Seismic Performance Design Criteria of Existing Bridge Bent Plastic Hinge Region and Rapid Repair Measures of Earthquake Damaged Bridges Considering Future Resilience

A K M Golam Murtuz
Portland State University

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Seismic Performance Design Criteria of Existing Bridge Bent Plastic Hinge Region and Rapid Repair Measures of Earthquake Damaged Bridges Considering Future Resilience

by

A K M Golam Murtuz

A dissertation submitted in partial fulfillment of the requirements for the degree of

Doctor of Philosophy

in

Civil and Environmental Engineering

Dissertation Committee:
Peter Dusicka, Chair
Franz Rad
Thomas Schumacher
Hormoz Zareh

Portland State University
2022
ABSTRACT

The main objective of this research was to evaluate the seismic performance of existing sub-standard reinforced concrete (RC) bridge column-spread footing subassemblies and to quantify the material strain limits through a full-scale experimental program. A total of six column-footing test specimens with pre-1990 construction details were subjected to reverse cyclic lateral loading, utilizing a conventional three-cycle symmetric loading protocol and a protocol representing the demands expected from a CSZ earthquake. Additionally, the tests were designed so that variable axial loading could be applied to simulate the secondary load effects experienced during an earthquake in a column that is part of a multi-column bent. Despite having sub-standard seismic detailing, all the test specimens with moderate lap splice length or continuous rebar demonstrated a ductile response, reaching a minimum displacement ductility of $\mu = 5.4$. The surprisingly ductile response can be attributed to the moderate splice length that ensured flexural plastic hinging and the low longitudinal steel ratio that resulted in significant rocking at the column-footing interface. Furthermore, flexural cracking of the accompanying spread footing and splice failure of the column dowel bars were also observed for specimens having different reinforcing and splice details. The performance of these test specimens was evaluated in terms of global and local deformation quantities, i.e., hysteretic load-deformation response, measured strains, flexural curvature profiles, etc. Finally, the experimentally obtained strain values at different damage states were used to define probabilistic operational and life safety performance criteria for seismic evaluation of the representative bridge bents. The spread of plasticity was also examined with respect to the existing plastic hinge model to be used for limit state evaluation.
A rapid repair method incorporating semi-permanent installation was also developed, anticipating the need for quick measures following the Cascadia Subduction Zone (CSZ) earthquake, which is expected to damage the existing bridges in the Pacific Northwest and spread geographically throughout the region. While the conventional repair methods are effective in restoring the strength in the damaged zones, often lead to higher stiffness and strength that would likely result in shifting the failure to other parts of the structure under future earthquake or aftershock demands. The proposed repair methodology uses capacity design principles to protect the remainder of the bridge from future earthquakes and eradicates the need for establishing rebar continuity, resulting in a less labor-intensive repair method. The adopted concept is to utilize U-shaped metallic plates as externally attached ductile fuses to be anchored to the non-damaged part of the column, hence bypassing the damaged zone to restore the lateral capacity. The damaged zone can then be repaired with strips of fiber-reinforced polymer (FRP) sheets to provide lateral confinement, preventing any further damage to the core concrete. A typical substandard reinforced concrete column-to-footing subassembly was damaged during a full-scale cyclic test under the CSZ loading protocol. The column was then repaired following the rapid repair methodology and retested to achieve the as-built column lateral capacity and self-centering behavior. Results showed the potential of this methodology to restore the lateral capacity while protecting the remainder of the column from any further damage. The targeted self-centering behavior was also evident under the applied axial load. Development of the repair methodology, analytical design approach, and results from a
full-scale cyclic test validating the performance of seismically substandard concrete bridge substructure is presented in this dissertation.
DEDICATION

Do you see the green carpet? Yes, this where I am from and this where I will be at the end. Dedicated to this small village named Goalbaria, where I lived like a bird roaming around the green paddy fields!

&

To my parents, sisters, and beautiful wife.
ACKNOWLEDGEMENTS

I would like to express my gratitude to my supervisor, Professor Peter Dusicka, for the opportunity to work with him and for his unwavering support and guidance, both intellectually and emotionally. Throughout my Ph.D. program, his inherent knack for research has motivated me, and I consider myself fortunate to have him as my supervisor.

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Chapter 1 Introduction

1.1 Background

Seismic design of new bridges and assessment of existing bridges in western half of Oregon needs to consider two-level performance objectives; life-safety and operational. The life-safety performance objective is intended to ensure that a bridge does not collapse under the design earthquake; however, the bridge is expected to sustain significant damage. The operational performance objective is intended to limit the seismic damage resulting from a specific demand level so that functionality of the bridge is minimally impacted. Within the Western parts of the state, the structural design of the substructure is often governed by the operational performance criteria and not the life safety criteria. The current design methodology for the operational performance objective has two main components: use of a specific seismic hazard that has traditionally been lower than that used for life safety performance, and limit of the material strains to stricter levels than those used for life safety.

While the operational performance objective often governs bridge bent design, but limited confidence exists in the selection of the appropriate material strain limits for achieving rapid return to operational condition. This lack of knowledge has cascading effects on the direct cost of construction, especially when considering the retrofit of existing bridges. This has recently been highlighted in retrofit assessment projects conducted on a select number of bridges where the designers found that the operational performance under the Cascadia Subduction Zone (CSZ) event governed the extent of retrofit required (Bazaez and Dusicka 2016b).
Seismic retrofit or replacement of the entire vulnerable bridge inventory in Oregon is unlikely in the foreseeable future, leaving us with a large bridge inventory of seismically vulnerable bridges. One of the major issues facing the transportation infrastructure during and following CSZ earthquake is not necessarily the magnitude of shaking at any particular site alone, but the vast and varied damage that will be distributed throughout the state. Damage is expected to be geographically spread and have a nearly simultaneous impact on transportation West of I-5 up and down the state.

Variability in intensity across the state combined with the actual individual bridge responses will mean that the extent of damage throughout the inventory will vary from minor to significant. Bridge repair in lieu of replacement will be needed following the CSZ earthquake. Priority will be placed on resuming mobility such that repairs will need to be implemented quickly and in many cases expected to remain for the useful life of the bridge as not all damaged bridges would be slated for replacement.

1.2 Performance Based Seismic Design Criteria

The performance-based seismic design/assessment philosophy is based on the approach that ensures certain performance objectives for new bridges or for retrofitting existing bridges. Performance objectives like operational and life safety are usually defined qualitatively based on the damage state in the structure. However, a quantitative definition of these damage states and hence the performance objective needs to be defined with respect to engineering deformation criteria such as strain, displacement, or curvature limits. Material strain values such as concrete compressive strain and steel tensile strain are inherent material properties that can be related to damage states. Past research has been
dedicated for the development of strain-based limit states for reinforced concrete bridge columns. The following sections presents significant past research using material strain-limit states for reinforced concrete bridges and the need for research.

### 1.2.1 Literature Review

In order to relate system to component level performance, five performance levels were proposed and is shown in Table 1.1 (Hose and Seible 1999). The assessment procedure developed was closely related to previous work done on buildings (SEAOC 1996). This parameterization of bridge components, sub-assemblages, and systems is useful for the development of a consistent performance-based design methodology for bridges in seismic zones.

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<tr>
<th>Performance Level</th>
<th>Qualitative Performance Description</th>
<th>Quantitative Performance Description</th>
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<td>I Cracking</td>
<td>Onset of hairline cracks.</td>
<td>Cracks barely visible.</td>
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<tr>
<td>II Yielding</td>
<td>Theoretical first yield of</td>
<td>Crack widths &lt; 1 mm.</td>
</tr>
<tr>
<td></td>
<td>longitudinal reinforcement</td>
<td></td>
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<tr>
<td>III Initiation of</td>
<td>Initiation of inelastic deformation.</td>
<td>Crack widths 1 - 2 mm.</td>
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<tr>
<td>Mechanism</td>
<td>Onset of concrete spalling.</td>
<td>Length of spalled region &gt; 1/10</td>
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<td></td>
<td>Development of diagonal cracks.</td>
<td>cross-section depth.</td>
</tr>
<tr>
<td>IV Full</td>
<td>Wide crack widths/spalling over</td>
<td>Crack widths &gt; 2mm.</td>
</tr>
<tr>
<td>Development of</td>
<td>full local mechanism region.</td>
<td>Diagonal cracks extended over 2/3</td>
</tr>
<tr>
<td>Local Mechanism</td>
<td></td>
<td>cross-section depth.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Length of spalled region &gt; 1/2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>cross-section depth.</td>
</tr>
<tr>
<td>V Strength</td>
<td>Buckling of main reinforcement.</td>
<td>Crack widths &gt; 2mm in</td>
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<tr>
<td>Degradation</td>
<td>Rupture of transverse reinforcement.</td>
<td>concrete core.</td>
</tr>
<tr>
<td></td>
<td>Crushing of core concrete.</td>
<td>Measurable dilation &gt; 5% of</td>
</tr>
<tr>
<td></td>
<td></td>
<td>original member dimension.</td>
</tr>
</tbody>
</table>
Priestley (2000) outlined three different methods of performance-based design, including direct displacement-based design, and compared them with a traditional force-based design approach. Two performance limit states namely “Fully Operational” and “Damage Control” were presented under the direct displacement design approach. Concrete and steel strain values were used to define the structural damage states whereas residual drift ratio was used to define the non-structural damage levels. The “Fully Operational” limit state was defined by the onset of concrete crushing and/or formation of residual crack widths exceeding 1 mm. The “Damage Control” limit state was defined by core concrete compressive strain to avoid non-repairable damage due to core crushing and/or peak longitudinal steel tensile strain to avoid bar buckling and low cycle fatigue. The proposed strain limit state values are summarized in Table 1.2.

<table>
<thead>
<tr>
<th>Performance Level</th>
<th>Concrete Compressive Strain</th>
<th>Steel Tensile Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fully Operational</td>
<td>0.004 cover concrete crushing</td>
<td>0.015 Residual crack widths within range of 0.5 – 1.00 mm</td>
</tr>
<tr>
<td>Damage Control</td>
<td>$\varepsilon_{cm} = 0.004 + 1.4 \rho_s f_{yh}$  (\varepsilon_{sub}/f'_{cc})</td>
<td>$\varepsilon_{sm} = 0.6 \varepsilon_{su}$ Bar buckling and low cycle fatigue</td>
</tr>
</tbody>
</table>

\(\rho_s = \text{volumetric ratio}, f_{yh} = \text{yield strength}, \varepsilon_{sub} = \text{strain at maximum stress}, f'_{cc} = \text{compression strength of the confined concrete}, \varepsilon_{sm} = \text{maximum longitudinal reinforcement tensile strain}, \) and \( \varepsilon_{su} = \text{strain at maximum stress}. \)

The research by Kowalsky (2000) developed dimensionless curvature relationships for serviceability and damage control limit states based on concrete compressive and steel strain limits. The serviceability concrete compressive strain limit was defined as the onset of concrete crushing. On the other hand, the serviceability steel tensile strain limit was
defined as the strain corresponding to a residual crack width exceeding 1.0 mm. The definition of the damage control concrete strain limit state was defined as the point up to repairable damage to core concrete. Whereas the steel strain at damage control limit was attributed to peak tension strain in longitudinal reinforcement before any visible bar buckling takes place. The proposed strain limit state values are tabulated in Table 1.3.

<table>
<thead>
<tr>
<th>Performance Level</th>
<th>Concrete Compressive Strain</th>
<th>Steel Tensile Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Serviceability</td>
<td>Concrete cover crushing</td>
<td>0.015 Residual crack widths &gt; 1 mm</td>
</tr>
<tr>
<td>Damage Control</td>
<td>$\varepsilon_{cu} = 0.018$ Limit of economical concrete repair</td>
<td>0.06 Tension based bar buckling</td>
</tr>
</tbody>
</table>

The work carried out by Sheikh et al., (2012) correlated seismic performance objectives (both qualitative and quantitative) with engineering parameters that are based on experimental investigations and field investigations from recent earthquakes. The limit state values along with the corresponding qualitative and quantitative performance description are presented in Table 1.4. The authors utilized a simplified assessment methodology based on pushover analysis procedures incorporating a substitute structure approach. The substitute structure approach uses a modified linear model for the structure while considering the effect of energy dissipation in the nonlinear range of displacement. They found that the method was fully validated with available experimental data.

<table>
<thead>
<tr>
<th>Limit states</th>
<th>Performance Level</th>
<th>Quantitative Performance Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>Fully Operational</td>
<td>$\sigma_c = f_{cr} = 0.4\sqrt{f'_c}$</td>
</tr>
<tr>
<td>1B</td>
<td></td>
<td>$\sigma_s = f_{sy}$</td>
</tr>
</tbody>
</table>
### Chapter 1 - Introduction

| 2 | Delayed Operational | \( \varepsilon_c = -0.004 \)  
\( \varepsilon_s = 0.007 \)  
\text{crack width} = 2 \text{ mm} |
|---|---|---|
| 3 | Stability | \( \varepsilon_c = \varepsilon_{cc50} \) (initial core crushing)  
\( \varepsilon_c = \varepsilon_{cu} \) (fracture of hoops)  
\( \varepsilon_s = \varepsilon_{su} = 0.07 \) (longitudinal reinforcement fracture)  
\( \varepsilon_s = \varepsilon_{scr} \) (onset of buckling) |

\( f'_c \) = concrete compressive capacity, \( \varepsilon_{cc50} \) = post peak axial strain in concrete when capacity drops to 50\% of confined strength, \( \varepsilon_{cu} \) = ultimate strain of concrete, \( \varepsilon_c \) = average tensile strain in longitudinal reinforcement, \( \varepsilon_{su} \) = tensile strain at fracture, and \( \varepsilon_{scr} \) = steel strain at onset of buckling of longitudinal bars.

The research undertaken by Lehman et al., (2004) assessed the seismic performance of 10 one-third scale models of well-confined, circular-cross-section, RC bridge columns. Key variables included were aspect ratio, longitudinal reinforcement ratio, spiral reinforcement ratio, axial load ratio, and length of the well-confined region adjacent to the zone where plastic hinging was anticipated. Data obtained from the tests were used to define different damage states, i.e., residual cracking, cover spalling, and core concrete crushing. These were defined with respect to engineering parameters such as concrete compressive strain and longitudinal steel tensile strain. Strains were obtained using foil strain gauges on the longitudinal and spiral reinforcement. Concrete compressive strains were approximated using external deformation measurements (LDVT). All columns showed similar damage progression, i.e., concrete cracking, longitudinal reinforcement yielding, initial spalling of concrete cover, complete spalling of concrete cover, spiral fracture, longitudinal reinforcement buckling, and longitudinal reinforcement fracture. It was found that the residual crack width was insignificant below the yield strength of
longitudinal reinforcement. Also, it is unrealistic to define residual crack width in terms of maximum previous reinforcement strain due to large scatter in the data analyzed. However, concrete cover spalling, and core concrete crushing were attributed to a range of measured strains of 0.0039 to 0.011 and 0.010 to 0.029, respectively. Lower spalling strains were observed for larger column aspect ratios whereas axial load ratios, reinforcement ratio, and confinement ratio were not found to have a significant influence on the considered range of damage. Finally, it was found that the longitudinal bar buckling/reinforcement fracture failure mode depends on the lateral loading history and cannot be captured by a single strain limit.

Moyer and Kowalsky (2003) presented a hypothesis regarding the bar buckling mechanism which states that the buckling of reinforcing bars upon load reversal are directly influenced by the peak tensile strain. They have experimentally validated the hypothesis on influence of tension strain on the buckling behavior of longitudinal reinforcing bar in RC columns. It was also observed that a cyclic load history had a pronounced effect on peak tension strain prior to bar buckling. In addition, the nature of the effect of load history was investigated and it was found that either extensive cycles at low response levels or only one cycle at a higher level of response can cause an accumulation of this tension strain.

An experimental study by Goodnight et al., (2013), investigated the effect of lateral displacement history on the bar buckling limit state of well-confined circular RC bridge columns. The results showed that the buckling of reinforcing bar can take place at a relatively low peak tensile strain under a lateral displacement history with a large enough compressive demand. This impact of the load history is mainly due to the accumulated
strain within the transverse and longitudinal reinforcing bars. However, it was also found that the strain and displacement envelopes were independent of the effect of displacement history.

Feng et al. (2015a; b) developed hybrid fiber based and finite element model to investigate the effect of the seismic load history on bar buckling limit states in RC bridge columns. Their analytical investigations showed that the buckling of a longitudinal reinforcing bar depends on bar diameter and the spacing and diameter of the transverse reinforcement. It was also observed that when the transverse reinforcement experienced inelastic strain, this adversely affected the peak tensile strain corresponding to the bar buckling limit state.

The most notable recent experimental endeavor by Goodnight et al. (2016a) consisted of thirty full-scale circular, well-confined, RC bridge columns to investigate the strain limits for different damage states, namely serviceability, spiral yielding, and reinforcing bar buckling. Key variables included in this study were lateral displacement history, axial load, longitudinal steel ratio, aspect ratio of the columns, and transverse reinforcement detailing. Strains were measured using non-contact 3D position sensors through tracking a network of markers in the 3D space with an accuracy of 0.1 mm, which allowed accurate measurement of strains comparable to conventional surface-mounted sensors. The sequence of damage observed during the test was concrete cracking, longitudinal reinforcement yielding, cover concrete spalling, transverse reinforcement yielding, longitudinal bar buckling, and fracture of previously buckled bars. Material
strains at the onset of the respective failure modes were recorded and presented with respect to the three performance limit states.

The “serviceability” limit state as defined by Goodnight et al., (2016a) as the point beyond which an interruption of use of a bridge is necessary without posing a safety concern and require some degree of intervention for the long-term serviceability of the structure. This limit state is attributed to cover concrete spalling or residual crack widths large enough to require epoxy injection to prevent future corrosion. The results from the experimental study showed that a concrete compressive strain limit of 0.0048 was reasonable to minimize the prediction error and measured crushing strains. However, a concrete compressive strain limit of 0.004 at the serviceability limit state was maintained. The conservatism in the strain limit value was counteracted by the equivalent moment curvature distribution proposed by Goodnight et al., (2016a) for calculating the target displacement. On the other hand, a steel tensile strain limit value of 0.015 as defined by Kowalsky (2000) corresponding to a serviceability limit state with 1 mm crack width was found to be conservative and implies that the serviceability limit state is governed by the concrete crushing strain limit state.

Transverse reinforcement yielding was stated as a limit state by Goodnight et al., (2016a) that prompts changes in the repair strategy from cover concrete patching or epoxy injection to additional transverse stiffness in the plastic hinge regions. The onset of transverse reinforcement yielding was expressed in terms of concrete compressive strain. An empirical equation was formulated to predict the concrete compressive strain at the onset of transverse reinforcement yielding. The empirical relationship as presented by
equation 1.1 relates the effect of longitudinal reinforcement ratio to the expected yield strain of the transverse reinforcement. Moreover, data obtained from the experimental study of Goodnight et al., (2016a) it was found that localization of compressive demand can occur in the regions with inelastic transverse steel reinforcement.

\[
\varepsilon_{c, \text{spiral yield}} = 0.009 - 0.3*(A_{st}/A_g) + 3.9*(f_{yhe}/E_s) \tag{1.1}
\]

Where, \( \varepsilon_{c, \text{spiral yield}} \) = Concrete compressive strain at the onset of transverse reinforcement yielding, \( A_{st}/A_g \) = Longitudinal reinforcement ratio, \( f_{yhe}/E_s \) = Expected yield strain of the transverse steel.

The bar buckling limit state was defined by Goodnight et al., (2016a) as the peak tensile strain in the longitudinal reinforcement prior to bar buckling. An empirical relationship was developed with respect to transverse steel ratio, expected yield strain of transverse reinforcement and column axial load ratio. Equation 1.2 presents the bar buckling limit state developed by Goodnight et al., (2016a). However, buckling of the longitudinal reinforcement was observed to take place while the bar is under net elongation but compressive stress.

\[
\varepsilon_{s, \text{bar buckling}} = 0.03 + 700* \rho_s*(f_{yhe}/E_s) - 0.1*(P/f'_{ce}*A_g) \tag{1.2}
\]

Where, \( \rho_s \) = Transverse volumetric steel ratio, \( P/f'_{ce}*A_g \) = Column axial load ratio with expected material properties.

Also, the Oregon Bridge Design Manual (BDM 2021) and Canadian Highway Bridge Design Code (CHBDC 2013) have defined the material strain limit states for different performance criteria and are tabulated in Table 1.5.
Table 1.5 Material strain limit states (BDM 2021; CHBDC 2013)

<table>
<thead>
<tr>
<th>Canadian Highway Bridge Design Code</th>
<th>Material</th>
<th>Minimal damage</th>
<th>Repairable Damage</th>
<th>Extensive Damage</th>
<th>Probable Replacement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>0.006</td>
<td>-</td>
<td>Extensive concrete spalling is permitted but $\varepsilon_{cc} \leq 0.8 * \varepsilon_{cu}$</td>
<td>Damage does not cause crushing of confined concrete core</td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td>0.010</td>
<td>0.025</td>
<td>0.05</td>
<td>0.075 0.06 for 35M or larger</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ODOT</th>
<th>Material</th>
<th>Operational</th>
<th>Life Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>$\varepsilon_{cc} = 0.005$</td>
<td>$\varepsilon_{cc} = 0.9 * \varepsilon_{cu}$</td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td>$2^*\varepsilon_{sh}$</td>
<td>$\varepsilon_R^{su}$</td>
<td></td>
</tr>
</tbody>
</table>

$\varepsilon_{cc} =$ The confined compressive strain, $\varepsilon_{cu} =$ The ultimate confined compressive strain, 
$\varepsilon_{sh} =$ Onset of strain hardening, $\varepsilon_R^{su} =$ Reduced ultimate tensile strain

1.2.2 The Need for Research

Current material strain limits are based on experimental data generated for reinforced concrete columns that are either based on 1) modern detailing requirements, or 2) exhibit excessively poor detailing relative to those used in existing bridges in Oregon, or 3) do not consider the cumulative damage effects from the long duration CSZ event. Recently completed tests of a bent representative of ODOT detailing were completed as part of a project on earthquake duration effects (Bazaez and Dusicka 2016b). One of the tangent results provided limited, but tantalizing, data that pointed to a seismic performance that was better than anticipated, given the lack of modern detailing. The possible contributing factors of this surprising result may be the intermediate lap-splice lengths (between excessively short and modern long splice details) utilized in vulnerable ODOT bridges prior to the 1980s, low longitudinal reinforcement ratio and the nearly constant...
axial load considered. More focused research is therefore needed to assess the influence of these variables. Specifically, on the types of detailing used in Oregon due to the potentially positive outcome on the overall seismic bridge design.

1.3 Low-Damage Seismic Repair

The Repair objectives of the earthquake damaged substandard bridge columns depend on the design details and seismic performance of the column in as built condition. Past experimental studies on representative substandard bridge columns of Pacific Northwest having moderate lap splice length in the plastic hinge region was controlled by flexural response (Bazaez and Dusicka 2016b; Lopez et al. 2020; Mehary et al. 2018; Murtuz et al. 2020). The bridge columns achieved moderate ductility prior to significant lateral strength degradation and without apparent loss of axial load carrying capacity. However, significant damage ranging from lap splice failure, rebar buckling, and rebar fracture is expected to take place in the column plastic hinge region. Post-earthquake repair of these bridge columns is therefore needed to be implemented with an aim to restore the strength and stiffness of the as built condition and maintain or enhance the displacement ductility capacity. Rapid implementation of the repair method is also of key interest for bridges along the lifeline route to restore the mobility following damaging earthquake.

1.3.1 Literature Review

Conventional repair methods such as repairing of cracked concrete with epoxy injection, encasing the column in concrete jacket (Bett et al. 1988), steel jacketing (Chail et al. 1991), FRP wrap (Chang et al. 2004; He et al. 2013; Rutledge et al. 2014; Saadatmanesh et al. 1997; Sheikh and Yau 2002; Vosooghi and Saiidi 2013), active
confinement with smart materials (Shin and Andrawes 2011), etc. aim to restore strength and stiffness to the damaged zones. These methods were found to be most effective for cases of low damage, whereby the steel reinforcement cage remains largely intact. More involved procedures are needed for cases of loss of lap splice, buckled or fractured rebar or merely loss of confidence at the remaining low cycle fatigue capacity for subsequent earthquakes. Most of the past research focused on restoring rebar continuity (Shin and Andrawes 2011), addition of longitudinal reinforcement (Lehman et al. 2001), applying externally bonded longitudinal reinforcement (He et al. 2013), plastic hinge relocation (Rutledge et al. 2014; Wu and Pantelides 2017), etc. for continuity prior to the encasement or wrap. While the conventional methods are effective at restoring the strength and stiffness of the damaged columns, they are not particularly suitable for rapid repair after a damaging earthquake like CSZ event that has the potential for further damage during large magnitude aftershocks that are expected for CSZ earthquake. Hence, a repair method that has the potential to be implemented rapidly and can ensure resilience under future earthquake events would be best suited for Oregon bridge inventory. Past research has been conducted on the development of low-damage system with externally attached dissipation devices and the principle of controlled rocking behavior. The following paragraphs outline past research conducted on developing low damage system utilizing different techniques such as use of ductile energy dissipation devices, low damage smart materials, posttensioning with or without additional supplemental damping etc. for the construction of new system or repair/retrofit of existing structures.
Kelly et al. (1972) investigated the feasibility of three different energy dissipating devices for earthquake engineering application. One of the devices were the U-shaped mild steel strips that relied on the plastic deformation of mild steel for energy dissipation. It was noted that the U-shaped devices can be in a region of the structure where they can be easily accessed for quick replacement following an earthquake event. Such an advantage of the U-shaped strips makes them suitable for rapid repair methods utilizing externally attached energy dissipation devices.

Mander and Cheng (1997) developed a “Damage Avoidance Design (DAD)” methodology for modular beam-column connection that allowed rocking under lateral loads to avoid damage in structural elements. The longitudinal reinforcements were discontinuous at the beam-column connection to facilitate rocking of the elements. The rocking toe of the connection was also detailed with a special steel-to-steel interface to prevent crushing of concrete resulting from stress concentration. Furthermore, the use of supplemental unbonded post-tensioned tendons improved the lateral strength of the system. The design methodology was validated with an experimental program on a near full-scale rocking column substructure where the column, connection and the foundation were found to remain damaged free. It was noted that the energy dissipation capacity of the rocking system alone is significantly lower and induces higher demand in the foundation system. The experimental program indicates the inability of rocking system alone to dissipate energy under earthquake demand. Hence, a rocking system with supplemental damping element will significantly improve the seismic performance where the energy will be
dissipated through the damping elements and the damage can be avoided with the rocking motion.

Pampanin et al., (2001a) outlines the inability of the classical section analysis methodology in predicting the response of precast beam-column system where presence of unbonded reinforcements at the critical section invalidates the concept of strain compatibility between steel and concrete material. The authors then introduced an iterative analytical procedure for sectional analysis of precast beam-column system. The iterative procedure uses a “monolithic beam analogy” concept where the equivalent plastic deformation of a monolithic beam is equated with the rigid rotation resulting from gap opening at the precast beam-column interface. The concept was found to be simple and effective to analytically predict the load-deformation response of precast beam-column connection.

Palermo et al., (2004) numerically investigated the feasibility and efficacy of the hybrid or controlled rocking connection for bridge piers and compared the results with a traditional monolithic system. The hybrid connection combines the self-centering properties to limit the residual displacement and energy dissipating properties to limit the damage within sacrificial devices. Result shows the promise of controlled rocking connection with negligible residual displacement compared to 10-15% residual displacement for monolithic bridge piers.

Palermo et al., (2007) experimentally investigated the seismic performance of hybrid jointed ductile connection for bridge piers and presented a simple design methodology along with modeling recommendations. In the hybrid connection, re-
centering feature was added with unbonded posttensioned tendons and deformed mild steel bars grouted into metallic sleeve were used as energy dissipating devices. Results obtained from the quasi-static cyclic test shows the enhanced seismic performance of hybrid connection in terms of minimal damage in structural elements, negligible residual displacement, and stable hysteretic response up to a high displacement ductility level. It was also found that the use of mild steel dissipating device can significantly reduce the repair cost by accumulating damage in the sacrificial element. Furthermore, the simplified design approach and lumped plasticity model was found to be satisfactory for the hybrid connection.

Newcombe et al., (2008) expanded the use of simplified analytical design procedure initially developed for precast concrete frame and column system to hybrid timber jointed ductile connection. The “monolithic beam analogy” concept was successfully used in the design procedure and the simplified method was verified with experimental results. When compared with a refined numerical analysis, the simplified method was found to be appropriate for design purposes. The authors also proposed recommendations for connection detailing with internal and external dissipative devices and estimated the strain penetration of internal energy dissipation system.

Palermo and Pampanin (2008) performed parametric analysis on different precast concrete hybrid system with unbonded posttensioned tendons to investigate the impact of different design variables (i.e., unbonded length of tendons, mild steel, element length and section parameters etc.) and develop design charts to be used with a simplified design procedure. An approximate closed form solution applicable only for the rectangular section
was also developed and compared with the charts-based design procedure. It was found that both the methods can be used reliably but the design chart method are more accurate while the closed form solution is more intuitive for design purpose.

Marriott et al., (2009) conducted experimental test on unbonded posttensioned bridge piers with externally attached replaceable mild steel hysteretic dissipator devices organized in different orientation. The results were compared with an equivalent reinforced monolithic benchmark bridge pier that shows the ability of the hybrid system with external dissipators in terms of enhanced stability, recentering and energy dissipation capacity without any significant damage to the structural elements. It was also found that the externally attached dissipators result in better stability and energy dissipation compared to the more traditional internally grouted mild steel reinforcement hybrid system. Furthermore, the use of properly calibrated multi spring model based lumped plasticity model was found to provide acceptable local and global response prediction for the hybrid system. This experimental study highlights the better performance of externally attached dissipators as compared to the internal ones for bridge pier system.

Solberg et al., (2009) experimentally validated the theoretical “Damage Avoidance Design” concept for bridge piers under bidirectional quasi static and pseudo dynamic loading. A steel-to-steel rocking interface connection with tension only energy dissipators were used to achieve the damage avoidance bridge pier system. The mild steel energy dissipating devices were used internally into an enlarged section at the base of the column. Results show that the damage avoidance bridge pier system can survive the design basis earthquake without sustaining any damage. However, the system was not convincing
enough to prevent complete collapse under maximum credible earthquake demand. Moreover, the impact of the internal energy dissipating devices was found to be insignificant in the seismic performance. It was indicated that a more efficient design approach could have resulted in better performance of the internal energy dissipating devices. However, it was found that the performance of the damage avoidance bridge pier was superior compared to the benchmark ductile bridge pier.

ElGawady and Sha’lan (2011) experimentally investigated the performance of four self-centering bridge bents constructed with precast posttensioned concrete filled fiber tube (PPT-CFFT) columns. Different construction details were used for each of these bents where mild steel angle dissipaters were added as externally attached dissipating device for one of the specimens. Seismic performance was compared with a monolithic moment resisting concrete bent and was found to have superior performance in terms of reducing damage, negligible residual displacement, and ultimate displacement capacity. However, the use of external energy dissipating device was found to introduce some damage in the bent. It was concluded that an efficient design of the external dissipaters will likely improve the seismic performance of the bents. The experimental program utilizes a different dissipating device where a mild steel angle section was connected at the beam-column and column-footing interface. The article also outlines the importance of properly designed device for better seismic performance.

Marriott et al., (2011) experimentally investigated the effect of biaxial earthquake loading on a unbonded posttensioned bridge pier with replaceable external mild steel dissipater and a monolithic ductile reinforced concrete bridge pier. Results obtained from
the experimental test was compared with a uniaxial test and significant reduction in strength and ductility capacity was observed for both the posttensioned and monolithic bridge pier under the biaxial loading. Rupture of the external mild steel dissipaters was found along with minor flexural cracking and small amount of concrete spalling in the column toe region. However, structural integrity of the posttensioned bridge pier was intact, and it was concluded that the repair cost of the damaged bridge pier would be minimal owing to the replaceable nature of the external energy dissipating devices. This study reflects the advantage of externally attached dissipaters over the internally attached ones due to easy accessibility, hence the ability to replace them quickly. It also identifies that the biaxial loading would require special detailing for better performance of the hybrid connection.

Newcombe et al., (2011) provides seismic design recommendation and analytical modeling methods for posttensioned timber wall system with U-shaped flexural plate coupling element. Existing design principles originally developed for posttensioned precast concrete member was modified to account for the elastic deformation of the timber wall and the influence of the floor system. An analytical equation was proposed for conservative approximation of the elastic deformation. Finally, the design approach was validated through nonlinear time history analysis.

Newcombe (2011) developed analytical design procedure for posttensioned timber frame and wall system. Analytical methods were then validated with experimental testing on a scaled down timber wall and frame building under unidirectional and bidirectional earthquake loading. Both the wall and frame system were tested as having only
posttensioned condition and a hybrid system where the U-shaped flexural plate elements were coupled with the posttensioning feature. Results show that the U-shaped flexural plate used as coupling element provides additional strength, stiffness, and energy dissipation capacity of the timber building system. Furthermore, it was found that bidirectional earthquake loading doesn’t significantly impact the seismic performance of the system and hence it was concluded that decoupled orthogonal response can be conservatively used for design purposes.

Palermo and Mashal (2012) discussed the trends and challenges in Accelerated Bridge Construction (ABC) and the use of low damage seismic resistant technology in the context of international practice and New Zealand perspective. Different concepts for low damage technology are also discussed along with the use of U-shaped flexural plates dissipating devices for the construction of new generation damage resistant bridges.

Baird et al., (2013) conducted experimental and numerical investigation for the use of U-shaped flexural plates in an innovative cladding connection to be used in multi-story building. Results show that the cladding connection was effective in reducing the hysteretic energy dissipated through structure and the inter-story displacement was also reduced. Furthermore, it was indicated that the cladding connection can be used for both new and retrofitted building.

Baird et al., (2014) identifies the requirements for choosing an ideal replaceable energy dissipators as the simplicity in design, low fabrication cost, flexibility in application, being robust and replaceable and indicates the U-shaped flexural plates as a perfect choice that met all the required criteria. The authors then outline the difficulty in
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predicting the load-deformation response of the UFP’s and conducted an experimental and numerical parametric study to determine the important design parameters such as pre and post yield stiffnesses. Finally, set of equations predicting the load-deformation responses of the UFP’s were proposed and preliminary design guideline was also recommended.

Eatherton et al., (2014b) developed design recommendation for a controlled rocking self-centering steel braced multi-story frame system where the self-centering features was added with posttensioned tendons and the replaceable butterfly shear fuse plates were used as energy dissipating devices. The enhanced performance of the self-centering rocking frame system in eliminating residual displacement and accumulating damage in the replaceable fuses was confirmed. Furthermore, the guideline for the key limit states was also developed for rocking steel frame design.

Sideris et al., (2014) introduced a novel precast segmental bridge system incorporating hybrid posttensioning and sliding-rocking joints. Two types of joints are used for the substructure column and the superstructure girders namely sliding dominant (SD) joints and rocking dominant (RD) joints respectively. In both the system, either straight or curved posttensioned tendons were used. Shake table testing of the specimens showed that both the specimens were efficient in limiting damage in the structural elements. However, the self-centering capability was higher for the RD joint, and the energy dissipation was higher for SD joint.

Guerrini et al., (2015) experimentally investigated the performance of a hybrid concrete-dual steel shell bridge column with internal and external energy dissipating device and discussed the design criteria. The proposed system was devised with an aim for
accelerated bridge construction and minimizing damaging utilizing energy dissipating fuse elements. Two types of energy dissipators were used, one being internal unbonded stainless steel bars and the other is external mild steel bar grouted in concrete sleeve to act like a buckling restraint bracing. Results show that both the internal and external energy dissipating devices fractured at the same drift ratio indicating no significant difference due to the internal or external nature of the devices.

He et al., (2015) summarized the state of experimental program on seismic repair of earthquake damaged bridges. The authors divided the repair techniques into two categories one for the columns with fractured longitudinal rebar and the other for columns with significant damage like extensive spalling or core crushing but without any longitudinal rebar fracture. Authors concluded that the traditional repair methods like jacketing (concrete, steel, FRP, SMA etc.) can be used effectively to restore or enhance the strength of the damaged column but changes in initial stiffness could lead to change in global behavior of the bridges that could potentially force damage in other parts of the structure. It was also concluded that the columns with fractured longitudinal rebar can be successfully repaired by replacing and mechanically restoring the rebar continuity. However, it was indicated that such a repair implementation is labor intensive and time consuming that makes it less unsuitable for rapid repair approach.

Mashal (2015) experimentally evaluated two types of accelerated bridge construction techniques namely emulative cast in place that forms plastic hinges to dissipate energy but offer accelerated construction due to the use of prefabricated segments and the other Dissipative Controlled Rocking (DCR) connection that uses unbonded
posttensioning to achieve self-centering and replaceable external energy dissipating devices as sacrificial element. Different types of innovative dissipaters i.e., bracing type UFP dissipater, mini UFP dissipater with or without self-centering capability etc. were developed to be used with the DCR connection. Results show that the DCR connection offers superior seismic performance over the emulative cast in place construction in terms limiting damage in the column and post-earthquake repair needs.

Mashal et al., (2016) presents the findings of quasi static cyclic testing on emulative cast in place bridge system with grouted duct connection and member socket connection. Results show that the connections were able to achieve significant energy dissipation despite the absence of external energy dissipating devices. It was also found that the displacement ductility of the system was like that of ductile monolithic construction. However, both the connections were found to form plastic hinges in the column region and sustained significant damage ranging from concrete spalling to rebar fracture. It was recommended that these types of connection along with external energy dissipating mechanism can significantly enhance the seismic performance while offering reduced construction time through accelerated bridge construction.

Sarti et al. (2016a) presented the design, detailing and experimental testing results of an alternative column-wall-column timber dissipative posttensioned rocking system to mitigate the vertical displacement incompatibility between the diaphragm and the wall resulting from the gap opening at the rocking interface. The system uses U-shaped flexural plates as the external energy dissipating devices. Results show that the system was able to reach significant energy dissipation while achieving a stable hysteretic response. However,
minor stiffness degradation was observed due to the ratcheting behavior of the dissipaters but the impact on overall response of the system was negligible. The stiffness degradation was however resulted in a reduction of the viscous damping of the system.

Tazarv and Saiidi (2016) experimentally investigated the seismic performance of precast bridges made with low damage materials like ultrahigh-performance concrete, engineered cementitious concrete (ECC) and shape memory alloy steel bar. While the system was efficient in reducing the residual displacement compared to the reference cast in place bridge column, but significant cover spalling, and large crack opening was attributed to the column.

White and Palermo (2016) experimentally investigated posttensioned system with emulative and non-emulative column-footing connection for bridge piers. Three different types of energy dissipaters were used for non-emulative socket and hybrid coupled connection. The emulative socket connection was found to achieve comparable energy dissipation and ductility compared to the cast in place construction but sustained significant damage ranging from spalling to more severe rebar buckling. The non-emulative socket connection, on the other hand, showed significantly less damage and a flag-shaped hysteretic curve representative of self-centering system was achieved at larger column drifts. Pre-mature failure of the non-emulative coupled connection dissipaters was observed at 3% drift ratio due to the difficulties in detailing of the dissipaters. Finally, it was concluded that the repair methodologies need further improvement to ensure reliability, robustness, and effectiveness in construction.
Thonstad et al., (2017) performed experimental cyclic test on precast pretensioned rocking bridge column-to-footing and column-to-cap beam subassemblies having unbonded rebar in the column. Ends of the columns near the rocking interface between footing and cap beam were also confined with steel tubes and annular end plates to prevent damage to the column concrete. Both the column-to-footing and column-to-cap beam reached 10% drift without significant loss in lateral load carrying capacity. It was also found that the unbonded rebar in the column offers significant recentering and retain the residual displacement within 1% limit even after 10% drift. However, fracture of reinforcing bar was reported around 6-7% drift ratio for the connections and cover concrete damage was limited to minor cosmetic damage.

