

Effect of Cumulative Cycling on Shear Strength of Older Reinforced Concrete Bridge Columns

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Abstract

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This research assessed the effect of cycling on the shear strength of older reinforced concrete bridge columns in Washington State, particularly those designed prior to 1975. These columns typically have low transverse reinforcement ratios ($<0.5\%$) and circular hoops with overlapped ends, making them susceptible to shear and flexure-shear failures.

A prototype column representative of 1960s Washington State bridge construction was scaled to a 1:3 ratio and used to design three nominally identical reinforced concrete columns that were subjected to loading protocols with identical drift-ratio targets but different numbers of cycles between drift targets. All three specimens exhibited shear-dominated damage progression and were classified as shear failures. For the relatively low cumulative plastic displacement demands these columns experienced before failure, increasing the number of cycles between drift-ratio targets did not yield a clear reduction in the effective lateral force at failure. However, the

specimen subjected to a higher number of cycles sustained greater deformation before failing in shear.

The results from these tests were combined with a compiled sample of 13 existing experimental tests of circular reinforced concrete columns with representative properties of pre-1975 bridge columns that exhibited shear damage, obtained from the UW/PEER and ACI 369 databases. These data were used to evaluate the Kowalsky and Priestley (2000) shear strength model, which overestimated shear strength in most tests with overlapped hoops, including those tested in this study. This overestimation is attributed to the reduced effectiveness of overlapped hoops in contributing to shear strength, together with trends suggesting that concrete's shear strength contribution degrades as cumulative plastic demands increase. Additional tests are needed to reach definitive conclusions regarding the effect of cumulative plastic displacement demand on shear strength.

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NOTATION

A_e	=	effective shear area
A_g	=	cross-sectional area
A_{hoop}	=	area of a spiral or hoop bar
A_s	=	total area of the longitudinal steel
A_{sT}	=	area of longitudinal reinforcement in tension
A_{str}	=	effective area at the end of diagonal strut
A_v	=	shear area provided by the transverse reinforcement hoops $(\frac{\pi A_{hoop}}{2})$ for circular columns, except for ACI codes, where it was taken as $2A_v$
$A_{v,min}$	=	minimum area of transverse reinforcement bar within spacing s from the model's referred code
b	=	width or diameter of the column
b_i^+	=	elastic displacement window limit in the positive direction at datapoint (i)
b_i^-	=	elastic displacement window limit in the negative direction at datapoint (i)
c	=	neutral axis depth
c_c	=	clear cover
d	=	distance from extreme compression fiber to centroid of longitudinal tension reinforcement

d_{bl}	=	diameter of longitudinal reinforcement bar
d_{bt}	=	diameter of transverse reinforcement bar
d_c	=	depth of elastic compression zone
d_e	=	depth of center of gravity of steel
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
D	=	column diameter
D''	=	diameter of transverse hoop or spiral measured to the center line of the hoop
D_r	=	diameter of the circle passing through the centers of the longitudinal reinforcing steel
E_s	=	modulus of elasticity of steel
f'_c	=	concrete compressive strength
f'_{ce}	=	expected concrete compressive strength
f_s	=	average volumetric transverse reinforcement stress
f_y	=	longitudinal reinforcement yield strength
f_{yt}	=	transverse reinforcement yield strength
$F_{0.004}$	=	idealized effective force at concrete strain of 0.004
F_{80}	=	80% of the absolute maximum effective force
F_{max}	=	maximum absolute effective force
F_{yi}	=	idealized effective force at calculated first yield point

h	=	height of the column cross-section
H	=	total height of the column
jd	=	internal lever arm of the concrete compression member
k_{nl}	=	shear strength degradation coefficient for nonlinear deformations in the Sezen & Moehle (2004) model
l_{north}	=	vertical displacement recorded from the rotational pot at the north face of the column at a determined height
l_{south}	=	vertical displacement recorded from the rotational pot at the south face of the column at a determined height
L_{pots}	=	horizontal distance between rotational pots' measurement points
L	=	distance from the critical section to the point of contraflexure in the column
M	=	moment at the critical section
$M_{0.004}$	=	idealized moment at concrete strain of 0.004 from moment-curvature analysis
n_l	=	number of longitudinal reinforcement bars
P	=	axial load (positive for compression)
$P\Delta_{type}$	=	lateral force-displacement history type for $P\Delta$ correction as discussed in 3.3.
s	=	spacing of the transverse reinforcement
s_x	=	crack spacing parameter, taken as the lesser of d_v or the maximum distance between layers of longitudinal crack control reinforcement per the AASHTO Bridge Design Specifications (2024)

s_{xe}	= crack spacing parameter as influenced by aggregate size in the AASHTO Bridge Design Specifications (2024)
V	= shear at the critical section
V_c	= shear strength attributed to the concrete
V_{Col0}	= shear strength of reinforced concrete columns at displacement ductility demands lower than 2.0 in ACI 369.1-22
V_{meas}	= peak shear force measured during the test
V_n	= nominal shear strength
V_p	= shear strength attributed to the axial load
V_s	= shear strength attributed to the transverse reinforcement
α	= concrete shear strength modification factor for aspect ratio in the Kowalsky et al. (2000) model
α'	= concrete shear stress adjustment factor in AASHTO LRFD-SBD (2023)
α_{Col}	= transverse reinforcement shear strength effectiveness contribution parameter in ACI 369.1-22.
α_v	= angle of inclination of the transverse reinforcement to longitudinal axis
β	= concrete shear strength modification factor for longitudinal reinforcement ratio in the Kowalsky et al. (2000) model
β_{AASHTO}	= factor for diagonally cracked concrete ability of transmitting tension and shear in the AASHTO Bridge Design Specifications (2024)

γ	=	concrete shear strength modification factor for ductility demand in the Kowalsky et al. (2000) model
γ_{req}	=	concrete shear strength modification factor for ductility demand required in the Kowalsky et al. (2000) model for $V_n = V_{meas}$
δ_{duct}	=	duct diameter correction factor, taken as 2.0 for grouted ducts in the AASHTO Bridge Design Specifications (2024)
$\delta_{p,i}$	=	incremental plastic displacement incursion at datapoint (i)
Δ_{80}	=	displacement corresponding to the last data point where the column resisted 80% of the absolute maximum effective force
Δ_i	=	displacement at data point (i)
Δ_p	=	peak plastic displacement ($\Delta_u - \Delta_y$)
$\Delta_{post-80}$	=	absolute maximum displacement after Δ_{80}
$\Delta_{post-zero}$	=	maximum displacement following another zero crossing after Δ_{80}
Δ_u	=	displacement at failure
Δ_y	=	idealized yield displacement
Δ_{yi}	=	displacement at initial yield
θ_{cr}	=	crack average angle to the vertical axis from the effective depth to the neutral axis
θ_{str}	=	diagonal compression strut average angle to the vertical axis
λ	=	modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal weight concrete of the same compressive strength in ACI 369.1-22 and ACI 318-25

λ_{AASHTO}	=	modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal weight concrete of the same compressive strength in AASHTO Bridge Design Specifications (2024)
λ_{duct}	=	shear strength reduction factor accounting for the presence of post-tensioning duct in AASHTO Bridge Design Specifications (2024)
λ_s	=	size effect modification factor in ACI 318-25
Λ	=	fixity factor
μ_{Δ}	=	displacement ductility
μ_{ϕ}	=	curvature ductility
v_c	=	adjusted concrete shear stress in AASHTO LRFD-SBD (2023)
ρ_l	=	longitudinal reinforcement ratio $\left(\frac{A_s}{A_g}\right)$
ρ_t	=	transverse reinforcement volumetric ratio: $\left(\frac{4 A_{hoop}}{sD''}\right)$ for circular columns
ρ_v	=	volumetric ratio of spiral or hoop reinforcement in circular columns $\left(\frac{2A_{hoop}}{sD''}\right)$ for circular columns
ρ_w	=	ratio of longitudinal reinforcement to bd
ξ	=	softening coefficient of reinforced concrete (after cracking) in Suselo et al. (2025) model
ϕ_{duct}	=	diameter of post-tensioning duct present in the girder web within depth d_v

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CHAPTER 1: INTRODUCTION

1.1. RESEARCH MOTIVATION

According to a database of 582 major highway bridges (Liu et al., 2022), approximately 68% of the bridges in Washington State are supported by at least one reinforced concrete column. Most of these bridges were constructed before the mid-1970's and, under the design provisions in place at that time, typically had low levels of transverse reinforcement to confine the column's concrete and help resist shear forces. The potential for shear or flexure-shear failure of these columns is of great concern because such failures can lead to a loss of axial load capacity and, in turn, to bridge collapse during an earthquake (Moehle & Eberhard, 1999).

The presence of the Cascadia Subduction Zone further contributes to the fragility of the bridge inventory. The Cascadia Subduction Zone is a megathrust fault located at the convergent boundary between the Juan de Fuca and North American plates (see Figure 1.1). The fault extends approximately 800 miles from Northern California to North Vancouver Island, British Columbia (McCrory et al., 2012). Geological evidence indicates that the most recent large-magnitude earthquake occurred in 1700 CE (Atwater et al., 2005), and the ongoing strain accumulation suggests the potential for a future large-magnitude megathrust event on the order of Mw 9. Such an earthquake would be characterized by long-duration ground motion (McCaffrey et al., 2013), which can subject structural components to a large number of loading cycles, a condition that has been shown to influence damage accumulation and deformation capacity (Ranf et al., 2006).

In this context, the accuracy of existing shear strength models needs to be revisited. These models were developed from experimental data on columns that, in many cases, are not representative of older bridges and were subjected to relatively limited cyclic demands. In contrast, long-duration ground motions, such as those associated with subduction earthquakes, may impose higher cyclic deformation demands, potentially accelerating shear strength degradation. Consequently, two columns subjected to similar peak displacement ductility demands may experience substantially different cumulative plastic displacement histories, depending on the earthquake source and magnitude.

Therefore, there is a need to evaluate the response of older reinforced concrete columns under such loading conditions to better understand their behavior and the potential implications for bridge infrastructure in Washington State and other regions subjected to long-duration ground motions.

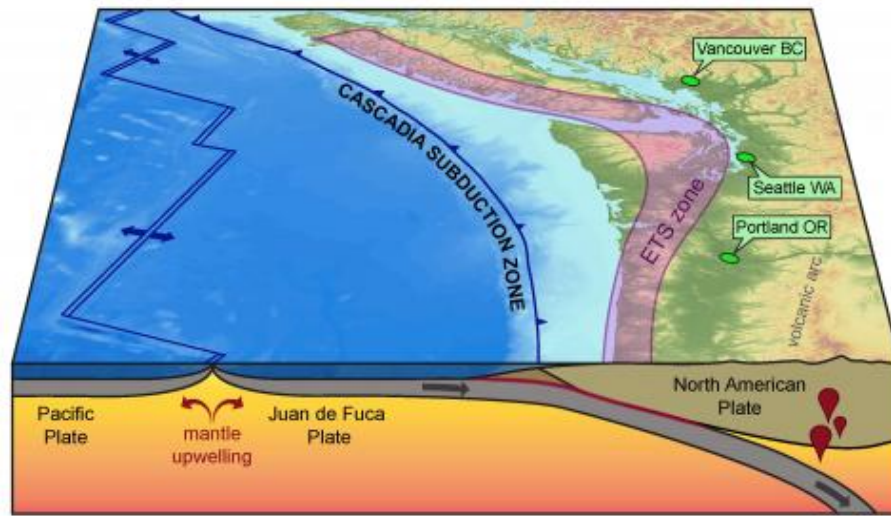


Figure 1.1 Cascadia Subduction Zone (USGS, 2020)

1.2. OBJECTIVES

The objective of this research is to evaluate the applicability of current shear strength models for older reinforced concrete columns subjected to long-duration lateral demands, and if necessary, to modify these models to account for the amount of cycling. To achieve this, the following objectives were developed:

- i. Identify reinforced concrete column tests with design properties and axial demands representative of pre-1975 WSDOT bridge columns.
- ii. Conduct tests of three poorly confined concrete columns under large cumulative plastic demands and subjected to high levels of shear stress.
- iii. Analyze the influence of ductility and cumulative plastic displacement on shear strength degradation for columns exhibiting shear and flexure-shear failure modes.

1.3. THESIS ORGANIZATION

This thesis is organized as follows:

- Chapter 2 reviews existing shear strength models for spiral- and hoop-reinforced concrete columns and discusses their assumptions and limitations.
- Chapter 3 reviews prior experimental tests representative of older bridge columns and introduces the research gap that this study addresses.
- Chapter 4 describes the experimental program, including the test matrix, loading setup, and instrumentation.
- Chapter 5 summarizes the damage observed during the tests.
- Chapter 6 includes the data processing procedure and the measured experimental response.
- Chapter 7 evaluates the accuracy of selected reinforced-concrete column shear-strength models and investigates the influence of cycling on shear-strength degradation.
- Finally, Chapter 8 presents the study's conclusions and provides recommendations for future research.

CHAPTER 2: SHEAR STRENGTH MODELS FOR CIRCULAR BRIDGE COLUMNS

2.1. OVERVIEW

Current shear strength models for circular reinforced concrete columns represent shear strength as the sum of contributions from the axial load, concrete, and transverse reinforcement, commonly idealized using a combination of truss and compression strut mechanisms. The models reviewed in this chapter are the “modified UCSD model” by Kowalsky and Priestley (2000), Sezen & Moehle (2004), Suselo et al. (2025), and those included in AASHTO, ACI, ASCE, and FHWA standards. These methods are described in the following sections. For clarity and consistency, notation has been standardized throughout this thesis.

2.2. KOWALSKY AND PRIESTLEY (2000)

The Kowalsky and Priestley (2000) shear strength model was proposed as a revision to the “UCSD shear model” developed by Priestley et al., (1994). Both original and modified models define the nominal shear strength of reinforced concrete members (V_n) as the sum of three separate components, as shown in Eq. 2.1: the shear strength attributed to the steel truss mechanism (V_s), the strength attributed to the axial load (V_p), and the strength of the concrete shear-resisting mechanism (V_c).

$$V_n = V_s + V_p + V_c \tag{2.1}$$

According to the model authors, the revised version accounts for the effect of the concrete compression zone on the mobilization of the transverse steel, as well as the effects of aspect ratio and longitudinal steel ratio on the strength of the concrete shear-resisting mechanism.

This model was calibrated using tests of 47 circular reinforced concrete columns that exhibited brittle shear failures (displacement ductility less than 2), ductile shear failures (displacement ductility greater than 2), and ductile flexural failures.

2.2.1. CONCRETE SHEAR-RESISTING MECHANISM (V_c)

The revised UCSD model assumes that the primary contribution to the concrete shear-resisting mechanism in flexurally cracked members is provided by aggregate interlock along rough shear cracks (Priestley et al., 2017). The model considers that the concrete component V_c varies as a function of the aspect ratio, longitudinal steel ratio, and ductility demand as defined in Eq. (2.2):

$$V_c = \alpha\beta\gamma\sqrt{f'_c}(0.8A_g) \quad (2.2)$$

where f'_c refers to the measured concrete compressive strength and A_g is the cross-sectional area.

According to Kowalsky and Priestley (2000), shear strength is greater for columns with smaller aspect ratios due to the confinement provided at the column ends by adjacent members. Therefore, it includes the factor α in Eq. (2.2) that accounts for the column aspect ratio as given in Eq. (2.3):

$$\alpha = 3 - \frac{M}{VD}; 1.0 \leq \alpha \leq 1.5 \quad (2.3)$$

where M and V are the moment and shear at the critical section, respectively, and D is the column diameter. For a cantilever or double-curvature column test, this ratio is equivalent to the aspect ratio L/D , where L refers to the distance from the critical section to the point of contraflexure in the column.

Additionally, lower longitudinal steel ratios are assumed to decrease concrete shear strength due to (i) a reduced dowel action from the longitudinal reinforcement, (ii) a more concentrated crack distribution which results in fewer, more widely spaced cracks that reduce the concrete aggregate interlock mechanism, and (iii) a smaller compression zone that results in a reduced compression shear transfer.

Thus, the model includes the factor β in Eq. (2.2), which accounts for the longitudinal steel ratio (ρ_l) as given in Eq. (2.4).

$$\beta = 0.5 + 20 \rho_l \leq 1.0 \quad (2.4)$$

The concrete's shear strength contribution is assumed to degrade with increasing ductility due to reduced aggregate interlock as cracks widen. Therefore, the model includes the factor γ in Eq. (2.2), a parameter that captures this behavior, as shown in Figure 2.1, which relates the factor γ to displacement ductility (μ_Δ).

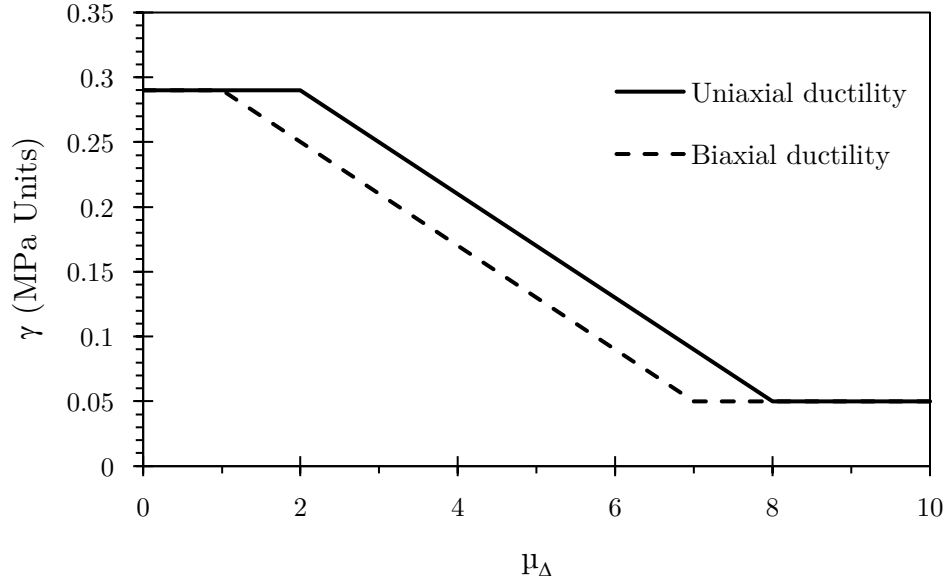


Figure 2.1 γ factor – Revised UCSD shear model (adapted from Kowalsky and Priestley, 2000)

2.2.2. AXIAL LOAD COMPONENT (V_p)

The revised UCSD model accounts for the shear strength enhancement provided by axial load as an independent component resulting from the horizontal component of a diagonal compression strut at an average θ_{str} angle. As shown in Figure 2.2, this strut forms between the column centroid at the top of the column and the center of the compression zone at the bottom for cantilever columns, or between the centers of the concrete compression zones for double-curvature columns.

The following equations define V_p :

$$V_p = \begin{cases} P \tan \theta_{str} = P \left(\frac{D - c}{2L} \right), & P > 0 \\ 0, & P < 0 \end{cases} \quad (2.5)$$

where P is the axial load including seismic effects (positive for compressive loads) and c is the neutral axis depth. This model does not account for the degradation of the axial load contribution as ductility demand increases.

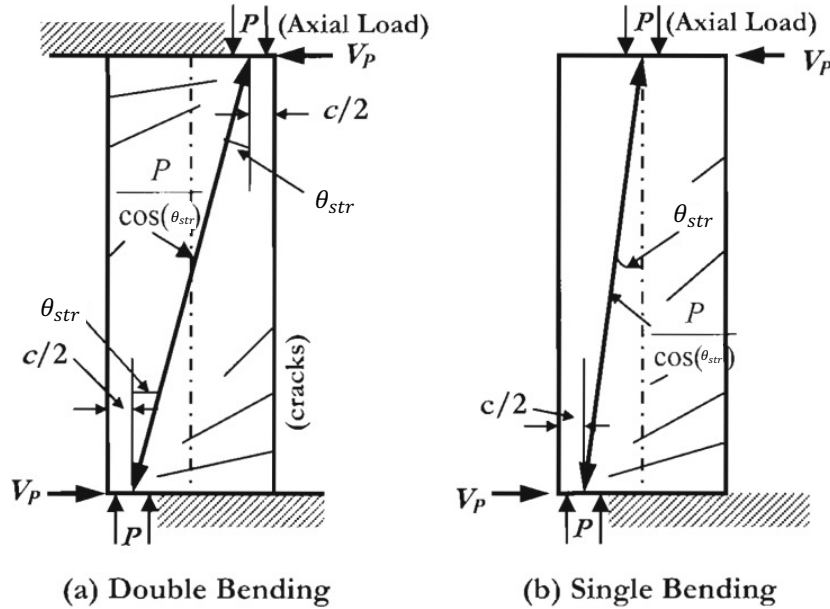


Figure 2.2 Contribution of axial load to column shear strength (adapted from Priestley et al., 2017)

2.2.3. STEEL TRUSS MECHANISM (V_s)

As shown in Figure 2.3, this model treats the transverse steel contribution to the column's shear strength as a truss mechanism that develops when the critical flexure-shear crack crosses the section at an average angle θ_{cr} to the vertical axis from the effective depth to the neutral axis. The maximum force transferred by a layer of transverse reinforcement varies as a function of its yield strength (f_{yt}) and the effective area with respect to the axis along which the shear is applied.

For a circular column, the authors of the model defined a simplified version that considers the effective area of the transverse reinforcement (either circular hoops or spirals) and the average number of hoops intersected by the inclined crack, as shown in Eq. (2.6), where A_v is the area of the hoop bar and s is the spacing of hoops or spirals along the member's axis.

$$V_s = \frac{\pi A_{hoop} f_{yt} (D - c - c_c) \cot(\theta_{cr})}{2s} \quad (2.6)$$

Priestley et al., (2017) comment that the standard practice in the U.S. adopts an angle of $\theta_{cr} = 45^\circ$, which can be overly conservative. They state that it has been found that using $\theta_{cr} = 35^\circ$ yields sufficiently conservative designs, and, according to Priestley et al. (1996), considering $\theta_{cr} = 30^\circ$ for evaluating existing structures is adequate. In this work, it was assumed that $\theta_{cr} = 30^\circ$ for this model.

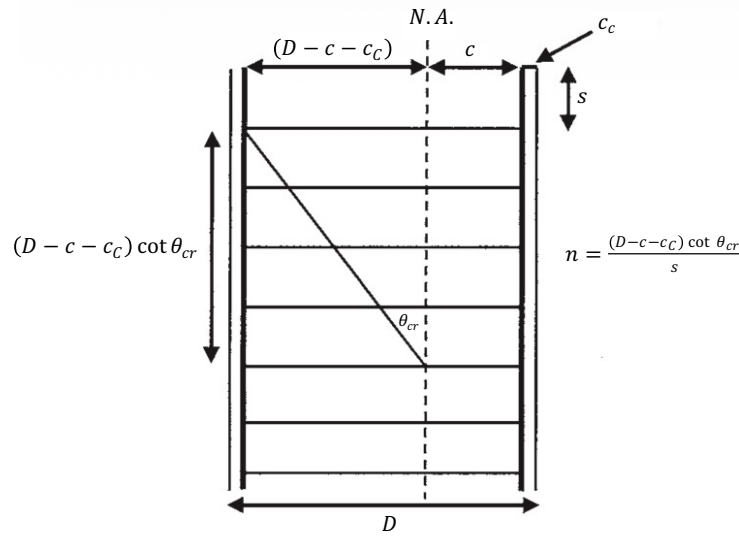


Figure 2.3 Effect of concrete compression zone on truss mechanism (adapted from Kowalsky and Priestley, 2000)

2.3. FHWA SEISMIC RETROFIT MANUAL FOR HIGHWAY STRUCTURES (2006)

The Federal Highway Administration (FHWA) Seismic Retrofit Manual for Highway Structures is a technical report developed for the FHWA by Buckle et al. (2006) and will be referred to as FHWA-SRM. It contains a shear strength model similar to that proposed by Kowalsky and Priestley (2000) that considers the total shear strength as the sum of the diagonal tension field in the concrete (V_c), truss action in the reinforcing steel (V_s), and strut action derived from the axial load (V_p), as shown in Eq. 2.1.

The calculations of V_s and V_p are defined in Eqs. 2.7 - 2.12. Most of these equations follow a formulation similar to that of Kowalsky and Priestley (2000).

$$V_s = A_v f_{yt} \frac{D''}{s} \cot \theta \quad (2.7)$$

$$\tan \theta_{cr} = \left(\frac{1.6 \rho_v A_e}{\Lambda \rho_l A_g} \right)^{0.25} \geq \tan \theta_{str} \quad (2.8)$$

$$\Lambda = \begin{cases} 2, & \text{for fixed - fixed columns} \\ 1, & \text{for fixed - pinned columns} \end{cases} \quad (2.9)$$

$$A_e = 0.8 A_g \quad (2.10)$$

$$\tan \theta_{str} = \frac{jd}{H} \quad (2.11)$$

$$V_p = \frac{\Lambda}{2} P \tan \theta_{str} \quad (2.12)$$

As for V_c , this model defines two distinct concrete contributions, depending on the column's plastic hinge rotation demands. The first is an initial concrete shear-strength component (V_{ci}) that retains the unreduced contribution (see Eq. 2.13) and applies for curvature ductility demands lower than 3. The second one (V_{cf}) applies for curvature ductility demands greater than 8, when, due to widely spaced tensile cracks and the yielding of longitudinal bars, the concrete shear-strength contribution is reduced (see Eq. 2.14). For curvature ductility demands between 3 and 8, the FHWA-SRM model assumes that shear strength degrades linearly from the initial-condition shear strength to the final-condition value

For initial conditions:

$$V_c = V_{ci} = 0.3 \sqrt{f'_{ce}} A_e \quad (2.13)$$

For final conditions:

$$V_c = V_{cf} = 0.05 \sqrt{f'_{ce}} A_e \quad (2.14)$$

These reduction factors closely match the γ reduction-factor boundaries for low and high displacement ductility demands in the Kowalsky and Priestley (2000) model; a notable difference is that the FHWA-SRM model uses curvature ductility for the parameter and different reduction-triggering values.

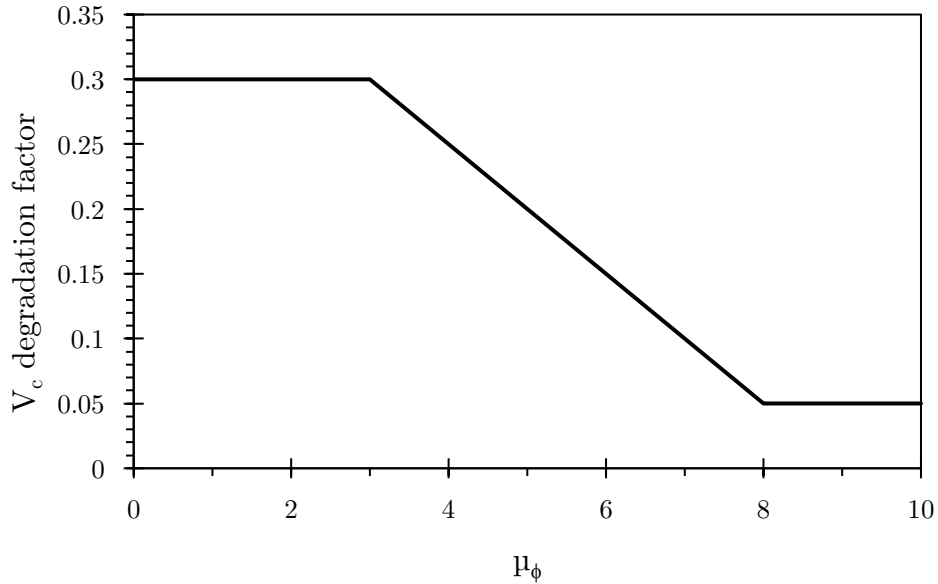


Figure 2.4 Concrete shear strength degradation factor according to FHWA-SRM

2.4. ACI 369.1-22 (2023)

The American Concrete Institute (ACI) Code for the Seismic Evaluation and Retrofit of Existing Concrete Buildings (ACI 369.1-22) defines a shear strength model that accounts for the interaction between the contributions of the concrete (V_c) and transverse reinforcement (V_s) to calculate the shear strength of a reinforced concrete column, as shown in Eq. 2.15, which applies a shear strength degradation factor (k_{nl}) that decreases with increasing displacement ductility demand.

$$V_n = k_{nl} V_{col0} = k_{nl} (V_c + V_s) \quad (2.15)$$

This shear strength equation is a modified version of the concrete column shear strength model developed by Sezen & Moehle (2004). These modifications include the addition of a lightweight concrete modification factor (λ) and an effectiveness of transverse

reinforcement in resisting shear forces factor (α_{col}), both discussed in Sections 2.4.1 and 2.4.2, respectively.

Originally, the Sezen & Moehle (2004) model was calibrated using experimental data collected from 51 rectangular reinforced columns subjected to unidirectional lateral forces parallel to one face of the column; shear span-to-depth ratios, $2 \leq L/d \leq 4$; concrete strength $13 \leq f'_c \leq 45$ MPa; longitudinal and transverse reinforcement yield stress, f_y and f_{yt} , from 300 to 650 MPa; longitudinal reinforcement ratio, $0.01 \leq \rho_l \leq 0.04$; transverse reinforcement index, $0.01f'_c \leq \rho_t f_{yt} \leq 0.12f'_c$; and reported shear distress at failure.

2.4.1. CONCRETE SHEAR STRENGTH CONTRIBUTION (V_c)

According to Sezen & Moehle (2004), their model assumes that the contribution of concrete to shear strength is determined by the shear stress to initiate its diagonal tension cracking capacity, which increases proportionally to the compressive axial load in the column. The model authors mention that it tends to overestimate the inclined cracking load, particularly for columns with larger shear span-to-depth ratios, because shear strength decreases with increasing aspect ratio.

The concrete shear strength contribution component (V_c), as defined in ACI 369.1-22, is given by Eqs. 2.16a and 2.16b. In contrast to its original version in Sezen & Moehle (2004), it now includes a λ reduction factor to account for the use of lightweight concrete. This factor is 0.75 for lightweight aggregate concrete and 1 for all other cases. The axial load (P) is considered equal to zero if it is causing axial tension.

In SI units, the equation for V_c takes the following form:

$$V_c = \lambda \left(\frac{0.5\sqrt{f'_c}}{\frac{M}{Vd}} \sqrt{1 + \frac{P}{0.5 A_g \sqrt{f'_c}}} \right) 0.8 A_g \quad (\text{N}) \quad (2.16a)$$

Using USCS units, the equation for V_c is:

$$V_c = \lambda \left(\frac{6\sqrt{f'_c}}{\frac{M}{Vd}} \sqrt{1 + \frac{P}{6 A_g \sqrt{f'_c}}} \right) 0.8 A_g \quad (\text{lb}) \quad (2.16b)$$

2.4.2. TRANSVERSE REINFORCEMENT CONTRIBUTION (V_s)

The transverse reinforcement contribution is defined as a simplified truss model in which the transverse reinforcement serves as a tension member within the shear-resisting mechanism.

Compared to its original version, the current definition of V_s in ACI 369.1-22 (Eq. 2.18) includes a parameter for the effective contribution of transverse reinforcement to shear strength (α_{col}), which is related to the spacing-to-effective depth ratio. This parameter degrades V_s when the transverse reinforcement spacing exceeds $0.75d$ (See Eq. 2.17).

$$\alpha_{col} = -4 \left(\frac{s}{d} \right) + 4 ; 0.00 \leq \alpha_{col} \leq 1.0 \quad (2.17)$$

$$V_s = \alpha_{col} \left(\frac{A_v f_{yt} d}{s} \right) \quad (2.18)$$

2.4.3. SHEAR STRENGTH DEGRADATION FACTOR (k_{NL})

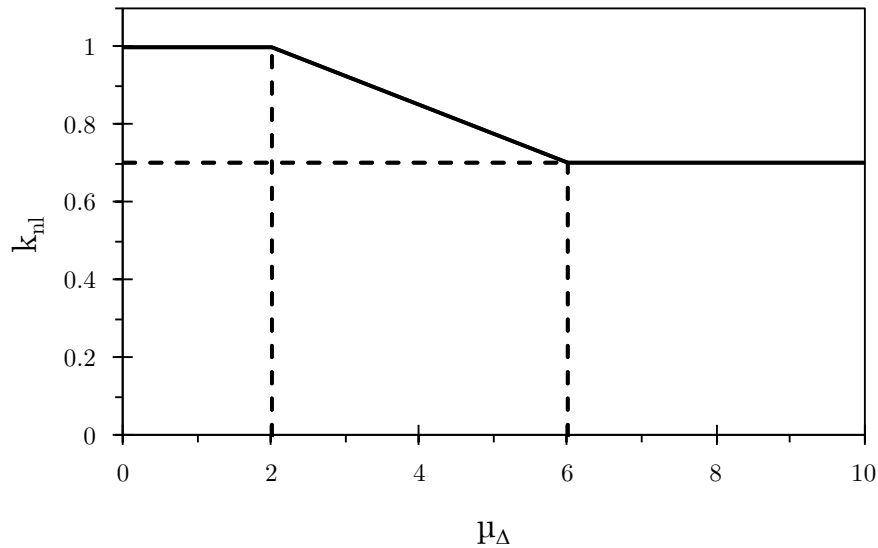


Figure 2.5 Shear strength degradation relation (adapted from Moehle, 2014)

In contrast to other empirical models, such as Kowalsky and Priestley (2000), this model assumes that the overall shear-resisting mechanism degrades as flexural deformation demand increases. This consideration stems from the degradation of V_c through crack openings and the degradation of V_s due to the loss of anchorage of the transverse

reinforcement as the concrete becomes more damaged. The effects of shear strength degradation due to inelastic cyclic loading are taken into account by the parameter k_{nl} , which is defined to be equal to 1.0 for displacement ductility demands lower than 2, to be equal to 0.7 for displacement ductility (μ_Δ) demands greater than 6, and to vary linearly for intermediate ductility demands (Sezen & Moehle, 2004). This variation is shown in Figure 2.5.

