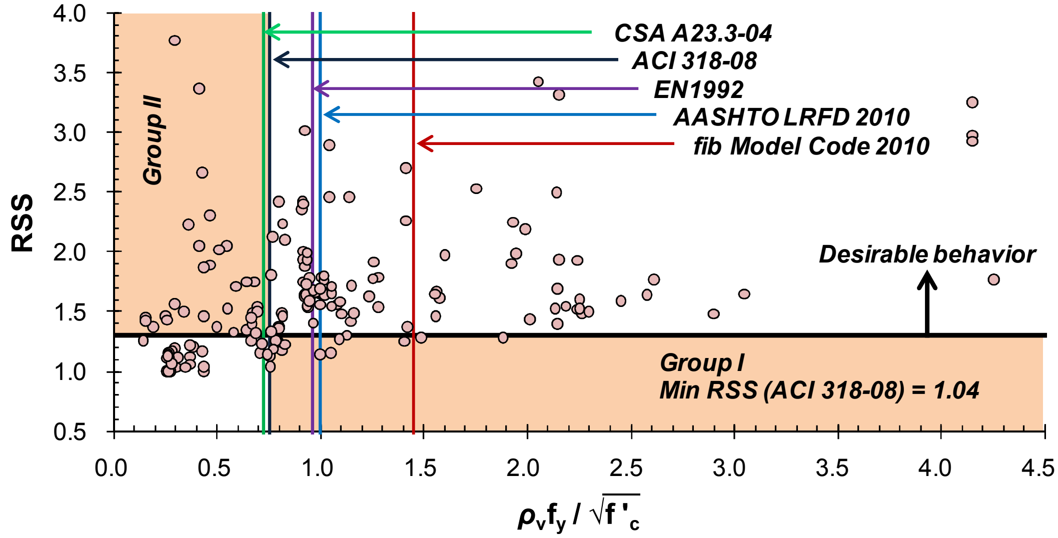
$$A_{v,\min} = 0.85\sqrt{f'_c}\,\frac{b_w s}{f_y} \qquad \text{(psi, in.)} \qquad \textbf{\textit{Equation 7-5}}$$

where:

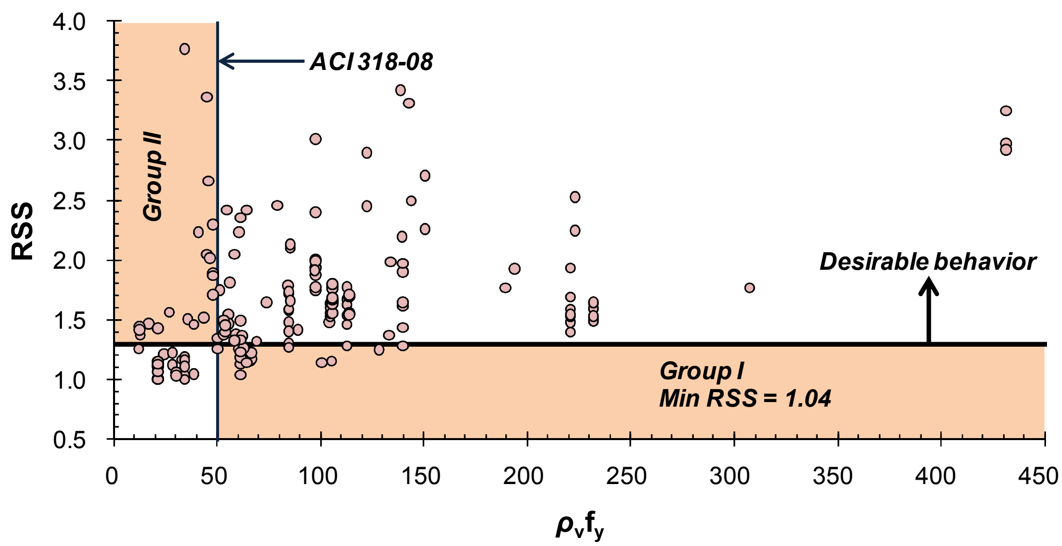
f'_c	=	specified compressive strength of concrete
b_w	=	web width
s	=	center-to-center spacing of transverse shear reinforcement
f_y	=	specified yield strength of transverse reinforcement

The relationship between the RSS values and the proposed limit for the minimum shear reinforcement is shown in Figure 7.2. The results presented in Table 7.3 indicated that the proposed limit reduces the number of tests in group I compared with that obtained by using the ACI 318-08 provisions. Another advantage of the proposed limit is that the minimum RSS value in group I is highly improved. In Figure 7.1, the minimum RSS value obtained by using the limit in the ACI-08 code is between 1.03 and 1.04. Meanwhile, the proposed limit increases the minimum RSS value to 1.14. Although the proposed limit increases the number of tests in group II in relation to the ACI 318-08 code, those advantages are of great significance to ensure ductile behavior after the formation of inclined cracks in prestressed concrete members. It should be emphasized that these advantages indicate that it is possible to reduce the current three types of the shear reinforcement requirement into one equation by using Equation 7.5. The adoption of Equation 7-5 contributes to reduce the complexity of the minimum shear reinforcement design.

Compared with Equation 7-1 in the ACI 318-08 code, the proposed limit leads to an increase in the amount of shear reinforcement by approximately 13.3% (= (0.85 - 0.75) / 0.75). However, it is expected that the possibility of sudden shear failure right after the formation of inclined cracks is greatly reduced.

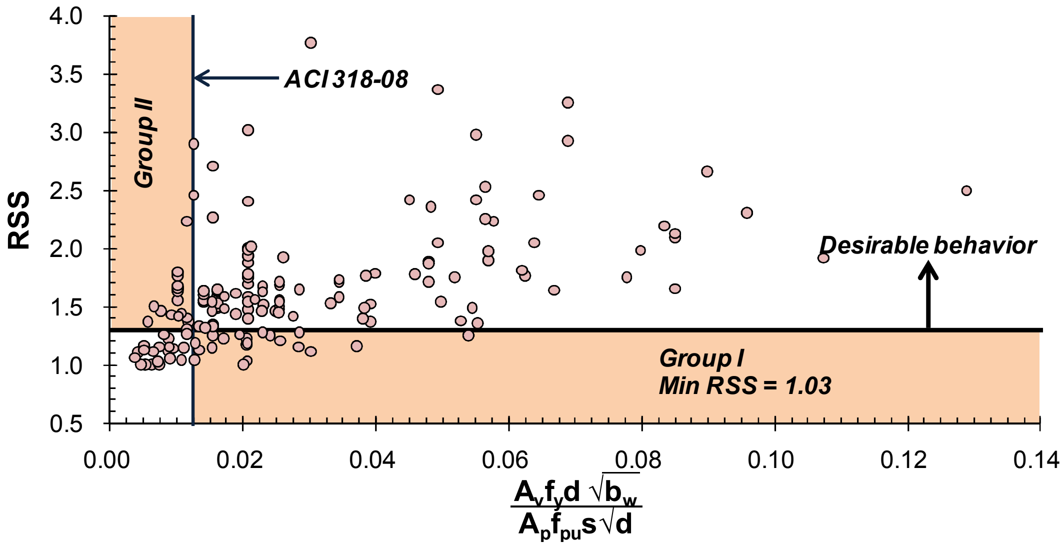


(a) Reserve shear strength versus $\dfrac{\rho_v f_v}{\sqrt{f'_c}}$



(b) Reserve shear strength versus $\rho_v f_v$

Figure 7.1 Reserve shear strength versus limit on minimum shear reinforcement (1/2)



(c) Reserve shear strength versus $\frac{A_v f_y d}{A_p f_{pu} s}\sqrt{\frac{b_w}{d}}$

Figure 7.1 Reserve shear strength versus limits on minimum shear reinforcement (2/2)

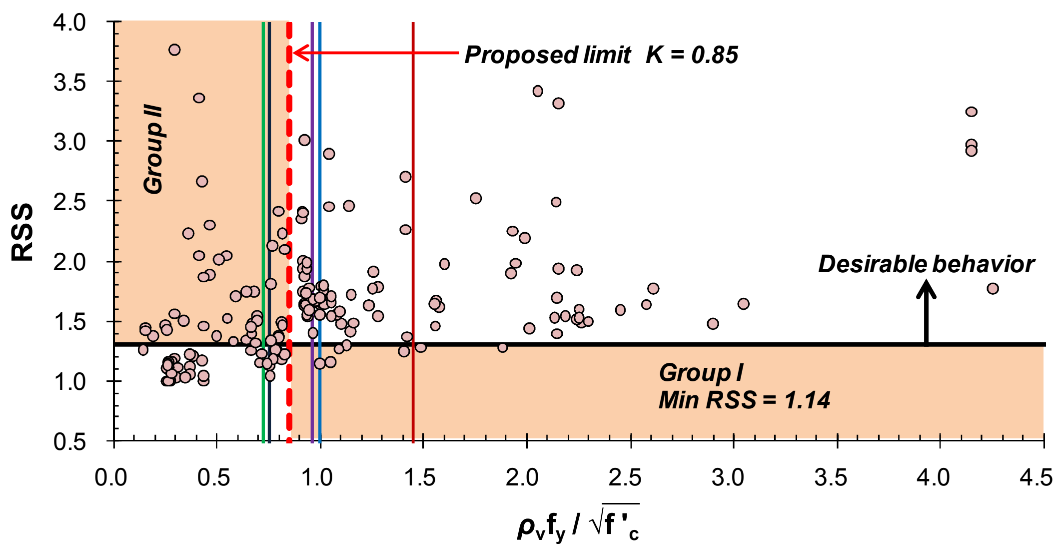


Figure 7.2 Reserve shear strength versus proposed limit on minimum shear reinforcement

Table 7.2 Reserve shear strength and limit on minimum shear reinforcement

Limit		RSS ≥ 1.3			RSS < 1.3		
			w/ $A_{v,min}$	w/o $A_{v,min}$ Group II		w/ $A_{v,min}$ Group I	w/o $A_{v,min}$
$K\sqrt{f'_c}\dfrac{b_w s}{f_y}$ (Eq. 7-1)	ACI 318-08 (K = 0.75)	130 (76.0%)	99 (57.9%)	31 (18.1%)	41 (24.0%)	13 (7.6%)	28 (16.4%)
	AASHTO LRFD 2010 (K = 1.00)		61 (35.7%)	69 (40.4%)		5 (2.9%)	36 (21.1%)
	CSA A23.3-04 (K = 0.72)		99 (57.9%)	31 (18.1%)		14 (8.2%)	27 (15.8%)
	fib MC 2010 (K = 1.45)		34 (19.9%)	96 (56.1%)		2 (1.2%)	39 (22.8%)
	EN 1992 (K = 0.96)		69 (40.4%)	61 (35.7%)		6 (3.5%)	35 (20.5%)
$\dfrac{50 b_w s}{f_y}$ (Eq. 7-2)			111 (64.9%)	19 (11.1%)		17 (9.9%)	24 (14.0%)
$\dfrac{A_{ps} f_{pu} s}{80 f_y d}\sqrt{\dfrac{d}{b_w}}$ (Eq. 7-3)			108 (63.2%)	22 (12.9%)		20 (11.7%)	21 (12.3%)

Note: The percentage of tests in each category is shown in parentheses.

Group I: specimens with RSS of less than 1.3 and with at least the minimum shear reinforcement

Group II: specimens with RSS of at least 1.3 and without the minimum shear reinforcement

Table 7.3 Reserve shear strength and proposed limit on minimum shear reinforcement

Limit		RSS ≥ 1.3			RSS < 1.3		
			w/ $A_{v,min}$	w/o $A_{v,min}$ Group II		w/ $A_{v,min}$ Group I	w/o $A_{v,min}$
$K\sqrt{f'_c}\dfrac{b_w s}{f_y}$	**Proposed** (K = 0.85)	130 (76.0%)	89 (55.6%)	41 (20.5%)	41 (24.0%)	6 (3.5%)	35 (20.5%)

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

A shear database for prestressed concrete members, UTPCSDB-2011, was utilized to evaluate existing shear design provisions. The conclusions derived from this evaluation are summarized as follows:

- A total of 1,696 tests were collected from the literature published in North America, Japan, and Europe from 1954 to 2010.

- An examination of various design codes showed that existing shear design provisions vary considerably. The differences were significant for (1) the shear strength calculation, (2) the minimum shear reinforcement design, and (3) the maximum shear strength limit.

- The shear behavior of prestressed concrete members was investigated by using the data accumulated in the UTPCSDB-2011. There was no evidence of size effect on prestressed concrete members with sufficient shear reinforcement. The shear strength of prestressed concrete members was observed to increase due to the effects of prestress force and shear reinforcement although there was an upper limit on the effectiveness of shear reinforcement.

- The variations in measured versus calculated shear strength using existing twelve shear design equations were evaluated with the UTPCSDB-2011. The MCFT-based design expressions produced the best strength estimations for prestressed concrete members that were reported to have failed in typical shear failure. The ACI 318-08 detailed method provided less conservative strength estimations than those of the MCFT-based design expressions.

