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Cyclic behavior and design of steel columns subjected to large drift

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Newell, James David

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Cyclic Behavior and Design of Steel Columns Subjected to Large Drift

A Dissertation submitted in partial satisfaction of the requirements for the degree

Doctor of Philosophy

in

Structural Engineering

by

James David Newell

Committee in charge:

Professor Chia-Ming Uang, Chair
Professor David Benson
Professor Thomas Bewley
Professor Tara Hutchinson
Professor P. Benson Shing

2008
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The Dissertation of James David Newell is approved, and it is acceptable in quality and form for publication on microfilm:

Chair

University of California, San Diego

2008
DEDICATION

To all those with whom I have shared the process.
Good judgment is usually the result of experience. And experience is frequently the result of bad judgment. But to learn from the experience of others requires those who have the experience to share the knowledge with those who follow.

*Barry LePatner*
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<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tbody>
<tr>
<td>AISC</td>
<td>American Institute of Steel Construction,</td>
</tr>
<tr>
<td>ASCE</td>
<td>American Society of Civil Engineers,</td>
</tr>
<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials,</td>
</tr>
<tr>
<td>BRB</td>
<td>Buckling-Restrained Brace,</td>
</tr>
<tr>
<td>BRBF</td>
<td>Buckling-Restrained Braced Frame,</td>
</tr>
<tr>
<td>CBF</td>
<td>Concentrically Braced Frame,</td>
</tr>
<tr>
<td>CDF</td>
<td>Cumulative Density Function,</td>
</tr>
<tr>
<td>EBF</td>
<td>Eccentrically Braced Frame,</td>
</tr>
<tr>
<td>ESW</td>
<td>Electro-Slag Welding,</td>
</tr>
<tr>
<td>FCAW</td>
<td>Flux-Cored Arc Welding,</td>
</tr>
<tr>
<td>FEMA</td>
<td>Federal Emergency Management Agency,</td>
</tr>
<tr>
<td>FLB</td>
<td>Flange Local Buckling,</td>
</tr>
<tr>
<td>IBC</td>
<td>International Building Code,</td>
</tr>
<tr>
<td>ICC</td>
<td>International Code Council,</td>
</tr>
<tr>
<td>IDC</td>
<td>Interstory Drift Capacity,</td>
</tr>
<tr>
<td>LA</td>
<td>Los Angeles,</td>
</tr>
<tr>
<td>LMSR</td>
<td>Large Magnitude, Small Distance,</td>
</tr>
<tr>
<td>LRFD</td>
<td>Load and Resistance Factor Design,</td>
</tr>
<tr>
<td>LTB</td>
<td>Lateral-Torsional Buckling,</td>
</tr>
<tr>
<td>MCE,</td>
<td>Maximum Considered Earthquake,</td>
</tr>
<tr>
<td>MRF</td>
<td>Moment Resisting Frame,</td>
</tr>
<tr>
<td>PGA</td>
<td>Peak Ground Acceleration,</td>
</tr>
<tr>
<td>PL</td>
<td>Plate,</td>
</tr>
<tr>
<td>RBS</td>
<td>Reduced Beam Section,</td>
</tr>
<tr>
<td>SDR</td>
<td>Story Drift Ratio,</td>
</tr>
<tr>
<td>SMRF</td>
<td>Special Moment Resisting Frame,</td>
</tr>
<tr>
<td>SRMD</td>
<td>Seismic Response Modification Device,</td>
</tr>
<tr>
<td>TSI</td>
<td>Testing Services and Inspection, Inc.,</td>
</tr>
<tr>
<td>TYP</td>
<td>Typical,</td>
</tr>
</tbody>
</table>
UCSD  University of California, San Diego,
WLB  Web Local Buckling,
kip  Kilo-pound (1000 pounds),
ksi  Kilo-pounds per square inch,
psf  Pounds per square foot,
rad.  Radians, and
sec.  Seconds.
## LIST OF SYMBOLS