Wu and Pantelides (2017) proposed a rapid repair method of severely damaged earthquake bridge column-footing and column-cap beam connection via plastic hinge relocation. The cast in place connections were damaged ranging from core concrete crushing, rebar buckling and fracture of the buckled longitudinal rebar. The damaged plastic hinge was then repaired with an enlarged section having epoxy-anchored headed steel bars and a CFRP shell filled with non-shrink concrete. Experimental results confirmed successful relocation of the plastic hinge away from the column base and enhanced the strength, and stiffness of the column. Furthermore, the repair implementation was rapid in nature and achieved similar or enhanced deformation capacity compared to the cast in place connection. However, significant damage ranging from spalling to core concrete crushing was reported for the newly formed plastic hinge region. This indicates the inherent problem of the traditional repair system that tries to restore the rebar continuity and forces the
damage in other parts of the structure requiring further repair with more complex and expensive techniques.

Yang and Okumus (2017) investigated the performance of segmental posttensioned precast concrete bridges constructed with ultrahigh performance concrete (UHPC) and different reinforcing condition. The specimens were tested under two different loading protocols representing design basis and the maximum considered earthquake. It was found that the impact of the UHPC was insignificant under the design basis earthquake which was controlled by the shear slip at the segmental joints of the column. However, the MCE protocol showed that the use of UHPC resulted in minimal damage and achieved higher strength and stiffness when the shear slip was prevented. It was also found that the UHPC column with and without mild steel reinforcing bars were similar indicating no significant impact of the steel reinforcing used along with UHPC.

Andisheh et al., (2018) investigated the effect of corrosion on hysteretic metal axial dissipative devices used as external dissipaters for dissipative controlled rocking bridge pier system. A total of 36 dissipaters were corroded with three levels and were tested under cyclic loading with two different deformation rates. Experimental results show that the quality and properties decrease because of corrosion. The impact of corrosion was more pronounced on the maximum strain compared to the number cycles to failure or maximum stress. It was also found that the energy dissipation capacity degrades significantly with corrosion and impacts the maximum achievable damping of the dissipaters. Corrosion was also found to result in an instable cyclic hysteresis because of buckling in the axial dissipative devices.
Kurama et al., (2018) summarized the development of seismic resistant precast concrete system in the context of moment frames, structural walls, floor diaphragms and bridges. Different methods of connecting the column to cap beam and column to foundation was also reviewed for the bridges and the use of supplemental energy dissipating devices are discussed. The review led to the conclusion that widespread use of the precast structures in high seismic zone is feasible and low damage system with jointed ductile connection can be achieved for resisting seismic forces.

Liu (2018) raised the issue of low redundancy and lack of robustness in avoiding collapse under maximum credible earthquake for dissipative controlled rocking (DCR) type connection. It was also indicated that the connection needs to be resilient under repetitive earthquake or significant aftershock following major earthquake event. The author introduced three different modifications to the existing DCR concept i.e., use of two sets of dissipaters with hierarchical activation, use of multiple rocking interface along column length with segmental construction and combining foundation rocking with DCR connection. Large-scale experimental testing along with numerical and analytical investigation was conducted to draw the conclusion of effectiveness in employing the modified DCR connection in achieving redundancy and robustness.

Higgins et al., (2020) conducted full-scale experimental research using titanium alloy bars (TiABs) for seismic retrofit of deficient bridge columns that are representative of the pre-1970 construction in the state of Oregon. Results show that the titanium alloy bars were effective in confining the plastic hinge region of the columns where premature lap splice failure was observed for the as-built specimen. The retrofit method utilizing
titanium alloy bars were effective in increasing the displacement ductility capacity of the vulnerable bridge columns.

A summary of the experimental studies on low damage system for construction of new structures or repair/retrofit of the existing structural system is presented in ‘Appendix A’.

1.3.2 The Need for Research

Conventional repair methods such as concrete jacket, steel jacket, or FRP wrap are most effective for cases of low damage where the column longitudinal rebar remains intact. However, more involved procedures are needed for cases of loss of lap splice, buckled or fractured rebar or merely loss of confidence at the remaining low cycle fatigue capacity for subsequent earthquakes. Past research had focused on coupling the rebar in various ways for continuity prior to the encasement or wrap. While effective at restoring the column capacity, there are three significant issues with these approaches:

a) restoring rebar continuity is labor intensive resulting in lengthy and potentially costly repairs that makes it unsuitable for rapid repair approach,
b) the affected area can be damaged again in an aftershock requiring new significant repairs or relocate the plastic hinges in the column, and
c) repair may result in increased strength and stiffness that would likely shift failures to other parts of the bridge under future earthquake demands.

An alternative post-earthquake repair method is therefore required that can be rapidly implemented and that also has the potential for increasing the resilience for future shaking. Dissipative controlled rocking connection with replaceable U-shaped flexural
plate fuse elements has shown great potentials for the repair of existing timber structures. Expansion of the repair concept for earthquake damaged concrete bridges needs to be experimentally validated and proper design methodology needs to be developed.

1.4 Research Goals and Scope

The main objective of this research is to identify the steel and concrete strain limits to be considered for the seismic assessment of existing reinforced concrete bridge bents considering the operational performance design criteria. Multi-column bents are typical for bridges in Oregon and are therefore more representative of the need to gather performance related data. The proposed research is for experimentally evaluating large-scale reinforced concrete subassemblies representing critical parts of the bents. These primarily represent column-to-foundation aspects of the bent. Of key interest from the experiments are the monitoring of material strains and deformations as the column reaches target performance levels.

Another important objective of the research program was to devise and experimentally validate a rapid repair measure utilizing externally attached energy dissipating device. U-shaped flexural plates are considered to be the externally attached dissipating devices and a dissipative controlled rocking connection (DCR) is assumed to be the most suitable for the representative bridge substructure types. The key objective is to identify a suitable load transfer mechanism between the column and footing without rebar continuation and also to develop an analytical design tool for the rapid repair method. Furthermore, examining the key design aspects in terms of the experimentally obtained data is of the key interest.
The scope of the research is to revisit the definition of performance levels with respect to specific damage states and identify engineering demand parameters for individual performance levels. Experimental study consisted of six full-scale square RC bridge column-spread footing subassemblies representing the details found in old bridge (pre-1990 bridges) construction in Oregon. The column-footing subassemblies were tested under reverse cyclic loading using standard laboratory and subduction zone loading protocols. Finally, preliminary results of limit states for the performance-based seismic design are presented. The investigation focused on defining and quantifying structural performance and the corresponding limit state. The two performance levels required by ODOT were studied, namely Operational and Life Safety. Results from previous and current experiments were used to compare and assess the performance limit states in terms of material strains.

The scope for the second phase of the experimental program includes full-scale testing of an as built specimen repaired according to the rapid repair measures. The specimen was tested using the same axial and lateral loading protocol. Results obtained from the test were compared with the as built specimen to validate the efficacy of the repair method.

1.5 Research Significance and Contribution

Recent bridge designs, especially for retrofit, have highlighted the need to better understand performance criteria because those bridge designs had been governed by the operational performance limits, driving the cost of the retrofit. The hypothesis, based on limited data collected in past research at PSU and those sampled from laboratory
assessment of building columns, is that the current strain limits may be conservative for the type of detailing encountered in Oregon. Hence, experimentally validated strain limits obtained from this research can directly influence the retrofit costs. The mobility aspect is especially important as ODOT aims to maintain dedicated lifeline routes following CSZ earthquake. The changes in operational criteria from a lower hazard to explicitly consider CSZ as an operational performance level will increase the seismic demand considerations for operational criterial in certain parts of the state. This emphasizes the need to better understand the underlying assumptions. The experimental data will provide knowledge on existing bridge performance at varying levels of demand and as such could also inform post disaster inspection.

The main product of the research will be the development of performance criteria recommendations for structural seismic bridge design in Oregon. The standards that engineers could use to guide their design and further the development of BDM. Given the unique data that will be generated, the findings will be of interest to the profession as the field of structural engineering in general moves toward performance-based design. Furthermore, the development of a new repair method utilizing external energy dissipating device will not only add a tool to the existing techniques but also provide a new direction for resilient repair of the vulnerable bridges.

1.6 Dissertation Outline

The dissertation is categorized according to the two different stages of experimental studies. The first part describes the tests on as-built specimens, and the second part
describes the tests on the repaired specimen. A total of seven chapters are organized with the following contents –

Chapter 1 discusses the background and significance of the research programs along with the objective and scope of the current experimental program. Review of the literatures for both the first and second stages of the experimental studies are also presented in this chapter. Finally, the chapter concluded with the outline of the entire dissertation.

Chapter 2 presents the detail of representative bridges in Oregon obtained through statistical analysis of over 100 bridge drawings. The key parameters that were investigated includes bridge characteristics like number of spans, lanes, length of bridge and typical bent characteristics like columns per bent, height, geometric and reinforcing detail of the bent components etc. This chapter also includes a discussion on the footing type and details accompanying the typical bridge bent columns.

Chapter 3 discusses the development of the experimental program including the selection of test specimen and geometric and reinforcing details of the component of the specimen. Properties of the materials used, construction stages, test setup, and instrumentation are also discussed in this chapter. Furthermore, the selection of the lateral and axial loading protocol is also presented, and the motivations are also discussed.

Chapter 4 presents the data obtained from the experimental test of six full-scale as-built specimens. Details of the data manipulation are discussed at the beginning of this chapter. Later sections of the chapter present the experimental results such as physical damage observation, load-displacement response, curvature, and strain profile etc. Finally,
the chapter concludes with the comparison of the key test results with past experimental program at Portland State University.

Chapter 5 outlines the performance limit states and correlates the experimentally obtained strain values for each of the specimens with different damage states. A global strain limit state based on the experimental program is also discussed and is compared with the strain limiting values obtained from the past experimental program. Throughout this chapter, operational and life safety performance limit state are explicitly discussed in terms of concrete and rebar strain values.

Chapter 6 details the experimental program for the repaired specimen starting with the details and design of the repair method followed by test details and instrumentation of the repaired specimen. Finally, the results obtained from the test is presented and is compared with the performance of the as built specimen. Efficacy of the repair method is also discussed with respect to key design parameters.

Chapter 7 builds on the repair method outlined in the earlier chapter. This chapter investigate the analytical response prediction method outlined earlier with respect to experimental data obtained from another research program conducted at Portland State University. Finally, the chapter concludes with recommendation and revision to the presented analytical response prediction equations.

Chapter 8 is the last chapter in the dissertation that provides summary based on the results presents in the earlier chapters and also provide recommendation for future studies.
Chapter 2 Representative Bridge Details

2.1 Introduction

A large number of bridge inventory in Oregon were built in 1990’s prior to the development of the current seismic design code. As a result, the design and detailing of these bridges focuses only on the gravity load system with minimal consideration to lateral load path. This coupled with the increase in seismic demand due to the CSZ earthquake event has led to the renewed interest in gathering experimental data for these potentially vulnerable bridges. However, an accurate presentation of the laboratory test specimens with representative detailing and bridge characteristics first needs to be evaluated through surveying key structural parameter of the bridges from available drawings of existing bridges. The following section discusses the details of representative bridges in Oregon built before 1990.

2.2 Oregon Bridge Inventory

The National Bridge Inventory (NBI 2021) database provides information about all bridges in United States of America (USA) with a span length of more than 20 ft. This database allows for general classification of bridges based on different information contained in 134 fields, referred to as items, in the NBI. In NBI Item 43, bridges are classified based on superstructure materials (Table 2.1), predominant type of design and/or type of construction (Table 2.2), and number of spans. The coding guide for the Oregon Bridge Inventory and Appraisal serves as a guide to the NBI and lists superstructure materials and types of construction. Table 2.1 shows bridge classification based on the type of material used for bridge construction.
Table 2.1 Kind of material and/or design, NBI Item 43A (FHWA 1995)

<table>
<thead>
<tr>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
</tr>
<tr>
<td>Concrete Continuous</td>
</tr>
<tr>
<td>Steel</td>
</tr>
<tr>
<td>Steel Continuous</td>
</tr>
<tr>
<td>Prestressed Concrete</td>
</tr>
</tbody>
</table>

Table 2.2 shows bridge classification based on the type of design and/or construction practices.

Table 2.2: Type of design and/or construction, NBI Item 43B (FHWA 1995)

<table>
<thead>
<tr>
<th>Description</th>
<th>Slab</th>
<th>Truss – Deck</th>
<th>Movable – Swing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stringer/Multi-beam or Girder</td>
<td>Truss – Thru</td>
<td></td>
<td>Tunnel</td>
</tr>
<tr>
<td>Girder and Floor Beam System</td>
<td>Arch – Deck</td>
<td></td>
<td>Culvert</td>
</tr>
<tr>
<td>Tee Beam</td>
<td>Arch – Thru</td>
<td></td>
<td>Mixed Types</td>
</tr>
<tr>
<td>Box Beam or Girders – Multiple</td>
<td>Suspension</td>
<td></td>
<td>Segmental Box Girder</td>
</tr>
<tr>
<td>Box Beam or Girders – Single or Spread</td>
<td>Stayed Girder</td>
<td></td>
<td>Channel Beam</td>
</tr>
<tr>
<td>Frame</td>
<td>Movable – Lift</td>
<td></td>
<td>Other</td>
</tr>
<tr>
<td>Orthotropic</td>
<td>Movable – Bascule</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

According to the 2014 NBI database, there are 8052 bridges and culverts in the state of Oregon. Among these, 3877 are multi-span bridges and 1802 of these bridges are owned by state agency, state park, forest or reservation agency, or other state agency. However, the number of state highway owned multi-span bridges built before the year 1990 is 1539. This research focuses on state-owned multi-span bridges in Oregon built prior to 1990, which are listed in Table 2.3.

Table 2.3: Classification of multi-span bridges in Oregon built prior to 1990, based on NBI item 43

<table>
<thead>
<tr>
<th>Bridge Type</th>
<th>Number of Bridges 2014 Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestressed Concrete Slab (PC Slab)</td>
<td>116</td>
</tr>
<tr>
<td>Concrete Continuous Stringer/Girder (CC SG)</td>
<td>448</td>
</tr>
<tr>
<td>Prestressed Concrete Stringer/Girder (PC SG)</td>
<td>210</td>
</tr>
<tr>
<td>Wood or Timber Stringer/Girder (Wood SG)</td>
<td>43</td>
</tr>
</tbody>
</table>
The graphical representation of Table 2.3 is presented in Figure 2.1. It can be seen from Figure 2.1, concrete continuous stringer/girder bridges (CCSG) are the most common type of bridge in the state of Oregon.
2.3 Bridge Characteristics

The NBI provides different information about the bridges such as year built, number of traffic lanes carried on the structure, design load, maximum span length, deck width, condition rating of the structure etc. However, detailed drawings for each of the bridge classes are necessary to extract typical details for each bridge classes. Thus, the following part of this chapter focuses on the most common bridge characteristics for concrete continuous stringer/girder bridges (CCSG) bridges in Oregon built prior to 1990.

Probability mass functions (PMF) were generated for discrete variables to determine the most common characteristic. PMF is defined as the probability that a discrete random variable, X takes on a particular value x, or \( P(X = x) \). For continuous variables, cumulative distribution functions (CDF) were computed. CDF gives the probability that a continuous variable, X takes a value less than or equal to x, or \( P(X \geq x) \).
2.3.1 Number of Spans

For this case, the number of CCSG bridges with equal number of spans was grouped together and each group was divided by the total number of CCSG bridges. Figure 2.2 shows the PMF for number of spans.

![PMF for number of spans of CCSG bridges built prior to 1990](image)

Figure 2.2 PMF for number of spans of CCSG bridges built prior to 1990

As can be seen from Figure 2.2, more than 50% of the bridges have three spans. Hence, it can be assumed that the most representative CCSG bridge is a three-span bridge.

2.3.2 Number of Lanes

The PMF for the number of lanes on the structure was also generated and is presented in Figure 2.3. It can be observed from Figure 2.3, over 80% of the CCSG bridges have two lanes on the main unit of the structure.
2.3.3 Length of Bridge

The total length of the bridges can be found from the NBI database and an empirical cumulative distribution function (CDFs) for the total length of the bridges are useful to describe the distribution for bridge length of CCSG bridges. Figure 2.4 represents the CDF for the total length of the bridges. The average total length was found to be 176.5 ft with a standard deviation of 108.5 ft and a median value of 150 ft.
In addition to the cumulative distribution function, the range of length for CCSG bridges are presented in Figure 2.5, bridges with span length between 120 ft–140 ft. are the most frequent.

![Frequency plot for bridge length of multi-span CCSG bridges built before 1990](image)

Figure 2.5 Frequency plot for bridge length of multi-span CCSG bridges built before 1990

### 2.4 Typical Bent Characteristics

The individual components of a bridge bent consist of columns, foundation, and cross beam. Geometric and reinforcing details for all the components are significant for seismic performance evaluation of a bridge substructure system. Hence, a detailed analysis from 113 available bridge drawings for concrete continuous stringer/girder bridges were conducted to establish the most representative bent details. The following sub-sections present the results obtained from the statistical data analysis for typical bent characteristics.

#### 2.4.1 Number of Columns per Bent

The frequency of bridges with certain number of columns per bent as extracted from the available drawings for CCSG bridges are presented in Figure 2.6. It should be
mentioned that only the intermediate bents were taken into consideration for counting the number of columns per bent.

![Number of columns per bent for multi-span CCSG bridges built before 1990](image)

Figure 2.6 Number of columns per bent for multi-span CCSG bridges built before 1990

It was found that most of the CCSG bridges consist of multi-column bents as the substructure. Further investigation shows that almost 60% of all multi-column bridge bents have two columns per bent.

### 2.4.2 Column Height

Column height is one of the important parameters in this research as it greatly affects the sensitivity of the earthquake response for bridge structures. Unfortunately, column height data are not tabulated in the NBI database and were hence extracted from the available bridge drawings. In most cases, column heights for end bents and interior bents differ largely and are thus recorded separately. Figure 2.7 represents the frequency of CCSG bridges over a particular range of column heights for end and intermediate bents. The column heights for the end bents are not presented here as the typical configuration and detailing for intermediate bent columns are considered while extracting the data from
Chapter 2-Representative Bridge Details

the available drawings. However, the typical range of column heights for end bents are in the range of 15 ft – 20 ft. Also, it can be observed, typical heights for intermediate bents are within a range of 20 ft – 25 ft.

![Graph a](image1.png)

![Graph b](image2.png)

**Figure 2.7** Variation of column heights for multi-span CCSG bridges built before 1990 (a) for end bents and (b) for intermediate bents

### 2.4.3 Column Details

Column details (i.e., cross sectional dimensions, reinforcement detailing, splice length etc.) were extracted from the available bridge drawings in order to reproduce a
typical bent column for CCSG bridges in Oregon. From the data extracted, a typical bent for a CCSG bridge has a square concrete column with cross sectional dimensions of 24 in x 24 in. Most of the bridges have four longitudinal reinforcement bars in the column section. Further investigation shows that the majority of the columns with four longitudinal reinforcement bars have either #8 or #10 rebars as the longitudinal reinforcement. The transverse reinforcement is typically provided by #3 hoops (65% of the cases) with a center-to-center spacing of 12 in throughout the column height. The same longitudinal bars are spliced with the foundation reinforcement having a total length of 6 ft – 7 ft. The majority of the columns have a concrete cover of 2 in. The cross sectional and elevation drawings of a typical column is presented in Figure 2.8.
2.4.4 Splice Length

Most of the bridge columns have lap splices in the foundation-column joint region, which has a profound impact on the seismic performance of the bridges. Hence, a detailed screening based on the available drawings were conducted to investigate the variation of lap splice lengths for typical bridge columns. It was found that the splice length varies with the variation of longitudinal reinforcement diameter. The splice length variation for #8 and #10 rebars are reported here. Figure 2.9(a) shows the distribution of splice length.
multipliers (numerical multiplier with respect to diameter of the longitudinal rebar) for #8 rebar and Figure 2.9(b) shows the cumulative percentage distribution for the splice length variation.

![Figure 2.9 (a) Variation of splice length multiplier for #8 splice bars and (b) Cumulative percentage changes for splice length multiplier](image)

Figure 2.10 shows the splice length and the cumulative distribution function for #10 rebars. It was found that 56% of the bent with #10 rebars had a splice length of either
less than or equal to 47 times the bar diameter. Similarly, for #8 rebars the splice length multiplier of \(47d_b\) was associated with cumulative percentage of 48.

![Histogram of Splice Length Multiplier](image1)

![Cumulative Percentage Changes](image2)

Figure 2.10 (a) Variation of splice length multiplier for #10 splice bars and (b) Cumulative percentage changes for splice length multiplier

### 2.4.5 Foundation Details

The majority of multi-span CCSG bridges with square concrete columns have a square/rectangular spread footing as the foundation. The cross-sectional dimensions of the footing ranges from 6 ft – 8 ft (square spread footing) and for more than 50% of the cases
the depth of the footing is 2 ft. Reinforcement detailing for the spread footing consists of only one bottom layer of reinforcement (in two directions) consisting of #4, #5, or #6 reinforcing bars. However, #5 reinforcing bars are most common and an investigation for the spacing of reinforcement with #5 rebar shows that majority of the foundations have a spacing of 6" – 7" (over 50% of the cases) in both directions. Figure 2.8 shows the elevation view of the foundation and the plan view is presented in Figure 2.11.

![Figure 2.11 Plan view of a typical foundation reinforcing detail](image)

### 2.4.6 Crossbeam Details

Typically, a cross beam has a sectional depth of about 5-6 ft and a width of 1.25 ft - 2 ft. The longitudinal reinforcement used typically consists of #9, #10, or #11 rebars separately or in combination with each other. Most of the cross beams have #4 or #5 U-
stirrups with a spacing of 6" – 9" near the column support and 12" – 15" in the mid-section. The concrete cover for the beam section is 2 in for the majority of cases. However, it should be mentioned that defining a typical cross beam section is not justified because of the large variability of their cross-sectional detail.

2.5 Summary

A statistical analysis of available data from the National Bridge Inventory (NBI) and the Oregon Department of Transportation (ODOT) were conducted to establish typical details for Oregon bridges built prior to 1990. Concrete Continuous Stringer/Girder (CCSG) bridges were found to be the most common bridge type in the state of Oregon. Further investigation established typical characteristics, i.e., number of spans, number of lanes on the bridge, length of bridges, etc. for CCSG bridges. It was found that a typical CCSG bridge has three spans with an average span length between 120ft – 140ft. In addition, the main unit of the structure usually has two lanes on the bridge.

A total of 113 bridge substructure drawings available from ODOT were studied to establish typical bridge bent characteristics including number of columns per bent, column height, and geometry and reinforcing details for different components of the substructure, i.e., column, foundation, and cross beam. Multi-column bridge bents are the most common type with an average column height of 15ft – 25ft. Finally, sectional details and other design specific parameters were investigated and typical bent details for the most common bridge substructure components in the state of Oregon were developed. It was found that a square reinforced concrete column with a square spread footing having a single layer of reinforcement at the bottom is most common. Most of the columns have a lap splice
reinforcing arrangement in the plastic hinge zone with a moderate splice length, which makes it potentially substandard for earthquake-type loadings. In addition, the transverse reinforcement spacing lacks the seismic detailing requirements as per current seismic design practice.
Chapter 3  Full-Scale Experimental Program

3.1  Introduction

The following sections describe the experimental program for the tested reinforced concrete bridge column-foundation subassembly subjected to reverse cyclic lateral loading. Test setup, specimen detail, and construction of specimens are presented as well. Finally, specimen external and internal instrumentation along with the loading protocols used for the study are presented. The experimental program was devised to study the behavior of a full-scale reinforced concrete bridge column-footing subassembly, measure local and global response quantities, and provide experimental evidence of different damage levels under longer duration shaking expected from a CSZ event.

3.2  Test Specimens

The reinforced concrete subassemblies representing critical part of the representative bridge bent primarily represent column-to-foundation and column-to-cross beam aspect of the bent. Past research on lightly reinforced existing bridge columns indicated that the plastic damage was concentrated at the column base near the column-foundation interface region (Bazaez and Dusicka 2016b; Lopez et al. 2020; Mehary et al. 2018). In addition to lightly reinforced non-seismically detailed bridge columns, existing bridge bent often use lightly reinforced spread footing for gravity load resistance. Lack of joint reinforcement at the interface and existence of different surface roughening at the end of column potentially led to a cold joint at the column-footing. However, no experimental data to date currently exists that considers the full-scale column and existing spread footing.
details having typical details of the representative bridges in Oregon. The overall behavior of the column-to-footing subassemblies and the influence of non-seismically detailed column-to-footing joint on the behavior of existing non-seismically detailed columns is of key interest. Hence, the column to spread footing portion of the bent was chosen for this study. A total of six reinforced concrete column-to-spread footing subassemblies representing full-scale details of a representative multi-column bridge bent built prior to 1990 were constructed and tested.

The construction of the specimens was done in two separate phases due to space limitation in the laboratory. The first phase of construction mentioned herein as ‘Phase-I’ consisted of three specimens with identical geometry and reinforcing details. The second phase of construction mentioned as ‘Phase-II’ consisted of three more full-scale specimens with same geometric details but with different reinforcing details. The geometry and reinforcing details of all the three column-footing subassemblies from Phase-I specimens represents the critical portion of a typical multi-column bridge bent of concrete continuous stringer/girder bridges in Oregon. The 24-inch square column was reinforced with 4-#8 longitudinal rebar on the four corners. Column longitudinal reinforcement bars were extended to the column-footing joint interface and were developed into the footing through a lap splice with #8 dowel bars. The lap splice length for the dowel bars starting from the column-footing joint was 47d_b. The spliced bars were developed up to the top of the footing reinforcement with a 90-degree bend at the end and then extended 12 times the rebar diameters.
All the three specimens in the Phase-II construction were also of 24-inch square column with a height of 102.5 inches. Two of the three specimens were constructed with 4-#8 longitudinal rebar with either lap splice in the plastic hinge zone or continuous longitudinal rebar throughout the column length. The remaining specimen of the series was constructed with 4-#10 longitudinal rebar having a lap length of $47d_b$ in the plastic hinge zone. The reinforcing detail of the six specimens constructed are presented in Table 3.1 below.

<table>
<thead>
<tr>
<th>Construction Phase</th>
<th>Specimen Name</th>
<th>Long. Rebar</th>
<th>Dowel Rebar</th>
<th>Splice Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase-I</td>
<td>SC</td>
<td>4-#8 spliced</td>
<td>4-#8</td>
<td>$47d_b$</td>
</tr>
<tr>
<td></td>
<td>SV</td>
<td>4-#8 spliced</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>LV</td>
<td>4-#8 spliced</td>
<td>4-#8</td>
<td>25$d_b$</td>
</tr>
<tr>
<td>Phase-II</td>
<td>MS#10</td>
<td>4-#10 spliced</td>
<td>4-#10</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>CR#8</td>
<td>4-#8 continuous</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>SS#8</td>
<td>4-#8 spliced</td>
<td>4-#8</td>
<td>25$d_b$</td>
</tr>
</tbody>
</table>

Naming convention used for the specimens are different for the two different construction phases. For the first phase of construction test variables considered was either the axial loading protocol or the lateral cyclic loading protocol. Hence, the name of the three specimens tested in the first phase started with either ‘S’ or ‘L’, where ‘S’ represents the CSZ cyclic loading protocol and ‘L’ represent the conventional three cycle symmetric laboratory loading protocol. The second letter in the naming conventions are after the axial loading protocol which was either ‘C’ that indicates a constant axial load level and ‘V’ indicating a variable axial loading protocol. For example, specimen SC was tested under CSZ lateral cyclic loading protocol and a constant axial loading throughout the test.
duration. On the other hand, variables considered in the second phase of testing was the reinforcing detail while the axial and lateral loading protocol was kept same for all the three specimens. The two-part naming convention for the specimens in Phase-II consists of splice length details where ‘MS’ represents moderate laps splice length (47$db$), ‘CR’ represents the specimen with continuous longitudinal rebar and finally ‘SS’ represents the specimen with short lap splice length (25$db$). The second part of the name followed the hashtag indicates the diameter of the longitudinal rebar used. Details of the test matrix are discussed in greater detail in the later sections in this chapter.

The transverse reinforcement consists of #3 square tie bars with 90-degree hooks at both ends with an extension of 10 times the diameter of the tie bars (4½ inch), constant for all the six specimens. The concrete clear cover from the external face of the tie bars to the face of the column cross section was 2 inches for all specimens. The first tie at the bottom of the column region is located 6 inches from the column-footing joint, with subsequent ties spaced at 12 inches. However, the tie bars in the top 19¼ inch of the column were spaced at only 3 inches to provide better concrete confinement of the core concrete and hence to negate any possibility of concrete crushing due to close proximity of axial load application over the top of the column. The hooks of the tie bars were placed in opposite corners alternatively to prevent weak spots in any column corner.

The 6 x 6 ft square footing were also same for all the six specimens and with only a bottom layer of reinforcement in two orthogonal directions with a concrete clear cover of 3 inch from the face of the bottom layer reinforcement. All the bars used for the footing reinforcement are #5 bars and are spaced at 6½ inch in both directions. The deformed bars
were laid flat in the foundation without any hook or any extension upward. Details of the column-footing sub-assemblies are presented in Figure 3.1 for specimen with #8 rebar and with 47d₀ lap splice length.

Figure 3.1 (a) Column-foundation longitudinal section details (b) Column cross sectional details (Section A-A), (c) Tie bar details, (d) Foundation cross section details

Geometry and reinforcing details of the rest of the specimens are presented in ‘Appendix B’ at the end of the document.
3.3 Material Properties

The specimens were built with normal weight concrete and Grade 60 deformed steel bars. Although, use of Grade 40 deformed bars was most common construction practice for the representative bridges, the production of Grade 40 deformed bars is now limited to a maximum diameter of #5. Hence, ASTM A615 (2018) Grade 60 deformed steel reinforcing bars were used for the construction of the specimens. Furthermore, the use of Grade 60 rebar were also found for existing bridges built prior to 1970 (Dagenais et al. 2018). However, the use of Grade 60 rebar will increase the demand in the vulnerable spread footings causing the most unfavorable situation for the already vulnerable spread footing. The reinforcing bars were tested under uniaxial tensile loading according to ASTM A370 (2019) standard. The test results are summarized in Table 3.2.

<table>
<thead>
<tr>
<th>Construction Phase</th>
<th>Bar Size</th>
<th>Bar Type</th>
<th>Yield Strength (ksi)</th>
<th>Tensile Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase-I</td>
<td>#8</td>
<td>Column Long. &amp; Dowel</td>
<td>69.4</td>
<td>96.5</td>
</tr>
<tr>
<td></td>
<td>#5</td>
<td>Footing Rebar</td>
<td>70.7</td>
<td>108.6</td>
</tr>
<tr>
<td></td>
<td>#3</td>
<td>Column Ties</td>
<td>75.6</td>
<td>104.7</td>
</tr>
<tr>
<td>Phase-II</td>
<td>#10</td>
<td>Column Long. &amp; Dowel</td>
<td>75.4</td>
<td>111.0</td>
</tr>
<tr>
<td></td>
<td>#8</td>
<td></td>
<td>71.5</td>
<td>100.6</td>
</tr>
<tr>
<td></td>
<td>#5</td>
<td>Footing Rebar</td>
<td>68.1</td>
<td>97.6</td>
</tr>
<tr>
<td></td>
<td>#3</td>
<td>Column Ties</td>
<td>68.0</td>
<td>97.7</td>
</tr>
</tbody>
</table>

Concrete mix was based on historic specifications that targeted an expected 28-day nominal concrete compressive strength of 3300 psi. Ready mix concrete was used for the construction with maximum aggregate size of ¾ inch, 4-inch slump, and water cement ratio of 0.47. All specimens were constructed at the same time and in two phases, starting with footing construction followed by column construction. The concrete compressive strength
for individual concrete batch was determined through standard cylinder tests following 
ASTM C39 (2020) and tested at 7, 21 and 28 days, as well as at or near the day of each 
individual test. A total of three samples were tested and the summary of the concrete 
compressive strength test results are shown in Table 3.3.

<table>
<thead>
<tr>
<th>Construction Phase</th>
<th>Component</th>
<th>28-Day Compressive Strength</th>
<th>Test Day Compressive Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Average (psi)</td>
<td>C.o.V (%)</td>
</tr>
<tr>
<td>Phase-I</td>
<td>Column</td>
<td>4740</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>Footing</td>
<td>5410</td>
<td>6.3</td>
</tr>
<tr>
<td>Phase-II</td>
<td>Column</td>
<td>3410</td>
<td>6.3</td>
</tr>
<tr>
<td></td>
<td>Footing</td>
<td>3910</td>
<td>10.4</td>
</tr>
</tbody>
</table>

Concrete cylinder compressive strength vs. age of concrete cylinder from the first 
phase of testing is plotted in Figure 3.2(a) and (b). The 28-day compressive strength for 
column and footing concrete were 4,740 psi and 5,410 psi, respectively. It can be seen from 
Figure 3.2 (a), 7-day strength of both the column and footing concrete were comparable (≈ 
3300 psi) and was almost the same as the expected 28-day nominal concrete compressive 
strength. However, a linear increase in compressive strength for both the column and 
footing concrete was observed up until the 28-day strength. It can also be observed that the 
28-day compressive strength of the footing concrete was higher than the one for column 
concrete. This can be attributed to the different batches of concrete mix design and 
variations in water content between footing and column concrete. However, post 28-day 
strength gain rate for the column concrete was almost twice the one for footing concrete.
3.4 Specimens Construction

All specimens were constructed in the iSTAR Laboratory at Portland State University and caution was taken to replicate the exact prototype column-footing subassemblies as described in earlier sections. However, formwork for one of the specimens failed in the bottom corner during casting, which resulted in a slightly larger
cross-sectional area in that corner of the specimen. After removing the forms, reinforcement alignment and position of the transverse reinforcement were checked and found to comply with the detailing of the column specimen. Hence, only the concrete clear cover was found to have increased due to the opening of the formwork in that corner. The construction sequence of the specimen includes formwork preparation, rebar cage construction, instrumenting the rebar with foil strain gages, placing the rebar cages in place, concrete casting and finally removing the forms off the specimens.

The two phase of testing consists of three specimens in each phase. The three specimens in each phase of testing were again constructed in two phases, where the first phase started with the construction of footing elements. All the three footing specimens were cast at once and hence the same concrete mix was used. The later phase of construction consists of column casting. Column dowel rebars for specimens with lap splices were positioned in place prior to the casting of footing concrete. The longitudinal rebars were later spliced with dowel bars before the casting of column concrete. All of the longitudinal rebars for specimen with continuous bar was positioned prior to the footing concrete casting. Footing surface underneath the column area was roughened in order to facilitate bonding between the column concrete and the previously casted hardened footing concrete. However, a cold joint as a result of phased construction resulted between the interface of column and the footing. Properties of the concrete for each phase of casting was measured through slump test and concrete cylinder testing. The sequence of specimen construction is presented in ‘Appendix-C’.
3.5 Test Setup

The quasi-static reverse cyclic lateral load was applied at the column top (Figure 3.3) using a servo-controlled hydraulic actuator with a maximum capacity of 220 kip in tension and 335 kip in compression. The maximum stroke length of the lateral actuator is ±10 inch. The lateral reverse cyclic load was applied in displacement-control mode. Built-in load cells and displacement transducers were used to monitor applied load and displacement of the lateral actuator during the test. The column top was free to rotate and translate while undergoing the lateral displacement cycles.

Constant and variable axial loads were applied to the column to simulate the self-weight of the superstructure. The axial load was applied using a lever arm concept where the initial force was generated with an actuator oriented in vertical position (Figure 3.3). The servo-controlled vertical actuator, having a maximum capacity of ±100 kip and a maximum stroke length of ±6 inch, was used for generating the axial force. In order to maintain the accurate load values, the vertical actuator was operated in force-controlled mode.

In addition to the lateral and vertical actuators, the test setup comprised of several different components, including a reaction frame supporting the lateral actuator, load transfer beams, and tension anchor rods. The detailed test setup is illustrated in Figure 3.3.
The reaction frame consisted of two column (W18x97) section resting on two floor beams (W18x97). The reaction frame column has holes spaced every 12 inch to facilitate the lateral actuator connection through a crossbeam. Two steel HSS diagonal braces were placed on the reaction frame as part of the load transfer mechanism. The floor beam of the reaction frame was post-tensioned and anchored to the laboratory strong floor. The lateral actuator was connected to the center of a cross beam, which was connected to the reaction frame column using threaded rods. In addition, the lateral actuator was vertically supported with a temporary wooden frame (not shown in Figure 3.3).

The test setup was designed to have the ability to vary the axial load in the column during the test for two of the specimens. The vertical actuator was placed on its own floor beam (W12x45), which was anchored to the strong floor and post-tensioned. Two Hollow
Steel Section (HSS) of 10-inch height were placed between the floor and the beam to provide additional space for the vertical actuator connection with the floor beam. The vertical actuator was connected with the floor beam through four 1½ inch diameter threaded rods. The test setup was also designed to overcome the capacity of the vertical actuator using a lever arm system. The details of the lever arm in the test setup are illustrated in Figure 3.4.

Figure 3.4 Illustration of lever arm arrangement for axial loading

The vertical actuator used for the application of axial load in the specimen column was limited to a maximum of ±100 kip. However, the maximum axial load on the column was determined to be 240 kip, which is significantly higher than the capacity of vertical actuator. Within the lever arm system, the center of the column-foundation specimen was positioned at 2L/3 from the vertical actuator and the opposite side was restrained with threaded rods at L/3 distance from the column center point. The column top was acting as the fulcrum point for the lever arm system. Hence, the following maximum forces could exist in the setup:
Vertical actuator tension force, $R_a = +80$ kip

Reaction (tension) force in threaded rods, $R_{tr} = \frac{80 \times 2L/3}{L/3} = +160$-kip

Axial compression load in column, $P_{\text{column}} = R_a + R_{tr} = -80$ kip - 160 kip = -240-kip

The lever arm consisted of a complex beam arrangement as shown in Figure 3.5. “Axial Beam (W14x145)” was mounted on top of the column and connected with the lateral actuator in the south end. In addition, the north end of the “Axial Beam” was connected to the vertical actuator at the south end with threaded rods. A beam parallel to the axial beam was mounted on top of the “Axial Beam” referred to as “Parallel Beam (W14x132) and was used to provide the necessary lever arm length. Steel spacer plates of 1-3/4-inch thickness was used on the compression side and wooden lumber on the tension side of the “Parallel Beam” to provide sufficient clearing for the lateral actuator during testing. Once a tension force was applied using the vertical actuator, the two restraint ends of the “Parallel Beam” were in tension whereas the center (at the point of the steel spacer blocks) in compression. In order to achieve composite action between the “Parallel Beam” and the “Axial Beam”, the north end of the “Parallel Beam” was restraint with a built-up section and four threaded rods (two on each side). Figure 3.5, illustrates free body diagrams of the lever arm components. It can be observed that the threaded rods restraint end experiences twice the load generated by the vertical actuator. Hence, at any instance of time, the column top experiences three times the load generated by the vertical actuator which is again the summation of the force generated by vertical actuator and the resultant resistance at the threaded rods end.
Figure 3.5 Beam arrangement for variable axial load application: Plan view, Elevation view, and Static equilibrium of the system.
3.6 Test Matrix

The experimental program was designed to investigate the performance limit states of substandard square RC bridge columns commonly found in the state of Oregon. Representative bridge bent details were formulated for the experimental research program and the test matrix was then developed based on the variables considered. The test matrix as shown in Table 3.4 consists of six full-scale bridge column-foundation subassemblies. The variables considered for the test matrix were (1) lateral displacement history, (2) axial loading history (3) splice length, (4) rebar content and bar diameter and (5) presence or absence of lap splice in the plastic hinge zone. Specimens were tested in two phases and a different designation were used for the different test phases. The three specimens tested in the first phase of experiment focuses on the effect of loading history (both axial and lateral) and are designated with two letters. The first letter starts with either S or L, designating the lateral loading protocol (S – Subduction Zone Lateral Loading Protocol and L – Standard Laboratory Loading Protocol). The second letter of the nomenclature designates the axial loading protocol which is either C (constant) or V (variable). The first specimen tested in ‘Phase I’ is designated as ‘SC’ indicating that the specimen was tested under Cascadia Subduction lateral loading protocol and with a constant axial load level. The variable considered for specimen SC (Test 1) and SV (Test 2) was the variable axial loading protocol whereas the variable considered between specimen SV (Test 2) and LV (Test 3) was the lateral loading protocol. Specimen geometry and reinforcing detail for ‘Phase I’ testing was same for all the three specimens. In ‘Phase II’ of the testing the objective was to investigate the effect of lap splice length and the rebar diameter and/or steel content. The
specimens were designated with two letters followed by a hashtag and a number indicating the size of the rebar. The first specimen in ‘Phase II’ testing is designated as ‘MS#10’ (Test 4) where ‘MS’ stands for the medium splice length (47d₀), and the number followed by the hashtag indicates the longitudinal rebar was #10 rebar (1.25” diameter). Similarly, specimen ‘CR#8’ indicates continuous longitudinal rebar without any lap splice with #8 (1.00” diameter) rebar. Finally, the last specimen in the series ‘SS#8’ indicates short lap spliced (25d₀) longitudinal reinforcement with #8 (1.00” diameter) rebar.