- The code provisions for the minimum shear reinforcement were evaluated by using the UTPCSDB-2011. On the basis of this evaluation, a new limit for the minimum shear reinforcement was proposed.

8.2 RECOMMENDATIONS

The following recommendations are made on the basis of the research findings of this study:

- All shear design equations evaluated in this thesis are likely to provide conservative strength estimations. However, it should be noted that different shear design equations provide different levels of safety margin and variation in the estimations of the shear strength.

- Although the MCFT-based design expressions provided the best strength estimations for prestressed concrete members that exhibited typical shear failure, these expressions produced less conservative estimations for members that failed in shear but displayed signs of horizontal shear damage and/or anchorage zone distress. It is clear that the MCFT-based design expressions were not calibrated to consider shear failure modes other than typical shear failures. Special attention should be paid to prevent such damage when the MCFT-based design expressions are used in design.

- The accuracy of the shear design equation for segmental post-tensioned box girders in AASHTO LRFD 2010 is highly improved by removing the limit on the K factor. The removal of the limit on the K factor shows no adverse effect on the safety margin and variation of strength estimations.

- In order to ensure a reserve shear strength of 1.3, the following minimum shear reinforcement design provision is proposed:

  $$A_{v,\min} = 0.85\sqrt{f'_c}\,\frac{b_w s}{f_y} \qquad \text{(psi, in.)}$$

  where:

$A_{v,min}$	=	minimum amount of shear reinforcement
f'_c	=	specified compressive strength of concrete
b_w	=	web width
s	=	center-to-center spacing of transverse shear reinforcement
f_y	=	specified yield strength of transverse reinforcement

- Since the UTPCSDB-2011 is the most comprehensive shear database for prestressed concrete members at the moment, it is recommended that the database be updated as new tests are reported so that it remains an indispensable tool for both research and design applications.

APPENDIX A

EVALUATION DATABASE

The detailed information of the 223 tests included in the evaluation database-level I is tabulated in this appendix. The following information is presented in Table A1.

Specimen I.D.	=	specimen's identification reported in original references
Cross section	=	type of cross section B: Box-beam, I: I-beam, R: rectangular beam, T: T-beam, U: U-beam, deck: specimen has a deck on top
f'_c	=	concrete compressive strength, psi
h	=	overall member height, in.
b_w	=	web width, in.
a/d	=	shear span to depth ratio
$\rho_v f_y$	=	shear reinforcement index, ksi
f_{pc}/f'_c	=	percentage of effective prestress in concrete at centroidal axis, f_{pc}, to concrete compressive strength, f'_c, %
f_{se}/f_{pu}	=	percentage of effective prestress in prestressing steel, f_{se}, to tensile strength of prestressing steel, f_{pu}, %
v_{test}	=	shear stress at failure, ksi
Loading condition	=	C: concentrated loads, U: uniform loads
Prestressing type	=	Pre: pretensioned, Post: post-tensioned
Failure mode	=	shear failure mode reported in original references S: shear failure, FS: flexural shear failure, WC: web crushing failure, SC: shear compression failure, ST: shear tension failure, HS: sign of horizontal shear damage, AD: sign of anchorage zone distress

Table A-1 Evaluation database-level I (1/14)

Specimen I.D.	Cross section	f'_c (psi)	h (in.)	b_w (in.)	a/d	$\rho_v f_y$ (ksi)	f_{pc}/f'_c (%)	f_{se}/f_{pu} (%)	v_{test} (ksi)	Loading condition	Prestresing type	Shear failure mode
Alshegeir & Ramirez (1992)												
Type I-4A	I	8,810	28.0	6.0	2.31	0.19	12.8	66.6	1.04	C	Pre	WC
Type II-1A	I	8,950	36.0	6.0	2.16	0.17	12.9	65.1	1.11	C	Pre	SC
Type I-3A	I	8,810	28.0	6.0	2.35	0.14	12.6	65.5	0.74	C	Pre	SC, AD
Avendaño & Bayrak (2008)												
TX28-I-L	I, deck	13,830	36.0	7.0	3.00	0.29	9.1	55.1	2.04	C	Pre	HS, WC, AD
TX28-I-D	I, deck	13,830	36.0	7.0	3.00	0.29	9.1	55.1	2.12	C	Pre	HS, WC, AD
TX28-II-L	I, deck	11,375	36.0	7.0	3.85	0.35	12.0	55.1	1.89	C	Pre	HS, WC, AD
TX28-II-D	I, deck	11,375	36.0	7.0	3.85	0.35	12.0	55.1	1.91	C	Pre	HS, WC, AD
Bennett & Balasooriya (1971)												
2F1	I	5,680	18.0	1.0	2.00	1.08	32.2	59.7	2.05	C	Post	WC
2F2	I	5,680	18.0	1.0	2.00	1.08	27.5	57.7	2.00	C	Post	WC
2F3	I	5,680	18.0	1.0	2.00	1.08	18.8	41.2	1.78	C	Post	WC
2F4	I	5,800	18.0	1.3	2.00	1.08	6.8	31.7	1.64	C	Post	WC
Bennett & Debaiky (1974)												
NM-6-240	I	5,256	13.0	2.0	3.02	0.14	21.4	44.6	0.85	C	Post	ST
NH-6-240	I	4,838	13.0	2.0	3.02	0.17	23.4	44.9	0.85	C	Post	ST
NL-6-160	I	5,337	13.0	2.0	3.02	0.16	21.2	44.7	0.86	C	Post	ST
NM-6-160	I	5,221	13.0	2.0	3.02	0.21	21.9	45.4	0.89	C	Post	ST
NL-6-80	I	5,430	13.0	2.0	3.02	0.32	21.0	45.0	1.02	C	Post	SC

Table A-1 Evaluation database-level I (2/14)

Specimen I.D.	*Cross section*	f'_c (psi)	h (in.)	b_w (in.)	a/d	$\rho_v f_y$ (ksi)	f_{pc}/f'_c (%)	f_{se}/f_{pu} (%)	v_{test} (ksi)	*Loading condition*	*Prestresing type*	*Shear failure mode*
Bennett & Debaiky (1974) continued…												
NH-6-80	I	5,291	13.0	2.0	3.02	0.52	21.5	44.9	1.09	C	Post	SC
NL-10-160	I	5,256	13.0	2.0	3.02	0.35	21.6	45.1	0.98	C	Post	ST
NM-10-160	I	5,221	13.0	2.0	3.02	0.57	21.9	45.4	0.98	C	Post	SC
PM-6-160	I	5,766	13.0	2.0	3.02	0.21	28.2	64.7	0.92	C	Post	ST
PH-6-160	I	5,511	13.0	2.0	3.02	0.26	29.1	63.5	0.89	C	Post	ST
PL-6-80	I	5,975	13.0	2.0	3.02	0.32	27.1	64.4	1.01	C	Post	SC
CM-6-240	I	7,774	13.0	2.0	3.02	0.14	14.8	45.5	0.95	C	Post	ST
CH-6-240	I	7,890	13.0	2.0	3.02	0.17	14.4	45.0	0.98	C	Post	ST
CL-6-160	I	8,006	13.0	2.0	3.02	0.16	14.2	45.1	0.98	C	Post	ST
CM-6-160	I	8,354	13.0	2.0	3.02	0.21	13.5	44.5	1.06	C	Post	ST
CH-6-160	I	8,099	13.0	2.0	3.02	0.26	14.0	45.0	1.07	C	Post	SC
CL-6-80	I	8,238	13.0	2.0	3.02	0.32	13.9	45.3	1.11	C	Post	SC
CM-6-80	I	8,122	13.0	2.0	3.02	0.42	14.2	45.7	1.20	C	Post	SC
CH-6-80	I	7,635	13.0	2.0	3.02	0.52	14.9	45.8	1.34	C	Post	SC
Choulli, Marí & Cladera (2008)												
C2TE	I	13,053	29.5	3.9	3.13	0.38	5.3	30.0	1.56	C	Pre	S
C2TW	I	13,053	29.5	3.9	3.13	0.38	5.3	30.0	1.48	C	Pre	S
C1TE	I	11,748	29.5	3.9	3.13	0.38	5.9	30.0	1.37	C	Pre	S
C1TW	I	11,748	29.5	3.9	3.13	0.38	5.9	30.0	1.62	C	Pre	S

Table A-1 Evaluation database-level I (3/14)

Specimen I.D.	**Cross section**	**f'_c (psi)**	**h (in.)**	**b_w (in.)**	**a/d**	**$\rho_v f_y$ (ksi)**	**f_{pc}/f'_c (%)**	**f_{se}/f_{pu} (%)**	**v_{test} (ksi)**	**Loading condition**	**Prestresing type**	**Shear failure mode**
De Silva, Mutsuyoshi, Witchukreangkrai & Takagi (2006)												
IPRC-1	I	6,062	19.7	5.9	3.30	0.21	7.2	57.4	0.84	C	Post	S
IPRC-2	I	7,150	19.7	5.9	3.30	0.12	6.1	57.4	0.76	C	Post	S
IPRC-3	I	6,526	19.7	5.9	3.30	0.21	6.7	57.4	0.74	C	Post	S
IPRC-4	I	6,265	19.7	5.9	3.18	0.21	6.9	57.4	0.85	C	Post	S
Durrani & Robertson (1978)												
3	T	6,688	20.0	3.0	3.52	0.14	7.6	51.5	0.93	C	Pre	S
4	T	6,401	20.0	3.0	3.52	0.14	8.8	57.0	0.99	C	Pre	S
5	T	6,473	20.0	3.0	3.52	0.13	8.6	56.7	1.00	C	Pre	S
8	T	5,720	20.0	3.0	3.52	0.10	9.8	56.7	1.02	C	Pre	S
10	T	6,097	20.0	3.0	3.52	0.10	9.1	56.3	1.03	C	Pre	S
11	T	6,061	20.0	3.0	3.52	0.19	9.1	55.9	1.07	C	Pre	S
Elzanaty, Nilson & Slate (1986)												
CW10	I	10,600	18.0	2.0	3.80	0.35	11.1	42.2	1.31	C	Pre	S
CW11	I	8,100	18.0	2.0	3.80	0.35	14.2	41.1	1.18	C	Pre	S
CW12	I	5,800	18.0	2.0	3.80	0.35	19.9	41.3	1.06	C	Pre	S
CW13	I	10,500	18.0	2.0	3.80	0.35	15.4	57.9	1.37	C	Pre	S
CW14	I	10,700	18.0	2.0	3.80	0.50	15.3	58.5	1.41	C	Pre	S
CW15	I	10,200	18.0	2.0	3.80	0.35	11.4	41.5	1.13	C	Pre	S
CW16	I	10,600	18.0	2.0	3.80	0.35	15.3	58.2	1.41	C	Pre	S

Table A-1 Evaluation database-level I (4/14)

Specimen I.D.	***Cross section***	***f'c (psi)***	***h (in.)***	***b_w (in.)***	***a/d***	***$\rho_v f_y$ (ksi)***	***f_{pc}/f'_c (%)***	***f_{se}/f_{pu} (%)***	***v_{test} (ksi)***	***Loading condition***	***Prestresing type***	***Shear failure mode***
Elzanaty, Nilson & Slate (1986) continued…												
CI10	T	10,600	14.0	3.0	5.80	0.29	10.3	42.2	1.01	C	Pre	SC
CI11	T	8,100	14.0	3.0	5.80	0.29	13.1	41.3	0.91	C	Pre	SC
CI12	T	5,800	14.0	3.0	5.80	0.29	18.4	41.5	0.88	C	Pre	SC
CI13	T	10,500	14.0	3.0	5.80	0.29	14.2	57.8	1.11	C	Pre	SC
CI14	T	10,700	14.0	3.0	5.80	0.46	14.2	58.8	1.18	C	Pre	SC
CI15	T	10,200	14.0	3.0	5.80	0.29	10.5	41.6	0.87	C	Pre	SC
CI16	T	10,600	14.0	3.0	5.80	0.29	14.2	58.4	1.17	C	Pre	SC
Hamilton, Lianos & Ross (2009)												
B1U4	I, deck	5,630	52.0	7.0	3.50	0.18	4.6	66.3	0.46	C	Pre	SC, AD
B4U4	I, deck	5,630	52.0	7.0	3.50	0.18	4.6	66.3	0.51	C	Pre	SC, AD
Hanson & Hulsbos (1964)												
F-X1-1st	I	6,650	18.0	3.0	3.04	0.12	16.2	48.8	0.68	C	Pre	WC
F-2-1st	I	6,550	18.0	3.0	2.53	0.12	15.5	45.9	0.84	C	Pre	WC
F-4-1st	I	6,340	18.0	3.0	3.16	0.12	17.6	50.3	0.80	C	Pre	ST
F-19-1st	I	7,410	18.0	3.0	3.16	0.16	14.3	47.8	0.84	C	Pre	ST
F-X1-2nd	I	6,650	18.0	3.0	3.04	0.12	16.2	48.8	0.79	C	Pre	WC
F-2-2nd	I	6,550	18.0	3.0	2.53	0.19	15.5	45.9	1.01	C	Pre	ST
F-3-2nd	I	6,840	18.0	3.0	2.53	0.29	15.1	46.7	1.06	C	Pre	WC
F-4-2nd	I	6,340	18.0	3.0	3.16	0.16	17.6	50.3	0.84	C	Pre	WC

