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CORRELATION OF SHEAR DESIGN BETWEEN AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS AND AASHTO GUIDE SPECIFICATIONS FOR THE LRFD SEISMIC BRIDGE DESIGN

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Abstract

This report presents the analytical study of the shear capacity of reinforced concrete columns using both the AASHTO LRFD Bridge Design Specifications and the AASHTO Guide Specifications for the LRFD Seismic Bridge Design. The study investigates various levels of axial load, transverse reinforcement and longitudinal reinforcement to determine how the two specifications compare. The AASHTO Guide Specifications for the LRFD Seismic Bridge Design permits the designer to use the AASHTO LRFD Bridge Design Specifications or equations within the AASHTO Guide Specifications for the LRFD Seismic Bridge Design with predetermined values.

In order to develop a comprehensive understanding of shear behavior, in addition to examining conventional reinforcement, the comparison will also be conducted on high strength concrete and reinforcement. Experimental data is limited for high strength concrete and reinforcement columns, but some test points will be provided to compare with the models.

A database of numerous tested reinforced concrete columns, including conventional and high strength reinforcement, was developed. A list of shear equations was used to predict shear strength, which shows how the concrete and reinforcement shear capacity was predicted using shear equations.

The key parameters investigated are column axial load, amount of transverse and longitudinal reinforcement and deformation ductility demand. The studied equations reveal differences in predicted shear strengths.

A parametrical study was extended to conventional full-scale columns, using both the AASHTO LRFD Bridge Design Specifications and the AASHTO Guide Specifications for the LRFD Seismic Bridge Design to predict shear strength in order to analyze the direct effects of the parameters on the shear strength predictions.

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1 Introduction

1.1 General Perspective

States like Nevada have not officially adopted the AASHTO Guide Specifications for the LRFD Seismic Bridge Design [1], but they use it as part for their design. The specifications were driven by the performance of bridges during the 1989 Loma Prieta and 1994 Northridge earthquakes. In these earthquakes reinforced concrete columns showed vulnerability to inadequate transverse reinforcement for longitudinal confinement and shear reinforcement.

Safe and sustainable transportation infrastructure is one of the key focus areas of Safety and Operations of Large-Area Rural/Urban Intermodal Systems (SOLARIS). In bridge design, shear failure must be prevented. Shear failure is brittle and sudden, therefore it does not permit the redistribution of load within a structure. In previous earthquakes, shear failures have been among the most devastating. The differences between the AASHTO LRFD Bridge Design Specifications [2] and the AASHTO Guide Specifications for the LRFD Seismic Bridge Design need to be understood to ensure a safe shear design. The inclusion of high strength reinforcement enables more sustainable structures through the possibility of more efficient designs.

The AASHTO Guide Specifications for the LRFD Seismic Bridge Design is a displacement based approach that ensures good shear performance by requiring detailed analysis to provide sufficient displacement/ductility capacity. The AASHTO LRFD Bridge Design Specifications provide a force based approach using the elastic seismic forces which are reduced by the Response Modification Factor. Displacements are predicted during the analysis. Designers tend to use the AASHTO LRFD Bridge Design Specifications more frequently due to its simplicity in comparison to the AASHTO Guide Specification for the LRFD Seismic Bridge Design.

There have been many experiments and studies that have focused on the flexural/bending aspects of column design. The number of studies that have investigated shear capacity is substantially less than those for flexure/bending. Those studies have been used to develop shear equations that are ductility based.

Recent column designs by the Nevada Department of Transportation have shown that the transverse reinforcement calculated from the AASHTO Guide Specifications for the LRFD Seismic Bridge Design can be less than what is calculated from the AASHTO LRFD Bridge Design Specifications. This research focuses on how shear strength is predicted by both AASHTO Specifications.

1.2 Objectives and Scope

The primary objective of this research is to investigate the shear capacity of reinforced concrete columns using both the AASHTO LRFD Bridge Design Specifications and the AASHTO Guide Specifications for the LRFD Seismic Bridge Design for various levels of axial load, transverse reinforcement and longitudinal reinforcement to determine how the two specifications compare. The parameters of interest are presented in Section 1.2.1. Chapter 2 is the literature review. The column tests selected for this study are detailed in Chapter 3. Chapter 4 is the parametric study, while Chapter 5 is conclusions.

The second objective is to evaluate previously proposed shear strength predicting equations and compare the results with the shear strength predictions provided by both AASHTO Specifications.

1.2.1 Studied Parameters

The first parameter is the axial load ratio$(P/A_g f'_c)$. P is the axial load, A_g is the gross area of the column section, and f'_c is the compressive strength of concrete that was determined on 28-days cylindrical (6-inch diameter by12-inch height) specimens.

The second parameter is the transverse reinforcement ratio $((A_v/bs)$, A_v is the area of transverse reinforcement within the hoop or spiral spacing, s, taken as $A_v = 2A_{sh}$, A_{sh} is the cross-sectional area of hoop or spiral, b is the width of the column section or the diameter of a circular column and f_y is the yield stress of the transverse reinforcement.

The third parameter is the deformation ductility ($\mu_\Delta = \Delta_u/\Delta_y$). Δ_y is the yield displacement, and Δ_u is the ultimate displacement under lateral force-displacement histories.

The forth parameter is the column aspect ratio (a / d) defined by the ratio of the shear span or length, a, to the effective depth of the column. For cantilever columns, the shear span is equal to the length of column, and for double fixed columns, the shear span is equal to a half of column length.

2 Literature Review

2.1 Introduction

The following section provides a review of various design codes and proposed shear strength predicting equations analyzed in Chapter 4. The equations were chosen to illustrate the difference in predicted shear strengths. Besides the AASHTO LRFD Bridge Design Specifications and the AASHTO Guide Specifications for the LRFD Seismic Bridge Design, the Japan Road Association Specification for Highway Bridges [3], ACI 318-14 [4], Standard New Zealand [5], ASCE-ACI 426 Shear Strength Approach[6] were added to the study. The shear strength equations for each approach are presented in the following sections.

2.2 Japan Road Association Specification for Highway Bridges (March 2002)

The shear strength was calculated using the following equations:

$$P_s = S_c + S_s \tag{2.1}$$

$$S_c = c_c c_e c_{pt} \tau_c bd \tag{2.2}$$

$$S_s = \frac{A_w \sigma_{sy} d(sin\theta + cos\theta)}{1.15a} \tag{2.3}$$

Where:

P_s= shear strength (N)

S_c= contribution of concrete in shear strength (N)

τ_c= average shear stress that can be borne by concrete (N/mm^2). Values are given in Table 2-1

c_c= modification factor on the effects of alternating cyclic loading, taken as 0.6 for Type I Earthquake Ground Motion and 0.8 for Type II

c_e= modification factor in relation to the effective depth of a column section. Values are given in Table 2-2

c_{pt}= modification factor in relation to the axial tensile reinforcement ratio p_t. Values are given in Table 2-3

b= width of column section perpendicular to the direction in calculating shear strength (mm)

d= width of column parallel to the direction in calculating shear strength (mm)

p_t= axial tensile reinforcement ratio. It is the value obtained by dividing the total sectional areas of the mail reinforcement on the tension side of the neutral axis by bd (%)

S_s= contribution of transverse steel in shear strength (N)

A_w= sectional area of hoop ties arranged with an interval of α and an angle of Θ (mm^2)

σ_{sy}= yield point of hoop ties (N/mm^2)

Θ = angle formed between hoop ties and the vertical axis (degree)

α= spacing of hoop ties (mm)

Table 2-1: Average Shear Stress of Concrete [3]

Design compressive strength of concrete (N/mm^2)	21	24	27	30	40
Average shear stress of concrete τ_c (N/mm^2)	0.33	0.35	0.36	0.37	0.41

Table 2-2: Modification Factor in Relation to Effective Height of Columns Section [3]

Effective height (mm)	Below 1000	3000	5000	Above 10000
c_e	1.0	0.7	0.6	0.5

Table 2-3: Modification Factor in Relation to Axial Tensile Reinforcement Ratio [3]

Tensile reinforcement ratio (%)	0.2	0.3	0.5	Above 1.0
c_{pt}	0.9	1.0	1.2	1.5

2.3 ACI 318-14

The shear strength was calculated using the following equations:

$$V_n = V_c + V_s \quad (2.4)$$

$$V_c = 0.166\left(1 + \frac{P}{13.8A_g}\right)\sqrt{f'_c}\,bd \quad (2.5)$$

$$V_s = \frac{A_v f_y d}{s} \quad (2.6)$$

Where:

V_n= shear strength (N)

V_c= contribution of concrete in shear strength (N)

V_s= contribution of transverse steel in shear strength (N)

P= axial load subjected to the column (N)

A_g= gross cross-sectional area of the column (mm^2)

f'_c= concrete compressive strength (N/mm^2)

b= width of column (mm)

d= effective depth of column (mm)

A_v= area of transverse reinforcement within the spacing s (mm^2)

f_y= yield stress of hoops or spirals (N/mm^2)

s= spacing of hoop ties (mm)

2.4 Standard New Zealand (1995)

The shear strength was calculated using the following equations:

$$V_n = V_c + V_s \tag{2.7}$$

$$V_c = \left(4(0.07 + 10\rho_w)\sqrt{f'_c}\sqrt{\frac{P}{f'_c A_g}} - 0.1 \right) bd \tag{2.8}$$

$$\rho_w = \frac{A_v}{bs} \tag{2.9}$$

$$V_s = \frac{\pi}{2}\frac{A_{sp} f_{yh} D_{sp}}{s} \tag{2.10}$$

Where:

V_n= shear strength (N)

V_c= contribution of concrete in shear strength (N)

V_s= contribution of transverse steel in shear strength (N)

ρ_w= transverse reinforcement ratio (%)

f'_c= concrete compressive strength (N/mm^2)

P= axial load subjected to the column (N)

A_g= gross cross-sectional area of the column (mm^2)

b= width of column (mm)

d= effective depth of column (mm)

A_v= area of transverse reinforcement within the spacing s (mm^2)

s= spacing of hoop ties (mm)

A_{sp}= cross-sectional area of spirals or hoops (mm^2)

f_{yh}= yield stress of transverse steel (N/mm^2)

D_{sp}= core diameter of circular column defined by the center-to-center diameter of hoops or spirals (mm)

2.5 ASCE-ACI 426 Shear Strength Approach

The shear strength was calculated using the following equations:

$$V_n = V_c + V_s \tag{2.11}$$

$$V_c = v_b \left(1 + \frac{3P}{f'_c A_g}\right) A_e \tag{2.12}$$

$$A_e = 0.8A_{gross} = 0.628D^2 \tag{2.13}$$

$$v_b = (0.066 + 10\rho_t\sqrt{f'_c} \leq 0.2\sqrt{f'_c} \tag{2.14}$$

$$V_s = \frac{\pi}{2}\frac{A_h f_{yh} D'}{s} \tag{2.15}$$

Where:

V_n= shear strength (N)

V_c= contribution of concrete in shear strength (N)

V_s= contribution of transverse steel in shear strength (N)

v_b= nominal shear stress carried by concrete (N/mm^2)

P= axial load subjected to the column (N)

f'_c= concrete compressive strength (N/mm^2)

A_g= gross cross-sectional area of the column (mm^2)

D= diameter of circular column (mm)

A_e= effective shear area of circular column with diameter D (mm^2)

ρ_t= longitudinal tension steel ratio, taken as $0.5\rho_l$ for columns (Priestley et al. 1994), where ρ_l is estimated by the ratio of longitudinal steel area within the column section to gross cross-sectional area of columns (%)

A_h= area of transverse reinforcement within the spacing s (mm^2)

f_{yh}= yield stress of transverse steel (N/mm^2)

D'= diameter of the spiral or hoop (mm)

s= spacing of hoop ties (mm)

2.6 AASHTO Guide Specifications for the LRFD Seismic Bridge Design and AASHTO LRFD Bridge Design Specifications

Ductile substructure with essentially elastic superstructure is the Earthquake Resisting System (Type 1) considered in this study from the AASHTO LRFD Bridge Design Specifications and the AASHTO Guide Specifications for the LRFD Seismic Bridge Design, hereinafter referred to as AASHTO LRFD Specification and AASHTO Guide Specifications, respectively. The Type 1 system includes conventional plastic hinging in columns that are the prime source of energy dissipation, thus a reliable column/substructure design is needed. The capacity design philosophy inhibits brittle and premature shear failure. It relies on ductile flexural response of plastic hinges to reduce strength requirements, and ensures that the shear strength exceeds the shear corresponding to maximum feasible flexural strength. The greatest shear force that the column can expect is designed to be less than its capacity, by using an overstrength factor.

2.6.1 AASHTO Guide Specifications

The ductile substructure with essentially elastic superstructure Earthquake Resisting System (ERS) is based on the expectation of significant inelastic deformation associated with ductility greater than 4. The other key premise of the provisions is the equal displacement theory assuming that displacements resulting from the inelastic response of a bridge are approximately equal to the displacements obtained from an analysis using the linear elastic response spectrum. As shown in Figure 2-1, it is assumed that Δ_C^L is approximately equal to Δ_D^L.

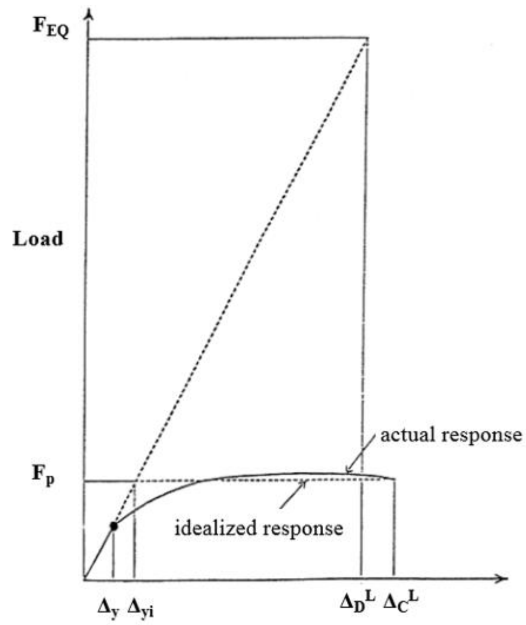


Figure 2-1: Design using Type I approach [1]

The State of Nevada is the third most seismically active state, and it is mapped for seismic design categories (SDC) corresponding to different seismic zones. They reflect the variation in seismic risk across the state. These seismic design categories are based on the 1-second period design spectral acceleration (S_{D1}) for the design earthquake.

The values given by the U.S. Geological Survey for the State of Nevada for the seismic design parameter S_1 have a minimum of 0.096g and maximum of 0.938g. As shown in Table 2-4 all four Seismic Design Categories are to be considered and discussed, since 0.096 is less than 0.15 and 0.938 is more than 0.50. F_v is the site factor for long-period range of acceleration spectrum.

Table 2-4: Seismic Design Categories

Value of S_{D1}=F_vS_1	***Seismic Design Category***
$S_{D1} < 0.15$	**A**
$0.15 \leq S_{D1} < 0.30$	**B**
$0.30 \leq S_{D1} < 0.50$	**C**
$0.50 \leq S_{D1}$	**D**

For Seismic Design Category A, no identification of the Earthquake Resisting System (ERS) or demand analysis is needed. No demand analysis, displacement capacity check or capacity design is required. Minimum detailing requirements for support length, connection design force and columns transverse steel shall be met.

For Seismic Design Category B, the ERS and column hinges are considered. Demand analysis and implicit, closed form solution for displacement capacity check are required. Capacity checks and prescriptive ductile detailing are also required.

For Seismic Design Category C, the ERS and column hinges are identified. Demand analysis and displacement capacity is required. Prescriptive and capacity-design column shear detailing are required.

For Seismic Design Category D, the ERS and column hinges are identified. Demand analysis and displacement capacity is required using pushover analysis. Capacity design including columns shear detailing is required.

In the analysis of the specification the following material, variable properties and equations were used.

2.6.1.1 Reinforcing Steel

Reinforcing bars shall conform to the AASHTO LRFD Specifications. The use of high strength bars with an ultimate tensile strength of 250 ksi is permitted for longitudinal reinforcement in case the low-cycle fatigue properties are not inferior to those of conventional reinforcement.

2.6.1.2 Concrete

Mander's model [14] for confined concrete was used in the sectional analysis.

2.6.1.3 Minimum Shear Reinforcement

The minimum transverse reinforcement required in the AASHTO Guide Specifications is presented in this section.

For SDC B: $\rho_s \geq 0.003$

$\rho_w \geq 0.002$

For SDC C and D: $\rho_s \geq 0.005$

$\rho_w \geq 0.004$

Where:

ρ_s= spiral or circular hoop reinforcement ratio (%)

ρ_w= rectangular web reinforcement ratio (%)

2.6.1.4 Maximum Shear Reinforcement

The shear strength provided by the reinforcing steel,V_s, shall not be taken greater than:

$$V_s \leq 0.25\sqrt{f'_c}A_e \tag{2.16}$$

Where:

A_e= effective area of the cross-section for shear resistance (in^2)

2.6.1.5 Shear Strength Equations

The shear strength within the plastic hinge for SDCs B, C and D was calculated using the following equations:

$$V_n = V_c + V_s \tag{2.17}$$

$$V_c = \upsilon_c A_e \tag{2.18}$$

$$A_e = 0.8A_g \tag{2.19}$$

In case of compressive axial force:

$$\upsilon_c = 0.032\alpha'(1 + \frac{P_u}{2A_g})\sqrt{f'_c} \leq \min(0.11\sqrt{f'_c}; 0.047\alpha'\sqrt{f'_c}) \tag{2.20}$$

Otherwise:

$$\upsilon_c = 0 \tag{2.21}$$

For circular reinforcement:

$$\alpha' = \frac{f_s}{0.15} + 3.67 - \mu_D \tag{2.22}$$

$$f_s = \rho_s f_{yh} \leq 0.35 \tag{2.23}$$

$$\rho_s = \frac{4A_{sp}}{sD'} \tag{2.24}$$

$$V_s = \frac{\pi}{2}\left(\frac{nA_{sp}f_{yh}D'}{s}\right) \tag{2.25}$$

For rectangular reinforcement:

$$\alpha' = \frac{f_w}{0.15} + 3.67 - \mu_D \tag{2.26}$$

$$f_s = 2\rho_{sw} f_{yh} \leq 0.35 \tag{2.27}$$

$$\rho_w = \frac{A_v}{bs} \tag{2.28}$$

$$V_s = \frac{A_v f_{yh} d}{s} \tag{2.29}$$

Where:

A_g= gross area of member cross-section (in^2)

P_u= ultimate compressive force acting on section (kip)

A_{sp}= area of spiral or hoop reinforcing bar (in^2)

s= pitch of spiral or spacing of hoops or ties (in)

D'= core diameter of column measured from center of spiral or hoop (in)

A_v= total cross-sectional area of shear reinforcing bars in the direction of loading (in^2)

b= width of rectangular column (in)

f_{yh}= nominal yield stress of transverse reinforcing (ksi)

μ_D= maximum local displacement ductility ratio of member as defined below

α'= concrete shear stress adjustment factor

n= number of individual interlocking spiral or hoop core sections

d= effective depth of section in direction of loading measured from the compression face of the member to the center of gravity of the tension reinforcement (in)

Outside the plastic hinge region the shear strength shall be determined in accordance with AASHTO LRFD Specification

2.6.2 AASHTO LRFD Specifications

Seismic zones are assigned to reflect the variation in seismic risk across the state. As shown in Table 2-5 all four seismic zones were discussed, since the S_1 for Nevada range from 0.096, which is less than 0.15 and 0.938, which is more than 0.50.

Table 2-5: Seismic Zones

Value of S_{D1}=F_vS_1	***Seismic Zone***
$S_{D1} < 0.15$	**A**
$0.15 \leq S_{D1} < 0.30$	**B**
$0.30 \leq S_{D1} < 0.50$	**C**
$0.50 \leq S_{D1}$	**D**

The method selected for the analysis depends on the seismic zone, regularity and operational classification of the specific bridge.

For single-span bridges no seismic analysis is required regardless of the seismic zone. Bridges in Seismic Zone 1 do not need to be analyzed for seismic loads. Depending on the regularity and seismic risk, multi-span bridges are required to be analyzed using uniform load elastic method, single-mode elastic method, multimode elastic method or time history method as specified in the AASHTO LRFD Specifications.

In the analysis of the specification the following material, variable properties and equations were used.

2.6.2.1 Reinforcing Steel

AASHTO LRFD Specifications limits the steel yield strength between 60 and 75 ksi. The design yield strength of transverse reinforcement was taken equal to the specified yield strength when this did not exceed 60 ksi. When the yield stress exceeded this value, the design yield strength was taken as the stress corresponding to a strain of 0.0035, with a maximum value of 75 ksi. High strength steel with yield strength over 75 ksi were analyzed in this research with a purpose of better understanding of the AASHTO Specifications equations on high strength reinforcing steel use in column design.

2.6.2.2 Concrete

AASHTO LRFD Specifications limits the use of normal weight concrete compressive strength between 2.4 and 10 ksi, except when higher strengths are allowed for normal weight concrete and tests establish the relationship between the concrete strength and other properties. The use of concrete with compressive strength less than 2.4 ksi for normal weight concrete is not allowed in structural applications. High strength concrete columns with compressive strength over 10 ksi were analyzed in this research with a purpose of better understanding of the AASHTO Specifications equations on high strength concrete columns.