The geometry and reinforcing details for the footing element was same for all the six specimens in the two different phases. However, the properties of concrete were different for different phases as the specimens were casted with two different batches of concrete. Similarly, the geometric details of the column like cross sectional dimension and the height were also same for all the six specimens tested.

<table>
<thead>
<tr>
<th>Testing Phase</th>
<th>Test No.</th>
<th>Test Name</th>
<th>Long. Reinforcement</th>
<th>Tran. Reinforcement</th>
<th>Lateral Loading</th>
<th>Axial Loading</th>
<th>(P/f_c<em>A₀)</em></th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase I</td>
<td>Test 1</td>
<td>SC</td>
<td>4#8-47d₀ spliced</td>
<td>3@12” C/C</td>
<td>CSZ</td>
<td>Constant</td>
<td>7.1%</td>
</tr>
<tr>
<td>Test 2</td>
<td>SV</td>
<td></td>
<td></td>
<td></td>
<td>CSZ</td>
<td>Standard</td>
<td>Min-4.7%</td>
</tr>
<tr>
<td>Test 3</td>
<td>LV</td>
<td></td>
<td></td>
<td></td>
<td>CSZ</td>
<td>Variable</td>
<td>Max-7.1%</td>
</tr>
<tr>
<td>Phase II</td>
<td>Test 4</td>
<td>MS#10</td>
<td>4#10-47d₀ spliced</td>
<td></td>
<td>CSZ</td>
<td></td>
<td>Min-6.9%</td>
</tr>
<tr>
<td>Test 5</td>
<td>CR#8</td>
<td></td>
<td>4#8-continuous</td>
<td></td>
<td>CSZ</td>
<td></td>
<td>Max-10.4%</td>
</tr>
<tr>
<td>Test 6</td>
<td>SS#8</td>
<td></td>
<td>4#8-25d₀ spliced</td>
<td></td>
<td>CSZ</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Nominal axial load ratio was calculated based on average test day compressive strength of the column concrete.

3.7 Instrumentation

The system and local behavior of the test specimens were captured using both external and internal sensors. The key global response quantities measured included the lateral column top displacement, column and foundation rotations, foundation uplift and
sliding, applied lateral load, and applied vertical loads. The following subsection summarizes the details of the instrumentation employed during the laboratory tests.

3.7.1 Column Instrumentation

The column was instrumented with several displacement transducer and strain gages to monitor the global and local response of the column specimen. The global response was monitored using displacement sensors connected to the column. Lateral load and displacement imposed by the lateral actuator was recorded using the load cell and LVDT attached to the actuator. Column top displacements were measured with a string potentiometer (string pot) attached to a fixed reference frame. Since the string pot was recording absolute column top displacements, this measurement was not influenced by any relative movement of the test setup. However, the foundation was not anchored to the floor and was allowed to rotate, slide, and uplift, and hence the measured top displacement was post-processed to exclude the column top displacement resulting from footing rotation due to uplift and sliding in the direction of loading.

The lateral displacement recorded by the actuator LVDT was initially used to set the magnitude of the loading protocol displacement history. Again, column top displacements did not necessarily match the magnitude targeted in the loading protocol. Thus, to maintain the intended lateral loading protocol, the displacement history was updated based on the displacement recorded by the string pot attached to the column top. It is noted that the column top displacement resulting from footing uplift was not excluded from the lateral loading protocol during the test and hence any contribution of the footing
rotation resulting from uplift was included in the lateral displacement protocol. Figure 3.6 shows the instrumentation details employed for column global response measurements.

In addition to the string potentiometer, seven curvature LVDTs were mounted on each side of the column as illustrated in Figure 3.6. A smaller spacing was chosen for the plastic hinge region (bottom section) where the most significant damage was anticipated. The last LVDT was placed on top of the column section to capture average curvature for the top two thirds of the column, which was expected not to undergo any significant damage. Threaded steel rods of 5/16 inch diameter were embedded into the concrete core to facilitate the mounting of the LVDT’s. Curvature rods of 36-inch length were cut in half.
and were inserted in two opposite sides along the centerline of the column as shown in Figure 3.6. Two thirds of each 18-inch-long curvature rod was embedded and bonded into the concrete while the rest was sticking out from the face to facilitate mounting of LVDTs. L-shaped aluminum brackets were used to attach LVDTs with curvature rods. These LVDTs measure the relative vertical displacement between two curvature rods from which the section rotation can be computed. Finally, average curvature between two rods was calculated by dividing the calculated average rotation with the gage length, i.e., the vertical distance between rods.

Strain gages were used to measure strain in both longitudinal and dowel bars at critical locations. In addition, three strain gages were placed at the transverse reinforcement to measure the strain of the tie bars resulting from cyclic displacement demand. Out of the four longitudinal and dowel bars, only two on each side were instrumented with strain gages at select locations. A total of 10 strain gages were mounted on the dowel bars, thus each bar having five gages. A total of 12 gages were placed on the longitudinal reinforcement. Locations are shown in Figure 3.7. Strain gages along the base of the column were closely spaced to have accurate resolution in the plastic hinge zone. Two of the dowel strain gages in each bar were placed in the column-footing to capture the strain profile within the footing. The first three transverse tie bars starting from the base were instrumented with similar strain gages at the center. For ‘Phase II’ testing the strain gages were also placed in the dowel bars at the column-footing interface or at zero distance from the footing top surface.
3.7.2 Footing Instrumentation

The test specimen consists of a spread footing and column subassembly. Experimental evaluation of seismic performance of the bridge components often focuses only on the behavior of column elements and hence the footing is constructed as a capacity protected member to facilitate the application of lateral loading. In these cases, the footing is usually tied to the strong floor to prevent any deformation (i.e., sliding, rotation, uplift etc.). However, the current experimental program focuses on the behavior of the column-spread footing subassembly. Hence, the spread footing was instrumented with external and internal sensors to capture the global and local deformation response quantities. Details of the external and internal instrumentation used for the spread footing alone are discussed in following subsections.
The foundation of the specimen rested on the laboratory floor and was allowed to rotate and translate. In order to quantify the movement of the foundation during a test, its uplift and sliding were measured. Along with these, LVDTs were placed on the footing top surface to measure relative deformation to compute concrete compression and curvature demand in the foundation. Figure 3.8 shows the foundation instrumentation details. Two LVDTs were placed vertically and attached at the mid-length of the foundation north and south face to measure any potential uplift. In order to measure the sliding of the foundation, three LVDTs were positioned in the horizontal direction on east, west and north faces. All three LVDTs reacted against a reference aluminum angle section, which was glued to the floor and hence allowed a relative measurement of sliding with respect to the laboratory strong floor. In addition, five LVDTs were used to measure the foundation’s extreme surface concrete compression and curvature along the direction of lateral loading. These LVDTs were also positioned in the horizontal direction to measure the relative displacement for a certain gage length. To fix these LVDTs, five zinc-coated steel threaded rods of 5/16 inch diameter were embedded 6 inches into the foundation prior to concrete casting. The first threaded rod was placed along the center of the foundation length at a distance of 3 inch from the column face. The next threaded rod was placed at 12 inch (parallel to the column edge) and the next three at 5-inch intervals as shown in Figure 3.8.
In order to measure the foundation rebar strain in the longitudinal and transverse direction of loading, several strain gages were placed on select foundation rebars. The locations are presented in Figure 3.9. Four strain gages were placed along the direction of lateral loading in the rebar running through the center of the foundation. The lateral load was applied in the N-S direction for all test specimens.
3.8 Loading Protocol

Two different types of cyclic lateral deformation protocols were utilized to assess the seismic performance of the column-footing subassemblies; an increasing amplitude protocol with three-cycle per amplitude as used in past research (Goodnight et al. 2016a; Mehary et al. 2018), and a cyclic protocol developed to capture the number and amplitude of cycles expected from a subduction earthquake (Bazaez and Dusicka 2016). The more traditional three cycle per amplitude loading protocol was divided into stages, starting with elastic cycles at $0.25\delta_i$, $0.50\delta_i$ and $0.75\delta_i$ where $\delta_i$ is the analytically predicted yield
displacement as obtained from monotonic moment curvature analysis of the column section. The elastic loading cycle concluded with one cycle at the analytically predicted yield displacement \((1.0\delta_i)\). The average of lateral force recorded during the push and pull cycle at \(1.0\delta_i\) was then used to determine the experimental effective yield displacement, \(\Delta_y\). Experimental effective yield displacement, \(\Delta_y\) was further used to calculate the displacement ductility and the subsequent loading cycles consists of three cycles at each increment of displacement ductility starting with \(1.0\Delta_y\). The symmetric three-cycle set loading history commonly known as conventional/standard laboratory loading protocol used is presented in Figure 3.10(a).

Contrary to the conventional laboratory loading protocol, the deformation protocol representative of subduction zone earthquake demands proposed by Bazaez & Dusicka (Bazaez and Dusicka 2016) was used to reflect the potentially increased damage in bridge columns that could result (Lopez et al. 2020; Mehary et al. 2018). This deformation protocol is characterized by higher number of cycles at the low ductility levels and lower number of cycles at the higher ductility level. The loading protocol consisted of two stages where the first stage included three nominally elastic cycles, each having displacements corresponding to at \(0.25\delta_i\), \(0.5\delta_i\) and, \(0.75\delta_i\) followed by one cycle at \(1.0\delta_i\) to capture the initial damage such as first cracking and the progression of cracks. The second stage of the protocol consisted of inelastic displacement cycles corresponding to increasing levels of ductility denoted by the solid lines in Figure 3.10(b). The subduction protocol targeted displacement ductility of \(\mu=8\) and a fundamental period of 0.5 sec which was found to be
representative of multi-column reinforced concrete bridge columns (Bazaez and Dusicka 2016).

The column-footing subassembly specimens used for the experimental program represent the critical part of existing multi-column bent where the column experience variation in axial load owing to the overturning moment resulting from the horizontal earthquake forces. Hence, an accurate representation of the axial load condition for seismic performance evaluation of existing bridge column requires a varying axial load protocol. Moreover, experimental investigation shows that the deformation capacity and flexural strength of reinforced concrete column was different under constant and varying axial loading protocol and the axial load path history significantly affect the flexural capacity of the column (Esmaeily and Xiao 2005). It was also found that the variation in axial load resulting from overturning moment is typically proportional and correspond to the direction of the applied lateral load (Esmaeily-Gh and Xiao 2002).

Two different axial loading protocols i.e., constant, and varying proportional to lateral loads were considered for the experimental program with an aim to investigate the effect of axial load variation on seismic performance of existing column-footing subassemblies. The design axial loads were obtained from the available drawings for existing bridges and were found to vary between 50 kip to 275 kip for a typical CCSG bridge. The average axial load was 130 kip with a standard deviation of 48.5 kip. An axial load of 240 kip representing slightly higher than the average design axial load plus two standard deviation value was then considered for the constant axial loading protocol. The axial load ratio \( P/f'_{\text{c}}A_g \) for the first phase of testing was calculated as 7% where the test day concrete
compressive strength, $f'_c$ of 5,850 psi) and a gross cross-sectional area of 576 in$^2$ was considered. The varying axial load utilized the same target axial load ratio of 7% for the maximum value and then varied to a minimum nominal load ratio of approximately 5%, corresponding to 240 kip and 160 kip respectively. The maximum variation in axial load in a column of a two-column representative bridge bent resulting from the overturning moment was found to be 40 kip. A base axial load of 200 kip was selected for the varying axial loading protocol and the variation from the base value was proportional to the lateral strength at each ductility level. The axial loading protocol is illustrated in Figure 3.10(c). The constant axial load of 240 kip was applied at the beginning of the test for constant axial loading protocol whereas the base axial load of 200 kip was initially applied for the specimens tested under varying axial load. The variation in axial column load was then implemented simultaneously alongside the application of lateral loading protocol.

![Diagram](image-url)
3.9 Summary

Chapter 3 presents details of the experimental program including test matrix, specimen construction, test setup, test variables studied, instrumentation, and loading protocols. The specimen details including geometry, reinforcing details, and loading conditions are established for the experimental program. Typical bridge bent details were first developed through statistical analysis of available bridge drawings and later used to define the test specimen.

A total of six full-scale specimens’ representative of bridge substructure subassemblies were constructed in the iSTAR Laboratory at Portland State University. A cantilever square reinforced concrete column with either 4 #8 or 4 #10 longitudinal rebar having a lap splice in the plastic hinge zone was selected for five of the six specimens. Where one specimen was constructed with 4 #8 continuous longitudinal reinforcement without any lap splice. A square spread footing with single layer of #5 rebars spaced at 6½ inch was selected as the foundation supporting the square column. These test specimens
were incorporated into a two-phase test matrix designed to investigate the effect of load variation, splice length and steel content. A unique experimental setup was developed to facilitate the investigation of the variables considered. Local and global instrumentation including external LVDT’s, and strain gages were placed to capture different deformation quantities.

Two sets of lateral loading protocols were considered for the test program, namely a subduction zone lateral loading protocol and a conventional three-cycle symmetric laboratory loading protocol. The standard laboratory loading protocol consisted of three cycles at each displacement ductility whereas the long duration subduction zone loading protocol mimics the higher displacement demands imposed by a full rupture subduction zone earthquake. Two different axial loading protocols were used, the first having a constant axial load and the second, variable axial loading resulting from lateral load reversal during an earthquake event.
Chapter 4  Experimental Results

4.1  Introduction

Results obtained from the experimental program of six full-scale reinforced concrete column-footing subassembly specimens are presented in this chapter. Physical observation of damage along with quantitative deformation quantities obtained from displacement transducers and strain gages are also presented. The deformation quantities presented include measured load-deformation response, column curvature, footing rotation vs column plastic hinge rotation, measured strain profile, etc. for each of the six tested specimens. Results obtained from different testes are also compared and presented to investigate the impact of individual variables considered for the experimental program. Finally, a brief comparison with past tests on bridge columns or subassemblies with similar details conducted at iSTAR laboratory are also presented.

Each specimen was visually inspected for damage throughout the test. In order to monitor the formation and progression of concrete cracking, crack maps were generated after each cycle during the pre-yield cycles. During the post-yield cycles, only the end cycles for significant ductility level were inspected unless any major damage occurred. The lateral loading was applied along the north-south direction. The north face of the column was under tension during the pull cycle and under compression during the push cycle of lateral loading. Loading cycles mentioned here are half cycles. A half cycle of loading is considered as the load reversal from a push cycle to a pull cycle or vice versa. Whereas a complete full cycle represents load reversal from a pull cycle to another pull cycle of loading.
4.2 Data Processing

Recorded data was post-processed to obtain the results from six different experimental tests. It was important to identify the sources of error while post processing the data to better represent the actual response of the column-footing specimen. Details of the data processing are presented in the following subsection prior to presenting the results from different tests.

4.2.1 Lateral Load and Displacement Data

The global force-displacement behavior is one of the most important parameters to evaluate the performance of reinforced concrete section under lateral earthquake loading. Hence, the test specimen was instrumented with external instrumentation to capture the load and deformation quantities. Details of the instrumentation to capture the global load and displacement quantities are described in earlier chapter. However, the data measured was post-processed to capture the actual load-displacement behavior for the tested specimens and are described below.

The column top was free to rotate and translate that resulted in a component of axial load which significantly contributed to resisting the applied lateral loading. Hence, the absolute lateral load that has been applied to the column was determined by subtracting the component of axial loading from the recorded lateral loads as shown in Figure 4.1.
Figure 4.1 Computation of absolute lateral load with axial component correction

Here, $\theta =$ column rotation, $\Delta =$ lateral column top displacement, $H =$ height of the column, $P_{\text{column}} =$ Applied column axial load, $F_{\text{actuator}} =$ Lateral load recorded by the actuator load cell, $F_h =$ Component of the applied axial load acting in the lateral direction.

Unlike most of the laboratory testing, the foundation used for the test was an integral part of the specimen and was placed on the laboratory strong floor without anchoring. Hence, the foundation specimen was allowed to rotate, uplift, and slide freely. The data measured from the test showed negligible sliding of the foundation specimen but experienced significant amount of uplifting. As the top displacement was measured with a string potentiometer attached to the column top, hence it does not exclude any contribution of the uplift in the measured lateral displacement. In order to get account for the lateral displacement resulting from the foundation uplift, measured data was post processed to exclude the uplift contribution into measured lateral displacement. Figure 4.2 shows the schematic diagram for lateral displacement correction resulting from foundation uplift.
Figure 4.2 Measured lateral displacement correction due to foundation uplift

Here, $U_S =$ Foundation uplift measured in the south face; $U_N =$ Foundation uplift measured in the north face; $D =$ Horizontal distance between the LVDT mounted on the north and south foundation face; $\Delta_m =$ Measured column top displacement; $\theta_u =$ Column rotation due to foundation uplift; $\theta_c =$ Corrected column rotation without foundation uplift; $\Delta_u =$ Column top displacement due to foundation uplift; $\Delta_c =$ Corrected column top displacement without foundation uplift.

### 4.2.2 Measured Curvature

The section curvature values were calculated using the strain gage data and the vertical LVDT’s placed on two opposite sides of the column. The strain gage data were primarily used to calculate the curvature but the strain gages in the proximity of column-footing interface were damaged at higher displacement ductility level. As a result, the average curvature values for the bottom section gage length were calculated using the relative displacement measured with the LVDT’s. A linear profile between the pair of LVDT’s was assumed to compute the rotation and the average curvature was then
calculated by dividing the rotation with the specific gage length. Curvature values will be plotted for different ductility level along the height of the column for different test specimens. Yield curvatures were computed using equation 4.1 where, φᵢ is experimental first yield curvature, Mₙ is the nominal moment capacity defined as the moment corresponding to concrete cover strain of 0.004 or reinforcing steel strain of 0.015, whichever occurs first and Mₒ is the experimental first yield moment. This definition of equivalent yield curvature is consistent with the recommendation from past studies on performance based seismic design approach (Goodnight et al. 2016b; Hines et al. 2004). The yield curvature is plotted here with a triangular profile having zero curvature at the top of the column and the computed yield curvature, φᵧ at column base. Both the push and pull direction of loading for all the three specimens showed a good agreement with the triangular yield curvature profile and the measured curvature values at effective yield displacement cycle of μ=1.

\[ \phi_y = \phi'_y \left( \frac{M_n}{M'_y} \right) \]  

(4.1)

Debonding of spliced rebar and strain penetration into the footing introduced a fixed end rotation at the column-footing interface. Consequently, the curvatures calculated from the bottom pair of LVDT’s between footing top and 3-inch height includes the contribution of bond-slip rotation. As a result, the calculated curvature values were orders of magnitude higher. A modified method of curvature calculation was hence used following the recommendation of Hines et al. (2004) for the bottom segment to filter out the bond-slip contribution. The method involves using a different gage length for the bottom segment that accounts for the strain penetration length, Lₛₚ as well as the instrument
gage length of 76 mm. The strain penetration length was calculated following the recommendation of Priestly et al. (2007) and using equation 4.2.

\[ L_{sp} = 0.022d_b f_{y-m} \]  

(4.2)

Here, \( d_b \) is the diameter (mm) of the dowel bar and \( f_{y-m} \) is the measured yield strength (MPa) of the dowel bar. The calculated strain penetration length was 10.4 inch and hence the gage length used for the calculation of the bottom segment curvature was 13.4 inch. The use of such an approach to filter the bond-slip contribution was further verified with the result of curvature obtained through moment-curvature analysis of the column cross-section (presented as “analytical” curvature in Figure 4.3) and the experimental section curvature calculated using the available strain gage data at that location. Equation 4.3 was used to compute the experimental flexural section curvature from the rebar tensile and compressive strain recorded with the strain gages placed near the column-footing interface and at the two opposite sides rebar.

\[ \phi_f = (\varepsilon_S - \varepsilon_N)/D \]  

(4.3)

Where, \( \varepsilon_S \) is the south side rebar strain, \( \varepsilon_N \) is the north side rebar strain, and \( D \) is the horizontal distance between the strain gages. The moment-curvature results obtained for specimen Test 2 (SV) using the three different methods are presented in Figure 4.3. The experimental curvature measured from the strain gage and the LVDT data are in close agreement with each other indicating the effectiveness of filtering the bond-slip rotation with the modified gage length calculation. A full range of comparison cannot be made due to the failure of strain gage at higher displacement ductilities and for the same reason experimental curvature of the bottom segment was not calculated using the strain gage.
data. The analytically derived moment-curvature values were also in close agreement with the experimental results prior to the concrete failure at lower ultimate strain value, further justifying the use of the method.

![Graph showing moment-curvature relationship](image)

**Figure 4.3 Interface moment curvature for specimen SV**

### 4.3 Test 1 (SC)

The first specimen in the series designated as specimen SC was tested under Cascadia Subduction lateral loading protocol and with a constant axial load level of 240 kip. Results obtained are presented in the following subsections.

#### 4.3.1 Physical Observation

The primary failure mode for specimen SC was flexural tension failure resulting in complete loss of cover concrete in the plastic hinge zone and buckling of longitudinal rebar. The first flexural crack for the specimen was observed at 18-inch height from column base. The subsequent loading cycles with increasing ductility level were associated with formation of new flexural cracks along the column height. Horizontal flexural cracks were
stabilized after the effective yield displacement cycles. A vertical crack was observed along the length of the south-east dowel bar immediately after yielding indicating bond-slip deterioration. The crack was extended longitudinally with an approximate length of 12 inch (location of first transverse reinforcement) measured from the column base. The vertical crack was stable until reaching the peak lateral load and started propagating diagonally afterward. Splitting of cover concrete along the crack was observed during this cycle and spalling of cover concrete was observed in later cycles along the length of this vertical crack. Initiation of cover crushing started in the corner region at the column-footing interface after the peak lateral load and eventually spread to the center of the column following the completion of initial spalling in the corner region. Subsequent loading cycles were associated with complete loss of cover in the south-east corner. Buckling of the south-east dowel bar was observed in the subsequent loading cycle when the bar was under compression loading. The final damage state was associated with buckling of the dowel bars in the three corners with a maximum spalled height of 20 inch in the south-east corner. Figure 4.4 shows the different damage levels observed during the test.
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4.3.2 Load-Deformation Response

The hysteretic load-deformation response and the backbone curve as obtained from the experimental result is presented in Figure 4.5. Measured lateral load is presented in ‘kip’ on the vertical axis while the column top displacement is presented in ‘inch’ unit along the X-axis. The X-axis for the backbone curve however represents the experimental displacement ductility calculated based on the experimental effective yield displacement. The experimental effective yield displacement was calculated as 0.68 inches for specimen SC. The occurrences of different damage levels are also presented in the plot. The load-deformation response of specimen SC is characterized by a stable and pinched hysteretic response with wide loops at higher displacement ductility cycles. A symmetric hysteretic curve can be observed for both the push and pull direction of loading. The peak lateral load recorded in the push direction of loading was 45.8 kip and the displacement corresponding
to the peak load was 1.26 inch. In the pull direction, the measured peak load was 42.5 kip and the displacement associated with the peak load was 1.85 inch. Experimental first yield displacement in the pull direction was measured to be 0.44 inch and the corresponding first yield force was 36.5 kip. The experimental effective yield displacement was calculated based on the recorded first yield displacement and the analytical stiffness as discussed in the earlier sections. Calculated average experimental effective yield displacement was 0.68 inch for specimen SC and the force associated with the average effective yield displacement was 40 kip. The experimental displacement ductility was then calculated as the ratio of the measured displacement and the experimental effective yield displacement. The ultimate displacement capacity was considered as the point where more than 20% strength degradation from the peak load was observed. The 20% strength degradation point was determined from the backbone curve and the displacement was recorded. The ultimate displacement capacity for specimen SC in the push and pull direction was recorded as 5.39 inch and 4.72 inch, respectively. The displacement ductility corresponding to the ultimate capacity was calculated as $\mu=7.9$ and $\mu=6.9$ in the push and pull direction respectively. Corresponding drift ratio in the push and pull direction at the ultimate displacement capacity was calculated as 5.2 and 4.6, respectively. Description of the different damage states are presented in the following subsection.
4.3.3 Concrete Cracking

Hairline cracks were first observed at the end of the 10th loading cycle (pull cycle at displacement ductility, $\mu=0.34$) on the north face of the column. The maximum lateral
displacement and force associated with this cycle was 0.23 inch and 30.1 kip (pull cycle), respectively. The first crack appeared at a height of 18 inch measured from the foundation face and extended across the entire north face of the column with an approximate length of 24.5 inch. Cracking at the column-foundation joint interface was also observed at this cycle on the north face of the column. With the increase of displacement levels numerous flexural cracks were found to form along the column height. The crack opening at the column-footing interface was significantly higher than other flexural cracks along the height of the column indicating significant rigid body rotation at the base resulting from strain penetration. The maximum crack width recorded at target displacement ductility of $\mu_{\text{target}}=1.0$ was 0.1 mm. The crack width measured at the interface between the column and footing during the loading cycle at $\mu_{\text{target}}=4.0$ was 14 mm in the south face and 8 mm in the north face. In the final loading cycle, the interface crack width was measured to be 23.5 mm (approximately 1 inch) in both the north and south side of the column. A vertical crack was also observed to form in the east face of the column during the yielding of the dowel bar. The location of the vertical crack was consistent with the presence of longitudinal reinforcement and the crack extended upward by approximately 5.5 inches from the base of the column. Crushing of cover concrete was observed in later displacement cycles along this vertical crack line.

### 4.3.4 First Yield

First yield of the north dowel bar was recorded at the end of 40th loading cycle at displacement ductility, $\mu=0.65$. The corresponding lateral displacement and load was 0.44 inch and 36.5 kip (pull cycle), respectively. Multiple cracks were observed on both the
north and south faces along the height of the column prior to yielding of the dowel bar. The maximum height of cracking observed at the end of this cycle was approximately 41 inches from the column-footing interface.

4.3.5 Concrete Spalling

The sign cover concrete spalling initiation was first observed at the end of 90th loading cycle (pull cycle at displacement ductility, $\mu=1.9$) on the south-east corner of the column. The maximum lateral force during this loading cycle was 41.9 kip and the column top displacement was 1.3 inch (pull cycle). The subsequent cycles at higher displacement ductility levels were associated with extensive spalling of cover concrete at the base of the column on both the north and south faces. The initiation of cover concrete spalling was also observed in the south face of the column in the subsequent push cycle at displacement of $\mu=2.0$. The lateral displacement and the force associated with this loading cycle was recorded as 2.1 inches and 45.3 kip. The subsequent loading cycles were associated with further progression of the cover concrete spalling. Significant spalling of the cover concrete was observed at 3.6 inches displacement in the north face of the column and 4.22 inches displacement in the south face of the column. Corresponding displacement ductility was $\mu=5.3$ in the pull direction and $\mu=6.2$ in the push direction. Associated lateral force recorded was 41.5 kip and 41.1 kip during the pull and push cycle, respectively. Complete loss of cover concrete was observed with a height of 9 inch at the end of displacement ductility $\mu=7$. 
4.3.6 Rebar Buckling

Visual observation of rebar buckling was observed in the south-east corner dowel bar of the column by the end of 104\textsuperscript{th} loading cycle (pull cycle at displacement ductility, \( \mu = 8.7 \)). The maximum-recorded lateral force at rebar buckling was 26.5 kip and the column top displacement was 5.9 inches. However, the ultimate capacity of the column as defined by 20\% degradation from peak lateral load was reached prior the observation of the longitudinal rebar buckling. It is noteworthy to mention that the buckled bar was visually observed once the spalled cover concrete in the south face was removed at the end of 104\textsuperscript{th} cycle. So, it was not evident if the bar buckled during this loading cycle, or it was already buckled from the previous pull cycle. However, since the concrete cover was not completely lost in the previous cycle (102\textsuperscript{nd} cycle) so, the later cycle (104\textsuperscript{th} cycle) was considered as the loading cycle associated with bar buckling damage state. In the subsequent push direction loading cycle, buckling of longitudinal rebar was also observed in the north side of the column when the rebars were in compression and the north face cover concrete was completely lost. It is also noted that the length of buckled bar was limited between the top of the footing and the first transverse reinforcement at 6 inches from the column base.

4.3.7 Measured Curvature

The curvature profile for specimen SC is presented in Figure 4.6 for both the push and pull direction and for increasing ductility level. Y-axis of the plot represent the moment in kip-in unit whereas the X-axis represent the curvature values in rad/in unit. The curvature profile in the push direction is presented with the solid lines and the curvature profile in
the pull direction is presented with the dashed lines. The experimental effective yield curvatures are also presented in the figure with red dashed lines for both the push and pull direction. It can be observed that both the push and pull direction of loading shows similar curvature profile where flexural curvature is mostly concentrated at the base of the column. The curvature values at the base increases with the increase in displacement ductility level. At displacement ductility $\mu=6$, the base curvature in the pull direction is slightly higher than the push direction with the measured values of 0.0028 rad/in and 0.0027 rad/in, respectively. But the difference in the measured curvature values is negligible indicating the impact of same axial load level for both the push and pull direction.

![Curvature Profile](image)

Figure 4.6 Measured curvature profile for specimen SC

The analytical yield curvature for specimen SC as obtained from the sectional analysis was 0.00018 rad/in, and the corresponding yield moment was 4179 kip-in. The experimental effective yield curvature calculated was 0.0002 rad/in. The experimental
effective yield curvature was plotted with a triangular profile where the curvature is zero at the top of the column and reaches the yield curvature at the base. It is evident that the plastic curvature was concentrated within the plastic hinge zone of the column. The experimental spread of plasticity was calculated based on the intersection of the yield curvature with the measured curvature profile. The calculated plastic hinge length for specimen SC was approximately 15 inch in the push direction and 16 inch in the pull direction. Whereas the spalled concrete height in the push and pull direction was found to be 11 inch in the north face and 22 inch in the south face of the column. For specimen SC, the ultimate curvature recorded at failure was 0.0033 rad/in for both the push and pull direction and the curvature ductility at failure was $\mu_\phi=16$.

### 4.3.8 Measured Strain Profile

The strain profile for the north-east and north-west rebar of specimen SC at different displacement ductility level is presented in Figure 4.7. It is noted that only the north side rebars in specimen SC were instrumented with strain gages. The yield strain is also presented as the red dashed line for both the push and pull direction. The pull direction strain profile is shown as the dashed line whereas the push direction is presented as the solid line. It can be observed the plastic strain is concentrated near the base of the column and the measured strain at 3 inches from the column base is orders of magnitude higher compared to the rest of the column region. In the push direction, the north side rebars were under compressive loading and hence compressive strain values were recorded up to the effective yield displacement at $\mu=1$. Following the effective yield displacement cycle, the strain at 3 inches from the column base experienced tensile strain values and the strain...
continue to increase in with the increase of displacement ductility level. This can be associated with the fact that the tension yielding of rebar leads to plastic strain accumulation at this location and while in the compressive excursion the plastic strain was not completely recovered. As a result, the rebar was under tensile strain while undergoing compressive excursion. This phenomenon leads buckling of longitudinal rebar at a later displacement ductility cycle once the cover concrete spalls off completely. However, the strain values in all other locations of the column showed compressive strain during the push cycle of loading. In the pull direction, the north side rebars were under tensile loading and tensile strain values were recorded along the column height. However, concentration of tensile can still be observed at the base of the column compared to the other locations of strain gages. But significant spread of the tensile strain can also be observed at 15 inches from the column base. However, plastic strain is only observed at 3 inches from the column base. In the pull direction of loading, significant strain penetration can also be observed into the adjoining footing element. The strain measured at 3 inches into the footing were also found to have crossed the yield strain indicating plastic strain penetration into the footing. The strain values at 9 inches into the footing element were below the yield limit up to a displacement ductility level of $\mu=3$. The strain penetration profile into the footing can be approximated as a bilinear curve with the maximum strain occurring at the base of the column. The following loading cycles at higher displacement ductility levels were associated with damage of strain gages and as a result the strain values were not incorporated into the profile.
The second specimen in the series designated as specimen SV was tested under Cascadia Subduction lateral loading protocol and with a variable axial loading protocol. The axial loading protocol consists of a maximum axial load level of 240 kip and a minimum axial load level of 160 kip. The variation in axial load level was proportional to the lateral load resisted by the column and varied from a base load of 200 kip. Results obtained from specimen SV are presented in the following subsections.

### 4.4.1 Physical Observation

The primary failure mode for specimen SV was also flexural tension failure resulting in significant concrete spalling and rebar buckling in the plastic hinge zone. The progression of damage for specimen SV was similar to specimen SC where the first flexural crack formed at 22 inches height from column base. Multiple vertical cracks with a short
height of approximately 3.5 inches was observed prior to yielding of the dowel bar. These vertical cracks were stable until the effective yield displacement after which upward diagonal progression was observed with increasing loading cycles. The vertical cracks were consistent with the length of the dowel bars and merged with the first horizontal flexural crack at 22 inch to form a cone shaped failure plane indicating lap splice failure. Complete loss of the cover concrete was observed in higher displacement cycles along this cone shaped failure plane. New flexural crack formed around the mid-height of the column prior to cover spalling indicating a tension shift in the column. But the initiation of spalling stabilized the newly formed horizontal cracks and diagonal flexure-shear crack propagation started within the bottom 1/3\textsuperscript{rd} height of the column. Residual crack width exceeding 1.0 mm was found immediately after the initiation of cover concrete spalling. The following loading cycles were associated with further progression of cover spalling and the final damage state was associated with complete cover loss in the north-east corner followed by buckling of dowel rebar. Compared to specimen SC, spalling of cover concrete for specimen SV was less extensive and was mostly limited in the corner region of the column. Figure 4.8 shows the different damage levels observed during the test.
4.4.2 Load-Deformation Response

The hysteretic load-deformation response and the backbone curves are presented in Figure 4.9. The experimental effective yield displacement was calculated as 0.67 inches for specimen SV. Different damage levels are also presented in the plot. The load-deformation response of specimen SV is characterized by a stable and pinched hysteretic
response with wide loops at higher displacement ductility cycles. Specimen SC experienced wider hysteretic loops compared to specimen SV indicating comparably less energy dissipation for SV. The result was consistent with physical observation of damage where less intensive concrete spalling was observed for specimen SV. An asymmetric hysteretic curve can be observed for both the push and pull direction of loading for specimen SV. The peak lateral load recorded in the push direction of loading was 46.1 kip and the displacement corresponding to the peak load was 1.77 inch. In the pull direction, the measured peak load was 39.6 kip and the displacement associated with the peak load was 2.16 inch. Experimental first yield displacement in both the push and pull direction was measured to be 0.52 inch and the corresponding first yield force was 40.9 kip in the push direction and 36.1 kip in the pull direction. Calculated average experimental effective yield displacement was 0.67 inch and the force associated with the average effective yield displacement was 39.6 kip. The ultimate displacement capacity defined as the 20% strength degradation from the peak was 5.35 inch in the push and 5.24 inch in the pull direction, respectively. For SV, the ultimate ductility in push and pull direction was $\mu=8.0$ and $\mu=7.8$, respectively. Corresponding drift ratio in the push and pull direction at the ultimate displacement capacity was calculated as 5.2 and 5.1, respectively. Description of the different damage states are presented in the following subsection.
4.4.3 Concrete Cracking

Flexural crack appears at the end of 8th loading cycle at target displacement ductility, $\mu=0.35$ on the north face of the column. The lateral load and top displacement
measured at the end of this loading cycle was 27.3 kip and 0.23 inch respectively. Horizontal crack formed at 22 inches from the column-foundation interface extending 20.5 inch in the north face of column. The following loading cycle was associated with the formation of first flexural crack in the south face of the column. Lateral load and column top displacement associated with the loading cycle was 27.9 kip and 0.22 inch. Residual crack width measured at the end of this cycle was 0.002” (0.05mm). First diagonal crack starting at the column-footing interface and extending 5.5 inch at 45-degree angle along the length of the column was observed at the end of 31\textsuperscript{st} loading cycle at a displacement ductility, $\mu=0.78$.

\textbf{4.4.4 First Yield}

Theoretical yielding of longitudinal rebar was observed by the end of 33\textsuperscript{rd} loading cycle in the push direction at displacement ductility, $\mu=0.78$. Several cracks formed at different height along the column length prior to yielding of the dowel bar. Lateral load and displacement associated with the first rebar yielding was recorded as 40.8 kip and 0.52 inch respectively. The maximum residual crack width measured at the end of this cycle was 0.008” (0.20mm). It was also observed that, most of the cracks formed were concentrated at the location of transverse reinforcement. In the pull direction, yielding of longitudinal rebar was first observed during the 46\textsuperscript{th} loading cycle at a displacement ductility of 0.78. The lateral load and displacement associated with the loading cycle was 36.4 kip and 0.52 inch, respectively.
4.4.5 Concrete Spalling

Splitting or flaking of cover concrete was observed at the end of 85th cycle at displacement ductility, \( \mu = 1.97 \). It was also observed that, the vertical crack in the east face was extended to 10 inches along the height of the column in the previous loading cycle. Lateral load and column top displacement recorded at the end of this loading cycle was 45.3 kip and 1.32 inch respectively. Cover concrete spalled off as a result of splitting crack on the east face of the column which was oriented perpendicular to the lateral loading direction. It should be noted that flaking of cover concrete was not due to the compressive crushing of cover rather splitting a portion of the cover during the tensile loading cycle. It was also concluded that the flaking of cover concrete resulted from lap-splice failure in the northeast dowel bars rather than crushing of concrete. The phenomenon was also observed for specimen SC. However, the initiation of traditional spalling of cover concrete due to compressive demand was observed at the end of 93rd loading cycle on the north face of column at a displacement ductility of \( \mu = 2.99 \). Measured lateral load and the column top displacement corresponding to the loading cycle was 46.1 kip and 2.0 inch, respectively. Initiation of cover spalling was also observed on the south face of the column under the pull direction during the 96th loading cycle at displacement ductility \( \mu = 3.23 \). Lateral load and displacement corresponding to the pull cycle was recorded as 39.6 kip and 2.17 inch. The following loading cycles at subsequent higher displacement ductility level was associated with significant progress in spalling of cover concrete. Complete loss of cover concrete was finally observed in the push and pull direction during the 103rd and 104th loading cycles at displacement ductility \( \mu = 8.1 \) and 7.8, respectively. Lateral load
corresponding to the complete loss of cover concrete was 36.6 kip and 36.0 kip in the push and pull direction respectively. Also, the column top displacement was recorded as 5.42 inch and 5.24 inch, respectively in the push and pull direction.