Table A-1 Evaluation database-level I (5/14)

Specimen I.D.	Cross section	f'_c (psi)	h (in.)	b_w (in.)	a/d	$\rho_v f_y$ (ksi)	f_{pc}/f'_c (%)	f_{se}/f_{pu} (%)	v_{test} (ksi)	Loading condition	Prestresing type	Shear failure mode
Hanson & Hulsbos (1964) continued…												
F-5-2nd	I	6,410	18.0	3.0	3.16	0.19	16.0	46.3	0.85	C	Pre	SC
F-7-2nd	I	6,620	18.0	3.0	3.80	0.13	16.7	49.9	0.73	C	Pre	WC
F-8-2nd	I	6,880	18.0	3.0	3.80	0.16	15.6	48.7	0.78	C	Pre	SC
F-19-2nd	I	7,410	18.0	3.0	3.16	0.19	14.3	47.8	0.84	C	Pre	WC
Hanson & Hulsbos (1969)												
G1-1	B	7,920	36.0	10.0	3.32	0.11	10.9	59.2	0.61	C	Pre	S
G4-3	I	7,580	36.0	6.0	2.76	0.09	10.2	60.1	0.70	C	Pre	SC
Hartman, Breen & Kreger (1988)												
2-1	I	10,800	21.5	2.0	3.00	1.61	14.0	55.2	2.63	C	Pre	WC
2-2	I	10,800	21.5	2.0	3.00	2.01	13.7	55.2	2.87	C	Pre	WC
2-3	I	10,800	21.5	2.0	3.00	2.01	13.4	55.2	2.82	C	Pre	WC
3-1	I, deck	13,000	20.3	2.0	3.21	0.90	6.6	50.4	1.98	C	Pre	S, AD
3-2	I, deck	13,160	20.3	2.0	3.21	0.90	6.8	51.3	2.04	C	Pre	S, AD
3-3	I, deck	11,500	20.3	2.0	3.21	0.45	5.3	56.2	1.30	C	Pre	S, AD
3-4	I, deck	11,500	20.3	2.0	3.21	0.45	5.4	56.2	1.54	C	Pre	WC
Hawkins & Kuchma (2007)												
G1E	I, deck	12,100	73.0	6.0	1.10	0.39	5.6	59.1	1.17	U	Pre	SC, HS, AD
G1W	I, deck	12,100	73.0	6.0	1.04	0.39	5.6	59.1	1.40	U	Pre	SC, HS, AD
G2E	I, deck	12,600	73.0	6.0	1.00	0.74	6.2	55.6	1.59	U	Pre	SC, HS, AD

Table A-1 Evaluation database-level I (6/14)

Specimen I.D.	***Cross section***	***f'_c* (psi)**	***h* (in.)**	***b_w* (in.)**	***a/d***	***$\rho_v f_y$* (ksi)**	***f_{pc}/f'_c* (%)**	***f_{se}/f_{pu}* (%)**	***v_{test}* (ksi)**	***Loading condition***	***Prestresing type***	***Shear failure mode***
Hawkins & Kuchma (2007) continued…												
G2W	I, deck	12,600	73.0	6.0	0.94	0.74	6.2	55.6	1.89	U	Pre	SC, HS, AD
G3E	I, deck	15,900	73.0	6.0	1.09	0.57	5.4	57.4	1.66	U	Pre	SC, HS
G3W	I, deck	15,900	73.0	6.0	1.09	0.57	5.4	57.4	1.80	U	Pre	SC, HS
G5E	I, deck	17,800	73.0	6.0	1.19	0.17	2.1	63.9	1.02	U	Pre	SF, HS
G6E	I, deck	12,700	73.0	6.0	1.09	0.56	7.3	62.1	1.57	U	Pre	SC, HS
G6W	I, deck	12,700	73.0	6.0	0.99	0.56	5.3	62.1	1.32	U	Pre	SC, HS, AD
G7E	I, deck	12,500	73.0	6.0	1.07	0.58	7.4	61.9	1.56	U	Pre	SC
G7W	I, deck	12,500	73.0	6.0	1.07	0.58	7.4	61.9	1.54	U	Pre	SC, HS
G8E	I, deck	13,300	73.0	6.0	1.73	0.58	6.6	58.7	1.54	U	Pre	SC
G8W	I, deck	13,300	73.0	6.0	1.09	0.58	6.6	58.7	1.39	U	Pre	SC
G9E	I, deck	9,600	73.0	6.0	0.84	1.04	8.5	62.3	1.64	U	Pre	SC, HS
G10E	I, deck	10,600	73.0	6.0	0.87	0.75	6.8	64.3	1.69	U	Pre	SC
G10W	I, deck	10,600	73.0	6.0	0.89	0.75	8.0	64.3	2.11	U	Pre	SC, HS
Heckmann (2008)												
B-C-70-1	I	12,100	40.0	7.0	2.20	0.14	11.1	62.0	1.56	C	Pre	ST
B-C-70-4	I	12,430	40.0	7.0	2.20	0.14	10.6	60.5	1.55	C	Pre	ST
B-C-70-5	I	12,500	40.0	7.0	2.20	0.14	10.4	60.1	1.48	C	Pre	ST
B-C-70-6	I	12,800	40.0	7.0	2.20	0.14	10.0	59.3	1.63	C	Pre	ST
B-C-60-1	I	12,300	40.0	7.0	2.20	0.14	10.9	62.0	1.59	C	Pre	ST

Table A-1 Evaluation database-level I (7/14)

Specimen I.D.	*Cross section*	f'_c (psi)	h (in.)	b_w (in.)	a/d	$\rho_v f_y$ (ksi)	f_{pc}/f'_c (%)	f_{se}/f_{pu} (%)	v_{test} (ksi)	*Loading condition*	*Prestresing type*	*Shear failure mode*
Heckmann (2008) continued…												
B-C-60-2	I	12,700	40.0	7.0	2.20	0.14	10.6	62.1	1.56	C	Pre	ST
Hernandez (1958)												
G7	I	4,660	12.0	1.7	3.55	0.06	13.4	45.5	0.80	C	Pre	S
G29	I	4,330	12.0	1.8	2.79	0.10	13.9	46.3	1.01	C	Pre	S
Hovell, Avendaño, Dunkman, Moore, Bayrak & Jirsa (2010)												
BB-01Q	B	11,300	28.0	10.0	2.93	0.12	7.3	61.4	0.99	C	Pre	WC
BB-01K	B	11,300	28.0	10.0	3.42	0.12	7.3	61.4	1.00	C	Pre	WC
BB-02Q	B	11,300	28.0	10.0	2.93	0.12	7.3	61.4	0.99	C	Pre	WC
BB-02K	B	11,300	28.0	10.0	3.42	0.12	7.3	61.4	1.02	C	Pre	WC
BB-03Q	B	11,160	28.0	10.0	2.93	0.12	7.4	61.4	1.18	C	Pre	WC
BB-03K	B	11,160	28.0	10.0	3.42	0.12	7.4	61.4	1.21	C	Pre	WC
BB-04Q	B	10,672	28.0	10.0	2.93	0.12	7.7	61.4	1.18	C	Pre	WC
BB-05Q	B	10,900	28.0	10.0	2.93	0.12	7.5	61.4	1.22	C	Pre	WC
BB-05K	B	10,900	28.0	10.0	3.42	0.12	7.5	61.4	1.16	C	Pre	WC
UB-1N	U, deck	11,900	62.8	10.3	2.59	0.64	4.4	58.1	1.09	C	Pre	HS
UB-1S	U, deck	11,900	62.8	10.3	2.62	0.64	4.4	58.1	1.01	C	Pre	HS
UB-2N	U, deck	11,500	62.8	10.3	2.59	0.83	5.1	58.1	1.01	C	Pre	HS
UB-3N	U, deck	11,400	62.8	10.3	2.63	0.63	2.5	58.1	1.09	C	Pre	HS
UB-3S	U, deck	12,100	62.8	10.3	2.63	0.63	2.2	58.1	1.10	C	Pre	HS

Table A-1 Evaluation database-level I (8/14)

Specimen I.D.	***Cross section***	**f'_c (psi)**	**h (in.)**	**b_w (in.)**	**a/d**	**$\rho_v f_y$ (ksi)**	**f_{pc}/f'_c (%)**	**f_{se}/f_{pu} (%)**	**v_{test} (ksi)**	***Loading condition***	***Prestresing type***	***Shear failure mode***
Hovell, Avendaño, Dunkman, Moore, Bayrak & Jirsa (2010) continued…												
UB-4N	U, deck	11,400	62.8	16.0	2.62	0.53	7.6	58.1	1.04	C	Pre	WC, HS
UB-5N	U, deck	13,200	62.8	10.3	2.60	0.98	2.1	58.1	1.39	C	Pre	FS
Imano, Ikeda, Kishi & Takemoto (2001)												
2	T	10,268	35.4	15.7	3.91	0.17	2.4	25.7	0.40	C	Pre	S
3	T	10,268	35.4	15.7	3.91	0.17	1.2	12.8	0.39	C	Pre	S
Kang, Wu, Wang & Xue (1989)												
YB8-2E	I	4,525	21.7	3.6	2.00	0.11	12.0	24.9	0.96	C	Pre	SC
YB8-2W	I	4,525	21.7	3.4	2.00	0.11	12.1	24.5	0.98	C	Pre	SC
YB10-2	I	4,525	21.7	3.7	2.00	0.10	14.6	31.0	0.92	C	Pre	ST
Kaufman & Ramirez (1988)												
I-2	I	8,340	28.0	6.0	2.20	0.12	12.9	64.8	0.95	C	Pre	WC
I-3	I	8,370	28.0	6.0	2.20	0.14	13.3	66.9	0.65	C	Pre	ST
I-4	I	8,370	28.0	6.0	2.20	0.12	13.4	68.0	0.72	C	Pre	ST
II-1	I	9,090	36.0	6.0	2.40	0.16	12.7	66.4	0.70	C	Pre	ST
Labonte & Hamilton (2005)												
S1-STDS	I	7,490	36.0	6.0	2.25	0.13	11.4	63.8	1.00	C	Pre	SC
Lee, Cho & Oh (2010)												
C40P2S10	I	6,584	47.2	7.9	2.47	0.16	8.1	54.4	1.02	C	Post	SC
C40P2S13	I	6,584	47.2	7.9	2.47	0.29	8.1	54.4	1.10	C	Post	SC

Table A-1 Evaluation database-level I (9/14)

Specimen I.D.	Cross section	f'_c (psi)	h (in.)	b_w (in.)	a/d	$\rho_v f_y$ (ksi)	f_{pc}/f'_c (%)	f_{se}/f_{pu} (%)	v_{test} (ksi)	Loading condition	Prestresing type	Shear failure mode
Lee, Cho & Oh (2010) continued…												
C60P1S10	I	10,645	47.2	7.9	2.47	0.16	2.7	59.4	0.97	C	Post	SC
C60P2S10	I	10,645	47.2	7.9	2.47	0.16	5.0	54.4	1.19	C	Post	SC
C60P2S13	I	10,645	47.2	7.9	2.47	0.29	5.0	54.4	1.33	C	Post	SC
C80P2S10	I	12,313	47.2	7.9	2.47	0.16	4.3	54.4	1.12	C	Post	SC
C80P2S13	I	12,313	47.2	7.9	2.47	0.29	4.3	54.4	1.35	C	Post	SC
Lyngberg (1976)												
2A-3	I	4,728	23.6	4.7	2.78	0.47	12.1	50.5	1.13	C	Post	WC
2B-3	I	4,917	23.6	4.7	2.78	0.49	11.6	50.3	1.15	C	Post	WC
3A-2	I	4,511	23.6	4.7	2.78	0.51	8.5	50.5	1.09	C	Post	WC
4A-1	I	4,569	23.6	4.7	2.78	0.49	4.3	52.1	1.05	C	Post	WC
4B-1	I	4,409	23.6	4.7	2.78	0.51	4.3	50.2	1.02	C	Post	WC
MacGregor, Sozen & Siess (1960)												
BV.14.30	I	4,020	12.0	3.0	3.56	0.05	14.0	48.2	0.41	C	Pre	S
Maruyama & Rizkalla (1988)												
PS2-S6M	T	6,309	19.3	3.8	2.69	0.14	3.0	32.8	0.54	C	Pre	S, AD
PS3-D2	T	6,483	19.3	3.8	2.69	0.18	2.9	32.8	0.55	C	Pre	S, AD
PS4-M2	T	6,265	19.3	3.8	2.69	0.11	3.0	32.8	0.52	C	Pre	S, AD
PS6-WD	T	5,526	19.3	3.8	2.69	0.14	3.5	32.8	0.49	C	Pre	S, AD
PS7-WSH	T	5,685	19.3	3.8	2.69	0.15	3.4	32.8	0.48	C	Pre	S, AD