2.6.2.3 Minimum Transverse Reinforcement

The minimum transverse reinforcement required in the AASHTO LRFD Specifications is presented in this section.

$$A_v \geq 0.0316\sqrt{f'_c}\frac{b_v s}{f_y} \tag{2.30}$$

Where:

A_v=area of a transverse reinforcement within distance s (in^2)

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

b_v= width of web adjusted for the presence of ducts as specified in Article 5.8.2.9 (in) [8]

s= spacing of transverse reinforcement (in)

f_y= yield strength of transverse reinforcement (ksi)

2.6.2.4 Maximum Spacing of Transverse Reinforcement

The maximum spacing of transverse reinforcement required is presented in this section.

If $v_u < 0.125f'_c$, then $s_{max} = 0.8d_v \leq 24.0\ in$ (2.31)

If $v_u \geq 0.125f'_c$, then $s_{max} = 0.4d_v \leq 12.0\ in$ (2.32)

Where:

v_u= the shear stress (ksi)

d_v= effective shear depth (in)

2.6.2.5 Shear Strength Equations

The shear strength was calculated using the lesser of equation 2.33 and 2.34.

$$V_n = V_c + V_s \quad (2.33)$$

$$V_n = 0.25 f'_c b_v d_v \quad (2.34)$$

$$V_s = \frac{A_v f_y d_v (cot\theta + cot\alpha) sin\alpha}{s} \quad (2.35)$$

$$V_c = 0.0316\beta\sqrt{f'_c} b_v d_v \quad (2.36)$$

For nonprestressed concrete sections not subjected to axial tension and containing at least the minimum amount of transverse reinforcement, or having an overall depth of less than 16.0 in., the following values for β and Θ may be used:

$$\beta = 2.0 \quad (2.37)$$

$$\theta = 45° \quad (2.38)$$

Alternatively for sections containing at least the minimum amount of transverse reinforcement, requirement met for columns, β and Θ can be determined as following:

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \quad (2.39)$$

$$\theta = 29 + 3500\varepsilon_s \quad (2.40)$$

Where:

β= factor indicating ability of diagonally cracked concrete to transmit tension and shear

V_n= nominal shear resistance of the section considered (kip)

V_c= nominal shear resistance provided by tensile stresses in the concrete (kip)

V_s= shear resistance provided by shear reinforcement (kip)

ε_s = net longitudinal tensile strain in the section at the centroid of the tension reinforcement

2.6.2.6 Seismic provisions

For Seismic Zone 1, where the response acceleration coefficient S_{D1} is greater than or equal to 0.1 and less than or equal to 0.15 the transverse reinforcement yield strength has to be less than the strength of the longitudinal reinforcement.

For Seismic Zone 2 the area of longitudinal reinforcement is limited between 0.01 and 0.06 times the gross cross-section area. The minimum transverse reinforcement shall not be less than specified in equation 2-16. In plastic hinge regions the concrete component for shear strength V_c, shall linearly decrease from the value obtained at the compressive force equal to $0.10f'_cA_g$ to zero when the compressive force is zero. These end regions shall be taken greater than the maximum cross-sectional dimension of the column, one-sixth of the clear height of the column or 18 inches. The yield strength of the transverse reinforcement for confinement has to be less than the yield strength of the longitudinal reinforcement.

For Seismic Zones 3 and 4 the area of longitudinal reinforcement is limited between 0.01 and 0.04 times the gross cross-section area. The minimum transverse reinforcement shall not be less than specified in equation 2-16. In plastic hinge regions the concrete component for shear strength V_c, shall linearly decrease from the value obtained at the compressive force equal to $0.10f'_cA_g$ to zero when the compressive force is zero. These end regions shall be taken greater than the maximum cross-sectional dimension of the column, one-sixth of the clear height of the column or 18 inches. The yield strength of the transverse reinforcement for confinement has to be less than the yield strength of the longitudinal reinforcement.

These approaches are further discussed in the parametric study in Chapter 4.

3 Database

3.1 Introduction

The database used for the comparison of predicted shear strengths was developed summarizing the seismic performance of previous column tests for conventional reinforced concrete and high strength concrete columns.

The database for the evaluation of shear strength capacity of conventional reinforced concrete column using the equations specified in Section 2, was constructed using the Pacific Earthquake Engineering Research (PEER) database available on the PEER website [6]. The database is found in Appendix A.

The high strength reinforcing steel and concrete column database is presented in Appendix A [7].

3.2 Conventional Reinforced Concrete Columns

Columns reported as failing due to shear stresses were used for the comparison of the shear strength predicting equations presented in Section 2. Later on, columns that reached the AASHTO LRFD Specifications and the AASHTO Guide Specifications shear capacities were also included in the study, for the purpose of analyzing how both AASHTO Specifications compare to a wider database.

3.2.1 Columns Configuration

The database includes cantilever, double-curvature, double-ended, hammerhead and flexible-base test configurations of columns. These were reduced in the PEER database to the case of an equivalent cantilever column in order to compare the behavior consistently for a wide range of testing configurations.

3.2.2 Conventional Reinforced Concrete Column Database

The tested columns used in this study for the comparison of the shear strength predicting equations presented in Chapter 2 are tabled in Appendix A, Table A-1. Columns in Table 1-1 failed in shear. To increase the number of data points and test the conservative level of the equations, specimens that failed in flexure were added to the study and tabulated in Table A-2. The shear strength of these columns exceeded predicted shear strengths by both AASHTO Specifications, therefore they could be used in the study.

3.3 High Strength Concrete and Reinforcing Steel Columns

High strength reinforcing steel and concrete column experiments were included in this study to analyze how the AASTHO Specifications predict shear strength for high strength materials. Longitudinal reinforcement SD685 with a specified yield strength of 685 MPa

(100 ksi) and transverse reinforcement SD785 with specified yield strength of 785 MPa (114 ksi) were used in these tests. The high strength concrete used had a specified compressive strength of 100MPa (14.5 ksi). The high strength concrete and reinforcing steel column database used is presented in Appendix A Table A-3.

4 Parametric Study

4.1 Introduction

This chapter presents the analytical study of shear strength predicting equations introduced in Chapter 2, using the databases from the previous chapter. Results of all the shear equations are discussed, followed by the comparison of the AASHTO LRFD Specifications and the AASHTO Guide Specifications shear strength of existing column tests. Parametric analysis on large-scale columns is conducted, and last, shear strength comparison using the AASHTO Specifications on high strength concrete and reinforcing steel column behavior is presented.

4.2 Shear Equations

Existing shear strength predicting equations are compared to gather knowledge on how conservative the AASHTO Specifications are in comparison to actual test results and to the equations previously mentioned: Japan Road Association Specification for Highway Bridges [3], ACI 318-14 [4], Standard New Zealand [5], ASCE-ACI 426 Shear Strength Approach [6]. For this comparison 24 column tests were included, presented in Table A-1 of Appendix A.

For a better overview of the results obtained, no reduction factors were used throughout this study.

Figures 4-1 to 4-6 show the ratio of shear strength at failure given from the testing of columns and predicted shear strength for each equation. Values over one show that the actual test strength exceeds the shear strength predicted by the equation, thus the equation is conservative.

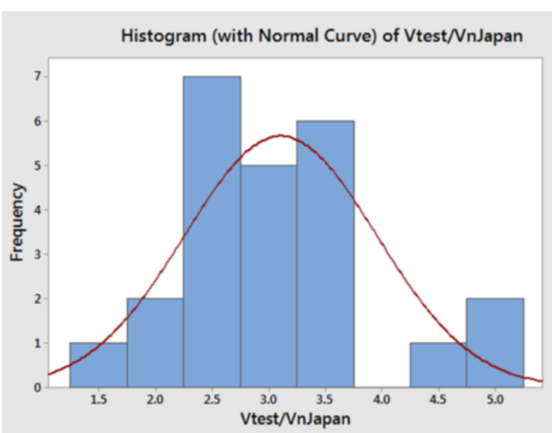


Figure 4-1: Japan Specification Histogram

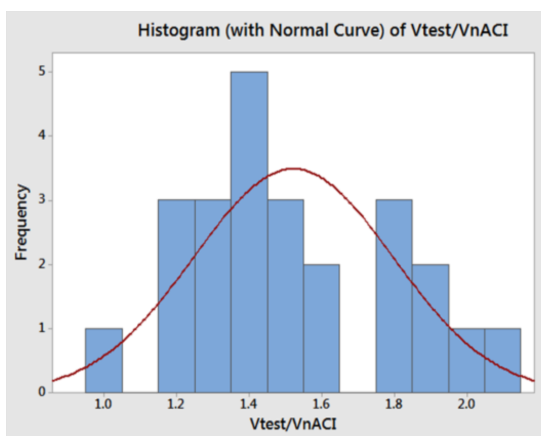


Figure 4-2: ACI 318-14 Histogram

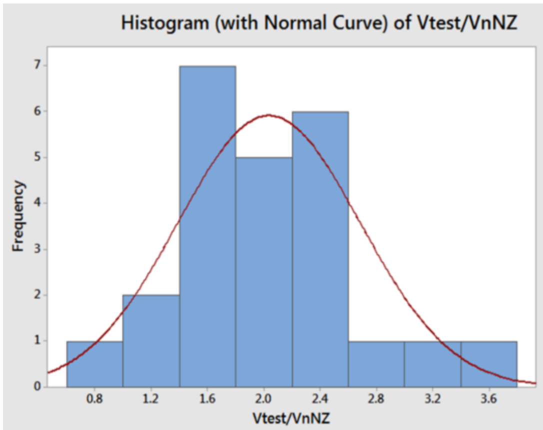


Figure 4-3: New Zealand Histogram

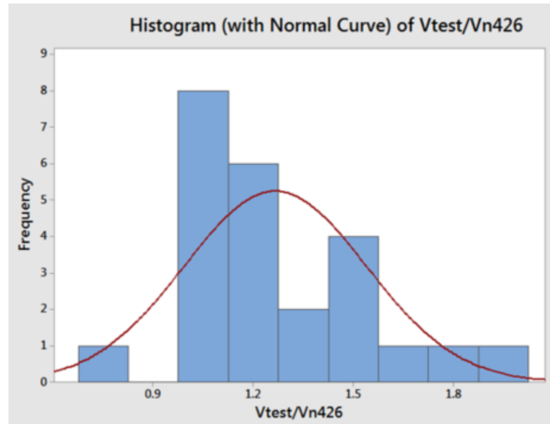


Figure 4-4: ASCE-ACI 426 Histogram

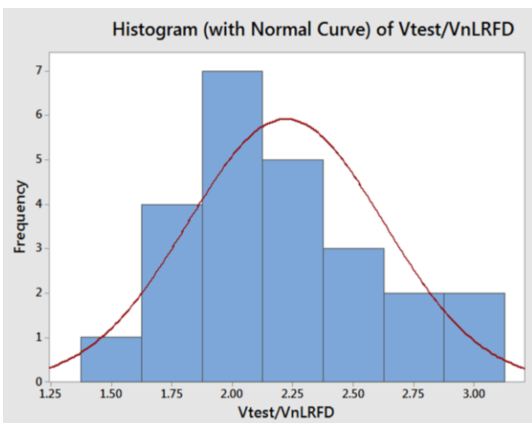


Figure 4-5: AASHTO LRFD Histogram

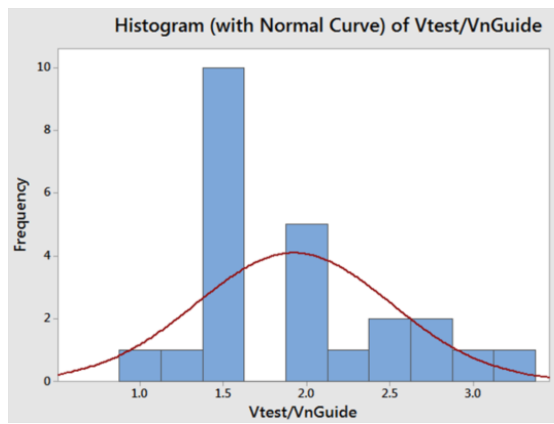


Figure 4-6: AASHTO Guide Histogram

Table 4-1 shows the means and standard deviations of each equation. The results are shown in descending magnitude of the mean.