4.4.6 Residual Crack Width

The residual crack width greater than 0.04" (1.0 mm) was considered as the damage state when intervention is required for long-term serviceability of the bridges. Following an earthquake event, any bridge column having residual crack width greater than 0.04" (1.0mm) need to be repaired with epoxy injection to prevent long term corrosion of reinforcing bars. The residual crack width exceeding 0.04" (1.0mm) was measured for this specimen at the end of 86th loading cycle at displacement ductility, \( \mu = 1.7 \). Measured lateral load and displacement corresponding to the residual crack width exceeding 1.0 mm was 38.2 kip and 1.12 inch respectively.

4.4.7 Rebar Buckling

The ultimate damage state as defined by 20% degradation from the peak lateral load capacity was observed at the end of 104th loading cycle (pull cycle at target displacement ductility, \( \mu = 7.8 \)). Lateral load measured at the end of this loading cycle was 36.0 kip. The following trailing loading cycle in the push direction at displacement ductility \( \mu = 5.0 \) was associated with the observation of buckled rebar in the north side of the column. It is noted that the earlier push cycle at displacement ductility \( \mu = 8.1 \) was associated with complete loss of cover concrete that facilitated buckling of longitudinal rebar. Considerable concrete spalling was observed prior to rebar buckling with 12-inch spall height from the column–
footing interface. The length of buckled rebar was again limited to the spacing between the top of the footing and the first transverse reinforcement.

### 4.4.8 Measured Curvature

The curvature profile for specimen SV is presented in Figure 4.10 for both the push and pull direction and for different ductility level. The curvature profile in the push direction is presented with the solid lines and the curvature profile in the pull direction is presented with the dashed lines. Experimental effective yield curvatures are also presented in the figure with red dashed lines for both the push and pull direction. Slightly asymmetric curvature profile is observed between the push and pull direction of loading where the pull direction experienced higher curvature at the base of the column during displacement ductility cycle $\mu=6.5$. The overall curvature profile is similar to specimen SC where the curvature at the base increases with the increase in displacement ductility level. At displacement ductility $\mu=6.5$, the base curvature in the pull direction is slightly higher than the push direction with the measured values of 0.003 rad/in and 0.0027 rad/in, respectively. Also, the curvature profile along the height of the column shows a linear pattern where the maximum curvature occurs near the base of the column and decreases with the increase of the column height.
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The theoretical average analytical effective yield curvature for specimen SV was 0.00018 rad/in and the experimental effective yield curvature was computed as 0.00019 rad/in. The analytical first yield curvature in the push and pull direction was 0.00017 rad/in and 0.00016 rad/in, respectively. Whereas the measured experimental first yield curvature for the push and pull direction was 0.00018 rad/in and 0.00017 rad/in, respectively. While the experimental first yield curvature was slightly higher than the analytically obtained values, but the difference was negligible. The corresponding analytical yield moment for the push direction was 4584 kip-in and in the pull direction the analytical moment was 3879 kip-in. The average yield moment for both the push and pull direction was thus calculated to be 4232 kip-in where the. The experimental yield moment in the pull direction was close to the analytically obtained value and was recorded as 3702 kip-in. Whereas in the push direction, the experimentally obtained yield moment was found to be 4299 kip-in which was lower than the analytically obtained yield moment for the specimen. It is also
evident that the plastic curvature was concentrated within the plastic hinge zone of the column. The experimental spread of plasticity was calculated based on the intersection of the yield curvature with the measured curvature profile. The calculated plastic hinge length for specimen SV was approximately 17 inch in the push direction and 19 inch in the pull direction. The spalled concrete height in the push and pull direction was found to be 13 inch in the north face and 12 inch in the south face of the column. For specimen SV, the ultimate curvature ductility in the push and pull direction was $\mu_{\phi}=15$ and $\mu_{\phi}=20$, respectively.

4.4.9 Measured Strain Profile

The strain profile for the north-east and south-east rebar of specimen SV at different displacement ductility level is presented in Figure 4.11. The yield strain is also presented as the red dashed line for both the push and pull direction. The pull direction strain profile is shown as the dashed line whereas the push direction is presented as the solid line. Strain profile similar to specimen SC was observed where the plastic strain is concentrated near the base of the column and the measured strain at 3 inches from the column base is orders of magnitude higher compared to the rest of the column region. In the push direction, the north side rebars were under compressive loading and compressive strain values were recorded up to the effective yield displacement at $\mu=1$. Following the effective yield displacement cycle, the strain at 3 inches from the column base experienced tensile strain values and the strain continue to increase with the increase of displacement ductility level. The strain values in all other locations of the column showed compressive strain during the push cycle of loading. In the pull direction, the north side rebars were under tensile loading
and tensile strain values were recorded along the column height. Concentration of tensile strain can still be observed at the base of the column compared to the other locations of strain gages. Compared to specimen SC, the spread of plastic strain was concentrated within shorter height of the column. Plastic strain can be observed up to column height of 9 inch from the column base. However, in the pull direction of loading, significant strain penetration can be observed into the adjoining footing element where plastic strain can be found 3 inches into the footing depth. The south side rebar on the other hand, showed a similar strain profile as the north side rebar but the amplitude of strain value at 3 inches from column base was higher compared to the north side rebar. The strain at 3 inches from column base during the displacement ductility cycle at $\mu=3$ was found to be 0.034 for the south side rebar whereas the north side rebar experienced a maximum strain of 0.032. It was also found that the strain penetration into the footing was higher for the south side rebar compared to the north side rebar. This can be attributed to the higher axial load ratio associated with the push direction that resulted in higher strain penetration into the footing element for south side rebar that was under tensile excursion during the push cycle.
4.5 Test 3 (LV)

The third specimen in the test series designated as specimen LV was tested under the conventional laboratory loading protocol consisted of three cycle symmetric protocol at each displacement ductility level and with a variable axial loading protocol. The axial loading protocol was same as the second specimen in the series and consisted of a maximum axial load level of 240 kip and a minimum axial load level of 160 kip. The variation in axial load was proportional to the lateral load and varied from a base load of 200 kip. Results obtained from specimen LV are presented in the following subsections.

4.5.1 Physical Observation

The first flexural crack for specimen LV formed at 17 inches from the column base and the first vertical crack was observed in the east face of the column during yielding of dowel bar. Numerous vertical cracks were found in the later cycles along the corner region.
of the column consistent with the location of the spliced bars. Initiation of cover concrete spalling started after reaching the peak lateral load. Like the previous specimens, spalling started in the corner region and eventually spread to the center of the column. Residual crack width was found to exceed 1.0 mm followed by the initiation of the spalling. Extensive spalling of the cover concrete started in the north face along the previously formed vertical crack and exposed the longitudinal bar in the north-east corner. The newly exposed bar was scrutinized for any sign of instability and was found to be straight without buckling. The following loading cycle was associated with extensive cover spalling in the south face of the column and the longitudinal rebar in the south-west corner was also exposed. In the following loading cycle, buckling of north-east dowel bar was observed under compression. The buckled dowel bar then fractured in tension during load reversal in the following cycle. The final damage state for specimen LV was associated with extensive spalling of cover concrete and fracture of two dowel bars in the south and north side. Furthermore, all the dowel bars were found to have buckled while reaching the final damage state. Compared to specimen SC and SV, spalling of cover concrete for specimen LV was extensive but was mostly limited in the corner region of the column. Figure 4.12 shows the different damage levels observed during the test.
4.5.2 Load-Deformation Response

The hysteretic load-deformation response and the backbone curves are presented in Figure 4.13. For specimen LV, the experimental effective yield displacement was
calculated as 0.70 inches. The load-deformation response of specimen LV is also characterized by a stable and pinched hysteretic response with wide loops at higher displacement ductility cycles. While the specimen LV was tested under a varying axial loading protocol, the cyclic hysteretic response showed almost a symmetric curve between the push and pull direction of loading. The peak lateral load recorded in the push direction of loading was 41.6 kip and the displacement corresponding to the peak load was 1.93 inch. In the pull direction, the measured peak load was 40.7 kip and the displacement of 1.93 inch associated with the peak load was same as the push direction. Experimental first yield displacement in both the push and pull direction was measured to be 0.43 inch and 0.58 inch, respectively. Corresponding first yield force was 34.2 kip in the push direction and 37.5 kip in the pull direction. Calculated average experimental effective yield displacement was 0.70 inch and the force associated with the average effective yield displacement was 37.1 kip. The ultimate displacement capacity defined as the 20% strength degradation from the peak was 4.69 inch in the push and 5.12 inch in the pull direction, respectively. For LV, the ultimate ductility in push and pull direction was $\mu=6.7$ and $\mu=7.3$, respectively. Corresponding drift ratio in the push and pull direction at the ultimate displacement capacity was calculated as 4.6 and 5.0, respectively. Description of the different damage states are presented in the following subsection.
4.5.3 Concrete Cracking

Hairline crack was observed in the north face of the column-foundation interface during the 2nd cycle of loading at target displacement ductility, $\mu=0.21$. In addition, a
vertical crack was observed in the north face along the north-east corner. In the later stages of loading, spalling of cover concrete was observed along the line of this vertical crack. Lateral load and column top displacement measured at the end of this loading cycle was 22.8 kip and 0.14 inch, respectively. The following loading cycle at displacement ductility $\mu=0.45$, the south column-footing interface also opened up along with a flexural crack at 17-inch height from the column base.

At the end of the 9th loading cycle at displacement ductility $\mu=1.0$ the crack formation tends to stabilize, and no new crack was found in the next three loading cycles. Three more new cracks were found at displacement ductility $\mu=1.0$ and $\mu=1.5$. No new crack was found to form at the later stages of loading. However, the existing cracks were found to increase in width with almost each increasing loading cycle.

4.5.4 First Yield

Flexural yielding of the dowel bar was first observed in the south-west corner of the column at the end of the 5th loading cycle at displacement ductility, $\mu=0.62$. Considerable crack of the column in the south face was observed prior to yielding the south dowel bar. The maximum crack width during this cycle was found to be 0.20 mm, and the residual crack width measured was 0.05 mm.

The north dowel bar at the north-west corner was found to exceed the yield strain in the following pull cycle (6th loading cycle) at displacement ductility, $\mu=0.82$. Similar to the south face, considerable cracking of the north face was also observed before yielding of the dowel bars. The maximum crack width recorded at this cycle was found to be 0.20 mm however, the crack at the column-footing interface opened up by 0.76 mm. The
residual crack width following the north dowel yielding was recorded to be less than 0.05 mm.

4.5.5 Concrete Spalling

Vertical crack in the column face was observed as early as 2nd cycle of loading at displacement ductility, $\mu=0.25$. The vertical cracks formed in the earlier cycles were found to be extending with subsequent loading cycles and were later contributed to the spalling of cover concrete along these cracks. Initiation of cover concrete spalling was noticed at the end of the third push cycle at displacement ductility, $\mu=2.78$. The final push cycle at target ductility, $\mu=5.58$ resulted in major spalling of cover concrete in the north face of the column with a spalled height of 8 inch and base width of 12 inch. The spalling in the north face of the column resulted in exposing the rebar in the northeast corner, which later facilitates buckling of the dowel bar in the subsequent loading cycle. South face of the column also experienced major spalling at the end of the second pull cycle at displacement ductility $\mu=5.56$ while exposing the longitudinal and dowel bar in the south-west corner.

4.5.6 Residual Crack Width

The residual crack width measured at the first 6 cycles of loading was insignificant and were close to 0.05 mm. At the end of the 7th cycle at displacement ductility, $\mu=0.82$, the residual crack width was measured to be 0.1 mm whereas the maximum crack opening recorded at the peak displacement was 0.70 mm. At the end of effective yield displacement at ductility $\mu=1.0$, the maximum recorded residual crack width during the push and pull cycle were 0.18 mm and 0.20 mm, respectively whereas the maximum crack opening measured at the peak displacement level for these two cycles were 1.02 mm and 1.52 mm,
respectively. The residual crack width increased to 0.25 mm in push cycle and 0.35 mm in pull cycle at the end of displacement ductility $\mu=1.5$. The maximum-recorded crack opening at this displacement ductility level was found to be 3.25 mm and 2.54 mm in the pull and push cycles respectively. The residual crack width remained stable during the subsequent push and pull cycles at target displacement ductility $\mu=2.0$. However, a trend in increasing residual crack width was observed starting from the beginning of displacement cycles at ductility $\mu=3.0$.

Finally, the serviceability limit state based on the residual crack width of 1.0 mm was found to be exceeding at the end of 32nd cycle at displacement ductility, $\mu=2.77$. The maximum-recorded residual crack width was found to be 1.02 mm in both the north and south side of the column. Lateral load and column top displacement associated with this cycle was 39.9 kip and 1.94 inch. Last recorded residual crack width at the end of target displacement ductility $\mu=6.0$, was recorded to be 6.50 mm while loss of cover concrete was observed in both the north and south face of the column.

### 4.5.7 Rebar Buckling & Fracture

The Buckling of the dowel bars in both the south and north face was observed during the test at higher displacement ductility cycles. Both the bar buckling was first initiated by the loss of cover concrete in previous loading cycle. At the end of the final push cycle at displacement ductility, $\mu=5.58$, the north-east corner of the column concrete cover was spalled off entirely and hence exposed the rebar in that corner. Observation shows no sign of bar buckling at the end of this cycle. However, the following loading cycle resulted in cover concrete loss in the south face of the column. While returning from
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this cycle at the first push cycle at displacement ductility $\mu=7.37$, the north face rebar was in compression and resulted in buckling without the support of cover concrete. Similarly, the previously exposed bar in the south-west corner buckled in the following loading cycle at the end of first pull cycle at displacement ductility, $\mu=7.50$, when the south side bar was in compression. At the end of second pull cycle at displacement ductility, $\mu=7.43$, the southeast corner bar was also found to be buckled. It is noted that the cover concrete in this corner was exposed in the previous pull cycle.

The final damage recorded for the column was bar fracture in both the north and south side. Significant drop in column lateral load carrying capacity was associated with the fracture of the bar. Previously buckled bar fractured shortly after the reversal of loading cycle. For instance, previously buckled bar in the north face fractured while reversal from push cycle to pull cycle at displacement ductility, $\mu=7.43$. Similarly, the buckled bar in the south-west corner fractured in the reversal of loading from pull cycle at displacement ductility, $\mu=6.1$.

4.5.8 Measured Curvature

The curvature profile for specimen LV is presented in Figure 4.14 for both the push and pull direction and for different ductility level. The curvature profile in the push direction is presented with the solid lines and the curvature profile in the pull direction is presented with the dashed lines. Experimental effective yield curvatures are also presented in the figure with red dashed lines for both the push and pull direction. Unlike specimen SV, a symmetric curvature profile is observed for specimen LV between the push and pull direction of loading. The overall curvature profile is similar to specimen SC and SV where
the curvature at the base increases with the increase in displacement ductility level. At displacement ductility $\mu=6$, the base curvature in the pull direction is slightly higher than the push direction with the measured values of 0.0026 rad/in and 0.0025 rad/in, respectively. Also, the curvature profile along the height of the column shows a linear pattern where the maximum curvature occurs near the base of the column and decreases with the increase of the column height.

![Curvature Profile](image)

**Figure 4.14** Measured curvature profile for specimen LV

Theoretical average analytical effective yield curvature for specimen LV was 0.00018 rad/in and the experimental effective yield curvature was computed as 0.00019 rad/in, same as specimen SV. The analytical first yield curvature in the push and pull direction was 0.00017 rad/in and 0.00016 rad/in, respectively. Whereas the measured experimental first yield curvature for the push and pull direction was 0.00018 rad/in and 0.00017 rad/in, respectively. The difference between the analytically obtained yield curvature values and the experimentally measured values were negligible. The
corresponding analytical yield moment for the push direction was 4619 kip-in and in the pull direction the analytical moment was 3958 kip-in. The average yield moment for both the push and pull direction was thus calculated to be 4289 kip-in. The experimental yield moment in the pull direction was close to the analytically obtained value and was recorded as 3685 kip-in. Whereas in the push direction, the experimentally obtained yield moment was found to be 4299 kip-in which was slightly lower than the analytically obtained yield moment for the specimen. It is also evident that the plastic curvature was concentrated within the plastic hinge zone of the column. The experimental spread of plasticity was calculated based on the intersection of the yield curvature with the measured curvature profile. The calculated plastic hinge length for specimen SV was approximately 17 inch in the push direction and 16.7 inch in the pull direction. The spalled concrete height in the push and pull direction was found to be 17.5 inch in the north face and 22 inch in the south face of the column. For specimen LV, the ultimate curvature ductility in the push and pull direction was $\mu_\phi = 16$ and $\mu_\phi = 20$, respectively.

### 4.5.9 Measured Strain Profile

The strain profile for the north-east and south-east rebar of specimen LV at different displacement ductility level is presented in Figure 4.15. The yield strain is also presented as the red dashed line for both the push and pull direction. The pull direction strain profile is shown as the dashed line whereas the push direction is presented as the solid line. Strain profile similar to specimen SC and SV was observed where the plastic strain is concentrated near the base of the column and the measured strain at 2 inches from the column base is orders of magnitude higher compared to the rest of the column region. In
the push direction, the north side rebars were under compressive loading and compressive strain values were recorded up to the effective yield displacement at $\mu=1$. Following the effective yield displacement cycle, the strain at 2 inches from the column base experienced tensile strain values and the strain continue to increase with the increase of displacement ductility level. The strain values in all other locations of the column showed compressive strain during the push cycle of loading. In the pull direction, the north side rebars were under tensile loading and tensile strain values were recorded along the column height. Concentration of tensile strain can still be observed at the base of the column compared to the other locations of strain gages. Plastic strain can be observed up to column height of 15 inch from the column base. However, in the pull direction of loading, significant strain penetration can be observed into the adjoining footing element where plastic strain can be found 3 inches into the footing depth. The south side rebar on the other hand, showed a similar strain profile where the plastic strain can be observed up to a height of 10 inch. The strain at 2 inches from column base during the displacement ductility cycle at $\mu=2$ was found to be 0.029 for the south side rebar whereas the north side rebar experienced a maximum strain of 0.024.
4.6 Test 4 (MS#10)

The fourth specimen in the test series designated as specimen MS#10 was tested under the subduction zone loading protocol and with a varying axial loading protocol. The axial loading protocol was same as SV and LV and consisted of a maximum axial load level of 240 kip and a minimum axial load level of 160 kip. The variation in axial load was proportional to the lateral load and varied from a base load of 200 kip. The reinforcing details of the specimen was different than the earlier three specimens in the test series. The specimen MS#10 was reinforced with 4 #10 longitudinal bar and was lap spliced with 47d_b lap length in the plastic hinge zone. The rest of the geometric and reinforcing details were same as the earlier specimens. Results obtained from specimen MS#10 are presented in the following subsections.
4.6.1 Physical Observation

The first hairline flexural crack was observed during the first push cycle at 18 inches and 31 inches height from the column base. Unlike other specimens, the vertical crack in specimen MS#10 was observed prior to the yielding of longitudinal or dowel bar. Numerous vertical cracks starting at the base of the column formed prior to reaching the peak lateral load. The vertical cracks were also observed to form from the existing horizontal cracks along the length of the dowel bars. It is noted that the cracking of the specimen MS#10 was more extensive compared to the first three tested specimens. Significant progression of horizontal flexural cracks in the diagonal direction was also evident as a result of increased shear demand in the column. It could be attributed to the higher reinforcing ratio of the column that correspond to higher flexural capacity and hence higher shear demand for the specimen. Exceedance of residual crack width by more than 1.0 mm was first observed at the load reversal from the second pull cycle at displacement ductility $\mu=1.90$ after attaining the peak load in the pull direction. Flaking of cover concrete was also observed at same displacement ductility cycle with a flaked width of 6 inches, height of 1.38 inches and a depth of 0.32 inches. No significant development was observed until the first push cycle at displacement ductility $\mu=2.6$ when cover concrete spalling was observed on the north face of the column. The following loading cycle in the pull direction was associated with cover spalling in the south face of the column. Subsequent loading cycles were associated with more spalling of cover concrete in the north face while the south face did not experience any progress in concrete spalling. Major spalling of north side cover concrete was observed during the first push cycle at $\mu=5.5$ with a spalled height
of 7 inches. The following pull loading cycle at $\mu=5.5$ was associated with the observation of a major crack in the concrete footing on the east and west face. The vertical cracks started at the bottom of the footing and extended upward along the edge of the column indicating flexural cracking resulting from significant uplift of the footing in the pull direction. It was also found that the strain gage attached to the footing longitudinal reinforcement has reached past the yield strain indicating plastic demand in the footing. The subsequent pull direction loading cycles were associated with significant uplift of the footing resulting from opening of the crack. The width of the footing crack measured at the peak of the first pull cycle at displacement ductility $\mu=6.5$ was 0.76 mm.

However, the response of the specimen in the push and pull direction was significantly different due to the damage in the footing concrete block. In the push direction the damage was concentrated at the column base where a plastic hinge formed. Extensive spalling of column cover concrete was observed on the north face of the column similar to other three specimens. However, buckling of longitudinal reinforcement was not observed at the final damage state. Pull direction on the other hand was associated with significantly different damage level for column base region. Initiation of spalling was observed under the pull direction of loading, but the concrete spalling was minimal compared to the push direction. In contrary, the damage in the pull direction was concentrated in the footing block as large vertical crack was found to form from the base of the footing. Extensive opening of the footing flexural crack led to significant uplift of the footing under pull direction loading. As a result, the damage in the column region was minimal and only limited to minor cover spalling. The final damage state for the specimen was associated
with major spalling of cover concrete in the north side of the column without any sign of rebar buckling. The south side of the column on the other hand sustained minimal spalling while flexural cracking of the spread footing was observed. Figure 4.16 shows the different damage levels observed during the test.
4.6.2 Load-Deformation Response

The hysteretic load-deformation response and the backbone curves are presented in Figure 4.17. For specimen MS#10, the experimental effective yield displacement was calculated as 0.76 inches. The load-deformation response of specimen MS#10 is characterized by an asymmetric hysteretic response for the push and pull direction of loading. The push direction of loading shows a typical hysteretic response for flexural dominated reinforced concrete column with wide and stable loops. In contrary, the pull direction of loading shows a bilinear hysteretic curve similar to rocking behavior. While the push direction was controlled by the flexural plastic hinging of the column region, the pull direction was dominated by flexural cracking of the footing block. The hysteretic response in the pull direction is indicative of brittle flexural failure of the concrete spread footing. The peak lateral load recorded in the push direction of loading was 58.6 kip and
the displacement corresponding to the peak load was 2.99 inch. In the pull direction, the measured peak load was 45.2 kip and the displacement of 0.92 inch associated with the peak load was recorded. Experimental first yield displacement in both the push and pull direction was same and measured to be 0.74 inch. The force corresponding to first yield displacement was 55.3 kip in the push direction and 43.5 kip in the pull direction. Calculated average experimental effective yield displacement was 0.76 inch and the force associated with the average effective yield displacement was 54.2 kip in the push direction and 43.2 kip in the pull direction. The ultimate displacement capacity defined as the 20% strength degradation from the peak lateral load was not reached at the end of the test where the maximum displacement ductility was $\mu=8.0$. The drift ration corresponding to the displacement ductility $\mu=8.0$ was 5.9% in the push and pull direction. Description of the different damage states are presented in the following subsection.


d) Backbone curve

Figure 4.17 Load-Deformation response for specimen MS#10

4.6.3 Concrete Cracking

For specimen MS#10, two hairline flexural cracks were observed in the south face of the column during the very first push cycle of loading at 18 inches and 31 inches from the column base. The following loading cycle in the pull direction was also associated with two more hairline flexural cracks at the same height from the column base and along with the opening of the column-footing joint interface. While no new crack formed in the subsequent loading cycle at displacement ductility $\mu=0.25$ but the existing cracks were found to extend on the east and west face of the column. The first push cycle at $\mu=0.5$ was associated with formation of two new horizontal crack at 47 inches and 63 inches from the column base. The first pull cycle at $\mu=0.5$ was associated with formation of three more horizontal cracks at 49 inches, 53 inches and 59 inches from the base. First vertical curve was also observed during this loading cycle on the north face of the column along the north-
west corner region. The height of the vertical curve was measured to be 3 inches from the base. Subsequent loading cycles at $\mu=0.5$ were associated with one more horizontal crack and a vertical crack in the south-east corner. The loading cycles at displacement ductility $\mu=0.75$ were associated with formation of three more vertical cracks on the east face along the north-east corner and the south and west face along the south-west corner. Three more horizontal cracks also formed during the loading cycles at $\mu=0.75$ between 10 inches and 55 inches from the column base. The loading cycles at analytical and experimental effective yield displacement at $\mu=1.0$ were associated with formation of four new vertical cracks. Also, the existing horizontal cracks started propagating diagonally indicating the increased shear demand in the column region. Following the effective yield displacement cycle, the cracks tend to stabilize, and no new crack formed until the loading cycles at $\mu=1.30$. The loading cycles between displacement ductility $\mu=1.30$ and $\mu=1.70$ were marked with formation of numerous vertical cracks and also propagation of existing cracks diagonally indicating flexural-shear crack.

### 4.6.4 First Yield

Flexural yielding of the dowel bar was first observed in the south-west corner of the column at the end of the 19th loading cycle at the first push cycle at analytical yield displacement ductility, $\mu_{\text{ana}}=1.0$. Considerable horizontal and vertical cracking of the column was observed prior to the yielding of the south dowel bar. The maximum crack width at the end of this cycle was measured for the north side column-footing interface opening and was found to be 1.27 mm. The residual crack width measured at zero force was between 0.05 mm and 0.10 mm. However, the maximum crack width for the flexural
crack was found to be 0.41 mm and the measured residual crack width was less than 0.10 mm. The north dowel bar at the north-west corner was found to exceed the yield strain in the following pull cycle (20\textsuperscript{th} loading cycle) at displacement ductility, $\mu_{\text{ana}}=1.0$. The maximum crack width for the flexural cracks in the column region was recorded as 0.41 mm however, the crack at the column-footing interface opened up by 1.02 mm. The residual crack width following the north dowel yielding was recorded to be less than 0.10 mm.

### 4.6.5 Concrete Spalling

Vertical crack in the column face was observed as early as 1\textsuperscript{st} loading cycle at displacement ductility, $\mu=0.25$. The vertical cracks formed in the earlier cycles of loading was later found to contribute to the initiation of concrete flaking in the south-west corner of the column. A small chunk of concrete was first found to flaked out during the third push cycle at displacement ductility $\mu=1.6$. The flaking of cover concrete was also observed in the north face of the column during the second pull cycle at displacement ductility $\mu=1.9$. However, it is noted that at both of these occasions the concrete was under tensile loading which indicates that the flaking of cover concrete was not due to compressive demand. Initiation of cover concrete spalling resulting from compressive demand was first noticed at the end of the first push cycle at displacement ductility, $\mu=2.6$. The first pull cycle at displacement ductility, $\mu=3.0$ was associated with the observation of first cover concrete spalling on the south face of the column. The following loading cycles were associated with further spalling of column cover concrete on the north face of the column. The south side of the column on the other hand experienced no further spalling in the cover concrete
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due to the large crack opening in the footing during pull cycles. Major spalling of cover concrete in the north face of the column with a spalled height of approximately 7 inch was observed during the first cycle at target displacement ductility of $\mu=6.5$. However, the extent of spalling was only limited to the cover without exposing any longitudinal or dowel rebar.

4.6.6 Residual Crack Width

The residual crack width measured up to the effective yield displacement cycles were insignificant and were less than 0.10 mm. At the end of effective yield displacement at ductility $\mu=1.0$, the maximum recorded residual crack width during the push and pull cycle was less than 0.10 mm, respectively whereas the maximum crack opening measured at the peak displacement level for these two cycles were 1.27 mm and 1.02 mm, respectively. The residual crack width started to increase after the loading cycle at displacement ductility $\mu=1.2$. At the end of the 37th loading cycle at displacement ductility, $\mu=1.2$, the residual crack width in the north side of the column was measured to be 0.35 mm whereas the maximum crack opening recorded at the peak displacement was 2.03 mm. In the south side of the column the residual crack width was 0.15 mm, and the opening was 2.03 mm at peak displacement.

Finally, the serviceability limit state based on the residual crack width of 1.0 mm was found to be exceeding at the end of 74th cycle at displacement ductility, $\mu=1.91$. The maximum-recorded residual crack width was found to be 1.02 mm in the north side of the column. Whereas the maximum residual crack width in the south side of the column was 0.74 mm. Lateral load and column top displacement associated with this cycle was 43.61
kip and 1.45 inch. Last recorded residual crack width at the end of target displacement ductility $\mu=3.0$, was recorded to be 2.74 mm in the south side and 1.96 mm in the north side.

4.6.7 **Rebar Buckling & Fracture**

The ultimate damage state for specimen MS#10 was cover concrete spalling in the north face of the column without exposing the longitudinal rebar and large flexural crack in the footing. Sign of bar buckling was not observed in this specimen as the longitudinal rebars were not exposed at the end of final loading cycle. Finally, bar fracture was not also observed for the specimen.

4.6.8 **Measured Curvature**

The curvature profile for specimen MS#10 is presented in Figure 4.18 for both the push and pull direction and for different ductility level. The curvature profile in the push direction is presented with the solid lines and the curvature profile in the pull direction is presented with the dashed lines. Experimental effective yield curvatures are also presented in the figure with red dashed lines for both the push and pull direction. The curvature profile between push and pull direction was significantly different at higher displacement ductility level. The overall curvature profile was similar to the first three specimens where the plastic curvature was concentrated at the column base in both the push and pull direction. However, the curvature in the push direction was significantly higher than the pull direction as the demand in the column during the pull cycles of loading was less due to the cracking of the footing. Although the magnitude was less, but the curvature at the base of the column in the pull direction was found to increase with higher displacement ductility level. At
displacement ductility $\mu=8$, the base curvature in the pull direction was measured as 0.001 rad/in whereas the measured curvature in the push direction was 0.0033 rad/in, respectively. This clearly indicates the effect of reduced curvature demand during the pull cycles of loading resulting from shifting the damage into footing region.

![Curvature profile](image)

**Figure 4.18 Measured curvature profile for specimen MS#10**

Theoretical average analytical effective yield curvature for specimen MS#10 was 0.000183 rad/in and the experimental effective yield curvature was computed as 0.00027 rad/in. The analytical first yield curvature in the push and pull direction was 0.00017 rad/in and 0.00016 rad/in, respectively. Whereas the measured experimental first yield curvature for the push and pull direction was 0.00027 rad/in and 0.00025 rad/in, respectively. The corresponding analytical yield moment for the push direction was 4989 kip-in and in the pull direction the analytical moment was 4368 kip-in. The average yield moment for both the push and pull direction was thus calculated to be 4679 kip-in. The experimental yield moment in the pull direction was close to the analytically obtained value and was recorded.
as 4465 kip-in. Whereas in the push direction, the experimentally obtained yield moment was found to be 5680 kip-in which was higher than the analytically obtained yield moment for the specimen. The experimental spread of plasticity was calculated based on the intersection of the yield curvature with the measured curvature profile. The calculated plastic hinge length for specimen MS#10 was approximately 24 inch in the push direction and 8.5 inch in the pull direction. The spalled concrete height in the push direction was found to be 10 inch in the north face and in the pull direction the spalling was insignificant. For specimen LV, the ultimate curvature ductility in the push and pull direction was $\mu_p=13$ and $\mu_p=4$, respectively.

### 4.6.9 Measured Strain Profile

The strain profile for the north-east and south-east rebar of specimen MS#10 at different displacement ductility level is presented in Figure 4.19. The yield strain is also presented as the red dashed line for both the push and pull direction. The pull direction strain profile is shown as the dashed line whereas the push direction is presented as the solid line. The strain profile for specimen MS#10 was similar to other specimens for the push direction of loading whereas the strain profile in the pull direction was significantly different. In the push direction, the plastic strain was concentrated near the base of the column and the measured strain at the column-footing interface was orders of magnitude higher compared to the rest of the column region. Figure 4.19(a) shows the north side rebar under push and pull direction of loading where the rebar was under tensile excursion during pull cycles and compressive excursion under push cycles. The strain profile during the pull cycles of loading shows a linear profile up to the yield displacement cycle. The subsequent
loading cycles up to a displacement ductility of $\mu=4$ indicated the accumulation of plastic strain at the column-footing interface and the tensile strain was linearly distributed over the length of the column. However, a sharp change in the strain profile can be observed during the displacement ductility cycle at $\mu=5.5$ where the strain at 9 inches from the column base increased significantly. It is noted that the north side of the column experienced major spalling in the prior push cycle of loading with a maximum spalled height of 7 inches. It is also noted that the plastic strain near the base of the column remained almost constant for the remaining of the test duration while the plastic strain at 9 inches from the column base increases with the increase in the displacement ductility level. This can be attributed to the significant tension shift in column region resulting from the major spalling in column over the 7 inches height and the flexural cracking in the footing region. In the push direction, the strain profile was similar to the first three specimens where the plastic strain concentration can be observed at the interface region with significant strain penetration into the adjoining footing. The accumulation of the plastic strain increases at the column-footing interface region with the increase in the displacement ductility level. Strain profile in the south-east rebar was also found to be similar to the first three specimens where the plastic strain concentration was observed near the base of the column with a linear strain distribution along the height of the column. Penetration of strain into the footing region is also evident from the strain profile and increases with the increase in the displacement ductility level.
4.7 Test 5 (CR#8)

The fifth specimen in the test series designated as specimen CR#8 was tested under the subduction zone lateral loading protocol and with a varying axial loading protocol. The reinforcing details of the specimen was different than the earlier specimens where the specimen CR#8 was reinforced with 4#8 continuous longitudinal reinforcement without any splice in the plastic hinge region. The rest of the geometric and other details were same as the earlier specimens. Results obtained from specimen CR#8 are presented in the following subsections.

4.7.1 Physical Observation

For specimen CR#8, a total of three flexural crack formed at 29 inches, 36 inches and 43 inches from the column base and few hairline cracks were also observed beyond the mid-height of the column. The following loading cycle was associated with formation
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of four more flexural cracks at 17 inches, 30.5 inches, 42 inches and 65 inches from the column base. Subsequent loading cycles were associated with formation of new flexural cracks. The first sign of vertical crack was observed prior to the analytical yield displacement cycle. The horizontal cracks tend to stabilize following the effective yield displacement loading cycle and started propagating diagonally and vertically along the depth of the column section. Significant flexural-shear cracking pattern was observed for the specimen and the cracks were interconnected to form a triangular shear failure plane within the column plastic hinge zone. Also, the opening of the crack width under tensile loading was significantly higher compared to the other specimens where the column-interface opening was most prominent. The crack opening during the push cycle of loading was higher compared to the pull cycle of loading. Initiation of cover concrete spalling started prior to the residual crack width exceedance of 1.0 mm. Major spalling of the cover concrete started in the north-east corner of the column with a maximum spalled height of around 9 inches. The spalling exposed the north-east longitudinal rebar over a height of 5.5 inches. The rebar was inspected for sign of instability but was not found to have buckled yet. The following push cycle of loading resulted in buckling of the north-east rebar with a buckled length between the base of the column and the first transverse reinforcement at 6 inches. The length of the buckled bar increased from 6 inches to 7.5 inches during the next push cycle. The next loading cycle in the pull direction was associated with major spalling of cover concrete in the south-west corner of the column with a maximum spalled height of around 3 inches. Although the spalling in the pull direction was not significant but the crack opening under tensile loading was so significant that the column cross section
effectively lost the confinement effect. The maximum crack opening on the south face of the column was recorded as 7 mm at the peak displacement during the pull cycle of loading. Subsequent loading cycles were associated with further opening of the crack and extensive concrete spalling on the north side of the column. The final damage state was associated with buckling of longitudinal rebar in all four corners. However, the buckled length of the rebar was different for the north and south side of the column. In the north side of the column, the maximum buckled length for the north-east and north-west corner bar was approximately 18 inches and spanned over the first transverse reinforcement. The hook of the first tie bar in the north-west corner failed as a result of the outward pressure imposed by the buckled longitudinal rebar during the final damage state. The buckling of longitudinal rebar was observed between the spacing of first and second transverse reinforcement for the south side of the column. The rebar length between the base of the column and the first transverse reinforcement was found to remain straight without any sign of buckling. Also, extensive spalling of the cover concrete was observed for the south face of the column. Figure 4.20 shows the different damage levels observed during the test.
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4.7.2 Load-Deformation Response

The hysteretic load-deformation response and the backbone curve for specimen CR#8 is presented in Figure 4.21. For specimen CR#8, the experimental effective yield
displacement was calculated as 0.79 inches. The load-deformation response of specimen CR#8 is characterized by a stable hysteretic response with significantly wide loops at higher displacement ductility cycles. Significant strength degradation was also observed for the specimen with maximum degradation from the peak load of 54% during the last push cycle and at displacement ductility $\mu=8.2$. The peak lateral load recorded in the push direction of loading was 44.6 kip and the displacement corresponding to the peak load was 1.84 inch. In the pull direction, the measured peak load was 37.9 kip and the displacement of 1.27 inch was associated with the peak load. Experimental first yield displacement in both the push and pull direction was measured to be 0.64 inch and 0.65 inch, respectively. Corresponding first yield force was 41.3 kip in the push direction and 35.7 kip in the pull direction. Calculated average experimental effective yield displacement was 0.79 inch and the force associated with the average effective yield displacement was 40.3 kip. The ultimate displacement capacity defined as the 20% strength degradation from the peak was 4.25 inch in the push and 5.49 inch in the pull direction, respectively. Ultimate ductility in push and pull direction was $\mu=5.4$ and $\mu=7.0$, respectively. Corresponding drift ratio in the push and pull direction at the ultimate displacement capacity was calculated as 4.1 and 5.3, respectively. Description of the different damage states are presented in the following subsection.
4.7.3 Concrete Cracking

A total of three hairline crack was observed in the south face of the column during the 1st cycle of loading at target displacement ductility, \( \mu = 0.25 \). The three flexural cracks
formed at 29 inches, 36 inches and 43 inches from the column base. Lateral load and column top displacement measured at the end of this loading cycle was 24.8 kip and 0.16 inch, respectively. The following loading cycle in the pull direction resulted in formation of four new cracks in the north face at 7 inches, 30.5 inches, 42 inches, and 65 inches. The lateral load and displacement correspond to this loading cycle was recorded as 24.1 kip and 0.18 inch. The subsequent loading cycles were associated with formation of further cracks and the first sign of vertical crack was observed in the north face of the column during the first pull cycle at displacement ductility of $\mu=0.5$. At the end of displacement ductility $\mu=1.0$ the crack formation tends to stabilize, and no new crack was found in the next few loading cycles. One new flexural crack was found to form during the displacement ductility cycle of $\mu=1.5$. No new crack was found to form at the later stages of loading. However, the existing cracks were found to increase in width with almost each increasing loading cycle.

### 4.7.4 First Yield

Flexural yielding of the longitudinal rebar was first observed in the south-west corner of the column at the end of the 19th loading cycle at displacement ductility, $\mu=0.81$. It is important to note that the yielding of longitudinal rebar was observed at 15 inches from the column base whereas the yielding was first observed near the column base for earlier specimens. This can be attributed to the extensive cracking and significant crack width opening at this location that resulted higher strain. The crack width opening at 15 inches height from the column base was measured to be 0.41 mm at the end of displacement ductility cycle of $\mu=0.81$. The maximum crack width opening during this loading cycle was
found at 20 inches height from the column base and was 0.51 mm. Lateral load and displacement during the first rebar yield in the push cycle was found to be 41.3 kip and 0.64 inches, respectively. The following loading cycle in the pull direction was associated with yielding of north side longitudinal rebar at the column-footing interface and 15 inches height from the column base. Similar to the push direction, significant crack opening at 17.5 inches height from the column base was observed and was found to be 1.02 mm. The lateral load and displacement associated with pull direction yielding was measured to be 35.7 kip and 0.65 inch.