2.5. AASHTO GUIDE SPECIFICATIONS FOR LRFD SEISMIC BRIDGE DESIGN (2023)

The AASHTO Guide Specifications for LRFD Seismic Bridge Design were developed by the American Association of State Highway and Transportation Officials (AASHTO) and will be referred to as (AASHTO LRFD-SBD). The specification defines the nominal shear strength of a reinforced concrete column as the sum of the concrete and transverse reinforcement contributions to it, as shown in Eq. 2.19.

$$V_n = V_c + V_s \quad (\text{kip}) \quad (2.19)$$

This model considers degradation of the concrete shear-strength contribution as a function of the transverse reinforcement volumetric ratio and displacement ductility demand; the latter is defined to vary with the assessed Seismic Design Category (SDC). The calculations are shown in Eqs. 2.20 - 2.26.

$$V_c = v_c A_e \quad (\text{kip}) \quad (2.20)$$

$$A_e = 0.8 A_g \quad (\text{in}^2) \quad (2.21)$$

$$v_c = \begin{cases} 0.032\alpha' \left(1 + \frac{P}{2A_g}\right) \sqrt{f'_c} \leq \min \begin{cases} 0.11 \sqrt{f'_c} \\ 0.047\alpha' \sqrt{f'_c} \end{cases}, & P > 0 \\ 0, & P < 0 \end{cases} \quad (2.22)$$

(f'_c in ksi)

$$\alpha' = \frac{f_s}{0.15} + 3.67 - \mu_\delta \quad (2.23)$$

$$f_s = \rho_t f_{yt} \leq 0.35 \quad (2.24)$$

(f_{yt} in ksi)

$$\rho_t = \frac{4 A_{hoop}}{s D''} \quad (2.25)$$

$$\mu_\Delta = \begin{cases} 2, & SDC \ B \\ 3, & SDC \ C \\ 1 + \frac{\Delta_p}{\Delta_y}, & SDC \ D \end{cases} \quad (2.26)$$

The shear-strength contribution from the transverse reinforcement is calculated with Eq. 2.27.

$$V_s = \frac{\pi}{2} \left(\frac{n A_{hoop} f_{yt} D''}{s} \right) \quad (\text{kip}) \quad (2.27)$$

(f_{yt} in ksi)

It is important to note that these specifications state that the shear provisions in the AASHTO LRFD Bridge Design Specifications described in 2.6 are not applicable to sections expected to accommodate significant plastic deformation. (AASHTO, 2023)

2.6. AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS (2024)

The AASHTO LRFD Bridge Design Specifications were developed by AASHTO and will be referred to as (AASHTO LRFD-BDS). It does not include a designated chapter for the evaluation of existing structures; hence, the following model refers to Section 5 about the design of concrete structures in the AASHTO LRFD-BDS, which, as discussed in Section 2.9.3, is required for the retrofit of Recovery and Critical bridges in Washington State.

In these specifications, the nominal shear strength model for reinforced concrete columns is directly influenced by their aspect ratio. Columns with an aspect ratio greater than 2.5 are subject to column considerations, while those with an aspect ratio lower than 2.5 are subjected to pier ones.

Considering that the scope of this study is focused on reinforced concrete columns, which are not usually prestressed members, the factors related to prestressing forces in the shear strength were, when possible, omitted in the following equations.

For columns with an aspect ratio above 2.5, these provisions establish the shear strength calculations shown in Eqs. 2.28 - 2.38, which are based on the Modified Compression-Field Theory (MCFT) proposed by Vecchio & Collins (1986)

$$V_n = \text{lesser of} \left\{ \begin{array}{l} V_c + V_s \\ 0.25f'_c b d_v \end{array} \right. \quad (\text{lb}) \quad (2.28)$$

$$d_v = 0.9 d_e \quad (\text{in}) \quad (2.29)$$

$$d_e = \frac{D}{2} + \frac{D_r}{\pi} \quad (\text{in}) \quad (2.30)$$

$$V_c = 0.0316 \beta_{AASHTO} \lambda_{AASHTO} \sqrt{f'_c} b d_v ; P > 0.1 f'_c A_g \quad (\text{lb}) \quad (2.31)$$

(If $P < 0.1 f'_c A_g$, V_c will decrease linearly to zero at $P = 0$)

$$V_s = \frac{A_v f_y d_v (\cot \theta_{cr} + \cot \alpha_v) \sin \alpha_v}{s} \lambda_{duct} \quad (\text{lb}) \quad (2.32)$$

For regular weight concrete: (2.33a)

$$\lambda_{AASHTO} = 1$$

For lightweight concrete where f_{ct} is not specified: (2.33b)

$$0.75 \leq \lambda_{AASHTO} = 7.5 w_c \leq 1.0$$

For lightweight concrete where f_{ct} is specified: (2.33c)

$$\lambda_{AASHTO} = 4.7 \frac{f_{ct}}{\sqrt{f'_c}}$$

$$\lambda_{duct} = 1 - \delta_{duct} \left(\frac{\phi_{duct}}{b} \right)^2 \quad (2.34)$$

$$\beta_{AASHTO} = \begin{cases} \frac{4.8}{(1 + 750\varepsilon_s)}, & A_v \geq A_{v,min} \\ \frac{4.8}{(1 + 750\varepsilon_s)} \frac{51}{(39 + s_{xe})}, & A_v < A_{v,min} \end{cases} \quad (2.35)$$

$$\theta_{cr} = 29 + 3500 \varepsilon_s \quad (^\circ) \quad (2.36)$$

$$\varepsilon_s = \frac{\frac{|M|}{d_v} + 0.5P + |V|}{E_s A_s} \quad (\text{in./in.}) \quad (2.37)$$

$$s_{xe} = \frac{1.38s_x}{a_g + 0.63} \quad (\text{in.}) \quad (2.38)$$

For the case of reinforced concrete columns with an aspect ratio lower than 2.5, these provisions refer to the wall-type pier section, whose nominal shear strength definition is included in Eq. 2.39. Nevertheless, it also states that for these cases, the forces in this section would not apply, considering that the forces resulting from plastic hinging will generally exceed the elastic design forces.

$$V_n = 0.063\lambda_{AASHTO}\sqrt{f'_c}bd \quad (\text{lb}) \quad (2.39)$$

Unlike other models, this one does not explicitly account for any degradation factor for ductility demand. The only reduction factor we find applies to axial load ratios below 10%.

2.7. ACI 318-25 (2025)

The ACI Building Code for Structural Concrete (ACI 318-25) is an alternative shear strength calculation method to ACI 369.1-22 for sections that comply with Chapter 18 of this code on seismic requirements. Note that older columns are unlikely to meet the minimum transverse reinforcement requirements of Chapter 18.

In this model, the nominal shear strength calculations for non-prestressed members are included in Eqs. 2.40 - 2.44. No shear strength degradation is considered in this model.

$$V_n = V_c + V_s \quad (\text{lb}) \quad (2.40)$$

$$V_c = \begin{cases} \left(2\lambda\sqrt{f'_c} + \frac{P}{6A_g}\right)bd, & A_v \geq A_{v,min} \\ \left[8\lambda\left(\rho_w^{\frac{1}{3}}\right)\sqrt{f'_c} + \frac{P}{6A_g}\right]bd, & A_v \geq A_{v,min} \\ \left[8\lambda\lambda_s\left(\rho_w^{\frac{1}{3}}\right)\sqrt{f'_c} + \frac{P}{6A_g}\right]bd, & A_v < A_{v,min} \end{cases} \quad (1b) \quad (2.41)$$

$$\lambda\sqrt{f'_c}bd < V_c < 5\lambda\sqrt{f'_c}bd$$

$$\rho_w = \frac{A_s}{bd} \quad (2.42)$$

$$\lambda_s = \sqrt{\frac{2}{1 + \frac{d}{10}}} \leq 1 \quad (2.43)$$

$$V_s = \frac{A_v f_{yt} d}{s} \quad (1b) \quad (2.44)$$

2.8. SUSELO ET AL. (2025)

This shear strength model was proposed by Suselo et al. (2025) as a revision to the shear strength model included in the ACI 369.1-22 and its application to both slender and short columns. It proposes three approaches as an updated version for the shear strength V_{col} as a function of aspect ratio: one for slender columns ($L/d > 2$), one for short columns ($L/d < 1.5$), and a weighted interpolation for the transition region ($1.5 < L/d \leq 2.5$) to eliminate discontinuities in strength and strength limits between the two first ones.

The model was calibrated using a subset of an expanded version of the ACI 369.1 rectangular column database, which included 94 experimental tests of rectangular, tie-reinforced columns that exhibited shear strength degradation prior to flexural yielding. These tests focused primarily on effective depths below 24 in and shear span-to-depth ratios below 2 (~62 tests), followed by specimens with aspect ratios between 2 and 4 (~26 tests). Axial load ratios ranged from 0 to 0.8 (concentrated in tests below 0.5), transverse

reinforcement volumetric ratios were concentrated in 0-0.4%, and longitudinal reinforcement ratios were concentrated between 2 and 4%.

Considering that the Suselo et al. (2025) ACI 369.1-22 update proposal is modifying the V_{col} calculations, it is not clear whether the nominal shear strength of the column (V_n) would still be affected by the parameter k_{nl} , which was not mentioned in the referred update, as well as the inclusion of the λ parameter for lightweight aggregate concrete considerations. Given that the proposed update does not explicitly redefine or include these parameters, for consistency with ACI 369.1-22, it is assumed that this shear strength model is intended to update shear strength calculations for $\mu_\theta < 2$ (V_{col0}), and that no λ factor is considered.

2.8.1. SHEAR STRENGTH OF SLENDER COLUMNS

Suselo et al. (2025) consider that, when assessing the shear strength of columns with an aspect ratio greater than 2, the mean principal tensile stress approach included in the ACI 369.1-22 method for shear strength is the most precise for slender columns according to their mean strength results. In this case, some of the modifications proposed to V_{col0} include a lower bound for V_s related to tension loads as well as an upper bound of $0.83\sqrt{f'_c}bd$ (MPa) ($10\sqrt{f'_c}bd$ [psi]) to reflect the maximum strength of slender columns in their database and to maintain consistency with Chapter 18 of ACI 318-25 (See Eqs. 2.45a and 2.45b).

Regarding the concrete contribution, the modifications proposed consider a higher diagonal tensile strength $0.623\sqrt{f'_c}$ (MPa) ($7.5\sqrt{f'_c}$ [psi]) in the V_c equation, along with an upper bound of $0.42\sqrt{f'_c}bd$ (N) ($5\sqrt{f'_c}bd$ [lb]) as shown in Eqs. 2.46a and 2.46b.

In SI units, the equations for $V_{Col0,slender}$ and V_c take the following form:

$$\begin{aligned} V_{Col0,slender} &= V_c + V_s \\ &\leq 0.83\sqrt{f'_c}bd \quad (N) \quad (2.45a) \\ &\geq V_s \text{ for tension axial load} \end{aligned}$$

$$V_c = \left(\frac{0.623\sqrt{f'_c}}{L/d} \sqrt{1 + \frac{P}{0.623 A_g \sqrt{f'_c}}} \right) 0.8 A_g \leq 0.415\sqrt{f'_c} \quad ; \quad (N) \quad (2.46a)$$

$$\left(1 + \frac{P}{0.623 A_g \sqrt{f'_c}}\right) \geq 0$$

In USCS units, the equations for $V_{Col0,slender}$ and V_c take the following form:

$$\begin{aligned} V_{Col0,slender} &= V_c + V_s \\ &\leq 10\sqrt{f'_c}bd \\ &\geq V_s \text{ for tension axial load} \end{aligned} \quad (1b) \quad (2.45b)$$

$$\begin{aligned} V_c &= \left(\frac{7.5\sqrt{f'_c}}{L/d} \sqrt{1 + \frac{P}{7.5 A_g \sqrt{f'_c}}} \right) 0.8 A_g \leq 5\sqrt{f'_c} \quad ; \\ &\left(1 + \frac{P}{7.5 A_g \sqrt{f'_c}} \right) \geq 0 \end{aligned} \quad (1b) \quad (2.46b)$$

As for the transverse reinforcement contribution (Eqs. 2.47a and 2.47b) the authors introduced an upper bound and a variable-angle truss model into the equation to account for the reinforcement's contribution to shear strength, which requires estimating the shear crack angle (θ_{cr}).

In SI units, the equations for V_s take the following form:

$$V_s = \alpha_{Col} \left(\frac{A_v f_{yt} d}{s} \right) \cot(\theta_{cr}) \leq 0.664\sqrt{f'_c} \quad (N) \quad (2.47a)$$

In USCS units, the equations for V_s are:

$$V_s = \alpha_{Col} \left(\frac{A_v f_{yt} d}{s} \right) \cot(\theta_{cr}) \leq 8\sqrt{f'_c} \quad (1b) \quad (2.47b)$$

To estimate the shear crack angle (θ_{cr}), the model uses the AASHTO LRFD Bridge Design Specifications method, which introduces upper and lower bounds for this angle, as shown in Eq. 2.48. The introduction of this new factor modifies the definition of the α_{Col} parameter from the original ACI 369.1-22 model, as shown in Eq. 2.49.

Equations for θ_{cr} and α_{Col} are independent of the unit system and are defined as:

$$\theta_{cr} = 29 + 3500 \left[\frac{\left(1 + \frac{L}{d}\right) V - 0.5P}{E_s A_{sT}} \right]; 35^\circ \leq \theta \leq 75^\circ \quad (2.48)$$

(It is permissible to assume $\theta_{cr} = 35^\circ$)

$$\alpha_{col} = -4 \left(\frac{s}{d \cot(\theta_{cr})} \right) + 5; 0.0 \leq \alpha_{col} \leq 1.0 \quad (2.49)$$

2.8.2. SHEAR STRENGTH OF SHORT COLUMNS

Suselo et al. (2025) propose a shear strength model for reinforced concrete columns with an aspect ratio lower than 2 based on a modified version of the strut-and-tie model developed by Weng et al. (2017) for squat walls.

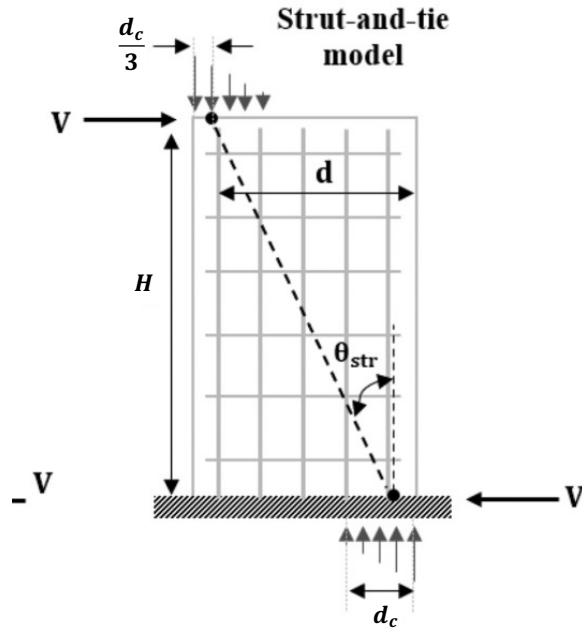


Figure 2.6 Strut-and-Tie model for short columns (Adapted from Suselo et al. (2025))

In their 2025 proposal, the model authors consider that shear strength is governed by the greater of the strength of diagonal struts oriented at an angle θ_{str} from the column's vertical axis (See Figure 2.6) and the strength of the variable-angle truss model defined in 2.8.1, which, for short columns, tends to be the first. That considered, along with a statistical analysis that yielded a constant coefficient of 1.85 to correlate the strut-and-tie index, ρ_t , f'_c , f_{yt} , and θ_{str} , the model authors developed Eqs. 2.50a - 2.54b to evaluate

the strength of the flexural compression block at both ends of the struts, accounting for the transverse steel contribution and concrete softening due to elongation (ξ).

In SI units, the column's shear strength equation is defined as:

$$V_{Col0,short} = 1.85 \xi f'_c A_{str} \sin(\theta_{str}) \quad \begin{aligned} &\leq \sqrt{f'_c} bd \\ &\geq V_s \text{ for tension axial load} \end{aligned} \quad \text{(N) (2.50a)}$$

In USCS units, the column's shear strength equation is:

$$V_{Col0,short} = 1.85 \xi f'_c A_{str} \sin(\theta_{str}) \quad \begin{aligned} &\leq 12\sqrt{f'_c} bd \\ &\geq V_s \text{ for tension axial load} \end{aligned} \quad \text{(lb) 2.50b}$$

Equations for the compressive strut are independent of the unit system and take the following form:

$$A_{str} = d_c b \quad (2.51)$$

$$d_c = \left(0.25 + 0.85 \frac{P}{A_g f'_c} \right) h \quad (2.52)$$

$$\theta_{str} = \tan^{-1} \left(\frac{h - \frac{2}{3} d_c}{H} \right) \quad (2.53)$$

In SI units, ξ is defined by:

$$\xi = \frac{3.35}{\sqrt{f'_c}} \leq 0.52 \quad \text{(N) (2.54a)}$$

In USCS units, ξ is defined as:

$$\xi = \frac{40}{\sqrt{f'_c}} \leq 0.52 \quad \text{(lb) (2.54b)}$$

The applicability of the proposed equations for θ_{cr} to cantilever columns is not explicitly addressed by the authors. The configuration illustrated in Figure 2.5 and geometric definitions are consistent with double-curvature idealization; however, this interpretation is inferred and not explicitly stated in the original publication.

2.8.3. SHEAR STRENGTH FOR COLUMNS IN THE TRANSITION REGION

In addition to the previous shear strength models, Suselo et al. (2025) propose Eq. 2.55 for reinforced concrete columns with an aspect ratio between 1.5 and 2.5, which they denote as the transition region. For this, they introduce a weighted-coefficient approach to the shear-strength calculation for these cases, providing a linear transition between the two shear-strength methods.

$$V_{Col0,transition} = \left(2.5 - \frac{L}{d}\right) V_{Col0,short} + \left(\frac{L}{d} - 1.5\right) V_{Col0,slender} \quad (2.55)$$

2.8.4. SHEAR STRENGTH FOR SPECIAL CASES

Finally, the model authors recommend the use of ACI 318 shear strength provisions (Chapter 22) as the only permitted method for columns with the following conditions: a) the effective depth exceeds 607 mm (24 in); b) the axial load is less than $0.15A_g f'_c$; and c) the amount of transverse reinforcement is less than $A_{v,min}$ as defined in Section 10.6.2.2 of ACI 318-25 (Eqs. 2.56a and 2.56b).

In SI units, $A_{v,min}$ is defined as:

$$A_{v,min} = \max \left(0.062 \sqrt{f'_c} \frac{b s}{f_{yt}} \quad , \quad 50 \frac{b s}{f_{yt}} \right) \quad (2.56a)$$

In USCS units, the equation takes the following form:

$$A_{v,min} = \max \left(0.75 \sqrt{f'_c} \frac{b s}{f_{yt}} \quad , \quad 50 \frac{b s}{f_{yt}} \right) \quad (2.56b)$$

2.9. ADOPTION OF MODELS FROM OTHER PROVISIONS

Some provisions do not include a shear strength model but instead reference the previously reviewed models as discussed next.

2.9.1. FHWA LRFD SEISMIC ANALYSIS AND DESIGN OF BRIDGES REFERENCE MANUAL (2014)

The FHWA LRFD Seismic Analysis and Design of Bridges Reference Manual is a technical report for the FHWA by Marsh et al. (2014) and does not introduce a new shear strength model; it refers to the one included in the AASHTO Guide Specifications for LRFD Seismic Bridge Design framework, which is discussed in 2.5.

2.9.2. ASCE/SEI 41-23 (2023)

The current version of the American Society of Civil Engineers (ASCE) Standard for Seismic Evaluation and Retrofit of Existing Buildings (ASCE/SEI 41-23) refers to the ACI 369.1-22.

2.9.3. WSDOT BRIDGE DESIGN MANUAL (2024)

Section 4.4 of the WSDOT Bridge Design Manual (WSDOT-BMD) states that the seismic retrofit of ordinary bridges shall comply with the FHWA-SRM, whose shear-strength model is described in 2.3.

As for the seismic retrofit of Recovery and Critical bridges, this Manual states that the project shall comply with the requirements set forth in WSDOT-BDM. Considering that these requirements refer to the new design section in the same manual, the shear design requirements for the retrofit of reinforced concrete columns refer to the “Simplified Procedure for Non-prestressed Sections” in AASHTO LRFD-BDS Section 5.7.3.4.1, which was discussed in 2.6.

2.10. LIMITATIONS OF EXISTING MODELS

Multiple models have been developed to assess the nominal shear strength on RC columns, considering contributions from the concrete, the transverse reinforcement, and, whether included in the concrete contribution or explicitly defined, the contribution of the compressive axial load on the column.

Most models account for the influence of key properties such as column aspect ratio, transverse reinforcement volumetric ratio, and the use of lightweight concrete. However,

not all models account for the degradation of shear strength due to displacement ductility demands on the element, particularly when the standard focuses on design rather than evaluation.

Among the models that account for this degradation effect, one thing to note is that models with statistically calibrated parameters, such as Kowalsky et al. (2000), ACI 369.1-22, and Suselo et al. (2025), depend heavily on the set of column experiments used to calibrate the shear-degradation factors. Hence, the applicability of these models to pre-1970's bridge column configurations should be assessed, particularly for overlapped hoops and low transverse reinforcement ratios, as those are unique to older bridges.

Additionally, the reviewed models primarily relate shear strength degradation to displacement ductility demand and geometric or reinforcement parameters. However, none explicitly considers the effect of cumulative plastic displacement demands on shear strength, despite the possibility that long-duration ground motions may impose large cyclic histories with similar peak displacement ductility demands but substantially different cumulative plastic deformation demands.

Therefore, this study evaluates whether, in addition to displacement ductility, the amount of cycling should be considered a parameter that influences shear-strength degradation in shear-strength models.

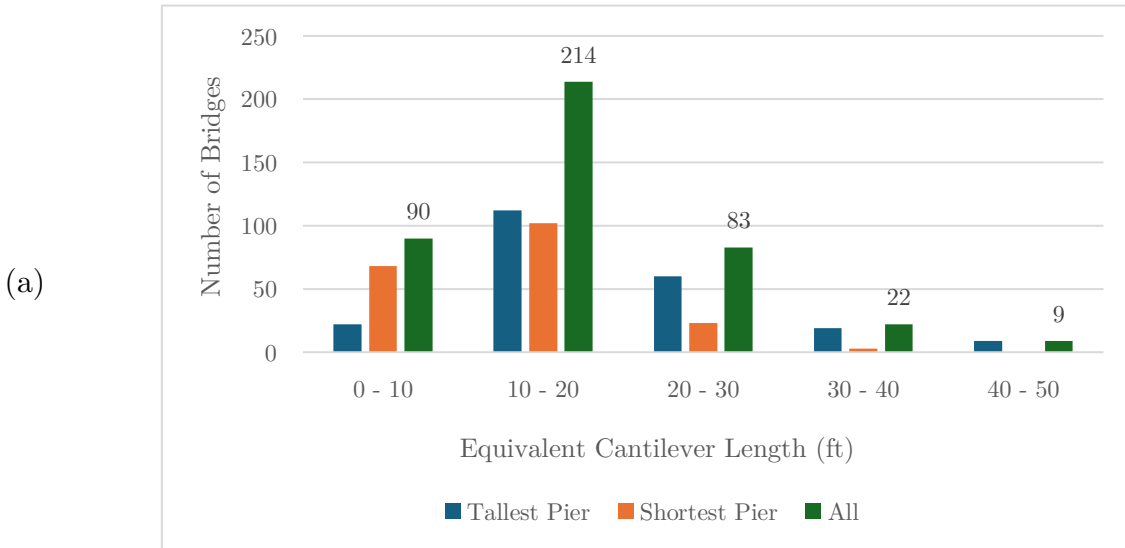
CHAPTER 3:

TARGET PROPERTIES OF TEST COLUMNS

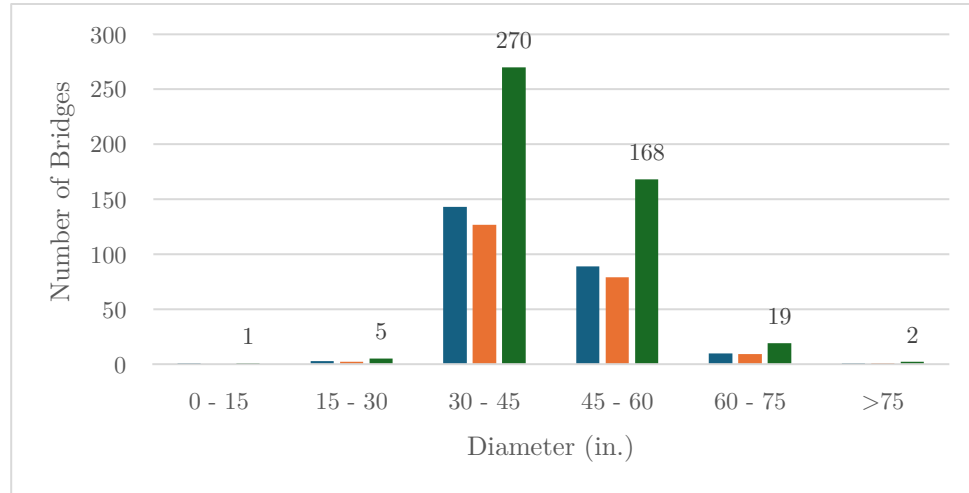
3.1. KEY PROPERTIES OF OLDER REINFORCED CONCRETE BRIDGE COLUMNS

Liu et al. (2022) developed a bridge inventory of 850 structures along major highways in the Puget Sound region, including 582 major highway bridges. This database contains key properties of WSDOT multi-span bridge components, including the year of construction (extracted from plan dates), continuity (simple span, continuous, or mixed), and geometric properties of the tallest and shortest reinforced concrete columns in the bridge.

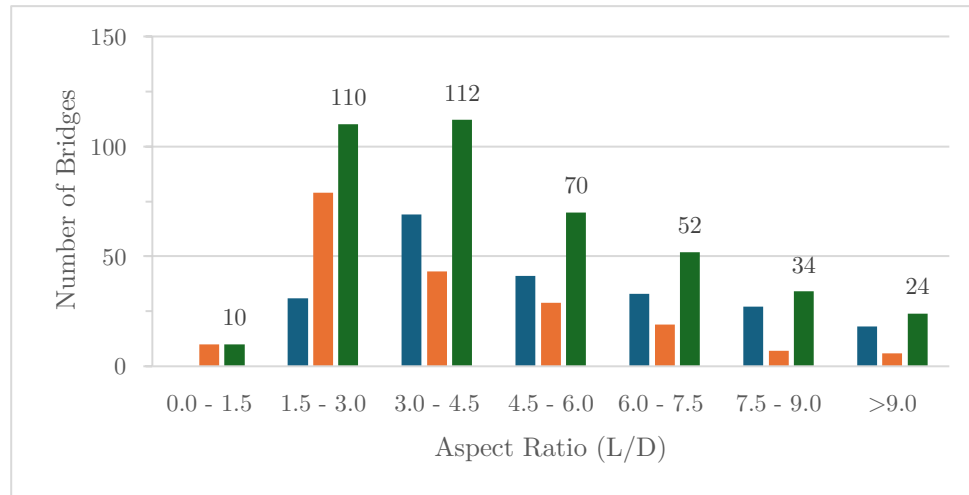
The inventory includes detailed information on 454 circular reinforced concrete columns for bridges built prior to 1971. The document column properties include diameter (D), clear height (H), longitudinal (ρ_l) and transverse (ρ_t) reinforcement ratios, and hoop size and spacing (s). The distributions of these properties were used to define key properties that represent a typical pre-1970's bridge column. The results of this analysis are summarized in Figure 3.1 (a) through (f).



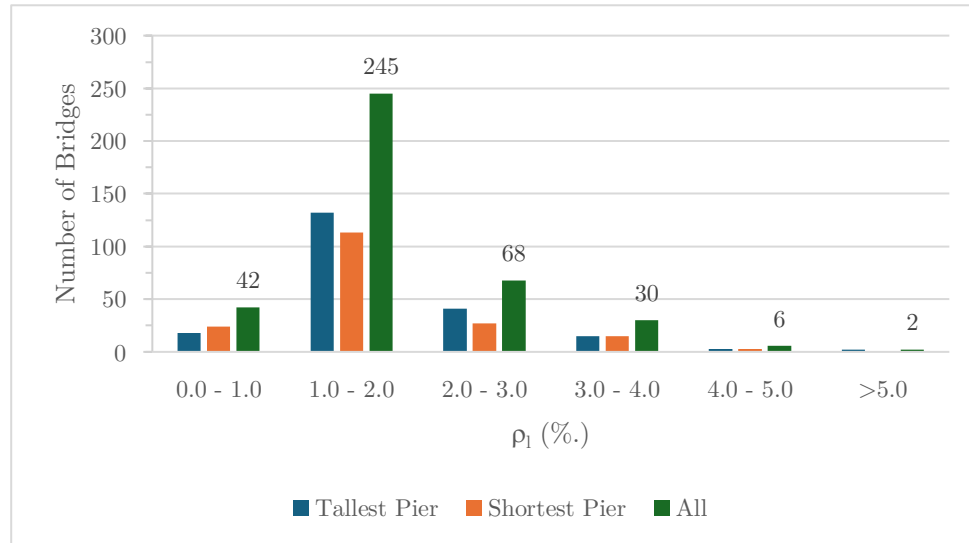
(b)



(c)



(d)



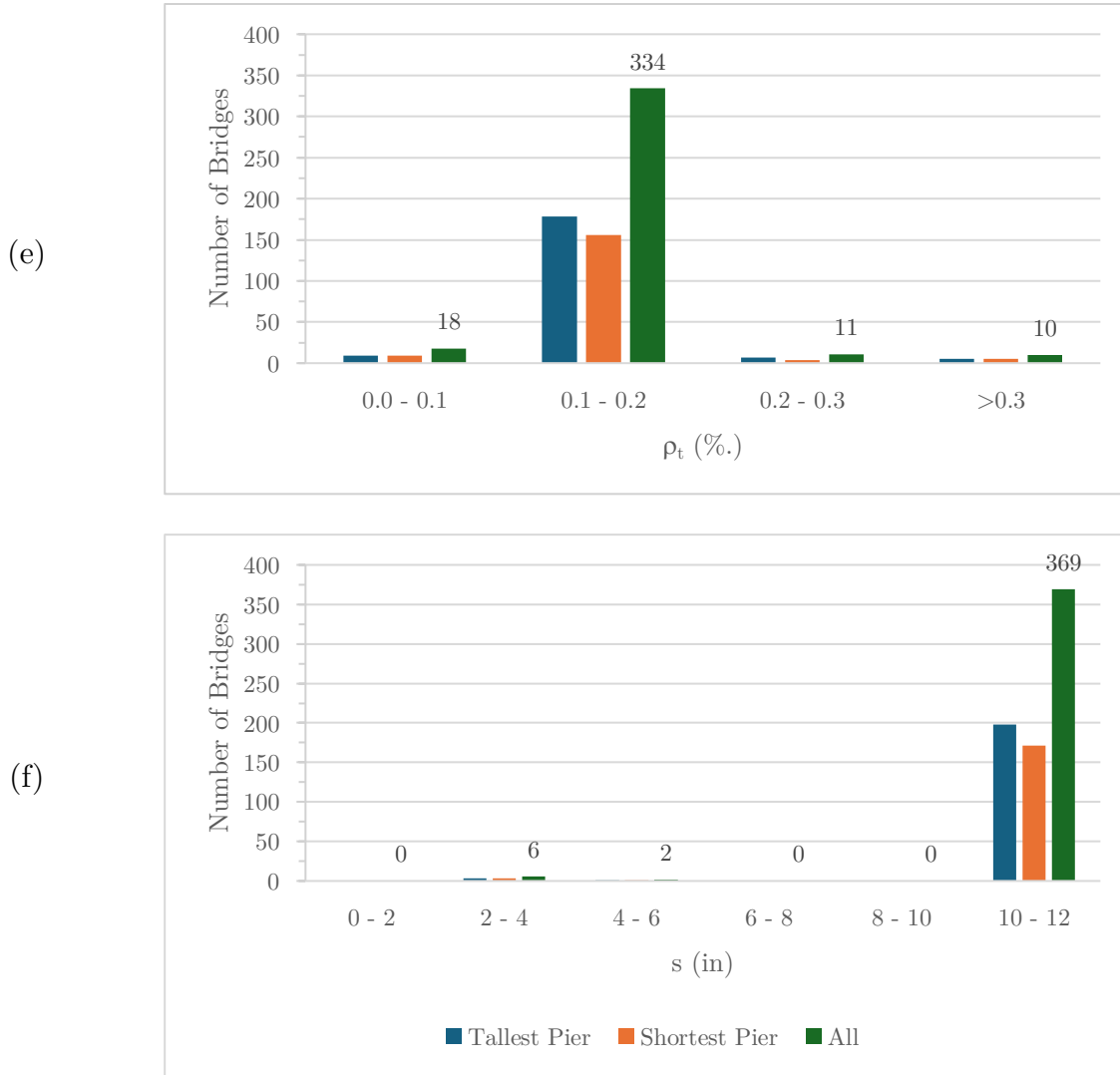


Figure 3.1 Distribution of key properties in pre-1970's bridge columns: (a) Equivalent cantilever length; (b) Diameter; (c) Aspect ratio (L/D); (d) Longitudinal reinforcement ratio (ρ_l); (e) Transverse reinforcement ratio (ρ_t); (f) Hoop spacing or spiral pitch (s). Results are shown separately for the shortest and longest columns in multi-span bridges, as well as for the combined dataset

Figure 3.1 (c) suggests that the shortest columns from the bridges included in the pre-1970's inventory sample have aspect ratios concentrated between 1.5 and 3.0, whereas the tallest ones are concentrated in ratios between 3.0 and 4.5. Additionally, Figure 3.1 (d) shows that longitudinal reinforcement ratios were typically below 4%, with most values in the 1%-2% range. As for the transverse reinforcement, Figure 3.1 (e) shows that most columns had transverse reinforcement ratios between 0.1% and 0.2%, with most sampled columns having a 304.8 mm (12 in.) hoop spacing (see Figure 3.1 (f)).