Table A-1 Evaluation database-level I (10/14)

Specimen I.D.	***Cross section***	***f'_c* (psi)**	***h* (in.)**	***b_w* (in.)**	***a/d***	***$\rho_v f_y$* (ksi)**	***f_{pc}/f'_c* (%)**	***f_{se}/f_{pu}* (%)**	***v_{test}* (ksi)**	***Loading condition***	***Prestresing type***	***Shear failure mode***
Maruyama & Rizkalla (1988) continued…												
PS8-WS	T	5,830	19.3	3.8	2.69	0.15	3.3	32.8	0.44	C	Pre	S, AD
PS9-WDH	T	6,019	19.3	3.8	2.69	0.15	3.2	32.8	0.45	C	Pre	S, AD
Moayer & Regan (1974)												
P8	T	6,190	12.6	5.9	3.60	0.10	13.8	62.7	0.63	C	Post	S
P13	T	5,710	12.6	5.9	3.48	0.10	5.1	64.4	0.48	C	Post	S
P18	T	6,450	12.6	5.9	3.65	0.10	13.3	62.7	0.57	C	Post	S
P49	T	5,480	12.6	5.9	3.56	0.15	15.6	62.7	0.66	C	Post	S
P50	T	5,970	12.6	5.9	3.56	0.27	14.3	62.7	0.80	C	Post	S
Naito, Parent & Brunn (2005)												
B1	I	9,183	45.0	7.0	2.21	1.25	9.8	54.4	1.67	C	Pre	WC
Nakamura, Takeuchi, Aoyama, Murakoshi & Kimura (2009)												
1	R	8,412	16.7	7.9	3.00	0.09	2.5	7.7	0.47	C	Pre	SC
Ramirez & Aguilar (2005)												
13.3-5.1-326P	I	13,340	28.0	6.0	3.62	0.31	7.2	63.8	1.18	C	Pre	SC
16.2-5.1-326P	I	16,150	28.0	6.0	3.66	0.31	5.9	63.8	1.41	C	Pre	SC
Rangan (1991)												
II-1	I	6,526	24.2	2.5	2.48	1.33	6.2	62.3	1.85	C	Pre	WC
II-2	I	4,569	24.2	2.5	2.48	2.24	9.0	62.3	1.55	C	Pre	WC
II-3	I	6,468	24.2	2.9	2.48	1.16	6.1	62.3	1.72	C	Pre	WC

Table A-1 Evaluation database-level I (11/14)

Specimen I.D.	***Cross section***	***f'c* (psi)**	***h* (in.)**	***b_w* (in.)**	***a/d***	***$\rho_v f_y$* (ksi)**	***f_{pc}/f'_c* (%)**	***f_{se}/f_{pu}* (%)**	***v_{test}* (ksi)**	***Loading condition***	***Prestresing type***	***Shear failure mode***
Rangan (1991) continued…												
II-4	I	6,236	24.2	2.9	2.48	1.91	6.3	62.3	1.67	C	Pre	WC
III-1	I	5,801	24.2	2.6	2.50	1.29	10.0	62.3	1.44	C	Pre	WC
III-2	I	5,366	24.2	2.6	2.50	2.14	10.8	62.3	1.53	C	Pre	WC
III-3	I	5,656	24.2	3.0	2.50	1.10	9.9	62.3	1.33	C	Pre	WC
III-4	I	5,366	24.2	2.8	2.50	1.96	10.6	62.3	1.63	C	Pre	WC
IV-1	I	5,381	24.2	2.4	2.62	2.28	19.6	62.3	1.64	C	Pre	WC
IV-2	I	4,786	24.2	2.5	2.62	1.33	21.9	62.3	1.43	C	Pre	WC
IV-3	I	5,221	24.2	2.8	2.62	1.96	19.5	62.3	1.75	C	Pre	WC
IV-4	I	4,162	24.2	2.8	2.62	1.18	24.5	62.3	1.47	C	Pre	WC
Runzell, Shield & French (2007)												
I	I, deck	10,130	63.0	8.0	3.01	0.16	3.0	46.4	0.83	C	Pre	WC
II	I	10,130	54.0	8.0	3.57	0.16	10.3	46.4	0.82	C	Pre	WC
Sato, Ishibashi, Yamashita & Takada (1987)												
1-3	T	6,286	15.7	5.9	3.06	0.13	7.5	32.7	0.83	C	Post	S
1-4	T	6,258	15.7	5.9	3.40	0.27	7.6	32.7	0.81	C	Post	S
1-5	T	6,130	15.7	5.9	3.06	0.13	7.7	32.7	0.67	C	Post	S
1-6	T	6,286	15.7	5.9	3.06	0.27	7.5	32.7	0.81	C	Post	S
1-7	T	6,315	15.7	5.9	3.06	0.13	12.5	54.5	0.89	C	Post	S
1-8	T	6,372	15.7	5.9	3.06	0.27	12.4	54.5	0.99	C	Post	S

Table A-1 Evaluation database-level I (12/14)

Specimen I.D.	Cross section	f'_c (psi)	h (in.)	b_w (in.)	a/d	$\rho_v f_y$ (ksi)	f_{pc}/f'_c (%)	f_{se}/f_{pu} (%)	v_{test} (ksi)	Loading condition	Prestresing type	Shear failure mode
Sato, Ishibashi, Yamashita & Takada (1987) continued…												
1-9	T	6,329	15.7	5.9	3.06	0.13	12.5	54.5	0.65	C	Post	S
1-10	T	5,831	15.7	5.9	3.40	0.27	13.6	54.5	0.74	C	Post	S
2-3	R	6,415	15.7	5.9	3.27	0.13	7.4	21.8	0.45	C	Post	S
2-4	R	6,400	15.7	5.9	3.06	0.13	7.4	21.8	0.53	C	Post	S
2-6	T	5,262	15.7	5.9	3.06	0.13	3.0	10.9	0.68	C	Post	S
2-7	T	5,191	15.7	5.9	3.06	0.13	6.1	21.8	0.64	C	Post	S
2-8	T	6,187	15.7	5.9	3.06	0.13	5.1	21.8	0.82	C	Post	S
2-9	T	6,343	15.7	5.9	3.06	0.13	10.0	43.6	0.79	C	Post	S
2-10	T	5,646	15.7	5.9	3.06	0.13	11.2	43.6	0.78	C	Post	S
4-1	T	5,831	15.7	5.9	3.06	0.27	5.4	21.8	0.72	C	Post	S
4-2	T	6,486	15.7	5.9	3.06	0.13	9.7	43.6	0.67	C	Post	S
4-3	T	6,030	15.7	5.9	3.06	0.27	2.6	10.9	0.75	C	Post	S
4-13	R	5,760	15.7	5.9	3.06	0.27	4.1	10.9	0.58	C	Post	S
4-14	R	5,860	15.7	5.9	3.06	0.12	8.1	21.8	0.47	C	Post	S
4-15	R	5,689	15.7	5.9	2.49	0.27	12.5	32.7	0.78	C	Post	S
4-16	R	5,931	15.7	5.9	2.49	0.27	4.0	10.9	0.65	C	Post	S
4-17	R	5,931	15.7	5.9	2.49	0.12	4.0	10.9	0.61	C	Post	S
Shahawy & Batchelor (1986)												
A0-00-R-N	I, deck	6,000	44.0	6.0	2.11	0.38	1.5	56.9	1.30	C	Pre	S, AD

Table A-1 Evaluation database-level I (13/14)

Specimen I.D.	***Cross section***	f'_c **(psi)**	h **(in.)**	b_w **(in.)**	a/d	$\rho_v f_y$ **(ksi)**	f_{pc}/f'_c **(%)**	f_{se}/f_{pu} **(%)**	v_{test} **(ksi)**	***Loading condition***	***Prestresing type***	***Shear failure mode***
Shahawy & Batchelor (1986) continued…												
A0-00-R-S	I, deck	6,000	44.0	6.0	2.11	0.38	1.5	56.9	1.14	C	Pre	S
A1-00-R/2-N	I, deck	6,000	44.0	6.0	2.53	0.13	1.5	56.9	0.69	C	Pre	S, AD
A1-00-R/2-S	I, deck	6,000	44.0	6.0	3.08	0.13	1.5	56.9	0.72	C	Pre	S, AD
A1-00-R-N	I, deck	6,000	44.0	6.0	2.53	0.25	1.5	56.9	0.87	C	Pre	S, AD
A1-00-3R/2-N	I, deck	6,000	44.0	6.0	2.53	0.38	1.5	56.9	0.86	C	Pre	S, AD
B0-00-R-N	I, deck	6,000	44.0	6.0	2.52	0.25	1.1	57.0	0.90	C	Pre	S, AD
B0-00-R-S	I, deck	6,000	44.0	6.0	3.06	0.25	1.1	57.0	0.85	C	Pre	S
Tawfiq (1995)												
R8N	I, deck	8,150	44.0	6.0	2.68	0.50	0.7	63.8	1.14	C	Pre	FS, AD
R10N	I, deck	10,130	44.0	6.0	2.68	0.50	0.6	63.8	1.16	C	Pre	FS
R12N	I, deck	11,040	44.0	6.0	2.68	0.50	0.5	63.8	1.15	C	Pre	FS, AD
2R8N	I, deck	8,120	44.0	6.0	2.68	1.00	0.7	63.8	0.96	C	Pre	FS, AD
2R10N	I, deck	9,910	44.0	6.0	2.68	1.00	0.6	63.8	0.99	C	Pre	FS, AD
2R12N	I, deck	11,040	44.0	6.0	2.68	1.00	0.5	63.8	1.15	C	Pre	FS, AD
R8S	I, deck	8,150	44.0	6.0	2.26	0.50	0.7	63.8	1.24	C	Pre	S, AD
R10S	I, deck	10,130	44.0	6.0	2.26	0.50	0.6	63.8	1.23	C	Pre	S, AD
R12S	I, deck	11,040	44.0	6.0	2.26	0.50	0.5	63.8	1.13	C	Pre	S, AD
2R8S	I, deck	8,120	44.0	6.0	2.26	1.00	0.7	63.8	1.05	C	Pre	S, AD
2R10S	I, deck	9,910	44.0	6.0	2.26	1.00	0.6	63.8	1.01	C	Pre	S, AD

Table A-1 Evaluation database-level I (14/14)

Specimen I.D.	***Cross section***	***f'_c* (psi)**	***h* (in.)**	***b_w* (in.)**	***a/d***	***$\rho_v f_y$* (ksi)**	***f_{pc}/f'_c* (%)**	***f_{se}/f_{pu}* (%)**	***v_{test}* (ksi)**	***Loading condition***	***Prestresing type***	***Shear failure mode***
Tawfiq (1995) continued…												
2R12S	I, deck	11,040	44.0	6.0	2.26	1.00	0.5	63.8	1.18	C	Pre	S, AD
Teoh, Mansur & Wee (2002)												
B6-12	I	15,417	27.6	5.9	2.71	0.13	4.3	60.0	0.63	C	Pre	S
Xuan, Rizkalla & Maruyama (1988)												
PSN2-WD	T	5,526	19.3	3.8	2.76	0.14	7.4	70.0	0.79	C	Pre	S
PSN3-D2	T	4,830	19.3	3.8	2.76	0.09	8.4	70.0	0.94	C	Pre	S
PSN4-WDH	T	4,569	19.3	3.8	2.76	0.12	8.9	70.0	0.90	C	Pre	S
PSN5-S6M	T	4,714	19.3	3.8	2.76	0.09	8.6	70.0	0.92	C	Pre	S
PSN6-WS	T	4,975	19.3	3.8	2.76	0.13	8.2	70.0	0.92	C	Pre	S

APPENDIX B

JAPANESE REFERENCES

A review of selected Japanese references that is focused on the shear behavior of prestressed concrete members is presented in this appendix. This review provides a brief summary of the research objectives, experimental procedures, and primary findings of the selected Japanese references.

B.1 HIGH STRENGTH CONCRETE

B.1.1 Public Works Research Institute (1995)

The public works research institute (1995) reported results obtained by testing 25 pretensioned concrete beams fabricated with high strength concrete. The specimens have different concrete strengths (f'_c = 5.8 to 11.6 ksi (40 to 80 MPa)), effective depths (d = 13.8 to 37.4 in. (350 to 950 mm)), shear span to depth ratios (a/d = 2.0 to 4.0), and levels of prestress force (f_{pe} = 0 to 1.7 ksi (0 to 12 MPa)). Note that f_{pe} is compressive stress in concrete due to effective prestress at the extreme tension fiber.