4.7.5 Concrete Spalling

Initiation of cover concrete spalling was noticed at the end of 73rd loading cycle (second push cycle) at displacement ductility, $\mu=2.04$. The load and displacement associated with the initiation of spalling was recorded as 44.1 kip and 1.61 inches. Subsequently spalling was observed during the pull cycle at the end of 76th loading cycle. The lateral load and displacement recorded during this loading cycle was 37.7 kip and 1.49 inches. The first push cycle at displacement ductility, $\mu=4.12$ was observed in the north-east corner exposing 5.5 inches of the longitudinal rebar and with a maximum spalled height of 6.25 inches. Loss of cover in the pull direction was also observed during the displacement ductility at $\mu=4.37$. Lateral load associated with the push and pull direction cover loss was 41.1 kip and 36.0 kip, respectively.

4.7.6 Transverse Reinforcement Yielding

Yielding of transverse reinforcement at 6 inches height from the column-footing interface was also observed for specimen CR#8 at the end of 94th loading cycle at
displacement ductility of $\mu=4.4$. The lateral load and displacement during this loading cycle was found to be 36.0 kip and 3.45 inches. It should be noted that, specimens with spliced bar in the plastic hinge zone was not observed to have yielding of transverse reinforcement. Loss of north side cover concrete was observed prior to yielding of the transverse reinforcement.

4.7.7 Residual Crack Width

The residual crack width measured up to the target displacement ductility level of $\mu=1.6$ was insignificant and was measured to be less than 0.1 mm. During the third pull cycle at displacement ductility $\mu=1.6$, the measured crack width for the north interface crack was recorded as 2.54 mm at peak and the residual crack width during load reversal was recorded as 0.35 mm. However, the residual crack width for flexural cracks in the column were still less than 0.1 mm. The subsequent loading cycles were associated with the increase in residual crack width for the interface opening. The first occurrence of residual crack width exceeding 1.0 mm was observed during the first push cycle at displacement ductility $\mu=2.73$ for the south interface opening. The lateral load and displacement associated with this loading cycle was recorded as 43.9 kip and 2.16 inches. The maximum residual crack width for the flexural cracks in the column was recorded as 0.76 mm at this loading cycle. The following loading cycle in the pull direction was associated with the exceedance of residual crack width for the north interface crack and the maximum residual crack width for column flexural crack was also 0.76 mm.
4.7.8 Rebar Buckling

For specimen CR#8, rebar buckling was observed for all the four longitudinal rebars and the length of buckled bar was significantly different compared to the other specimens tested. The first instance of rebar buckling was observed for the north-east corner rebar during the first push cycle at $\mu=4.65$. Major spalling in the north-east corner was observed in the previous push cycle that resulted in exposing the longitudinal rebar. The lateral load and displacement corresponding the first occurrence of longitudinal bar buckling was recorded as 39.9 kip and 3.67 inches, respectively. The following pull cycle was associated with significant spalling in the south face of the column. However, buckling of longitudinal rebar in the south face was not yet observed during this loading cycle. The push cycle at target displacement ductility of $\mu=6.5$ was associated with buckling of longitudinal rebar in the north-west corner. The buckled length of the rebar extended beyond the spacing of the first transverse reinforcement at 6 inches from the column base. Yielding of transverse reinforcing bar was recorded in earlier loading cycles and the transverse rebar was found to have failed during this loading cycle. The following pull cycle resulted in buckling of the south side longitudinal rebar. However, the buckling of the south side rebar was observed between the first and second transverse reinforcement as opposed to the base of the column and hence the buckled length spanned 12 inches between the two tie bars. The final damage state was associated with buckling of all the four rebars where the buckled length for the north side rebars were 18 inches spanning from the base of the column to the second transverse reinforcement.
4.7.9 Measured Curvature

The curvature profile for specimen CR#8 is presented in Figure 4.22 for both the push and pull direction and for different ductility level. The curvature profile in the push direction is presented with the solid lines and the curvature profile in the pull direction is presented with the dashed lines. Experimental effective yield curvatures are also presented in the figure with red dashed lines for both the push and pull direction. The curvature profile for specimen CR#8 was different than the other specimens where the distribution of curvature along the length of the column was not entirely linear. Significant concentration of plastic curvature was observed at the base of the column and also at 15 inches height from the column base. The overall spread of plastic curvature is similar to other specimens where the curvature at the base increases with the increase in displacement ductility level. The curvature in the push direction was significantly higher than the curvature in the pull direction. At displacement ductility $\mu=6.5$, the base curvature in the pull direction is smaller than the push direction with the measured values of 0.0039 rad/in and 0.0021 rad/in, respectively. The maximum curvature in the pull direction was found to be 0.0025 rad/in at displacement ductility $\mu=5.5$. Significant strength degradation resulting from bar buckling was observed at this ductility level and hence the following pull direction at $\mu=6.5$ was associated with a decrease in plastic curvature at the base of the column. Similar phenomenon was also observed in the push cycle of loading where the maximum curvature at the base of the column was recorded at $\mu=5.5$. However, the plastic curvature at 15 inches height from the column was found to have increased in plastic curvature with the increase
in the displacement ductility. This indicates the tension shift in column following major damage near the base of the column.

![Curvature profile graph](image)

Figure 4.22 Measured curvature profile for specimen CR#8

Theoretical average analytical effective yield curvature for specimen CR#8 was 0.00018 rad/in and the experimental effective yield curvature was computed as 0.00019 rad/in. The analytical first yield curvature in the push and pull direction was 0.00019 rad/in and 0.00017 rad/in, respectively. Whereas the measured experimental first yield curvature for the push and pull direction was 0.00017 rad/in and 0.00019 rad/in, respectively. The difference between the analytically obtained yield curvature values and the experimentally measured values were relatively small. The corresponding analytical yield moment for the push direction was 4619 kip-in and in the pull direction the analytical moment was 3958 kip-in. The average yield moment for both the push and pull direction was thus calculated to be 4289 kip-in. The experimental yield moment in the pull direction was slightly smaller than the analytically obtained value and was recorded as 3662 kip-in. Whereas in the push
direction, the experimentally obtained yield moment was found to be 4236 kip-in which was also slightly lower than the analytically obtained yield moment for the specimen. The experimental spread of plasticity was calculated based on the intersection of the yield curvature with the measured curvature profile. The calculated plastic hinge length for specimen CR#8 was approximately 20 inch in the push direction and 20.5 inch in the pull direction. The approximated plastic hinge length was calculated based on the linear regression analysis of the plastic curvatures up to a height of 33 inch. However, the maximum height where the plastic curvature was observed was found to be 35 inches in the push direction and 34 inches in the pull direction. The spalled concrete height in the push and pull direction was found to be 25 inch in the north face and 33 inch in the south face of the column. For specimen CR#8, the curvature ductility at failure was $\mu_\phi=11$ in the push direction and $\mu_\phi=13$ in the pull direction. The maximum curvature ductility of $\mu_\phi=21$ was measured at displacement ductility $\mu=6.5$ for the push direction and a maximum curvature ductility of $\mu_\phi=20$ was measured at displacement ductility $\mu=8$ in the pull direction.

4.7.10 Measured Strain Profile

The strain profile for the north-east and south-east rebar of specimen CR#8 is presented in Figure 4.23 for different displacement ductility level. The pull direction strain profile is shown as the dashed line whereas the push direction is presented as the solid line. The strain profile for CR#8 showed a profile similar to other specimens up to a displacement ductility level of $\mu=0.5$. During the pull cycle of loading the north side rebar was under tensile excursion and showed concentration of strain at the column-footing
interface following the effective yield displacement loading cycle and up to $\mu=2.0$. With the increase of displacement ductility level, the plastic strain tends to spread higher up along the column height and the tension shift is evident at $\mu=3.0$. The strain distribution in the adjoining footing element showed a perfectly linear profile up to the displacement ductility level of $\mu=3.0$. On the other hand, the south side rebar was under tensile excursion during the push cycle of loading and showed similar strain distribution as the north side rebar. However, the south side strain gages were available until the displacement ductility cycle of $\mu=5.5$ and shows the impact of tension shift where significant plastic strain was observed at 9 inches and 15 inches from the column base. Also, the strain distribution in the footing element showed a bilinear distribution where the plastic strain can be traced at 3 inches into the footing element. The compressive excursion of north side rebar also showed the distribution of plastic strain up to a height of 32 inches from the column base. Similarly, the south side rebar under compressive excursion showed a plastic strain distribution up to 27 inches height. It is evident that the strain distribution for the specimen with continuous rebar (CR#8) warrant higher spread of plastic strain and significant tension shift following damage near the column-footing interface.
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4.8 Test 6 (SS#8)

The sixth specimen in the test series designated as specimen SS#8 was tested under the subduction zone lateral loading protocol and with a varying axial loading protocol. The reinforcing details of the specimen was different than the earlier specimens where the specimen SS#8 was reinforced with 4#8 spliced longitudinal reinforcement with 25d_b splice length in the plastic hinge region. Results obtained from specimen SS#8 are presented in the following subsections.

4.8.1 Physical Observation

For specimen SS#8, formation of flexural crack was first observed during the first pull cycle at target displacement ductility $\mu=0.25$. A total of three flexural cracks were observed at 7 inches, 27.5 inches and 32 inches from the column base and opening of the north interface crack was also observed. The following loading cycle, second push cycle
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at target displacement ductility of $\mu=0.25$ was associated with formation of two more flexural cracks on the south face of the column. The height of the crack formation was 16.5 inches, and 55 inches from the column base. Subsequent loading cycles at target ductility of $\mu=0.25$ and $\mu=0.50$ were associated with formation of five more flexural cracks between 30 inches to 65 inches from the column base. Sign of vertical crack was observed during the first push cycle at target displacement ductility of $\mu=0.75$. Numerous vertical cracks were observed prior to the effective yield displacement cycles at $\mu=1.0$. The previously formed horizontal cracks were also found to have started propagating diagonally before reaching the effective yield displacement. By the point of effective yield displacement, a network of horizontal and vertical cracks was observed within the mid-height of the column. Significant vertical and diagonal crack propagation was observed following the effective yield displacement cycles and the cracks were concentrated around in the corner region along the length of the dowel bars. Toe crushing in the north-east corner region was observed as early as third push cycle at target displacement ductility of $\mu=1.1$. At this point, the residual crack width measured at the column-footing interface was 0.15 mm. First sign of cover concrete spalling was observed during the third push cycle at target displacement ductility of $\mu=1.50$. The residual crack width at the onset of cover concrete spalling was 0.20 mm for the south interface crack and 0.10 mm for flexural crack in the column region. Following the initiation of cover spalling in the south face of the column at target ductility of $\mu=2.4$, significant diagonal shear crack starting at the top of the north corner dowel bar and propagating toward the base of the column was observed. The following push cycle at $\mu=2.6$ was associated with the exceedance of residual crack width by more than 1.0 mm.
Sign of splice failure was observed for the north-east corner rebar in the following pull cycle of loading. Vertical cracks starting from the base of the column to the whole length of the dowel bar was prominent on the north and west face of the column. The width of the vertical crack measured at the peak load was 1.52 mm. The subsequent pull cycle at $\mu=3.0$ was associated with further deterioration of the spliced region in the north-east and north-west corner. Measured width of the vertical crack during this loading cycle was 3.56 mm. This loading cycle was also associated with sudden drop in the peak lateral load capacity of the column and the splice failure was evident from physical observation. The final damage state for the specimen was significant strength degradation resulting from extensive vertical and diagonal cracking due to the splice failure in the north-east and north-west dowel bar. The push cycle of loading was able to maintain the lateral load carrying capacity of the column. Figure 4.24 shows the different damage levels observed during the test.
4.8.2 Load-Deformation Response

The hysteretic load-deformation response and the backbone curve for specimen SS#8 is presented in Figure 4.25. For specimen SS#8, the experimental effective yield displacement was calculated as 0.70 inches. The load-deformation response of specimen SS#8 is characterized by a rapid strength degradation in the pull cycle resulting from lap splice failure. Relatively closely spaced hysteretic loops are indicative of low energy dissipation for the specimen. Significant strength degradation was also observed for the specimen with a maximum degradation of 58% from the peak load during the last pull cycle and at displacement ductility $\mu=3.22$. The peak lateral load recorded in the push direction of loading was 41.8 kip and the displacement corresponding to the peak load was 1.67 inch. In the pull direction, the measured peak load was 36.9 kip and the displacement of 1.67 inch was associated with the peak load. Experimental first yield displacement in
the push direction was 0.48 inches and, in the pull, direction was 0.30 inches. Corresponding first yield force was 36.2 kip in the push direction and 27.6 kip in the pull direction. Calculated average experimental effective yield displacement was 0.70 inches and the force associated with the average effective yield displacement was 37.5 kip. The ultimate displacement capacity defined as the 20% strength degradation from the peak was 2.03 inches in the pull direction while the push direction didn’t experience strength degradation less than 20% from the peak. Ultimate ductility in pull direction was \( \mu = 2.9 \) and the corresponding drift ratio was calculated as 2.0%. Description of the different damage states are presented in the following subsection.

(a) Cyclic hysteresis response
4.8.3 Concrete Cracking

Three hairline cracks were observed in the north face of the column during the 2\textsuperscript{nd} cycle of loading at target displacement ductility, $\mu=0.25$. Lateral load and column top displacement measured at the end of this loading cycle was 23.9 kip and 0.16 inch, respectively. The following pull loading cycle at displacement ductility $\mu=0.25$, the south column-footing interface also opened up along with a flexural crack at the column region. Lateral load and displacement associated with the pull cycle cracking was 22.3 kip and 0.16 inch. The moment associated with the first flexural crack formation was 2452 kip-in and the cracked stiffness of the column section was calculated to be 146 kip/in. The ratio of the cracked to gross section EI was found to be 0.41.
4.8.4 First Yield

Flexural yielding of the dowel bar was first observed during the pull cycle at the north-east corner dowel bar of the column at the end of the 8th loading cycle at displacement ductility, \( \mu = 0.70 \). Considerable flexural cracking was observed prior to yielding of the dowel bar. The maximum crack width during this cycle was found to be less than 0.10 mm, and the residual crack width measured was less than 0.05 mm. Lateral load and column top displacement associated with the first yield in the pull direction was 27.7 kips and 0.30 inch. Calculated yield moment for the pull direction was 2843 kip-in.

The south dowel bar at the south-east corner was found to exceed the yield strain during the 13th loading cycle at displacement ductility, \( \mu = 0.70 \). Similar to the north face, considerable cracking of the north face was also observed before yielding of the dowel bars. The maximum crack width recorded at this cycle was found to be 0.12 mm and the residual crack width following the north dowel yielding was recorded to be less than 0.05 mm. Lateral load and displacement for the first yield in the push direction was recorded as 36.2 kips and 0.48 inch.

4.8.5 Concrete Spalling

Minimal spalling of cover concrete was found for the specimen with most of the spalling being concentrated in the corner region. The maximum height of cover concrete spalling was 6 inches. However, the spalling was limited to an average depth of only 0.50 inches and none of the dowel bars were exposed at the end of the final loading cycle. Initiation of cover concrete spalling was observed in the push cycle at displacement ductility \( \mu = 1.52 \). The lateral load and column top displacement corresponding to the first
cover spalling was 40.5 kips and 1.05 inch. The maximum crack width at peak displacement level was recorded to be 2.54 mm and the residual crack width at zero force was found to be 0.20 mm. In the pull direction, cover spalling was limited to a height of only 2 inches from the column base. Initiation of spalling in the pull direction was first observed at displacement ductility of $\mu=2.40$. Associated lateral load and the column top displacement was 41.8 kips and 1.67 inch. The maximum crack width at this cycle during the peak displacement was 5.08 mm and the residual crack width at zero force was 0.76 mm.

### 4.8.6 Residual Crack Width

At the end of effective yield displacement at ductility $\mu=1.0$, the maximum recorded residual crack width in the push and pull direction was less than 0.10 mm whereas the maximum crack opening measured at the peak displacement level for these two cycles were 0.76 mm and 0.64 mm, respectively. The residual crack width measured up to the displacement ductility of $\mu=1.1$ was insignificant and was measured to be less than 0.10 mm. At the end of this loading cycle the measured residual crack width was 0.15 mm whereas the maximum crack width at the peak displacement was 1.52 mm.

Finally, the serviceability limit state based on the residual crack width of 1.0 mm was found to be exceeding at the end of 81st loading cycle at displacement ductility, $\mu=2.62$. The maximum-recorded residual crack width was found to be 1.02 mm for the south side column-footing interface crack. Lateral load and column top displacement associated with this cycle was 41.7 kip and 1.81 inch. The following push cycle was associated with the exceedance of the residual crack width by more than 1.0 mm for the north column-footing
interface crack. Last recorded residual crack width at the end of target displacement ductility $\mu=3.0$, was recorded to be 1.30 mm while splice failure was observed in the pull direction of loading.

4.8.7 Lap Splice Failure

The final damage recorded for the column was due to significant lateral strength degradation resulting from the lap splice failure of the north side rebars. The column lateral load and top displacement associated with the lap splice failure was recorded to be 25.9 kip and 2.15 inch. The previous pull cycle was associated with a column lateral load of 40 kips. The strength degradation between the two loading cycles was calculated to be 35% and was more than the specified failure strength degradation of 20% from the peak load.

4.8.8 Measured Curvature

The curvature profile for specimen SS#8 is presented in Figure 4.26 for both the push and pull direction and for different ductility level. The curvature profile in the push direction is presented with the solid lines and the curvature profile in the pull direction is presented with the dashed lines. Experimental effective yield curvatures are also presented in the figure with red dashed lines for both the push and pull direction. The curvature profile for specimen SS#8 was different than the other specimens where the distribution of curvature along the length of the column shows segmental curvature. While significant concentration of plastic curvature was observed at the base of the column but the column curvature at 25 inches (right above the end of the dowel bars) from the base was also significantly higher. This is more pronounced in the pull direction of loading compared to the push direction. This can be attributed to the large crack opening at that location and
also the effect of splice failure that led to an increased demand immediately above the splice bars. The overall spread of plastic curvature within 3 inches from the column base was similar to other specimens where the curvature at the base increases with the increase in displacement ductility level. Although the failure was apparent due to splice failure in the pull direction, the curvature at the column base in the push and pull direction was comparable. At displacement ductility $\mu=3.0$, the base curvature in the push and pull direction was measured to be 0.0012 rad/in and 0.001 rad/in, respectively. These curvature values were significantly lower than the experimentally measured curvature for other specimens.

![Graph](image)

Figure 4.26 Measured curvature profile for specimen SS#8

Theoretical average analytical effective yield curvature for specimen SS#8 was 0.00018 rad/in and the experimental effective yield curvature was computed as 0.00015 rad/in. The analytical first yield curvature in the push and pull direction was 0.00017 rad/in and 0.00016 rad/in, respectively. Whereas the measured experimental first yield curvature
for the push and pull direction was 0.00016 rad/in and 0.0001 rad/in, respectively. The difference between the analytically obtained yield curvature in the pull direction was significant whereas in the push direction same curvature values were obtained. The experimental spread of plasticity wasn’t calculated for this specimen as the curvature profile varies significantly from a typical flexural dominated column section. However, the plastic curvature was recorded to a maximum height of 36 inches from the column base. For specimen SS#8, the curvature ductility at failure was $\mu_\phi=7$ in the push direction and $\mu_\phi=6$ in the pull direction. Again, the curvature ductility at failure for this specimen was significantly lower than the other specimens tested in the experimental program.

4.8.9 Measured Strain Profile

The strain profile for the north-east and south-east rebar of specimen SS#8 is presented in Figure 4.27 for different displacement ductility level. The pull direction strain profile is shown as the dashed line whereas the push direction is presented as the solid line. The strain profile for the north side rebar showed that the plastic strain was concentrated only at the column-footing interface region. The remaining of the column length shows essentially elastic strain values, and the strain profile was found to be a linear for the entire range of the column above 10 inches. It is important to note that the lap splice failure of the north side rebar was the primary reason of failure for this specimen. It can be observed that the strain value at the interface increase with the increase in displacement up to a displacement ductility level of $\mu=2.0$. Following the splice failure at $\mu=3.0$, the strain values at the column-footing interface zone dropped which is indicative of the slip due to splice pull out. However, inelastic strain penetration into the footing can be observed for
the south side rebar and plastic strain can be found up to a distance of approximately 9 inches into the footing. The strain profile for the north side rebar was similar to the south side rebars where concentration of plastic strain can be observed at the base of the column. However, contrary to the north side rebars, a drop in strain value at the interface was not observed between the displacement ductility level of $\mu = 2.0$ to $\mu = 3.0$. This is again indicative of the fact that the push cycle of loading was able to maintain the lateral load capacity up to a displacement ductility of $\mu = 3.0$.

In the push direction, the north side rebars were under compressive loading and compressive strain values were recorded up to the displacement ductility of $\mu = 0.5$. Following the displacement cycle, the strain at column-footing interface experienced tensile strain values and the strain continue to increase with the increase of displacement ductility level. The strain values in all other locations of the column showed compressive strain during the push cycle of loading. Similar phenomenon was observed in the pull direction for the south side rebars. The maximum tensile strain at the column-footing interface during the displacement ductility cycle at $\mu = 3$ was found to be $0.032$ for the south side rebar whereas the north side rebar experienced a maximum tensile strain of $0.015$. 
4.9 **Hysteretic Envelope Comparison**

This section will compare the envelope curve for all the six test specimens and focuses on the impact of different variables for the tested specimens.

4.9.1 **Impact of Varying Axial Loading Protocol**

The first two specimens in phase I testing were devised to investigate the impact of varying axial loading protocol on seismic performance of the as built column-footing subassemblies. Specimen SC was tested with a constant axial loading protocol having an axial load of 240 kips whereas specimen SV was tested under a varying axial loading protocol with a maximum axial load of 240 kips and a minimum of 160 kips. The variation axial load was proportional to the lateral load capacity of the column at any given displacement ductility level. The envelope response obtained from the two tests are compared in Figure 4.28. The load-displacement response for both the specimens were
flexural dominated. The push direction of specimen SV experienced higher axial loading level and it can be observed that the load-displacement envelope response for the push direction of SV was comparable to that of the specimen SC. However, in the pull direction, specimen SV experienced a lower lateral strength compared to the specimen SC. This is due to the lower axial load level experienced in the pull direction of loading for SV. In terms of strength degradation, specimen SV showed lower degradation from the peak load especially in the pull direction and resulted in higher ultimate displacement level. It can also be observed that the peak lateral load was achieved earlier for the specimen SC.

![Figure 4.28 Impact of varying axial loading protocol](image)

The impact of axial load variation was also investigated with a series of monotonic moment-curvature analysis. The moment-curvature analysis was conducted for the column section with different axial load using a numerical program named Response-2000 (Bentz and Collins 2001). The yield and ultimate condition as obtained from these analyses are presented in Figure 4.29(a) through (f). It was found that the higher axial load ratio results in higher peak strength as well as higher strength degradation. On the other hand, lower
axial load ratio reduced the peak lateral capacity for the column and resulted in lesser strength degradation following the peak lateral strength.

Figure 4.29 Effect of axial load variation on (a) Yield curvature, (b) Ultimate curvature, (c) Curvature ductility, (d) Yield moment, (e) Ultimate moment and (f) Peak moment capacity
Figure 4.29(a) and (b) showed the variation of yield and ultimate curvature values along with axial load variation. It was found that the yield curvature increases linearly with the increase in axial load ratio whereas the ultimate curvature decreases almost linearly following an axial load of 100 kips which correspond to 3.6% axial load index ($0.036A_{g}f_{c}$) for the tested specimens. As a result, the difference between yield curvature and ultimate curvature reduces with the increase in axial load ratio which results in a reduction of curvature ductility for the column section. Figure 4.29(c) showed the variation of curvature ductility with respect to different axial load values. A trend similar to the ultimate curvature variation is evident where the curvature ductility reduces almost linearly with the increase in axial load. The effect of axial load on moment capacity for the column section is presented in Figure 4.29 (d) through (f), where the yield moment, ultimate moment and the peak moment is plotted against different axial load values. It can be seen that the moment capacity of the column section linearly increases with the increase in axial load values. Even though it was concluded that higher axial load decreases the curvature ductility and increases the maximum moment capacity. Results obtained from the numerical analysis was in line with the findings from the experimental results.

4.9.2 Impact of Lateral Loading Protocol

Specimen SV and LV was tested with an objective to investigate the impact of lateral loading protocol variation on the seismic performance of the representative column-footing specimens. Specimen SV was tested under Cascadia Subduction loading protocol whereas specimen LV was tested under conventional three cycle symmetric laboratory loading protocol. However, the axial loading protocol for both the specimens were kept
constant. The load-displacement response for the two specimens is compared in Figure 4.30. It can be observed that the specimen tested with conventional three cycle symmetric loading protocol (specimen LV) shows a symmetric response for the push and pull direction despite the varying axial loading protocol. This indicates that the impact of varying axial loading protocol was more pronounced for the long duration subduction loading protocol as compared to the conventional protocol. In the push direction, the peak lateral load for LV was 41.6 kips whereas for specimen SV the peak load was 46.07 kips. Similarly, the lateral load at first yield was also significantly lower for specimen LV in the push direction of loading. In the pull direction, the load-displacement response was comparable for both the specimens, but a sudden loss in lateral load carrying capacity was observed for specimen LV following the rebar fracture. More extensive damage was observed for LV and also the damage spread to a greater height compared to specimen SV.

![Figure 4.30 Impact of lateral loading protocol](image)
4.9.3 Impact of Steel Ratio and Lap Splice Detail

The second phase of experimental program focuses on the impact of design variables such as steel ratio, presence of lap splices in the plastic hinge zone and length of lap splice. Specimen MS#10, CR#8 and SS#8 were devised to investigate the effect of above-mentioned design variables for the representative bridge column-footing subassemblies. All of these four specimens were tested with the varying axial protocol and the Cascadia Subduction lateral loading protocol. Load displacement responses for all the three specimens are presented in Figure 4.31 and is compared with the specimen SV from first phase of testing. Specimen MS#10 shows the maximum peak lateral load capacity among all the four specimens as was expected due to the higher steel content. The recorded peak load for specimen MS#10 was 58.6 kips in the push direction and 45.2 kips in the pull direction. However, unlike other specimens the failure mode for specimen MS#10 is expected to be a brittle flexural failure of the spread footing where the other specimens showed either flexural failure (SV and CR#8) or lap splice failure (SS#8). The steel ratio for specimen MS#10 was 0.88% whereas the steel ratio for all other specimens were 0.55%. Hence, it can be concluded that any representative bridge column-spread footing substructure where the column steel ratio is higher than 0.55% should be carefully investigated for the footing failure condition.

Specimen CR#8 showed a relatively stable hysteretic response compared to specimen SV. However, the specimen with continuous rebar experienced significant damage and higher strength degradation was also observed. The impact of continuous rebar was more pronounced in limiting the rocking of the column at the column-footing interface
connection. As a result, the damage was spread over a greater height of the column. It is important to note that the yielding of transverse reinforcement was observed only for CR#8. This indicates that the widely spaced transverse reinforcements were more effective with continuous longitudinal rebar as compared to the lap spliced longitudinal rebar. The peak lateral load for specimen CR#8 was recorded to be 44.6 kips in the push direction and 37.9 kips in the pull direction. Compared to the other specimens the peak load was achieved earlier for specimen CR#8.

The last specimen in the series was SS#8 having a very short lap splice length of 25d₀ in the plastic hinge zone. The failure mode for the specimen was due to splice failure. However, yielding of dowel bars were observed prior to the lap splice failure. The peak lateral load recorded for the specimen was 41.8 kips in the push direction and 36.9 kips for the pull direction. Significant strength degradation was observed in the pull direction.
immediately after the peak lateral load which led to failure of the specimens. However, lower strength degradation was observed in the push direction due to the higher clamping force resulting from the higher axial loading level for this loading direction. The ultimate displacement ductility capacity for this specimen was significantly lower than the other specimens. A summary of the test results including the first yield, effective yield, peak lateral load, and the ultimate displacement capacity are presented in Table 4.1.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cycle</th>
<th>First Yield</th>
<th>Effective Yield</th>
<th>Peak</th>
<th>Ultimate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\Delta y$ (in.)</td>
<td>$F_y$ (kip)</td>
<td>$\Delta y$ (in.)</td>
<td>$F_y$ (kip)</td>
</tr>
<tr>
<td>SC</td>
<td>Push</td>
<td>N/M</td>
<td>0.68</td>
<td>40.0</td>
<td>45.8</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>0.44</td>
<td>36.6</td>
<td></td>
<td>42.5</td>
</tr>
<tr>
<td>SV</td>
<td>Push</td>
<td>0.52</td>
<td>40.9</td>
<td>0.67</td>
<td>39.6</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>0.52</td>
<td>36.2</td>
<td></td>
<td>39.6</td>
</tr>
<tr>
<td>LV</td>
<td>Push</td>
<td>0.43</td>
<td>34.2</td>
<td>0.70</td>
<td>37.1</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>0.58</td>
<td>37.5</td>
<td></td>
<td>40.7</td>
</tr>
<tr>
<td>MS#10</td>
<td>Push</td>
<td>0.75</td>
<td>55.3</td>
<td>0.76</td>
<td>48.7</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>0.74</td>
<td>43.5</td>
<td></td>
<td>45.2</td>
</tr>
<tr>
<td>CR#8</td>
<td>Push</td>
<td>0.64</td>
<td>41.3</td>
<td>0.79</td>
<td>40.3</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>0.65</td>
<td>35.7</td>
<td></td>
<td>37.9</td>
</tr>
<tr>
<td>SS#8</td>
<td>Push</td>
<td>0.48</td>
<td>36.2</td>
<td>0.70</td>
<td>37.6</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>0.30</td>
<td>27.6</td>
<td></td>
<td>36.9</td>
</tr>
</tbody>
</table>

### 4.10 Comparison with Past Test

Results obtained from past experimental tests on reinforced concrete bridge columns with similar detailing were compiled to evaluate the performance of the representative bridge components. Results were compared in terms of experimentally
obtained column capacities like peak load, ultimate displacement ductility, drift ratio etc. All of these experimental projects were conducted at InfraStructure Testing and Applied Research laboratory at Portland State University (Bazaez and Dusicka 2016b; Lopez et al. 2020; Mehary et al. 2018). Detail of the variables are presented in Table 4.2.
Table 4.2 Summary of variables considered for experimental testing of representative bridge columns

<table>
<thead>
<tr>
<th>Reference</th>
<th>Specimen/Test</th>
<th>Specimen Name</th>
<th>X-Section Geometry</th>
<th>Aspct Ratio</th>
<th>$f_c$ (ksi)</th>
<th>$f_y/f_u$ (ksi)</th>
<th>$\rho_i$ (%)</th>
<th>$\rho_t$ (%)</th>
<th>$l_0$ (Dia mm)</th>
<th>$P/f'_c$ A% (%)</th>
<th>Lateral Protocol</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Bazaez and Dusicka 2016b)</td>
<td>Column to Cap Beam/Quasi Static Cyclic</td>
<td>As built</td>
<td>Ø18&quot;-Cylindrical</td>
<td>6.2</td>
<td>4.9</td>
<td>50/75.8</td>
<td>1.2 (1-10-#5)</td>
<td>0.2 (10-#6-5)</td>
<td>40d b</td>
<td>6.7 (10-#5)</td>
<td>Subduction</td>
</tr>
<tr>
<td>(Mehary et al. 2018)</td>
<td>Column/Quasi Static Cyclic</td>
<td>C-7</td>
<td>24&quot;x2 4&quot; Square</td>
<td>4.2</td>
<td>4.4</td>
<td>70.3/100.9</td>
<td>0.8 (4-10-#5)</td>
<td>0.1 (10-#6-5)</td>
<td>28d b</td>
<td>5.9 (10-#5)</td>
<td>Laboratory</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S-7</td>
<td></td>
<td></td>
<td>4.1</td>
<td>4.1/4</td>
<td></td>
<td></td>
<td></td>
<td>6.2 (10-#5)</td>
<td>Subduction</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S-17</td>
<td></td>
<td></td>
<td>4.6</td>
<td>4.6/4</td>
<td></td>
<td></td>
<td></td>
<td>13.1 (10-#5)</td>
<td>Subduction</td>
</tr>
<tr>
<td>(Lopez et al. 2020)</td>
<td>Column/Dynamic Shake Table</td>
<td>C1C-L</td>
<td>Ø18&quot;-Cylindrical</td>
<td>5.3</td>
<td>4.7</td>
<td>51.5/79</td>
<td>1.2 (10-#5)</td>
<td>0.2 (10-#6-5)</td>
<td>25d b</td>
<td>9.0 (10-#5)</td>
<td>Loma-Prieta</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C2C-M</td>
<td></td>
<td></td>
<td>4.8</td>
<td>4.8/5</td>
<td></td>
<td></td>
<td></td>
<td>Maule</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C3C-T</td>
<td></td>
<td></td>
<td>4.9</td>
<td>4.9/2</td>
<td></td>
<td></td>
<td></td>
<td>Maule</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C4S1-L</td>
<td></td>
<td></td>
<td>4.2</td>
<td>4.2/7</td>
<td></td>
<td></td>
<td></td>
<td>Tohoku</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C5S1-M</td>
<td></td>
<td></td>
<td>4.5</td>
<td>4.5/5</td>
<td></td>
<td></td>
<td></td>
<td>Loma-Prieta</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C6S2-M</td>
<td></td>
<td></td>
<td>4.6</td>
<td>4.6/5</td>
<td></td>
<td></td>
<td></td>
<td>Maule</td>
<td></td>
</tr>
<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Tohoku</td>
<td></td>
</tr>
<tr>
<td>Current</td>
<td>Column to Footing/Quasi Static Cyclic</td>
<td>SC</td>
<td>24&quot;x2 4&quot; Square</td>
<td>4.3</td>
<td>5.8</td>
<td>69.4/96.5</td>
<td>0.5 (4-#8)</td>
<td>0.5 (4-#8)</td>
<td>47d b</td>
<td>7.1 (4-#8)</td>
<td>Subduction</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Max - 7.1 Min - Subduction</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>LV</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Laborat</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>MS#1-0</td>
<td></td>
<td></td>
<td>4.0</td>
<td>75.4/111</td>
<td>0.8 (4-#1)</td>
<td>0.1 (4-#10)</td>
<td>47d b</td>
<td>Max - 10.4 Min - Subduction</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>CR#8</td>
<td></td>
<td></td>
<td>4.0</td>
<td>71.5/100</td>
<td>0.5 (4-#8)</td>
<td>0.5 (4-#8)</td>
<td>25d b</td>
<td>Max - 10.4 Min - Subduction</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>SS#8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Max - 6.9 Min - Subduction</td>
<td></td>
</tr>
</tbody>
</table>
Key results obtained from the tests are compared with the results obtained from the current experimental program and is presented in Table 4.3.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Specimen Name</th>
<th>(\Delta_{yi}) (in)</th>
<th>(\Delta_{ye}) (in)</th>
<th>(F_{max}) (kip)</th>
<th>(\Delta_{F_{max}}) (in)</th>
<th>(\Delta_{max}) (in)</th>
<th>Ductility, (\mu)</th>
<th>Drift Ratio (%)</th>
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</thead>
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<tr>
<td>(Bazaez and Dusicka 2016)</td>
<td>As built</td>
<td>0.56</td>
<td>0.73</td>
<td>35.1</td>
<td>2.44</td>
<td>4.92</td>
<td>6.7</td>
<td>4.4</td>
</tr>
<tr>
<td>(Mehary et al. 2018)</td>
<td>C-7</td>
<td>0.78</td>
<td>0.97</td>
<td>44.3</td>
<td>2.6</td>
<td>5.98</td>
<td>6.2</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>S-7</td>
<td>0.75</td>
<td>0.99</td>
<td>40.0</td>
<td>2.4</td>
<td>4.41</td>
<td>4.4</td>
<td>4.4</td>
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<td></td>
<td>S-17</td>
<td>1.15</td>
<td>1.33</td>
<td>57.3</td>
<td>2.44</td>
<td>4.96</td>
<td>4</td>
<td>5</td>
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<td>(Lopez et al. 2020)</td>
<td>C1C-L</td>
<td>0.52</td>
<td>0.84</td>
<td>18.2</td>
<td>2.32</td>
<td>3.31</td>
<td>3.9</td>
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<td>0.85</td>
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<td>3.78</td>
<td>4.5</td>
<td>3.9</td>
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<tr>
<td></td>
<td>C3C-T</td>
<td>0.75</td>
<td>0.94</td>
<td>17.5</td>
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<td>5.3</td>
<td>4.7</td>
<td>4.7</td>
</tr>
<tr>
<td></td>
<td>C4S1-L</td>
<td>0.67</td>
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<td>3.1</td>
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<td></td>
<td>C5S1-M</td>
<td>0.59</td>
<td>1.27</td>
<td>21.6</td>
<td>2.68</td>
<td>5.28</td>
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<td></td>
<td>C6S2-M</td>
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<td>0.92</td>
<td>20</td>
<td>2.68</td>
<td>3.66</td>
<td>4</td>
<td>3.8</td>
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<tr>
<td>Current</td>
<td>SC</td>
<td>0.44</td>
<td>0.68</td>
<td>45.8</td>
<td>1.26</td>
<td>5.39</td>
<td>7.9</td>
<td>5.3</td>
</tr>
<tr>
<td></td>
<td>SV</td>
<td>0.52</td>
<td>0.67</td>
<td>46.1</td>
<td>1.77</td>
<td>5.35</td>
<td>8</td>
<td>5.2</td>
</tr>
<tr>
<td></td>
<td>LV</td>
<td>0.43</td>
<td>0.70</td>
<td>41.6</td>
<td>1.93</td>
<td>4.69</td>
<td>6.7</td>
<td>4.6</td>
</tr>
<tr>
<td></td>
<td>MS#10</td>
<td>0.74</td>
<td>0.76</td>
<td>58.6</td>
<td>2.99</td>
<td>6.04</td>
<td>8</td>
<td>5.9</td>
</tr>
<tr>
<td></td>
<td>CR#8</td>
<td>0.64</td>
<td>0.79</td>
<td>44.6</td>
<td>1.84</td>
<td>4.26</td>
<td>5.4</td>
<td>4.2</td>
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<tr>
<td></td>
<td>SS#8</td>
<td>0.39</td>
<td>0.70</td>
<td>41.8</td>
<td>1.67</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Mean</td>
<td>0.63</td>
<td>0.90</td>
<td>33.9</td>
<td>2.10</td>
<td>4.67</td>
<td>5.6</td>
<td>5.6</td>
<td>4.7</td>
</tr>
<tr>
<td>COV (%)</td>
<td>29</td>
<td>24</td>
<td>40</td>
<td>26</td>
<td>25</td>
<td>30</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>Mean (Square Column)</td>
<td>0.65</td>
<td>0.84</td>
<td>44.2</td>
<td>1.91</td>
<td>4.92</td>
<td>6.4</td>
<td>6.4</td>
<td>4.8</td>
</tr>
<tr>
<td>COV (Square Column)</td>
<td>37</td>
<td>26</td>
<td>40</td>
<td>26</td>
<td>25</td>
<td>20</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Mean (Circular Column)</td>
<td>0.63</td>
<td>1.0</td>
<td>20.4</td>
<td>2.53</td>
<td>4.49</td>
<td>4.6</td>
<td>4.6</td>
<td>4.6</td>
</tr>
<tr>
<td>COV (Circular Column)</td>
<td>14</td>
<td>22</td>
<td>26</td>
<td>11</td>
<td>30</td>
<td>26</td>
<td>26</td>
<td>26</td>
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</table>
The key objective for the comparison of the results from past studies was to develop a failure probability based on the drift ratio of the column specimens. It is important to note that the test specimens varied significantly in terms of variables considered for each of the experimental program. However, a failure probability based on the dimensionless drift ratio would provide a closer look into the global behavior of the vulnerable representative bridge substructure specifically for the columns. In order to calculate the failure probability, the ultimate drift ratio for the individual specimens were sorted from smallest to largest. The individual rank was then divided by the total number of specimens to determine the probability of failure for each of the specimens. The failure probability obtained from the past and the current experimental studies are presented in Figure 4.32.