3.2. PREVIOUS TESTS OF OLDER REINFORCED CONCRETE BRIDGE COLUMNS

Researchers have compiled tests on reinforced concrete columns into several databases, including the UW/PEER Circular Column Database, NEES: ACI 369 Circular Columns Database (Ghannoum et al., 2015), Circular Column Database (Azadi Kakavand et al., 2019), and SDSU Circular Column Database (Tazarv et al., 2021). These databases contain test results for columns with a wide range of characteristics.

To identify tests representative of pre-1970's bridge columns, the results from Section 3.1 were used to establish selection criteria for querying the database. These criteria included:

- i) circular geometry;
- ii) axial load ratios between 2% and 15%, considering that bridge columns typically experience relatively low-to-moderate axial load ratios;
- iii) longitudinal reinforcement ratios lower than 2.5%;
- iv) transverse reinforcement volumetric ratios below 0.65%;
- v) no special experimental features, such as flexible bases, retrofit techniques, or other non-conventional configurations;
- vi) cyclic, quasi-static, unidirectional lateral loading; and vii) available force-displacement histories.

3.2.1. NELSON AND PRICE TESTS

The Nelson and Price test series consists of six reinforced concrete circular column specimens tested in the Large-Scale Structural Engineering Testing Laboratory at the University of Washington by Nelson (2000) and Price (2000), as described in Ranf et al. (2006). These tests are referred to herein as the Nelson & Price (N&P) tests.

According to the test report, a cantilevered-equivalent prototype column was defined based on a survey conducted by WSDOT engineers in contact with the research team, which identified representative properties of WSDOT bridge columns constructed before the 1970's, including overlapped-end hoops as transverse reinforcement. The test set was designed with six columns sharing the same configuration, corresponding to a 1:3 scale of

this prototype column, and subjected to different cyclic loading histories. Key properties of the prototype and test specimens are shown in Table 3.1.

Table 3.1 N&P Test Column Properties (Adapted from Ranf et al., (2006))

Property	Prototype	N&P Test Column	Remarks
Column Length	4.57 m (15 ft)	1.52 m (5 ft)	1:3 scale
Column Diameter	1.52 m (60 in.)	0.51 m (20 in.)	1:3 scale
Longitudinal Steel Ratio	$\rho_l = 1.0 \%$	$\rho_l = 0.99 \%$	Maintained during scaling
Transverse Hoops	#4 Grade 60 @304.8 mm (12 in.) spacing	W2.5 wire @101.6 mm (4 in.) spacing	1:3 scale on spacing
Transverse Reinforcement Ratio	$\rho_t = 0.12 \%$	$\rho_t = 0.15 \%$	Significantly lower than current code requirements
Cover to Hoop	38.1 mm (1.5 in.)	14.48 mm (0.57 in.)	1:2.6 scale
Axial Load	$0.07f'_c A_g$	$0.1f'_c A_g$	
Maximum Aggregate Size	25.4 mm (1 in.)	9.53 mm (3/8 in.)	1:2.7 scale

The N&P tests were designed as cantilevered columns with a footing that was attached to the testing floor through post-tensioned rods. Axial load was applied through two post-tensioned high-strength rods from the floor to a welded steel crosshead mounted on a spherical bearing at the top of the column; lateral loads were applied by a servo-controlled hydraulic actuator spanning between the reaction wall and the column.

The specimen designation used for these tests refers to the displacement history applied during testing. A description of each column is included in Table 3.2.

Although these six tests are included in the UW/PEER Circular Column Database reviewed in 3.2.2, they are discussed separately because their configurations are representative of pre-1970's bridge columns, and their test matrix specifically investigates

the effects of cyclic loading histories. Additionally, these specimens served as the basis for the design of the columns tested in this study.

Table 3.2 N&P Specimens Description (Adapted from Ranf et al. (2006))

ID	Description	Reported Shear Damage
NP-S1	1 standard cycle between each displacement level, with displacement increments each half cycle until failure.	Yes
NP-S3	3 standard cycles at each displacement level until failure.	Yes
NP-S15	15 standard cycles between each displacement level, with displacement increments each half cycle until failure.	Yes
NP-C2	Constant amplitude cycling at a drift ratio of 2%.	Yes
NP-C4	Constant amplitude cycling at a drift ratio of 4%.	No
NP-C3R	Constant amplitude cycling at a drift ratio of 3%, followed by a decrease in the amplitude of cycling.	Yes

3.2.2. ADDITIONAL TESTS

The UW/PEER Circular Column Database is a collaborative database developed through contributions from numerous researchers that provides an expanded version of PEER's existing Structural Performance Database for circular columns (PEER, 2003). This database currently includes 343 specimens and incorporates test data from PEER's database, other experimental databases, original reference documents, and information obtained directly from researchers. From this database, geometric and material properties, as well as force-displacement histories, were obtained for most tests in the study sample, including the Nelson and Price specimens, four of which were already part of the original PEER database.

All the specimens were characterized using the geometric and material properties available in the UW-PEER database. However, the force-displacement histories of two

specimens were digitized from the original reference documents, and for five specimens these were obtained from the ACI 369 Circular Column database. No additional tests were incorporated from the remaining reviewed databases because force-displacement histories were unavailable for tests that met the selection criteria.

Once the selection criteria were applied, the sample included 24 tests: 19 conducted under cantilevered conditions and 5 under double-curvature setups. Of these tests, 12 used spiral reinforcement and 12 used hoop reinforcement. The complete list of these tests, references, and sources is included in Table A.1. Additionally, key properties used in the analyses presented in this study for each test are provided in Table A.2.

3.3. KEY PROPERTIES OF PREVIOUS TESTS

3.3.1. P-Δ AND EFFECTIVE FORCE CORRECTIONS

For each test, force-displacement histories were processed to identify P -Δ effects, if present, and determine consistent force histories: the horizontal shear force, $F_{i,shear}$, which excludes the force component induced by P -Δ effects, and the effective force, $F_{i,eff}$, which includes the P -Δ contribution. Four types of correction approaches were defined, similar to those stated in Berry et al. (2004):

- a) **Type 1:** Reported forces are effective horizontal forces, and P -Δ effects occur in the same direction as the lateral force applied.

$$F_{i,shear} = F_{i,meas} - \frac{P\Delta_i}{L} \quad (3.1a)$$

- b) **Type 2:** Reported forces are horizontal forces, and no P -Δ effects are included.

$$F_{i,shear} = F_{i,meas} \quad (3.1b)$$

- c) **Type 3:** Reported forces are effective horizontal forces, and the axial load decreases the shear in the column. Additionally, the axial load ram does not point toward the bottom of the column, but to the point L_{bot} below it.

$$F_{i,shear} = F_{i,meas} - \frac{P\Delta_i}{(L + L_{bot})} \quad (3.1c)$$

d) **Type 4:** Reported forces are effective horizontal forces, and the axial load increases the shear in the column. Additionally, the axial load ram rotates to increase shear at a distance L_{top} from the lateral load application.

$$F_{i,shear} = F_{i,meas} + \frac{P\Delta_i}{\sqrt{\Delta_i^2 + L_{top}^2}} \quad (3.1d)$$

Once the horizontal force is defined, the effective horizontal force ($F_{i,eff}$) and the moment at the base of the column ($M_{i,base}$) are:

$$F_{i,eff} = F_{i,shear} + \frac{P\Delta_i}{L} \quad (3.2)$$

$$M_{i,base} = F_{i,eff}L \quad (3.3)$$

3.3.2. DISPLACEMENT DUCTILITY

The displacement ductility (μ_Δ) of each test was calculated as the ratio of the displacement at failure (Δ_u) and the idealized yield displacement (Δ_y), as shown in Eq. 3.4.

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} \quad (3.4)$$

The definitions of Δ_y and Δ_u used in this study are based on Camarillo (2003). To calculate these two points, a moment-curvature analysis of the section was performed considering: i) even distribution of the longitudinal reinforcement in the section; ii) Mander et al. (1988) and Hognestad (1951) concrete constitutive models for the confined and unconfined sections, respectively; and iii) an elastic-perfectly plastic steel model with an elastic modulus of 200 GPa.

To define Δ_y , the moment at “first” yield is determined from the moment-curvature analysis at the yield of the extreme layer of tensile reinforcement or at the maximum compressive strain of 0.002, whichever occurred first. This moment was then divided by

L , to obtain the effective horizontal force at “first” yield (F_{yi}). Then, the displacement at “first” yield (Δ_{yi}) was obtained as the displacement on the effective force envelope corresponding to F_{yi} . For cases in which F_{yi} was larger than the maximum measured effective force (F_{max}), Δ_{yi} was taken as the displacement at 95% of the maximum effective force (Camarillo, 2003).

The idealized yield force ($F_{0.004}$) is defined the same way as F_{yi} , with a concrete compressive strain target of 0.004. The displacement corresponding to the crossing between a line extended from the origin to (Δ_{yi}, F_{yi}) and a horizontal line at $F_{0.004}$ is then considered Δ_y .

Furthermore, following the failure definition procedure in Camarillo (2003), Δ_u was defined as the maximum displacement the column was subjected to prior to the displacement (Δ_{80}) corresponding to the last data point where the column resisted 80% of the absolute maximum effective force (F_{80}), given that one of the following cases occurred:

- a) The absolute maximum displacement after the identified 80% force ($\Delta_{post-80}$) exceeds Δ_{80} (i.e., $\Delta_{post-80} \geq \Delta_{80}$).
- b) The maximum displacement following another zero crossing ($\Delta_{post-zero}$) exceeds 95% of Δ_{80} (i.e., $\Delta_{post-zero} \geq 0.95\Delta_{80}$).
- c) The force corresponding to the maximum displacement after the zero crossing ($F_{post-zero}$) is less than or equal to the proportional force set by the force and displacement at the 80% location (i.e., $F_{post-zero} \leq F_{80} \frac{\Delta_{post-zero}}{\Delta_{80}}$).

If none of the previous cases occurred, the column was assumed not to have failed, and thus, Δ_u would be undetermined.

Calculated values for μ_Δ , Δ_y , and Δ_u for each specimen are included in Table A.3.

3.3.3. FAILURE MODE CLASSIFICATION PROCEDURE

The failure classification criteria used were those implemented in Camarillo (2003) and Berry et al. (2004), as summarized in Figure 3.2.

Three failure modes were defined: flexure, flexure-shear, and shear. If no shear damage was reported, it was classified as flexural failure; if shear damage was reported and either

i) the absolute maximum effective force was lower than 95% of the idealized effective force at concrete strain of 0.004 ($F_{max} < 0.95F_{0.004}$), or ii) displacement ductility was lower than or equal to 2 ($\mu_{\Delta} \leq 2$), it was classified as a shear failure; otherwise it was classified as a flexure-shear failure.

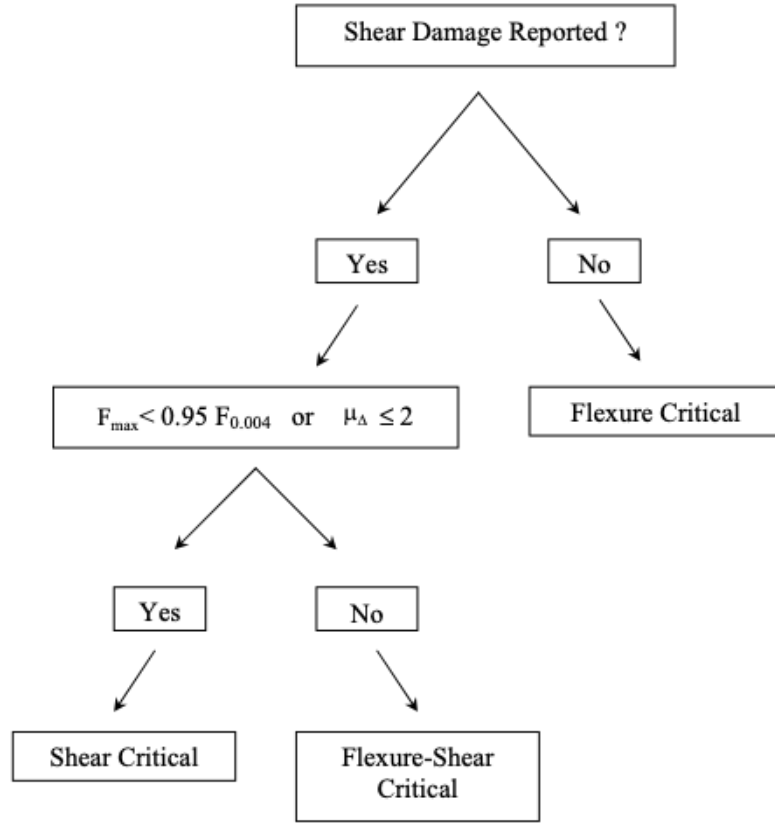


Figure 3.2 Failure Classification Flowchart (Adapted from Berry et al. (2004))

Table 3.3 presents the number of tests included per failure-mode classification, considering different filtering criteria. The first row, “All”, considers all circular tests from the UW/PEER and ACI 369 databases with conventional test configurations and available force-displacement histories (120 tests); from those, different filter combinations for transverse (ρ_t) and longitudinal (ρ_l) reinforcement ratios, along with axial load ratios, were applied for limiting the sample to be representative of pre-1970’s bridge column properties as found in Section 3.1.

Once ρ_l , ρ_t , and axial load ratio in the sample were limited to the target properties, the number of tests in the sample was 24. Following the failure mode classification

procedure, only 1 experienced a shear failure; 11 had a flexure-shear failure; 9 failed in flexure; and 3 did not qualify to have failed.

Table 3.3 Study Sample Failure Mode Classification

FILTER CRITERIA			NUMBER OF TESTS PER FAILURE MODE				L/D		TOTAL
ρ_t	ρ_l	$\frac{P}{A_g f'_c}$	SHEAR	FLEXURE-SHEAR	FLEXURE	NO FAILURE	MIN	MAX	
-	-	-	3	19	65	33	1.5	10	120
$\leq 0.65\%$	-	-	3	16	11	4	1.5	6.2	34
-	$\leq 2.5\%$	-	3	15	53	25	1.5	10	96
-	-	[2% – 15%]	1	13	51	24	1.5	10	89
$\leq 0.65\%$	$\leq 2.5\%$	[2% – 15%]	1	12	8	3	1.5	6.2	24

Specimens classified as “No failure” were kept in the study sample for comparison of key properties but were not used to evaluate shear strength degradation. Subsequent analyses of shear strength degradation focus primarily on specimens classified as shear or flexure-shear failures.

3.4. KOWALSKY AND PRIESTLEY (2000) MODEL EVALUATION

Once the key properties were defined for each specimen in the study sample, as described above and in Appendix A, the Kowalsky and Priestley (2000) model was used to estimate the nominal shear strength (V_n) of columns in the study sample and then to compute the ratio of the measured absolute maximum shear demand (V_{meas}) to V_n . The calculated ratios and intermediate results for each specimen are included in Table A.4.

The statistical results for the shear strength ratio (V_{meas}/V_n) analysis were grouped by reported shear damage and are included in Table 3.4. The mean ratio for shear and flexure-shear failure modes for this model is conservative and exceeds 1, whereas for flexure

failures, as expected, it is much lower, which makes sense given that these columns did not develop any shear damage.

Table 3.4 Statistical parameters for ratios of measured-to-nominal shear strength for columns with shear and flexure-shear failure modes

GROUP	TESTS	$\mu_{(V_{meas}/V_n)}$	$\sigma_{(V_{meas}/V_n)}$	COV
Observed shear damage (shear and flexure-shear failure modes)	13	0.93	0.21	0.22
No shear damage reported (flexural failure mode)	9	0.58	0.30	0.52

Figure 3.3 shows the influence of factors such as shear span-to-depth ratio, longitudinal and transverse reinforcement ratios, axial load ratio, and displacement ductility on the shear strength ratio V_{meas}/V_n .

From Figure 3.3 (a) and (c), we observe that the effectiveness of the Kowalsky and Priestley (2000) model does not depend on the shear span-to-depth ratio or the transverse reinforcement volumetric ratio. As expected, data points closer to 1 correspond to low aspect and transverse reinforcement volumetric ratios, which are associated with the concentration of shear and flexure-shear failures; however, many experiments fall in the unconservative region of the model.

As for the longitudinal reinforcement ratio, ρ_l , Figure 3.3 (b) shows that more variation appears to exist for lower ρ_l values, and that the Kowalsky and Priestley (2000) model tends to be more conservative for lower ρ_l values. Figure 3.3(d) suggests that the model is more conservative as the axial load ratio decreases. Finally, in Figure 3.3 (e), the model is observed to produce more conservative results as displacement ductility demands increase, but still presents many experiments in the unconservative region.

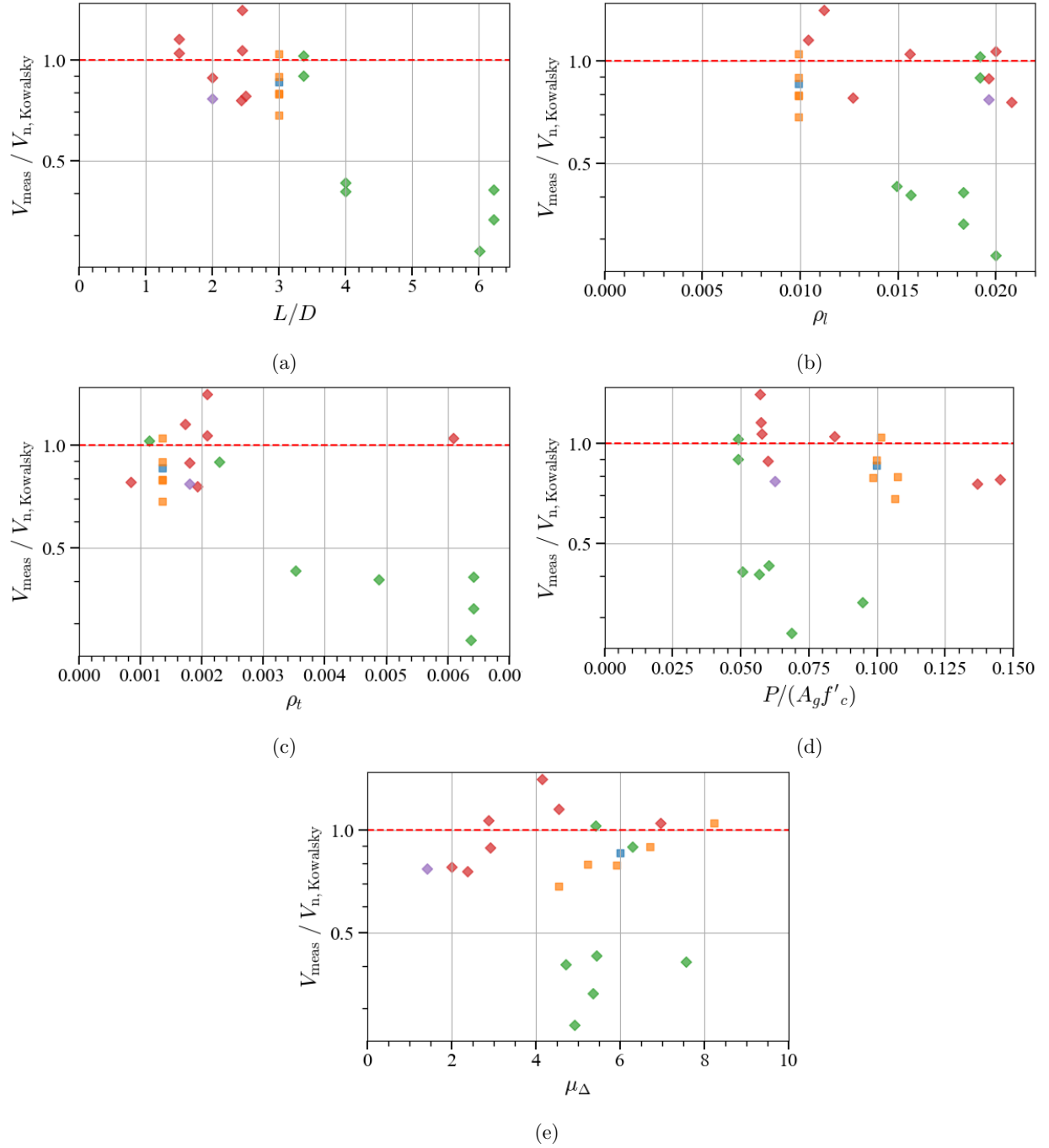


Figure 3.3 Kowalsky and Priestley (2000) shear strength ratio (V_{meas}/V_n) as a function of: (a) aspect ratio (L/D), (b) longitudinal reinforcement ratio (ρ_l), (c) transverse reinforcement volumetric ratio (ρ_t), (d) axial load ratio ($P/A_g f'_c$), and (e) displacement ductility (μ_Δ)

The influence of displacement ductility on shear strength is studied further here. Recall that the reduction in shear strength for ductility demand in the Kowalsky and Priestley (2000) model is implemented through the parameter γ . Here, its effectiveness under different ductility demands in tests classified as shear and flexure-shear failures is studied by computing the required value of the gamma parameter (γ_{req}) such that the nominal shear strength, V_n , matched the measured shear strength, V_{meas} . Individual results for γ_{req} per experiment are available in Table A.4.

Figure 3.4 (a) shows that γ_{req} decreases with increasing displacement ductility, following the general trend assumed by the γ factor in the Kowalsky and Priestley (2000) model. Figure 3.3(b), which shows the variation of the $\gamma_{req}/\gamma_{Kowalsky}$ ratio, indicates noticeable dispersion in the model, suggesting that it may be unconservative for specimens with hoops for transverse reinforcement.

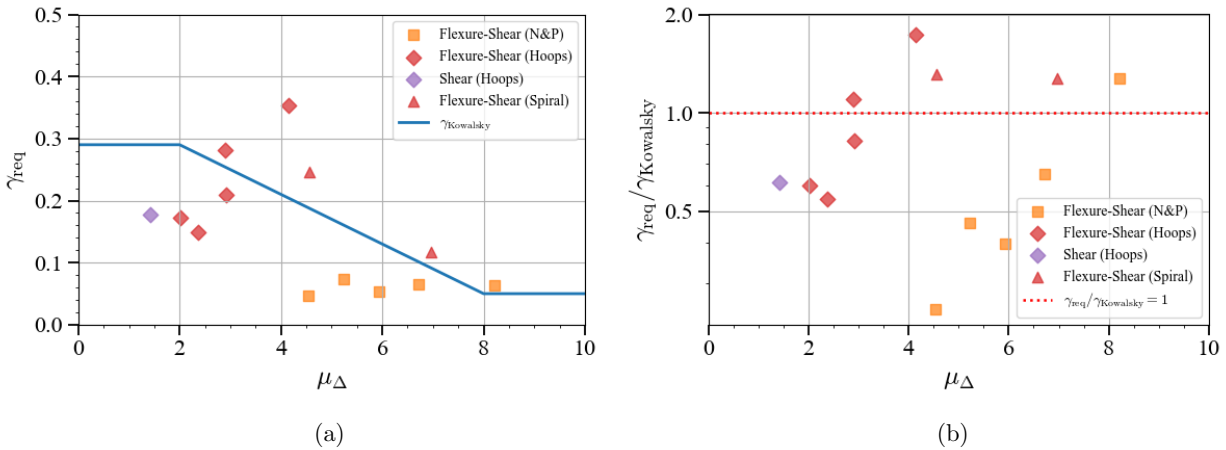


Figure 3.4 (a) Required concrete shear strength reduction parameter, γ_{req} , versus Ductility Demand for the Kowalsky and Priestley (2000) model to match the selected experimental results and (b) Normalized by γ computed from Kowalsky and Priestley (2000) versus Ductility Demand

3.5. EFFECT OF CUMULATIVE PLASTIC DISPLACEMENT

To assess the effects of cycling and load history on shear strength, a variable, Cumulative Plastic Displacement (CPD), was introduced. CPD was defined as the cumulative sum of incremental plastic displacement excursions occurring outside a moving elastic displacement window. A larger CPD indicates that the specimen experienced a

greater number of cycles and/or cycles with larger displacement amplitude beyond the yielding displacement. Thus, two columns with the same displacement ductility demand may be subjected to different CPD demands depending on the number and amplitude of the cycles it experienced.

The process to calculate CPD starts by defining a “mobile window” of elastic displacement, whose initial limits b_0^+ and b_0^- would be defined by Δ_y in both directions (Eq. 3.5). The range of this window ($2\Delta_y$) was defined considering systems loaded in one plane whose elastic region is equal in both directions.

When a displacement at instant i (Δ_i) falls beyond these window limits, the absolute of the difference of Δ_i and the existing window limit will be stored as the incremental plastic displacement at instant i ($\delta_{p,i}$), as shown in Eq. 3.6. The elastic window limits would be updated when Δ_i has fallen outside the elastic range (Eq. 3.7).

The CPD is calculated by summing the absolute value of all the recorded $\delta_{p,i}$ values as defined in Eq. 3.8. Individual results for CPD computations per experiment are available in Table A.3.

$$b_0^+ = \Delta_y \quad (3.5)$$

$$b_0^- = -\Delta_y$$

$$\delta_{p,i} = \begin{cases} \Delta_i - b_{i-1}^+, & \text{if } \Delta_i > b_{i-1}^+ \\ \Delta_i - b_{i-1}^-, & \text{if } \Delta_i < b_{i-1}^- \\ 0, & \text{if } b_{i-1}^- \leq \Delta_i \leq b_{i-1}^+ \end{cases} \quad (3.6)$$

$$b_i^+ = b_{i-1}^+ + \delta_{p,i} \quad (3.7)$$

$$b_i^- = b_{i-1}^- + \delta_{p,i}$$

$$CPD = \sum_{i=1}^N |\delta_{p,i}| \quad (3.8)$$

Additionally, the Normalized Cumulative Plastic Displacement was proposed as a dimensionless derived variable of CPD, obtained by normalizing it by 4 times the peak plastic displacement (Δ_p) of the test (Eqs. 3.9 and 3.10) in one direction. The factor 4 represents the four plastic displacement excursions experienced during a full symmetric cycle: loading and unloading in both positive and negative directions. This normalization

provides a comparable measure across tests, accounting for differences in the plastic displacement range. Individual results for NCPD computations per experiment are available in Table A.3.

$$\Delta_p = \Delta_u - \Delta_y \quad (3.9)$$

$$NCPD = \frac{CPD}{4\Delta_p} \quad (3.10)$$

The influence of the previous plastic displacement variables on the idealized γ_{req} parameter defined above necessary to make V_{meas} equal to V_n is shown in Figure 3.5. against both NCPD and CPD/L. Figure 3.5 (a) and (b) show a trend that suggests that additional strength reduction is required as more plastic displacement incursions are induced on the column.

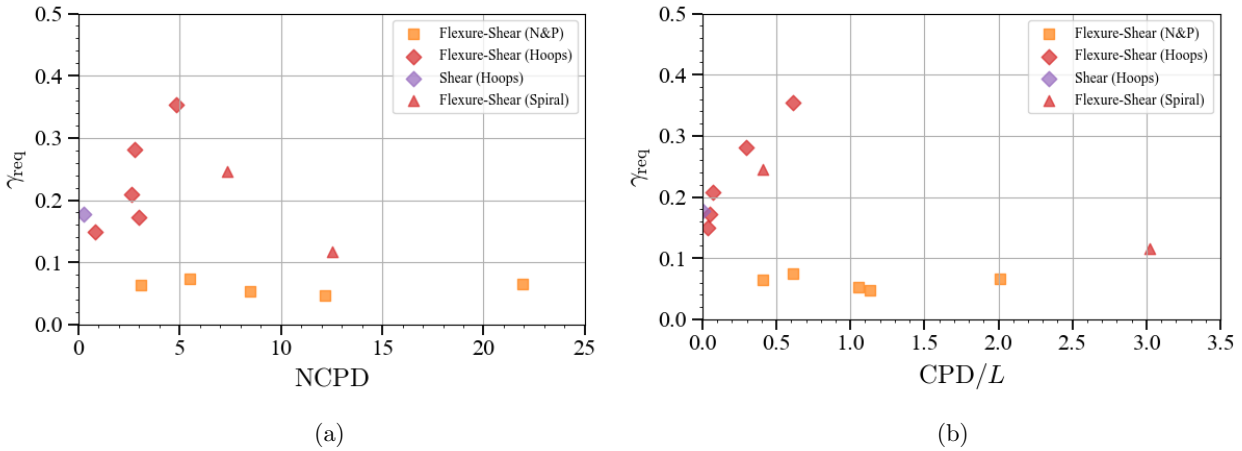


Figure 3.5 Influence of cumulative plastic displacement in concrete shear strength degradation in the Kowalsky and Priestley (2000) model: (a) NCPD, (b) CPD/L

Figure 3.6 evaluates an additional variable that normalizes the maximum plastic displacement (Δ_p) of the test by its shear span, showing that shear strength decreases as the maximum plastic displacement increases. However, conceptually, this variable does not account for the effects of multiple cycles and is more closely related to the trends observed in displacement ductility analyses, such as those shown in Figure 3.4(a).

The influence of plastic displacement variables on the conservatism of the γ parameter in the Kowalsky and Priestley (2000) model is shown in Figure 3.7. A trend of decreasing conservatism as more NCPD is achieved was observed in Figure 3.7 (a), and increased conservatism with higher CPD/L ratios was observed in Figure 3.7 (b). Finally,

in (c), it was observed that higher maximum plastic displacement yield increases conservatism and dispersion, as expected given this variable's conceptual similarity to displacement ductility.

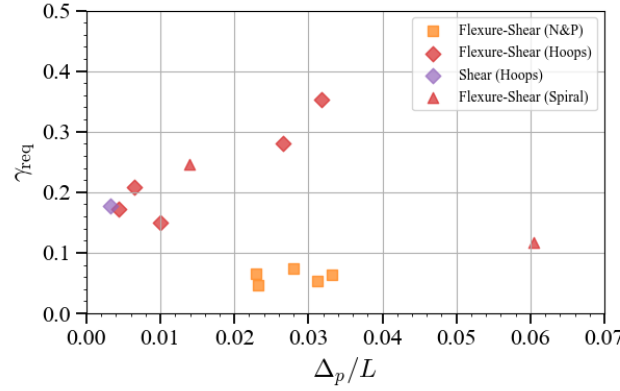


Figure 3.6 Influence of plastic range in concrete shear strength degradation in the Kowalsky and Priestley (2000) model

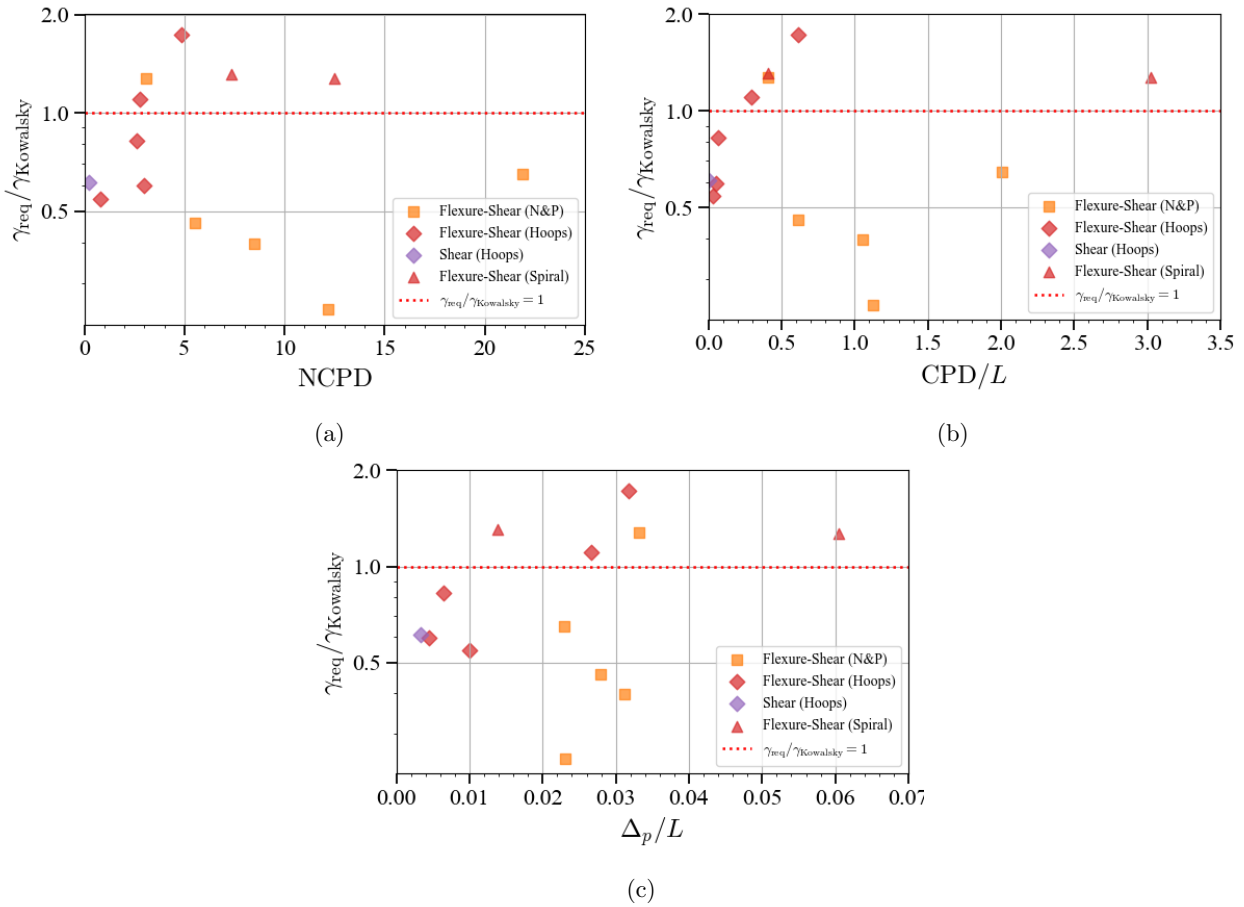


Figure 3.7 Influence of plastic displacement variables in concrete shear strength degradation parameter conservatism in Kowalsky and Priestley (2000) model: (a) NCPD, (b) CPD/L, (c) Δ_p/L

3.6. RESEARCH GAP

The previous analyses indicated that the number of column tests with properties similar to those of pre-1970's bridge columns is limited, particularly for shear and flexure-shear failures. As a result, the calibration and evaluation of shear strength models may be influenced by data from more modern configurations. This limitation highlights the need to assess the accuracy of current shear strength evaluation models for older bridge columns.