The specimens with different effective depths are shown in Figure B.1. The researchers studied the influence of size effect on the shear strength of prestressed concrete members without shear reinforcement by testing these specimens.

On the basis of their experimental results, the researchers clarified that the shear strength at the formation of the flexural shear crack was highly increased due to prestress force. An increase in the shear strength due to prestress force was more notable than that caused by a use of high strength concrete. The researchers reported that there was no additional increase in the shear strength at the formation of the flexural shear crack by using high strength concrete of larger than 8.7 ksi (60 MPa). These facts imply that introduction of prestress force is more effective to enhance the shear strength than a use of high strength concrete.

Additionally, the researchers revealed that the shear strength at the formation of the flexural shear crack decreased as the effective depth increased. This trend indicates that size effect affects the shear strength at the formation of the flexural shear crack in prestressed concrete members without shear reinforcement. However, the researchers emphasized that the increase in the shear strength due to prestress force was unrelated to the effective depth of specimens. In other words, there is no influence of size effect on the increase in the shear strength due to prestress force.

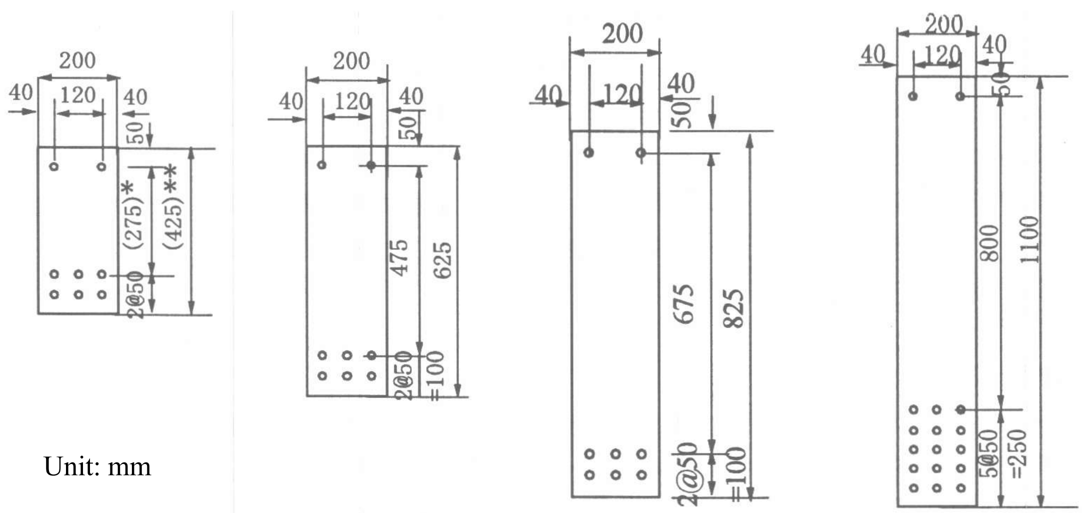


Figure B.1 Cross section of specimens with different effective depths
(Adopted from Watanabe, et al. (1995))

B.1.2 Funakoshi, et al. (1979, 1981, 1982, 1984)

Funakoshi, et al. (1979, 1981, 1982, 1984) tested more than 80 post-tensioned concrete beams with high strength concrete up to 16 ksi (110 MPa). All specimens have I-shaped cross sections and member heights of less than 12 in. (305 mm).

The researchers reported that most of the specimens failed in web crushing or shear compression failures. In particular, specimens with concrete strengths of larger than 12 ksi (80 MPa) were likely to fail in explosive manners. The researchers also indicated that the concrete strength affected the shear strength of specimens that had relatively small shear span to depth ratios. An increase in the concrete strength contributed to enhance the shear strength. In contrast, shear reinforcement was not fully effective in specimens that had relatively small shear span to depth ratios.

B.2 LIGHTWEIGHT CONCRETE

B.2.1 Tamura, et al. (2001), Hamada, et al. (1999)

This research group investigated the shear strength of prestressed concrete members built with lightweight concrete. Tamura, et al. (2001) conducted loading tests of four large pretensioned concrete beams that had an effective depth of 39 in. (1,000 mm) as shown in Figure B.2. All specimens contained no shear reinforcement. The researchers compared the measured shear strength with the calculated shear strengths with the JSCE specifications and the strut-and-tie model.

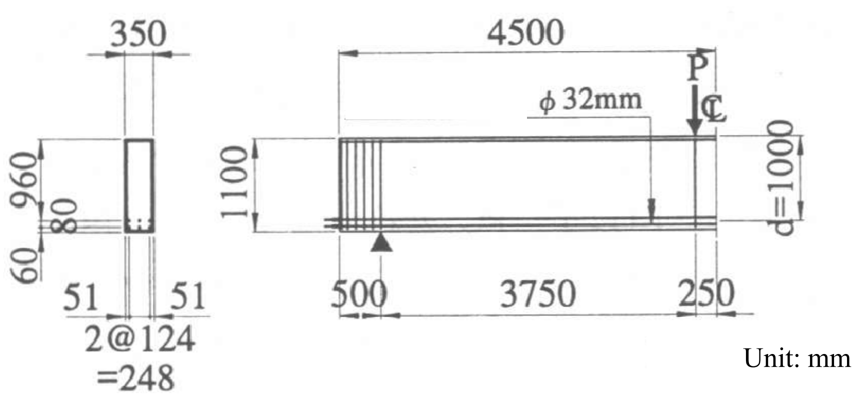


Figure B.2 Details of specimen of Tamura, et al. (2001)
(Adopted from Tamura, et al. (2001))

On the basis of their experimental results, the researchers concluded that there was no need to reduce the V_c term in prestressed concrete members with lightweight concrete when the members were supposed to fail in shear compression failure. The researchers recognized that the tensile strength of lightweight concrete was lower than that of normal concrete, and the concrete compressive strength had more dominant effects on the shear strength of specimens that failed in shear compressive failure. Moreover, the researchers indicated that the JSCE shear design equation and the strut-and-tie model were capable of providing reasonable estimations of the shear strength of the four specimens.

B.3 EFFECT OF PRESTRESS FORCE

B.3.1 De Silva, et al. (2006)

De Siva, et al. (2006) investigated the influence of prestress force, concrete cover, shear reinforcement ratio, and flexural reinforcement ratio on the shear cracking behavior by testing six reinforced and partially prestressed concrete beams shown in Figure B.3.

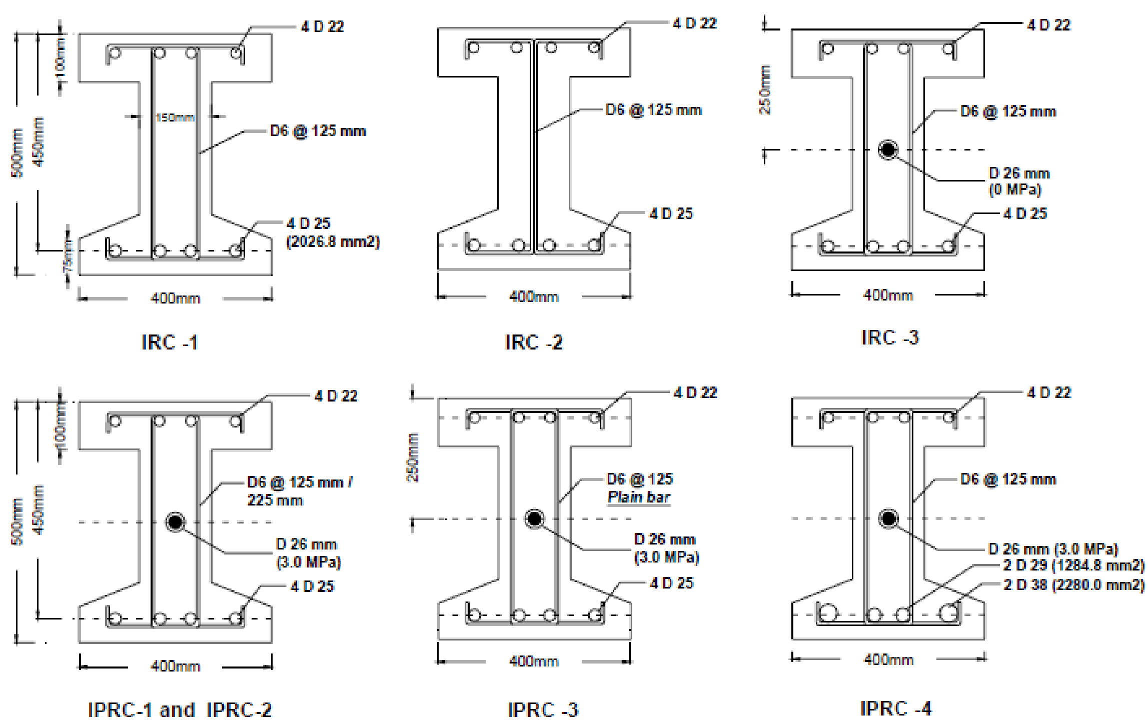


Figure B.3 Details of specimens of De Silva, et al. (2006)

(Adopted from De Silva, et al. (2006))

The researchers revealed that prestress force contributed to delay the formation of inclined cracks. However, the effect of prestress force was limited before the formation of the inclined cracks because there was no clear difference in the width of the inclined cracks between the reinforced and partially prestressed concrete beams after the formation of the inclined cracks.

B.3.2 Mikata, et al. (2001)

Mikata, et al. (2001) also investigated the effect of prestress force on the shear strength. The researchers conducted loading tests of post-tensioned concrete beams that had member heights of less than 12in. These specimens have three types of distributions of prestress force across the cross section: (1) triangle; (2) trapezoid; and (3) rectangle. The triangle distribution is that specimens have compressive stress at the extreme tension fiber but zero stress at the extreme compression fiber. The trapezoidal distribution is that specimens have compressive stress at both extreme fibers but compressive stress at the extreme tension fiber is larger than that at the extreme compression fiber. The rectangular distribution is a uniform compression stress across the cross section.

The researchers concluded that the distribution of prestress force highly affect the shear strength of prestress concrete members. Their experimental results showed that the shear strength increased as compression stress at the extreme tension fiber increased. The shear strength at the formation of the inclined cracks in specimens with the trapezoidal and rectangular distributions is larger than that in specimens with the triangle distribution. The angle of the inclined crack became lower in specimens with the trapezoidal and rectangular distributions and with large prestress force at the extreme tension fiber.

Additionally, the researchers proposed revisions to the JSCE specifications on the basis of the MCFT. Their revisions intended to predict the angle of the inclined crack and the effect of prestress force on the shear strength.

B.3.3 Sato, et al. (1987)

Sato, et al. (1987) also studied the effect of prestress force on the shear strength by conducting loading tests of 51 post-tensioned concrete beams. 27 of the 51 specimens contained shear reinforcement, and 20 of the 51 specimens had draped prestressing steel. Figure B.4 shows the details of the specimens.

The researchers concluded that although the JSCE shear design equation produced conservative estimations, there was still a room for improvement in the accuracy. The method adopting the decompression moment to account for the effect of prestress force in the JSCE specifications was found to produce scattered estimations.

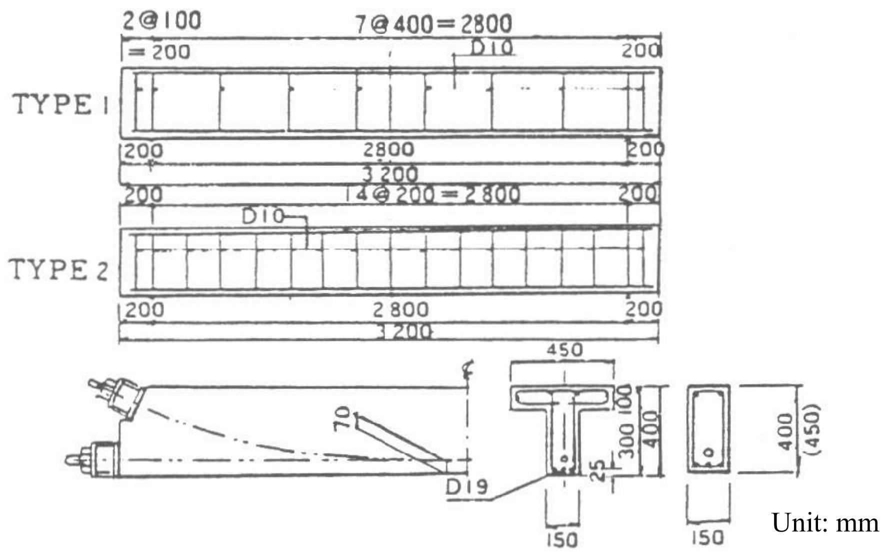


Figure B.4 Details of specimens of Sato, et al. (1987)

(Adopted from Sato, et al. (1987))

B.4 EXTERNALLY POST-TENSIONED PRESTRESSED CONCRETE MEMBERS

B.4.2 Sivaleepunth, et al. (2007)

Sivaleepunth, et al. (2007) conducted loading tests of externally post-tensioned concrete beams, and proposed the finite element model to predict the shear strength of externally post-tensioned concrete beams. On the basis of their analysis results, the proposed model is capable of providing more accurate estimations of the shear strength on externally post-tensioned concrete beams than the JSCE shear design equation.