The lowest ultimate drift ratio of 2% was obtained for the column-footing specimen tested during the current experimental program. The specimen showed a lap splice failure mode where only 25dₖ splice length was used in the column plastic hinge region. A circular column tested under dynamic shake table testing (Lopez et al. 2020) was found to have an
ultimate drift ratio of 3.1%. Based on the results from the two experimental program it can be concluded that the lowest ultimate drift ratio is associated with the lap splice failure and a short lap splice in the range of $25d_b$ is most vulnerable under seismic loading. However, lightly reinforced concrete columns with intermediate lap splice length such as within the range of $28d_b$ to $47d_b$ were able to develop a minimum ultimate drift ratio of 3.8% and a maximum ultimate drift ratio of 6%. Furthermore, results from the past tests shows that a 4% drift ratio has a 25% probability of failure. Similarly, 4.7% drift ratio was associated with a 50% probability of failure and a 5.4% drift ratio was associated with a 75% probability of failure. Hence, it can be concluded that a representative bridge column with intermediate lap splice length will fall within the range of 4% to 5.4% drift ratio. However, the failure mode and damage state will depend on design variables and the loading condition.

A general trend was also observed from the past tests that the dynamic shake table testing of the scaled down specimen showed lower average ultimate drift ratio compared to the full-scale reversed cyclic testing of the similar bridge columns. Hence, further research should be undertaken to investigate the effect of dynamic properties and loading condition on a full-scale specimen having representative detail.
Chapter 5  Performance Limit State

5.1  Introduction

The seismic design philosophy for new bridges or retrofit of existing bridges has evolved from strength-based design to a target performance-based design which is also known as “Performance Based Seismic Design” (PBSD). The performance based seismic design is defined as the design methodology to reliably achieve the targeted performance objective for a specific bridge category (importance category). Each of these performance objectives are specified for a particular hazard level, which are usually referred in terms of probability of exceedance. Moreover, the performance based seismic design methodology uses performance for a wide range of hazard levels whereas the traditional strength-based design follows a single hazard and performance level. The performance objectives are usually defined in terms of component parameters (i.e., rebar buckling) or global structural parameters (i.e., stability). Both qualitative and quantitative definitions of performance levels are in use to define specific performance objectives. Engineering limit states (i.e., material strain) are usually used to quantitatively define each aspect of the performance level.

Growing needs to define performance objectives in terms of engineering limit states has led to many studies that have resulted in a multi-level bridge design methodology. This methodology has been adopted and implemented by a few departments of transportation, such CALTRANS, Oregon DOT, and South Carolina DOT (NCHRP 2013). However, the lack of experimental data on performance limit states of representative bridge bents has limited the use of performance-based philosophy for evaluation of existing structure. In
this section, the performance metrics for performance-based seismic design and retrofit of bridges based on ODOT’s requirements are evaluated experimentally.

5.2 Two-Level Design Philosophy

Oregon DOT uses a displacement-based design philosophy with a two-level performance criterion, namely “Life Safety” and “Operational” for seismic design of new bridges. In order to satisfy the life safety performance criteria, new bridges should be designed for a 1000-year return period earthquake (7% probability of exceedance in 75 years) whereas a full rupture Cascadia subduction zone earthquake should be used to satisfy the operational performance criteria. Hence, the performance is described with two discrete performance levels and two seismic hazards, as shown in Table 5.1.

<table>
<thead>
<tr>
<th>Performance Level</th>
<th>Seismic Hazard (Return Period)</th>
<th>New Bridge Design</th>
<th>Existing Bridge Evaluation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Steel Strain</td>
<td>Concrete Strain</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Operational</td>
<td>CSZE</td>
<td>$\varepsilon_s \leq 2\varepsilon_{sh}$</td>
<td>$\varepsilon_{cc} = 0.005$</td>
</tr>
<tr>
<td>Life Safety</td>
<td>1000-year</td>
<td>$\varepsilon_s \leq \varepsilon^{R}_{su}$</td>
<td>$\varepsilon_{cc} = 0.9\varepsilon_{cu}$</td>
</tr>
</tbody>
</table>

Where, $\varepsilon_s$ is the reinforcing steel strain; $\varepsilon_{sh}$ is the reinforcing steel strain at the onset of strain hardening; $\varepsilon^{R}_{su}$ is the reduced ultimate tensile strain in the reinforcing steel; $\varepsilon_{cc}$ is the strain in the confined section of columns; $\varepsilon_{cu}$ is the ultimate concrete strain computed using Mander’s model.

*As adequate hoops are qualified those that meet the definition of “seismic hooks” in Article 8.8.9 of AASHTO Guide Specifications for Seismic Bridge Design and are spaced no more than 6 inches apart.

**As adequate lap splice is qualified those that meet the requirements of Article 5.10.8.4.3a of AASHTO LRFD Bridge Design Specifications for Class B splice.

Table 5.1 ODOT’s performance level and currently used strain limit (BDM 2021)

Figure 5.1 illustrates four-level performance criteria adapted (Deierlein and Moehle 2004) to satisfy ODOT requirements.
On the other hand, for the seismic retrofit design of existing bridges, ODOT has adopted the design philosophy according to the publication “Seismic Retrofitting Manual for Highway Structures, Part 1-Bridges” (Buckle et al. 2006). The two-level performance objectives adopted by ODOT are presented in Table 5.2. It should be noted that the performance objectives for the lower-level ground motion as adopted by ODOT differs from the seismic retrofit manual guideline (Buckle et al. 2006).
Part-I: As Built Performance and Strain Limit State

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Table 5.2 Minimum performance level for retrofitted bridges adopted by ODOT

<table>
<thead>
<tr>
<th>Earthquake Ground Motion</th>
<th>Bridge Importance and Service Life Category</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Standard</td>
</tr>
<tr>
<td></td>
<td>ASL 1</td>
</tr>
<tr>
<td>Lower-Level Ground Motion</td>
<td>PL0</td>
</tr>
<tr>
<td>Cascadia Subduction Zone Earthquake – Full Rupture</td>
<td></td>
</tr>
<tr>
<td>Upper-Level Ground Motion</td>
<td>PL0</td>
</tr>
<tr>
<td>7 percent probability of exceedance in 75 years; Return period is about 1,000 years</td>
<td></td>
</tr>
</tbody>
</table>

The stated performance levels (PL) are defined as follows –

Performance Level 0 (PL0): No minimum level of performance is recommended.

Performance Level 1 (PL1): Life safety. Significant damage is sustained during an earthquake and service is significantly disrupted, but life safety is assured. The bridge may need to be replaced after a large earthquake.

Performance Level 2 (PL2): Operational. Damage sustained is minimal and full service for emergency vehicles should be available after inspection and clearance of debris. Bridge should be repairable with or without restrictions on traffic flow.

Performance Level 3 (PL3): Fully Operational. Damage sustained is negligible and full service is available for all vehicles after inspection and clearance of debris. Any damage is repairable without interruption to traffic.

5.3 Experimental Strain Limit State

The current section examines the material strain values at different damage levels along with the displacement ductility and drift ratio to investigate the effectiveness of each...
of the engineering demand parameter for performance-based evaluation of representative bridge sub-structures.

5.3.1 Minor Damage

Minor damage levels are defined as the onset of flexural cracking and yielding of reinforcing bars. While these damage levels don’t require any retrofit or repair intervention but marks a significant change in the stiffness of the column. Similarly, yielding of the reinforcing bar indicates the excursion into plastic deformation and hence determining these damage states has some importance in terms of seismic evaluation of the existing bridges. The onset of flexural cracking was physically observed for all the specimens and the cracks were found to form in the column region for five out of the six specimens. For specimen MS#10, flexural cracks were observed in the footing as well. Table 5.3 presents the displacement ductility, drift ratio and material strain values at the onset of first flexural crack and reinforcement yielding. The instance of flexural crack appearance is usually defined in terms of the reinforcing steel tensile strain. The average tensile strain at the occurrence of first flexural crack was 0.001 with a coefficient of variation of 43%. While the tensile strain value can be used as an indicator for the cracking, but it was found that displacement ductility and drift ratio better capture the occurrence of this damage level. The average drift ratio and displacement ductility at the cracking damage level was 0.22% and 0.30, respectively. The coefficient of variation for the drift ratio and ductility was 21% and 29%, respectively.
### Table 5.3. Minor damage levels

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cycle</th>
<th>Cracking</th>
<th>First Yield</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Drift (%)</td>
<td>µ</td>
</tr>
<tr>
<td>SC</td>
<td>Push</td>
<td>0.23 0.34</td>
<td>0.0015</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>0.22 0.33</td>
<td>0.0006</td>
</tr>
<tr>
<td>SV</td>
<td>Push</td>
<td>0.23 0.35</td>
<td>0.0039</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>0.31 0.45</td>
<td>0.0018</td>
</tr>
<tr>
<td>LV</td>
<td>Push</td>
<td>0.28 0.42</td>
<td>0.0021</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>0.18 0.24</td>
<td>0.0023</td>
</tr>
<tr>
<td>MS#10</td>
<td>Push</td>
<td>0.18 0.24</td>
<td>0.0024</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>0.18 0.24</td>
<td>0.0024</td>
</tr>
<tr>
<td>CR#8</td>
<td>Push</td>
<td>0.16 0.20</td>
<td>0.0021</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>0.18 0.23</td>
<td>0.002</td>
</tr>
<tr>
<td>SS#8</td>
<td>Push</td>
<td>0.25 0.24</td>
<td>0.0016</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>0.25 0.23</td>
<td>0.0018</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>0.22 0.30</td>
<td>0.002</td>
</tr>
<tr>
<td>COV (%)</td>
<td></td>
<td>21</td>
<td>29</td>
</tr>
</tbody>
</table>

The first rebar yield was defined in terms tensile strain values recorded during the test. The yield strain was calculated as the ratio of the measured yield stress, $f_{y\text{-me}}$, and the steel modulus of elasticity of $E = 29000$ Psi. The calculated yield strain was 0.0024. The reliance on strain values to define the yielding damage level made it impossible to define the damage state qualitatively through physical observation. However, the residual crack width measured at zero displacement during load reversal was correlated and found that a width between 0.05mm – 0.1mm is indicative of the yielding damage state. The drift ratio and displacement ductility were again found to predict the first yield within reasonable limit. The average drift ratio and ductility at the first yield of rebar was found as 0.54% and 0.76, respectively with a 25% coefficient of variation for drift ratio and 20% coefficient of variation for the displacement ductility values. Average concrete strain values at first
flexural cracking and yielding of longitudinal rebar was $\varepsilon_c=0.002$ and $\varepsilon_c=0.0036$, respectively with a coefficient of variation of 40% and 45%. Significant scatter is noted in the measured concrete strain, and it is evident that these damage states cannot be correlated with the cover concrete strain.

5.3.2 Operational Performance Level

The operational performance objective is qualitatively defined as the point after which a repair intervention is required. The damage level used to define the operational performance objective is the exceedance of residual crack width by more than 1mm requiring epoxy injection and the onset of concrete cover spalling requiring concrete patching to ensure long term durability.

The residual crack width was measured at zero displacement during load reversal. The residual forces at zero displacements were negligible up to the point of residual crack width exceeding 1mm. Residual crack width was not measured for the specimen SC. For other specimens, the crack width was measured for multiple cracks in the column and the gap opening at the column-footing interface. For all the specimens, the interface opening was found to have reached the 1 mm residual width threshold prior to any other cracks. Although, the gap opening at the interface is not considered as flexural crack but the implication from the long-term durability point of view is same as other flexural cracks in the column. These opening will require epoxy injection to prevent moisture intrusion into the column and hence to prevent corrosion of the rebar. The drift ratio, displacement ductility and strain values at the onset of operational performance level is presented in Table 5.4. The average drift ratio and the displacement ductility at the exceedance of
residual crack width by 1 mm was 1.7% and 2.4, respectively with a coefficient of variation of 20% and 18%. The residual crack width damage level is usually defined in terms of the peak tensile strain reached in the preceding loading cycle. The average tensile strain at the onset of residual crack width exceeding 1 mm was $\varepsilon_s=0.024$ with a standard deviation of 0.01 and a coefficient of variation of 42%. Large scatter can be observed in the measured peak tensile strain values prior to the exceedance of residual crack width. The highest strain value at residual crack width exceeding 1 mm was observed for specimen tested under conventional three cycle symmetric lateral loading protocol. Hence, it is evident that the operational strain limit state is controlled by the Cascadia Subduction zone loading protocol. Furthermore, it was found that the average tensile strain in the push and pull direction of SV was $\varepsilon_s=0.018$ with a coefficient of variation of 11% and for LV the average strain was $\varepsilon_s=0.04$ with 11% coefficient of variation. It can thus be stated that the variation in axial load has very little significance for the tensile strain at the residual crack width damage level. The lateral loading protocol, on the other hand, was found to impact the strain values significantly. Moreover, the strain values at the onset of residual crack width damage state were considerably lower for the subduction protocol compared to the conventional three cycle symmetric loading protocol. The average minus the first standard deviation value of tensile strain was 0.014 which was lower than the generally accepted operational tensile strain limit state value of $\varepsilon_s=0.015$ for well confined bridge columns. Also, the lowest recorded strain value for the tested specimens was $\varepsilon_s=0.012$, still lower than the specified tensile strain limiting value of $\varepsilon_s = 0.015$. Hence, a limiting tensile strain value of $\varepsilon_s=0.015$ cannot be retained for the seismically sub-standard bridge columns with
representative details. A more conservative strain limit state value of 0.01 deems adequate for the representative bridge column-footing subassemblies.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cycle</th>
<th>Residual Crack Width &gt;1 mm</th>
<th>Onset of Cover Spalling</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Drift (%)</td>
<td>$\mu$</td>
</tr>
<tr>
<td>SC</td>
<td>Push</td>
<td>N/M</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>1.3</td>
<td>1.9</td>
</tr>
<tr>
<td>SV</td>
<td>Push</td>
<td>1.3</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>1.1</td>
<td>1.7</td>
</tr>
<tr>
<td>LV</td>
<td>Push</td>
<td>1.9</td>
<td>2.8</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>1.9</td>
<td>2.8</td>
</tr>
<tr>
<td>MS#10</td>
<td>Push</td>
<td>1.4</td>
<td>1.9</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>1.4</td>
<td>1.9</td>
</tr>
<tr>
<td>CR#8</td>
<td>Push</td>
<td>2.1</td>
<td>2.7</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>1.9</td>
<td>2.5</td>
</tr>
<tr>
<td>SS#8</td>
<td>Push</td>
<td>1.8</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>1.8</td>
<td>2.6</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>1.7</td>
<td>2.4</td>
</tr>
<tr>
<td>COV (%)</td>
<td></td>
<td>20</td>
<td>18</td>
</tr>
</tbody>
</table>

The onset of cover spalling is usually defined with respect to the concrete compressive strain at the column. Initiation of cover spalling was identified through physical observation and the corresponding compressive strain at the extreme face of the column was calculated from the LVDT measurements. The average compressive strain at the location of the LVDT’s were calculated by dividing the recorded displacement with the gauge length of the LVDT. The average compressive strain at the face of the column was then interpolated using a linear strain distribution and the rotation calculated from the readings of the two opposite LVDT’s. It is noted that the maximum compressive strain within the gauge length would either be higher or equal to the average strain value.
computed. The mean of the average calculated compressive strain at spalling onset was $\varepsilon_c=0.0094$ with a standard deviation of 0.005 and a coefficient of variation of 55%. The generally accepted concrete compressive strain for the operational performance criteria is $\varepsilon_c=0.004$ which is about a standard deviation away from the mean value. Hence, considering a concrete compressive strain value of $\varepsilon_c=0.004$ can be used conservatively to evaluate the operational performance criteria. However, the concrete strain at spalling for the specimen with very short lap splice (specimen SS#8) was 0.0017 and 0.0036 for the push and pull direction, respectively. Hence, the operational strain limit state of $\varepsilon_c=0.004$ should not be used for columns with short lap splices or for columns anticipated to have splice failure. The average drift ratio and displacement ductility values at the onset of spalling was 1.8% and 2.6, respectively with a coefficient of variation of 23% and 25%, respectively for the drift and ductility. Finally, it was concluded that choosing a definitive tensile and compressive strain values for the operational performance objective was not insightful due to the significant scatter in the measured data. It was rather judicious to use the above stated strain values as confirmatory to the already accepted strain limit state for the operational objective. An exceedance probability plot was developed based on measured strain data. The steel tensile and concrete compressive strain values were listed in the ascending order and were ranked from ‘1’ to ‘n’ values where n is the total number of data points. The probability of exceedance corresponding to each strain value was then computed by dividing the rank of the individual data by ‘n’ (i.e., 1/n). The exceedance probability was then plotted by linear curve fit and is presented in Figure 5.2. Existing operational tensile strain limit state of $\varepsilon_s=0.015$ was found to have 24% probability of
exceedance and the maximum measured tensile strain of $\varepsilon_s = 0.043$ was associated with 100% probability of exceedance. On the other hand, existing operational concrete compressive strain value of $\varepsilon_c = 0.004$ was also associated with a 24% probability of exceedance including the specimen with lap splice failure. The operational strain limit state excluding the specimen with lap splice failure was associated with a 0% probability of exceedance for the spalling strain of $\varepsilon_c = 0.004$. This indicates the conservatism in choosing the operational concrete limit state of $\varepsilon_c = 0.004$ when splice failure is not the predominant failure mode. The concrete strain value of $\varepsilon_c = 0.0177$ was associated with a 100% probability of exceedance at the operational performance level.

![Graphs showing probability of exceedance](image)

(a) Tensile strain at res. crack $> 1$mm  
(b) Concrete strain at onset of spalling

Figure 5.2 Exceedance probability of operational strain limit state

### 5.3.3 Life Safety Performance Level

The definition of the ‘life-safety’ performance level needs to be established qualitatively since the same damage level has been used interchangeably to define the
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Damage control and life-safety performance criteria. For instance, Goodnight et. al., (2016) defined the life-safety performance level with respect to the fracture of the previously buckled bar whereas significant core crushing and/or buckling of rebar was defined as the damage control limit state. Oregon Department of Transportation (ODOT) defines the life-safety performance level in terms of rebar buckling or fracture and was adopted from the component performance level prescribed by Hose et. al, (2000). Clearly, the distinction between the damage control and the life-safety performance level is not present in ODOT definition. While the difference in the limiting strain values might be small between the damage control and life-safety limit state but the implication in the repairability of the structure is significant. The life safety limit state is defined to prevent complete collapse of the structure and if reached feasible repair becomes expensive and often uneconomic (Priestly, MJN 2000). In contrary, the damage control limit state is specifically defined to limit the damage to an extent where repair is feasible and economically viable. So, it was prudent to clearly define the ‘life-safety’ performance level prior to the evaluation of the limiting strain values.

The life-safety concrete compressive strain is defined as the peak compressive strain on the face of the column in the preceding displacement ductility cycle to the onset of complete cover loss that exposes the dowel bar. This varies from the qualitative definition of the damage control performance level for well confined bridge columns where crushing of core concrete is observed, and the core concrete strain as opposed to the strain at the column face is used to define the damage control compressive strain limit state (Goodnight et. al, (2016), Lehman et. al., (2004), Priestly, MJN (2000)). However, a similar
definition would be impractical for existing bridge columns where lack of confinement essentially diminishes the concept of core concrete. Similarly, the peak tensile strain in the preceding ductility cycle at the onset of complete cover loss was used to define the life-safety steel tensile strain. It was evident from all the three specimens that the bar buckling is contingent upon the loss of cover concrete that exposes the unsupported length of the dowel bars between the first transverse reinforcement and the top of the footing. Buckling of dowel bars in all the cases were observed over this unsupported length of 6 inches ($\frac{1}{4}$ of the column cross sectional depth, $h$) in the subsequent loading cycle. The objective here was not to predict the tensile strain at the onset of bar buckling rather to limit the strain value to a stricter level that will prevent reaching the buckling strain. Again, the definition varies from the damage control steel strain limit state of well confined bridges where the peak tensile strain in the rebar prior to bar buckling is used to define a tension-based bar buckling strain limit state.

The tensile strains at center of the bar location and the concrete compressive strains at the face of the column were calculated from the LVDT measurements and are presented in Table 5.5. The strain data recorded with the strain gages were erroneous at higher displacement ductility level which led to the calculation of average tensile strain values based on the LVDT measurements. The fixed end rotation component at the column base was filtered out prior to the calculation of the tensile strain from LVDT measurements. It is important to note that life safety strain values presented here only represent the first phase of experimental studies. The specimens from the second phase of studies were excluded due to the very different failure mode for these specimens (footing cracking, lap
splice failure, etc.). A separate set of concrete strain ($\varepsilon_{c-cl}$) for the cover loss and steel tensile strain ($\varepsilon_{s-rb}$) for the rebar buckling is shown in Table 5.5. These are the strain values at the onset of complete cover loss and rebar buckling. However, the strain values are presented with an intention to compare the actual strain at the onset of damage levels and the limiting strain values as per the adopted qualitative definition of the life-safety performance level. Interestingly, the rebar strain at the onset of buckling was associated with less scatter with a coefficient of variation of 17% compared to the adopted limiting strain values having a coefficient of variation of 25%.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cycle</th>
<th>Cover Loss &gt; 0.25h*</th>
<th>Rebar Buckling</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Drift (%) $\mu$ $\varepsilon_{c-cl}$ $\varepsilon_{c}$ $\varepsilon_{s}$</td>
<td>Drift (%) $\mu$ $\varepsilon_{c}$ $\varepsilon_{s-rb}$ $\varepsilon_{s}$</td>
</tr>
<tr>
<td>SC</td>
<td>Push</td>
<td>5.1 8 0.022 0.014 N/M</td>
<td>6.2 9.3 0.057 N/M N/M</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>4.7 7 0.034 0.013 0.09</td>
<td>5.7 8.7 0.074 0.115 0.085</td>
</tr>
<tr>
<td>SV</td>
<td>Push</td>
<td>5.3 8 0.047 0.017 0.07</td>
<td>5.3 8.1 0.047 0.119 0.096</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>5.1 8 0.067 0.015 0.06</td>
<td>N/A N/A N/A N/A 0.064</td>
</tr>
<tr>
<td>LV</td>
<td>Push</td>
<td>3.8 6 0.023 0.012 0.11</td>
<td>5 7.4 0.041 0.135 0.070</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td>3.8 6 0.019 0.015 0.08</td>
<td>5.1 7.5 0.047 0.087 0.050</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>4.6 7 0.035 0.014 0.08</td>
<td>5.5 8.2 0.053 0.114 0.073</td>
</tr>
<tr>
<td>COV (%)</td>
<td></td>
<td>15 16 53 12 22</td>
<td>9 10 24 17 25</td>
</tr>
</tbody>
</table>

Table 5.5 Life-Safety performance level

$\varepsilon$ is the column cross sectional depth

The average drift ratio and displacement ductility cycle corresponding to the concrete cover loss damage level was 4.6% and 7.0 with a coefficient of variation of 15% and 16%, respectively. The lowest recorded drift and ductility was found for specimen LV with the damage level reaching at 3.8% drift and at $\mu$=5.6. It is also noted that the push and pull direction reaches the damage level almost at the same ductility level for all the three specimens indicating no significant influence of the axial load variation on the damage.
state. The average concrete compressive strain at the face of the column was $\varepsilon_c = 0.014$ with a COV of 12%. A lower limit of the concrete compressive strain of $\varepsilon_c = 0.01$ which corresponds to average minus two standard deviation value can be considered as the life-safety concrete limiting strain. The average drift ratio at the rebar buckling damage level was 5.5% with a COV of 9%. Whereas the average displacement ductility at rebar buckling was 8.2 and the COV was 10%. The average limiting tensile strain was $\varepsilon_s = 0.073$ with a COV of 25%. It is evident that a significant scatter exists in the data representing the limiting tensile strain. The minimum calculated rebar tensile strain of $\varepsilon_s = 0.05$ was obtained for specimen LV whereas the maximum strain of $\varepsilon_s = 0.096$ was for specimen SC. The exceedance probability of the life-safety concrete and steel strain limit state are presented in Figure 5.3. The steel strain value $\varepsilon_s = 0.04$ was associated with a 2% probability of exceedance and the maximum measured strain value of $\varepsilon_s = 0.096$ was associated with 100% probability of exceedance. As for the life-safety concrete limit state, $\varepsilon_c = 0.011$ was associated with 0% probability of exceedance $\varepsilon_s = 0.017$ with 100% probability of exceedance.
5.4 Comparison with Past Tests

In this section, results obtained from past research at iSTAR are summarized and are compared with the results obtained from the current project.

5.4.1 Square RC column [Mehary & Dusicka (2015)]

Mehary and Dusicka (2015) experimentally investigated the performance of full-scale square reinforced concrete columns consisted of four test specimens intended to represent typical bridge columns in the state of Oregon as illustrated in Figure 5.4. All the four specimens have same material properties, cross-sectional dimensions and reinforcement ratios and were tested under quasi-static reverse cyclic loading. The variables considered for the testing program were the lateral loading protocol (conventional laboratory and subduction loading protocol), applied axial loading (8% to 17% of the gross cross-sectional strength) and the column conditions (as built and retrofitted).

Each column specimen consisted of 4 - #10 longitudinal reinforcement on four corners with #3 stirrups with 90° hooks at 12 inches center to center spacing and 2 inches of clear cover concrete confining the column core. There are dowels that extend through the footing to 36 inches (914 mm) from the top of the footing. The longitudinal steel extended through the stubs to 15 mm from the end.

Normal weight concrete was used to construct the test specimens with a target 28-day strength of 3500 psi (24.1 N/mm²). All reinforcing steel used to construct the test
specimens consisted of Grade 60 deformed bar conforming to the American Society of Testing and Materials (ASTM) Designation A615.

![Test setup and geometry of RC column specimen](image)

Figure 5.4 Test setup and geometry of RC column specimen [Mehary & Dusicka (2015)]

The following figures show the RC square column performance during the experiments:

![Force-Displacement hysteresis curve](image)

![Force-Displacement envelope](image)

Figure 5.5 RC square column performance. (a) Force-Displacement hysteresis curve, (b) Force-Displacement envelope [Mehary & Dusicka (2015)]
The following table shows the RC square column strain limit state during the experiments:

<table>
<thead>
<tr>
<th>Level</th>
<th>Limit State</th>
<th>Steel Strains ($\varepsilon_s$)</th>
<th>Concrete* Strains ($\varepsilon_c$)</th>
<th>% Drift</th>
<th>Ductility ($\mu$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Cracking</td>
<td>0.00017</td>
<td>0.0004</td>
<td>0.26</td>
<td>0.4</td>
</tr>
<tr>
<td>II</td>
<td>First Yield</td>
<td>0.0024</td>
<td>0.0016</td>
<td>0.46</td>
<td>0.8</td>
</tr>
<tr>
<td>III</td>
<td>Effective Yield</td>
<td>x</td>
<td>x</td>
<td>0.60</td>
<td>1</td>
</tr>
<tr>
<td>IV</td>
<td>Onset of Spalling</td>
<td>x</td>
<td>x</td>
<td>0.86</td>
<td>1.4</td>
</tr>
<tr>
<td>V</td>
<td>Buckling/Rupture</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
</tbody>
</table>

*The extreme concrete compressive strains of the columns were obtained using curvature data.

Strain for performance levels III, IV and V could not be computed due to the failure of the strain gauges prior to these levels. There also was rocking at the base of the column where a cold joint between the column and footing exists which made acquiring those strains impossible.

5.4.2 **RC Bent [Bazaee (2017)]**

In this section, the performance of a half scale RC bridge bent retrofitted utilizing Buckling Restrained Braces (BRBs) is assessed by presenting and discussing the steel-reinforcement strains and concrete strains. The experimental program consisted of three tests evaluating half-scale models of a RC bridge bent as illustrated in Figure 5.6. The first two experiments consisted of different BRBs options in an effort to assess the influence of BRB stiffness on the overall structural performance. In the third test, the bent was evaluated in the as-built non-retrofitted condition, hereinafter referred to as “As-built”.

The longitudinal reinforcing steel used to construct the test specimens consisted of Grade 40, $f_y = 40$ ksi, $f_u = 60$ ksi, deformed bar conforming to the American Society of Testing and Materials (ASTM) designation A615. The measured yield stress for the longitudinal reinforcement was 50 ksi. The transverse steel consisted of deformed wire D5 conforming the ASTM A496.

Normal weight concrete was used to construct the test specimens with a target 28-day strength ($f'_c$) of 3.3 ksi. Standard compression testing of 6-inch by 12-inch concrete cylinders was performed at 7-day, 28 days and at the day of test completion. The day of testing the compressive strength was 4.8 ksi approximately.

The following figures show the RC bent performance during the experiments:
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Figure 5.7 RC bent performance. (a) Force-Displacement hysteresis curve, (b) Force-Displacement envelope [Bazaez (2017)]

The following table show the RC bent strain limit state during the experiments:

<table>
<thead>
<tr>
<th>Level</th>
<th>Limit State</th>
<th>Steel Strains ($\varepsilon_s$)</th>
<th>Concrete** strains ($\varepsilon_c$)</th>
<th>% Drift</th>
<th>Ductility ($\mu$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Cracking</td>
<td>0.0008</td>
<td>0.0007</td>
<td>0.21</td>
<td>0.3</td>
</tr>
<tr>
<td>II</td>
<td>First Yield</td>
<td>0.0017</td>
<td>0.0012</td>
<td>0.46</td>
<td>0.8</td>
</tr>
<tr>
<td>III</td>
<td>Effective Yield</td>
<td>0.0020</td>
<td>0.0017</td>
<td>0.60</td>
<td>1</td>
</tr>
<tr>
<td>IV</td>
<td>Onset of Spalling</td>
<td>0.010</td>
<td>0.0042</td>
<td>0.99</td>
<td>1.7</td>
</tr>
<tr>
<td>V</td>
<td>Buckling/Rupture</td>
<td>0.048</td>
<td>0.0080</td>
<td>2.87</td>
<td>4.8</td>
</tr>
</tbody>
</table>

**The extreme concrete compressive strains of the columns were obtained using the results from the strains in the reinforcing steel and a linear strain profile for a circular section. The actual values of concrete strain in the confined section are expected to be lower since these values are the maximum compressive strain and not the strain in the confined section.

5.4.3 Circular RC Column [Dusicka & Lopez (2016)]

In this section, the dynamic performance of scaled circular RC bridge columns is assessed by presenting and discussing the steel-reinforcement strains and concrete strains.

The experimental program consisted of six test specimens intended to represent scale models of typical circular bridge columns as illustrated in Figure 5.8. All six specimens
have the same material properties, cross-sectional dimensions, and reinforcement ratios. The variables in the testing program were the ground motion duration and lap splice length.

The longitudinal reinforcing steel used to construct the test specimens consisted of Grade 40, $f_y = 40$ ksi, $f_u = 60$ ksi, deformed bar conforming to the American Society of Testing and Materials (ASTM) designation A615. The measured yield stress for the
longitudinal reinforcement was 50 ksi. The transverse steel consisted of deformed wire D5 conforming the ASTM A496.

Normal weight concrete was used to construct the test specimens with a target 28-day strength ($f'_c$) of 3.3 ksi. Standard compression testing of 6-inch by 12-inch concrete cylinders was performed at 7-day, 28 days and at the day of test completion. The day of testing the compressive strength was 4.8ksi approximately.

The following figures show the circular RC column performance during the experiments:

![Figure 5.9](image)

Figure 5.9 Circular RC column performance. (a) Force-Displacement hysteresis curve, (b) Force-Displacement envelope [Dusicka & Lopez (2016)]

The following table show the RC bent strain limit state during the experiments:

<table>
<thead>
<tr>
<th>Table 5.8 Bridge performance parameters (Limit States) for circular RC columns [Dusicka &amp; Lopez (2016)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>I</td>
</tr>
<tr>
<td>II</td>
</tr>
<tr>
<td>III</td>
</tr>
<tr>
<td>IV</td>
</tr>
</tbody>
</table>
5.4.4 *Square RC Column [Current Project]*

The current experimental program investigating seismic performance of square reinforced concrete columns-foundation sub-assembly consisted of three test specimens intended to represent full-scale models of typical bridge column-footing assembly. All the specimens have the same material properties and cross-sectional dimensions. The variables in the testing program were the lateral and axial loading protocol, splice length and presence of lap splice in the plastic hinge zone. The longitudinal reinforcement in each prototype column consisted of either 4 No. 8 or 4 No. 10 bars on four corners with No. 3 stirrups with 90° hooks at 12 inches center to center spacing and 2 inches of clear cover concrete confining the column core. There are dowels that extend through the footing to 47d_b or 25d_b from the top of the footing.

Normal weight concrete was used to construct the test specimens with a target 28-day strength of 3300 psi (22.75 N/mm²). Standard compression testing of 6-inch by 12-inch concrete cylinders was performed at 7-day, 28 days and at the day of test completion. The 28-day compressive strength was 4.25 ksi for the column and 5.1 ksi for the footing. All reinforcing steel used to construct the test specimens consisted of Grade 60 deformed bar conforming to the American Society of Testing and Materials (ASTM) Designation A615.
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The concrete and steel strain values were computed for associated damage level and were finally used to define the performance limits based on the computed strain values. Lateral force and displacement associated with different damage levels are presented in Table 5.9 for the three tested specimens.

Table 5.9 Bridge performance parameters (Limit States) for square RC column-footing subassemblies

<table>
<thead>
<tr>
<th>Level</th>
<th>Limit State</th>
<th>Steel Strains ($\varepsilon_s$)</th>
<th>Concrete** strains ($\varepsilon_c$)</th>
<th>% Drift</th>
<th>Ductility ($\mu$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Cracking</td>
<td>0.001</td>
<td>0.002</td>
<td>0.22</td>
<td>0.30</td>
</tr>
<tr>
<td>II</td>
<td>First Yield</td>
<td>0.0025</td>
<td>0.0036</td>
<td>0.54</td>
<td>0.76</td>
</tr>
<tr>
<td>III</td>
<td>Effective Yield</td>
<td>-</td>
<td>-</td>
<td>0.68</td>
<td>1.0</td>
</tr>
<tr>
<td>IV</td>
<td>Onset of Spalling</td>
<td>0.026</td>
<td>0.0094</td>
<td>1.8</td>
<td>2.6</td>
</tr>
<tr>
<td>V</td>
<td>Buckling/Rupture</td>
<td>0.073</td>
<td>0.014</td>
<td>4.7</td>
<td>5.7</td>
</tr>
</tbody>
</table>

**The extreme concrete compressive strains of the columns were obtained using the results from the strains in the reinforcing steel and a linear strain profile for a circular section. The actual values of concrete strain in the confined section are expected to be lower since these values are the maximum compressive strain and not the strain in the confined section.

5.5 Experimental Plastic Hinge Length

The performance based seismic design and evaluation approach uses the curvatures corresponding to the strain limit state values to compute the lateral displacement using an equivalent curvature distribution model. The plastic hinge method proposed by Priestly et al. (2007) is widely accepted for modeling the reinforced concrete column responses based on a simple moment curvature analysis. However, the proposed equivalent rectangular curvature distribution within the plastic hinge length does not account for actual profile of plastic curvature distribution rather uses as a numerical substitution to compute the lateral displacement resulting from flexural deformation and fixed end rotation due to strain penetration (Goodnight et al. 2016b). While the method is useful to capture the lateral load
displacement response of concrete columns but lacks accuracy in determining the
displacement corresponding to the onset of performance limit state (Goodnight et al. 2014).

A modified plastic hinge method for well confined circular concrete bridge columns was
proposed by Goodnight et al. (2016b) where the rectangular equivalent plastic hinge length,
$L_p$ was replaced by a triangular plastic curvature profile length, $L_{pr}$. Furthermore, the strain
penetration component of the deformation was separated from the plastic hinge expression.

The expression of plastic method proposed by Priestly et al. (2007) is presented in equation
(5.1) and expression proposed by Goodnight et al. (2016b) are presented in equation (5.2)
through (5.4).

\[
L_p = kL_c + 0.022f_{ye}d_b \geq 0.044f_{ye}d_b \quad (5.1)
\]
\[
L_{pr_t} = 2kL_c + 0.75D \quad (5.2)
\]
\[
L_{pr_c} = 2kL_c \quad (5.3)
\]
\[
k = 0.2\left(\frac{f_u}{f_y} - 1\right) \leq 0.08 \quad (5.4)
\]

Here, $L_p$ is the plastic hinge length (mm), $f_{ye}$ is the expected yield stress of the
longitudinal rebar (MPa), $L_c$ is the clear height of the column (mm) and D is the diameter
of circular columns (mm). The equations contain different expression for tensile and
compressive plastic hinge region length, $L_{pr_t}$ and $L_{pr_c}$ to be used with the limiting rebar
tensile strain and concrete compressive strain values respectively.

The experimental spread of plasticity was calculated following the
recommendation of Hines et al. (2004). The procedure is explained in Figure 5.10(a) for
the push direction of specimen SV where the plastic section curvature profiles at different
ductility level are plotted along the column height and the least square regression lines at
each ductility level are presented with the linear dashed line. The triangular yield curvature profile is presented with the red dashed line. The intersection of this triangular yield curvature line and the regression line representing the plastic curvature distribution at each ductility level is defined as the plastic hinge region length, $L_{pr}$. The section curvature at the interface of the column-footing were also extracted from linear extrapolation of the regression lines. The plastic hinge region lengths, $L_{pr}$ starting at the effective yield displacement ductility cycle are plotted against the curvature ductility in Figure 5.10(b). The plastic hinge region length was found to increase with the curvature ductility. A logarithmic relationship was found to correlate well with the data. The three plastic hinge length expression from Goodnight et al. (2016b) and Priestly et al. (2007) were also plotted to compare the adequacy of the hinge length expression. The compressive plastic hinge length expression, $L_{prc}$ was found to predict the upper bound of the measured spread of plasticity with adequate accuracy. It is noted that the Goodnight et al. (2016b) expression for the plastic hinge length was based on the upper bound data of the measured data extent of plasticity. On the other hand, the tension based plastic hinge length $L_{prt}$ grossly overestimates the spread of plasticity for the specimen. This was due to the incorporation of tension shift in the $L_{prt}$ expression which was not prevalent for the existing bridge columns with representative details. The plastic hinge length expression from Priestly et al. (2007) was found to underestimate the spread of plasticity and was inadequate to predict the upper bound of the measured extent of plasticity in the specimen. It should be noted that the Priestly et al. (2007) expression plotted here uses only the first part of expression $(kL_c)$ that accounts for the moment gradient and not the strain penetration effect. It was
judicious to compare only the effect of moment gradient since strain penetration component was filtered from the curvature data presented here.