Table 4-1: Mean and standard deviation of shear equations

	Mean	***Standard Deviation***
Japan	3.11	0.85
AASHTO LRFD	**2.22**	0.40
New Zealand	2.04	0.65
AASHTO GUIDE	**1.92**	0.59
ACI 318	1.52	0.27
ASCE-ACI 426	1.27	0.27

As shown in Table 4-1, Japan Road Association Specification for Highway Bridges provides the most conservative shear strength for the 24 analyzed columns, the average actual strength was 3 times higher than the estimated strength. The shear strength calculated using ASCE-ACI 426 Shear Strength Approach resulted being the least conservative.

AASHTO LRFD Specifications exceeded two times the actual shear strength in 75% of the cases, showing that it is a very conservative approach for the analyzed columns.

AASHTO Guide Specifications predicted shear strengths very close (with a strength divided by prediction ratio of less than 1.5) to the actual test results for 50% of the cases. On the other hand, cases with a strength divided by prediction of greater than 1.5, the other 50% of the cases resulted in very conservative shear strengths for the columns. This inconsistency was explained in the comparison of the AASHTO Specifications in figure 4-10. Note that no reduction factors were used in the study.

4.3 AASHTO Specifications on Small-Scale Columns

The original database had 24 shear dominate columns presented in Table A-1 in Appendix A. In order to increase the number of columns in the database, additional 22 columns were added that did not fail in shear but in flexure for the comparison of the two AASHTO Specifications. They are presented in Table A-2 in Appendix A. These columns reached the shear strength given by the AASHTO Specifications or more, therefore they could be introduced in the analysis for the purpose of this study. Since these columns were flexure dominated, they were not analyzed in the previous section.

Figure 4-7 displays the correlation between the two AASHTO Specifications. Values of the test result and predicted shear strength ratios (represented by the dots in the figure) by the AASHTO LRFD Specifications are higher than the ratios by the AASHTO Guide Specifications, with 95% accuracy. The line represents the relation between the two means of the test result and predicted shear strength ratios for the AASHTO Guide Specifications and the AASHTO LRFD Specifications. This observation confirms that the AASHTO LRFD Specification is a more conservative shear design than the AASHTO Guide Specification; the predicted capacity is safely exceeded for both AASHTO Specifications, nonetheless.

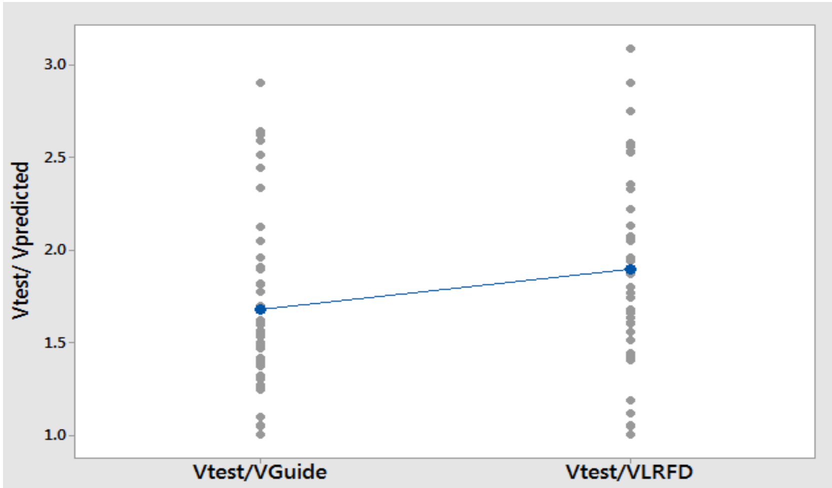
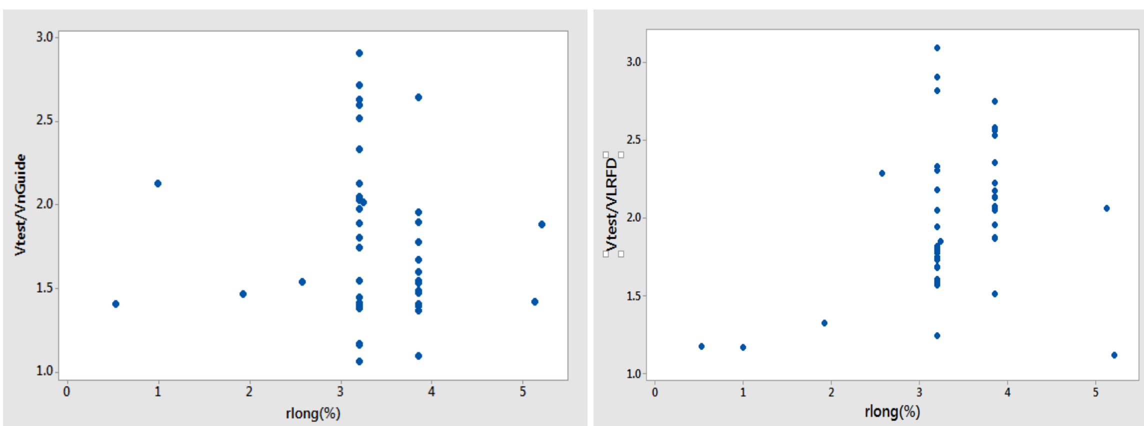


Figure 4-7: AASHTO Specifications shear strength and actual shear strength ratio

To achieve shear failure in the columns, a majority of the columns tested exceeded the 1%-2.5% longitudinal steel ratio used for conventional bridge column design, thus the longitudinal steel ratio varies between 0.5-5.5. A parametric study with conventional bridge column longitudinal steel ratio was conducted and presented in section 4.4 to obtain results on columns with parameters in interest. The frequency of the longitudinal steel ratios of the database can be seen in Figures 4-8 (a) and (b).

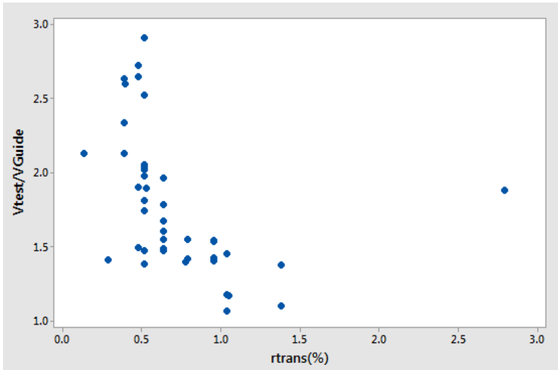


(a) AASHTO Guide Specifications longitudinal steel ratios

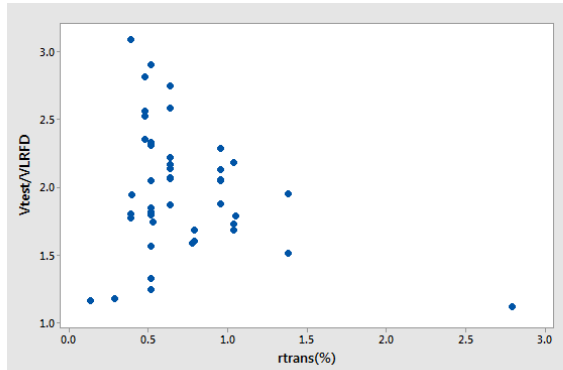
(b) AASHTO LRFD Specifications longitudinal steel ratios

Figures 4-8 (a) and (b): Longitudinal steel ratios

The transverse steel ratio of the considered columns varies between 0.2-1.4, lower in some cases than the 0.5% required by the AASHTO Specifications. Nonetheless, the columns with less transverse steel ratio than the required resulted in conservative predictions of shear strength. The frequency of the transverse steel ratios of the database can be seen in Figures 4-9 (a) and (b).



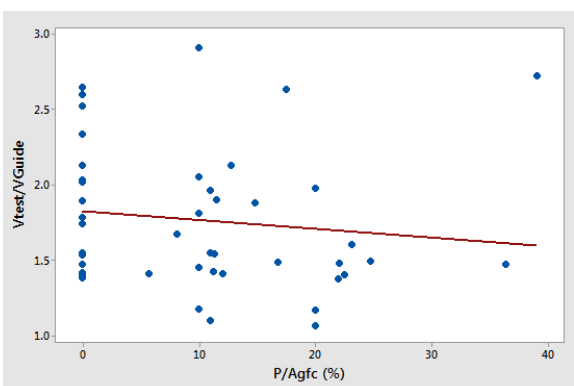
(a) AASHTO Guide Specifications
transverse steel ratios



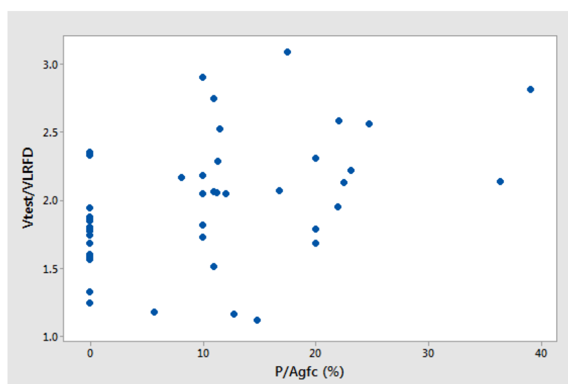
(b) AASHTO LRFD Specifications
transverse steel ratios

Figures 4-9 (a) and (b): Transverse steel ratios

In Figures 4-10 (a) and (b) the test result and predicted shear strength ratios for the AASHTO Guide Specifications and the AASHTO LRFD Specifications were plotted as a function of the axial load index. The accuracy increases in predicting the shear strength of the AASHTO Guide Specifications with the increase of axial load index as seen in Figure 4-10 (a). It was observed that for axial load index values of 0.1 and greater, the AASHTO Guide Specifications provides less conservative shear strength predictions. This explained the inconsistency between conservative and unconservative shear strength prediction of the AASHTO Guide Specifications. On the other hand, axial load is not accounted for in the AASHTO LRFD Specifications, thus no trend is observed in Figure 4-10 (b).