- $A_g$: Gross area of member,
- $A_{sc}$: Area of Buckling-Restrained Brace steel core,
- $A_w$: Web area ($=d_t t_w$),
- $B$: Constant to be determined from regression,
- $C$: Constant to be determined from regression,
- $C_a$: Ratio of required strength to available strength,
- $C_b$: Lateral-torsional buckling modification factor for nonuniform moment diagrams when both ends of the unsupported segment are braced,
- $C_{eu}$: Elastic base shear ratio,
- $C_p$: Structural performance parameter,
- $C_s$: Design base shear ratio,
- $C_y$: Base shear ratio at structural yield level,
- $D$: Dead load,
- $D$: Total damage,
- $E$: Earthquake load,
- $E$: Modulus of elasticity,
- $F_{cr}$: Critical stress,
- $F_e$: Elastic critical buckling stress,
- $F_y$: Specified minimum yield stress,
- $F_{yu}$: Actual yield stress,
- $F_{ye}$: Expected yield stress,
- $G$: Shear modulus of elasticity,
- $H$: Column clear length,
- $I_x$: Moment of inertia about the x-axis,
- $K$: Effective length factor,
- $K$: Initial stiffness,
- $K_b$: Rotational flexural stiffness,
- $K_s$: Rotational shear stiffness,
- $K_0$: Rotational stiffness,
$L$ Laterally unbraced length of a member,
$L$ Live load,
$L_b$ Unbraced length,
$M$ Column end moment,
$L_p$ Limiting laterally unbraced length for the limit state of yielding,
$L_r$ Limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling,
$M_{cx}$ Available flexural strength about x-axis,
$M_{cy}$ Available flexural strength about y-axis,
$M_{max}$ Maximum moment,
$M_n$ Nominal flexural strength,
$M_p$ Plastic moment,
$M_{pa}$ Actual plastic moment,
$M_{pc}$ Reduced plastic moment,
$M_{pn}$ Nominal plastic moment,
$M_{rx}$ Required flexural strength about x-axis,
$M_{ry}$ Required flexural strength about y-axis,
$M_u$ Required flexural strength,
$N$ Number of damaging cycles,
$P$ Axial load,
$P_c$ Available axial compressive strength,
$P_G$ Axial load from gravity load,
$P_L$ Measured longitudinal force of the platen,
$P_n$ Nominal axial strength,
$P_r$ Required axial compressive strength,
$P_S$ Axial load from seismic load,
$P_T$ Target column compressive axial load,
$P_u$ Required axial strength in compression,
$P_y$ Yield strength,
$P_{yu}$ Actual axial yield strength,
$P_{yc}$ Compressive yield strength,
$P_{ye}$ Expected axial yield strength,
$P_{yn}$ Nominal axial yield strength,
$P_{yt}$ Tensile yield strength,
$R$ Epicentral distance,
$R$ Response modification coefficient,
$R_i$ Rotational degree of freedom (where $i = 1, 2, 3$),
$R_p$ Plastic rotation capacity,
$R_y$ Ratio of expected yield stress to the specified minimum yield stress,
$S_a$ Site-specific MCE spectral response acceleration,
$S_{D1}$ 5% damped design spectral response acceleration parameter at a period of 1 sec.,
$S_{DS}$ 5% damped design spectral response acceleration parameter at short periods,
$S_x$ Elastic section modulus about the x-axis,
$T$ Column web to base plate fillet weld size,
$T$ Fundamental period of the building,
$T_0 = 0.2 \frac{S_{D1}}{S_{DS}},$
$T_1$ First mode period of vibration,
$T_2$ Second mode period of vibration,
$T_a$ Approximate fundamental period of the building,
$T_S = \frac{S_{D1}}{S_{DS}},$
$U_i$ Translational degree of freedom (where $i = 1, 2, 3$),
$V$ Measured lateral force of the platen,
$Z_x$ Plastic section modulus about the x-axis,
$a$ Distance from origin to $P-M$ target point,
$b$ Distance from origin to $P-M$ interaction surface along line from origin to $P-M$ target point,
$c_p$ Another structural performance parameter
$b_f$ Flange width,
$d_c$ Column depth,
Gravitational acceleration, 
Clear distance between flanges less the fillet or corner radius for rolled shapes, 
Height above base to uppermost level of building, 
Radius of gyration, 
Radius of gyration about y-axis, 
Coefficient of determination, 
Flange thickness, 
Web thickness, 
Imposed lateral displacement of the platen, 
Deformation range of cycle $i$, 
Overstrength factor, 
Exponent to be determined from regression, 
Exponent to be determined from regression, 
Cyclic overstrength factor, 
Exponent to be determined from regression, 
Exponent to be determined from regression, 
Yield strain, 
Exponent to be determined from regression, 
Exponent to be determined from regression, 
Exponent to be determined from regression, 
Stiffness factor ($=\sqrt{P/EI_x}$), 
Resistance factor for flexure, 
Resistance factor for compression, 
Member end rotation, 
Plastic rotation, 
Total end rotation or $IDC$, 
Yield rotation 
Actual yield rotation modified to include the effect of axial compression,
\( \lambda_p \) Limiting slenderness parameter for compact element, and
\( \lambda_{ps} \) Limiting slenderness parameter for seismically compact element.
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VITA

2001 Bachelor of Science, Clarkson University
2001-2003 Research Assistant, Oregon State University
2003 Master of Science, Oregon State University
2003-2008 Research Assistant, University of California, San Diego
2008 Doctor of Philosophy, University of California, San Diego

PUBLICATIONS

Referred Technical Journals


Conference Proceedings


Technical Reports


ABSTRACT OF THE DISSERTATION

Cyclic Behavior and Design of Steel Columns Subjected to Large Drift

by

James David Newell

Doctor of Philosophy in Structural Engineering

University of California, San Diego, 2008

Professor Chia-Ming Uang, Chair

During an earthquake, steel braced frame columns are potentially subjected to high axial forces combined with inelastic rotation demand resulting from lateral story drift. Little design guidance is available concerning the reliability of steel columns under this level of combined loading. To evaluate the performance of wide-flange columns under high axial load and drift demand, specimens have been subjected to laboratory and analytical investigation.

Nine W14 wide-flange columns were tested at different levels of axial force demand (35% to 75% of column yield strength) combined with lateral story drift demand of up to 10%. Since a specified loading sequence does not exist in building
codes, one was developed based on the results of nonlinear earthquake time-history analysis of 3-story and 7-story buckling-restrained braced frame models. The first step in the loading sequence consisted of imposing simulated gravity load. Then in-phase, increasing amplitude cyclic axial load and story drift were applied. Experimental results showed that flange local buckling was the dominant buckling mode. Specimens achieved interstory drift capacities of 0.07 rad. to 0.09 rad., in part, due to the delay in flange local buckling resulting from the stabilizing effect provided by the stocky column web of the W14 specimens.

The finite element analysis program Abaqus was used to model the column specimens and to perform a parametric study investigating the effect of flange and web local buckling, lateral-torsional buckling, and axial load. Analysis and experimental results were observed to be well correlated. The behavior of deep columns (W18 and W24) with higher web slenderness than the tested W14 sections was also investigated. Models of deep column sections showed significant strength degradation due to the interaction of flange and web local buckling.

Testing and finite element analysis indicated that the plastic rotation capacities currently predicted by *Seismic Rehabilitation of Existing Buildings* (ASCE 41) are very conservative for axial load ratios above 0.5. At axial load ratios below 0.5 the plastic rotation capacity of some finite element models was less than that predicted by ASCE 41. Using the database of finite element analysis results and regression analysis, nonlinear models were developed to more accurately predict rotation capacity of steel wide-flange columns. These models consider the interaction of flange and web local buckling, column slenderness, and axial load.
1 INTRODUCTION

1.1 Building Columns

Columns are arguably one of the most critical elements in a building system. They support vertical gravity loads (i.e., self weight of the structure, and occupancy loads) and enable multi-story frame buildings to rise from the ground. Failure of only a few columns has the potential to results in partial or complete collapse of a structure. Certain columns are also a part of the lateral load resisting system, and form an integral link in the load path to resist wind and seismic forces. Columns in the lateral load resisting system are often subjected to a combination of axial and flexural loading. Such members are termed beam-columns because they function as both a beam and a column simultaneously. This dissertation focuses on column members that are part of the lateral load resisting system and are subjected to both axial load and bending. Therefore, they are actually beam-columns. However, for simplicity, in this dissertation the term column will generally refer to a member under combined axial and flexural loading.