![Experimental strain penetration length](image)

Figure 5.10: Experimental spread of plasticity

The experimental strain penetration length for specimen SV was computed using the strain data recorded with the gages placed into the footing region and near the column-footing interface region. A bilinear strain distribution into the footing was used and the length of strain penetration was calculated by extrapolating to the location where the strain value is zero. The experimentally obtained strain penetration length for specimen SV are plotted against the curvature ductility and is presented in Figure 5.11.
The average length of strain penetration was 252 mm (10.1 inches) with a coefficient of variation of 21.5%. A significant scatter in the data is evident from the plot and the high coefficient of variation especially at the higher displacement ductility levels. However, the plastic hinge length expression proposed by both Goodnight et al. (2016b) and Priestly et al. (2007) doesn’t directly use the actual length of strain penetration into the adjoining member. Instead, a constant equivalent strain penetration length, $L_{sp}$ is approximated to compute the fixed end rotation resulting from strain penetration. The expression to compute column top displacement resulting from the fixed end rotation due to strain penetration component is proposed by Goodnight et al. (2016b) and is presented in equation (9). The experimental equivalent strain penetration length $L_{sp}$ was calculated by equating the two sides of equation (5.5) and therefore dividing the experimentally measured slip rotation, $\theta_{sp}$ with the effective base curvature $\phi_{base}$. The equivalent strain penetration length $L_{sp}$ was found to remain nearly constant with the curvature ductility.
The average $L_{sp}$ was 245 mm (9.8 inches) with a coefficient of variation of only 3%. The equivalent strain penetration length calculated using the Priestly et al. (2007) was found to be 264 mm (10.5 inches) and is also presented in Figure 5.11 as a reference line. The experimentally obtained $L_{sp}$ and the expression proposed by Priestly et al. (2007) was in close agreement. Hence, it was concluded that the existing equivalent strain penetration length expression from Priestly et al. (2007) along with the strain penetration model proposed by Goodnight et al. (2016b) can be adequately used to compute the column top displacement resulting from the bond slip contribution.

$$\Delta_{sp} = \theta_{sp}L_c = L_{sp}\phi_{base}L_c \quad (5.5)$$

Finally, the shear deformation component of the lateral displacement should be considered independently and can be calculated using Equation 5.6 (Moehle 2015). Here, $V$ is the shear force, $A_v$ is the effective shear area and can be approximated as $A_v = (5/6)A_g$ where $A_g$ is the gross cross-sectional area and $G_{eff}$ is the effective shear modulus equivalent to $0.2E_c$. However, the lateral displacement resulting from the shear deformation was comparably insignificant.

$$\Delta_{v} = VL_c/A_v G_{eff} \quad (5.6)$$

The set of equations that can be used to predict the lateral load deformation response representative bridge columns in conjunction with the results of analytical moment-curvature analysis are presented in equation 5.7 through equation 5.13. These set of equations are based on a triangular curvature distribution and were adopted from Goodnight et al. (2016b).

$$\Delta_{e} = \phi_{base}L_c^2/3; \quad \text{when,} \phi_{base} < \phi_y \quad (5.7)$$
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\[
\Delta_e = \phi' \left( \frac{M_n}{M'_y} \right) L_c^2 / 3; \quad \text{when, } \phi_{base} \geq \phi_y \tag{5.8}
\]

\[
\Delta_p = \phi_p \left( \frac{L_{pr}}{2} \right) \left( L_c - \frac{L_{pr}}{3} \right); \quad \text{when } \phi_{base} > \phi_y \tag{5.9}
\]

\[
\phi_p = \phi_{base} - \phi'_y \left( \frac{M_n}{M'_y} \right) \tag{5.10}
\]

\[
\Delta_{sp} = L_{sp} \phi_{base} L_c \tag{5.11}
\]

\[
\Delta_V = V L_c / A_v G_{eff} \tag{5.12}
\]

\[
\Delta_r = \Delta_e + \Delta_p + \Delta_{sp} + \Delta_V \tag{5.13}
\]
Chapter 6 – Full-Scale Experimental Evaluation

6.1 Introduction

The Seismically substandard existing bridges in the Pacific Northwest of United States are typically characterized by multicolumn bridge bents, which can develop damage in the column plastic hinge regions near the foundations and the bent beams (Bazaeez and Dusicka 2016b; Murtuz et al. 2020). The damage can range from rebar yielding and concrete cracking to more severe lap-splice pull-out or longitudinal rebar buckling and fracture (ElGawady et al. 2010; Lopez et al. 2020; Mehary et al. 2018; Murtuz et al. 2020). Yet, the collapse mechanism continues to depend on the component's ability to carry gravity loads. Numerous bent and column experiments along with observations from post-earthquake reconnaissance have shown that collapse is not inevitable just because the lateral system is damaged (Bazaeez and Dusicka 2016b; Lopez et al. 2020; Mehary et al. 2018). Despite severe degradation of the lateral strength in specific areas, bridges can continue to carry design level gravity loads as the gravity structure can remain sufficiently damage free. Hence, restoring the lateral load path would essentially restore the functionality of the bridges.

Repair objective of the existing bridges depend on design details and seismic performance of as built condition. Depending on the deficiencies, the repair can be implemented not only to restore the strength and stiffness of the existing columns but also to enhance flexural capacity, improve displacement ductility or change the failure mode from brittle shear failure to ductile flexural failure (He et al. 2015). Conventional seismic
repair methods such as repairing of cracked concrete with epoxy injection, encasing the column in concrete jacket (Bett et al. 1988), steel jacketing (Chail et al. 1991), FRP wrap (Chang et al. 2004; He et al. 2013; Rutledge et al. 2014; Saadatmanesh et al. 1997; Sheikh and Yau 2002; Vosooghi and Saiidi 2013), active confinement with smart materials (Shin and Andrawes 2011), etc. aims to restore strength and stiffness to the damaged zones. These methods were found to be most effective for cases of low damage, whereby the steel reinforcement cage remains largely intact. More involved procedures are needed for cases of loss of lap splice, buckled or fractured rebar or merely loss of confidence at the remaining low cycle fatigue capacity for subsequent earthquakes. Most of the past research focused on restoring rebar continuity (Shin and Andrawes 2011), addition of new longitudinal reinforcement (Lehman et al. 2001), applying externally bonded longitudinal reinforcement (He et al. 2013), plastic hinge relocation (Rutledge et al. 2014; Wu and Pantelides 2017), etc. for continuity prior to the encasement or wrap. While effective at restoring the column capacity, there are three significant issues with these approaches:

a) restoring rebar continuity is labor intensive resulting in lengthy and potentially costly repairs that makes it unsuitable for rapid repair approach,
b) the affected area can be damaged again in an aftershock requiring new significant repairs, and
c) repair may result in increased strength and stiffness that would likely shift failures to other parts of the bridge under future earthquake demands.

An alternative post-earthquake repair method is proposed that can be rapidly implemented and that also has the potential for increasing the resilience for future shaking.
The method utilizes a nonemulative column-to-footing connection with externally mounted brackets having supplemental energy dissipating devices, to achieve a low damage system through dissipative controlled rocking (DCR) concept. The use of DCR concept has been successfully implemented for bridge pier system (Mander and Cheng 1997; Marriott et al. 2009, 2011; Mashal 2015; Mashal and Palermo 2019b; Palermo et al. 2004, 2007; Solberg et al. 2009; Thonstad et al. 2017; White and Palermo 2016), steel braced frame system (Eatherton et al. 2014a; b), timber wall system (Sarti et al. 2016a; b), concrete filled steel tube bridge columns (Zhang et al. 2021) etc. for construction of new generation damage resistant structures and repair/retrofit of existing structure to ensure resilience through damage avoidance in future earthquake.

Researchers at Portland State University have developed a similar concept for retrofit of slender equipment support structures (Palnikov 2000) and a similar approach has also been proposed for new precast bridge construction in New Zealand (Mashal et al. 2014). The proposed repair measure would be best suited for bents that had significant damage, but not lost gravity capacity. This is expected to encompass most of the existing bridges in the Pacific Northwest built prior to 1970’s. Damage outside of the columns is possible for vulnerable bridge types in Oregon and repairing for strength in those components can be relatively conventional. The difficulty is in repairing for ductility and providing future resiliency, which this rapid approach would offer. The proposed methodology incorporates externally attached ductile fuses to bypass the damaged zone and restore the lateral capacity. The advantage of this approach is bypassing the internal rebar continuity within the damaged zone significantly simplifies the repair, which is made
in part by saw cutting cover depth around the perimeter and removing loose debris. The number of replaceable fuses and their individual capacity then control the lateral behavior, leaving the rest of the repair to be relatively generic and conventional. Following capacity design, subsequent earthquake damage would be forced into the replaceable fuses and thereby provide significant resilience in the future.

The proposed research aims to develop, and experimentally validate a practical post-earthquake repair methodology that can be rapidly implemented and that incorporates low damage earthquake resilience for future shaking. In order to evaluate the proposed repair method, damaged components of a typical bridge bent were repaired and tested under reversed cyclic lateral loading representative of long duration Cascadia Subduction zone earthquake demand.

6.2 Detailing of the Connection

The dissipative controlled rocking (DCR) connection was implemented for the as built specimen MS#10. The reinforcing and geometric details of the column and the footing subassembly are discussed in 0 and the performance of the as built specimen under reversed cyclic lateral loading is discussed in Section 4.6 of Chapter 4. The column consisted of 4-#10 longitudinal reinforcing bar in the four corners with 47d₀ length splice in the plastic hinge zone. The transverse reinforcement consists of #3 bar spaced at 12 inches c/c. The footing consisted of only one layer of reinforcing at the bottom. The final damage state for the specimen was associated with major spalling of column cover concrete without rebar buckling and significant flexural cracking of the spread footing without failure. Hence, the repair objective was to ensure the stability of the spread footing as well as to meet the
strength and ductility requirement of the column. Details of the DCR connection between the column and the footing is presented in Figure 6.1.
The DCR connection was designed with two components where one being the column confinement and the other is the externally attached energy dissipating arrangement. The column confinement was achieved with carbon fiber reinforced polymer (CFRP) wrapping of the column plastic hinge zone. A total of four segments of the plastic wraps were used to confine the column. It is noted that a single segment of CFRP wrap covering the entire plastic hinge length of the column would have been ideal but four separate segments were chosen to accommodate the anchor rods used for attaching the external dissipating device arrangement. Each of these four segments were wrapped with a total of 3 layers of Tyfo®SCH-41 CFRP having unidirectional fibers and with a nominal
thickness of 0.04 inch. It was determined that the three layers of CFRP would provide a displacement ductility of 9.3 for the confined column plastic hinge region. The height of the different segment of CFRP layers were slightly different.

The externally attached energy dissipating device was again arranged in two parts one connecting to the top of the footing and the other connecting to the column face. Each of this part consists of a hold down made from a vertical angle section and the U-shaped flexural plate. The footing side hold down arrangement consists of a 36 inches vertical angle section (L4x4x1/2) welded to a base plate and then the base plate was anchored to the footing with post installed adhesive anchors. The embedment depth of the adhesive anchor rods were 9 inches with a nominal outside diameter of ¾ inches. A total of three adhesive anchors were used for each of the hold down and a total of four hold downs were used for DCR connection. Similarly, the column side hold down having a 28 inches long angle section (L4x4x1/2) was directly anchored to the column face with post installed anchors and a total of five anchors were used for each of the column side hold downs. The anchors were staggered vertically in order to avoid a single plane that could potentially result in cracking or failure of the anchor rods. Finally, one face of the U-shaped flexural plates was attached to the column side angle section and the other face was attached to the footing side angle section connecting the two parts. Thus, the connection between the footing and column was established through these U-shaped flexural plates where the two faces of the U-shaped plates undergo relative displacement under lateral cyclic loading. It is noted that the U-shaped flexural plates connection to the column side and footing side
angle section were post-tensioned to avoid slippage between the surface. The following section discusses the analytical design steps of the DCR connection.

6.3 Connection Design

The DCR connection system was designed with following objectives –

- Elastic primary structure ensures damage free structural element
- Plastic hinge relocation is avoided
- Rapid implementation of the repair method
- Simple design and construction for easy implementation
- Economically competitive with respect to the traditional repair methods
- Resiliency against future seismic event and aftershocks
- Replaceable external energy dissipating devices that are easily accessible

The proposed design method aims to eliminate yielding in the column when subjected to lateral loading. Cross sectional moment-curvature analysis can be used to determine the column yield moment, $M_y$, and the plastic moment, $M_p$ which can further be used to determine the maximum allowable moment immediately at the top of the CFRP jacket. It is important to ensure that the moment at the top of the CFRP layer is less than the yield moment in order to avoid yielding of the column section and hence the relocation of plastic hinge. The jacket height should consider plastic hinge length of the column, maximum height of inelastic damage in the column resulting from cyclic loading, and confinement required for the column section to develop the full capacity of the DCR connection.
The target design strength for the DCR connection was to match the lateral strength of the as built column-footing specimen. However, seismic performance of the as built specimen shows instability of footing resulting from large flexural cracking. Hence, it was important to account for the force associated with formation of footing flexural cracks. The maximum lateral force for specimen MS#10 was found to be 58.6 kip in the push direction and 45.2 kip in the pull direction. The 45 kips load associated with the pull direction was determined to be the force resulted in cracking and subsequent hinging of the footing. Hence, the target strength for the DCR connection was considered to be 44 kips. Finally, the target strength was used to determine the number and properties of the U-shaped flexural plates (UFP) for the DCR connection. Remaining element of the DCR connection was then detailed to capacity protected. Figure 6.2 shows the target design strength for the DCR connection with different numbers of UFP’s compared to the pull direction strength of the as built specimen MS#10. It was found that 10 UFP’s per hold down would result in a design strength of 44 kips in the pull direction and hence 10 UFP’s per hold down were used for the connection. Hence, a total of 40 UFP’s were used for the four hold downs in the column-footing specimen. Based on the target design strength, the maximum device force is determined where the maximum device force is considered to be 1.5F_P. Where, the overstrength of the device is taken from the UFP plastic force and determined to be 1.5 based on component testing of various UFP geometries at Portland State University.
An iterative moment-rotation analysis procedure was used to determine the load-deformation response of the DCR connection. The procedure was first developed by Pampanin et al. (2001b) and was used to design precast concrete frames with ductile fuses.
The procedure was modified to meet the current repair arrangement and is outlined through step-by-step procedure here.

**Step-1: Load-Displacement Response of As-Built Column**

The design response prediction procedure starts with the analysis of the as-built column to determine the capacity of the column in its as-built condition. A moment-curvature analysis of the column cross section can be used to predict the overall moment-rotation response of the as-built column. The load-displacement behavior can therefore be predicted from the obtained moment-rotation response of the column. However, a more refined non-linear pushover analysis can also be performed to predict the load displacement behavior of the as-built column. The following analysis procedure to predict the deformation response of the repair arrangement is based on the experimentally obtained as-built load-deformation response.

**Step-2: Total Connection Rotation**

The total rotation at the column connection can be approximated from the lateral column top displacement and the height of the column using Equation 6.1.

\[ \Theta_T = \frac{\Delta_T}{H_c} \]  

(6.1)

Where, \( \Delta_T \) = Column top displacement  
\( H_c \) = Height of the column

**Step-3: Flexural Rotation**

The total rotation calculated using Equation 6.1 can be divided into two components i.e., the flexural component resulting from the flexural deformation of the column and the connection rotation at the column-footing interface resulting from gap opening as shown...
in Figure 6.3. The flexural rotation can then be approximated using Equation 6.2 with the moment at the desired location and considering an appropriate rotational stiffness for the flexural component of the column deformation.

\[ \Theta_F = \frac{M_F}{K_C} \]  

Here,

\( M_F \) = Flexural moment resulting from column top displacement = \( H_F \times K_C \)

\( H_F \) = Height of the column where flexural deformation is expected = \( H_C - H_R \)

\( H_R \) = Height of rigid portion of the column jacketed with CFRP wrap

\( K_C \) = Column rotational stiffness = \( 3 \times E_C \times I_C / H_C \) (Considering a pin connection at the base)

**Step-4: Connection Rotation**

The connection rotation at the column-footing interface can then be calculated using Equation 6.3 below.
\[ \Theta_{\text{Conn}} = \Theta_T - \Theta_F \]  

(6.3)

The imposed connection rotation as calculated from Equation 6.3 can then be used to compute the neutral axis depth following an iterative procedure. Connection moment capacity can be finally achieved using subsequent computations following sectional analysis and section equilibrium concept.

**Step-5: Determine Neutral Axis Depth, c (Iterative Procedure)**

The determination of neutral axis depth is an iterative process that starts with the assumption of an initial value of neutral axis depth \(c\). The first approximation can be started as one quarter of the cross-sectional depth of the column as shown in Equation 6.4.

\[ C = \frac{1}{4}h \]  

(6.4)

Here, \(h\) = Column cross sectional depth.

**Step-5a: Dissipaters Displacement's**

The dissipaters displacement resulting from the imposed connection rotation can be calculated as the function of the deformed geometry of the column and the neutral axis depth and using Equation 6.5a through Equation 6.5c. The deformed geometry of the connection under imposed rotation is shown in Figure 6.4 and the nomenclature used in following equations.
Figure 6.4 Deformed geometry of the column and the hold downs

Displacement of the tension dissipaters at the column face under tension,

\[ \Delta_{ufp, sf} = \Theta_{conn}^* (h+s'-c) \]  \hspace{1cm} \text{(6.5a)}

Displacement of the tension dissipaters at the middle of the column section,

\[ \Delta_{ufp, sm} = \Theta_{conn}^* (h/2-c) \]  \hspace{1cm} \text{(6.5b)}

Displacement of the compression dissipaters at the extreme face of the column under compression,

\[ \Delta_{ufp, cf} = \Theta_{conn}^* (s+c) \]  \hspace{1cm} \text{(6.5c)}

**Step-5b: Dissipaters Force’s**

Figure 6.5 presents the section equilibrium and the forces in each component at the connection. Forces in each of the dissipaters can be computed using the force displacement behavior of individual UFP’s. The force-displacement behavior of the UFP’s can be modeled using the Ramberg-Osgood steel model following Baird et. al. (2014) and as outlined in Equation 6.5d.
Here,

\[ \Delta_{\text{ufp}} = \frac{F}{K_o} \left[ 1 + \left( \frac{F}{F_y} \right)^{(r-1)} \right] \]  \hspace{1cm} (6.5d)

\[ \Delta_{\text{ufp}} \]

Here,

\[ F_y = \text{Effective yield displacement of the UFP dissipaters} \]
\[ K_o = \text{Initial stiffness} \]
\[ r = \text{Ramberg-Osgood factor} \]

**Figure 6.5 Section equilibrium at column-footing interface**

**Step-5c: Section Equilibrium**

Once the dissipaters forces are known, the concrete compression resultant \((C_c)\) can be calculated using the strain compatibility at the section and applying the principal of monolithic beam analogy theorem for a rocking element. According to the monolithic beam
analogy the displacement of the rocking element at the column-footing interface can be assumed to be the same as the monolithic beam element. Finally, the concrete compressive force can be computed using Equation 6.5e.

\[ C_c = N + T_{sf} + (2*T_{sm}) - C_s \]  

(6.5e)

Here,

\[ N = \text{Axial load acting on the column} \]

**Step-5d: Determination of Neutral Axis Depth, c**

A new value of neutral axis depth can finally be determined using the Whitney’s stress block assumption for concrete compressive resultant using Equation 6.5f.

\[ c = \frac{C_c}{0.85f_c\beta_1b} \]  

(6.5f)

Here, \( a = \beta_1c \) where \( \beta_1 \) is the ratio of depth of rectangular stress block to the depth of the neutral axis and \( f_c' \) is the concrete compressive strength for the existing bridge column.

If the initially assumed value of \( c \) is equal to the newly obtained neutral axis depth, then the \( c \) value can be used for further calculation otherwise iteration with a new value must be adopted and the steps 5a through 5d should be followed for convergence.

**Step-6: Connection Moment Capacity**

The connection moment capacity can finally be calculated using the neutral axis depth \( c \). Summing moment at \( a/2 \) distance for the section –

\[ M_{Conn} = [C_s*(s+a/2)] + [N*(h-a)/2] + 2*[T_{sm}*(h-a)/2] + [T_{sf}*(h+s'-a/2)] \]  

(6.6)

Finally, the lateral force can be calculated using Equation 6.7.

\[ F = \frac{M_{Conn}}{H_C} \]  

(6.7)
The iterative procedure can be implemented repeatedly to achieve the final load-deformation response of the repaired column.

6.4 Repair Implementation

The repair implementation started with the restoration of the damaged concrete in the column. The damaged and loose concrete was first removed from the column section and the longitudinal rebars were cut near the column-footing interface. The reduced section of the column was then patched with grouted concrete. Prior to the patching of the column few screws and wire mesh was installed to improve the bonding of the concrete between the existing and the new surface. Figure 6.6 shows the restoration of damaged concrete of the column section.

(a) Removal of loose concrete and cutting of longitudinal rebar
The next step in repair implementation was to install the adhesive anchor rods. Firstly, the holes for the anchor rods were prepared with a ¾ inch diameter drill bit and then the holes were cleaned with a wire brush to remove any loose debris inside the hole. This is important for proper bonding of the adhesive with the existing concrete surface. Finally, the holes were further cleaned with pneumatic air gun. Figure 6.7 shows the steps in preparation for the installation of adhesive anchor rods.
Figure 6.7 Preparation for the installation of anchor rods
The next step in the process was to install the FRP wraps for confining the column plastic hinge length. The process started with rounding the corner of the columns to avoid sharp corners of the square column. This is beneficial to avoid stress concentration in the column corner region and hence prevent failure of the CFRP wrap resulting from the concentrated stress in the region. Later the column concrete surface was sand grinded to remove paintings or any other fine particles from the surface of the column. Once the concrete surface was prepared the CFRP sheets were cut according to the height of each segment and were mixed with the epoxy resin. The wet CFRP layers were then placed on the wrapped around the column and any air bubble inside the wrap was removed for proper bonding between the layers of CFRP. Figure 6.8 shows the steps associated with the installation of the CFRP wrap.
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(a) Rounding of the column corner

(b) Surface preparation

(c) Mixing epoxy resin

(d) CFRP wrapping
Once the column was wrapped with CFRP layers, the next step was to install the hold downs. It is important to note that the holes for anchor rods in the column side hold downs were intentionally oversized for construction tolerance issue. However, the sequence of hold down installation was installation of the foundation side angles, loosely tightening the foundation anchor rods, installation of the UFP’s to the foundation side angle, post tensioning of the UFP’s on the foundation side angle, installation of the column side angle sections, post tensioning the UFP’s to the column side angle, tightening the column side anchor rods, and finally tightening the foundation anchor rods. The last step in hold down installation was to install a metal bar connecting the two individual hold downs in each face of the column. This straight metal bar was used to prevent out of plane movement of the UFP connection. Figure 6.9 shows the sequence of hold down installation.
With the completion of the hold down installation the specimen was repaired and ready to proceed with the required instrumentation.

6.5 Test Details and Instrumentation

The repaired specimen was tested under the same setup as the as built specimen where the lateral load was applied with a servo controlled hydraulic actuator and the axial load was applied with another servo controlled hydraulic actuator positioned in vertical direction. The subduction zone lateral reversed cyclic loading protocol was same as the specimen MS#10 and the varying axial loading protocol was also same as MS#10. However, the ductility based lateral loading protocol used for the repaired specimen use the same yield displacement value as calculated for the as built specimen. While the
experimental yield displacement for the repaired system could potentially be different but the loading protocol was kept same with an aim to compare the results with the as built specimen. Details of the test setup is discussed in Section 3.5 and the axial and lateral loading protocol is discussed in Section 3.8. Figure 6.10 below shows the test setup and the instrumentation used for the data collection.
The repaired specimen was instrumented with several LVDT’s, and string potentiometer measuring the global and local displacement quantities. Column lateral displacement was measured with a string potentiometer attached to the top of the column. The footing of the repaired was also instrumented with five LVDT’s placed vertically on
the east face to measure the uplift of the footing. Two other LVDT’s were placed horizontally on the north and south face of the footing to measure any sliding of the footing relative to the laboratory strong floor. Column curvature was measured similar to the as built specimen with a series of vertically placed LVDT’s on curvature rods. Slip between the column side angle and the surface of the column was measured with a LVDT placed vertically on the column surface and measuring the relative displacement on top of the column side angle section. The two angle sections on the north side of the column were instrumented to measure the relative displacement to the column surface. Furthermore, the relative displacement between the column side angle section and the footing side angle section was also measured with a vertically placed LVDT as shown in Figure 6.10(b). Furthermore, the strain gages used to measure the rebar strain for the as built specimen were also used to record the rebar strains withing the column section. However, it is important to note that most of the strain gages within the concrete crushing damaged zones were not available due to the crushing the lead wire connected to the strain gages.

6.6  Test Results

Results obtained from the experimental testing of the repaired specimen is presented in the following sub-sections.

6.6.1  Load-Deformation Response

The lateral load-displacement response for the repaired specimen is shown in Figure 6.11. The load-displacement response showed a stable hysteretic response for both the push and pull direction of loading without any significant strength degradation. It can also be observed that the load-displacement response point to a so-called flag shaped
hysteretic response that is typical of a self-centered system with additional energy dissipating capabilities. It is thus indicative of the effectiveness of the repair system with external energy dissipating devices. Figure 6.11(b) also compares the hysteretic response with the as built specimen. It can be observed that the repaired specimen shows a significant stability for the pull direction of loading as compared to the as built specimen. However, in the push direction, the wider loop for the as built specimen is indicative of higher energy dissipation capability compared to the repaired specimen. It is also prominent that the overall lateral strength of the repaired specimen is significantly less than the as built specimen.

(a) Hysteretic load-displacement response of repaired specimen
The peak lateral load from the as-built column was 58.6 kips in the push direction and 45.2 kips in the pull direction. The target peak lateral load for the repaired specimen was 44 kips. The peak lateral load measured from the repaired test was 41.6 kips in the push direction and 30.6 kips in the pull direction. Hence, the measured peak lateral load was 5.5% less than target strength in the push direction and 30.5% less in the pull direction. The strength degradation for the repaired specimen was significantly less than the strength degradation for the as built specimen. In the push direction, the strength degradation from the peak lateral load during the final loading cycle at displacement ductility, $\mu=8.0$ was 14.8% for the as built specimen and 5.7% for the repaired specimen. In the pull direction, the strength degradation was 4% for the as built specimen and 2.6% for the repaired specimen. However, it is important to note that the strength degradation in both the push and pull direction of the repaired specimen was due to the secondary effect of axial loading.
and not due to notable failure of any component. The ultimate displacement capacity as defined by 20% degradation from peak strength was never reached for the repaired specimen. A summary of the experimental results comparison is presented in Table 6.1 below.

<table>
<thead>
<tr>
<th>Test</th>
<th>Loading Cycle</th>
<th>Effective Stiffness, k (kips/inch)</th>
<th>Peak Lateral Load (kips)</th>
<th>Displacement at Peak Lateral Load (in.)</th>
<th>Degradation from peak at μ=8 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>As Built</td>
<td>Push</td>
<td>147</td>
<td>58.6</td>
<td>2.99</td>
<td>14.8</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td></td>
<td>45.2</td>
<td>0.92</td>
<td>4.0</td>
</tr>
<tr>
<td>Repaired</td>
<td>Push</td>
<td>120</td>
<td>41.6</td>
<td>3.32</td>
<td>5.7</td>
</tr>
<tr>
<td></td>
<td>Pull</td>
<td></td>
<td>30.6</td>
<td>4.31</td>
<td>2.6</td>
</tr>
</tbody>
</table>

**6.6.2 Envelope Comparison**

The backbone curve or the envelope responses for the as built and the repaired specimen are presented in Figure 6.12 below. The key performance parameter like effective yield displacement, peak lateral load and displacement during the final loading cycle is also shown for the as built specimen. Similarly, the peak lateral load for the repaired specimen is also presented in Figure 6.12. It can be observed that the initial stiffness in the push direction of loading was comparable between the as built and the repaired specimen whereas in the pull direction the initial stiffness of the repaired specimen was lower than the as built. The peak lateral load in the push direction was 23% lower than the peak load for the as built specimen whereas the peak lateral load was 31% lower compared to the peak load of the as built specimen. While the design target strength of the repaired specimen was set to be at 44 kips for the pull direction, it has certainly achieved lower
strength than expected. This could be attributed the oversized holes for the column side anchor rods that allowed slip between the hold downs and the column surface. In addition, gap opening below the base plate in the footing side angle section was also observed that played into potentially achieving less strength for the repaired specimen. These issues are further discussed in the following sections.

Figure 6.12 Comparison of envelope curve for as built and repaired specimens

6.6.3 Strength and Stiffness Degradation

The comparison of strength and stiffness degradation for the as built and the repaired specimen is presented in Figure 6.13. The push and pull cycles of loading are presented separately where the push cycles are presented with a solid and the pull cycles with a dashed line. The ultimate degradation from the peak strength was found to be below
the 20% threshold for both the as built and repaired specimens. In the push cycle of loading, the as built specimen achieved peak strength at an earlier displacement ductility level and also showed higher strength degradation compared to the repaired specimen. The final loading cycle was associated with a 14.8% strength degradation for as built specimen and whereas the repaired specimen was associated with only 5.7% strength degradation. In the pull direction, however, the as built specimen and repaired specimen showed comparable strength degradation during the final loading cycle. The as built specimen showed a maximum strength degradation of 4.8% at displacement ductility $\mu=1.3$. Following the initial strength degradation, a straight line representing no significant strength degradation was observed for the as built specimen. This was due to the elastic perfectly plastic load displacement response observed in the pull direction of the as built specimen where the strength was dictated by the flexural capacity of the footing element. The lack of strength degradation at higher displacement ductility level in the pull direction of the as built specimen could be indicative of brittle flexural failure resulting from the rupture of bottom layer of the footing reinforcements. However, the final strength degradation for the repaired specimen was only 2.6%.
The stiffness degradation for the as built and the repaired specimen is presented in Figure 6.13(b). The stiffness degradation in the push direction of loading was comparable for the as built and the repaired specimen whereas in the pull direction the repaired specimen showed lower stiffness degradation throughout the loading cycles. It was also found that the analytical equation representing stiffness degradation as the inverse of
displacement ductility was inadequate for the repaired specimen. However, a nonlinear relationship between the displacement ductility and the stiffness degradation can be established where the axial load level dictates the order of degradation.

### 6.6.4 Gap Opening – Hold Down Contribution

The contribution of the hold down rotation compared to the total base rotation and the column rotation is presented in Figure 6.14 below. The column rotation was calculated from the experimentally measured lateral displacement at the top of the column divided by the height of the column. It is noted that the footing flexibility was not excluded from the column top displacement. However, the column top displacement resulting from the footing rotation was found to be insignificant. The DCR connection rotation at the base of the column was calculated from the experimentally measured LVDT data recorded at 9 inches from the column base. Two LVDT’s were placed on the north and south side of the column directly measuring the gap opening at the base. These values were used to calculate the total connection rotation at the base. Similarly, a LVDT was placed vertically to measure the relative displacement between the two angle sections in each of the hold downs. Both the north and south side hold downs were again instrumented to measure the hold down displacement. These data were then used to calculate the hold down rotation for the specimen. It can be observed from Figure 6.14, that for a specific moment at the connection, the total connection rotation at the base was comparable to the hold down rotation whereas the column rotation was higher. This is indicative of the fact that the total base rotation was mostly translated into the hold down but other sources of deformation like flexural contribution of the column, shear deformation etc. led to higher column
rotation. A closer look into the base rotation and the hold down rotation shows that in an average the hold down rotation accounts for 85% of the base rotation in the system. The remaining rotation can be associated with the slip in the hold down connection.

![Comparison of total connection rotation to hold down rotation](image.png)

Figure 6.14 Comparison of total connection rotation to hold down rotation

### 6.6.5 **Column Curvature – Flexural Contribution**

Experimental column curvature profile is presented in Figure 6.15 below. The curvature values were calculated from the data recorded with vertically placed LVDT’s on two opposite side of the column. These LVDT’s were measuring the vertical deformation of the column section for a specific gage length. The curvature profile shows in Figure 6.15 also presents the analytical yield curvature for the column cross section with red dashed lines. It can be observed that the measured curvature up to a displacement ductility, \( \mu = 8.0 \) was significantly lower than the yield curvature indicating that the column remained elastic for the entirety of the test. One of the important objectives of the repair system was to ensure that the column section remained elastic without any major damage in the column.
to ensure future resilience. Hence, the experimental data presented here validates the design objective for the repair system where an elastic column was ensured with supplemental external energy dissipating system.

Figure 6.15 Column curvature profile
Chapter 7  Analytical Response Prediction

7.1  Introduction

This chapter outlines an analytical response prediction method for the current repair method and investigates individual parameters of the analytical equations based on measured data from the experimental testing of rapid repair measures utilizing steel collar and external U-shaped flexural plate dissipaters. Test setup and the repair methodology is shown in Figure 7.1 and the test matrix is presented in Table 7.1. Further details and experimental results can be found in (Murtuz et al. 2021).

Figure 7.1 Experimental test setup layout
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Section 6.3 of the earlier chapter discusses the proposed analytical response prediction method. The iterative section analysis procedure was used to develop the analytical response prediction for the repaired columns. The total column lateral displacement was divided into two main components i.e., column top displacement resulting from hold-down rotation and the contribution of flexural deformation along the length of the repaired column. Each of these two components were measured with displacement transducers during the experimental testing and were evaluated for the justification of the initially developed analytical equations. Section 7.2 of this chapter presents the findings of experimentally evaluated data and compares the result with proposed analytical method. Finally, the chapter concludes with proposed modification to the analytical equations, or co-efficient of equations based on the experimental data evaluation and are presented in subsequent sections.

<table>
<thead>
<tr>
<th>Test</th>
<th>As Built Column Designation</th>
<th>Hold-down Orientation</th>
<th>Lateral Loading Protocol</th>
<th>UFP $F_p$ (kips)</th>
<th>Expected Hold-down Force, $F_{max}$ (kips)*</th>
<th>Expected Lateral Load $V_{max}$ (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SV Corner</td>
<td>Subduction</td>
<td></td>
<td>5.45</td>
<td>32.73</td>
<td>46.4</td>
</tr>
<tr>
<td>2</td>
<td>LV Face</td>
<td>Conventional</td>
<td></td>
<td>5.43</td>
<td>32.60</td>
<td>48.9</td>
</tr>
<tr>
<td>3</td>
<td>LV Face</td>
<td>Subduction</td>
<td></td>
<td>5.43</td>
<td>32.60</td>
<td>48.9</td>
</tr>
<tr>
<td>4</td>
<td>LV Face</td>
<td>Subduction</td>
<td></td>
<td>8.83</td>
<td>52.97</td>
<td>61.9</td>
</tr>
</tbody>
</table>

*The maximum expected hold-down force was calculated as $F_{max} = (1.5F_p) * (4$ UFP’s per hold-down)*

One of the main objectives of the research undertaking was to validate the design of the rapid repair methodology through experimental testing. The design principal adopted for the repair method was aimed at to minimize the damage in the substructure while
restoring the strength and stiffness of the as-built column after an earthquake event. A simplified static analysis procedure can be used to determine the initial design parameters e.g., collar details, hold-down arrangements, UFP properties etc. However, a more refined method is necessary to accurately predict the load-deformation response of the repaired column and hence to ensure similar strength and stiffness parameters as compared to the as-built column. An iterative moment-rotation analysis procedure was used to predict the load-deformation response of the repaired column. The procedure was first developed by Pampanin et al. (2001b) and was used to design precast concrete frames with ductile fuses. The procedure was modified to meet the current repair arrangement and is outlined through step-by-step procedure in the following section.

7.2  Measured Displacement Components

Displacement data measured with the LVDT’s were used to investigate the contribution of column flexural deformation and the hold down rotation into the total column top displacement. Flexural deformation of the column above the shell was determined from average column rotation measured with a series of LVDT’s mounted on curvature rods for different segments along the height of the column. The measured data was compared with the analytically predicted flexural moment-rotation hysteresis for the hold-down arrangement. The hold-down displacement was also measured with displacement transducers. The data measured was then used to determine the column rotation resulting from the hold-down displacement.
7.2.1 Load-Displacement Response

The load-deformation response of the repaired column for the push and pull cycles of loading is plotted in Figure 7.2 and Figure 7.3, respectively. The column top displacement measured with an independent string potentiometer is plotted against the lateral load measured with the built-in load cell of the lateral actuator. The column displacement presented here was corrected for any contribution of foundation uplift and the lateral load was corrected for the component of the axial load resulting from secondary P-Δ effect.