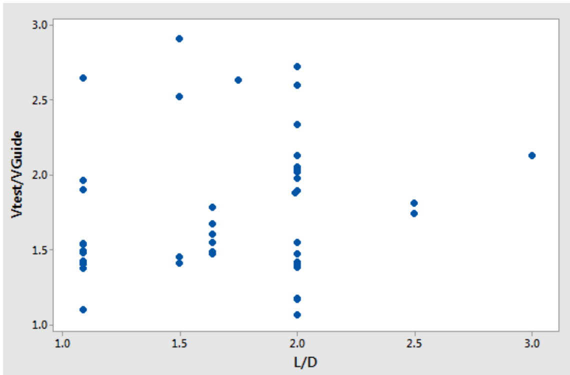
(a) AASHTO Guide Specifications
axial load index



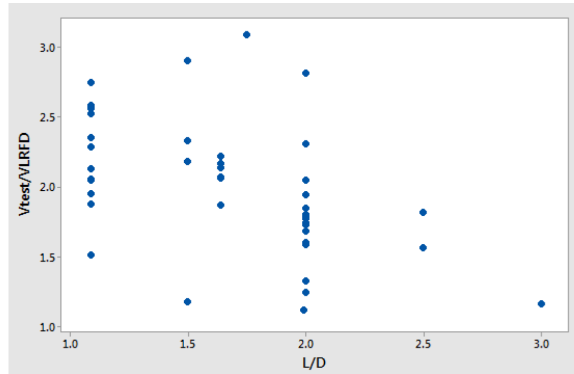
(b) AASHTO LRFD Specifications
axial load index

Figures 4-10 (a) and (b): Axial load index

The studied columns aspect ratio shown in Figure 4-11 has no concluding results visible in the study of the tested columns.



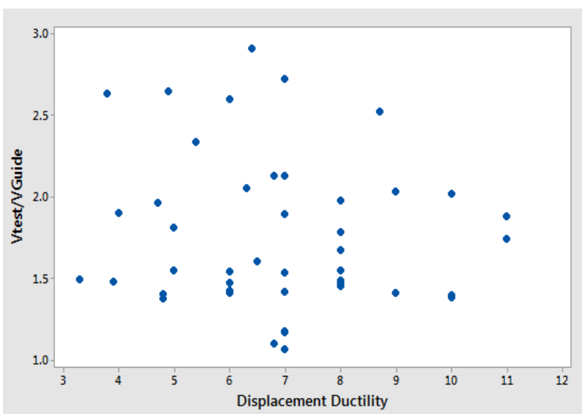
(a) AASHTO Guide Specifications aspect ratio



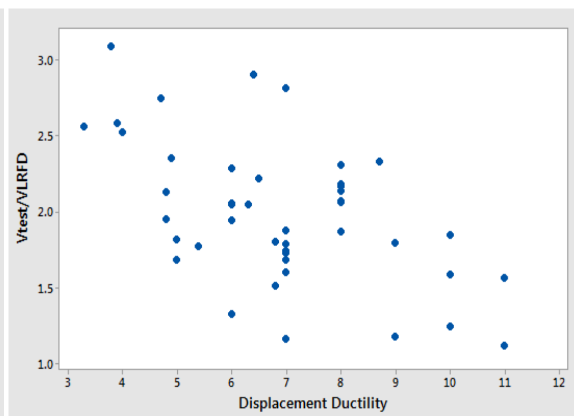
(b) AASHTO LRFD Specifications aspect ratio

Figures 4-11 (a) and (b): Aspect ratio

As the displacement ductility increases, the shear strength decreases in the shear strength predictions by the AASHTO Guide Specifications, but the pattern cannot be seen in Figure 4-12 (a) due to the number of variables.



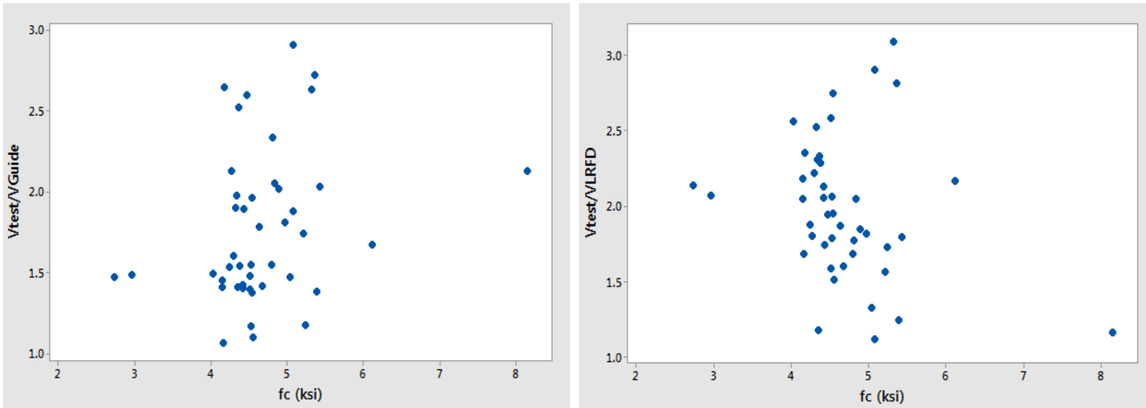
(a) AASHTO Guide Specifications displacement ductility



(b) AASHTO LRFD Specifications displacement ductility

Figures 4-12 (a) and (b): Displacement ductility

The concrete compressive strength for more than 90% of the considered columns was between 4-5.5ksi as shown in Figures 4-13 (a) and (b).



(a) AASHTO Guide Specifications concrete compressive strength

(b) AASHTO LRFD Specifications concrete compressive strength

Figures 4-13 (a) and (b): Concrete compressive strength

The AASHTO Guide Specifications predict a higher shear strength resisted by the transverse reinforcement than the AASHTO LRFD Specifications. Thus the higher the tranverse reinforcement ratio is, the higher the predicted shear strength is for the AASHTO Guide Specifications in comparison to the AASHTO LRFD Specification, as seen in Figure 4-14.

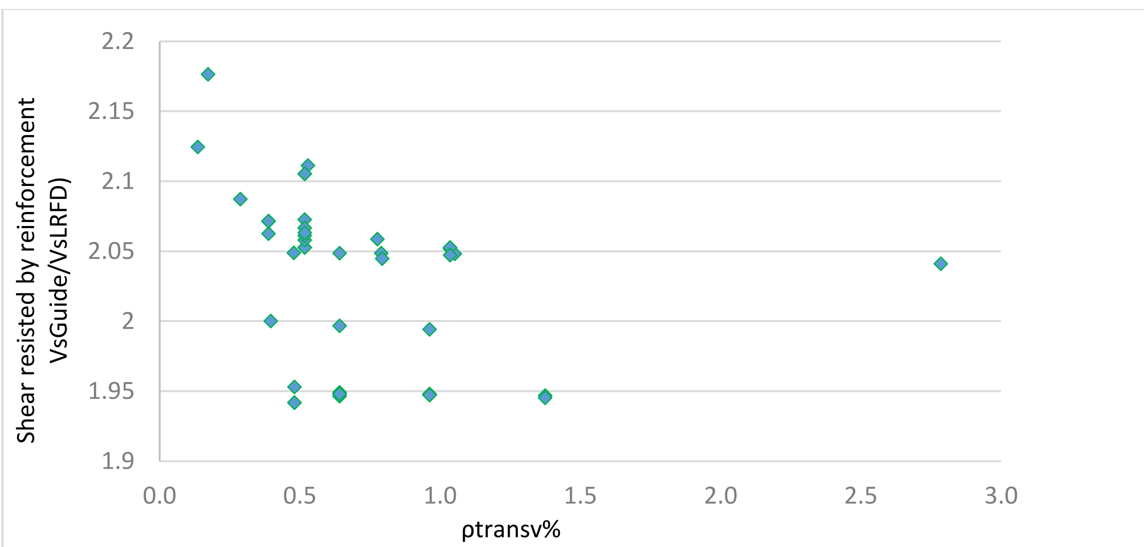


Figure 4-14: Shear resisted by reinforcement vs. ρ_{transv}%

The concrete component V_c of the shear strength predicted by AASHTO Guide Specifications is zero when the axial load on the columns is zero. As the axial load increases, V_c adds to the nominal shear strength V_n. Over 10% axial load index, the values of V_c predicted by the AASHTO Guide Specification increase significantly as shown in Figure 4-15 and as observed in previous work.

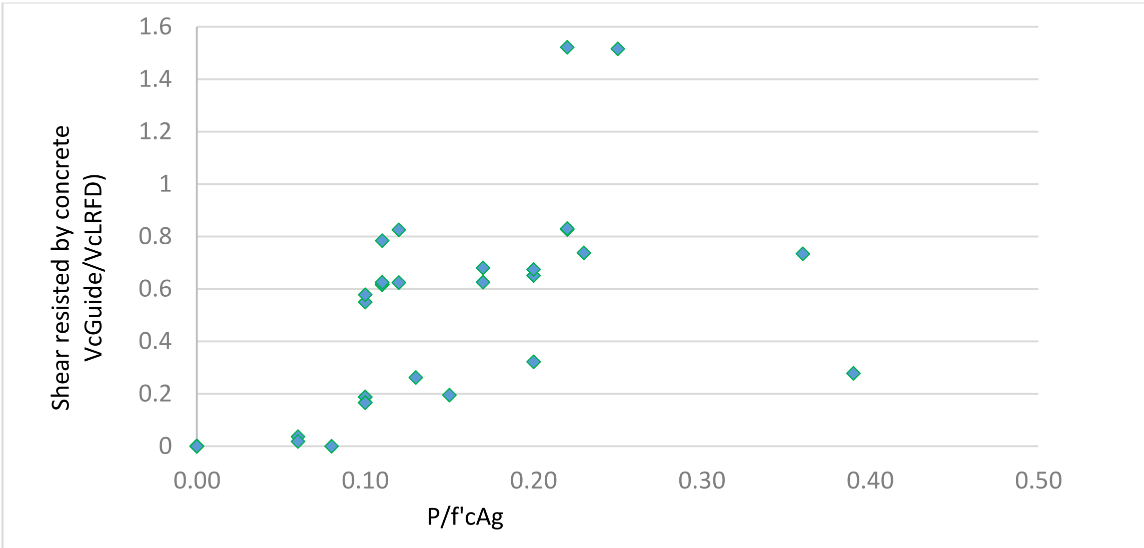


Figure 4-15: Shear resisted by reinforcement vs. $P/(f'_c A_g)$

4.4 Parametric Study of AASHTO Specifications on Full-Scale Columns

A full-scale column parametric study was added to this study to analyze frequently used parameters for conventional bridges and summarize how the predicted shear strength is influenced by each of them. A 6-foot diameter and 23-foot high column was analyzed for this purpose.