1.2 Current Seismic Design Philosophy

For design of gravity framing, member sizes are selected so that members remain elastic under the expected demand. While it is also possible to design a structure to remain completely elastic during an earthquake or other extreme loading event, the resulting building would be impractical to construct, occupy, and be very uneconomical. Typically, buildings are designed for a lower force level by relying on the ductility (inelastic deformation capacity) of properly designed and detailed
systems. The concept of a “structural fuse” is applied in a manner similar to circuit breakers in electrical systems. The overall forces in a structure are limited by the capacity of the structural fuses and the remaining members of the system are design to remain elastic under the maximum demand that can be imposed by the structural fuses.

This seismic design philosophy is illustrated in Figure 1.1, which shows a plot of base shear ratio versus story drift (Uang 1991). The required elastic base shear ratio, $C_{eu}$, is reduced to the design level base shear ratio, $C_s$, by dividing by the response modification coefficient, $R$. $R$ accounts for the ductility of the structural fuse of a particular seismic load resisting system (SLRS). The $C_s$ level corresponds to “first significant yield” or the first significant deviation from elastic global response. Structural fuse members are designed at this force level and only elastic structural analysis is required. The base shear ratio at structural yield, $C_y$, represents the idealized maximum base shear demand. This amplified force level is determined by multiplying $C_s$ by a system overstrength factor, $\Omega_o$. Members other than the structural fuse are designed for the expected capacity of the structural fuse based on the amplified force level ($C_y = \Omega_o C_s$). For steel seismic load resisting systems (SLRSs) $R$ ranges from 3.25 to 8 and $\Omega_o$ from 2 to 3 (ASCE 2005).

1.3 Steel Seismic Load Resisting Systems

Traditionally, steel SLRSs have been divided into moment frame and braced frame systems. Examples of these SLRSs are shown in Figure 1.2. Moment resisting frames (MRFs) are comprised of rigidly connected beams and columns that
resist lateral loading through flexural action. The structural fuse mechanism in a MRF is yielding, or plastic hinging in the ends of the beams adjacent to the columns. Braced frame systems act like a vertically cantilevered truss to resist lateral loading. In a conventionally braced frame (CBF) tension yielding and compression buckling of the diagonal braces is the structural fuse mechanism. Buckling-Restrained Braced Frames (BRBFs) can be considered as a higher performance CBF. Buckling-Restrained Braces (BRBs) are designed such that global brace buckling is prevented. Tension and compression yielding of the BRB steel core provides the ductile structural fuse behavior. Unlike CBFs and BRBFs the diagonal braces of an eccentrically braced frame (EBF) are not the structural fuse and are designed to remain elastic. Ductile performance of EBFs is achieved by shear yielding or a combination of shear and flexural yielding of specially detailed link beams located between the diagonal braces. The performance of the various structural fuses has been well established based on the results of a significant body of research (Malley and Popov 1984, Kasai and Popov 1986, Engelhardt et al. 1998, FEMA 2000b, Black et al. 2002, Newell et al. 2005).

### 1.4 Capacity Design for Columns

Members other than the structural fuse are designed to remain elastic using the concept of capacity based design. For all steel SLRSs the columns are designed to remain essentially elastic at a force level corresponding to the maximum expected demand from the capacity of the structural fuses. The 1985 and prior editions of the *Uniform Building Code* (UBC) did not consider the effect of amplified seismic axial
load or moment in column design for SLRSs. Beginning with the 1988 UBC (ICBO 1988) the significant axial load developed in columns due to system overstrength was treated in the form of amplified seismic force; however, moment demand was still neglected for braced frames. Current state of the practice design procedures consider amplified seismic axial column force but still, with no rational basis, neglect moment demand on braced frame columns.

1.5 Statement of Problem

The BRBF system has been introduced into US practice over about the last 10 years. The system has recently been adopted by building codes and specifications and is included in the 2005 AISC Seismic Provisions for Structural Steel Buildings (AISC 2005b) and 2006 International Building Code (ICC 2006). Prior to the inclusion of the BRBF SLRS in building codes BRBFs were designed as an unclassified system using more sophisticated analysis methods than would be typically used for other SLRSs. Nonlinear time-history analysis of BRBFs conducted for research purposes and design validation revealed that columns in BRBFs are subjected to high axial loads combined with large lateral drift demand. Analysis results have shown expected maximum story drift ratios of about 2% (Sabelli 2001). Experimental testing has shown stable hysteretic behavior of individual BRBs tested in a uniaxial and subassembly configuration well beyond 2% drift (Wantanabe et al. 1988, Wantanabe 1992, Aiken et al. 2000, Black et al. 2002, Merritt et al. 2003a, 2003b, 2003c, Uang and Nakashima 2004, Newell et al. 2005, 2006). However, for column members this level of drift results in inelastic rotational demand combined
with high axial force demand in bottom-story columns and often at intermediate
stories. Columns in concentrically braced frames, eccentrically braced frames, and
moment frames are also subjected to inelastic rotational demand and significant axial
forces. The reliability of column under this level of combined loading had not been
previously experimentally validated.

To provide a basis for performance evaluation of columns in braced frames
subjected to reversing high axial loads combined with high drift demand, the
objectives of this research were to: (1) develop a statistically based loading sequence
for experimental testing of braced frame columns, (2) cyclically test nine full-scale
W14 wide-flange section columns subjected to reversed, increasing amplitude axial
load and drift demand, (3) perform additional parametric studies using finite element
analysis to expand the experimental database to section sizes that were beyond the
scope of experimental testing, and (4) develop an analytical model for prediction of
rotation capacity that is appropriate for incorporation into design standards.