Additionally, it was found that cumulative plastic displacement demand contributed to shear strength reduction in the previously collected and analyzed test results. However, the reviewed shear strength models for reinforced concrete columns assume that shear strength is related to displacement ductility, without explicitly accounting for the effects of cumulative plastic displacement demand. This could result in different shear strengths among columns subjected to similar displacement ductility demands but having different cumulative plastic displacement histories.

This limitation is particularly important for regions where long-duration ground motions, such as subduction-zone earthquakes, impose repeated loading cycles that accumulate plastic displacement demands on columns before peak ductility demand is reached. Therefore, there is a need to evaluate the effect of cyclic loading on shear strength degradation and assess whether current shear strength models adequately capture this effect.

To address these gaps, three reinforced concrete columns with design configurations representative of pre-1970's WSDOT bridge columns were designed and tested under cyclic loading histories with the same ductility demands but different cumulative plastic displacement levels. The specimens were designed to meet shear demands in the middle of the flexure-shear region of the Kowalsky and Priestley (2000) model, thereby increasing the number of tests available for older bridge columns exhibiting flexure-shear-critical behavior. The results of these tests are used to assess the impact of cyclic loading on shear strength degradation and to evaluate the accuracy of current shear strength models in accounting for cumulative plastic displacement effects.

CHAPTER 4: EXPERIMENTAL PROGRAM

Three nominally identical reinforced concrete columns were designed based on a 1960's prototype column and tested under different displacement histories at the Large-Scale Structural Engineering Testing Laboratory at the University of Washington. This chapter provides information on the test column design, the test matrix of displacement histories, the test setup, and the instrumentation plan.

4.1. PROTOTYPE

The prototype column was selected as a representative design of a 1960's bridge column, based on trends in pre-1970's bridge column designs reported by Liu et al. (2022) and discussed in Section 3.1.

A prototype diameter of 1.52 m (60 in.) was selected to maintain similarity with the N&P tests and to remain consistent with the bridge inventory distribution shown in Figure 3.1 (b). Given that most of the shortest columns in the multi-span bridges inventory have an aspect ratio between 1.5 and 3.0 (Figure 3.1 (c)), this variable was set to 2.5; therefore, the prototype length was set to 3.81 m (150 in.).

According to bridge design specifications prior to the 1970's, the required concrete compressive strength was 24.8 MPa (3600 psi) before 1963 and 27.6 MPa (4000 psi) after 1963. Therefore, the specified concrete compressive strength for the prototype column was set at 27.6 MPa (4 ksi), which, assuming a 50% increase to account for long-term strength gain and differences between the specified and mean strengths, would yield a probable concrete strength of 41.4 MPa (6 ksi). The axial load ratio, based on the specified concrete strength, was set to 10%. Considering the expected long-term strength gain, this ratio would yield an expected value of 7%; however, the 10% value was retained to provide a conservative representation of the axial demand relative to the nominal column strength.

Additionally, prior to the use of spiral reinforcement introduced in 1977 bridge specifications, transverse reinforcement in circular columns typically consisted of individual hoops with lap splices. Figure 3.1 (e) and (f) indicate that the transverse reinforcement for pre-1970's bridge columns typically included volumetric ratios below

0.2% and a 12 in. spacing. Therefore, the prototype's transverse reinforcement was defined as #4 circular hoops with overlapped splices and a 12 in. spacing, which implies a $\rho_t = 0.15\%$; the same ratio and spacing as the N&P prototype.

The prototype's longitudinal reinforcement ratio was set as $\varphi_l = 1.5\%$, which, according to Figure 3.1 (d), is included in the most frequent bin of the inventory sample. Using the Kowalsky and Priestley (2000) model and a moment-curvature analysis of the column cross-section, it was assessed that the maximum shear demand from flexural strength of the column was in the middle height of the flexure-shear region of the shear strength model.

Key properties of the prototype column are summarized in Table 4.1.

4.2. TEST COLUMNS

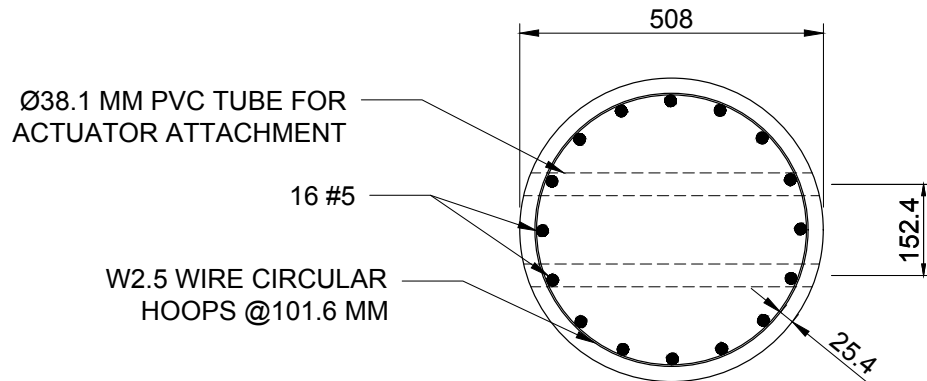
The three specimens had the same design and nominal properties, consisting of a footing and a column as shown in Figure 4.1. Their key properties are summarized in Table 4.1, and detailed plans are included in Appendix B.1. Test Columns

The footing was included solely to allow the column to be attached to the testing floor, and the column was designed with a 1:3 scale factor relative to the prototype column. Therefore, the test columns had a diameter of 0.51 m (20 in.) and a shear span of 1.52 m (50 in.) measured from the top of the footing to the loading point. The prototypical longitudinal and transverse reinforcement ratios were maintained, and the scaled reinforcement consisted of 16 No. 5 Gr60 bars for longitudinal reinforcement and W2.5 wire hoops, spaced at 101.6 mm (4 in.) and overlapped by 101.6 mm (4 in.) for transverse reinforcement. W2.5 wire was commercially available only as smooth wire; thus, poorer bond properties are expected for the scaled specimens made with it.

The test column axial load ratio was set to 10%, consistent with the prototype ratio before accounting for long-term concrete strength gain. Aggregate size was scaled to the nearest commercial size, and clear cover was defined as 25.4 mm (1 in.) due to construction constraints.

Table 4.1 Details of prototype and test columns

Property	Prototype	Test Column	Remarks
Column Length	3.81 m (150 in.)	1.52 m (50 in.)	1:3 scale
Column Diameter	1.52 m (60 in.)	0.51 m (20 in.)	1:3 scale
Longitudinal Steel Ratio	$\rho_l = 1.5 \%$	$\rho_l = 1.56 \%$	Maintained during scaling
Transverse Hoops	#4 Grade 60 @304.8 mm (12 in.) spacing	W2.5 wire @101.6 mm (4 in.) spacing	1:3 scale on spacing
Transverse Reinforcement Ratio	$\rho_t = 0.15 \%$	$\rho_t = 0.15 \%$	Significantly lower than current code requirements
Cover to Hoop	38.1 mm (1.5 in.)	25.4 mm (1 in.)	1:1.5 scale
Axial Load	$0.1f'_c A_g$	$0.1f'_c A_g$	
Maximum Aggregate Size	25.4 mm (1 in.)	9.53 mm (3/8 in.)	1:2.7 scale



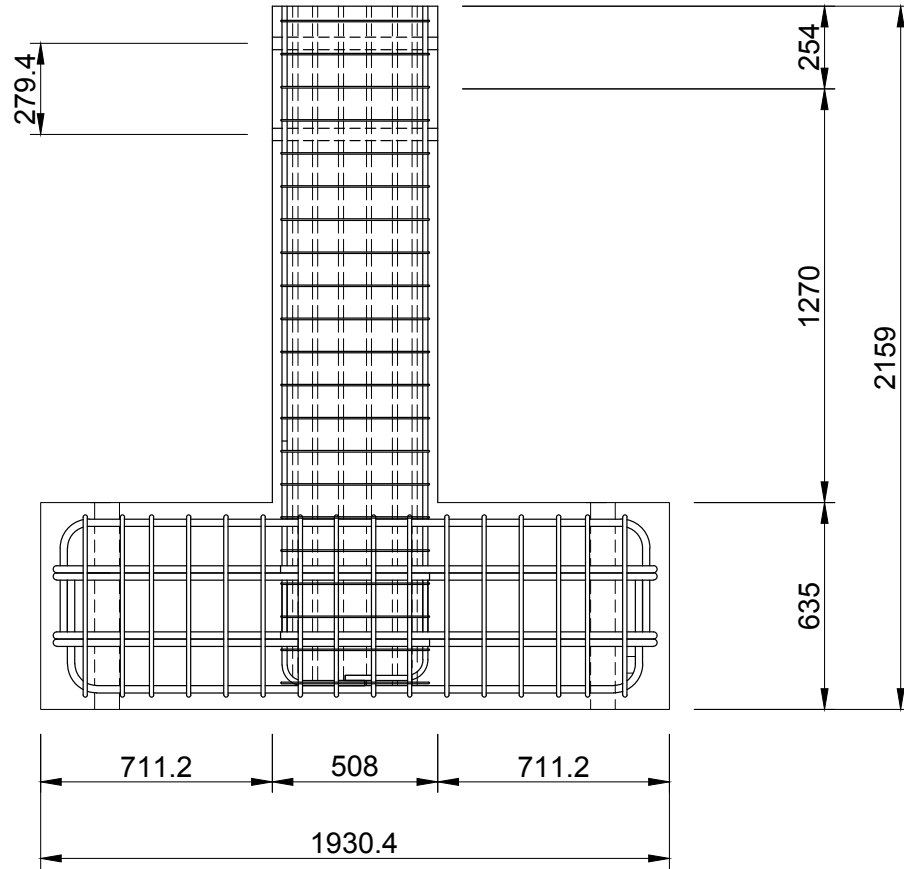


Figure 4.1 Test column geometry and reinforcement (elevation view)

4.3. TEST MATRIX

The three specimens had the same nominal design, and the variable among them was the displacement history, as introduced in Table 4.2. A preliminary estimate of the yield displacement was defined as 5.08 mm (0.2 in.) and used to define two initial cycles at 0.25 and 0.5 times the yield displacement. The following cycles considered target drift ratios defined with a 1.5-fold increment starting from 0.33% drift ratio. Each specimen considered a different number of intermediate cycles between each drift ratio target: S1 considered 1, S3 considered 3, and S6 considered 6.

The name of each specimen refers to the displacement history to which it was subjected, as shown in Table 4.2. A detailed description of these histories is provided in Section 4.3.

Table 4.2 Test column names and description

Specimen	Description
G-S1	1 cycle between each displacement level, with displacement increments each half cycle until failure.
G-S3	3 cycles between each displacement level, with displacement increments each cycle until failure.
G-S6	6 cycles between each displacement level, with displacement increments each half cycle until failure.

Specimens S1 and S6 were subjected to half-cycle increments of $1.5^{1/6}$ and $1.5^{1/12}$ between drift ratio targets, respectively. Specimen S3 was subjected to symmetric incremental cycles between each target drift ratio. Each increment had a $1.5^{1/3}$ -fold increment. Target displacement histories and cycle designations are summarized in Table 4.3,

Table 4.4, and Figure 4.2.

During testing, minor deviations from the target drift amplitude and cycle sequence occurred due to actuator-control adjustments and equipment calibration. All response histories and derived analyses included in this thesis are based on the recorded displacement histories during testing. Additional information on the testing cycles can be found in Appendix B.1.3. Displacement Histories.

Table 4.3 Target displacement levels

Drift Ratio (%)	Multiple of Estimated Yield Displacement	Displacement	
		(mm)	(in.)
0.1	$0.25\Delta_y$	1.27	0.05
0.2	$0.5\Delta_y$	2.54	0.1
0.33	$0.83\Delta_y$	4.32	0.17
0.5	$1.25\Delta_y$	6.35	0.25
0.75	$1.88\Delta_y$	9.65	0.38
1.13	$2.81\Delta_y$	14.22	0.56
1.69	$4.22\Delta_y$	21.34	0.84
2.53	$6.33\Delta_y$	32.26	1.27
3.80	$9.49\Delta_y$	48.26	1.90
5.70	$14.24\Delta_y$	72.39	2.85

Table 4.4 Cycle designations

Drift Ratio (%)	Cycle Numbers at Each Drift Level		
	S1	S3	S6
0.1	1	1	1
0.2	2		
0.33	3	2	2-7
0.5	4	3-5	8-13
0.75	5	6-8	14-19
1.13	6	9-11	20-25
1.69	7	12-14	26-31
2.53	8	15-17	32-37
3.80	9	18-20	38-43
5.70	10	21-23	44-49

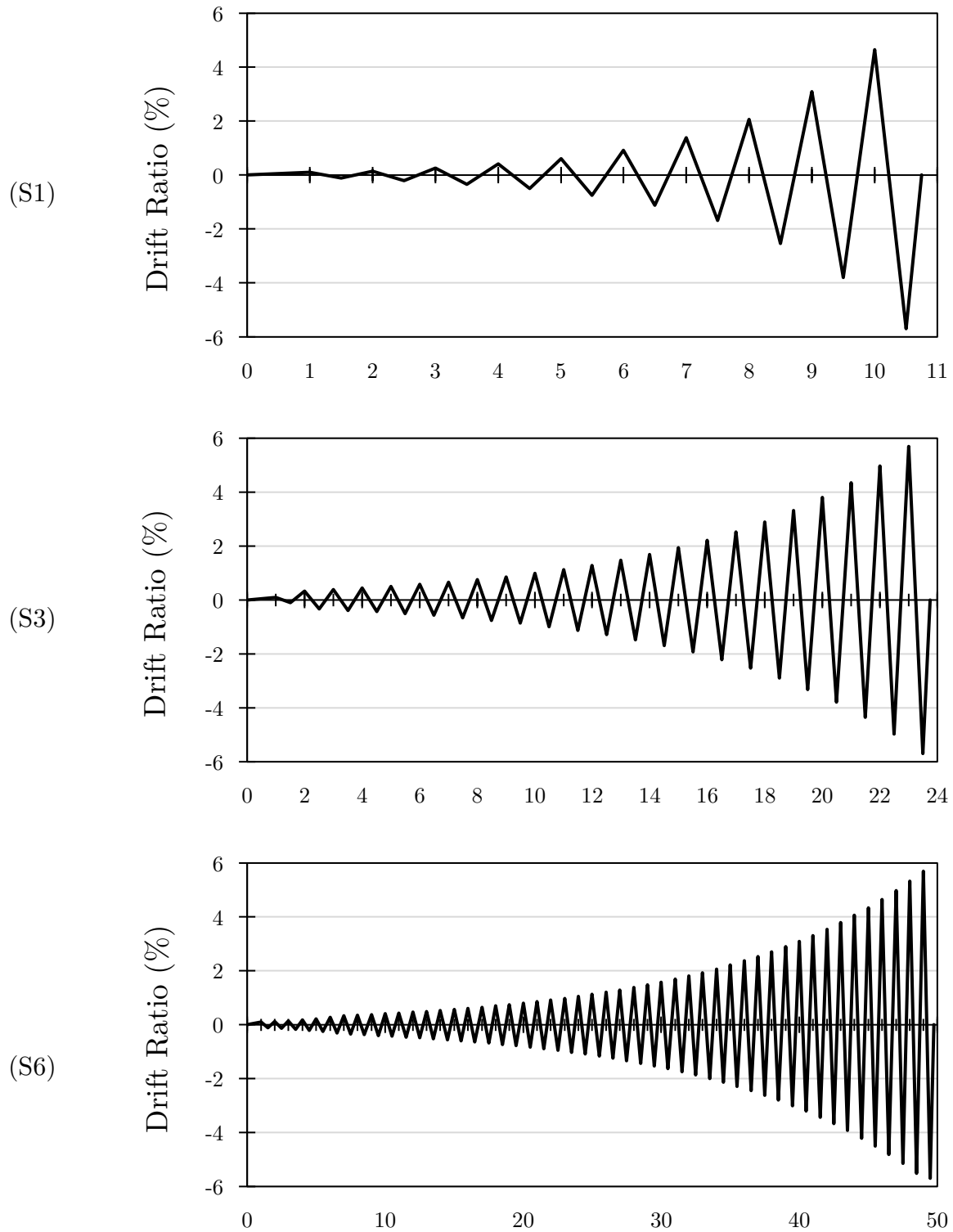


Figure 4.2 Target displacement history per specimen

4.4. APPLIED AXIAL LOAD

The axial load ratio was set to 10% for each specimen based on the average concrete compressive strength measured on the test day. Applied axial loads are shown in Table 4.5, while the axial load application system is described in Section 4.6.

Table 4.5 Applied axial load per specimen

Specimen	Axial Load	
	(kN)	(kip)
S1	868.9	195.7
S3	789.1	177.4
S6	918.5	206.9

Specimen S3 was subjected to an accidental axial load of 1,241 kN (279 kip) during the initial cycle that caused vertical cracks at the top of the column.

4.5. TEST SETUP

These tests used the self-reacting frame developed by Angela Kingsley (see Figure 4.3 and Figure 4.4). This frame was centered beneath the 10,675-kN (2,400-kip) Baldwin Universal Testing Machine, which maintained a constant axial load at the top of the column, as described in Section 4.6. A 978-kN (220-kip) MTS actuator installed on the reaction frame was used to apply the lateral load as described in Section 4.7.

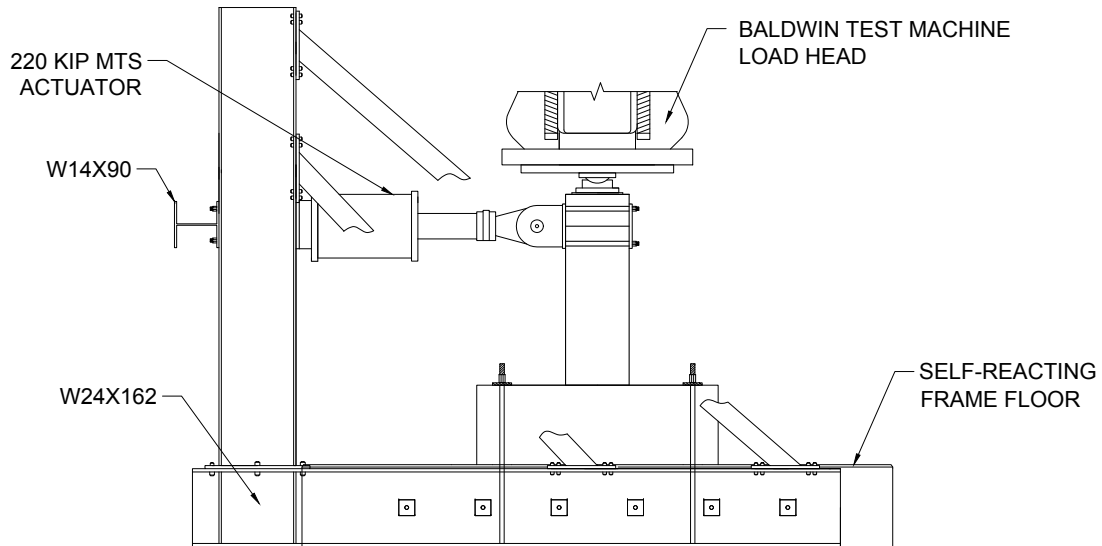


Figure 4.3 Experimental test rig (lateral view)

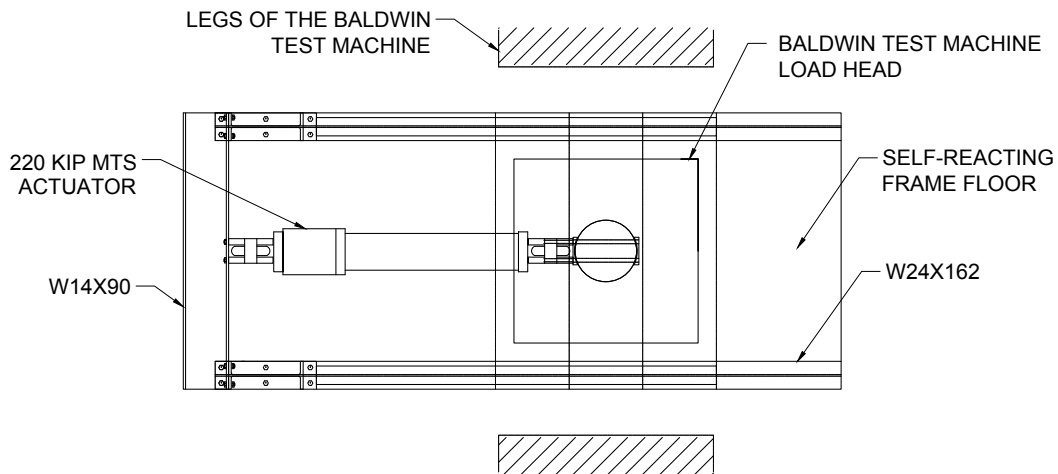


Figure 4.4 Experimental test rig (plan view)

Specimens were attached to the reaction frame using 1 ¼ in. 150 ksi William all-thread bars post-tensioned to 556 kN (125 kip) from the top of the footing to attachments at the top of the rig floor connecting to the bottom of the reaction block (see Figure 4.5). Additionally, a layer of Hydrostone was placed between the footing and the rig floor to ensure a uniform bearing surface.

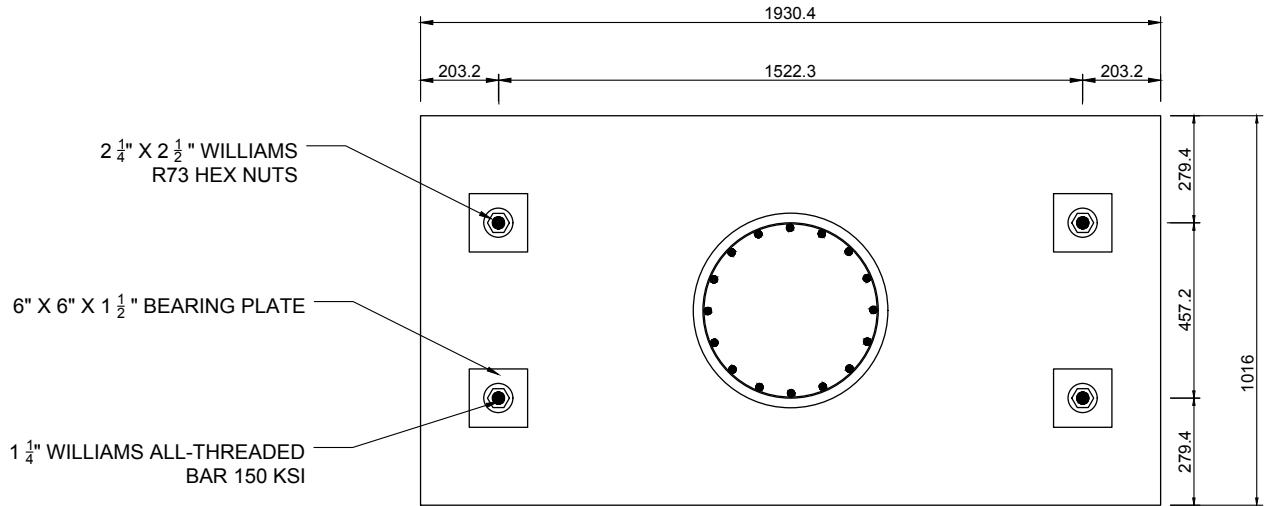


Figure 4.5 Footing attachment layout

4.6. AXIAL LOAD APPLICATION SYSTEM

Constant axial loads were applied through the Baldwin Universal Testing Machine at the top of the column. To maintain a constant vertical axial load while allowing lateral displacement of the column, a spherical swivel-head bearing was attached to the top of the column with Hydrostone, as shown in Figure 4.6.

The top of the spherical bearing sat within a recessed plate with three polytetrafluoroethylene (PTFE) surfaces in contact with stainless-steel surfaces, which were attached to a C-channel fixed to the Baldwin head. A thin layer of silicone grease was applied between the PTFE and stainless-steel sliding surfaces to reduce friction.

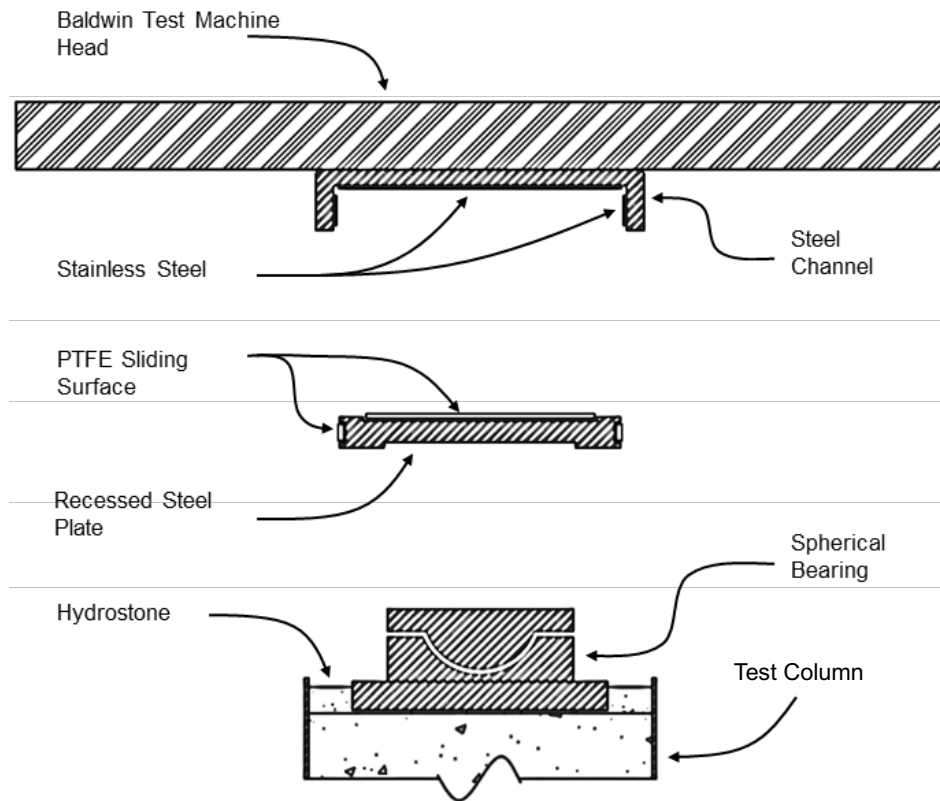


Figure 4.6 Axial load attachment (adapted from (Berg, 2014))

4.7. LATERAL LOADING SYSTEM

Lateral loads were applied using a 978-kN (220-kip) displacement-controlled hydraulic MTS actuator attached to a cap beam within the self-reacting frame, as shown in Figure 4.3 and Figure 4.4. The actuator was connected to the column at a height of 1,270 mm (50 in.) above the top of the footing using four 1-in.-diameter threaded rods that passed through 38.1 mm (1.5 in.) transverse holes in the column. To provide a flat connection surface between the actuator and the circular column, steel bearing plates were used. Each plate had a flat exterior face and a 508-mm (20-in.)-diameter concave interior surface to match the column curvature. Rubber pads were placed between the plates' concave surfaces and the column to improve contact and reduce local stress concentrations.

4.8. INSTRUMENTATION

The instrumentation plan was divided into three categories: lateral displacement pots, rotation and uplift pots, and longitudinal reinforcement strain gauges. Each of them is described in the following sections.

4.8.1. LATERAL DISPLACEMENT

Four lateral displacement measurements were recorded on each tested specimen using four string pots, as shown in Figure 4.7. Three were associated with different column heights: one at 25.4 mm (1 in.) from the top of the footing, one at 508 mm (20 in.), and one at 1270 mm (50 in.). On one end, the upper string pot was attached to the center of the actuator attachment, while the bottom two pots were attached to $\frac{1}{2}$ in.-diameter threaded rods embedded in the column. These rods were debonded along their length except for a 127 mm (5 in.) bonded region at the center of the column cross-section to minimize any provided confinement. On the other end, all these pots were attached to a reference column on the north side of the test setup.

Additionally, to measure the actuator-induced displacement absorbed by the cap beam in the reaction frame, a string pot was attached to a reference column on the south side of the test setup and to the cap beam.

4.8.2. ROTATION AND UPLIFT

Four Duncan pots were attached for uplift measurements: two in the interface footing-reaction block, and two in the interface reaction block-lab floor. For each location, one Duncan pot was placed on the north side of the experiment setup and one on the south side, as shown in Figure 4.8.

Additionally, rotation of the column was measured using 2 Duncan pots at 25.4 mm (1 in.) from the footing height and 2 string pots at 508 mm (20 in.). One pot per height was located on both the north and south faces of the experimental setup and was attached to the embedded threaded bars described in Section 4.8.1 (see Figure 4.8).

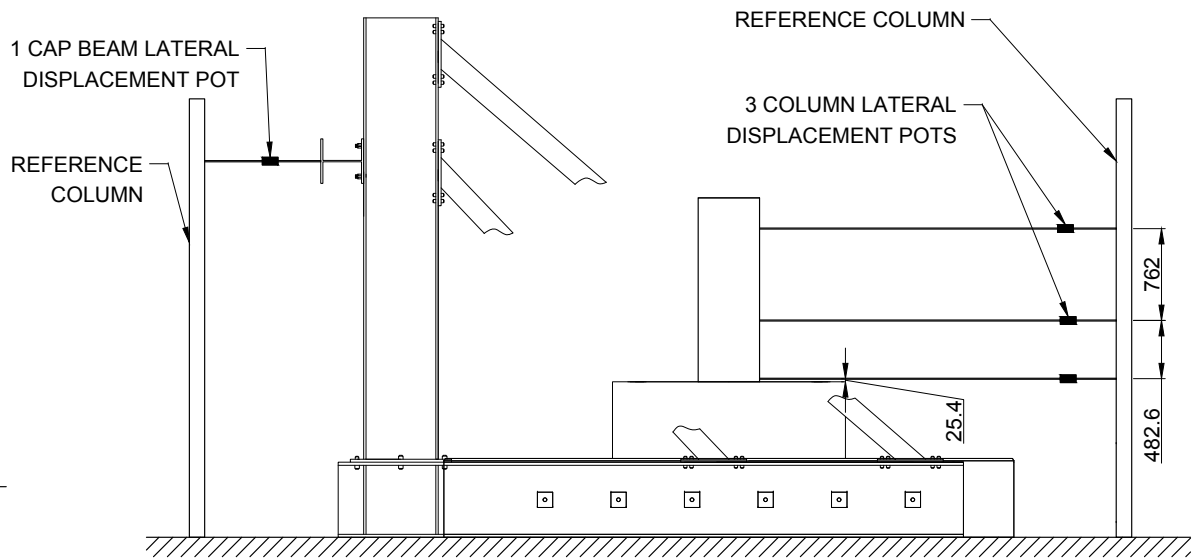


Figure 4.7 Lateral displacement potentiometers

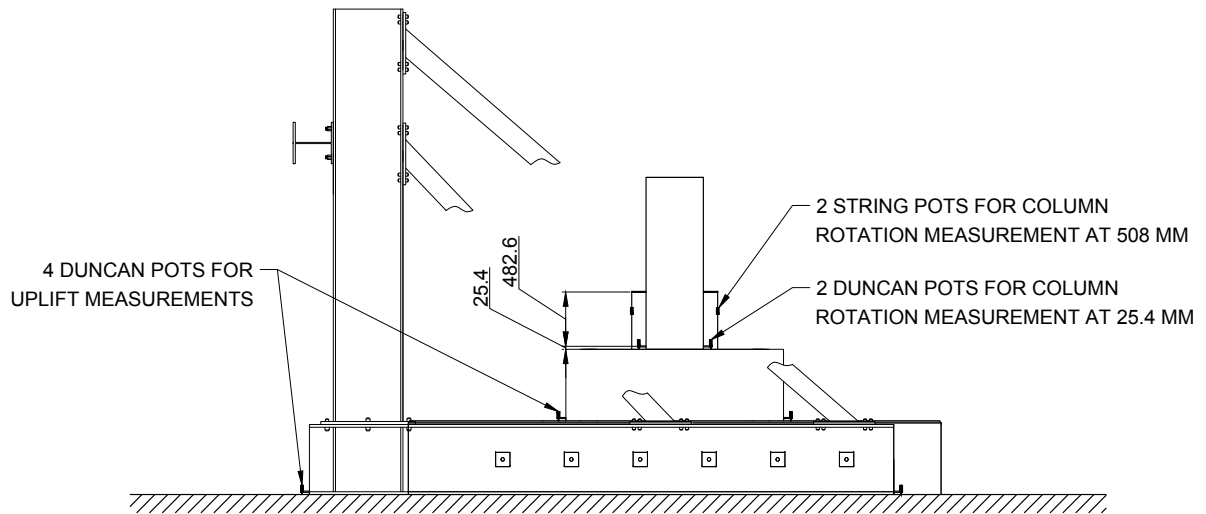


Figure 4.8 Rotation and uplift potentiometers

4.8.3. COLUMN LONGITUDINAL REINFORCEMENT STRAIN

Strain in longitudinal reinforcement bars was measured using a 12-strain-gauge layout, as shown in Figure 4.9. For all specimens, strain gauges were located in the two longitudinal bars nearest to the south and north faces of the column. Per bar, two strain gauges, one on the inner side of the bar and one on the outer side, were placed at three

different heights: interface footing-column, 190.5 mm (7.5 in.) from the interface (inside the column), and 190.5 mm (7.5 in.) below the interface (inside the footing).