The ratio of shear strength predicted by AASHTO Guide Specifications and AASHTO LRFD Specifications was plotted in Figure 4-16 as a function of concrete compressive strength. This was varied between 5 ksi and 10 ksi. Zero compression force was assumed. The reinforcing steel yield strength was 60 ksi. For longitudinal reinforcement assumed was 34 #11 bars, resulting in a longitudinal reinforcement ratio of 1% and #8 bars at a spacing of 4 inches, resulted in a transverse reinforcement ratio of 1.5%. Even though the increase of the concrete compressive strength did not change the shear strength value predicted by the AASHTO Guide Specifications, the nominal shear strength from the AASHTO Guide Specification was higher than the strength predicted by the AASHTO LRFD Specifications.

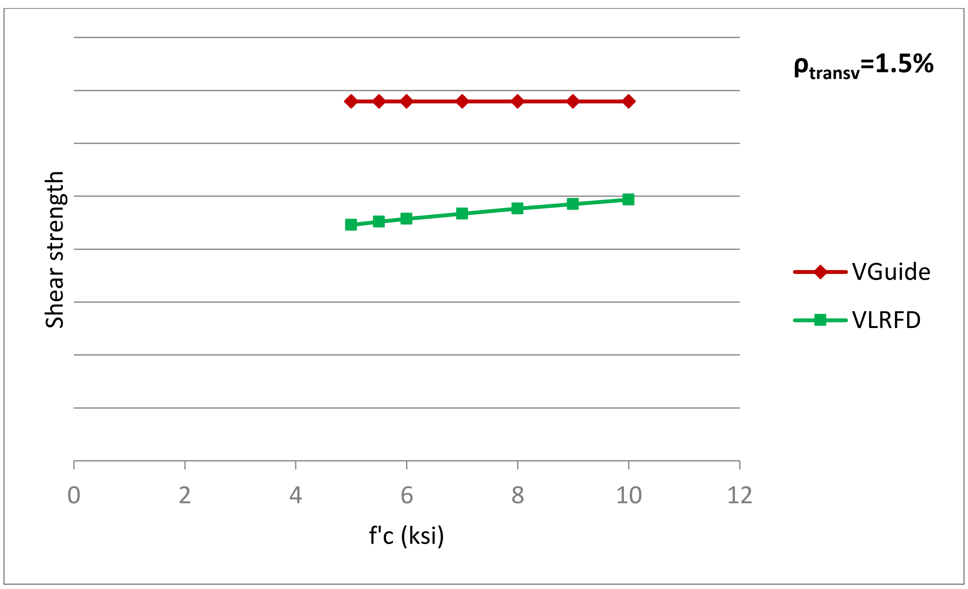


Figure 4-16: AASHTO Specifications shear strength and actual shear strength ratio vs. concrete compression strength (transverse reinforcement ratio 1.5%)

Figure 4-17 presents the effect of transverse reinforcement ratio decrease when the axial load index remains equal to zero. The ratio values above one show that for high transverse reinforcement ratios AASHTO Guide Specifications predicts higher shear strength than the AASHTO LRFD Specifications. At 0.5% transverse reinforcement ratio AASHTO LRFD Specifications exceed these, thus values under one can be observed.

For an axial load ratio of 0%, the variation of longitudinal reinforcement ratio does not influence the shear strength predicted by the AASHTO Guide Specifications regardless of the variation of the transverse reinforcement, because the concrete component V_c is not taken in consideration in shear strength prediction.

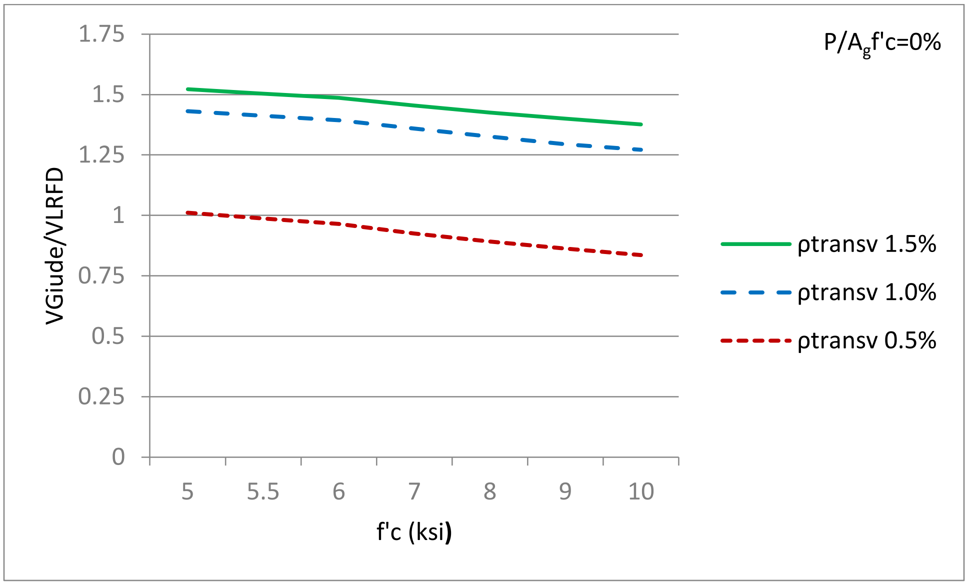


Figure 4-17: AASHTO Specifications shear strength and actual shear strength ratio vs. concrete compression strength (transverse reinforcement ratio 0.5%, 1.0%, 1.5%)

For an axial load ratio of 5%, shown in Figure 4-18, it is observed that the shear strength prediction by the AASHTO Guide Specifications exceeds the values obtained by the AASHTO LRFD Specifications. The steeper slope confirms that the concrete component significantly adds to the nominal shear strength as the axial load ratio is increased.

The longitudinal reinforcement ratio variation does not have a significant effect on the concrete component of the shear strength predicted by the AASHTO Guide Specifications; neither does for the AASHTO LRFD Specifications.

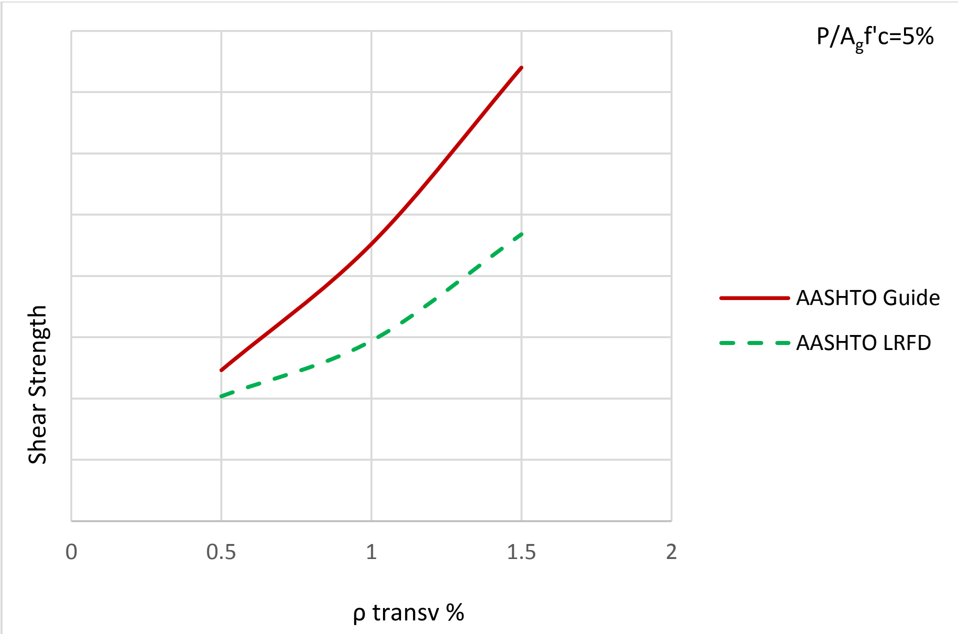


Figure 4-18: AASHTO Specifications shear strength vs. transverse reinforcement ratio (5% axial load index)

At an axial load ratio of 10%, in Figure 4-19 it is observed that the shear strength prediction by the AASHTO Guide Specifications exceeds the values obtained by the AASHTO LRFD Specifications. The increase is more abrupt than in Figure 4-18 confirming that by increasing the axial load ratio, the concrete component significantly adds to the nominal shear strength.

The longitudinal reinforcement ratio variation does not have a significant effect on the concrete component of the shear strength predicted by the AASHTO Guide Specifications; nor does it for the AASHTO LRFD Specifications.

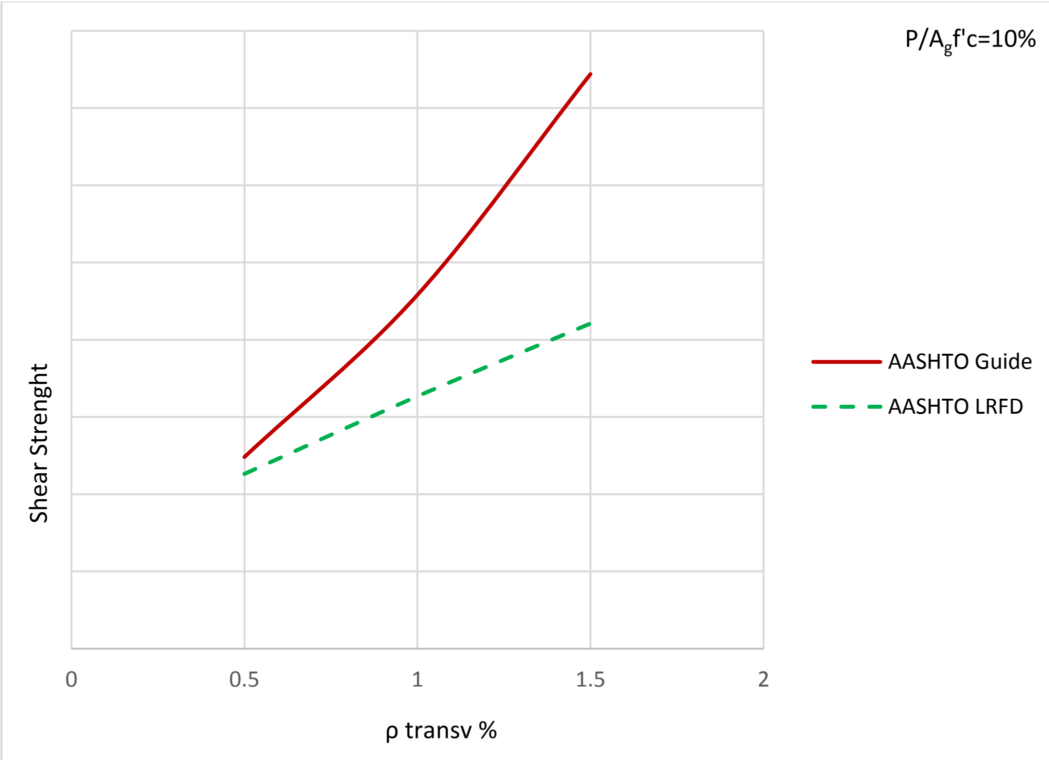


Figure 4-19: AASHTO Specifications shear strength vs. transverse reinforcement ratio (10% axial load index)

As shown in Figure 4-20, the concrete component by the AASHTO LRFD Specifications stays constant when the axial load index is increased, while the concrete component by the AASHTO Guide Specifications predictions shows a linear increase until 10% where it exceeds the shear strength predicted by AASHTO LRFD Specifications and continues to increase in a linear trend.