1.6 Dissertation Outline and Chapter Summary

This dissertation begins with a discussion of capacity-based seismic design. Both
system and member level behavior are discussed and the limitations of current
state of the practice for design of seismic load resisting system columns are identified.
Chapters 3-6 describe the results of experimental and analytical work to overcome the
shortcomings of current practice, as identified in Chapter 2. A brief summary of
each chapter follows.
1.6.1 Chapter 1

This chapter has introduced current seismic design philosophy along with the concepts of a “structural fuse” and capacity-based design. A flaw with the capacity-based design concept for steel columns has been revealed that provides motivation for the work undertaken as part of this dissertation. This chapter also provides an outline for the chapters to follow.

1.6.2 Chapter 2

A review of the current state of the practice for seismic design of steel beam-columns is presented in Chapter 2. The results of previous research and its limitations to seismic design are discussed. Current building code based determination of column demand and capacity are presented. The inconsistencies with and limitations of current practice are highlighted.

1.6.3 Chapter 3

Chapter 3 describes the experimental testing program, test setup, instrumentation, and loading sequence. The development of a new and unique dual parameter loading sequence for testing of columns under combined axial and flexural loading is presented. This loading sequence was developed based on the results of nonlinear time-history analysis of 3-story and 7-story prototype BRBF building models subjected to a suite of earthquake ground motion records. Story drift ratio and column axial load and end moment time-history responses were extracted from the analysis results and were converted into series of cycles using a simplified
rainflow cycle counting procedure. This data was then evaluated to develop a loading sequence that conservatively represented the expected column demand. The developed loading sequence was prescribed in terms of story drift ratio and target column compressive axial load.

1.6.4 Chapter 4

The results of laboratory testing of nine full-scale W14 column specimens are presented in Chapter 4. Specimens representing a practical range of flange and web width-to-thickness ratios were subjected to different levels of axial force demand (35%, 55%, and 75% of nominal axial yield strength) combined with up to 10% story drift. No global buckling was observed in all test specimens. Flange local buckling was the dominant buckling mode. Specimens achieved interstory drift capacities of 0.07 to 0.09 rad. These large deformation capacities were, in part, achieved due to the delay in flange local buckling resulting from the stabilizing effect provided by the stocky column web of the W14 section specimens.

1.6.5 Chapter 5

This chapter describes finite element simulation used to populate a database for further analytical investigation. Finite elements models were calibrated using the experimental results discussed in Chapter 4. Models of W12, W14, W18 and W24 column sections with varying column heights and levels of axial load were subjected to the braced frame loading sequence developed in Chapter 3. Analysis data clearly showed a decrease in interstory drift capacities for the deeper W18 and W24 columns.
due to more significant flange and web local buckling. This database of finite element analysis results will be used in Chapter 6 to develop rotation capacity predictive models.

1.6.6 Chapter 6

Chapter 6 discusses limits on allowable column interstory drift and rotation capacity. The ASCE 41 predicted plastic rotation is introduced and compared with the plastic rotation capacity determined from the experimental results discussed in Chapter 4 and finite element results presented in Chapter 5. New interstory drift and plastic rotation capacity models are presented which address the limitations of the ASCE 41 criteria. These new models consider the interaction of flange local buckling, web local buckling, global buckling, and axial load ratio. Modifications to the general models, making them more appropriate for inclusion in design standards, are discussed along with other design recommendations.

1.6.7 Chapter 7

This chapter provides a summary of the experimental and analytical work presented in this dissertation and highlights the original contributions made for the design of steel wide-flange section columns. Conclusions and design recommendations are presented as well as suggestions for future work building upon the results of this dissertation.
Figure 1.1 Base Shear Ratio versus Story Drift
(a) Moment Resisting Frame

(b) Concentrically Braced Frame

(c) Buckling-Restrained Braced Frame

(d) Eccentrically Braced Frame

Figure 1.2 Steel Seismic Load Resisting Systems
2 LITERATURE REVIEW AND CURRENT STATE OF THE PRACTICE

2.1 Introduction

The primary function of a column is to support large vertical loads. For these “force-controlled” members the general design consideration is does the intended member size have adequate strength to carry the required loads without global or local instability. The SLRS structural fuses (i.e., BRBs, EBF link beams, etc.) described in Chapter 1 are considered to be “deformation-controlled” members that must have adequate deformation capacity to undergo the expected maximum single cycle deformation and cumulative deformation demand without excessive loss of strength.

The concern that motivated the American Institute of Steel Construction to initiate this project centered around from the results of time-history analysis of BRBFs that indicated that force-controlled column members are also expected to exhibit at least some minimum level of plastic deformation capacity similar to that of a deformation-controlled member. This chapter discusses previous research and current practice for evaluation of strength and deformation capacity of columns with combined axial and flexural loading.

2.2 Previous Steel Wide-Flange Column Research

2.2.1 Monotonic Testing

Development of beam-column strength interaction equations has been based on tests of a large number of relatively small size hot-rolled wide-flange section beam-columns of ASTM A7 and A36 steel subjected to constant axial load and monotonically increasing end moment applied to failure (Austin 1961, Van Kuren and
Galambos 1964, Bjorhovde et al. 1978, Nakashima et al. 1983, Nakashima et al. 1990). The effects of residual stress (Beedle and Tall 1960, Bjorhovde 1988) and column out-of-straightness have inherently been included in testing and analysis of as-rolled column sections. Historical overview of beam-column interaction equation development has been provided by Sputo (1993).

A database of beam-column test results, as reported in the Japanese literature, has been compiled (Nakashima et al. 1991). The database includes 237 beam-column tests on steel (similar to A36) wide-flange sections subjected to constant applied axial load and monotonically increasing end moments, for a variety of end restraint conditions. Specimens were generally small-scale; for 80% of the specimens the section depth and flange width were less than 6 in. In general, good agreement was observed between specimen capacity and the capacity predicted by current design specifications. The average specimen was 16% stronger than predicted by the LRFD interaction equation using actual material properties and neglecting the strength reduction factors. The variation of steel beam-column ductility capacity was also investigated using the results from this database (Nakashima 1994). Plastic rotation capacities up to approximately 20 were observed. This plastic rotation capacity study will be discussed further in Chapter 6. Parametric studies revealed that residual stress and initial out-of-straightness did not significantly affect specimen ductility.