A spring analogy, as stated earlier in Section 6.3, was used to determine the component of column top displacement resulting from hold-down rotation and the flexural deformation of the column section beyond the elastic shell height. Hold-down rotation was calculated based on the measured data of the north and south hold-down displacement and using the stated Equation 7.1. Flexural deformation of the column above the shell was determined from average column rotation measured with the series of LVDT’s mounted on curvature rods for different segments along the height of the column. Finally, the cumulative displacement resulting from column flexural deformation and the hold-down rotation were calculated through algebraic summation of these two components.

\[ \text{HD}_{Rotation} = \frac{S_{HD} - N_{HD}}{D_{HD}} \]  \hspace{1cm} (7.1)

Figure 7.2 presents the backbone envelope displacement for all the four tested specimens in the push cycle of loading along with the measured displacement components for the tests. Results showed a good agreement between the column top displacement measured with an independent string potentiometer and the cumulative displacement.
calculated from the individual displacement components calculated from measured LVDT readings. The flexural deformation component for the column showed a linear increase in displacement with the increase in lateral load. A constant slope representing the rotational stiffness of the flexural spring can be used to predict the flexural deformation response of the column. Further investigation into the flexural deformation response of the column is discussed in later section. It can also be observed that the column top displacement resulting from the rotation of the hold-down is significantly higher than the flexural deformation. This can be attributed to the accumulation of plastic deformation in the UFP’s and hence accounting for majority of the column top displacement resulting from hold-down rotation. Also, the linear flexural deformation component showed that the remainder of the column section behaves essentially elastically.
Figure 7.2 Contribution of flexural deformation and hold-down rotation into column top displacement for push cycle

Figure 7.3 presents the backbone envelope displacement for tested specimens in the pull cycle of loading along with the measured displacement components. Similar trend in pull cycles of loading were also observed where the cumulative displacement calculated from the independent displacement component reasonably capture the total column top displacement measured independently. Also, most of the displacement for the pull cycle of loading is resulting from the hold-down rotation. The column top displacement resulting from flexural deformation of the column was again found to be accounting for only a minor portion of the total column top displacement and increases linearly with the increase of lateral load. Hence, the efficacy of the repair method to limit damage in the column region can be experimentally verified for both the push and pull cycles of loading.
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Test 1

Test 2
Further investigation into the push and pull cycles of loading for test 4 was conducted to compare the effect of axial load variation on the displacement components. It was found that the column top displacement resulting from hold-down rotation for both
the push and pull cycles of loading accounts for 88% of the total displacement at the maximum ductility cycle. Whereas the column top displacement resulting from the flexural deformation accounts for 3% in the push cycle and 4.5% in the pull cycle at the maximum ductility loading cycle. While the maximum ductility loading cycle accounts for the maximum variation in axial loading for push and pull cycles, the effect on displacement components is negligible. Similar results were obtained for the rest of the tested specimens where the effect of axial load variation has no significant impact on the efficacy of the repair method to limit damage in the column. Figure 7.4 and Figure 7.5 show the distribution of column top displacement resulting from different displacement components i.e., flexure and hold-down rotation as a percentage of the total column top displacement for individual loading cycles in the push and pull direction, respectively. Also, the difference between the column top displacement measured independently and the calculated column top displacement resulting from individually measured components are shown in red color. Ideally, the difference should be zero if the column top displacement is resulting only from the contribution of the hold-down displacement and flexural deformation. But shear deformation and any slip due to the lateral loading was not measured and was not accounted in the calculated cumulative top displacement. Hence, the difference in each loading cycle accounts for other sources of column displacement that was not directly measured during the experimentation. However, the difference was significantly lower for all the test specimens except test 3.
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**Test 1**

- Column Displacement_HD
- Column Displacement_Flexure
- Difference from Measured Displacement

**Test 2**

- Column Displacement_HD
- Column Displacement_Flexure
- Difference from Measured Displacement
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The majority of the column displacement resulted from the hold-down rotation. However, the impact of hold-down rotation was more pronounced in the higher ductility cycles where more than 85% of the total column top displacement was due to the hold-down rotation. This can be attributed to the yielding of UFP’s in the hold-down and the
accumulation of plastic deformation within the fuse elements. The column top
displacement resulting from the flexural deformation accounts for almost 1/3 of the total
displacement for the initial loading cycles and gradually decreases during the final loading
cycles at higher displacement ductility. It can also be observed that the pull cycles of
loading experienced less flexural deformation than the push cycles of loading. It is noted
that the pull cycles of loading were associated with lower axial load compared to the push
cycles of loading.
Part-II: Rapid Repair Measures
Chapter 7- Analytical Response Prediction

Test 2

Test 3
7.2.2 **Hold-Down Displacement**

Experimentally measured and the analytically predicted hold-down displacement is plotted against the lateral load and is presented in Figure 7.7 for the push cycles of loading and Figure 7.8 for the pull cycles of loading. An analogous spring model was used to predict the hold-down displacement response for the tested specimen.
Figure 7.6 UFP hysteresis response adopted from Baird et. al. (2014)

Measured hold-down response agrees well with the analytical prediction and shows that the analogous spring model can be efficiently used to predict the overall response of the hold-down displacement. However, it should be noted that the analytically predicted response is dictated by the adopted UFP hysteretic response and hence an accurate model of the UFP response would yield a closer prediction of the overall response. It can also be noted that the analytical prediction does not account for degradation in the higher ductility cycles due to the adopted hysteretic response of the UFP’s as shown in Figure 7.6. The UFP hysteretic response was adopted after the recommendation of Baird et. al. (2014). It is important to note that the UFPs used for the current experimental program is different in terms of how they were bend to U-shapes. The process of bending the steel plates can cause a difference in the response of the UFPs. Hence, the degradation could be a result of different load-deformation behavior of the UFPs. Furthermore, few other potential sources of degradation such as slip in the system at larger displacement cycles, strain penetration...
in the anchor rods, softening of steel at the reduced section of the hold-down leg (angle sections), deterioration of the concrete due to toe crushing, etc. can contribute to the degradation. But it should be stressed that the result shows a significant reduction in strength degradation for the repaired system as compared to the as-built column. The system reached significant ductility level without significant loss of strength and hence the small degradation in the overall system behavior can be disregarded.
The initial stiffness for the pull cycles of loading were well aligned with the analytically predicted initial stiffness. The analytical prediction closely captures the lateral load versus the column top displacement resulting from hold-down rotation for test 1 and test 2. For, test 3 the analytical equation overestimates the lateral versus hold-down component of column displacement whereas it underestimates the behavior for test 4. It is noted that both the specimens were tested under same loading protocol but with different UFP geometries. However, it should also be noted that the behavior presented here is an effective means of looking at the efficacy of the analytical response prediction, but the load presented in the vertical axes does not represent the hold-down forces. Rather, it represents the lateral load resisted by the column. Hence, a more refined analysis into the hysteretic response of the hold-down forces versus hold-down displacements would help investigate the efficacy of the adopted UFP model and any modification to the individual model.
parameter. For this report, the task was considered out of the scope and the adopted UFP model proposed by Baird et. al. (2014) was considered to be accurate enough for the overall analytical response prediction.
7.2.3 *Column Flexural Displacement*

The flexural deformation component of the total column top displacement was further investigated to determine the rotational spring constant of the assumed spring model. Figure 7.9 and Figure 7.10 shows the measured flexural displacement response
plotted against the lateral load for push and pull loading cycles, respectively. Cluster of data in the higher displacement cycles indicate the decrease in flexural deformation of column and indicate the accumulation of plastic displacement in the replaceable U-shaped flexural plates (UFPs).
Figure 7.9 Envelope of hysteretic response of flexural displacement for push cycles of loading

The lateral load versus column top displacement resulting from flexural deformation for the pull cycles of loading shows similar trend and can be approximated with a linear equation. However, the slope of the linear regression line for the pull cycles
of loading was lower than the push cycles of loading and can be attributed to the lower axial load associated with the pull cycles of loading.

\[ y = 220.18x \]
\[ R^2 = 0.9689 \]

\[ y = 205.15x \]
\[ R^2 = 0.9696 \]
A linear regression analysis shows that the flexural displacement of the column can be closely predicted with the linear line having a constant stiffness coefficient. The analytical equation to predict the column top displacement resulting from the flexural
deformation of the column can be approximated using the linear equation presented with Equation 7.2 below.

\[ F = K_r \Delta \]  
\hspace{1cm} (7.2)

Where, \( K_r = \text{Stiffness for flexural displacement component (slope of the linear line in Figure 7.9 and Figure 7.10)} \)

The stiffness coefficient, \( K_r \) for flexural displacement prediction for both the push and pull cycle of loading can be taken as the slope of the linear regression line and are presented in the Table 7.2 Flexural stiffness coefficient.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Flexural Stiffness, ( K_r ) (kip/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Push Cycle</td>
</tr>
<tr>
<td>Test 1</td>
<td>266</td>
</tr>
<tr>
<td>Test 2</td>
<td>253</td>
</tr>
<tr>
<td>Test 3</td>
<td>237</td>
</tr>
<tr>
<td>Test 4</td>
<td>261</td>
</tr>
<tr>
<td>Mean</td>
<td>254</td>
</tr>
<tr>
<td>Average for Push and Pull</td>
<td>247</td>
</tr>
</tbody>
</table>

The average stiffness coefficient approximated from the experimental data for the push cycle is 254 kip/in while the average coefficient for pull cycle is 240 kip/in. However, the average stiffness coefficient for both the push and pull cycles was found to be 247 kip/in.

7.2.4 **Column Moment-Rotation**

The moment-rotation response for the 37" to 40" segment (immediately on top of the shell height) is presented in Figure 7.11 for the push cycles of loading. Experimentally measured response was compared with the analytically predicted behavior. The linear
regression analysis used in the previous section was adequate to capture the overall flexural load-deformation response of the column. However, the computation of individual points in the moment-rotation plot using a single rotational stiffness coefficient leads to overestimation of the bending moment at higher displacement ductility cycles. By definition, the slope of the analytically predicted response is the rotational stiffness of the adopted analogous spring model (Section 6.3). Figure 7.11 shows that a linear equation with a single stiffness coefficient e.g. \(3EI/H\) is somewhat insufficient to predict the accurate moment-rotation response of the column section. Hence, a second order polynomial equation was more adequate to capture the moment-rotation response. Figure 7.11 shows both the linear regression line and the polynomial regression line for all the tests specimen in the push cycle.
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Test 2

\[ y = -3 \times 10^9 x^2 + 5 \times 10^6 x \]
\[ R^2 = 0.9249 \]

Test 3

\[ y = -1 \times 10^9 x^2 + 3 \times 10^6 x \]
\[ R^2 = 0.9962 \]
Figure 7.11 Flexural moment-rotation response for 37” to 40” segment for push cycles

Figure 7.12 presents the moment-rotation response for the 37” to 40” segment in the pull cycles of loading. A similar trend for the pull cycle of loading was also found where the analytical equation representing a single rotational stiffness coefficient is somewhat inadequate to accurately capture the moment-rotation response. However, it should be noted that the accuracy of the linear regression line in predicting the moment-rotation response during the initial cycles is more accurate for pull cycles of loading than the push cycles. The push cycles of loading completely overestimate the initial loading cycles.
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Test 1

\[ y = 9 \times 10^8 x^2 + 3 \times 10^6 x \]

\[ R^2 = 0.9574 \]

Test 2

\[ y = 3 \times 10^9 x^2 + 5 \times 10^6 x \]

\[ R^2 = 0.9716 \]
The experimental spring constant was derived from the measured response and is used to validate the initially assumed spring constant for the analogous spring model in the analytical response prediction. Although it was found that a second order polynomial equation can accurately predict the moment-rotation response of the column section, using
such equation would compromise the objective of developing a simple set of equations for response prediction. Hence, an alternative method of predicting the moment-rotation response with a linear equation having a single initial rotational stiffness coefficient was considered along with a stiffness degradation model. The experimentally obtained data were used to plot the degradation of the secant stiffness and plotted for the push and pull cycles of loading in Figure 7.13 and Figure 7.14, respectively.

\[ y = -3.7688x + 1.1627 \]

\[ R^2 = 0.9824 \]
Chapter 7 - Analytical Response Prediction

Test 2

$y = -6.6205x + 1.3672$

$R^2 = 0.9701$

Test 3

$y = -3.9264x + 0.8934$

$R^2 = 0.6961$
Figure 7.13 Degradation of rotational flexural stiffness for 37" to 40" segment for push cycles

The experimentally obtained rotational flexural stiffness degradation for the pull cycles of loading are shown in the following figure.
Part-II: Rapid Repair Measures
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Test 2

\[ y = 4.2137x + 1.1579 \]
\[ R^2 = 0.966 \]

Test 3

\[ y = 4.569x + 1.0751 \]
\[ R^2 = 0.9493 \]
A linear regression line can be used to accurately predict the rotational stiffness degradation of the column. A linear equation presented here as Equation 7.3 can be used along with the initial rotational stiffness value.

\[
\frac{K_{sec}}{K_y} = m\Theta + c \tag{7.3}
\]

Where, \(m\) is the slope of the regression line and \(c\) is the constant. The values of the slope and the constant for push and pull cycles for all the four specimens are listed in Table 7.3 Coefficient of the linear regression line for predicting stiffness degradation along with the average values.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Slope, (m)</th>
<th>Constant, (c)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Push Cycle</td>
<td>Pull Cycle</td>
</tr>
<tr>
<td>Test 1</td>
<td>-3.77</td>
<td>-3.55</td>
</tr>
<tr>
<td>Test 2</td>
<td>-6.62</td>
<td>-4.21</td>
</tr>
<tr>
<td>Test 3</td>
<td>-3.92</td>
<td>-4.57</td>
</tr>
<tr>
<td>Test 4</td>
<td>-2.62</td>
<td>-2.41</td>
</tr>
<tr>
<td>Mean</td>
<td>-4.23</td>
<td>-3.69</td>
</tr>
<tr>
<td>--------------</td>
<td>-------</td>
<td>-------</td>
</tr>
<tr>
<td>Average for Push and Pull</td>
<td>-3.96</td>
<td></td>
</tr>
</tbody>
</table>
Chapter 8  Conclusions and Recommendations for Future Research

8.1  Summary and Conclusions

A total of six full-scale column-footing subassemblies representing critical part of multi-column bridge bent in Oregon constructed prior to 1990 was experimentally evaluated under reversed cyclic lateral loading protocol. In the first phase of experimental program, three nominally identical subassemblies were tested under two different lateral loading protocols i.e., conventional three cycle symmetric lateral loading protocol and a cyclic loading protocol developed to implement cyclic demand resulting from a long duration Cascadia Subduction zone earthquake. Two different types of axial loading were also implemented to represent a constant axial load level and a varying axial loading protocol typical of multi-column bent under lateral loading. The second phase of study consists of another three column-footing subassemblies having no splice rebar in the column plastic hinge zone, very short lap splice (25d_b) and different steel ratio for the column. Cascadia subduction zone lateral loading protocol and varying axial loading protocol were used for all of these three specimens. The summary of the test results are as follows –

- All the three subassemblies (SC, SV, and LV) from first phase of experimental studies showed ductile response with a minimum displacement ductility of \( \mu = 6.7 \) (4.6% drift) and a maximum displacement ductility of \( \mu = 7.9 \) (5.2% drift) despite having substandard seismic detailing. The surprisingly ductile behavior was attributed to the moderately long lap splice length (47d_b) and low reinforcement ratio (\( \rho_s = 0.55\% \)) that led to significant opening of the cold joint at the column-
footing interface resulting in rocking that delayed damage to the column. Average peak lateral load was 44.5 kips in the push direction and 40.9 kips in the pull direction.

- The failure mode for all the three subassemblies (SC, SV, and LV) were flexural tension failure with crushing of cover concrete and formation of plastic hinges at the base of the column. All the damage was concentrated in the column region whereas the spread footing accompanying the square column was undamaged for all the three subassemblies without any cracking or joint shear failure despite having only a single layer of longitudinal reinforcement at the bottom. The final damage state for all the three subassemblies were associated with significant lateral strength degradation but without any apparent loss of axial load carrying capacity.

- The specimen with continuous longitudinal rebar in the column (CR#8) showed similar response where flexural plastic hinging of the column was observed and the primary failure mode was flexural tension failure. A minimum displacement ductility of $\mu=5.4$ (4.2% drift) and a maximum of $\mu=6.9$ (5.3% drift) was determined. The peak lateral load in the push and pull direction was 44.6 kips and 37.9 kips, respectively. Yielding of transverse reinforcing bar at 6 inches from the column base was also observed only for this specimen indicating better confinement due to the rebar continuity.

- The specimen (MS#10) having a column longitudinal reinforcement ratio $\rho_s=0.88\%$ (4-#10 rebar) resulted in undesirable flexural cracking of the accompanying spread footing. The flexural brittle failure of the footing was anticipated from the load-
displacement response of the subassembly. The increase in reinforcing ratio resulted in increased plastic hinge capacity of the substandard column that led to increased demand in the spread footing. This along with lower axial load level was associated with the cracking of the footing element. The minimum and maximum displacement ductility achieved at the end of the test was $\mu = 6.9$ (5.1% drift) and $\mu = 8.0$ (5.9% drift). The peak lateral load in the push and pull direction was 58.6 kips and 45.2 kips, respectively.

- The flexural cracking moment for the footing section with a single layer of reinforcement at the bottom was calculated to be 251 kips-ft and 223 kips-ft with an associated axial load of 240 kips and 160 kips, respectively. The column lateral load corresponding to the footing cracking moment was calculated as 48 kips in the push direction and 45 kips in the pull direction. While specimen MS#10 was found to have achieved the column lateral load capacity for both the push (58.6 kips) and pull (45.2 kips) direction but none of the other specimens reached the lateral load capacity required to produce the footing cracking moment. As a result, footing flexural cracking was only observed for specimen MS#10.

- The column-footing subassembly specimen (SS#8) having very short lap splice length ($25d_b$) achieved a maximum displacement ductility of $\mu = 2.9$ (2.0% drift). The final damage state for the specimen was splice failure resulting in early strength degradation. The peak lateral load in the push and pull direction was 41.8 kips and 36.9 kips, respectively.
Following conclusions were drawn in the context of the experimental results obtained from the test program –

- Despite having substandard detail, columns having moderate lap splice length or continuous rebar and light reinforcement will result in flexural plastic hinging at the base of the column. However, flexural cracking of gravity designed spread footing should also be investigated as a potential failure mode.

- Columns having very short lap splice in the plastic hinge zone was inadequate to develop flexural plastic hinging at the column base and failed due to early strength degradation resulting from splice failure.

- The effect of varying axial loading protocol was more pronounced for subassembly tested under long duration Cascadia Subduction lateral loading protocol. Varying axial loading protocol resulted in an asymmetric hysteretic response where higher peak lateral load was associated with higher axial load level. Furthermore, the axial load path associated with varying axial loading protocol resulted in achieving slightly higher displacement ductility compared to the subassembly tested under constant axial load level but having same amplitude as the maximum of the varying axial loading protocol.

- Post peak strength degradation was influenced by the axial loading protocol where a symmetric strength degradation was observed in the push and pull direction for subassembly tested under constant axial load level. Conversely, subassemblies tested under varying axial loading protocol shows asymmetric strength degradation
with higher degradation corresponding to higher axial load level. It was also found that higher axial load increases the rate of strength degradation.

- In comparison with the behavior of previously tested columns at Portland State University, it can be concluded that the moderate splice length along with foundation rocking resulted in achieving higher displacement ductility for the column-footing subassembly than the previously tested columns with lower splice length. It can also be concluded that specific damage states (i.e., concrete spalling, rebar buckling etc.) were delayed due to the rocking of the foundation compared to the past tests that focused only on the column component behavior (i.e., footings were anchored to the floor).

- The strain-limit values for the operational performance level were found to be governed by the specimens tested under the subduction zone lateral loading protocol.

- The operational performance limit state was defined with respect to the concrete strain at the initiation of cover concrete spalling and steel strain corresponding to the exceedance of a residual crack width exceeding 1.0 mm. The average concrete and steel strains corresponding to the operational performance limit state was \( \varepsilon_c = 0.0094 \) (standard deviation = 0.005) and \( \varepsilon_s = 0.024 \) (standard deviation = 0.01), respectively.

- The strain-limit state for the operational performance criteria, as obtained from the experimental results, were compared with the existing strain limit states specified by Oregon Department of Transportation (ODOT) for the evaluation of existing
bridges. The ODOT recommended concrete strain value of 0.002 for columns having inadequate hoops and hoop spacing but adequate splice length was found to be conservative. Rather, the widely used operational concrete strain value of 0.004 was found to be more suitable for such columns.

- Existing strain penetration length proposed by Priestly et al. (2007) was found adequate for the representative column-footing subassemblies and can be used to adequately compute the bond-slip rotation for subassembly. Also, the triangular curvature distribution based plastic hinge model can be useful for performance based seismic evaluation purposes and the modified compressive plastic hinge length model proposed by Goodnight et al. (2016b) was found more accurate to predict the length of experimental spread of plasticity.
The experimental program on rapid repair measures validated the design goal of achieving restored or controlled strength, while isolating damage to replaceable ductile fuses and, in turn, enhancing the resilience to aftershocks or future seismic events. The experiments have shown the potential of this methodology to rapidly repair earthquake damaged columns with a relatively generic approach. The key takeaways from the experiments are as follows:

- This study provided strong evidence to validate the design goal of achieving restored or controlled strength. While the peak lateral load capacity achieved with the repaired specimen was lower than the as built specimen, but comparable strength can be achieved with better construction controls.
- The repaired specimen achieved a displacement ductility of $\mu = 8.0$ without any significant damage to the column or the associated spread footing. Flexural cracking of the spread footing observed in the as built specimen was successfully mitigated with the repaired method and dissipative rocking at the column-footing interface was successfully established.
- An additional benefit to the proposed repair methodology is reduced strength degradation at high drifts and during long duration cyclic loading. The experimental results from the repair test showed a significant reduction in strength degradation from the as-built state.
- The experimental program demonstrated the feasibility of the proposed repair methodology to be rapidly implemented. The components should be fabricated with adequate tolerances for constructability. Oversized hole and plate washers on the
exterior hold-down leg greatly reduced the installation time. Prior to the CSZ earthquake, the proposed repair methodology should be pre-manufactured and inventoried for rapid access in the aftermath of an earthquake. Earthquake preparation should include training and practice for workers to implement the repair methodology.

In conclusion, this study successfully demonstrated the development of a resilient repair methodology for earthquake damaged bridge columns that can be rapidly implemented following a damaging earthquake.

8.2 Recommendations for Future Research

Following recommendations are proposed for further studies –

- Effect of bidirectional earthquake loading, and different lateral loading protocol should be considered to investigate the seismic performance and limiting strain values for the representative bridge bents.
- The effect of soil-structure interaction on the different damage states should be investigated and the performance of pile foundations can be investigated.
- Conventional instrumentation limits the evaluation of strain values for damage states at higher displacement ductility level. Advanced measurement approaches (i.e., three-dimensional position sensors) could be used to capture the strain values more accurately, especially for life-safety damage levels.
- Limited confidence exists in the strain values for the life-safety performance criteria due to the difficulties in capturing strain data at higher displacement levels. Further
studies could be used to investigate the material strain values at life-safety performance criteria for existing bridge types in the state of Oregon.

The practicality and effectiveness of the rapid repair method needs to be further investigated for a column buried under deep fill or submerged in water. Following are some of the recommendations for future studies –

- The proposed repair method uses replaceable hold-downs made of regular steel plates and angle sections. Such a setup is susceptible to corrosion when exposed to atmospheric condition and the corrosion can be greatly enhanced under direct exposure to moisture content and salt water. Hence, a preventive measure against corrosion should be considered while designing the components of the repair method. Further research can be conducted to find a suitable solution for corrosion prevention of the hold-down components to ensure long-term functionality of the repair method. The Steel Bridge Design Handbook (Kogler 2015) provides a list of alternative methods that can be considered as a preventive measure against corrosion. Furthermore, suitability of smart materials that are resilient against corrosion such as stainless steel, shape memory alloy (SMA) etc. can be considered as an alternative to regular steel. However, economic viability of such an alternative should be critically investigated.

- Toe concrete crushing of the column within the gap between the CFRP layer and the footing was observed during the experimental test of the repaired column. The crushing was not significant and does not have a significant impact on the response of the repaired system. However, the long-term performance of the repaired system
must be ensured for resiliency and hence the integrity of the column concrete in the gap region must be ensured. Further research investigating different alternatives to prevent the toe crushing of the column should be considered.

- Self-centering behavior of the proposed repair method was dependent solely on the gravity load of the bridges. The current experimental program used an axial load ratio of 10\% and 7\% for push and pull cycles, respectively. Self-centering behavior was observed under these axial load condition but the behavior of a bridge substructure with significantly lower axial load level should be investigated. Especially, the ability of the repair method to provide self-centering behavior with lower axial load level should be scrutinized. Use of unbonded post-tensioned system along with the axial load can also be investigated to establish a pronounced self-centering behavior of the repair method.

- Development of a finite element model to predict the seismic response of the repaired system can be undertaken as future research endeavor. Furthermore, the global behavior of concrete bridges with replaceable hold-downs needs to be scrutinized. Transfer of load between the superstructure and the substructure should be further investigated and the component response should be examined.
References


References


## Appendix A

### Summary of Past Research on Low-Damage Structural System

<table>
<thead>
<tr>
<th>Author (Year)</th>
<th>Research Objective</th>
<th>Structural System</th>
<th>Application</th>
<th>DCR (Y/N)</th>
<th>ED Loc.</th>
<th>ED Type</th>
<th>PT (Y/N)</th>
<th>Relevant Findings</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Kelly et al. 1972)</td>
<td>Investigating different ED's</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>UFP, mild steel beams</td>
<td>N/A</td>
<td>N/A</td>
<td>Potentials of UFP for seismic application</td>
</tr>
<tr>
<td>(Mander and Cheng 1997)</td>
<td>Development of damage avoidance design</td>
<td>Precast bridge column</td>
<td>New</td>
<td>Y</td>
<td>N/A</td>
<td>No ED</td>
<td>Y</td>
<td>*Unbonded PT improves lateral strength&lt;br&gt;*Rocking alone is insufficient to dissipate energy</td>
</tr>
<tr>
<td>(Palermo et al. 2004)</td>
<td>Feasibility of DCR for bridge piers</td>
<td>Bridge column</td>
<td>New</td>
<td>Y</td>
<td>Internal</td>
<td>Mild steel</td>
<td>Y</td>
<td>*DCR is efficient and promising alternative to monolithic system&lt;br&gt;*DCR results in negligible residual displacements and damage</td>
</tr>
<tr>
<td>(Palermo et al. 2007)</td>
<td>Design, modeling, and experimental validation of DCR system</td>
<td>Bridge column</td>
<td>New</td>
<td>Y</td>
<td>Internal</td>
<td>Mild steel</td>
<td>Y</td>
<td>*Stable hysteretic response with minor flexural cracking and self-centering capability of jointed ductile connection</td>
</tr>
<tr>
<td>(Marriott et al. 2009)</td>
<td>Experimental validation of PT bridge pier with external dissipaters</td>
<td>Bridge column</td>
<td>New</td>
<td>Y</td>
<td>External</td>
<td>Mild steel</td>
<td>Y</td>
<td>*External dissipaters result in greater stability and energy dissipation compared to the grouted internal ones&lt;br&gt;*Ability to replace dissipaters quickly&lt;br&gt;*Superior performance compared to monolithic construction</td>
</tr>
<tr>
<td>(Solberg et al. 2009)</td>
<td>Experimental validation of damage avoidance design with</td>
<td>Bridge column</td>
<td>New</td>
<td>Y</td>
<td>Internal</td>
<td>Mild steel</td>
<td>N</td>
<td>*Damage avoidance system survived DBE with minor damage&lt;br&gt;*Collapse prevention of damage avoidance system under MCE is not guaranteed&lt;br&gt;*Damage avoidance system had superior performance compared to benchmark monolithic specimen&lt;br&gt;*Internal dissipaters had insignificant impact</td>
</tr>
<tr>
<td>Author</td>
<td>Research Objective</td>
<td>Structural System</td>
<td>Application</td>
<td>DCR (Y/N)</td>
<td>ED Loc.</td>
<td>ED Type</td>
<td>PT (Y/N)</td>
<td>Relevant Findings</td>
</tr>
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<td>--------</td>
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<td>-----------</td>
<td>--------</td>
<td>----------------</td>
<td>--------</td>
<td>--------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>(ElGawady and Sha’ljan 2011)</td>
<td>Investigation precast segmental bent with different construction details</td>
<td>Precast segmental bridge bent</td>
<td>New</td>
<td>Y</td>
<td>External Mild-steel Angle</td>
<td>Y</td>
<td>*External mild steel angle ED resulted in better performance but sustained damage in the structural elements</td>
<td></td>
</tr>
<tr>
<td>(Marriott et al. 2011)</td>
<td>Effect of biaxial loading on DCR connection with external ED</td>
<td>Bridge column</td>
<td>New</td>
<td>Y</td>
<td>External Mild steel</td>
<td>Y</td>
<td>*Biaxial loading reduces the strength and ductility of DCR system compared to the uniaxial loading, but system remains intact with minor cracking and spalling damage *Requires special detailing for biaxial loading</td>
<td></td>
</tr>
<tr>
<td>(Baird et al. 2013)</td>
<td>Use of innovative cladding connection with UFP</td>
<td>RC frame and precast panel</td>
<td>New &amp; retrofit</td>
<td>N/A</td>
<td>N/A</td>
<td>UFP</td>
<td>N/A</td>
<td>*Energy dissipation through UFP's reduced demand in the structure *Indicated the concept can be implemented for both new construction and retrofit</td>
</tr>
<tr>
<td>(Eatherton et al. 2014a)</td>
<td>Investigating rocking steel frame system and validating analytical models, establishing performance limit states, etc.</td>
<td>Dual and single steel braced frame</td>
<td>New</td>
<td>Y</td>
<td>N/A</td>
<td>Shear type steel butterfly fuse</td>
<td>Y</td>
<td>*Elastic response of the steel frame and PT was maintained up to 2.5% drift ratio *Inelastic response was concentrated at the fuses *Residual drift ratio was negligible with the new system</td>
</tr>
<tr>
<td>(Sideris et al. 2014)</td>
<td>Investigating a novel precast segmental bridge system for earthquake resistance</td>
<td>Precast bridge super and substructure</td>
<td>New</td>
<td>N/A</td>
<td>N/A</td>
<td>No ED</td>
<td>Y</td>
<td>*Both the sliding dominated (SD) and rocking dominated (RD) joints were successful in limiting damage to the structure *RD joints were more effective for self-centering while SD joints were more effective for energy dissipation</td>
</tr>
<tr>
<td>Author</td>
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<tr>
<td>(Guerrini et al. 2015)</td>
<td>Validating precast posttensioned composite hollow core column with supplemental ED</td>
<td>Hollow core bridge column</td>
<td>New</td>
<td>Y</td>
<td>Internal &amp; External</td>
<td>Mild steel</td>
<td>Y</td>
<td>*Metallic aggregate mortar bed with polypropylene fibers used at the joint interface showed excellent performance beyond 5% drift ratio compared to mortar bed without polypropylene *ED fracture occurred at 7.5% drift ratio and crushing of mortar bed was observed priorly</td>
</tr>
<tr>
<td>(Mashal 2015)</td>
<td>Development of low damage earthquake resistant system incorporating accelerated construction</td>
<td>Bridge column</td>
<td>New</td>
<td>Y</td>
<td>External</td>
<td>Grooved dissipaters (GD), Mini UFP dissipaters (MUD)</td>
<td>Y</td>
<td>*DCR connection offer superior seismic performance over emulative cast in place system in terms of limiting damage and post-earthquake repair needs</td>
</tr>
<tr>
<td>(Mashal et al. 2016)</td>
<td>Emulative cast in place construction for accelerated bridge construction in moderate to high seismic region</td>
<td>Bridge column</td>
<td>New</td>
<td>N</td>
<td>N/A</td>
<td>No ED</td>
<td>Y</td>
<td>*Emulative cast in place system offers accelerated bridge construction but sustain significant damage in plastic hinge *Alternative nonemulative solution with ED should be used for important bridges under moderate to high seismic zone *Grouted duct connection showed lower displacement capacity with shorter PH length and member socket connection showed higher displacement capacity and a longer PH length</td>
</tr>
<tr>
<td>(Sarti et al. 2016a)</td>
<td>Development of alternative DCR timber wall connection with boundary column</td>
<td>Timber wall</td>
<td>New</td>
<td>Y</td>
<td>External</td>
<td>UFP</td>
<td>Y</td>
<td>*PT timber wall system are efficient and robust seismic resistant system for multistory building but may cause damage to the floor diaphragm *Alternative system showed stable hysteretic response with significant energy dissipation capacity</td>
</tr>
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<td>(Sarti et al. 2016b)</td>
<td>Estimating response of a posttensioned wall with alternative arrangements and combination of dissipaters</td>
<td>Timber wall</td>
<td>New</td>
<td>Y</td>
<td>Intern and External</td>
<td>Mild steel BRD</td>
<td>Y</td>
<td>*All posttensioned configurations showed excellent performance *Minor damage sustained by the structural element after repetitive test *PT bar and timber element are expected to remain elastic up to MCE demand</td>
</tr>
<tr>
<td>(Tazarv and Saiidi 2016)</td>
<td>Use of UHPC, ECC and SMA to achieve low damage precast pier for accelerated bridge construction</td>
<td>Bridge column</td>
<td>New</td>
<td>N</td>
<td>N/A</td>
<td>No ED</td>
<td>N</td>
<td>*HCS column was efficient in reducing residual displacement compared to CIP *Large crack opening and spalling of ECC was evident in the system, but the damage was significantly less than CIP</td>
</tr>
<tr>
<td>(White and Palermo 2016)</td>
<td>Effectiveness of nonemulative column-footing connection for accelerated bridge construction</td>
<td>Bridge column</td>
<td>New &amp; Repair</td>
<td>Y</td>
<td>Intern and External</td>
<td>Groove d dissipaters (GD) and turned mild steel dissipaters</td>
<td>Y</td>
<td>*Emulative connection achieved comparable energy dissipation and ductility to CIP *Nonemulative socket connection offer significant damage reduction and a flag-shaped hysteretic curve *Premature failure of dissipaters occurred at 3% drift due to detailing error for nonemulative coupled connection indicating construction difficulties with internal ED</td>
</tr>
<tr>
<td>(Thonstad et al. 2017)</td>
<td>Use of precast pretensioned column with confining shoe for low damage self-centering system</td>
<td>Precast column- CIP footing and Precast column- precast cap beam</td>
<td>New</td>
<td>Y</td>
<td>Intern al</td>
<td>Unbonded mild steel rebar</td>
<td>Preten sioned</td>
<td>*No significant strength degradation even after 10% drift *Excellent recentering capacity compared to non-prestressed column *System experienced minimal damage with cosmetic spalling *Longer debonding length could potentially eliminate or delay fracture of rebar</td>
</tr>
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<td>(Yang and Okumus 2017)</td>
<td>Use of UHPC for low damage precast posttensioned self-centering piers</td>
<td>Bridge column</td>
<td>New</td>
<td>Y</td>
<td>N/A</td>
<td>N/A</td>
<td>Y</td>
<td>*Insignificant improvement in strength and stiffness, energy dissipation and self-centering was noticed with UHPC when shear slip and gap opening was allowed but damage limitation showed moderate improvement *When shear slip and gap opening was disallowed UHPC significantly reduced damage and enhanced strength and stiffness with no significant improvement on energy dissipation</td>
</tr>
<tr>
<td>(Andish eh et al. 2018)</td>
<td>Investigating the effect of chloride-induced corrosion on hysteretic and cyclic response of axial metal dissipative devices</td>
<td>Mild steel buckling restrained dissipative devices</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>*Corrosion results in significant degradation of energy dissipation capacity and maximum achievable damping *Unstable hysteretic response resulting from buckling was noticed for supported type dissipaters without continuous support *Impact of corrosion was less pronounced for epoxy grouted anti-buckling dissipaters over other anti-buckling dissipaters</td>
</tr>
<tr>
<td>(Liu 2018)</td>
<td>Explore ways to improve redundancy and robustness of DCR system</td>
<td>Bridge column</td>
<td>New &amp; Repair</td>
<td>Y</td>
<td>External</td>
<td>Mild steel groove d dissipaters</td>
<td>Y</td>
<td>*HHDCR system increases lateral stiffness and energy dissipation following activation of the second set of dissipaters but reduced self-centering capacity *PCDCR configuration performed best in terms of increasing redundancy</td>
</tr>
<tr>
<td>(Mashal and Palermo 2019a)</td>
<td>Development of compact dissipaters to be used with low damage DCR system</td>
<td>Precast bridge bent</td>
<td>New &amp; Repair</td>
<td>Y</td>
<td>External</td>
<td>Mini UFP dissipaters</td>
<td>Y</td>
<td>*Innovative dissipaters offer advantages such as cost effectiveness, fabrication simplicity and promise for excellent seismic performance</td>
</tr>
</tbody>
</table>
### Table A 1: Review of low-damage structural system

<table>
<thead>
<tr>
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<tr>
<td>(Mashal and Palermo 2019b)</td>
<td>Development of a low damage seismic design technology for accelerated bridge construction</td>
<td>Precast bridge bent</td>
<td>New</td>
<td>Y</td>
<td>Y</td>
<td>grooved and mini UFP dissipaters</td>
<td>*ABC low damage system offers superior seismic performance over CIP and emulative system in terms of limiting damage to structural elements and self-centering capability</td>
<td></td>
</tr>
</tbody>
</table>
| (Mashal and Palermo 2019c) | Investigates the use of two emulative connections for accelerated bridge construction in high seismic zone | Precast bridge bent | New | N | No ED | N | *Emulative bent performed like CIP where energy dissipated through formation of plastic hinge  
*Capacity protected members sustained no inelastic damage  
*Member socket connection experienced higher strength degradation and spalling compared to grouted duct connection  
*Emulative bent is expected to remain operational following DBE and prevent collapse under MCE  
*While offering advantage of accelerated construction emulative bent will sustain significant damage necessitating repair or even replacement like CIP construction |
| (Mashal et al. 2019) | Development of concepts, experimental validation of the innovative metallic dissipaters | Brace type and mini UFP dissipaters | New & Repair | Y | N/A | N/A | *Results showed excellent performance of brace type dissipaters with superior energy dissipation capacity and without any sign of strength degradation  
*Application of mini UFP dissipater resulted in a damage free precast bridge bent with full re-centering capability |
### Table A 1: Review of low-damage structural system

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<tr>
<td>(Chen and Popovski 2020)</td>
<td>Propose a material-based modeling method to predict seismic performance of CLT wall system with energy dissipaters</td>
<td>CLT shear wall</td>
<td>New</td>
<td>Y</td>
<td>Exter</td>
<td>Mild steel UFP</td>
<td>Y</td>
<td>*Material based model was able to capture the behavior of PT only CLT wall system with reasonable accuracy</td>
</tr>
<tr>
<td>(Zhang et al. 2020)</td>
<td>Investigating seismic performance of underwater bridge with novel self-centering segmented concrete filled steel tube columns</td>
<td>Single span bridge</td>
<td>New</td>
<td>Y</td>
<td>Exter</td>
<td>Mild steel bars</td>
<td>Y</td>
<td>*PSCFST columns showed good seismic performance with minimal damage and negligible residual displacement *ED bars improved the energy dissipation capacity of the columns through yielding *Presence of water was found to reduce the acceleration response of the system due to added damping properties</td>
</tr>
<tr>
<td>(Nikouk alam and Sideris 2021a)</td>
<td>Investigating seismic performance of polyurethane (PU) enhanced rocking column with energy dissipation links</td>
<td>Bridge column</td>
<td>New &amp; Repair</td>
<td>Y</td>
<td>Exter</td>
<td>Groove type mild steel</td>
<td>Y</td>
<td>*PUED column showed excellent damage resistant properties where fracture of first ED was observed at 6.2% drift *PU column showed no major damage up to 8.2% drift *Repaired PUED column showed similar damage resistant response as to the original column</td>
</tr>
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<tr>
<td>(Nikoukalam and Sideris 2021b)</td>
<td>Focused on quantifying the mechanical properties of ED links and their capability in achieving the properties of undamaged system</td>
<td>Bridge column</td>
<td>New &amp; Repair</td>
<td>Y</td>
<td>External</td>
<td>Grooved type mild steel</td>
<td>Y</td>
<td>*RC rocking column exhibited stable hysteretic response up to 3% drift ratio following which a linear strength and stiffness degradation was *PUED column showed much higher energy dissipation and equivalent damping capability compared to RC rocking *Residual deformation of PUED column can be successfully recovered by releasing and retightening the ED links</td>
</tr>
<tr>
<td>(Thapa and Pantelides 2021)</td>
<td>Design and experimentally validate a hybrid system with tension only hysteretic dissipaters</td>
<td>Two-column precast bridge bent</td>
<td>New</td>
<td>Y</td>
<td>External</td>
<td>Stretch Length Anchors (SLA)</td>
<td>Y</td>
<td>*Hybrid system achieved adequate strength and energy dissipation with reasonable recentering capability *System achieved design objective of reaching 2% drift without any damage and yielding of PT or mild steel bars *Final damage state was cover spalling without core crushing and buckling or fracture of longitudinal rebar which allows rapid repair through concrete patching *SLA bars did not fracture up to 6% drift and polyurethane plate at the interface prevented column toe crushing with improved PT elongation capacity</td>
</tr>
<tr>
<td>(Zhang et al. 2021)</td>
<td>Investigating seismic performance of PSCFST bridge columns with internal and external ED</td>
<td>Bridge column</td>
<td>New &amp; Repair</td>
<td>Y</td>
<td>Internal &amp; External</td>
<td>Mild steel bars</td>
<td>Y</td>
<td>*PSCFST system with ED showed superior seismic performance with minimal damage and excellent self-centering *Repaired system showed potentials for rapid repair following damaging earthquake and achieved favorable seismic performance with negligible residual displacement</td>
</tr>
</tbody>
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Table A 1: Review of low-damage structural system

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DCR=Dissipative Controlled Rocking, ED=Energy Dissipaters, PT=Post-tensioned, N/A=Not Applicable, UFP=U-shaped Flexural Plate, Y=Yes, N=No, DBE=Design Basis Earthquake, MCE=Maximum Credible Earthquake, RC=Reinforced Concrete, PH=Plastic Hinge, GD=Grooved Dissipaters, MUD=Mini UFP Dissipaters, BRD=Buckling Restraint Dissipaters, HCS=Headed reinforcement Coupler Column with SMA, CIP=Cast in Place, ECC=Engineered Cementitious Composite, SMA=Shape Memory Alloy, UHPC=Ultra High-Performance Concrete, HHDCR=Horizontal Hierarchical Activated DCR, PCDCR=Pile-cap Rocking DCR, CLT=Cross-Laminated Timber, PSCFST=Precast Segmental Concrete-Filled Steel-Tube, PU=Polyurethane, PUED=PU-enhanced column with ED links, SLA=Stretch Length Anchors
Figure B-1 Geometry and reinforcing detail of specimens SC, SV, and LV
Figure B-2 Longitudinal and tie bar detail of specimens SC, SV, and LV
Figure B-3 Geometry and reinforcing detail of specimens MS#10
Figure B-4 Longitudinal and tie bar detail of specimens MS#10
Figure B-5 Longitudinal and tie bar detail of specimens CR#8
Figure B-6 Longitudinal and tie bar detail of specimens CR#8
Figure B-7 Longitudinal and tie bar detail of specimens SS#8
Figure B-8 Longitudinal and tie bar detail of specimens SS#8
Appendix C
Specimen Preparation

Figure C-1 Formwork preparation

Figure C-2 Prepared footing formwork for concrete casting
Figure C-3 Footing concrete pour

(a) Column rebar with strain gages

(b) Concrete pour

Figure C-4 Column pour
Appendix C

(a) Slump test  (b) Cylinder casting  (c) Cylinder testing

Figure C-5 Preparation for concrete properties testing

Figure C-6 Loading beam rigging
Figure C-7 Loading apparatus

(a) Specimen painting  (b) Instrumented test ready specimen

Figure C-8 Testing preparation