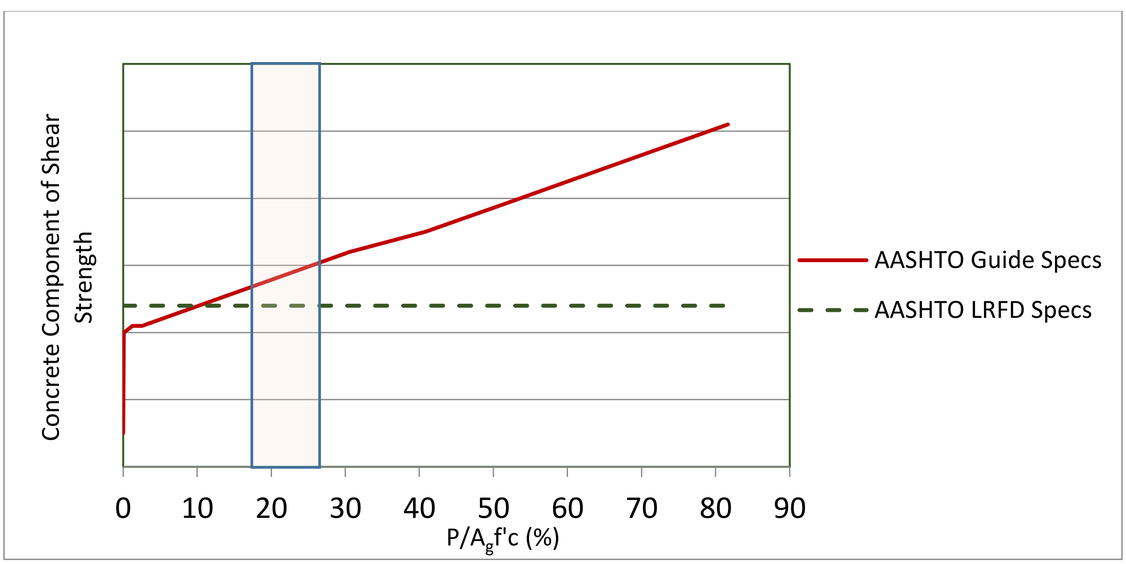


Figure 4-20: Shear strength vs. axial load index

Figure 4-21 illustrates that the increase of axial load causes a decrease of ductility in the columns especially from 0 to 20% axial load index. The AASHTO LRFD Specifications is a force based approach and it does not take in consideration the effect of ductility and axial load relationship. Advanced concrete analysis is required for the AASHTO Guide Specification.

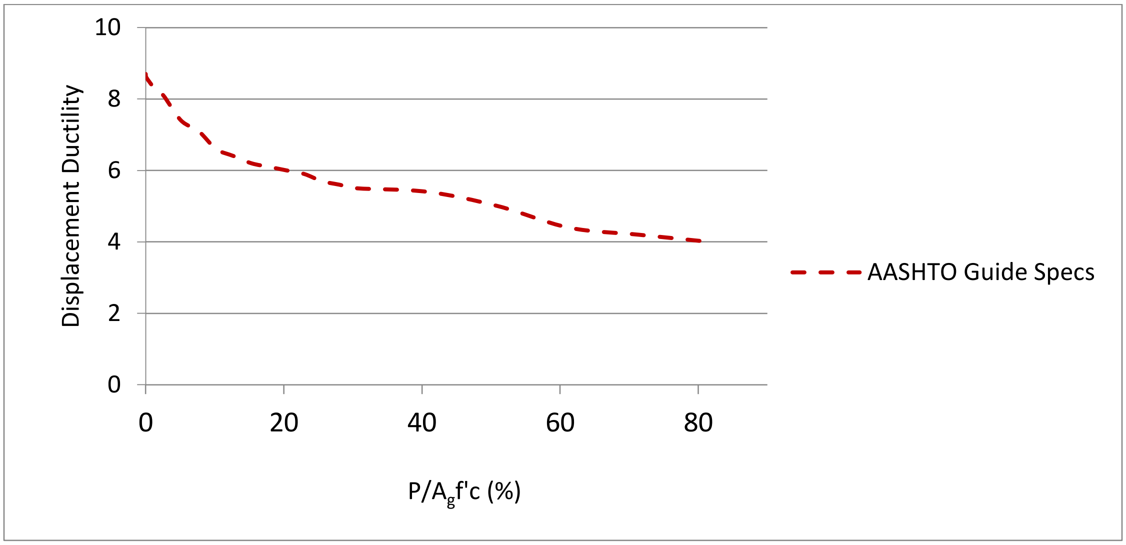


Figure 4-21: Displacement ductility vs. axial load index

4.5 Parametric Study of AASHTO Specifications on High-Strength Columns

The comparison of shear strength predicted by the AASHTO LRFD Specifications and the AASHTO Guide Specifications of high strength concrete and high strength reinforcing steel columns is discussed in this section for the plastic hinge region. The database used is presented in Table A-3 of Appendix A.

As shown in Figure 4-22, both the AASHTO Guide Specifications and the AASHO LRFD Specifications conservatively predict the actual shear test capacity. The shear predicted by the AASHTO Guide Specifications was more conservative than the AASHTO LRFD Specifications.

The use of high strength concrete resulted in higher shear capacity prediction by the AASHTO LRFD Specifications because of the high concrete component.

The concrete compressive strength of the analyzed columns was between 10 ksi and 16 ksi as presented in Figure 4-22.

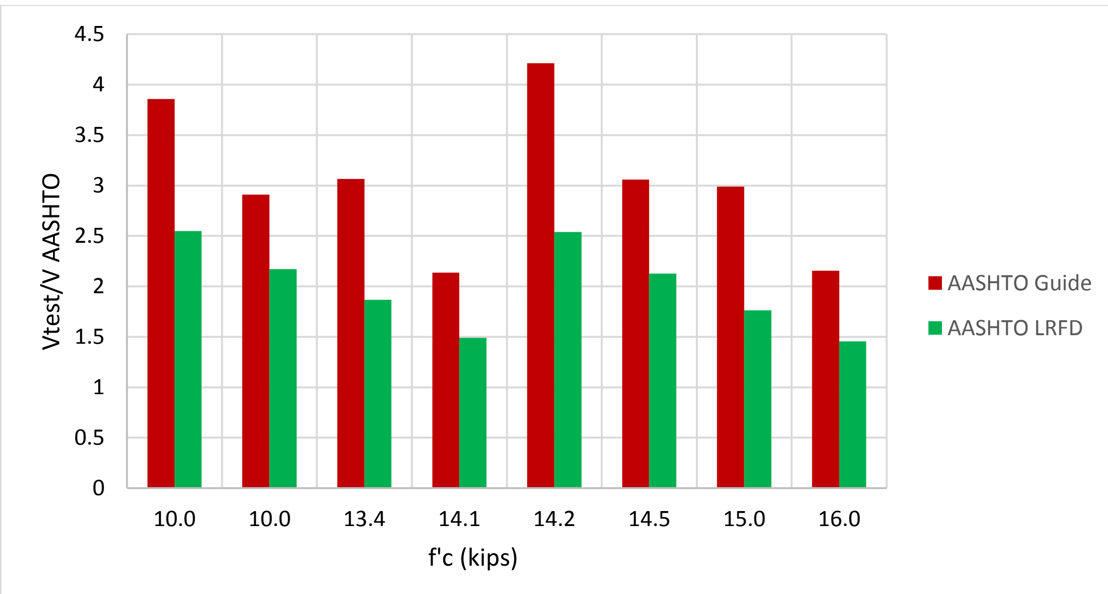


Figure 4-22: V_{test}/V_{AASHTO} vs. compression concrete strength

The transverse reinforcement ratio in the columns was 0.4% and 0.75 as illustrated in Figure 4-23.

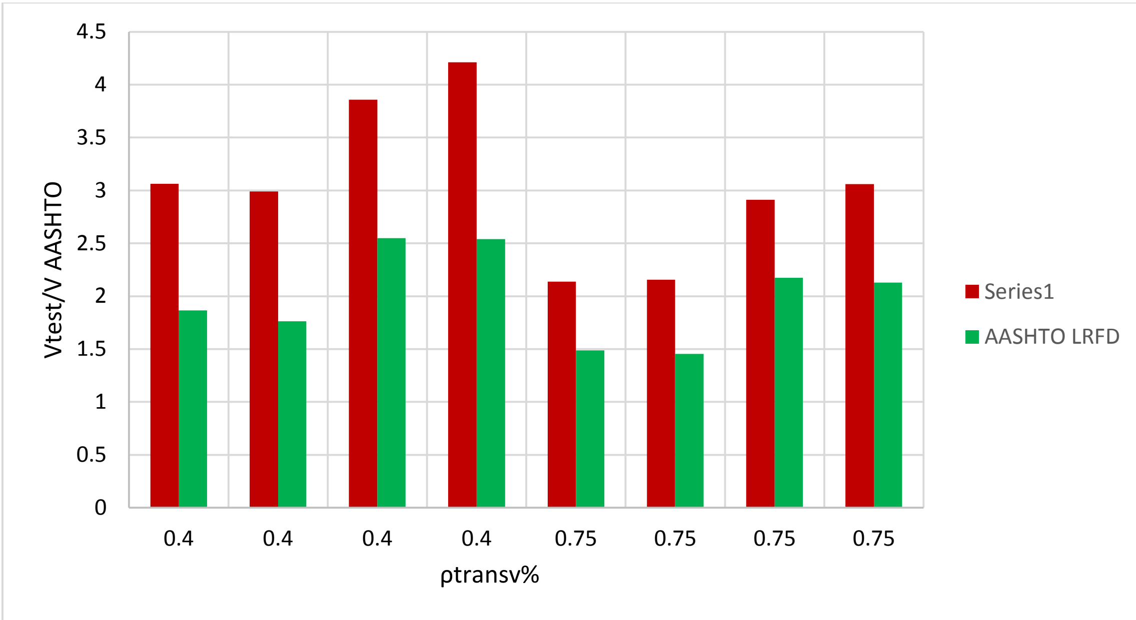


Figure 4-23: V_{test}/V_{AASHTO} vs. transverse reinforcement ratio

The axial load index was 10% and 20% for the included columns, that is typical for bridge column design and it is shown in Figure 4-24.