These US and Japanese experimental studies have limited applicability to seismic design of steel columns as only small sections have been tested; ASTM A7 and A36 material specifications are obsolete for hot-rolled, wide-flange sections; and
loading has consisted of constant compressive axial load combined with monotonically increasing end moments.

### 2.2.2 Cyclic Testing

Beam-column sub-assemblages (Popov and Pinkney 1969, Bertero et al. 1972, Roeder et al. 1993, Schneider et al. 1993) have been subjected to cyclic axial and lateral loading, providing a limited quantity of experimental data on performance and ductility of small-scale beam-columns similar to those under consideration. Weak column-strong beam moment frame testing with constant axial loads (less than 0.25 times the nominal axial yield strength) and both pseudo-dynamic and cyclic lateral loading (Schneider et al. 1993) has shown that an increase in column axial load significantly degraded hysteretic behavior. This has lead to concerns about ductility of columns due to the combination of high column axial load and flexural demand. However, it is believed that column testing under constant compressive axial loads is conservative. During an earthquake, columns would be subjected to reversed loading conditions, and local buckling developed in compression would be partially straightened as the column went into tension.

Results from cyclic testing of five HP14×89 piles subjected to combined vertical and horizontal displacements has shown ductile performance with formation of a plastic hinge below the reinforced concrete pile cap (Astaneh-Asl and Ravat 1997). Strength degradation associated with flange local buckling was observed at large displacements. Specimens sustained rotations exceeding 0.06 rad. before
fracture of the pile flange in the plastic hinge region or before global buckling occurred.

2.3 LRFD Strength Evaluation of Column Members

When a column is subjected to combined axial and flexural loading a portion of the members capacity is consumed by the axial force demand and a portion by the moment demand. An interaction equation, that is dependent on the ratio of axial force demand to capacity, is used to evaluate whether or not the member has adequate strength for the imposed demand. Before evaluating member strength for combined loading the independent axial and flexural capacities must be determined. These capacities are influenced by the member width-to-thickness ratios and corresponding propensity for local buckling.

2.3.1 Classification of Sections for Local Buckling

Section B4 of the AISC Specification for Structural Steel Buildings (AISC 2005c) prescribes certain limits on flange and web width-to-thickness ratios for compact members ($\lambda_p$). Figure 2.1 shows the dimensions used to calculate the flange width-to-thickness ratio ($b_f/2t_f$) and web width-to-thickness ratio ($h/t_w$). These compact section limits are intended to ensure sections are capable of developing the plastic capacity of the section and a rotation capacity of approximately 3 before the onset of local buckling (Yura et al. 1978). For certain members in the SLRS Section 8.2 of the AISC Seismic Provisions for Structural Steel Buildings (AISC 2005b) places further limitations on the width-to-thickness ratio for seismically compact
members ($\lambda_{ps}$). These stricter limits are intended to ensure that seismically compact members are adequate for ductilities to 6 or 7 (Sawyer 1961, Lay 1965). The $\lambda_{ps}$ limits were adapted from those originally developed for plastic design (ASCE 1971). Table 2.1 compares the limiting width-to-thickness ratios for compact and seismically compact members. Note that $\lambda_{ps}$ for the web is dependent on the ratio of axial force demand to axial capacity. Wide-flange shapes used for SLRS columns generally meet these requirements for seismically compact sections.

2.3.2 Axial Compressive Strength

The load and resistance factor design (LRFD) axial compressive strength of doubly symmetric wide-flange members is governed by the limit state of flexural buckling. Compression strength is controlled by the member slenderness ratio, $KL/r$. The effective length factor, $K$, accounts for member end conditions (i.e., fixed, pinned, etc.). For braced frame compression members Section C1.3a of the AISC Specification indicates that a value of $K=1.0$ (pinned-pinned end conditions) shall be used unless analysis indicates a smaller value is appropriate. Chapter E of the AISC Specification defines the design compressive strength using Eqs. 2.1 to 2.4. Note that the AISC Specification equation number is provided in brackets.

\[
\phi_c P_n = \phi_c F_{cr} A_g \tag{2.1}
\]

When $\frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{F_y}}$, \[F_{cr} = \left[ 0.658 \frac{F_y}{F_e} \right] F_y \tag{2.2}\]

When $\frac{KL}{r} > 4.71 \sqrt{\frac{E}{F_y}}$, \[F_{cr} = 0.877 F_e \tag{2.3}\]
\[
F_e = \frac{\pi^2 E}{(KL/r)^2}
\]  

(2.4) [E3-4]

where

\[A_g\] = gross area of member,
\[E\] = modulus of elasticity,
\[F_{cr}\] = critical stress,
\[F_e\] = elastic critical buckling stress,
\[F_y\] = specified minimum yield stress,
\[K\] = effective length factor,
\[L\] = length of member,
\[P_n\] = nominal axial strength,
\[r\] = radius of gyration, and
\[\phi_c\] = resistance factor for compression.

For braced frame columns this design compressive strength is usually governed by out-of-plane (weak-axis) buckling.

These column strength equations include the influence from initial residual stress and out-of-straightness (Galambos 1998). For steel rolled-shapes residual stress result primarily from uneven cooling after the hot-rolling manufacturing process. Residual stress results in yielding of a column earlier than that of an initially stress free section (Beedle and Tall 1970, Salmon and Johnson 1996). Initial out-of-straightness (geometric imperfections) resulting from mill, fabrication, and erection tolerances also reduce the axial strength of columns (Bjorhovde 1988).

2.3.3 Flexural Strength

For a doubly symmetric compact I-shaped member bent about its strong-axis the applicable LRFD flexural strength limit states are yielding and lateral-torsional buckling (LTB). When the member unbraced length, \(L_b\), is less than the limiting
laterally unbraced length for the limit state of yielding, $L_p$, the LTB limit state does not apply and flexural strength is determined by the plastic moment capacity of the section (Eq. 2.5). When $L_b$ is between $L_p$ and the limiting laterally unbraced length for the limit state of inelastic LTB, $L_r$, the flexural capacity is calculated using Eq. 2.6.