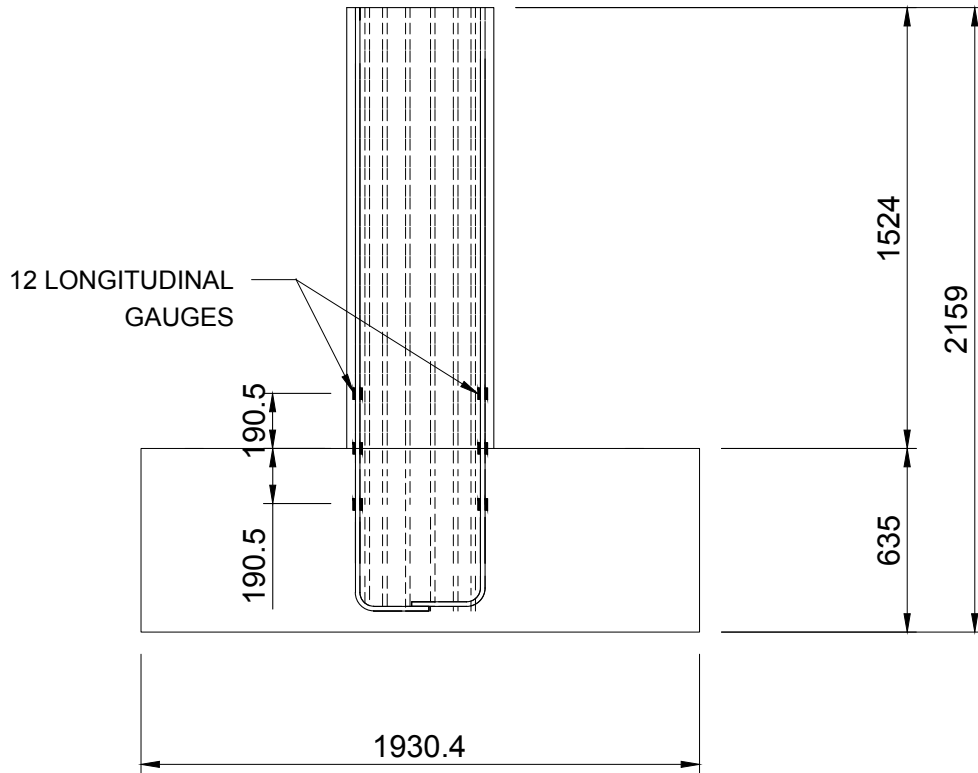


Figure 4.9 Strain gauge layout

4.8.4. DATA ACQUISITION

LabVIEW, a National Instruments data acquisition system, was used during these experiments to read the initial voltage and its variations for each potentiometer and strain gauge from the instrumentation plan. LabVIEW converted these voltage variations into mechanical units of strain and displacement accordingly.

4.9. SPECIMEN CONSTRUCTION

The specimen construction process began with the reinforcement steel assembly for the footing, followed by the column assembly Figure 4.10(a). Then, once the footing formwork was complete, the assembled steel was placed inside it.

Casting took place in two phases: first, the three footings were cast on January 23, 2026, as shown in Figure 4.10(b). Later, once the test setup was in place, the formwork for the three columns was built (Figure 4.10(c)), and lastly the columns were cast on March 18, 2026 (Figure 4.10(d)).

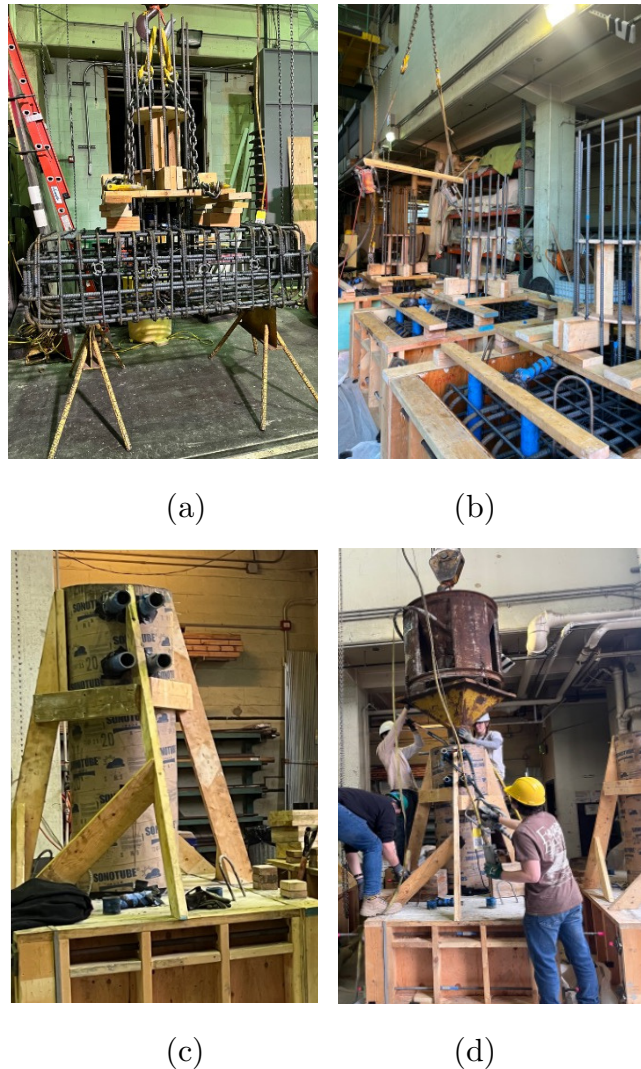


Figure 4.10 Specimen construction process

CHAPTER 5: OBSERVED DAMAGE

This chapter summarizes the damage progression observed during testing. The damage states considered included:

- first yield of longitudinal reinforcement,
- significant flexural cracking,
- initiation of shear cracking,
- significant spalling,
- bar buckling, hoop fracture,
- loss of 20% lateral strength,
- and loss of axial load capacity.

Some of these damage states were not observed in the tested specimens, such as bar buckling and hoop fracture. They are included here to maintain consistency with N&P tests. Cycle designations refer to the last identifiable data point at which the damage state was observed or evaluated. This designation includes the cycle number and the stage of the cycle, using ‘N’ for the north-direction loading peak, ‘S’ for the south-direction loading peak, and ‘E’ for the end of the cycle.

5.1. FIRST YIELD OF COLUMN LONGITUDINAL REINFORCEMENT

From tensile tests on longitudinal reinforcement, a yield strain of $\varepsilon_y = 0.0023$ was determined. The first yield displacement was computed as the displacement at the data point where the first strain gauge installed on the longitudinal bars farthest from the neutral axis in the loading direction reached ε_y . Table 5.1 summarizes the measured first yield results for each specimen, along with the measured crack widths at this stage.

Table 5.1 Effective lateral force and column top displacement at first yield

Specimen	Cycle Designation	Maximum Observed Crack Width		Effective Force at First Yield		Displacement at First Yield	
		(mm)	(in.)	(kN)	(kip)	(mm)	(in.)
S1	3N	0.3	0.01	289.2	65.0	6.29	0.25
S3	4N	0.3	0.01	271.5	61.0	6.79	0.27
S6	11N	0.25-0.38*	0.01-0.015*	282.6	63.5	5.39	0.21

*Crack width from the closest cycles with crack measurements

5.2. SIGNIFICANT FLEXURAL CRACKING

The significant flexural cracking damage state was defined as the point at which flexural cracks reached a width of at least 0.5 mm (0.02 in.). Table 5.2 includes the cycle at which cracks of this width were observed as well as the maximum effective lateral force and displacement recorded until that point. Figure 5.1 shows this damage state in each specimen.

Table 5.2 Effective lateral force and column top displacement at significant flexural cracking damage state

Specimen	Cycle Designation	Maximum Observed Crack Width		Maximum Effective Force		Maximum Displacement	
		(mm)	(in.)	(kN)	(kip)	(mm)	(in.)
S1	4S	0.5	0.02	339.5	76.3	6.9	0.27
S3	6N	2.0	0.08	306.9	69.0	8.7	0.34
S6	13S	0.5	0.02	336.6	75.7	6.8	0.27

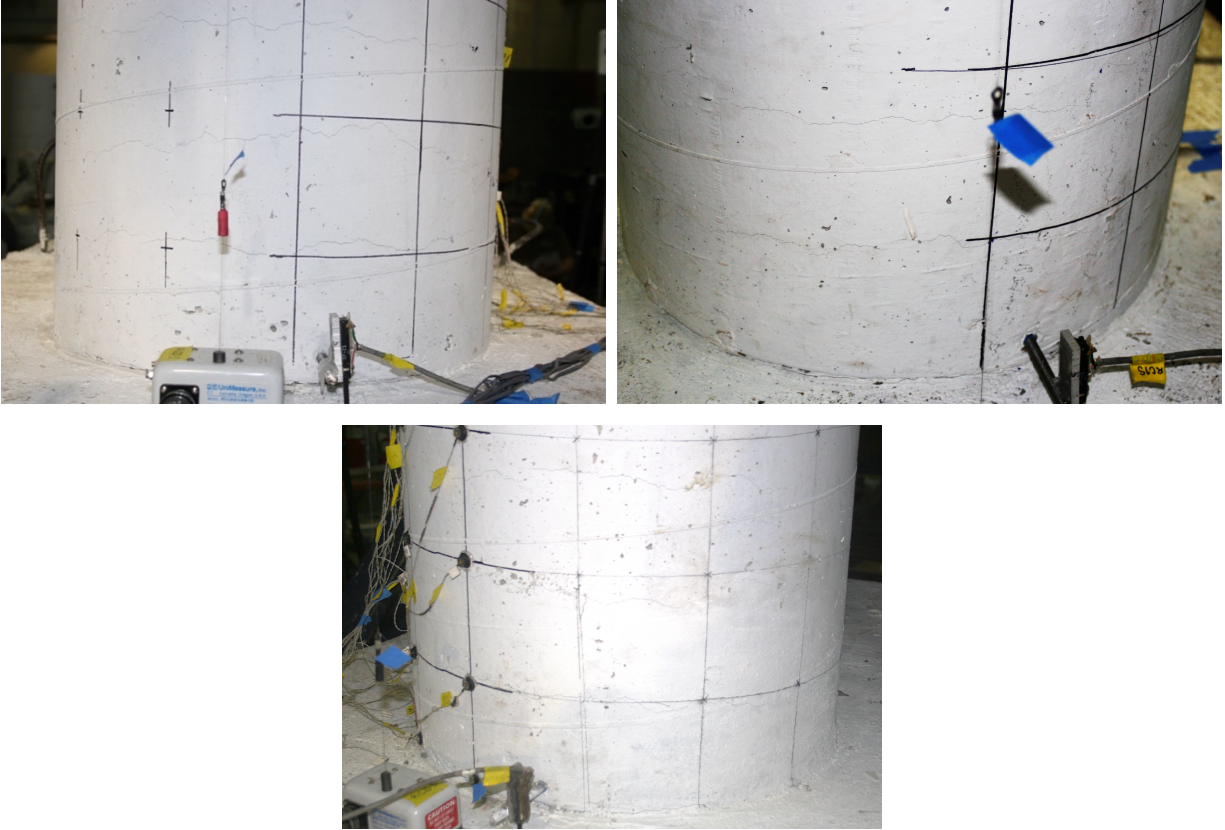


Figure 5.1 Significant flexural damage state in specimen S1 (top-left), S3 (top-right), and S6 (bottom)

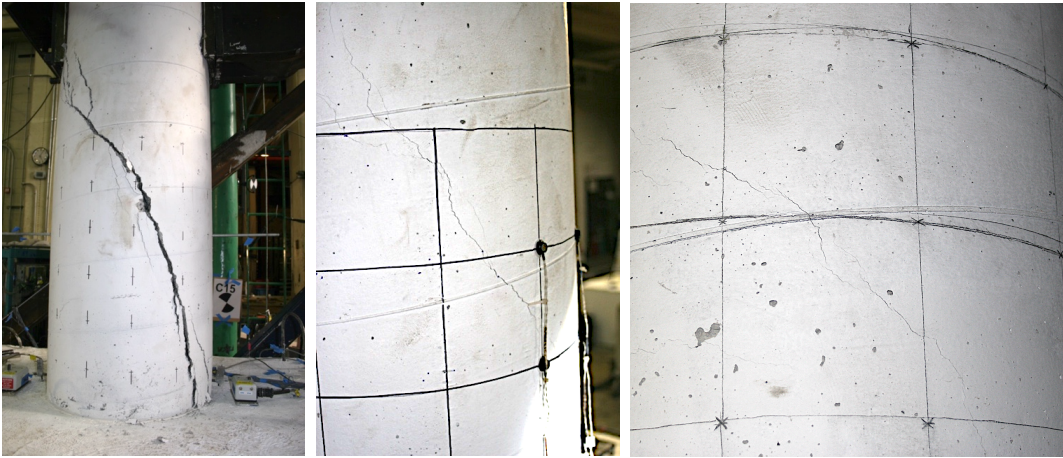
5.3. INITIATION OF SHEAR CRACKING

This damage state was defined as the first observation of a clear diagonal crack appearing in the column associated with shear damage. The observed cycle and maximum effective force and displacement per specimen at this damage state are presented in Table 5.3.

Additionally, Figure 5.2 shows how the length and depth of the first diagonal crack varied per specimen. Specimen S1 (Figure 5.2 (left)) presented a long diagonal crack with a 0.76 mm (0.03 in.) width; the initial shear crack on specimen S3 (Figure 5.2 (center)) was shorter and had a 0.65 mm (0.26 in.) width; the initial shear crack on S6 (Figure 5.2 (right)) was the shortest and had a 0.25 mm (0.01 in.) width.

Table 5.3 Effective lateral force and column top displacement at initiation of shear cracking

Specimen	Cycle Designation	Maximum Observed Crack Width		Maximum Effective Force		Maximum Displacement	
		(mm)	(in.)	(kN)	(kip)	(mm)	(in.)
S1	5S	0.75	0.03	388.8	87.4	10.8	0.43
S3	5-1S	0.8	0.03	306.9	69.0	8.7	0.34
S6	16S	0.5	0.02	354.7	79.7	8.2	0.33

**Figure 5.2 Initiation of shear cracks per specimen: (left) S1, (center) S3, and (right) S6**

5.4. RESIDUAL CRACKING

The criterion for defining the residual cracking damage state was the presence of cracks with widths of at least 0.25 mm (0.01 in.) at unloaded phases of the cycle. Table 5.4 presents the identified cycles for this damage state, along with the associated maximum effective force and displacement per specimen.

Table 5.4 Effective lateral force and column top displacement at residual cracking damage state

Specimen	Cycle Designation	Maximum Observed Crack Width		Maximum Effective Force		Maximum Displacement	
		(mm)	(in.)	(kN)	(kip)	(mm)	(in.)
S1	5S	0.75	0.03	388.8	87.4	10.8	0.43
S3	5-1E	0.25	0.01	306.9	69.0	8.7	0.34
S6	16E	0.25	0.01	354.7	79.7	8.2	0.33

5.5. SIGNIFICANT FLEXURAL SPALLING

The significant flexural spalling damage state criterion defined in Ranf et al.(2006) was related to spalling reaching a height of 1/10 of the cross-section depth. However, no significant spalling due to flexural damage was observed in any of the specimens. Local concrete cover loss associated with diagonal shear cracking was observed during the final stages of the tests, but it was not considered part of this category's definition.

5.6. BAR BUCKLING

Bar buckling is a damage state commonly associated with flexure failures due to plastic hinge deformation. In the present tests, no clear buckling of longitudinal reinforcement was observed at the base of the specimens. Instead, local deformations of longitudinal bars were observed along the diagonal shear-cracking region during the final cycles before axial load capacity was lost. Thus, these deformations were considered reinforcement distortion associated with shear failure rather than bar buckling.

5.7. HOOP FRACTURE

No clear evidence of hoop fracture was observed. Several hoops appeared discontinuous near the diagonal shear crack; however, these discontinuities coincided with hoop overlap locations and were interpreted as opening of the lap region rather than fracture of the hoop reinforcement.

Specimens S1 and S3 had overlapped ends over the west and east faces of the column. In specimen S1 (Figure 5.3 (left)), one open hoop was observed on the west face at 533.4 mm (21 in.) above the column base. In specimen S3 (Figure 5.3 (middle)), two open hoops were observed on the east face at heights of 406.4 mm (16 in.) and 609.6 mm (24 in.), and two on the west face at heights of 533.4 mm (21 in.) and 736.6 mm (29 in.).

Specimen S6 (Figure 5.3 (right)) had overlapped ends located randomly and exhibited one open hoop on the east face, 431.8 mm (17 in.) from the base of the column.



Figure 5.3 Opened hoops observed per specimen: S1 (left), S3 (middle), and S6 (right)

Once all tests were complete, the clear cover over the area where the overlapping ends of the hoops met was removed to assess for existing hoop fractures or to confirm hoop opening. As shown in Figure 5.3, some hoops exhibited a reduced overlapping length and evidence of sliding from its original position, confirming that no hoop fractures were observed at any of the tested specimens.



Figure 5.4 Final state of overlapped ends in hoops after clear cover removal, showing a reduction in length (initially was 4 in.)

5.8. LOSS OF 20% OF LATERAL STRENGTH

The cycle at which the effective force last achieved 80% of the maximum recorded lateral force for each specimen is presented in Table 5.5.

Table 5.5 Effective lateral force and column top displacement at 20% loss of the lateral strength

Specimen	Cycle Designation	Maximum Observed Crack Width		Maximum Effective Force		Maximum Displacement	
		(mm)	(in.)	(kN)	(kip)	(mm)	(in.)
S1	5S	0.75	0.03	388.8	87.4	10.8	0.43
S3	9N	>7.0*	>0.28*	306.9	69.0	9.7	0.38
S6	26S	>5.08*	>0.2*	375.2	84.3	15.9	0.62

* From the last cycle with crack measurements (7N for S3 and 26N for S6)

5.9. LOSS OF AXIAL LOAD CAPACITY

The loss of axial load capacity damage state was characterized by a sudden drop in the axial load, and instability was observed that required removal of axial load from the column. This damage state occurred during the subsequent loading excursion in the same direction after the 20% loss-of-lateral-strength threshold was reached.

The maximum effective forces and lateral displacements recorded at this point are shown in Table 5.6 per specimen. The cycle designation for this damage state refers to the loading direction at the time of failure; in each case, failure occurred before the target displacement for that peak was reached.

Additionally, Figure 5.5, Figure 5.6, and Figure 5.7 show the final state of each specimen from multiple angles. Shear failure damage mechanisms are observed in all three tests.

Table 5.6 Effective lateral force and column top displacement at loss of axial load capacity

Specimen	Cycle Designation	Maximum Observed Crack Width**		Maximum Effective Force		Maximum Displacement	
		(mm)	(in.)	(kN)	(kip)	(mm)	(in.)
S1	6S*	>0.75	>0.03	388.8	87.4	17.0	0.67
S3	10N*	>7.0	>0.28	306.9	69.0	12.8	0.5
S6	27S*	>5.08	>0.2	375.2	84.3	19.2	0.75

*Failure occurred before reaching the target displacement for this peak in the cycle

** From the last cycle with crack measurements (5S for S1, 7N for S3, and 26N for S6)



Figure 5.5 Specimen S1 final state: southeast face (left), southwest face (center), and north face (right)



Figure 5.6 Specimen S3 final state: southwest face (left), northeast face (center), and north face (right)



Figure 5.7 Specimen S6 final state: southeast face (left), west face (center), and northwest face (right)

CHAPTER 6: MEASURED EXPERIMENTAL RESPONSE

6.1. MATERIAL PROPERTIES

6.1.1. CONCRETE

The three column footings were cast on one day, and the three columns were cast on another. Early measurements of the compressive strength for the footing concrete indicated that the compressive strength on the test day (Table 6.1) would exceed the target value of 6000 psi, so the concrete mix was modified to cast the columns.

Table 6.1 and Table 6.2 present the results of the concrete tests performed on 152 x 305 mm (6 x 12 in.) concrete cylinders collected from the footing and column concrete batches, respectively. These material tests were conducted within two days of the specimen test date. Each value represents the average of the measured values from two tests.

The measured compressive strengths for the footing concrete exceeded the target strength by approximately 30% on average. The column concrete had an average compressive strength approximately 2% above the target, and each individual strength was within 10% of the target value.

Table 6.1 Footing concrete properties at time of testing

Specimen	Age at Test (days)	SI			USCS		
		f'_c	f'_t	E_c	f'_c	f'_t	E_c
		(MPa)	(MPa)	(GPa)	(psi)	(psi)	(ksi)
S1	101	55.6	3.7	39.2	8062	529	5686
S3	89	53.2	3.4	39.7	7714	497	5758
S6	108	54.9	3.9	38.5	7970	565	5588

Table 6.2 Column concrete properties at time of testing

Specimen	Age at Test (days)	SI			USCS		
		f'_c (MPa)	f'_t (MPa)	E_c (GPa)	f'_c (psi)	f'_t (psi)	E_c (ksi)
S1	47	42.9	3.1	37.2	6219	451	5389
S3	35	38.6	3.0	37.5	5603	432	5440
S6	54	45.4	2.8	36.9	6586	412	5355

6.1.2. STEEL

Longitudinal reinforcement bars for the three specimens were ordered from the same steel batch to limit variation among the specimens. Tension tests were performed on samples of the longitudinal and transverse reinforcement to determine their properties; yield and ultimate stresses are summarized in Table 6.3 for both types of reinforcement. The yield stress of the longitudinal steel was approximately 10% above the minimum value, and the yield stress of the transverse reinforcement was approximately 5% above the minimum value.

Figure 6.1 and Figure 6.2 show the measured stress-strain curves from the longitudinal and transverse reinforcement samples, respectively. The average measured yield strain of longitudinal reinforcement was 0.0023 mm/mm. Note that the transverse reinforcement had been bent into hoops before arriving at the test laboratory, so although an attempt was made to straighten them, the strains at low stresses (derived from an extensometer) should not be considered to be reliable.

Table 6.3 Steel properties

Material	SI		USCS	
	f_y (MPa)	f_u (MPa)	f_y (ksi)	f_u (ksi)
Longitudinal Reinforcement Bar (#5 Gr60)	457.2	752.3	66.3	109.11
Transverse Reinforcement Wire (W2.5)	436.2	646.5	63.3	93.8

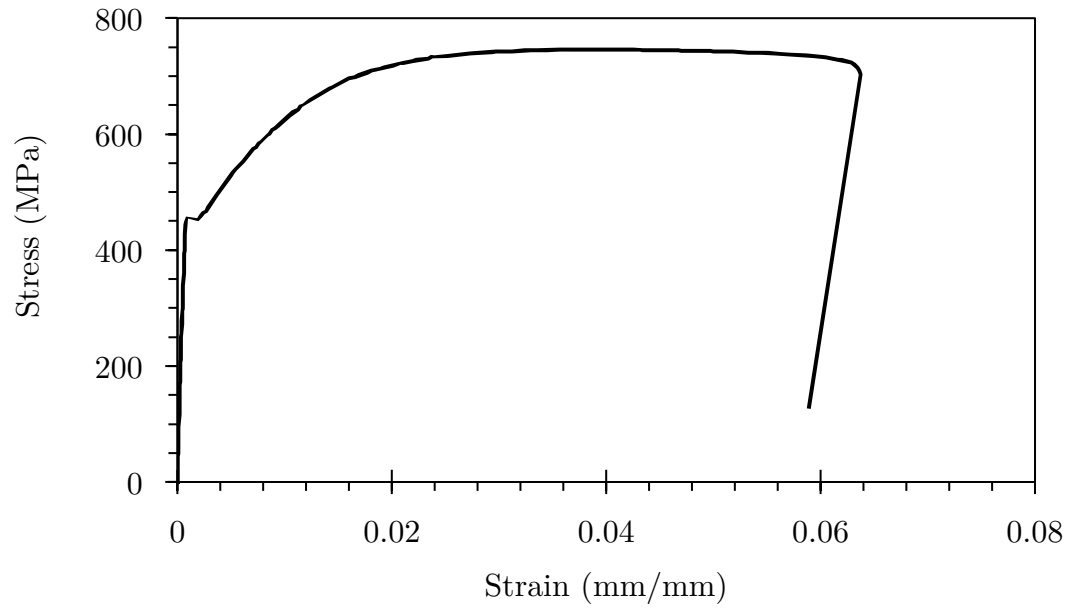


Figure 6.1 Stress-strain response of longitudinal reinforcement

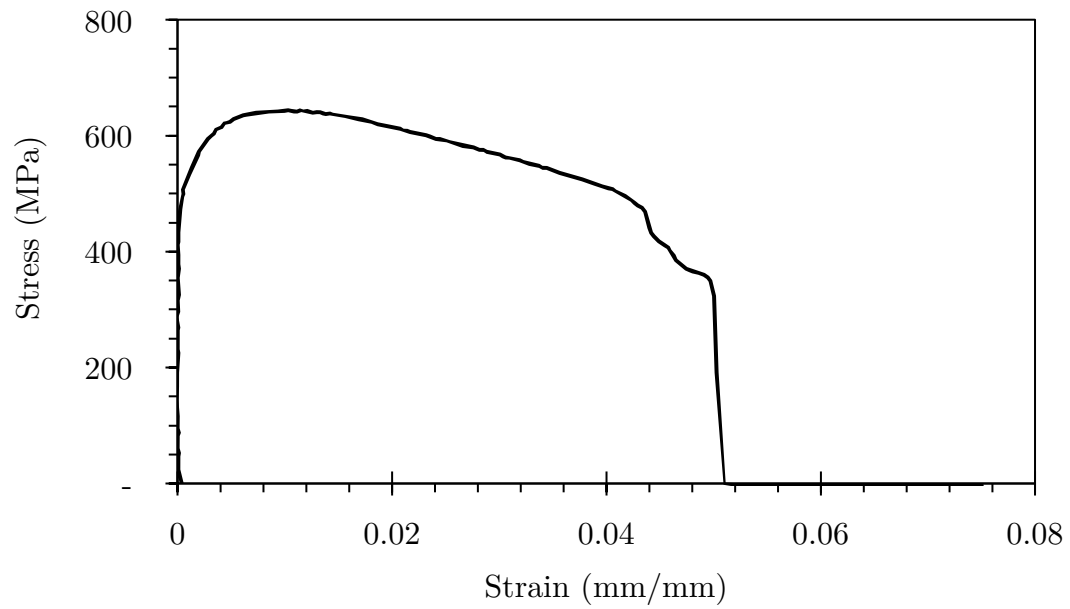


Figure 6.2 Stress-strain response of transverse reinforcement

6.2. EFFECTIVE FORCE-DISPLACEMENT RESPONSES

6.2.1. COLUMN DISPLACEMENT ACCURACY

The displacement at the top of the column was taken as the recorded displacement from the string potentiometer located at the load-application height of 1270 mm (50 in.). This displacement was validated by comparing it with the sum of the displacements of the loading beam (where the actuator was attached to the reaction frame) measured from a fixed reference and the actuator LVDT measurement.

No uplift corrections were applied to the measurements because the drift induced by footing and reaction-frame uplift was negligible relative to the drift ratio measured at the top of the column. Across all specimens, the induced drift from footing uplift and reaction-frame uplift did not exceed 0.01% and 0.013%, respectively.

6.2.2. P-DELTA EFFECT ADJUSTMENTS

The forces recorded from the MTS actuator load cell corresponded to the horizontal shear force, excluding the effects on moment (or effective force) associated with P - Δ effects. Following the P - Δ effect adjustments defined in Section 3.3.1, the effective force was computed using Eqs. 3.1b, 3.2, and 3.3.

6.2.3. NOISE CORRECTIONS

Two data channels were post-processed to reduce measurement noise: top-of-column displacement and rotation at 20 in. on the north face. The top-of-column displacement exhibited occasional spikes, which were filtered using a rolling median and a local deviation threshold, thereby removing isolated data-point spikes while preserving the hysteretic trend.

The rotation at 20 in. on the north face exhibited high-frequency oscillations, which were treated without altering the broader recorded response by applying a four-pass weighted moving average with weights of 0.4 for neighboring values and 0.2 for the current value. Raw data backups were kept and compared with the noise-reduced records for quality control, exhibiting the same trends as the original measurements.

6.2.4. EFFECTIVE FORCE VS DRIFT RATIO

The recorded histories of the effective force and drift ratio for the three specimens are shown in Figure 6.3, illustrating the shared drift-ratio demands and the increasing number of cycles per target drift ratio as described in Section 4.3.

The results in Figure 6.3 (top and middle) show the brittle behavior observed in Specimens S1 and S3, characterized by a sudden drop in effective lateral strength once the first diagonal crack formed. Specimen S6 exhibited a more gradual strength degradation and sustained larger drift demands before loss of axial load capacity, as shown in Figure 6.3 (bottom) and Figure 6.4.

Figure 6.4 includes the envelope of the effective force versus drift response for the three specimens. Figure 6.4 shows that, in terms of peak effective lateral force, Specimen S6 reached the largest value, followed by Specimen S1 and then Specimen S3.

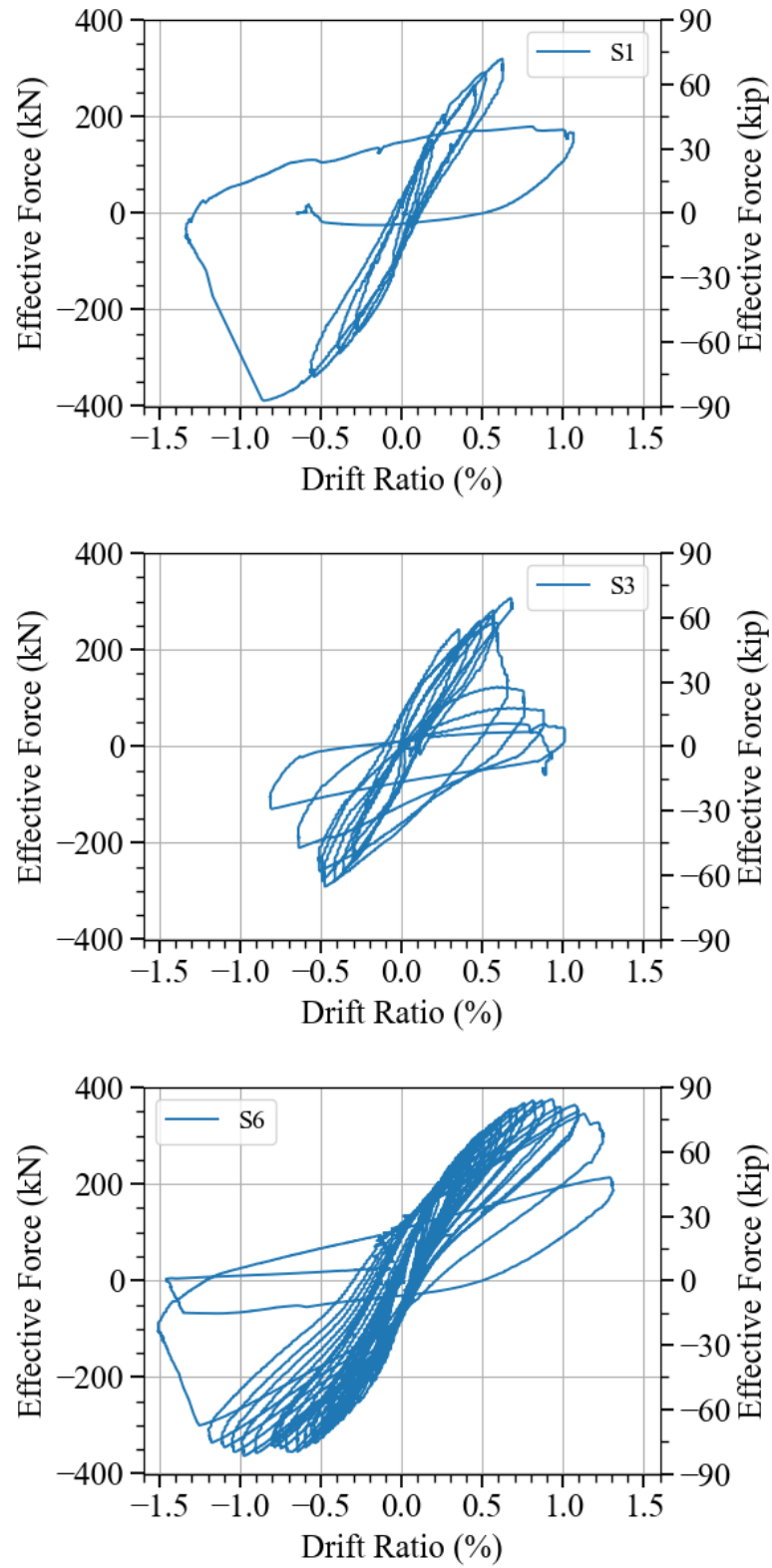


Figure 6.3 Effective force vs drift ratio histories: S1 (top), S3 (middle), and S6 (bottom)

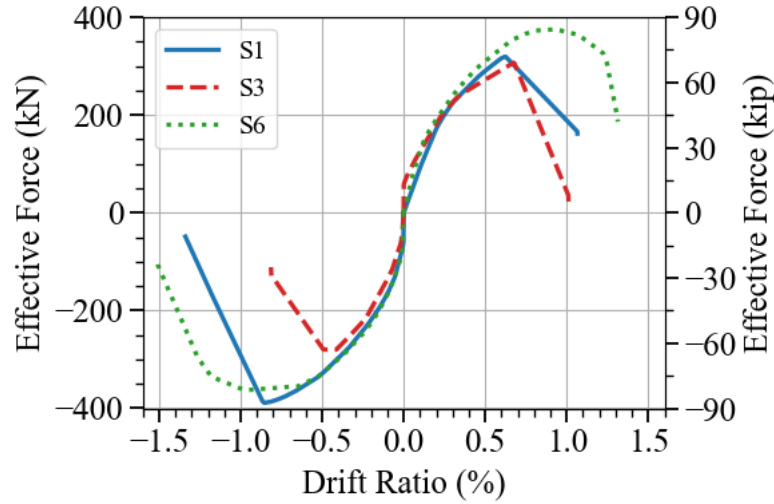


Figure 6.4 Effective force vs drift ratio envelopes

6.3. DISTRIBUTION OF LATERAL DEFORMATION

6.3.1. DRIFT RATIO CONTRIBUTIONS PER HEIGHT

The contribution of the total drift at different heights was computed by normalizing the recorded lateral displacements at heights of 508 mm (20 in.) and 25.4 mm (1 in.) above the column base to the shear span of the specimens, 1,270 mm (50 in.).