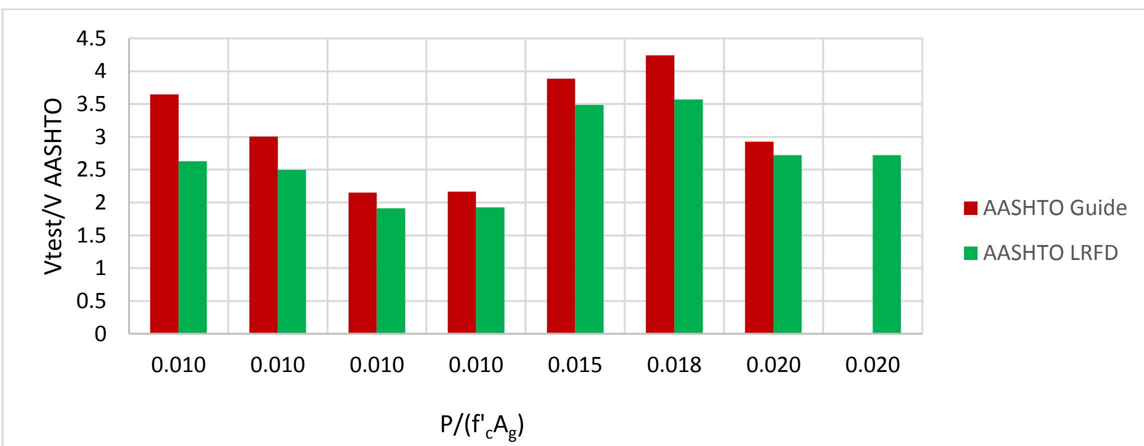


Figure 4-24: V_{test}/V_{AASHTO} vs. axial load index

The high strength reinforcing steel provides the same strength for both AASHTO Specification, thus AAHTO LRFD Specifications provides less conservative shear strength.

Figures 4-22 to 4-23 show that both the AASHTO LRFD Specifications and the AASHTO Guide Specifications provide very conservative shear strength approximations. The shear strength provided is in some cases more than three times the necessary strength.

For normal strength concrete and reinforcement 72% of the specimens had a ratio of V_{test}/V_{AASHTO} that ranged between 1.5-2.5 for AASHO LRFD Specifications and between 1-2 for the AASHTO Guide Specifications, while for columns with high strength concrete and steel the ratio ranged between 1.5-3 and 2-4.5, respectively, showing that the conservatism of the AASHTO Specifications equations increase with the use of high strength materials.

Table 4-1: V_{test}/V_{AASHTO} ratio

	Normal strength		High strength	
	V_{test}/V_{LRFD}	V_{test}/V_{Guide}	V_{test}/V_{LRFD}	V_{test}/V_{Guide}
V_{test}/V_{AASHTO} ratio	Percentage on total specimens (%)			
1-1.5	12	41	-	-
1.5-2	41	31	50	-
2-2.5	31	14	25	25
2.5-3	14	14	25	25
3-3.5	2	-	-	25
3.5-4	-	-	-	12.5
4-4.5	-	-	-	12.5

5 Conclusions

This report presents an analytical study of the shear capacity using the AASHTO LRFD Bridge Design Specifications and the AASHTO Guide Specifications for the LRFD Seismic Bridge Design. The previous chapter discussed how the analyzed parameters influence the predicted shear strength.

For normal strength reinforced concrete, when no axial load acts on the columns, the concrete component is neglected in the AASHTO Guide Specifications, thus the shear stresses are assumed to be resisted only by the transverse steel. These are the cases in which the displacement based approach has the more conservative results. In axially loaded columns, which are realistic cases in bridge design, shear capacity is predicted more accurately as the axial load index increases, adding to the shear strength and through detailed analysis predicting a closer result to the actual shear strength. For bridge columns, the AASHTO Guide Specifications predict a higher strength, thus it resulted in more cost efficiency while providing a conservative design. Note that safety factors were not used in the study when comparing with experimental results.

Both AASHTO Specifications predicted highly conservative shear strength for the analyzed columns. To achieve the same level of conservatism the AASHTO Guide Specifications requires less transverse reinforcement, resulting in a more ductile and cost-efficient design.

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APPENDIX A

Table A-1: Database used for the comparison of shear equations

Column	f'_c (ksi)	f_{yl} (ksi)	f_{yt} (ksi)	$P/(f'_c A_g)$	L/D	r_{long} (%)	r_{trans} (%)
Ang et al. 1985, No. 4	4.4	63	46	0.00	2.0	0.03	0.5
Ang et al. 1985, No. 6	4.4	63	48	0.00	1.5	0.03	0.5
Ang et al. 1985, No. 7	4.3	65	54	0.00	2.0	0.03	0.4
Ang et al. 1985, No. 16	4.8	63	47	0.10	2.0	0.03	0.5
Ang et al. 1985, No. 18	5.1	63	47	0.10	1.5	0.03	0.5
Ang et al. 1985, No. 19	5.0	63	47	0.10	1.5	0.03	0.4
Ang et al. 1985, No. 20	5.3	70	47	0.17	1.8	0.03	0.4
Ang et al. 1985, No. 21	4.8	63	47	0.00	2.0	0.03	0.4
Ang et al. 1985, No. 22	4.5	63	45	0.00	2.0	0.03	0.4
Arakawa et al. 1987, No. 1	4.2	53	53	0.00	1.1	0.04	0.5
Arakawa et al. 1987, No. 2	4.2	53	53	0.00	1.1	0.04	1.0
Arakawa et al. 1987, No. 4	4.3	53	53	0.12	1.1	0.04	0.5
Arakawa et al. 1987, No. 6	4.1	53	53	0.12	1.1	0.04	1.0
Arakawa et al. 1987, No. 8	4.6	53	53	0.11	1.1	0.04	1.4
Arakawa et al. 1987, No. 9	4.4	53	53	0.11	1.1	0.05	1.0
Arakawa et al. 1987, No. 12	4.0	53	53	0.25	1.1	0.04	0.5
Arakawa et al. 1987, No. 13	4.4	53	53	0.22	1.1	0.04	1.0
Arakawa et al. 1987, No. 14	4.5	53	53	0.22	1.1	0.04	1.4
Arakawa et al. 1988, No. 17	4.5	53	55	0.11	1.1	0.04	0.6
Arakawa et al. 1988, No. 19	4.5	53	55	0.11	1.6	0.04	0.6
Arakawa et al. 1988, No. 22	3.0	53	55	0.17	1.6	0.04	0.6
Arakawa et al. 1988, No. 24	4.5	53	55	0.22	1.1	0.04	0.6
Arakawa et al. 1988, No. 25	4.3	53	55	0.23	1.6	0.04	0.6
Arakawa et al. 1988, No. 27	2.7	53	55	0.36	1.6	0.04	0.6

Table A-2: Database used for the comparison of AASHTO LRFD Bridge Design Specifications and the AASHTO Guide Specifications for the LRFD Seismic Bridge Design

Column	f'_c (ksi)	f_{yl} (ksi)	f_{yt} (ksi)	$P/(f'_cA_g)$	L/D	r_{long} (%)	r_{trans} (%)
Ang et al. 1985, No. 1	5.4	63	48	0.00	2.0	3.2	0.5
Ang et al. 1985, No. 2	5.4	43	48	0.00	2.0	3.2	0.5
Ang et al. 1985, No. 3	5.2	63	48	0.00	2.5	3.2	0.5
Ang et al. 1985, No. 5	4.5	63	48	0.00	2.0	3.2	0.8
Ang et al. 1985, No. 8	4.2	65	54	0.20	2.0	3.2	1.0
Ang et al. 1985, No. 10	4.5	65	48	0.20	2.0	3.2	1.1
Ang et al. 1985, No. 11	4.3	65	54	0.20	2.0	3.2	0.5
Ang et al. 1985, No. 12	4.1	63	48	0.10	1.5	3.2	1.0
Ang et al. 1985, No. 13	5.3	63	47	0.10	2.0	3.2	1.0
Ang et al. 1985, No. 14	4.9	61	47	0.00	2.0	3.2	0.5
Ang et al. 1985, No. 15	5.0	63	47	0.00	2.0	1.9	0.5
Ang et al. 1985, No. 17	5.0	63	47	0.10	2.5	3.2	0.5
Ang et al. 1985, No. 23	4.7	63	48	0.00	2.0	3.2	0.8
Ang et al. 1985, No. 24	4.8	63	45	0.00	2.0	3.2	0.8
Wong et al. 1990, No. 2	5.4	69	49	0.39	2.0	3.2	0.5
Arakawa et al. 1987, No. 10	4.4	53	53	0.11	1.1	2.6	1.0
Arakawa et al. 1988, No. 15	4.6	53	55	0.00	1.6	3.9	0.6
Arakawa et al. 1988, No. 23	6.1	53	55	0.08	1.6	3.9	0.6
Benzoni & Priestley 1994, NR1	4.4	67	52	0.00	1.5	0.5	0.3
Benzoni & Priestley 1994, NR2	4.4	67	52	0.00	1.5	1.0	0.2
Vu et al. 1998, NH4	5.1	68	63	0.15	2.0	5.2	2.8
Nelson & Price 2000, Col1	8.2	66	66	0.13	3.0	1.0	0.1

Table A-3: High strength column database used for the comparison of AASHTO LRFD Bridge Design Specifications and the AASHTO Guide Specifications for the LRFD Seismic Bridge Design

Column	f'_c (ksi)	f_{yl} (ksi)	f_{yt} (ksi)	$P/(f'_cA_g)$	L/D	r_{long} (%)	r_{trans} (%)
HS Column-1	13.4	106	125	0.010	3	3.5	0.4
HS Column-2	15.0	106	125	0.010	3	3.5	0.4
HS Column-3	14.1	106	125	0.010	3	3.5	0.75
HS Column-4	16.0	106	125	0.010	3	3.5	0.75
HS Column-5	10.0	106	125	0.015	3	3.5	0.4
HS Column-6	14.2	106	125	0.018	3	3.5	0.4
HS Column-7	10.0	106	125	0.020	3	3.5	0.75
HS Column-8	14.5	106	125	0.020	3	3.5	0.75