B.4.2 Mitamura, et al. (2001)

Mitamura, et al. (2001) conducted loading tests of externally post-tensioned concrete beams fabricated with lightweight concrete. The experimental results showed that the shear strength was highly increased due to prestress force that was introduced by external cables. The researchers indicated that the increase of the shear strength in externally post-tensioned concrete beams was caused mainly by the arch action that was developed by the external cables and the diagonal concrete strut.

B.4.3 Takagi, et al. (2000)

Takagi, et al. (2000) focused on the effect of bond between concrete and prestressing steel on the shear strength of prestressed concrete members. The researchers conducted loading tests of beams prestressed by inner cables and external cables. The beams with the external cables have no bond between concrete and external cables. The details of the specimens are presented Figure B.5.

The failure mode was completely different between beams with the internal cables and those with the external cables. The beams with the internal cables failed in shear failure, while the beams with the external cables failed in flexural failure. The researchers indicated that these different failure modes were derived from a difference in the bond condition between the two types of cables. In the beams with the external cables, there was no bond between the concrete and the external cables, and this resulted in a development of the arch action that contributed to increase the shear strength. As a result, the beams with the external cables failed in flexural shear. This explanation agrees with the test results presented by Mitamura, et al. (2001).

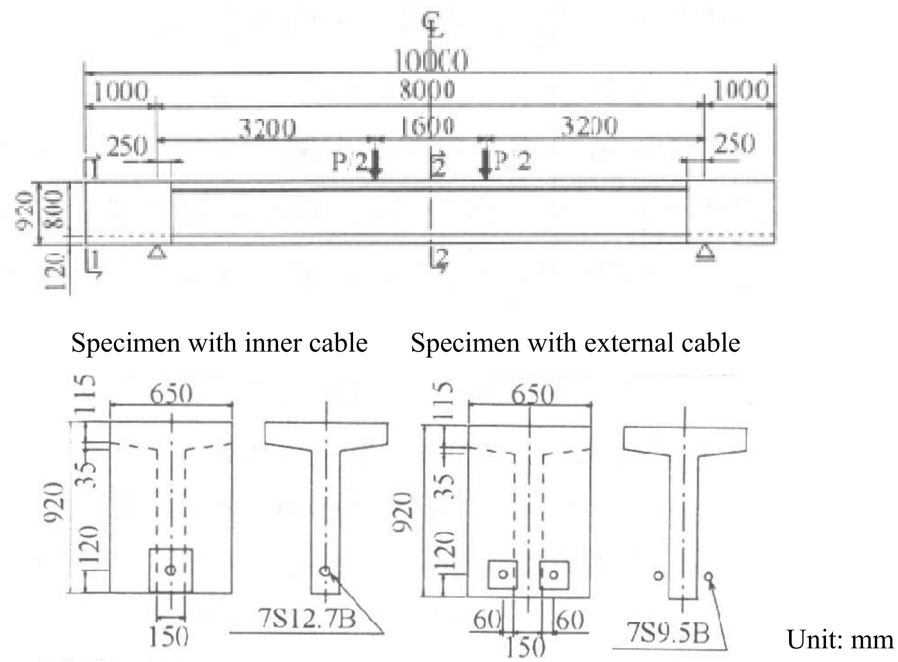


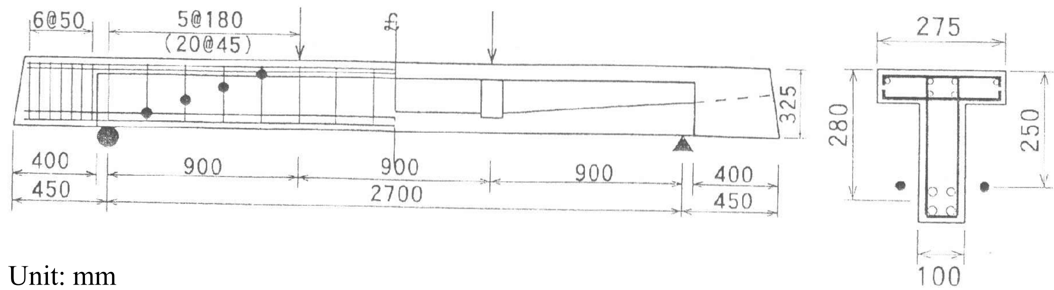
Figure B.5 Details of specimens of Takagi, et al. (2000)

(Adopted from Takagi, et al. (2000))

B.4.4 Niitsu, et al. (1999), Kondo, et al. (1994)

This research groups investigated the effect of external post-tensioning as one of the strengthening methods for reinforced concrete members. Kondo, et al. (1994) tested both reinforced and externally post-tensioned concrete beams. The details of the specimens are shown in Figure B.6.

The experimental results showed that the external post-tensioning contributed to increase the shear strength of reinforced concrete beams. Additionally, the researchers reported that the JSCE shear design equation provides reasonable estimations of the increased shear strength.



Unit: mm

Figure B.6 Details of specimens of Kondo, et al. (1994)
(Adopted from Kondo, et al. (1994))

B.5 SEGMENTAL POST-TENSIONED PRESTRESSED CONCRETE MEMBERS

B.5.1 Nguyen, et al. (2010), Sivaleepunth, et al. (2009)

This research groups proposed the finite element models to predict the shear strength of segmental post-tensioned concrete beams and segmental externally post-tensioned concrete beams. The researchers also conducted loading tests of those beams and verified the accuracy of their models.

B.5.2 Hosoda, et al. (2002), Ito, et al. (1997)

Hosoda, et al. (2002) and Ito, et al. (1997) conducted loading tests of segmental post-tensioned concrete beams and segmental externally post-tensioned concrete beams. The researchers revealed that the external cables contributed to develop the arch action in the segmental beams and increase the shear strength of the specimens.

REFERENCES

1. AASHTO, Guide Specifications for Design and Construction of Segmental Concrete Bridges, 2nd Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 1999.

2. AASHTO, Guide Specifications for Design and Construction of Segmental Concrete Bridges, 2nd Edition, 2003 Interim, American Association of State Highway and Transportation Officials, Washington, D.C., 2003.

3. AASHTO, Standard Specifications for Highway Bridges, 17th Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 2002.

4. AASHTO, LRFD Bridge Design Specifications, 4th Edition, American Association of State Highway and Transportation Officials, Washington, D. C., 2007.

5. AASHTO, LRFD Bridge Design Specifications, 4th Edition, 2008 Interim, American Association of State Highway and Transportation Officials, Washington, D. C., 2008.

6. AASHTO, LRFD Bridge Design Specifications, 5th Edition, American Association of State Highway and Transportation Officials, Washington, D. C., 2010.

7. ACI Committee 318, Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary, American Concrete Institute, Farmington Hills, MI, 2008.

8. ACI Committee 318, Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-63), American Concrete Institute SP-10, American Concrete Institute, Farmington Hills, MI, pp.78-84, 1965.

9. ACI-ASCE Commettee 326, Shear and Diagonal Tension, ACI Journal, Vol.59, No.1, January, pp.1-30, 1962, Vol.59, No.2, February, pp.277-333, 1962, Vol.59, No.3 March, pp.353-359, 1962.

10. Alshegeir, A., and Ramirez, J. A., Strut-Tie Aapproach in Prestressed Deep Beams, ACI Structural Journal, Vol.89, No.3, May-June, pp.296-304, 1992.

11. Arthur, P. D., The Shear Strength of Pre-Tensioned I Beams with Unreinforced Webs, Magazine of Concrete Research, Vol.17, No.53, pp.199-210, 1965.

12. Arthur, P. D., Bhatt, P., and Duncan, W., Experimental and Analytical Studies on the Shear Failure of Pretensioned I-Beams under Distributed Loading, PCI Journal, Vol.18, No.1, pp.50-67, 1973.

13. Avendaño, A. R., and Bayrak, O., Shear Strength and Behavior of Prestressed Concrete Beams, Technical Report: IAC-88-5DD1A003-3, Center for Transportation Research, the University of Texas at Austin, 2008.

14. Bennett, E. W., and Balasooriya, B. M. A., Shear Strength of Prestressed Beams with Thin Webs Failing in Inclined Compression, ACI Journal, Vol.68, No.3, March, pp.204-212, 1971.

15. Bennett, E. W., and Debaiky, S. Y., High Strength Steel as Shear Reinforcement in Prestressed Concrete Beams, American Concrete Institute SP-42, Shear in Reinforced Concrete, American Concrete Institute, Farmington Hills, MI, pp. 231-248, 1974.

16. Bentz, E. C., and Collins, M. P., Development of the 2004 CSA A23.3 Shear Provisions for Reinforced Concrete, Canadian Journal of Civil Engineering, NRC Research Press, Canada, Vol.33, No.5, pp.521-534, 2006a.

17. Bentz, E. C., Vecchio, F. J., Collins, M. P., The Simplified MCFT for Calculating the Shear Strength of Reinforced Concrete Elements, ACI Structural Journal, Vol.103, No.4, July-August, pp.614-624, 2006b.

18. Birrcher, D., Tuchscherer, R., Huizinga, M., Bayrak, O., Wood, S., and Jirsa, J. O., Strength and Serviceability Design of Reinforced Concrete Deep Beams, FHWA/TX-09/0-5253-1, Report No.0-5253-1, Center for Transportation Research, The University of Texas at Austin, 2009.

19. Brown, M. D., Bayrak, O., and Jirsa, J. O., Design for Shear Based on Loading Conditions, ACI Structural Journal, Vol.103, No.4, July-August, pp.541-550, 2006.

20. Bruce, R. N., An Experimental Study of the Action of Web Reinforcement in Prestressed Concrete Beams, Ph.D. Dissertation, University of Illinois, 1962.

21. Bruggeling, A. S. G., Brunekreef, S. H., and Walraven, J. C., Tests on Partially Prestressed T-Beams Subjected to Shear and Bending, Partially Prestressed Concrete, Theory and Experiments, Delft University of Technology, pp.18-35, 1978.

22. Burgueño, R., and Bendert, D. A., Experimental Evaluation and Field Monitoring of Prestressed Box Beams for SCC Demonstration Bridge, Research Report RCC-1489, Michigan State University, 2007.

23. CEB, CEB-FIP Model Code for Concrete Structures, CEB Bulletin, No.125E, 1978.

24. CEB, CEB-FIP Model Code 1990, Thomas Telford, London, United Kingdom, 1991.

25. Cederwall, K., Hedman, O., and Loeberg, A., Shear Strength of Partially Prestressed Beams with Pretensioned Reinforcement of High Grade Deformed Bars, American Concrete Institute SP-42, Shear in Reinforced Concrete, American Concrete Institute, Farmington Hills, MI, pp. 215-230, 1974.

26. Choulli, Y., Mari, A. R., and Cladera, A., Shear Behaviour of Full-Scale Prestressed I-Beams Made with Self Compacting Concrete, Materials and Structures, Vol.41, No.1, pp.131-141, 2008.

27. Cladera, A., Marí, A. R., Shear Strength in the New Eurocode 2. A Step Forward?, Structural Concrete, Journal of the fib, Vol.8, No.2, pp.57-66, 2007.

28. Collins, M. P., and Mitchell, D., Prestressed Concrete Structures, Response Publications, Canada, 1997.

29. Collins, M. P., Bentz, E. C., and Sherwood E. G., Where is Shear Reinforcement Required? Review of Research Results and Design Procedures, ACI Structural Journal, Vol.105, No.5, September-October, pp.590-600, 2008.

30. CSA, CSA-A23.3-04 Design of Concrete Structures, A National Standard of Canada (approved July 2007) (Reaffirmed 2010), Canadian Standards Association, Canada, 2010.

31. Cumming, D. A., Shield, C. K., and French, C. E., Shear Capacity of High-Strength Concrete Pre-Stressed Girders, MN/RC-1998/12, Center for Transportation Studies, University of Minnesota, 1998.

32. De Silva, S., Mutsuyoshi, H., Witchukreangkrai, E., and Takagi, M., Experimental Study on Shear Cracking Behaviour in I-shaped Partially Prestressed Concrete Beams, Transactions of the Japan Concrete Institute, Vol.28, No.2, pp. 817-822, 2006.

33. Dill, J. C., Development Length of 0.6-inch Diameter Prestressing Strand in High-Performance Concrete, Master Thesis, Georgia Institute of Technology, 2000.

34. Durrani, A. J., and Robertson, I. N., Shear Strength of Prestressed Concrete T Beams with Welded Wire Fabric as Shear Reinforcement, PCI Journal, Vol.32, No.2, March-April, pp.46-61, 1987.