The normalized lateral displacement for the three specimens is included in Figure 6.5Figure 6.6 for 508 mm (20 in.) and 25.4 mm (1 in.), respectively.

Figure 6.5 shows that once diagonal cracks appeared in Specimens S1 and S3, the lateral displacement experienced at 508 mm no longer followed the approximately linear displacement profile observed before diagonal cracking and started contributing more to the total drift at the top of the column, whereas for Specimen S6, this happened once the diagonal crack was significantly wider and the lateral strength of the column experienced a 20% loss.

Results from Figure 6.6 indicate that the displacement at the base of the column did not contribute significantly at any point to total displacement in the history of any specimen.

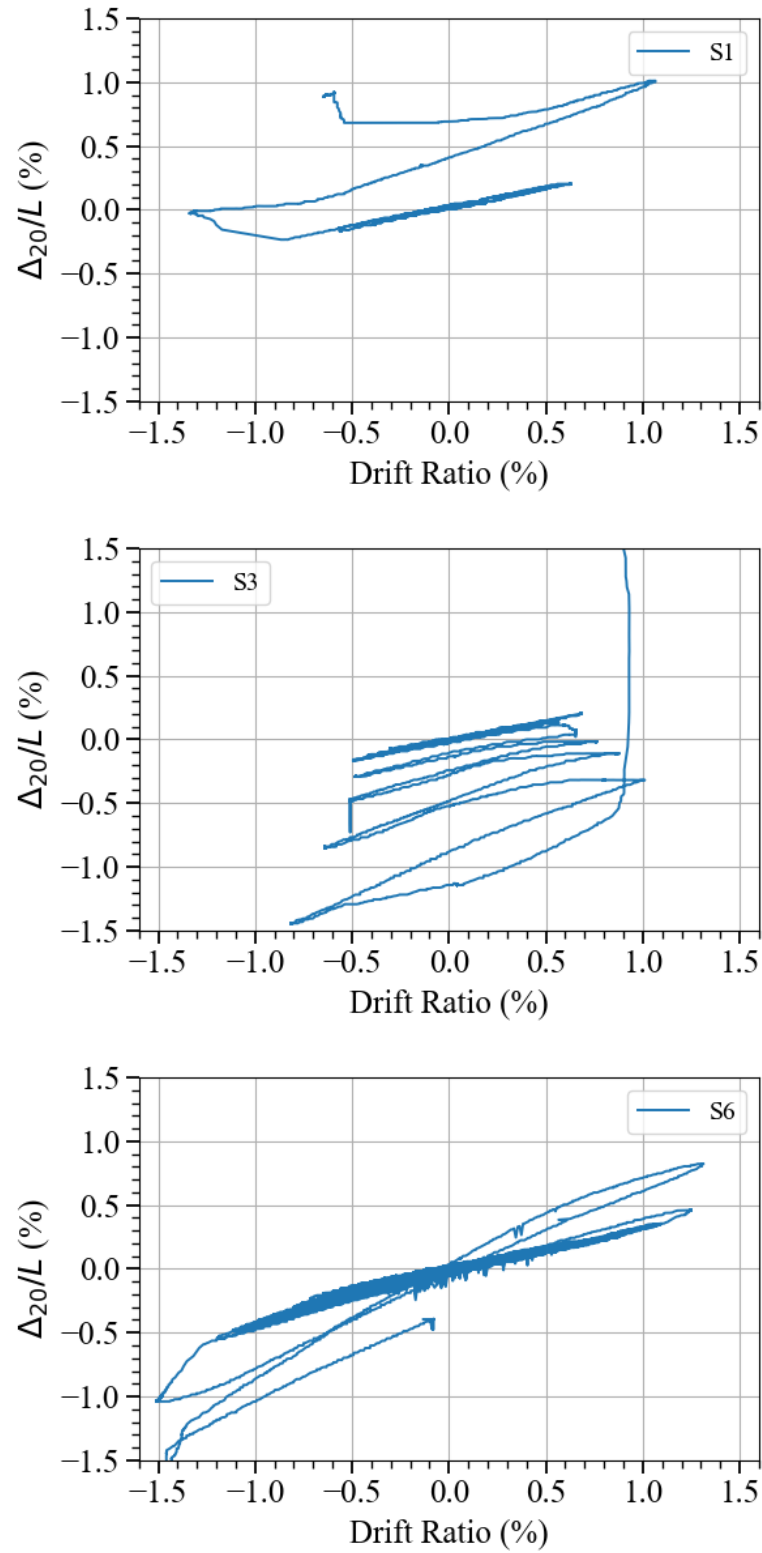


Figure 6.5 Normalized drift ratio at 508 mm (20 in.) height vs drift ratio histories: S1 (top), S3 (middle), and S6 (bottom)

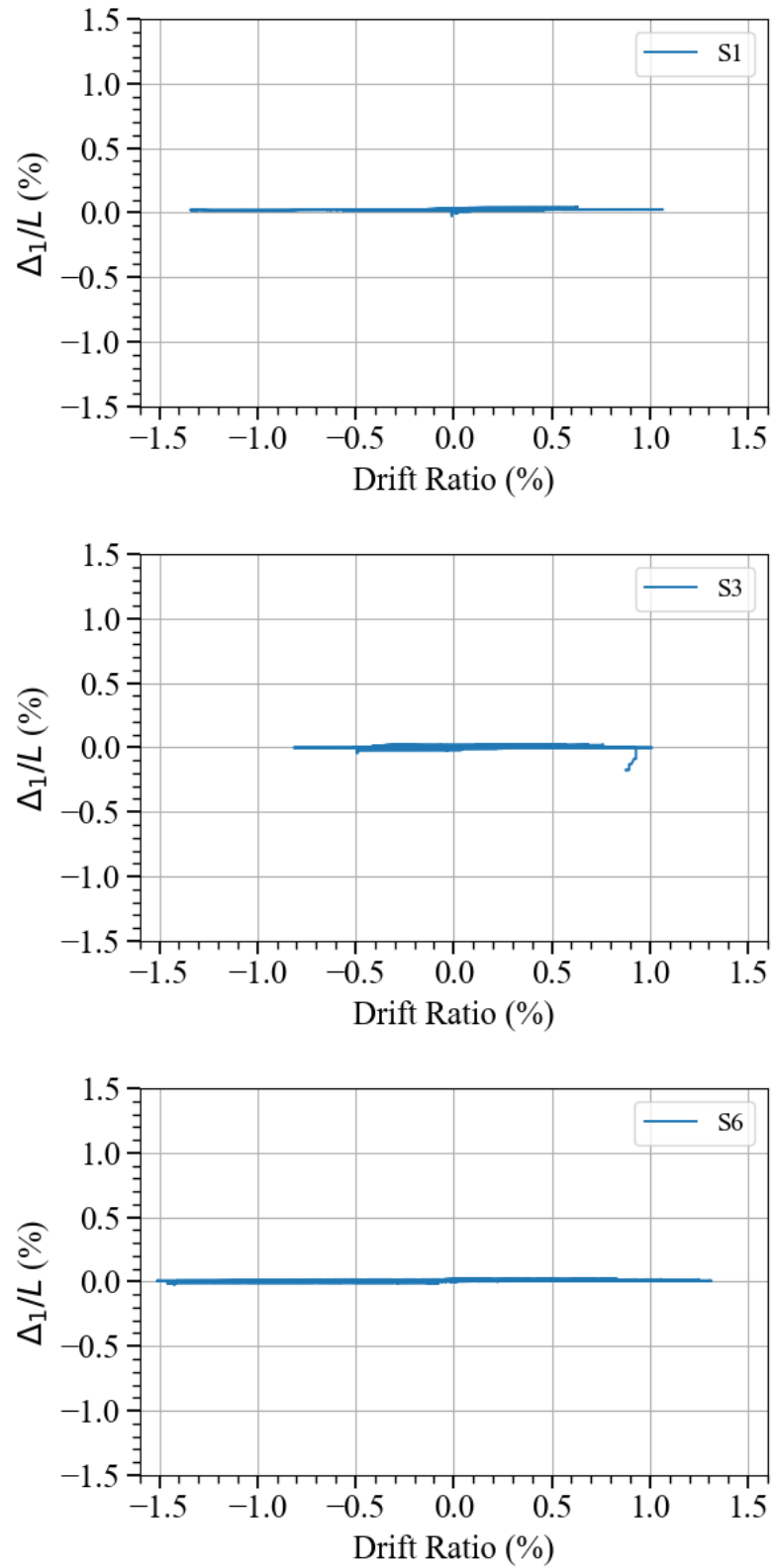


Figure 6.6 Normalized drift ratio at 25.4 mm (1 in.) height vs drift ratio histories: S1 (top), S3 (middle), and S6 (bottom)

6.3.2. DRIFT RATIO PROFILES

Drift ratio profiles in Figure 6.7 show, for each specimen, displacements at the three measured column heights, normalized by the specimen's shear span, for selected drift-ratio demands.

Figure 6.7 (top) describes the response from Specimen S1. It exhibited an approximately linear displacement profile before diagonal shear cracking. After the shear crack formed at 0.62% drift ratio, the displacement measured at 508 mm (20 in.) increased disproportionately relative to the top displacement, indicating a localized deformation mechanism associated with diagonal crack opening, as shown in the 0.75% drift ratio profile. As the diagonal crack opened wider at the following drift-ratio targets, the displacement at this mid-height point increased until failure at 0.80% drift target, where it was unable to follow the displacement of the column at the top.

Figure 6.7 (middle) depicts the drift ratio profiles for Specimen S3. It shows a response similar to that of Specimen S1: a cantilever-like deflection for smaller drift-ratio demands until the shear crack formed at 0.68% drift ratio while loading towards the south direction. After the diagonal crack formed, the displacement at a height of 508 mm (20 in.) remained leaning towards the south; it increased with drift-ratio demands but did not match the displacement at the top of the column until failure at the 0.80% drift target.

Figure 6.7 (bottom) shows the drift ratio response of Specimen S6. This Specimen exhibited a more linear profile than the others under relatively higher drift demands. Results show that the top- and middle-height displacements continued to increase, the latter at a lower rate than the former, consistent with a response initially dominated by flexural deformation near the base. It was on the half-cycle prior to failure (1.28% target drift at north), right after the 20% lateral strength loss, that the displacement increase-rate from the 508 mm (20 in.) height went up, followed by the failure when loading to the south, where the displacement at the middle was almost the same as the displacement at the top.

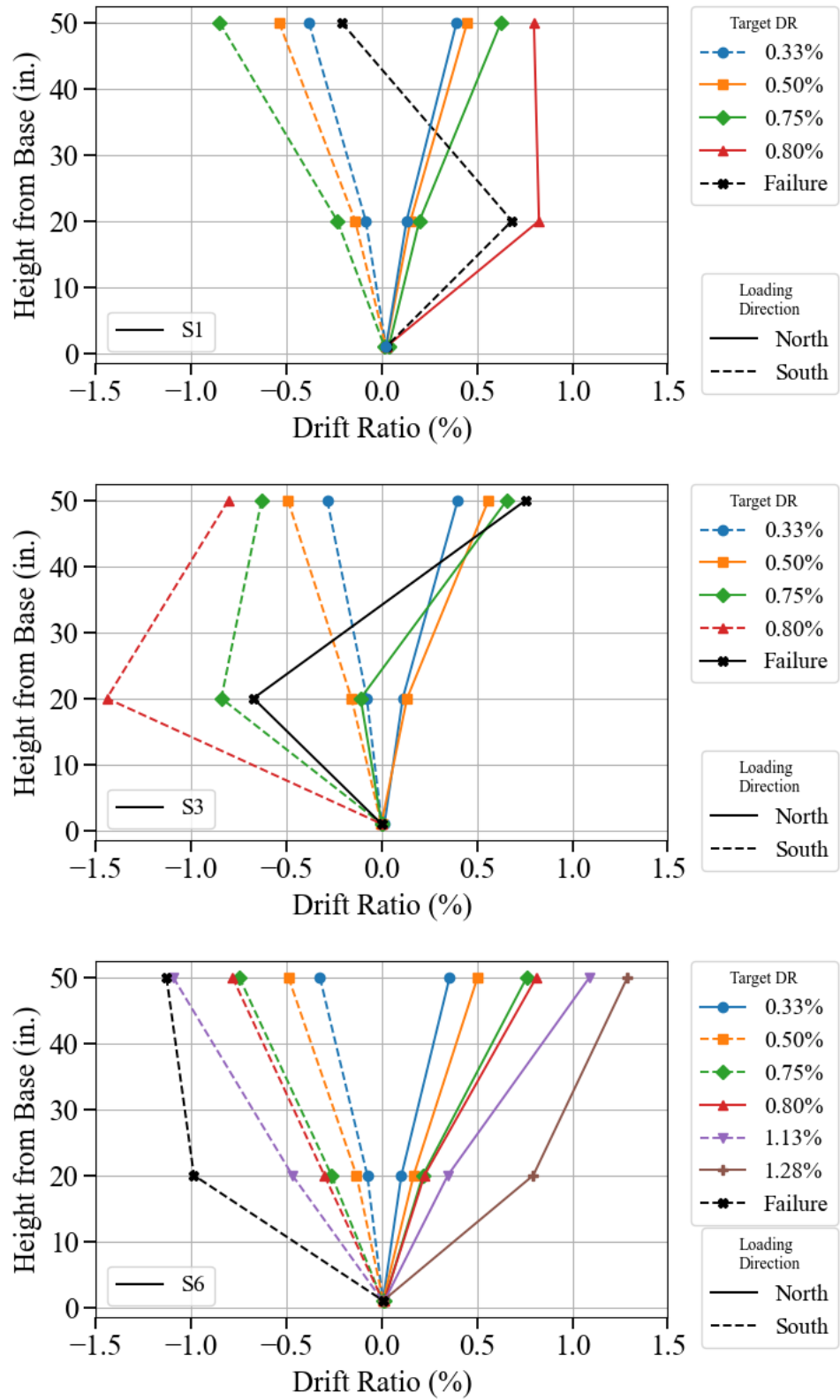


Figure 6.7 Drift ratio profiles per specimen: S1 (top), S3 (middle), and S6 (bottom)

6.4. COLUMN ROTATION

Column rotation relative to their base was measured at heights of 508 mm (20 in.) and 25.4 mm (1 in.) using vertical displacements recorded from the potentiometers, from the top of the footing to the debonded bars across the column at these heights. Rotation was computed as:

$$Rotation = \frac{l_{south} - l_{north}}{L_{pots}} \quad (6.1)$$

where l_{south} and l_{north} are the vertical displacements recorded at the pots located on the south and north faces of the column, respectively, and L_{pots} is the horizontal distance between these potentiometers.

Figure 6.8 and Figure 6.9 show the column rotation histories for each specimen at 508 mm (20 in.) and 25.4 mm (1 in.), respectively. The results in these figures indicate that, for all specimens, the rotation experienced at 1 in. represented most of the rotation recorded at 508 mm (20 in.). This suggests that localized deformation at the column-footing interface, presumably attributable to bar anchorage slip, contributed significantly to the measured rotation, especially for higher drift demands.

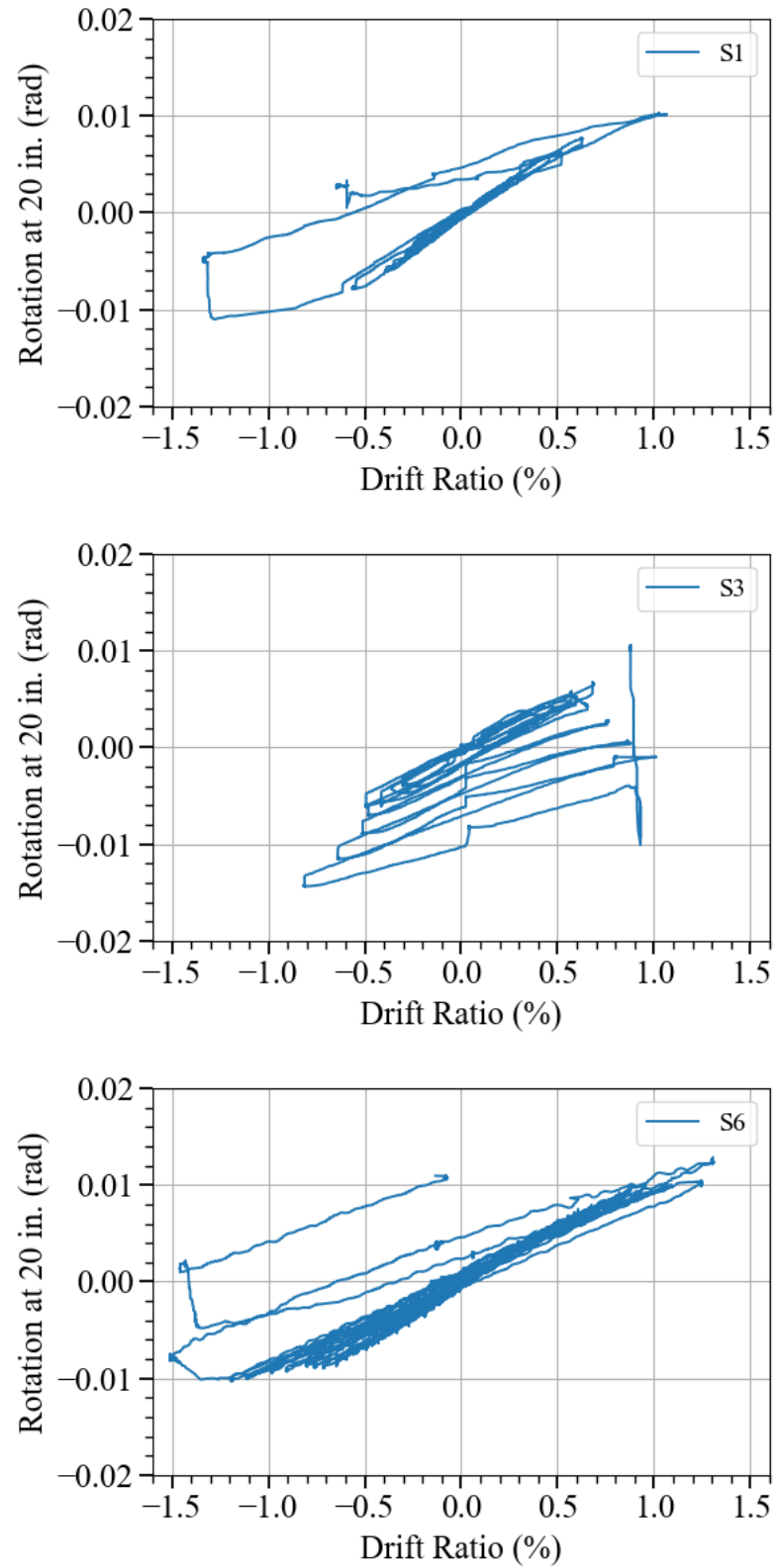


Figure 6.8 Column rotation at 508 mm (20 in.) height vs drift ratio histories: S1 (top), S3 (middle), and S6 (bottom)

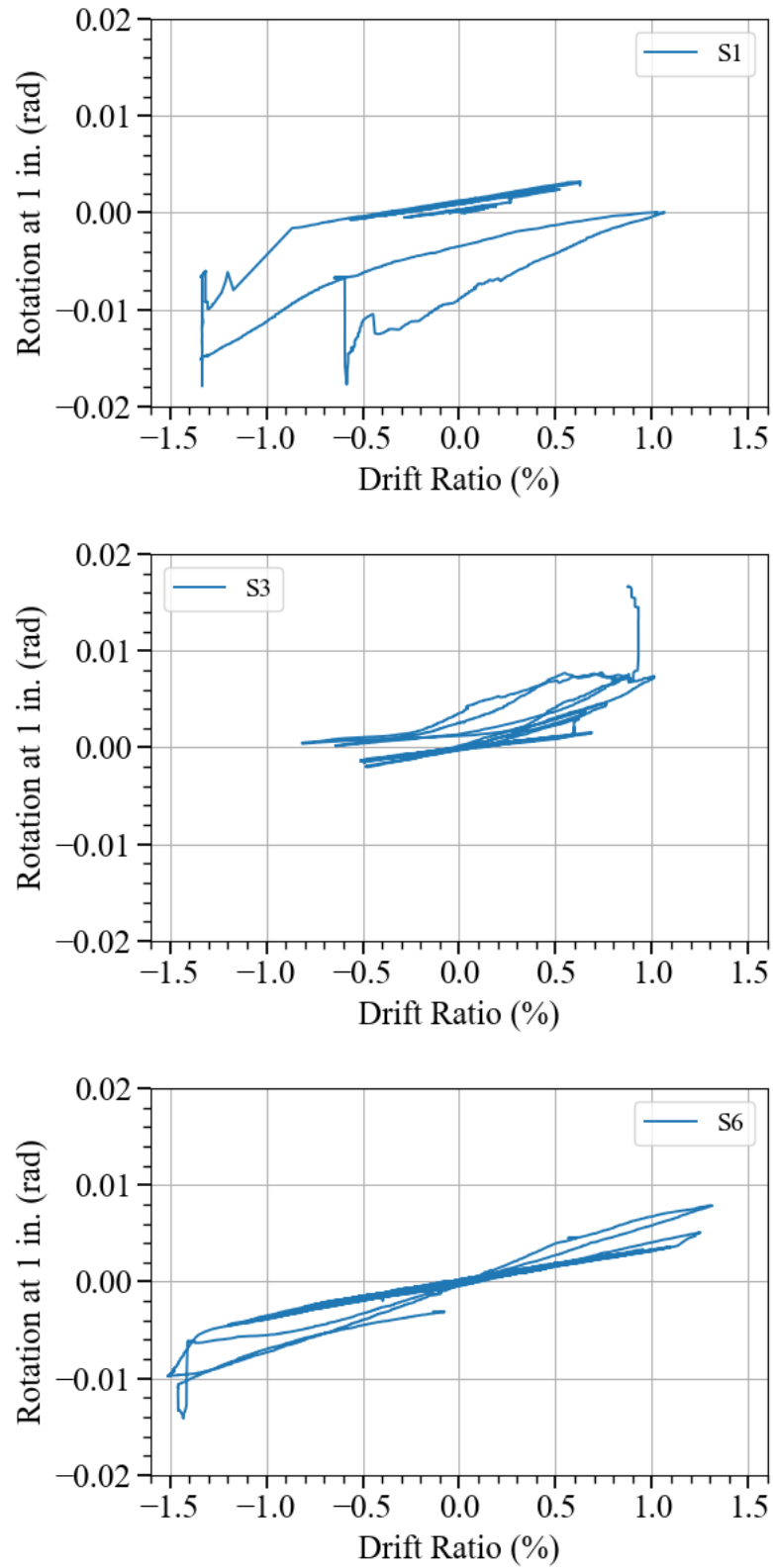


Figure 6.9 Column rotation at 25.4 mm (1 in.) height vs drift ratio histories: S1 (top), S3 (middle), and S6 (bottom)

6.5. COLUMN LONGITUDINAL REINFORCEMENT STRAINS

6.5.1. STRAIN HISTORIES

As described in Section 4.8.3, each specimen had a strain gauge layout of 12 gauges. The gauges were located at the column-footing interface and at 7.5 in. above and below the interface, for the two bars located on the North or South faces of the columns. At each location, two gauges were installed: one on the inner face of the bar and one on the outer face of the monitored bar.

Three strain gauges were damaged before testing, so the entire recording for these cases was based solely on the remaining gauge at that height. One of these damaged strain gauges was on Specimen S3 (south bar – inner face at +7.5 in. from the interface), and two were on Specimen S6 (north bar -inner face at the interface, and south bar – outer face at the interface).

Strain histories recorded from the strain gauges located at the column-footing interface on the north and south bars per specimen are shown in Figure 6.10 Figure 6.11, respectively. The strain displayed was computed as the average of the strains measured by the gauges at that location, on both the inner and outer faces of the bar, when both gauges were available, and once one of them stopped working due to damage during the experiment, the strain was that of the other gauge still recording. Once both strain gauges stopped working, the measurement and graph stopped too.

Figure 6.10 Figure 6.11 show that all the instrumented bars passed the yielding strain (0.0023). Differences in the number of cycles are reflected in the strain histories, particularly in the greater number of strain reversals recorded for specimens subjected to more cycles. The characteristic bottom deformation at failure of Specimens S1 and S3 is shown in Figure 6.10 (top and middle), where higher tensile strains are reached on the north bar, with the middle region leaning in the opposite direction of the top of the column displacement, as discussed in Section 6.3.2.

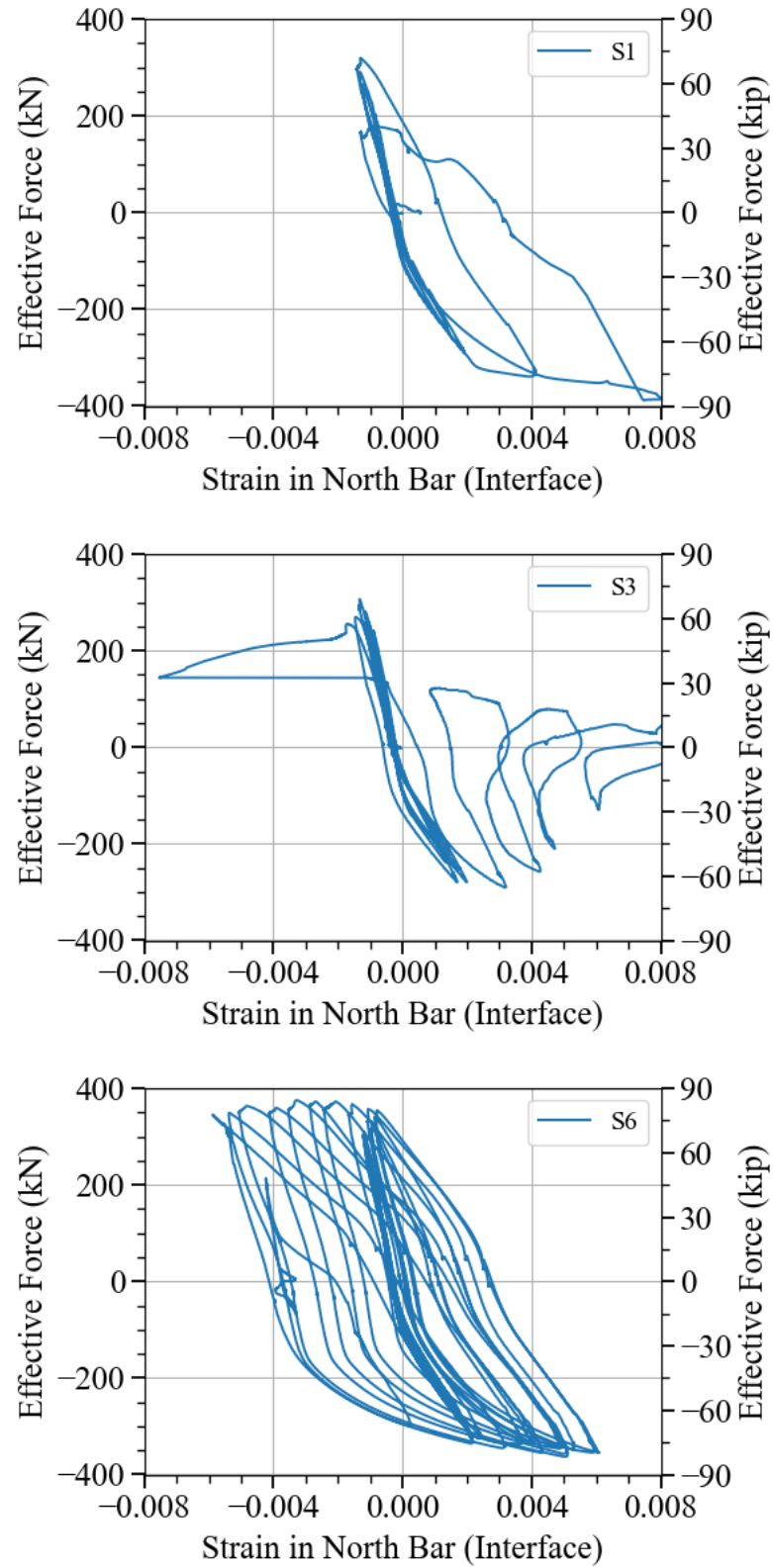


Figure 6.10 Strain history in the North bar at the interface per specimen: S1 (top), S3 (middle), and S6 (bottom)

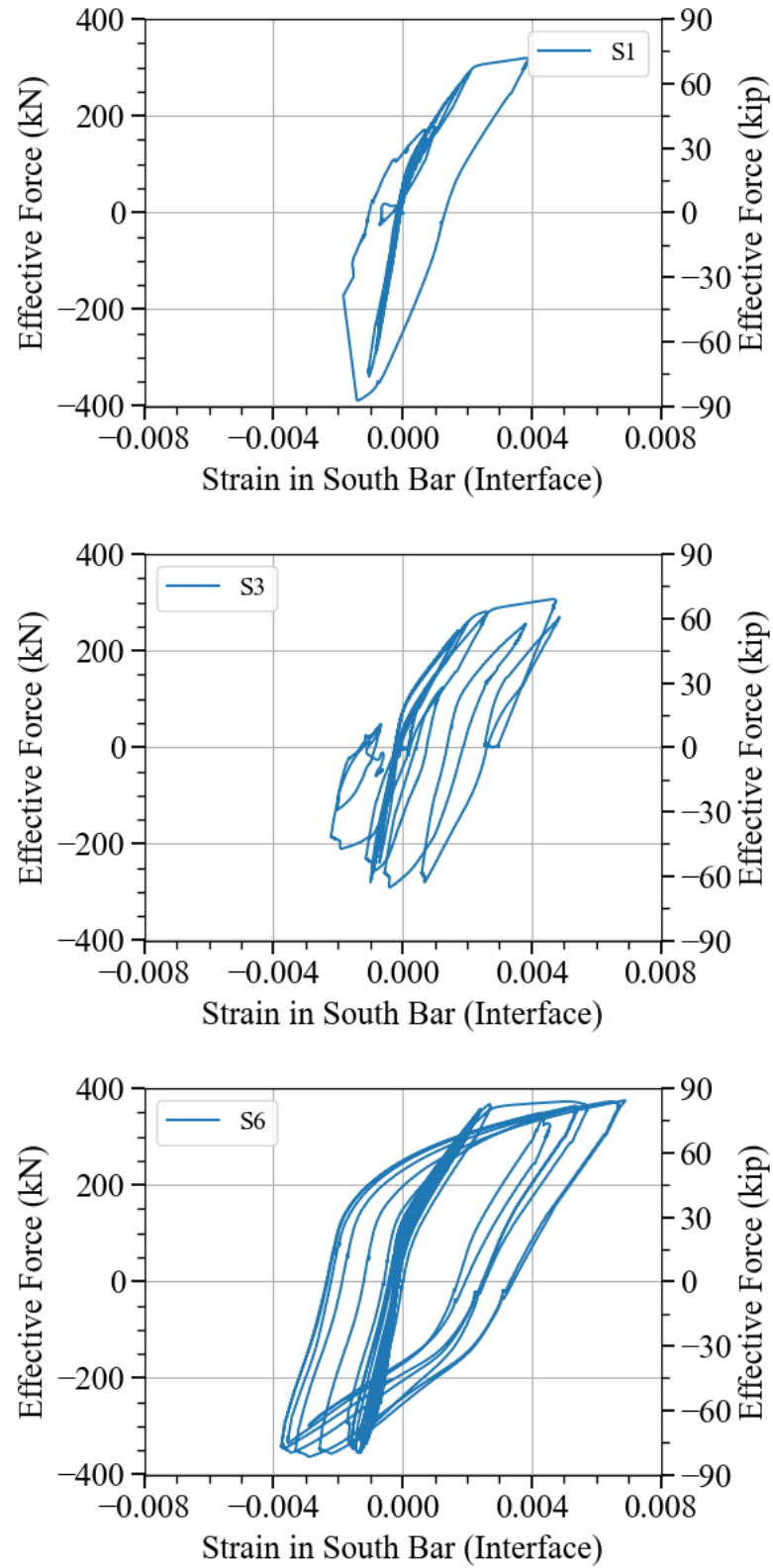


Figure 6.11 Strain history in the South bar at the interface per specimen: S1 (top), S3 (middle), and S6 (bottom)

6.5.2. STRAIN PROFILES

Based on the target drift ratios shown in Figure 6.7, the strain at the three measured elevations in the North and South bars of each specimen was identified and shown in Figure 6.12Figure 6.13. These drift ratios correspond to selected loading peaks in the North and South directions, respectively. The same average strain criterion defined in Section 6.5.1 was used to plot the strain at each data point. Once both gauges stopped recording, the strain point in the profile and the following points at that location were omitted.