When $L_b \leq L_p$

$$
\phi_b M_n = \phi_b M_p = \phi_b F_y Z_x
$$

(2.5) \[\text{[F2-1]}\]

When $L_p < L_b \leq L_r$

$$
\phi_b M_n = \phi_b C_b \left[ M_p - \left( M_p - 0.7 F_y S_x \left( \frac{L_p - L_p}{L_r - L_p} \right) \right) \right] \leq \phi_b M_p
$$

(2.6) \[\text{[F2-2]}\]

where

- $C_b$ = lateral-torsional buckling modification factor,
- $F_y$ = specified minimum yield stress,
- $L_b$ = laterally unbraced length,
- $L_p$ = limiting laterally unbraced length for the limit state of yielding,
- $L_r$ = limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling,
- $M_n$ = nominal flexural strength,
- $M_p$ = plastic bending moment,
- $S_x$ = elastic section modulus about the x-axis,
- $Z_x$ = plastic section modulus about the x-axis, and
- $\phi_b$ = resistance factor for flexure.

For the tested W14 specimens the values of $L_p$ ranged from 13.3 ft to 15.1 ft and $L_r$ from 56 ft to 148 ft. Consideration of LTB did not significantly reduce the flexural capacity of these specimens.

### 2.3.4 Combined Axial and Flexural Strength

Most members in a structure are subjected to combined axial and bending loads. If the magnitude of one of these load effects is relatively small it may be neglected and the member designed as a pure axial or pure bending member.
However, when neither can be neglected, consideration must be given to their combined effect. A linear interaction equation in which the ratio of actual to allowable axial stress plus the ratio of actual to allowable bending stress shall not exceed unity was first introduced in the 1936 AISC Specification. Separate yielding and stability checks were later introduced into design specifications. For LRFD the two separate checks were replaced by a single criterion (split into two regions) that accounts for both yielding and stability. Eqs. 2.7 and 2.8 mathematically represent the interaction surface defined in Chapter H of the AISC Specification (see Figure 2.2).

\[
\begin{align*}
\text{When } & \frac{P_u}{\phi_c P_n} \geq 0.2 \quad \frac{P_u}{\phi_c P_n} + \frac{8}{9} \left( \frac{M_u}{\phi_b M_n} \right) \leq 1.0 \quad \text{(2.7)} \\
\text{When } & \frac{P_u}{\phi_c P_n} < 0.2 \quad \frac{P_u}{2\phi_c P_n} + \frac{M_u}{\phi_b M_n} \leq 1.0 \quad \text{(2.8)}
\end{align*}
\]

where
\begin{align*}
M_n &= \text{nominal flexural strength}, \\
M_u &= \text{required flexural strength}, \\
P_n &= \text{nominal axial strength}, \\
P_u &= \text{required axial strength}, \\
\phi_b &= \text{resistance factor for flexure}, \text{ and} \\
\phi_c &= \text{resistance factor for compression}.
\end{align*}

These equations are also valid for combined axial tension and bending by replacing the axial compression capacity ($\phi_c P_n$) with the tension capacity ($\phi_t P_n$). For braced frame columns the axial capacity is usually controlled by out-of-plane (weak-axis) buckling and the moment capacity by in-plane (strong-axis) bending. Eqs. 2.7 and 2.8 are presented for bending about one axis, but with appropriate modifications are valid for bi-axial bending as well.
2.4 Seismic Design of Steel Columns

2.4.1 Required Column Strength

The AISC Seismic Provisions Section 8.3 requires that when the column axial load ratio \((P_u/\phi_c P_n)\) for LRFD is greater than 0.4, without consideration of the amplified seismic load, the required axial compressive strength, considered in the absence of any applied moment, shall be determined using the load combinations from Minimum Design Loads for Buildings and Other Structures, ASCE 7 (ASCE 2005) that include the amplified seismic load \((\Omega_o E)\). The required axial compressive strength need not exceed the maximum load that can be transferred to the column considering \(1.1R_y\) times the nominal strengths of the connecting beam or brace elements of the building. \((R_y\) is the ratio of expected yield stress to the specified minimum yield stress and the 1.1 factor accounts for the increased demand due to cyclic strain hardening.)

These requirements imply that when the axial load ratio is less than 0.4 the column would possess adequate reserve capacity for any reasonably expected additional axial and/or flexural demand. For higher axial load ratios it has been deemed prudent to consider amplified seismic loads representing reasonable limits on the axial force that can be imposed (based on the expected capacity of structural fuse members). These axial forces are permitted to be applied neglecting any column moment demand. This practice is more based on favorable column performance in past earthquakes and engineering judgment than from rational analysis.

Traditionally, seismic design has been viewed as being closely related to plastic (inelastic) design. AISC Specification Appendix 1.5.1 limits the required
axial strength for braced frame columns designed on the basis of inelastic analysis to $0.85\phi_c F_y A_g$ (equal to $0.765 F_y A_g$). For moment frames, the required axial strength of columns shall not exceed $0.75\phi_c F_y A_g$ (equal to $0.675 F_y A_g$). For both braced frames and moment frames the required axial strength of columns shall not exceed the design strength, $\phi_c P_n$. These limits provide a basis for the upper limit of axial load ($0.75 F_y A_g$) used for experimental testing.

2.4.2 Column Plastic Hinging

In practical applications, the column fixed-base condition in the bottom-story is created by either extending and embedding steel columns in basement walls, if used, or connecting the column base to grade beams. Fixed-base columns are often used for MRFs in order to satisfy drift requirements. Braced frame column bases typically exhibit at least partial fixity due to the brace gusset plate connection to the column and column base plate. Plastic hinging (inelastic rotation demand) of these columns is difficult to avoid. For MRF columns Nakashima (2000) has shown that there is essentially no level of overstrength that will prevent column plastic hinging (i.e., increasing column strength does not prevent plastic hinging). It should be noted that isolated column plastic hinging is not considered to be a significant problem for seismically compact columns with low or moderate axial loads. However, plastic hinging of all columns in any one story results in a very undesirable collapse mechanism (FEMA 2000c).
2.4.3 \textit{P-Δ Effect}

Elastic first-order structural analysis methods are used for hand calculations to determine member internal forces (moment, shear, and axial force) resulting from design loads. The equilibrium equations for these first-order methods are based on the original undeformed geometry of the structure and assume that internal forces are not sufficiently affected by the change in shape due to the applied loads. For unbraced frame (moment frame) columns subjected to lateral loading (wind or seismic) a second-order analysis is required that considers the additional moment demand resulting from the column axial load, $P$, that has been laterally displaced by an amount, $Δ$. For braced frames, second-order effects do not generally significantly contribute to member internal forces and the $P-Δ$ effect can be neglected. However, computer analysis software can easily include $P-Δ$ effects, which can be more significant for the relatively flexible BRBF system where story drifts up to approximately 2% can be expected.