35. Dymond, B. Z., Shear Strength of a PCBT-53 Girder Fabricated with Lightweight, Self-Consolidating Concrete, Master Thesis, Virginia Polytechnic Institute and State University, 2007.

36. Elzanaty, A. H, Nilson, A. H., and Slate, F. O., Shear Capacity of Prestressed Cocnrete Beams Using High-Strength Concrete, ACI Structural Journal, Vol.83, No.2, March-April, pp.359-368, 1987.

37. European Committee for Standardization, Eurocode 2: Design of Concrete Structures – Part 1-1: General Rules and Rules for Buildings, BS EN 1992-1-1:2004, 2004.

38. Evans, R. H., and Schumacher, E. G., Shear Strength of Prestressed Beams without Web Reinforcement, Journal of the American Concrete Institute, Vol.60, No.11, November, pp.1621-1642, 1963.

39. fib, Model Code 2010, First Complete Draft, Vol.1, fib Bulletin No.55, 2010a.

40. fib, Model Code 2010, First Complete Draft, Vol.2, fib Bulletin No.56, 2010b.

41. Funakoshi, M., and Okamoto, A., Fundamenal Study on Diagonal Compression Strength of Web in Prestressed Concrete Beam, Transactions of the Japan Concrete Institute, Vol.1, pp.325-328, 1979 (in Japanese).

42. Funakoshi, M., and Okamoto, A., Experimental Study on Ultimate Strength of Prestressed Concrete Beams with High Strength Concrete, Transactions of the Japan Concrete Institute, Vol.3, pp.365-368, 1981 (in Japanese).

43. Funakoshi, M., Okamoto, A., and Tanaka, E., Study on Resisting Behavior of Prestressed Concrete Beams with High Strength Concrete, Transactions of the Japan Concrete Institute, Vol.4, pp.297-300, 1982 (in Japanese).

44. Funakoshi, M., Tanaka, H., and Tani, M., Strength and Failure Mode of Prestressed Concrete Beams with High Strength Cocrete, Transactions of the Japan Concrete Institute, Vol.6, pp.469-472, 1984 (in Japanese).

45. Gregor, T., and Collins, M. P., Tests of Large Partially Prestressed Concrete Girders, ACI Structural Journal, Vol.92, No.1, January-February, pp.63-72, 1995.

46. Haines, R. A., Shear Testing of Prestressed High Performance Concrete Bridge Girders, Master Thesis, Georgia Institute of Technology, 2005.

47. Hamada, Y., Tamura, H., Maehori, S., and Niwa, J., Shear Strength of Prestressed Concrete Beams with Lightweight Concrete, Proceedings of the Japan Prestressed Concrete Engineering Association, Vol.9, pp.739-744, 1999 (in Japanese).

48. Hamilton III, H. R., Llanos, G., and Ross, B. E., Shear Performance of Existing Prestressed Concrete Bridge Girders, Final Report, BD 545-56, University of Florida, 2009.

49. Hanson, J. M., and Hulsbos, C. L., Ultimate Shear Tests of Prestressed Concrete I-Beams under Concentrated and Uniformed Loadings, PCI Journal, Vol.9, No.3, June, pp.15-28, 1964.

50. Hanson, J. M., and Hulsbos, C. L., Ultimate Shear Tests of Large Prestressed Concrete Bridge Beams, American Concrete Institute SP-26, Concrete Bridge Design, American Concrete Institute, Farmington Hills, MI, pp.523-549, 1969.

51. Hartman, D. L., Breen, J. E., and Kreger, M. E., Shear Capcity of High Strength Prestressed Concrete Girders, Research Report 381-2, FHW/TX-88+381-2, Center of Transportation Research, The University of Texas at Austin, 1988.

52. Hawkins, N. M., and Kuchma, D. A., Application of LRFD Bridge Design Specifications to High-Strength Structural Concrete: Shear Provisions, NCHRP Report 579, Transportation Research Board, National Research Council, Washington, D.C., 2007.

53. Hawkins, N. M., Kuchma, D. A., Mast, R. F., Marsh, M. L., and Reineck, K. H., Simplified Shear Design of Structural Concrete Members, NCHRP Report 549, Transportation Research Board, National Research Council, Washington, D.C., 2005.

54. Hawkins, N. M., Sozen, M. A., and Siess, C. P., Strength and Behavior of Two-Span Continuous Prestressed Concrete Beams, Civil Engineering Studies, Structural Research Series No.225, University of Illinois, 1961.

55. Heckmann, C. P., Effects of Increasing the Allowable Compressive Stress at Release on the Shear Strength of Prestressed Concrete Girders, Master Thesis, The University of Texas at Austin, 2008.

56. Hernandez, G, Strength of Prestressed Concrete Beams with Web Reinforcement, Civil Engineering Studies, Structural Research Series No.153, University of Illinois, 1958.

57. Hicks, A. B., The Influence of Shear Span and Concrete Strength upon the Shear Resistance of a Pre-Tensioned Prestressed Concrete Beam, Magazine of Concrete Research, Vol.10, No.30, pp.115-122, 1958.

58. Horibe, K., and Ueda, T., Effect of Axial Force on Shear Behavior of Reinforced Concrete Beams, Proceedings of Annual Conference of the Japan Society of Civil Engineers, Vol.40, No.5, pp.319-320, 1986 (in Japanese).

59. Hosoda, A., Kobayashi, K., and Kino, J., Shear Failure Mode of Segmental Prestressed Concrete Beams with External Cables, Transactions of the Japan Concrete Institute, Vol.24, No.2, pp. 589-594, 2002 (in Japanese).

60. Hovell, C., Avendaño, A., Dunkman, D., Moore, A., Bayrak, O., and Jirsa, J., Behavior of Texas U-Beams and Box-Beams at Prestress Transfer and Under Shear Loads, TX-10/0-5831-1, Center of Transportation Research, The University of Texas at Austin, 2010.

61. Iguro, M., Shioya, T., Nojiri, Y., and Akiyama, H., Experimental Studies on the Shear Strength of Large Reinforced Concrete Beams under Uniformly Distributed Load, Journal of Japan Society of Civil Engineers, No.348/V-1, pp.175-184, 1984 (in Japanese).

62. Imano, H., Ikeda, K., Kishi, N., and Kakemoto, S., Loading Tests of Large Prestressed Reinforced Concrete Beams with Different Shear Safety Margin, Transactions of the Japan Concrete Institute, Vol.23, No.3, pp.631-636, 2001 (in Japanese).

63. Ito, T., Suzuki, Y., Yamaguchi, T., and Ikeda, N., Shear Resisting Behavioir of Prestressed Concrete Beams with Artificial Lightweight Aggregate, Transactions of the Japan Concrete Institute, Vol.18, No.2, pp.1133-1138, 1996 (in Japanese).

64. Ito, T., Yamaguchi, T., and Ikeda, S., Experimental Study on Flexural Shear Behavior of Segmental Precast Beams, Journal of Prestressed Concrete, Vol.39, No.1, pp.83-96, 1997 (in Japanese).

65. Japan Road Association, Specifications for Highway Bridges Part III Concrete Bridges, Japan Road Association, 2002.

66. Japan Society of Civil Engineers, Structural Specifications for Concrete Structures 2007 "Design", Japan Society of Civil Engineers, 2007 (in Japanese).

67. Kang, G. Y., Wu, Z. M., Wang, L. Y., and Xue, B. H., Ultimate Shear Tests of Prestressed Concrete I-Beams, Proceedings of the Institution of Civil Engineers, Part 2, Vol.87, No.9, pp.401-414, 1989.

68. Kar, J. N., Shear Strength of Prestressed Concrete Beams without Web Reinforcement, Magazine of Concrete Research, Vol.21, No.68, pp.159-170, 1969.

69. Kaufman, M. K., and Ramirez, J. A., Re-evaluation of the Ultimate Shear Behavior of High-Strength Concrete Prestressed I-Beams, ACI Structural Journal, Vol.85, No.3, May-June, pp.295-303, 1987.

70. Kobayashi, K., and Nieda, T., Study on Shear Resisting Behavior of Partially Prestressed Concrete Beams, Proceedings of Annual Conference of the Japan Society of Civil Engineers, Vol.46, No.5, pp.656-657, 1991 (in Japanese).

71. Kondo, E., Mutsuyoshi, H., Takahashi, H., and Sano, M., Experimental Study on Shear Strengthening Effect by External Cables, Transactions of the Japan Concrete Institute, Vol.16, No.2, pp.1015-1020, 1994 (in Japanese).

72. Kuroda, I., Shimoyama, K., Furuya, N., and Nakamura, S., Static and Rapid Speed Shear Loading Experiment on PC Beam Members, Journal of Structural Engineering, Vol.47A, pp.1299-1308, 2001 (in Japanese).

73. Labonte, T., and Hamilton III, H. R., Self-Consolidating Concrete (SCC) Structural Investigation, Final Report, DB545, PRWO#21, University of Florida, 2005.

74. Laskar, A., Wang, J., Hsu, T. T. C., and Mo, Y. L., Rational Shear Provisions for AASHTO LRFD Specifications: Technical Report, FHWA/TX-07/0-4759-1, University of Houston, 2007.

75. Lee, S. C., Cho, J. Y., and Oh, B. H., Shear Behavior of Large-Scale Post-Tensioned Girders with Small Shear Span-Depth Ratio, ACI Structural Journal, Vol.107, No.2, March-April, pp.137-145, 2010.

76. Lin, T. Y., Strength of Continuous Prestressed Concrete Beams Under Static and Repeated Loads, Journal of the American Concrete Institute, Vol.51, No.6, June, pp.1037-1059, 1955.

77. Lyngberg, B. S., Ultimate Shear Resistance of Partially Prestressed Reinforced Concrete I-Beams, ACI Journal, Vol.73, No.4, April, pp.214-222, 1976.

78. Losberg, A., and Hedman, O., Design of Concrete Structures with regard to Shear Forces, CEB Bulletin, No.126, Shear and Torsion, 1978.

79. Ma, Z., Tadros, M. K, and Baishya, M., Shear Behavior of Pretensioned High Strength Concrete Bridge I-Girders, ACI Structural Journal, Vol.97, No.1, January-February, pp.185-192, 2000.

80. MacGregor, J. G., and Hanson, J. M., Proposed Changes in Shear Provisions for Reinforced and Prestressed Concrete Beams, ACI Journal, Vol.66, No.4, pp.276-288, 1969.

81. MacGregor, J. G., Sozen, M. A., and Siess, C. P., Effect of Draped Reinforcement on Behavior of Prestressed Concrete Beams, Journal of the American Concrete Institute, Vol.57, No.12, December, pp.649-677, 1960a.

82. MacGregor, J. G., Sozen, M. A., and Siess, C. P., Strength and Behavior of Prestressed Concrete Beams with Web Reinforcement, Civil Engineering Studies, Structural Research Series No.201, University of Illinois, 1960b.

83. Magnel, G., Prestressed Concrete, Third Edition, McGraw-Hill Book Company, Inc., New York, Concrete Publications Limited, London, pp.180-188, 1954.

84. Mahgoub, M. O., Shear Strength of Prestressed Concrete Beams without Web Reinforcement, Magazine of Concrete Research, Vol.27, No.93, pp.219-228, 1975.

85. Maruyama, K., and Rizkalla, S. H., Shear Design Consideration for Pretensioned Prestressed Beams, ACI Structural Journal, Vol.85, No.5, September-October, pp.492-498, 1988.

86. Mattock, A. H. and Kaar, P. H., Precast-Prestressed Concrete Bridges, 4. Shear Tests of Continuous Girders, Journal of the PCA Research and Development Laboratories, Vol.3, No.1, pp.19-46, 1961.

87. Meyer, K. F., Transfer and Development Length of 0.6-inch Diameter Prestressing Strand in High Lightweight Concrete, Ph.D. Dissertation, Georgia Institute of Technology, 2002.

88. Mikata, Y., Inoue, S., Kobayashi, K., and Nieda, T., Effect of Prestress on Shear Strength of Prestressed Concrete Beams, Journal of Japan Society of Civil Engineers, Vol.669/V-50, pp.149-159, 2001 (in Japanese).

89. Mitamura, H., Ikeda, K., Hishiki, Y., and Fukuda, I., Experiment on Shear Strength of Prestressed Concrete Beams with External Cable and High Performance Lightweight Concrete, Transactions of the Japan Concrete Institute, Vol.23, No.3, pp.619-624, 2001 (in Japanese).

90. Moayer, M., and Regan, P. E., Shear Strength of Prestressed and Reinforced Concrete T-Beams, American Concrete Institute SP-42, Shear in Reinforced Concrete, American Concrete Institute, Farmington Hills, MI, pp. 183-213, 1974.

91. Morice, P. B., and Lewis, H. S., The Ultimate Strength of Two-Span Continuous Prestressed Concrete Beams as Affected by Tendon Transformation and Un-Tensioned Steel, Second Congress of the Federation Internationale de la Precontrainte, Amsterdam, pp.561-584, 1955.