Figure 6.12Figure 6.13 show that, as expected, the strain of the longitudinal reinforcement bars at the footing height varied less than the other strain measurement points across the drift ratio demand increments.

Additionally, Figure 6.12Figure 6.13 indicate that the gages in the column and at the interface height measured similar responses at lower drift ratios. Higher drift demands lead to shear crack formation and widening until failure, resulting in permanent deformations in the lower part of the column. As discussed in Section 6.3.2, this prevented the mid-height column displacement measurement from following the column top displacement and also caused higher strains at the interface level, with even opposite signs to the strain in the column away from the interface.

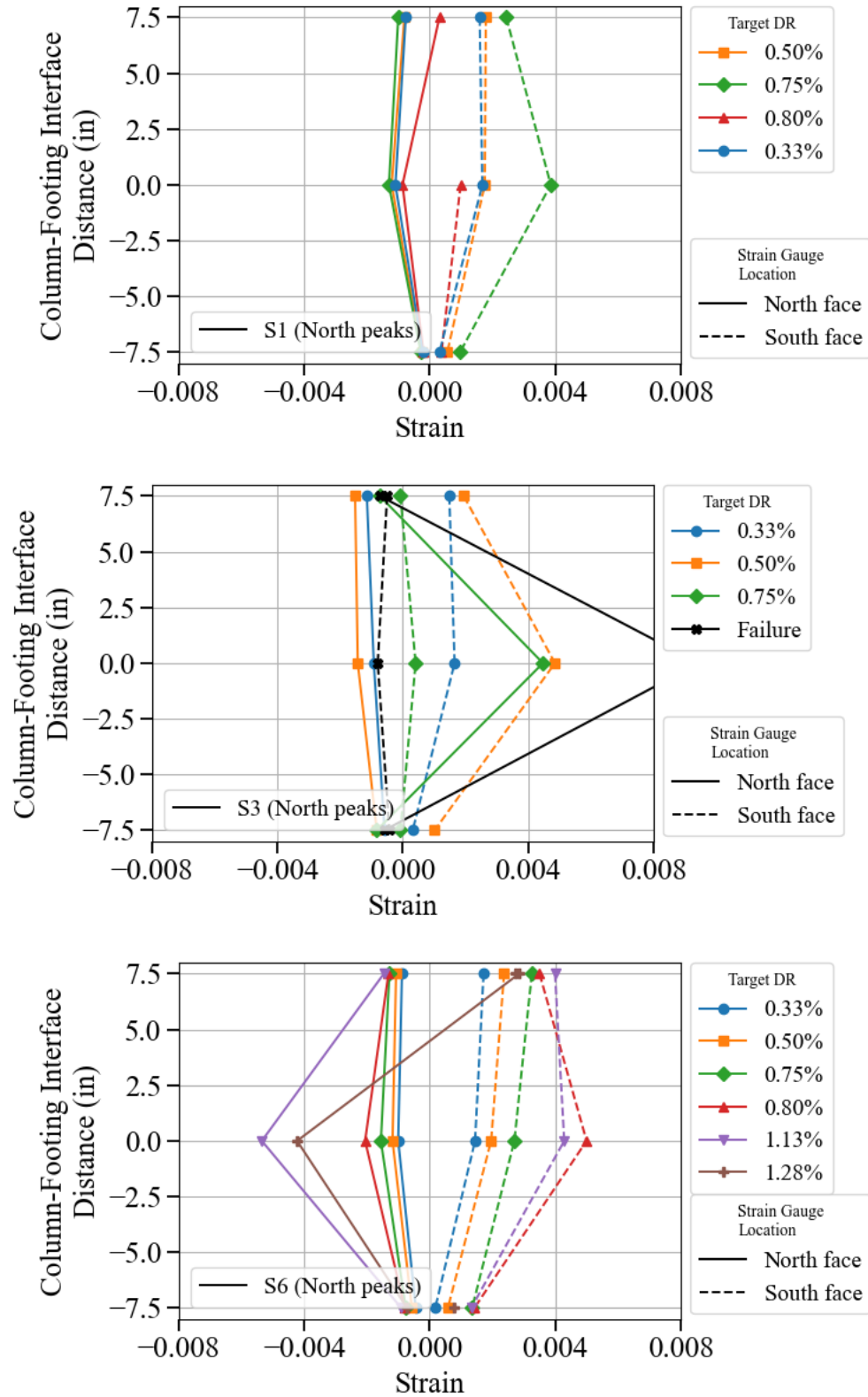


Figure 6.12 Strain profiles per specimen at the North direction loading peaks: S1 (top), S3 (middle), and S6 (bottom)

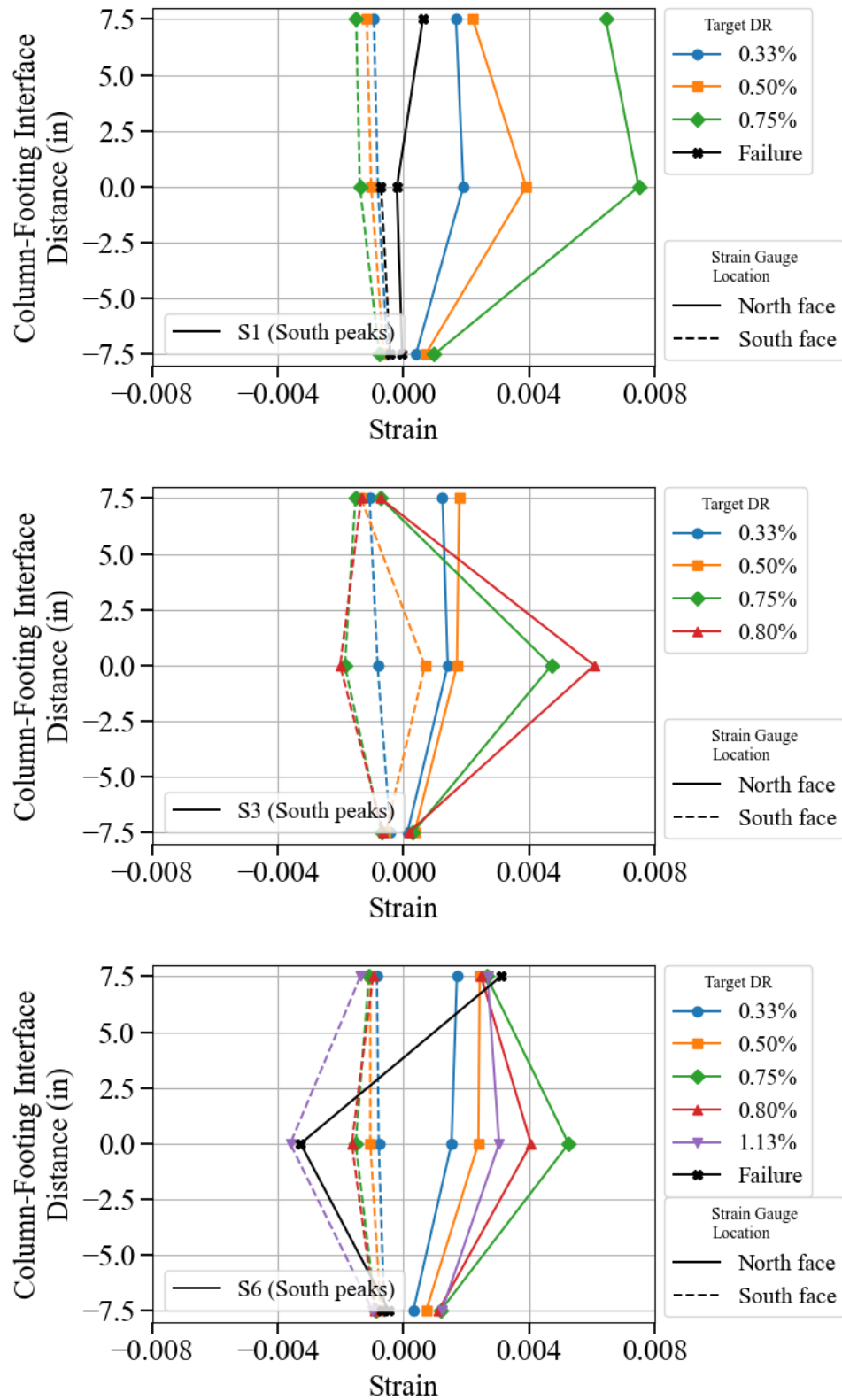


Figure 6.13 Strain profiles per specimen at the South direction loading peaks: S1 (top), S3 (middle), and S6 (bottom)

6.6. DISPLACEMENT DUCTILITY

The experimental data for the “first” yield point per specimen, as shown in Table 5.1, along with the idealized yield force ($F_{0.004}$) from a moment-curvature analysis of the sections, are presented in Table 6.4. These data were processed to compute the idealized yield and failure displacements (Δ_y , Δ_u) following the procedures described in Section 3.3.2.

The results included in Table 6.5 show that Specimens S1 and S3 experienced a displacement ductility demand lower than 2, which, combined with the observed shear damage, qualify these tests as shear-critical failures according to the failure mode classification defined in Section 3.3.3. Specimen S6 exhibited a displacement ductility slightly larger than 2 (2.44). Although this value exceeded a value of 2.0, the failure was still classified as a shear failure because the failure did not occur in the flexural hinge. As discussed in Chapter Chapter 5:, Specimen S6 did not exhibit the usual progression of flexure damage.

Table 6.4 First yield and idealized yield data inputs per specimen

Specimen	F_{yi}		Δ_{yi}		$F_{0.004}$	
	(kN)	(kip)	(mm)	(kN)	(kN)	(kip)
S1	289.2	65.0	6.29	289.2	331.5	74.52
S3	271.5	61.0	6.79	271.5	317.5	71.37
S6	282.6	63.5	5.39	282.6	339.9	76.42

Table 6.5 Idealized yield and failure displacement per specimen

Specimen	Δ_y		Δ_u		μ_Δ
	(mm)	(in.)	(mm)	(in.)	
S1	7.2	0.28	10.9	0.43	1.51
S3	7.9	0.31	9.7	0.38	1.22
S6	6.5	0.26	15.8	0.62	2.44

6.7. CUMULATIVE PLASTIC DISPLACEMENT

Cumulative plastic displacement (CPD) was computed for each specimen according to the procedure defined in Section 3.5 to assess the plastic displacement demands to which each specimen was subjected.

As expected, the results in Table 6.6 show that Specimen S6 exhibits significantly larger Δ_p and CPD values than the other specimens, which is directly related to the greater displacement ductility achieved under the loading program that included six intermediate cycles between target drift ratios.

Specimens S1 and S3 experienced low displacement ductility demands before a brittle shear failure occurred, which, along with the loading protocol with reduced intermediate cycles between drift ratio targets compared to Specimen S6, yielded low Δ_p and CPD values.

Table 6.6 Plastic displacement demand analysis per specimen

Specimen	Δ_p		<i>CPD</i>	
	(mm)	(in.)	(mm)	(in.)
S1	10.9	0.43	17.2	0.68
S3	9.7	0.38	13.6	0.54
S6	15.8	0.62	261.1	10.28

CHAPTER 7:

SHEAR STRENGTH ANALYSIS

This chapter analyzes the shear strength of the three specimens tested as part of this thesis, as well as the previous experimental results discussed earlier in Section 3.2. The analysis focuses on the Kowalsky and Priestley (2000) model and examines whether the model captures the measured shear strength of columns with overlapped hoops. The influence of displacement ductility and cumulative plastic displacements on model conservatism is then evaluated.

7.1. EVALUATION OF SHEAR DEMAND AND CAPACITY

The nominal shear strengths for Specimens S1, S3, and S6 were computed using the Kowalsky and Priestley (2000) model. In the following graphs, Specimens S1 and S3, which were classified as shear failures, are included within the ‘Shear (Gonzalez)’ group, whereas Specimen S6, classified as a shear failure with ductility demands within the flexure-shear region of the model was included in the ‘Flexure-shear (Gonzalez)’ group.

Figure 7.1(a) shows that nearly all the tests with overlapped hoops fall within the model's unconservative region. This was the case for the three tested specimens: the measured peak effective shear force was lower than the nominal shear strength predicted by this model, even though all three exhibited clear shear-related damage. Figure 7.1 (b) shows decreasing conservatism with higher CPD/L values; however, even the low CPD/L tests are mostly within the model's unconservative region.

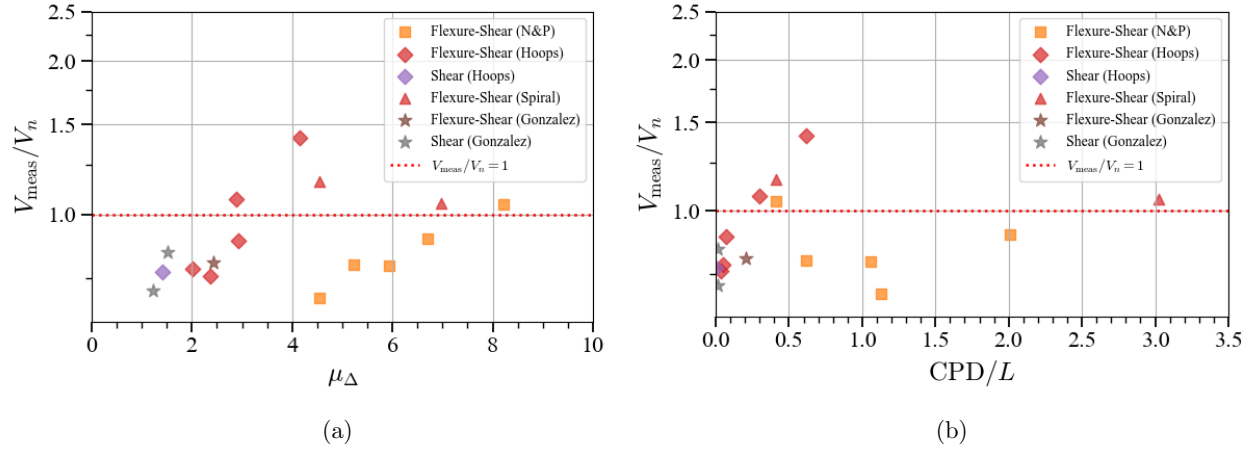


Figure 7.1 Kowalsky and Priestley (2000) shear strength ratio (V_{meas}/V_n) as a function of: (a) displacement ductility (μ_Δ) and (b) CPD/L

7.2. INFLUENCE OF DISPLACEMENT DUCTILITY

Following the analyses presented in Figure 3.4 (a) and (b), Figure 7.2 (a) and (b) add the tested specimens to the computed γ_{req} influence with displacement ductility demand graphs. Figure 7.2 (a) suggests that, while the measured shear strength is above the calculated strength for the two spiral-reinforced column tests, lower values of $\gamma_{Kowalsky}$ would be required for the model to match the measured strengths of specimens with overlapped hoops. Additionally, Figure 7.2 (b) shows that the unconservative trend observed for most tests with overlapped hoops becomes more pronounced as displacement ductility increases.

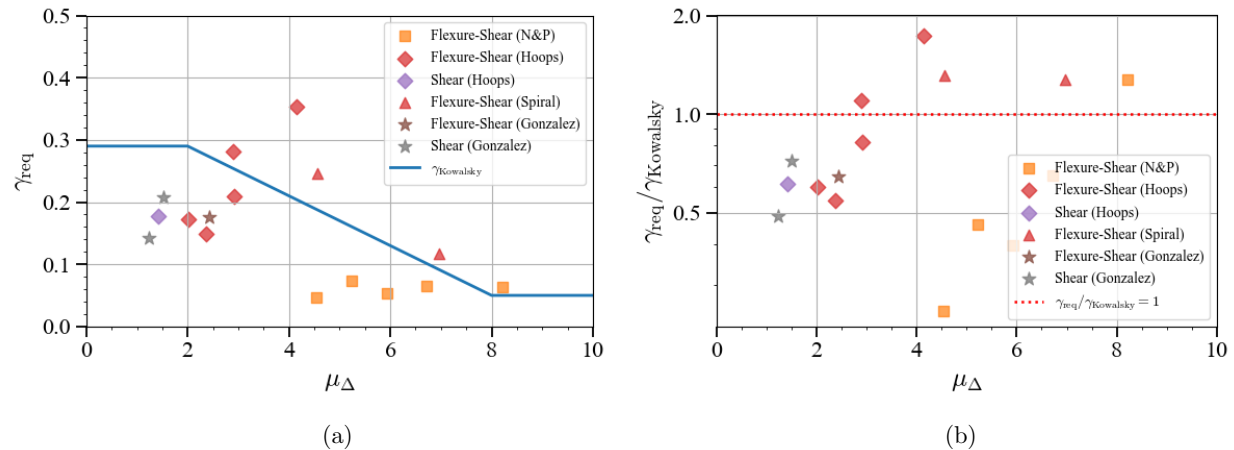


Figure 7.2 (a) Required concrete shear strength reduction parameter, γ_{req} , versus Ductility Demand for the Kowalsky and Priestley (2000) model to match the selected experimental results, and (b) Normalized by γ computed from Kowalsky and Priestley (2000) versus Ductility Demand

7.3. INFLUENCE OF CUMULATIVE PLASTIC DISPLACEMENT

The influence of cumulative plastic displacement (CPD) was assessed in Figure 7.3(a) and (b) by adding the three tested specimens to the CPD/L graphs from Figure 3.5(b) Figure 3.7(b), which evaluate the Kowalsky and Priestley (2000) model.

Figure 7.3(a) shows that, relative to the broader experimental sample, the three specimens tested in this study occupied the lower range of CPD/L values, although Specimen S6 accumulated substantially larger CPD values than Specimens S1 and S3.

Additionally, Figure 7.3(a) suggests a trend toward a decreasing contribution of concrete shear strength with increasing CPD/L values, especially in overlapped-hoops tests. Figure 7.3(b) shows an increasing dispersion of the model's conservatism for higher CPD/L tests.

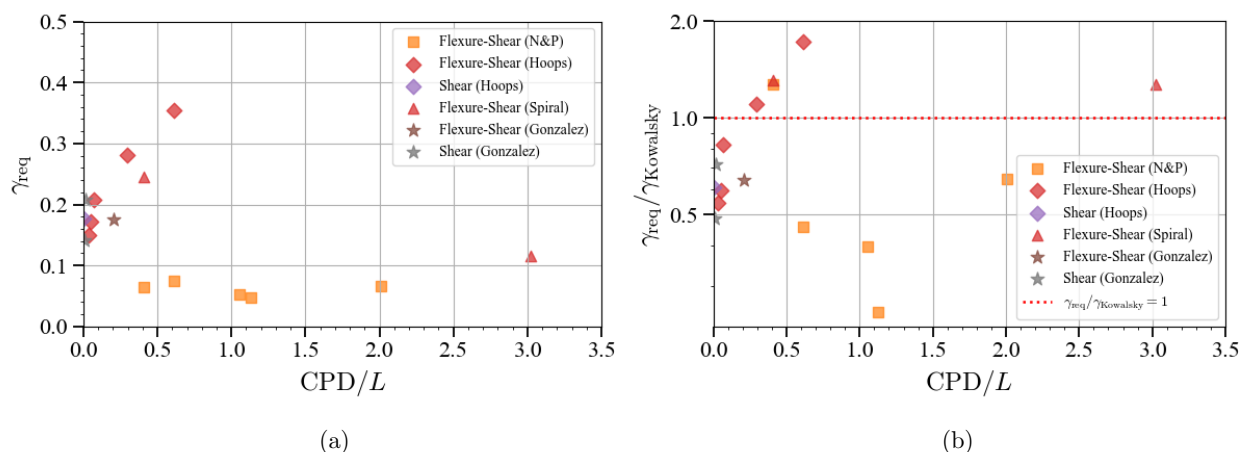


Figure 7.3 (a) Required concrete shear strength reduction parameter, γ_{req} , versus CPD/L for the Kowalsky and Priestley (2000) model to match the selected experimental results, and (b) Normalized by γ computed from Kowalsky and Priestley (2000) versus CPD/L

7.4. DISCUSSION

Camarillo (2003) observed that assuming the shear strength contribution of the steel-truss component (V_s) equal to zero while computing the nominal shear strength of Nelson & Price columns yielded better results when assessing their shear strength using the Kowalsky and Priestley (2000) model. This assumption was motivated by the use of

overlapped hoops in those specimens, rather than hooked or welded ends, or a spiral configuration.

The observed unconservative trend of this model for most tests with overlapped hoops, together with the observation of open hoop lap regions in the present specimens, supports the possibility that the transverse reinforcement did not fully develop the shear-resisting contribution assumed by the model. Therefore, the sensitivity of the model to neglecting the transverse reinforcement contribution to shear strength was assessed in these tests, as shown in Figure 7.4.

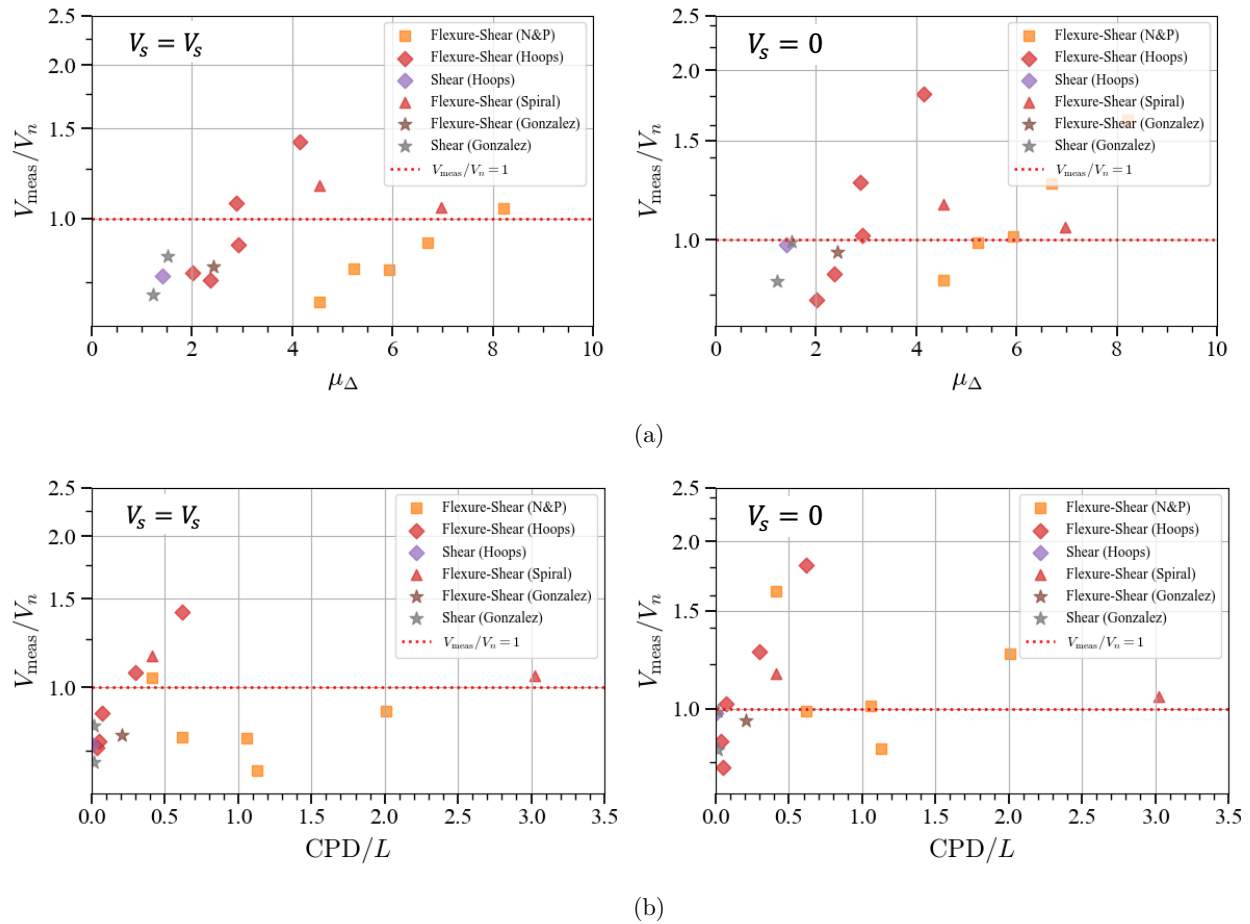


Figure 7.4 Kowalsky and Priestley (2000) shear strength ratio (V_{meas}/V_n), as a function of: (a) displacement ductility (μ_Δ) and (b) CPD/L. Results show the original model with V_s (left) and neglecting V_s for overlapped hoops (right)

Figure 7.4(a) shows improved shear strength predictions for most tests; however, some remain within the unconservative region, especially at lower displacement-ductility demands. Figure 7.4(b) suggests that the implemented consideration yields more

conservative results in general, while still showing some unconservative results for low CPD/L values.

These results suggest that neglecting V_s overlapped-hoop specimens improves the model predictions, but it is not sufficient to avoid unconservative results across all the samples. Additional adjustment factors for the contribution of concrete shear strength, considering displacement ductility and/or cumulative plastic displacement demands, may be needed.

Considering that some of the tests analyzed in Figure 7.4(a) still lie within the unconservative region of the model, the required γ reduction factor for the concrete shear strength contribution was computed by neglecting V_s in overlapped hoops tests, as shown in Figure 7.5(a).

Results in Figure 7.5(b) suggested that the γ factors from the Kowalsky model were still unconservative for overlapped-hoop tests at lower displacement ductility demands, even while neglecting the transverse reinforcement shear-strength contribution. According to Figure 7.5(a), lowering the value of the γ parameter for small displacement ductility demands would improve conservatism in this region, and increasing the value of the γ parameter for larger displacement ductility demands could yield less over-conservative results when neglecting the shear-strength contribution from the hoops.

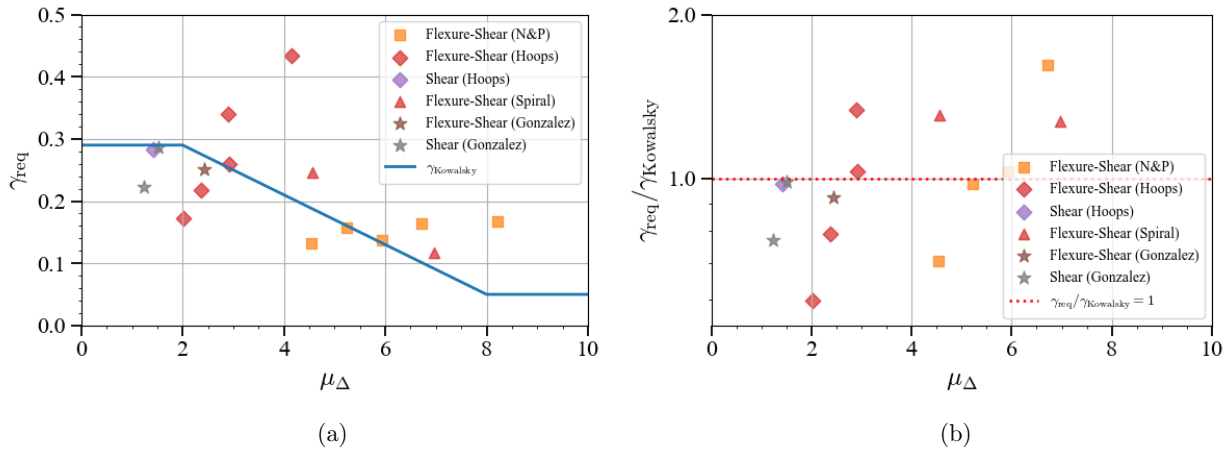


Figure 7.5 (a) Required concrete shear strength reduction parameter, γ_{req} , versus Ductility demand for the Kowalsky and Priestley (2000) model to match the selected experimental results, neglecting transverse reinforcement shear strength contribution for overlapped hoops, and (b) Normalized by $\gamma_{computed}$ from Kowalsky and Priestley (2000) versus displacement ductility demand

When assessing the influence of cycling through the variable CPD/L in the data sample, considering the negligible transverse reinforcement shear-strength contribution in tests with overlapped hoops, a decreasing trend in the required γ parameter is observed as CPD/L values increase (see Figure 7.6 (a)). Figure 7.6 (b) suggests that the current values of the γ parameter in the Kowalsky and Priestley (2000) model are unconservative for some tests in the lower- CPD/L -demand region.

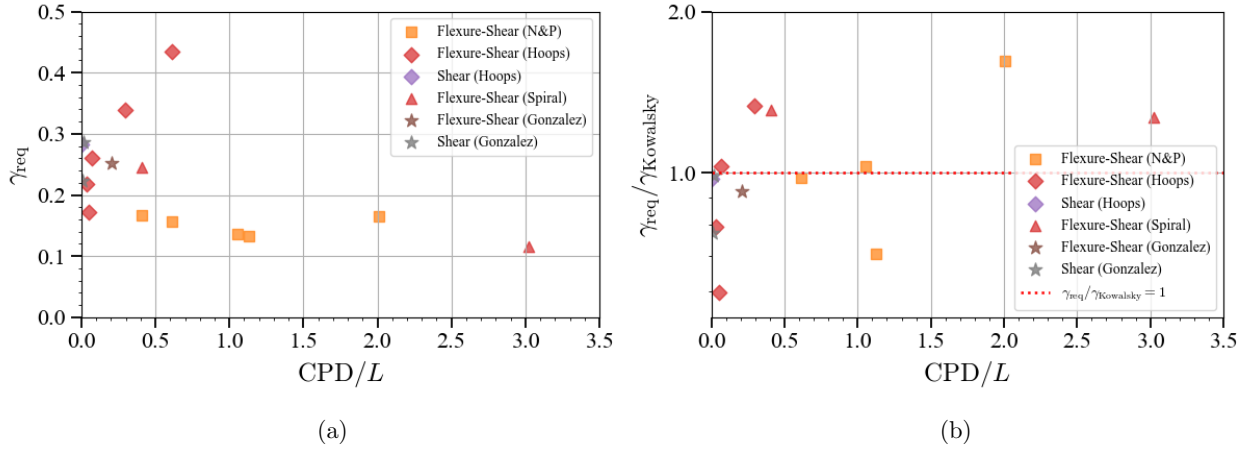


Figure 7.6 (a) Required concrete shear strength reduction parameter, γ_{req} , versus CPD/L for the Kowalsky and Priestley (2000) model to match the selected experimental results, neglecting transverse reinforcement shear strength contribution for overlapped hoops, and (b) Normalized by $\gamma_{computed}$ from Kowalsky and Priestley (2000) versus CPD/L

CHAPTER 8:

CONCLUSIONS AND RECOMMENDATIONS

8.1. SUMMARY

This research assessed the effect of cycling on the shear strength of older reinforced concrete bridge columns in Washington State, particularly those designed prior to 1975. These columns typically exhibit low transverse reinforcement ratios ($<0.5\%$) and circular hoops with overlapping ends, making them susceptible to shear and flexure-shear failures. This vulnerability is particularly relevant to the Pacific Northwest, where long-duration ground motions expected from a future large earthquake from the Cascadia Subduction Zone may impose a large number of cycles with inelastic displacement demand on these columns; thus, evaluating the effects of cumulative cyclic demands on the shear strength of older bridge columns is critical.

Chapter 2 reviewed existing shear strength models for reinforced concrete columns and showed that none explicitly accounts for the amount of cycling as a variable. Chapter 3 then defined representative properties of older bridge columns based on a bridge inventory developed by Liu et al. (2022). Using these characteristics, a sample of 24 existing experimental tests of circular reinforced concrete columns was compiled from the UW/PEER and ACI 369 databases. Of these specimens, only 13 exhibited shear or flexure-shear failures and were used to evaluate the Kowalsky and Priestley (2000) model.

Given the limited number of available experiments representative of older bridge column configurations with shear and flexure-shear failures, Chapter 4 defined a prototype column representative of 1960s bridge construction in Washington State and scaled it to a 1:3 ratio. This prototype was used to design three nominally identical reinforced concrete columns, designated S1, S3, and S6, which were subjected to loading protocols with identical drift-ratio targets but different numbers of cycles between targets.

Chapters 5 and 6 presented the observed and measured responses for each specimen. Shear-related damage was observed in all three columns, classifying them as shear failure modes. Specimen S6 sustained the largest effective lateral force, drift-ratio demand, and cumulative plastic displacement. In all specimens, the overlapping hoops did not fracture during testing; instead, they slid and opened after diagonal cracking developed.

Chapter 7 evaluated the nominal shear strength of the experimental specimens using the Kowalsky and Priestley (2000) model. The results showed that most tests with overlapped hoops fell within the unconservative region of the model, whereas specimens with spiral reinforcement and higher transverse reinforcement ratios were generally in the conservative region. A sensitivity analysis that neglected the shear-strength contribution of overlapped hoops improved strength predictions for most tests, although some still fell within the unconservative region.