2.5 \textbf{Rehabilitation of Existing Structures}

Existing structures are commonly seismically upgraded when a change in use or occupancy occurs, or because an owner desires improved performance of a building during and after an earthquake. The seismic rehabilitation process typically involves a combination of strengthening the structure and improving the performance of existing components.
2.5.1 Column Strengthening

For seismic rehabilitation of existing structures the designer must develop a rehabilitation scheme that does not overload existing members, or alternatively strengthens the existing members to carry the additional demand. One strengthening scheme for steel wide-flange columns is the addition of cover plates (see Figure 2.3) to increase the capacity of the highly stressed portion of a column. For older buildings, the weldability of the existing column and new cover plate is an important consideration (FEMA 2006). Also, strengthening of the column may necessitate strengthening of the column-to-foundation anchorage or the foundation itself. This column strengthening (and associated potential anchorage and foundation strengthening) is expensive and could potentially be avoided if research results and standards of practice provided guidelines for the force and deformation capacity of columns under high axial load and inelastic rotation demand.

2.5.2 ASCE 41

Little guidance is available in the literature concerning the cyclic plastic rotation capacity (ratio of the inelastic rotation attained to the idealized elastic rotation at first yield) of steel columns subjected to combined axial and flexural loading. Sections 5.4.2.4.2 and 5.4.2.4.3 of Seismic Rehabilitation of Existing Buildings, ASCE 41 (ASCE 2007) do provide plastic rotation capacities for moment frame columns subjected to combined loading. These have been used by designers for braced frame columns in the absence of other guidelines.
According to ASCE 41, columns with a required axial strength ($P_r$) to available axial strength ($P_c$) ratio of 0.5 or less are considered to be deformation-controlled elements with a plastic rotation capacity that is dependent on axial load ratio, flange and web width-to-thickness ratios, and performance objective. Columns with an axial load ratio greater than 0.5 are considered to be force-controlled for combined axial and flexural loading and shall satisfy Eq. 2.9.

$$\frac{P_r}{P_c} + \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \leq 1.0 \quad (2.9)$$

The ASCE 41 criteria, therefore, assume that columns loaded above one-half their available axial strength possess no plastic rotation capacity. These acceptance criteria for nonlinear analysis procedures are summarized in Table 2.2. The plastic rotation in Table 2.2 is dependent on the flange and web slenderness parameters. The slenderness limits for line A are similar to the current AISC Seismic Provisions seismically compact limits. Linear interpolation is allowed for slenderness values between lines A and B. Interpolation for both flange and web slenderness shall be conducted and the lower resulting value used. The plastic rotation versus axial load ratio for seismically compact sections (line A) and the collapse prevention limit state is shown in Figure 2.4. These conservative criteria were developed based on limited available test data and will be compared to the experimental and analytical results of this study in Chapter 6.
2.6 Summary

The current methods for evaluation of column capacity have been developed based on experimental testing of small-scale specimens of a steel grade which is no longer used in new construction. In experimental testing beam-columns have generally been subjected in constant axial load and monotonically increasing moment demand. This method of testing does not adequately represent the axial force reversals expected in a column member during a seismic event.

Little information is available to assist the designer in evaluating the suitability of columns with combined axial load and plastic rotation demand. ASCE 41 contains plastic rotation acceptance criteria for steel moment frame columns for use when nonlinear analysis procedures are used to evaluate building performance. These criteria have also been applied for braced frame columns due to a lack of other applicable guidelines.
### Table 2.1 Limiting Width-to-Thickness Ratios for Compression Elements

<table>
<thead>
<tr>
<th>Description of Element</th>
<th>Limiting Width-to-Thickness Ratio</th>
<th>( \lambda_p ) (compact)</th>
<th>( \lambda_{ps} ) (seismically compact)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Flange</strong></td>
<td>( \frac{b_f}{2t_f} ) = 0.38 ( \frac{E}{F_y} )</td>
<td>0.30 ( \frac{E}{F_y} )</td>
<td>( 3.14 \frac{E}{F_y} (1 - 1.54C_a) )</td>
</tr>
<tr>
<td><strong>Web</strong></td>
<td>( \frac{h}{t_w} ) = 3.76 ( \frac{E}{F_y} )</td>
<td>( C_a \leq 0.125 )</td>
<td>( 1.12 \frac{E}{F_y} (2.33 - C_a) \geq 1.49 \frac{E}{F_y} )</td>
</tr>
</tbody>
</table>

For LRFD \( C_a = \frac{P_u}{\phi b F_y} \)