92. Muguruma, A., Watanabe, F., and Fujii, M., Experimental Study on Shear Resisting Behavior of Prestressed Reinforced Concrete Beams, Transactions of the Japan Concrete Institute, Vol.5, pp.453-456, 1983.

93. Naito, C., Brunn, G., Parent, G., and Tate, T., Comparative Performance of High Early Strength and Self Consolidating Concrete for Use in Precast Bridge Beam Construction, ATLSS Report No.05-03, Lehigh University, 2005.

94. Nakamura, E., Takeuchi, Y., Aoyama, H., Murakoshi, J., and Kimura, Y., Study on Shear Resisting Behavior of Corroded Prestressed Concrete Beams, Proceedings of the Concrete Structure Scenarios, Vol.9, pp.411-416, 2009 (in Japanese).

95. Nguyen, D. H., Watanabe, K., Niwa, J., and Hasegawa, T., Modified Model for Shear Carrying Capacity of Segmental Concrete Beams with External Tendons, Journal of Japan Society of Civil Engineers E, Vol.66, No.1, pp.53-67, 2010.

96. Niitsu, M., Mutsuyoshi, H., Sano, M., and Tonokawa, K., Study on Shear Strengthening Effect on Reinforced Concrete Beams by External Cables, Transactions of the Japan Concrete Institute, Vol.21, No.3, pp.949-954, 1999 (in Japanese).

97. Niwa, J., Yamada, K., Yokozawa, K., and Okamura, T., Revaluation of the Equation for Shear Strength of Reinforced Concrete Beams without Web Reinforcement, Journal of Japan Society of Civil Engineers, No.372/V-5, pp.167-176, 1986 (in Japanese).

98. Niwa, J., Equation for Shear Strength of Reinforced Concrete Deep Beams based on FEM Analysis, Proceedings of Second Colloquium on Analytical Research on Shear Failure in Reinforced Concrete Structures, Japan Concrete Institute, pp.119-126, 1983 (in Japanese).

99. Nunnally, L., Shear Behavior of Prestressed Self-Consolidating Concrete Bulb-Tee Beams, Master Thesis, University of Virginia, 2005.

100. Okada, K., and Toyofuku, T., Effect of Prestress on Shear Behavior and Strength of Prestrerssed Reinforced Concrete Beams, Transactions of the Japan Prestressed Concrete Engineering Association, Vol.22, pp.16-17, 1982 (in Japanese).

101. Okada, K., Kobayashi, K., and Toyofuku, T., Flexural Shear Behavior of Prestressed Reinforced Concrete Beams, Transactions of the Japan Cement Association, Vol.34, pp.425-428, 1980 (in Japanese).

102. Okamura, H., and Hihai, T., Proposed Design Equation for Shear Strength of Reinforced Concrete Beams without Web Reinforcement, Journal of Japan Society of Civil Engineers, No.300, pp.131-141, 1980.

103. Olesen, S. O., Sozen, M. A., and Siess, C. P., Investigation of Prestressed Concrete for Highway Bridges, Part IV, Strength in Shear of Beams with Web Reinforcement, Engineering Experiment Station Bulletin No.493, University of Illinois, 1967.

104. Ozcebe, G., Ersoy, S., Tankut, T., Evaluation of Minimum Shear Reinforcement Requirements for Higher Strength Concrete, ACI Structural Journal, Vol.96, No.3, May-June, pp.361-369, 1999.

105. Public Works Research Institute, Collaborative Research Report on Design Method of High Strength Concrete Members, Collaborative Research Report No.122, 1995.

106. Ramirez, J. A., and Aguilar, G., Shear Reinforcement Requirements for High-Strength Concrete Bridge Girders, Final Report, FHWA/IN/JTRP-2005/19, Purdue University, 2005.

107. Ramirez, J. A., and Breen, J. E., Review of Design Procedures for Shear and Torsion in Reinforced and Prestressed Concrete, FHWA/TX-84/36+248-2, Research Report No.248-2, Center for Transportation Research, The University of Texas at Austin, 1983a.

108. Ramirez, J. A., and Breen, J. E., Experimental Verification of Design Procedures for Shear and Torsion in Reinforced and Prestressed Concrete, FHWA/TX-84/37+248-3, Research Report No.248-3, Center for Transportation Research, The University of Texas at Austin, 1983b.

109. Ramirez, J. A., and Breen, J. E., Proposed Design Procedures for Shear and Torsion in Reinforced and Prestressed Concrete, FHWA/TX-84/38+248-4F, Research Report No.248-4F, Center for Transportation Research, The University of Texas at Austin, 1983c.

110. Ramirez, J. A., and Breen, J. E., Evaluation of a Modified Truss-Model Approach for Beams in Shear, ACI Structural Journal, Vol.88, No.5, September-October, pp.562-571, 1991.

111. Ramirez, J., Olek, J., Rolle, E., and Malone, B., Performance of Bridge Decks and Girders with Lightweight Aggregate Concrete, Final Report, FHWA/IN/JTRP-98/17, Purdue University, 2000.

112. Rangan, B. V., Web Crushing Strength of Reinforced and Prestressed Concrete Beams, ACI Structural Journal, Vol.88, No.1, January-February, pp.12-16, 1991.

113. Raymond, K. K, Bruce, R. N., and Roller, J. J., Shear Behavior of HPC Bulb-Tee Girders, American Concrete Institute SP-228, Seventh International Symposium on the Utilization of High-Strength/High-Performance Concrete, pp.705-722, 2005.

114. Reineck, K. H., Kuchma, D. A., Kim, K. S., and Marx, S., Shear Database for Reinforced Concrete Members without Shear Reinforcement, ACI Structural Journal, Vol.100, No.2, March-April, pp.240-249, 2003.

115. Roller, J. J., and Russell, H. G., Shear Strength of High-Strength Concrete Beams with Web Reinforcement, ACI Structural Journal, Vol.87, No.2, March-April, pp.191-198, 1990.

116. Runzell, B., Shield, C., and French, C., Shear Capacity of Prestressed Concrete Beams, Final Report, MN/RC 2007-47, University of Minnesota, 2007.

117. Sakurada, M., Suzuki, M., Watanabe, H., and Murai, N., Loading Tests of Prestressed Concrete Beams with High Performance Aggregate of Coal Ash, Transactions of the Japan Concrete Institute, Vol.23, No.3, pp.559-564, 2001 (in Japanese).

118. Saqan, E. I., and Frosch, R. J., Influence of Flexural Reinforcement on Shear Strength of Prestressed Concrete Beams, ACI Structural Journal, Vol.106, No.1, January-February, pp.48-62, 2009.

119. Sato, T., Ishibashi, T.,Yamashita, Y., and Takada, S., Shear Strength and Failure Mode of Prestressed Concrete Beams, Transactions of the Japan Concrete Institute, Vol.9, No.2, pp.323-328, 1987 (in Japanese).

120. Sethunarayanan, R., Ultimate Strength of Pre-Tensioned I Beams in Combined Beanding and Shear, Magazine of Concrete Research, Vol.12, No.35, pp.83-90, 1960.

121. Shahawy, M. A., and Batchelor, B., deV., Shear Behavior of Full-Scale Prestressed Concrete Girders: Comparison between AASHTO Specifications and LRFD Code, PCI Journal, Vol.41, No.3, May-June, pp.48-62, 1996.

122. Shioya, T., Shear Properties of Large Reinforced Concrete Member, Special Report of Institute of Technology, Simizu Corporation, No.25, 1989 (in Japanese).

123. Sivaleepucth, C., Niwa, J., Nguyen, D. H., Hasegawa, T., and Hamada, Y., Shear Carrying Capacity of Segmental Prestressed Concrete Beams, Journal of Japan Society of Civil Engineers E, Vol.65, No.1, pp.63-75, 2009.

124. Sivaleepunth, C., Niwa, J., Diep, B. K., Tamura, S., and Hamada, Y., Simplified Truss Model for Externally Prestressed Concrete Beams, Journal of Japan Society of Civil Engineers, Vol.63, No.4, pp.562-574, 2007.

125. Sozen, M. A., and Hawkins, N. M., Discussion of Shear and Diagonal Tension by ACI-ASCE Committee 326, ACI Journal, Vol.59, No.9, September, pp.1341-1347, 1962.

126. Sozen, M. A., Zwoyer, E. M, and Siess, C. P., Investigation of Prestressed Concrete for Highway Bridges, Part I, Strength in Shear of Beams without Web Reinforcement, Engineering Experiment Station Bulletin No.452, University of Illinois, 1959.

127. Takagi, Y., Umetsu, K., Taira, Y., and Mizuno, K., Shear Resisting Behavior of Prestressed Concrete Beams with Inner and External Cables, Transactions of the Japan Concrete Institute, Vol.22, No.3, pp.685-690, 2000 (in Japanese).

128. Tamura, H., Hamada, Y., Maehori, S., and Niwa, J., Shear Resisting Behavior of Large Prestressed Concrete Beams with Lightweight Concrete, Transactions of the Japan Concrete Institute, Vol.23, No.3, pp.709-714, 2001 (in Japanese).

129. Tan, K. H., and Mansur, M. A., Partial Prestressing in Concrete Corbels and Deep Beams, ACI Structural Journal, Vol.89, No.3, May-June, pp.116-128, 1992 (in Japanese).

130. Tan, K. H., and Ng, C. K., Partial Prestressing in Concrete Corbels and Deep Beams, ACI Structural Journal, Vol.95, No.2, March-April, pp.116-128, 1998.

131. Tan, K. H., Lu, H. Y., and Teng, S., Size Effect in Large Prestressed Concrete Deep Beams, ACI Structural Journal, Vol.96, No.6, November-December, pp.937-946, 1999.

132. Tawfiq, K. S., Cracking and Shear Capacity of High Strength Concrete Girders, Final Report, FL/DOT/RMC/612(1)-4269, FAMU/FSU College of Engineering, 1995.

133. Teng, S., Kong, F. K., and Poh, S. P., Shear Strength of Reinforced and Prestressed Concrete Deep Beams, Proceedings of the Institution of Civil Engineers, Structures and Buildings, Vol.128, No.2, pp.112-143, 1998.

134. Teoh, B. K., Mansur, M. A., and Wee, T. H., Behavior of High-Strength Concrete I-Beams with Low Shear Reinforcement, ACI Structural Journal, Vol.99, No.3, May-June, pp.299-307, 2002.

135. Toyofuku, T., Study on Shear Bahavior and Strength of Prestressed Reinforced Concrete Beams, Transactions of the Japan Cement Association, Vol.38, pp.456-459, 1984 (in Japanese).

136. Vecchio, F. J., and Collins, M. P., The Modified Compression-Field Theory for Reinforced Concrete Elements Subjected to Shear, ACI Journal, Vol.83, No.2, March-April, pp.219-231, 1986.

137. Walraven, J. C., Shear in Prestressed Concrete Members, CEB Bulletin, No.180, 1987.

138. Watanabe, H., Kawano, H., Funai, Y., Tanaka, M., Study on Flexural Shear Cracking Strength on Pretensioned Concrete Beams with High Strength Concrete, Transactions of the Japan Concrete Institute, Vol.17, No.2, pp.673-678, 1995 (in Japanese).

139. Watanabe, H., Kawano, H., Suzuki, M., and Sato, S., Study on Shear Strength of Prestressed Concrete Beams with High Strength Lightweight Concrete, Proceedings of Concrete Research and Technology, Vol.14, No.1, pp.13-22, 2003 (in Japanese).

140. Xuan, X., Rizkalla, S., and Maruyama, K., Effectiveness of Welded Wire Fabric as Shear Reinforcement in Pretensioned Prestressed Concrete T-Beams, ACI Structural Journal, Vol.85, No.4, July-August, pp.429-436, 1998.

141. Zekaria, I., Shear Failure of Two-Span Continuous Prestressed Concrete Beams without Web Rainforcement, PCI Journal, Vol.3, No.1, June, pp.10-53, 1958.

142. Zwoyer, E. M., and Siess, C. P., Ultimate Strength in Shear of Simply-Supported Prestressed Concrete Beams without Web Reinforcement, Journal of the American Concrete Institute, Vol.51, No.10, October, pp.181-200, 1954.

VITA

Eisuke Nakamura was born in Fukuoka, Japan in 1978. After graduating from Miyako High School in Fukuoka, Japan in March of 1997, he entered Nagoya University in Japan. He graduated with a Bachelor of Engineering in March of 2002. In March of 2004, he also received a Master of Engineering from Nagoya University.

In April of 2004, he joined in the Ministry of Land, Infrastructure, Transport and Tourism of the Government of Japan. He started his career as a researcher at the Public Works Research Institute. In April of 2009, he moved to the National Institute for Land and Infrastructure Management at the Ministry of Land, Infrastructure, Transport and Tourism.

In August of 2009, he enrolled in the graduate program at the University of Texas at Austin. He graduated with a Master of Science in Engineering in May of 2011.

E-mail: eisuke87s@gmail.com

This thesis was typed by the author.