Finally, the effects of displacement ductility demand and cumulative plastic displacement were assessed using both the compiled database sample and the newly tested specimens. The analyses showed that the required shear strength degradation factor for the concrete contribution tended to decrease as displacement ductility and cumulative plastic displacement demands increased. However, the number of tests with significant cumulative plastic displacement remains limited, preventing definitive conclusions regarding the influence of cycling on shear strength degradation.

8.2. CONCLUSIONS

The findings of this study support the following conclusions:

- a) Only a limited number of existing experimental tests with available force-displacement histories and representative properties of WSDOT pre-1975 bridge columns were found. Of the 24 tests found, only 13 exhibited shear and flexure-shear failures. This highlights the need for additional tests on older bridge column configurations to assess shear strength models, given that these designs are particularly susceptible to shear and flexure-shear failures due to their low transverse reinforcement ratios and the presence of overlapped hoops.
- b) The three nominally identical specimens tested in this study exhibited a shear-dominated damage progression. Increasing the number of cycles between drift-ratio targets did not produce a clear reduction in the effective lateral force at failure at these relatively low cumulative plastic displacement demands. However, Specimen S6 had a higher number of cycles and sustained greater deformation before failing in shear.

- c) The overlapped hoops did not fracture after failure in any of the three tests. Instead, they slid and opened as diagonal cracks formed and widened, suggesting their effectiveness may be less than assumed in the steel-truss mechanism considered by shear-strength models. Consistent with this observation, the Kowalsky and Priestley (2000) model overestimated the shear strength of the three specimens tested in this study and most tests involving overlapped hoops in the studied sample. Neglecting the transverse reinforcement's shear-strengthening contribution in tests with overlapping hoops improved shear strength predictions; however, some tests remained in the model's unconservative region.
- d) The concrete shear-strength degradation factor required for the Kowalsky and Priestley (2000) model to match the measured shear strength tended to decrease with increasing displacement ductility and normalized cumulative plastic displacement demand, CPD/L. The required values were generally lower for tests with overlapped hoops, suggesting that the concrete shear-strength contribution may degrade more rapidly than assumed by the model for this reinforcement detail, particularly at low and moderate displacement ductility demands. This behavior may be related to reduced confinement effectiveness after hoop lap regions open, as well as to the accumulation of cyclic damage. Additional tests on pre-1975 bridge column configurations are needed to isolate these effects that lead to definitive conclusions regarding both the influence of overlapped-hoop effectiveness and the additional shear-strength reduction associated with cumulative plastic displacement demand.
- e) Overall, the results suggest that the shear strength evaluation of older reinforced concrete bridge columns with overlapped hoops should account for the reduced effectiveness of transverse reinforcement and the potential influence of cumulative plastic displacement on shear-strength degradation.

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APPENDIX A:
INTERMEDIATE RESULTS OF SHEAR STRENGTH MODEL
EVALUATION

A.1 KEY PROPERTIES OF EXPERIMENTAL TESTS

Table A.1 Experimental Test Sample List

ID	Specimen Name	Cross-section	Observed Damage	Database
PR-M1E1	Petrovsky and Ristic 1984, M1E1 (Petrovski & Ristic, 1984)	Circular	Flexure	UW/PEER
PR-M1E2	Petrovsky and Ristic 1984, M1E2 (Petrovski & Ristic, 1984)	Circular	Flexure	UW/PEER
PR-M2E1	Petrovsky and Ristic 1984, M2E1 (Petrovski & Ristic, 1984)	Circular	Shear	UW/PEER
PR-M2E2	Petrovsky and Ristic 1984, M2E2 (Petrovski & Ristic, 1984)	Circular	Shear	UW/PEER
NIST-FS	NIST, Full Scale Flexure (Cheok & Stone, 1986)	Circular	Flexure	UW/PEER
BP-NR1	Benzoni & Priestley 1994, NR1 (Priestley & Benzoni, 1996)	Circular	Shear	UW/PEER
BP-NR2	Benzoni & Priestley 1994, NR2 (Priestley & Benzoni, 1996)	Circular	Shear	UW/PEER
NP-S3	Nelson and Price 2000, Col1 (Ranf et al., 2006)	Circular	Shear	UW/PEER
NP-C4	Nelson and Price 2000, Col2 (Ranf et al., 2006)	Circular	Flexure	UW/PEER
NP-C2	Nelson and Price 2000, Col3 (Ranf et al., 2006)	Circular	Shear	UW/PEER
NP-C3R	Nelson and Price 2000, Col4 (Ranf et al., 2006)	Circular	Shear	UW/PEER

ID	Specimen Name	Cross-section	Observed Damage	Database
NP-S1	Nelson and Price 2000, Col5 (Ranf et al., 2006)	Circular	Shear	UW/PEER
NP-S15	Nelson and Price 2000, Col6 (Ranf et al., 2006)	Circular	Shear	UW/PEER
H-415S	Henry 1998, 415s (Henry & Mahin, 1999)	Circular	Flexure	UW/PEER
GD-14	Goodnight 2015, Test 14 (Goodnight et al., 2015)	Circular	Flexure	UW/PEER
YN-A1	Yoneda et al. 2001, A1 (Yoneda et al., 2001)	Circular	Flexure	UW/PEER
YN-B1	Yoneda et al. 2001, B1 (Yoneda et al., 2001)	Circular	Flexure	UW/PEER
MK-2-3	Makido 2007, 2-3 (Makido, 2007)	Circular	Flexure-Shear	UW/PEER (Digitized)
EL-CS-A1	Elsanadedy 2002, UnitCS-A1 (Elsanadedy, 2002)	Circular	Shear	ACI 369
EL-CS-A2	Elsanadedy 2002, UnitCS-A2 (Elsanadedy, 2002)	Circular	Shear	UW/PEER (Digitized)
YL-BR-C1	Yalcin 1997, BR-C1 (Yalcin, 1997)	Circular	Shear	UW/PEER (Digitized)
JR-S1	Jaradat 1996, Specimen S1 (Jaradat, 1996)	Circular	Shear	ACI 369
JR-S3	Jaradat 1996, Specimen S3 (Jaradat, 1996)	Circular	Shear	ACI 369
YR-CR-C	Yarandi (2007), Specimen CR-C (Yarandi, 2007)	Circular	Shear	ACI 369

Table A.2 Key Properties of the Study Sample

ID	D (mm)	c_c (mm)	L (mm)	$\frac{L}{D}$	d (mm)	f'_c (MPa)	d_{bl} (mm)	n_t	f_y (MPa)	ρ_t (%)	d_{bt} (mm)	f_{yt} (MPa)	s (mm)	ρ_t (%)	P (kN)	$P\Delta_{type}$	$\frac{P}{A_g f'_c}$ (%)
PR-M1E1	307	33	1910	6.2	262	38.8	12.0	12	240.0	1.8	6.0	240.0	75.0	0.6	145.0	U ¹	0.05
PR-M1E2	307	33	1910	6.2	262	36.2	12.0	12	240.0	1.8	6.0	240.0	75.0	0.6	254.0	U ¹	0.09
PR-M2E1	307	33	900	2.9	262	35.9	12.0	12	240.0	1.8	6.0	240.0	75.0	0.6	145.0	U ¹	0.05
PR-M2E2	307	33	895	2.9	262	34.4	12.0	12	240.0	1.8	6.0	240.0	75.0	0.6	254.0	U ¹	0.10
NIST-FS	1520	50.8	9140	6.0	1431.9	35.8	43	25	475	2.0	15.9	493	88.9	0.64	4450	2	0.07
BP-NR1	610	13	915	1.5	585	30.0	12.7	12	462.0	0.5	6.4	361.0	76.2	0.3	503.0	1	0.06
BP-NR2	610	13	915	1.5	585	30.0	12.7	24	462.0	1.0	6.4	361.0	127.0	0.2	503.0	1	0.06
NP-S3	508	19	1524	3.0	476	56.2	16.0	10	470.0	1.0	4.5	455.0	101.6	0.1	1223.0	3	0.11
NP-C4	508	19	1524	3.0	476	56.3	16.0	10	470.0	1.0	4.5	455.0	101.6	0.1	1139.0	3	0.10
NP-C2	508	19	1524	3.0	476	57.0	16.0	10	470.0	1.0	4.5	455.0	101.6	0.1	1139.0	3	0.10
NP-C3R	508	19	1524	3.0	476	52.7	16.0	10	470.0	1.0	4.5	455.0	101.6	0.1	1139.0	3	0.11
NP-S1	508	19	1524	3.0	476	36.3	16.0	10	426.0	1.0	4.5	455.0	101.6	0.1	747.0	3	0.10
NP-S15	508	19	1524	3.0	476	40.5	16.0	10	426.0	1.0	4.5	455.0	101.6	0.1	818.0	3	0.10
H-415S	610	19	2438	4.0	576	37.2	15.9	22	462.0	1.5	6.4	606.8	63.5	0.4	654.0	3	0.06

¹ Unknown; type 2 was assumed.

ID	D (mm)	c_c (mm)	L (mm)	$\frac{L}{D}$	d (mm)	f'_c (MPa)	d_{bl} (mm)	n_l	f_y (MPa)	ρ_l (%)	d_{bt} (mm)	f_{yt} (MPa)	s (mm)	ρ_t (%)	P (kN)	$P\Delta_{type}$	$\frac{P}{A_g f'_c}$ (%)
GD-14	610	13	2438	4.0	578	45.8	19.1	16	469.5	1.6	9.5	445.4	101.6	0.5	756.0	3	0.06
YN-A1	400	32	1350	3.4	354	30.0	16.0	12	374.0	1.9	6.0	363.0	150.0	0.2	185.0	1	0.05
YN-B1	400	32	1350	3.4	354	30.0	16.0	12	374.0	1.9	6.0	363.0	300.0	0.1	185.0	1	0.05
MK-2-3	610	38	915	1.5	553	32.4	19.1	16	430.9	1.6	9.5	468.8	88.9	0.6	800.0	2	0.08
EL-CS-A1	610	19	1219	2.0	575	36.8	19.1	20	299.3	2.0	6.4	210.3	127.0	0.2	645.0	2	0.06
EL-CS-A2	610	19	1219	2.0	575	35.4	19.1	20	481.0	2.0	6.4	444.3	127.0	0.2	645.0	2	0.06
YL-BR-C1	610	55	1485	2.4	532.8	45	25.4	12	445	2.1	9.5	425.0	300.0	0.2	1800.0	2	0.1
JR-S1	254	14	622	2.5	230	28.9	12.7	8	359.2	2.0	3.8	210.0	98.0	0.2	84.5	2	0.06
JR-S3	254	14	622	2.5	231	26.2	9.5	8	370.9	1.1	3.8	210.0	98.0	0.2	75.6	2	0.06
YR-CR-C	600	45	1500	2.5	539	35.0	19.5	12	465.0	1.3	6.4	491.0	300.0	0.1	1436.0	2	0.15

Table A.3 Study Sample Experimental Parameters

ID	Failure mode	Δ_{yi} (mm)	Δ_y (mm)	Δ_u (mm)	Δ_p (mm)	μ_Δ	CPD (mm)	$NCPD$	F_{yi} (kN)	$F_{0.004}$ (kN)	F_{max} (kN)	$M_{0.004}$ (kN-mm)	V_{meas} (kN)
PR-M1E1	Flexure	6.74	9.15	69.25	60.10	7.57	2553.01	10.62	20.68	28.10	32.69	53672.99	33.60
PR-M1E2	Flexure	7.09	9.22	49.44	40.22	5.36	1358.34	8.44	25.17	32.72	35.64	62501.56	36.98
PR-M2E1	No Fail	2.31	3.13	-33.03	29.89	10.55	1083.58	9.06	43.51	59.08	76.43	53173.37	85.94
PR-M2E2	No Fail	2.69	3.48	-36.03	32.55	10.36	784.05	6.02	53.43	69.08	82.19	61825.58	92.95
NIST-FS	Flexure	82.54	109.91	540.99	431.08	4.92	22555.55	13.08	986.39	1313.39	1454.20	12004358.40	1293.40
BP-NR1	No Fail	1.80	2.29	-32.56	30.27	14.21	1066.71	8.81	256.23	326.86	399.52	298915.81	403.28
BP-NR2	Flexure-Shear	2.71	3.57	16.24	12.67	4.55	373.57	7.37	375.24	495.20	587.36	452862.30	578.93
NP-S3	Flexure-Shear	8.35	10.07	52.65	42.58	5.23	936.96	5.50	223.43	269.42	318.19	410595.17	286.21
NP-C4	Flexure	7.72	9.41	56.59	47.18	6.01	2055.19	10.89	215.33	262.49	298.79	400032.79	280.01
NP-C2	Flexure-Shear	7.92	9.63	57.11	47.48	5.93	1609.01	8.47	215.62	262.19	277.50	399570.55	260.80
NP-C3R	Flexure-Shear	8.24	9.95	45.23	35.28	4.54	1720.96	12.20	213.91	258.46	272.19	393887.63	251.45
NP-S1	Flexure-Shear	5.62	7.02	57.68	50.67	8.22	622.12	3.07	159.24	198.88	218.85	303086.29	207.30
NP-S15	Flexure-Shear	4.93	6.13	-41.10	34.97	6.71	3063.87	21.90	168.43	209.46	218.53	319217.81	221.11
H-415S	Flexure	17.84	23.67	128.93	105.26	5.45	3090.32	7.34	190.02	252.21	293.60	614977.53	259.25
GD-14	Flexure	25.64	34.17	161.24	127.08	4.72	3314.17	6.52	209.70	279.47	305.58	681461.97	281.67
YN-A1	Flexure	6.60	8.83	-55.63	46.80	6.30	1843.77	9.85	83.68	111.87	114.35	151026.57	113.95

ID	Failure mode	Δ_{yi} (mm)	Δ_y (mm)	Δ_u (mm)	Δ_p (mm)	μ_Δ	CPD (mm)	$NCPD$	F_{yi} (kN)	$F_{0.004}$ (kN)	F_{max} (kN)	$M_{0.004}$ (kN-mm)	V_{meas} (kN)
YN-B1	Flexure	6.48	8.65	46.79	38.15	5.41	1493.41	9.79	83.89	111.89	126.38	151054.60	123.91
MK-2-3	Flexure-Shear	7.20	9.27	-64.54	55.27	6.96	2765.88	12.51	503.06	647.81	835.53	592421.93	798.12
EL-CS-A1	Flexure-Shear	3.02	4.11	12.01	7.90	2.92	82.80	2.62	331.96	452.02	458.83	551101.58	454.16
EL-CS-A2	Shear	7.18	9.52	-13.48	3.97	1.42	3.97	0.25	466.18	618.34	473.15	753882.00	469.21
YL-BR-C1	Flexure-Shear	9.01	10.88	25.82	14.94	2.37	48.66	0.81	491.40	593.76	589.14	881732.46	562.48
JR-S1	Flexure-Shear	6.56	8.78	25.35	16.58	2.89	183.60	2.77	48.08	64.29	82.41	40010.13	79.36
JR-S3	Flexure-Shear	4.80	6.28	26.04	19.76	4.15	382.59	4.84	32.89	43.02	80.21	26773.89	77.13
YR-CR-C	Flexure-Shear	5.41	6.49	13.09	6.60	2.02	78.76	2.98	344.28	412.95	435.51	619428.07	423.58

A.2. KOWALSKY AND PRIESTLEY (2000) EVALUATION

Table A.4 Kowalsky and Priestley (2000) Evaluation Parameters

ID	α	β	γ	$V_{cKowalsky}$ (kN)	$V_{sKowalsky}$ (kN)	$V_{pKowalsky}$ (kN)	$V_{nKowalsky}$ (kN)	V_{meas} (kN)	$\frac{V_{meas}}{V_{nKowalsky}}$	γ_{req}
PR-M1E1	46.00	1.00	0.87	0.07	21.53	50.97	9.11	81.61	33.60	0.41
PR-M1E2	47.00	1.00	0.87	0.16	48.00	47.72	15.08	110.81	36.98	0.33
NIST-FS	53.00	1.00	0.90	0.17	1353.45	3188.78	271.47	4813.70	1293.40	0.27
BP-NR2	105.00	1.50	0.71	0.19	255.67	113.46	130.90	500.02	578.93	1.16
NP-S3	135.00	1.00	0.70	0.16	136.56	70.73	152.73	360.02	286.21	0.80
NP-C4	136.00	1.00	0.70	0.13	110.07	71.44	143.59	325.09	280.01	0.86
NP-C2	137.00	1.00	0.70	0.13	113.56	71.62	143.94	329.13	260.80	0.79
NP-C3R	138.00	1.00	0.70	0.19	154.71	70.70	142.18	367.60	251.45	0.68
NP-S1	139.00	1.00	0.70	0.05	34.11	70.62	93.15	197.89	207.30	1.05
NP-S15	140.00	1.00	0.70	0.10	73.23	71.08	102.63	246.93	221.11	0.90
H-415S	142.00	1.00	0.80	0.15	173.05	367.90	62.48	603.44	259.25	0.43
GD-14	188.00	1.00	0.81	0.18	232.64	390.51	73.20	696.35	281.67	0.40
YN-A1	205.00	1.00	0.88	0.12	57.43	49.29	20.33	127.05	113.95	0.90
YN-B1	208.00	1.00	0.88	0.15	74.74	24.64	20.33	119.72	123.91	1.04
MK-2-3	245.00	1.50	0.81	0.09	148.32	414.72	194.74	757.79	798.12	1.05

ID	α	β	γ	$V_{c_{Kowalsky}}$ (kN)	$V_{s_{Kowalsky}}$ (kN)	$V_{p_{Kowalsky}}$ (kN)	$V_{n_{Kowalsky}}$ (kN)	V_{meas} (kN)	$\frac{V_{meas}}{V_{n_{Kowalsky}}}$	γ_{req}
EL-CS-A1	250.00	1.00	0.89	0.25	320.07	65.66	124.90	510.63	454.16	0.89
EL-CS-A2	251.00	1.00	0.89	0.29	359.48	131.42	118.58	609.48	469.21	0.77
YL-BR-C1	252.00	1.00	0.92	0.28	395.28	97.69	249.68	742.65	562.48	0.76
JR-S1	253.00	1.00	0.90	0.25	49.91	11.50	12.77	74.17	79.36	1.07
JR-S3	255.00	1.00	0.72	0.20	30.66	12.06	11.94	54.66	77.13	1.41
YR-CR-C	259.00	1.00	0.75	0.29	291.78	52.49	196.92	541.19	423.58	0.78

APPENDIX B: EXPERIMENTAL PROGRAM

B.1. TEST COLUMNS

This section includes detailed plans of the column and footing geometry and reinforcement.

B.1.1. COLUMN GEOMETRY AND REINFORCEMENT

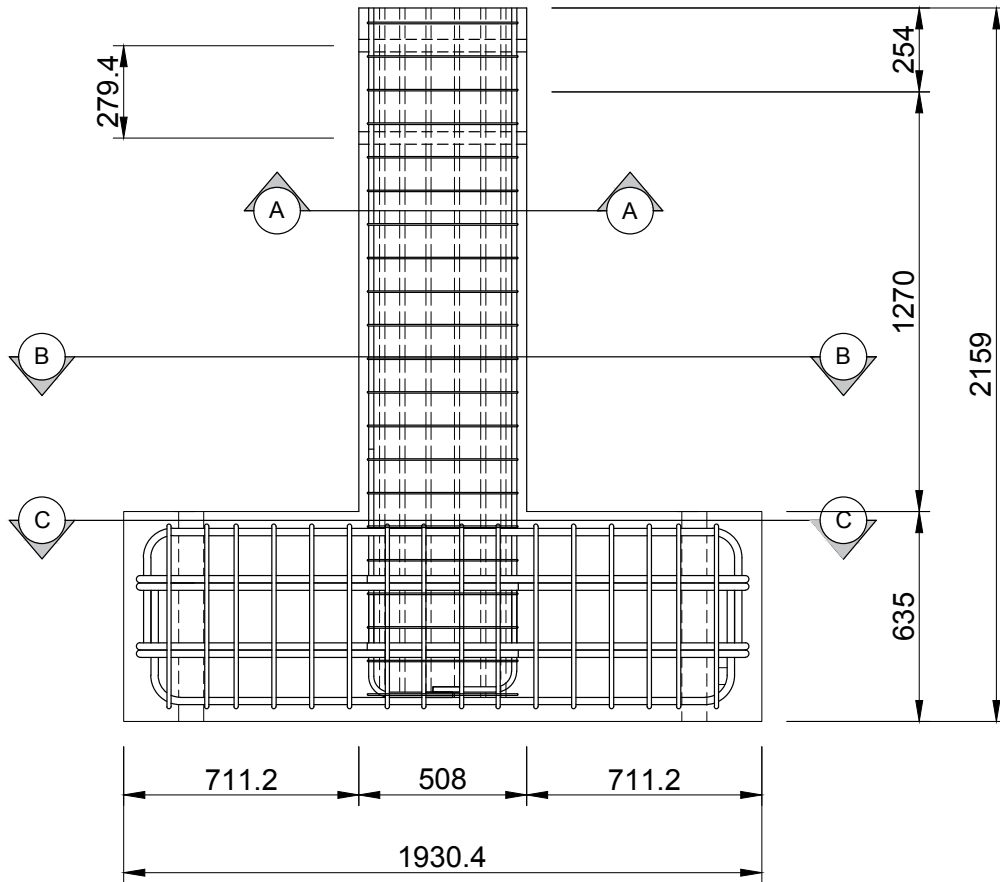


Figure B.1 Elevation view of test column reinforcement

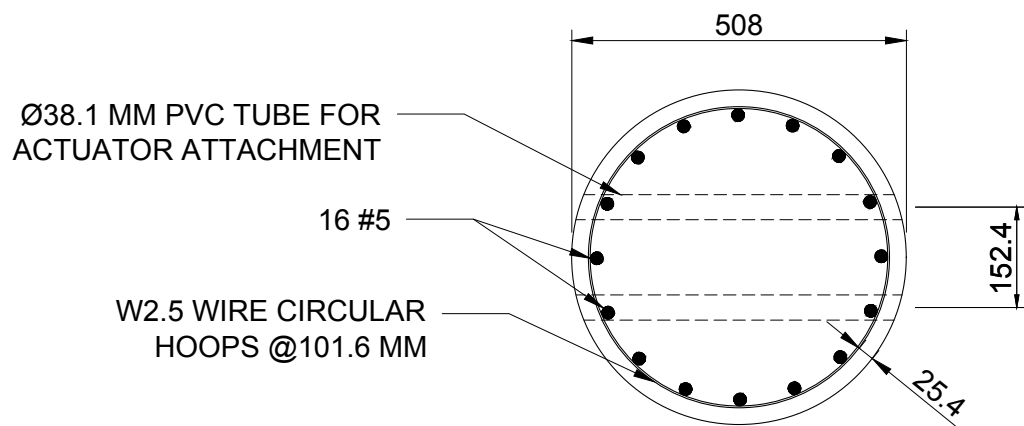
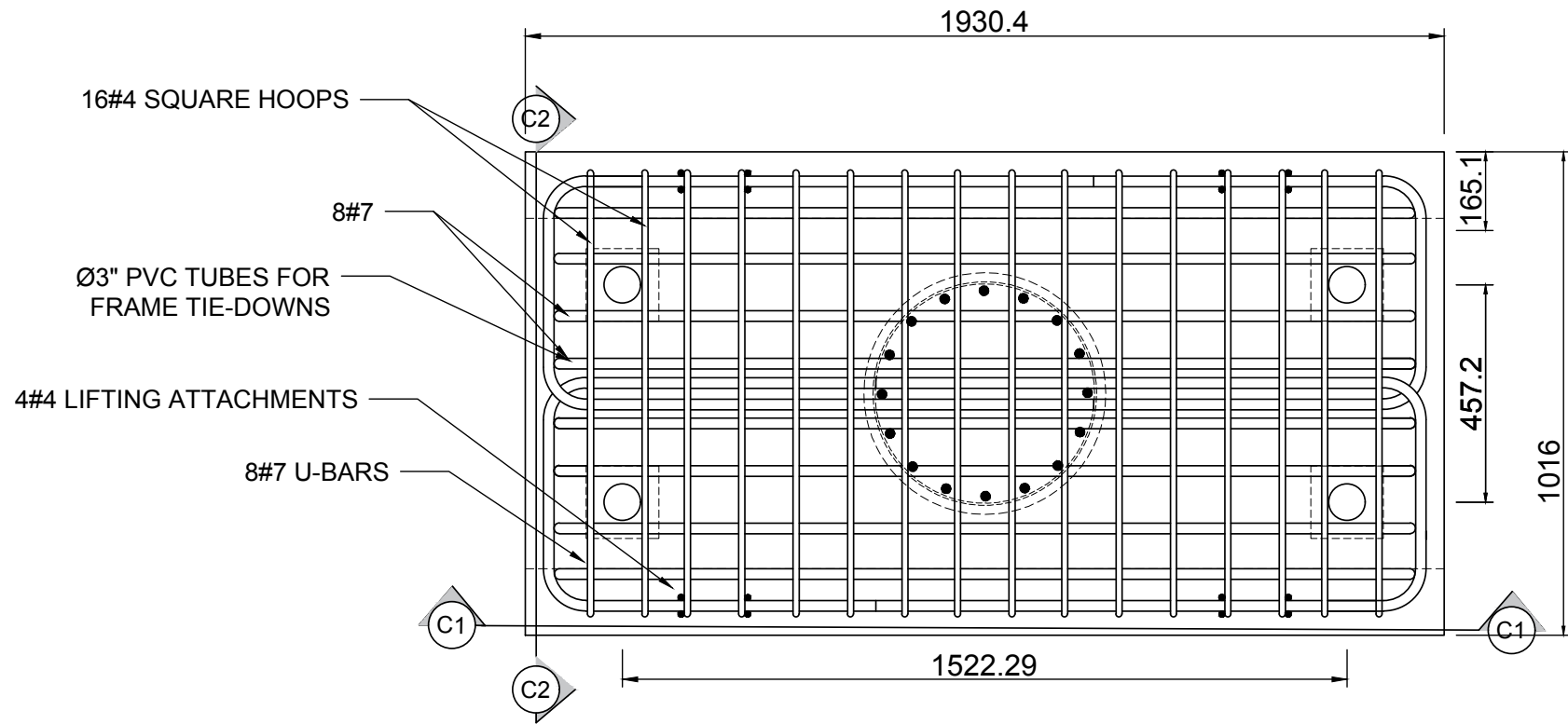


Figure B.2 Section A-A: Plan view of column reinforcement



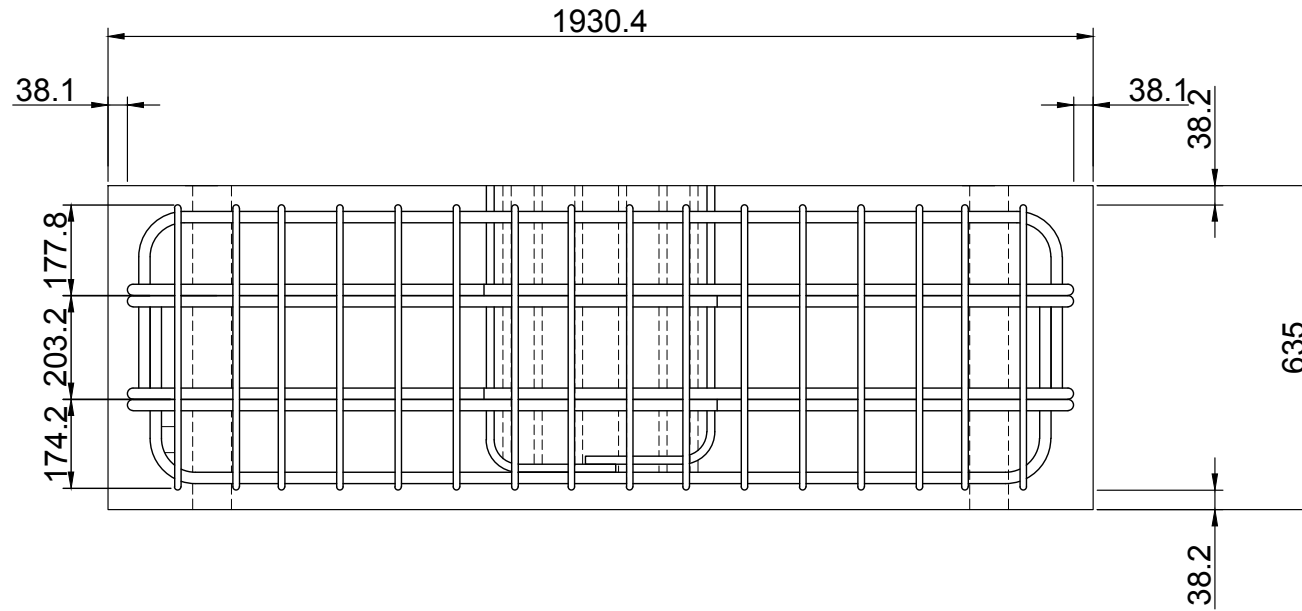


Figure B.4 Section C1-C1: Elevation view of footing reinforcement

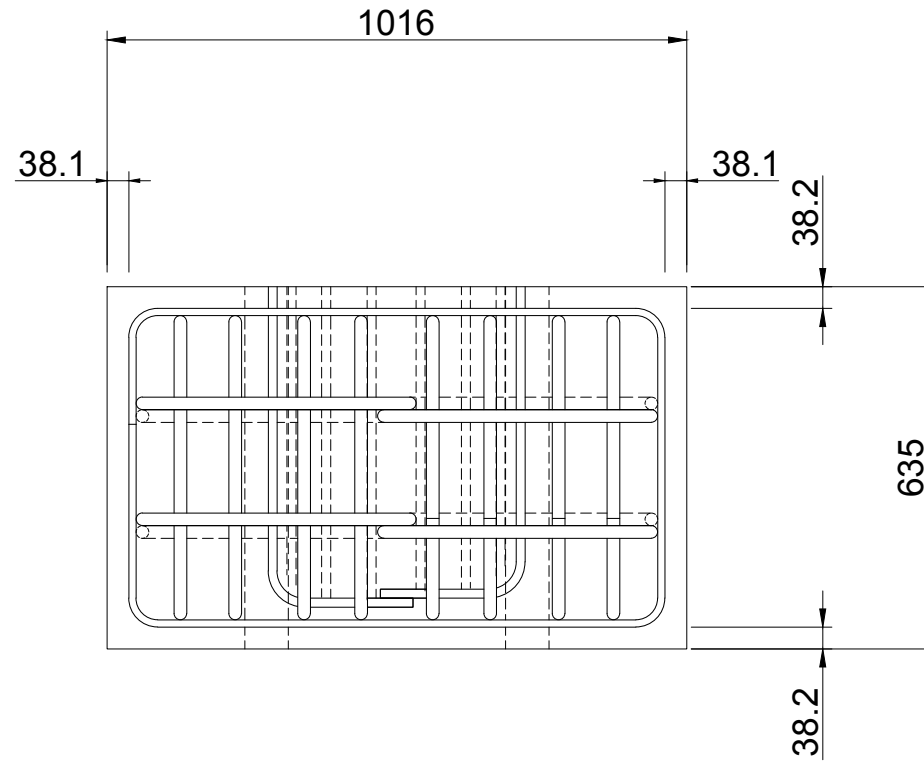


Figure B.5 Section C2-C2: Elevation view of footing reinforcement

B.1.3. DISPLACEMENT HISTORIES

The three specimens had the same nominal design, and the variable among them was the displacement history, as introduced in Table 4.2. A preliminary estimate of the yield displacement was defined as 5.08 mm (0.2 in.) and used to define two initial cycles at 0.25 and 0.5 times the yield displacement. The following cycles considered target drift ratios defined with a 1.5-fold increment starting from 0.33% drift ratio.

Specimens S1 and S6 were subjected to half-cycle increments between drift ratio targets of $1.5^{1/2}$ and $1.5^{1/12}$, respectively. Specimen S3 was subjected to symmetric incremental cycles between each target drift ratio with a $1.5^{1/3}$ -fold increment.

Target displacement histories discussed in Section 4.3 showed minor deviations from the target drift amplitudes due to actuator control adjustments and equipment calibration during testing. All response histories presented in this study correspond to the measured displacements during the tests.

Mechanical issues with the actuator's load cell during specimen S3 tests required the column to undergo 6 cycles at drift ratios below 0.2% as part of the load cell calibration process. These cycles occurred between cycles 1 and 2, and no lateral load recordings were available before cycle 2. Loads and displacement histories for specimen S3 analyses consider cycles from cycle 2. Additionally, due to electrical issues at the cycle 5 north peak (5N), the test had to stop before completing the cycle in the south direction. This cycle was repeated and nominated 5-1.

B.2. MATERIAL PROPERTIES

B.2.1. CONCRETE

The three specimens were cast in two pours inside the Large-Scale Structural Engineering Testing Laboratory at the University of Washington. The three footings were cast on the first pour from the same concrete batch on January 23rd, 2026. The concrete mix used for casting the three footings was provided by Stoneway Concrete, and the mix ID was 560374. Details about this mix are included in Table B.1.

Table B.1 Concrete mix used for footings

Component	Weight (lb/yd³)	Volume (fl oz/yd³)
Cement	550	
GGBFS (Slag)	110	
Coarse Aggregate	1786	
Fine Aggregate	1417	
Water	260	
HRWR		16.5
WR		29.7

Weeks after the footings were cast, the second pour happened on April 18, 2026. The three columns were cast from the same concrete batch during this pour. The concrete mix used for casting the three columns was provided by the same supplier, and the mix ID was 330174. Details about this mix are included in Table B.2.

Table B.2 Concrete mix used for columns

Component	Weight (lb/yd³)	Volume (fl oz/yd³)
Cement	282	
GGBFS (Slag)	282	
Coarse Aggregate	1787	
Fine Aggregate	1500	
Water	255	
HRWR		20
WR		25.38