### Table 2.2 ASCE 41 Plastic Rotation Capacity

<table>
<thead>
<tr>
<th>Axial Load Ratio</th>
<th>Plastic Rotation (×( \theta_{yc} ) rad.)</th>
<th>Immediate Occupancy</th>
<th>Life-Safety</th>
<th>Collapse Prevention</th>
</tr>
</thead>
<tbody>
<tr>
<td>For ( P_r/P_c &lt; 0.20 )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A) ( \frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_y}} ) and ( \frac{h}{t_w} \leq \frac{300}{\sqrt{F_y}} )</td>
<td></td>
<td>1</td>
<td>6</td>
<td>8</td>
</tr>
<tr>
<td>B) ( \frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_y}} ) or ( \frac{h}{t_w} \geq \frac{460}{\sqrt{F_y}} )</td>
<td></td>
<td>0.25</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>For 0.2 &lt; ( P_r/P_c &lt; 0.50 )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A) ( \frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_y}} ) and ( \frac{h}{t_w} \leq \frac{260}{\sqrt{F_y}} )</td>
<td>0.25</td>
<td>8 ( 1 - \frac{5 P_r}{3 P_c} )</td>
<td>11 ( 1 - \frac{5 P_r}{3 P_c} )</td>
<td></td>
</tr>
<tr>
<td>B) ( \frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_y}} ) or ( \frac{h}{t_w} \geq \frac{400}{\sqrt{F_y}} )</td>
<td>0.25</td>
<td>0.5</td>
<td>0.8</td>
<td></td>
</tr>
<tr>
<td>For ( P_r/P_c &gt; 0.50 )</td>
<td></td>
<td></td>
<td></td>
<td>Force-Controlled (see Eq. 2.9)</td>
</tr>
</tbody>
</table>
Figure 2.1 Flange and Web Dimensions for Calculation of Width-to-Thickness Ratios

Figure 2.2 $P-M$ Interaction Curve for Beam-Column Elements
Figure 2.3 Column Cover Plate Strengthening Scheme

Figure 2.4 ASCE 41 Plastic Rotation Capacity (Collapse Prevention Limit State)
3 TESTING PROGRAM AND LOADING SEQUENCE DEVELOPMENT

3.1 Introduction

The experimental portion of this project consisted of testing nine full-scale fixed-base columns. ASTM A992 steel wide-flange sections typical of braced frame columns, representing a practical range of flange and web width-to-thickness ratios, and 15 ft story height were subjected to different levels of axial force demand combined with story drift demand for these simulated bottom-story columns. The test matrix is summarized in Table 3.1. Specimens were designated with the column section size and target axial load, i.e., Specimen W14×132-75 was a W14×132 wide-flange section with a compressive axial load target of 0.75 times the column nominal yield strength, \( P_{yn} \). A total of four W14 sections were selected such that the effect of width-to-thickness ratios on cyclic local buckling behavior could also be investigated. The axial load capacity of the test facility precluded testing of a W14×233 specimen at an axial load of 0.75\( P_{yn} \) and a W14×370 specimen at an axial load of 0.55\( P_{yn} \) or 0.75\( P_{yn} \). However, these two sections were tested at axial loads within the equipment capacity.

3.2 Test Setup

Testing used the University of California, San Diego (UCSD) Seismic Response Modification Device (SRMD) Test Facility as shown in Figure 3.1. Column specimens were tested in a horizontal configuration with one end of the specimen attached to a reaction fixture that was attached to a strong-wall. The other end of the specimen was attached to a reaction fixture attached to the SRMD shake
table platen. Longitudinal (E–W) and lateral (N–S) movement of the shake table platen imposed load in both the column axial and strong-axis bending directions. Control software for the SRMD Test Facility automatically resolved applied loads into longitudinal and lateral components.

### 3.3 Test Specimens

In practical applications, the column fixed-base condition in the bottom-story is created by either extending and embedding steel columns in basement walls, if used, or connecting the column base to grade beams. Since the objective of this research was focused on strength and ductility capacities of steel columns, not the column base connection to surrounding members, both ends of the column specimen were strengthened by stiffeners or re-usable haunches to simulate the fixed-end condition (see Figure 3.1). As described above, this fixed-end condition is appropriate for column bases but underestimates the potential for column lateral-torsional buckling (due to higher moment gradients in an actual building column). Accounting for the flexibility of the column top connection would have required a more complicated test setup and an additional variable would have been added to the testing matrix. Except for both ends, specimens were not braced laterally to inhibit weak-axis buckling. The clear length of column between the haunches at both ends represented the column height of 15 ft. Testing in this manner allowed for investigation of steel column behavior without placing excess demand on the welded column to base plate connection.
Specimens consisted of an 18 ft wide-flange section with 3 in. thick base plates welded on each end (see Figure 3.2). Column flange to base plate welds were electro-slag welds and the column web was fillet welded to the base plate. Specimens were attached to the reaction fixtures with 28 1-1/2 in. diameter high-strength threaded rods per base plate. Re-usable haunches, shown in Figures 3.3 and 3.4, were bolted to both flanges and each side of the web on both ends of the specimen. The 18 in. length of haunches on each end of the specimen resulted in a clear column length of 15 ft. Shim plates were used between the haunches and column to accommodate the variation in column section dimensions for the different specimen sizes.

3.4 Material Properties

Wide-flange column sections were specified to be ASTM A992 material. The values shown in Table 3.2 are the material properties of the column sections obtained from tension coupon testing by Testing Services & Inspection (TSI), Inc. and Certified Mill Test Reports.

Column base plates and plate material for the haunches were specified to be ASTM A36. Haunch-to-column bolts were 1-1/2 in. diameter ASTM A490 high-strength structural bolts and were tensioned to a minimum of 148 kips, the minimum specified for pretensioned joints (AISC 2005c).
3.5 **Instrumentation**

A combination of displacement transducers, strain gage rosettes, and uni-axial strain gages were placed in specific locations on the specimen to measure global and local response. Displacement transducers were positioned as shown in Figure 3.5. The various strain gage rosettes and uni-axial strain gages were used to measure strain throughout the column length (see Figure 3.6). In addition, longitudinal and lateral displacement of the SRMD platen and longitudinal and lateral load applied to move the platen (load applied to specimen) were recorded.

3.6 **Loading Sequence Development**

The moment frame beam-to-column connection loading history in the AISC Seismic Provisions is a sequence based on interstory drift angle. Similarly, the eccentrically braced frame link-to-column connection loading history is based on the link rotation angle. Unlike the moment frame and eccentrically braced frame loading sequences, for testing of columns under axial load and drift a new dual-parameter (i.e., axial load and story drift) loading sequence was required. To develop a rational, statistically based loading sequence for braced frame column testing, 3-story and 7-story BRBF prototype building models were designed and analyzed. Nonlinear time-history analysis of frame models, subjected to a suite of 20 earthquake ground motions, was conducted. Time-histories of bottom-story drift ratio for the 20 records were processed using a rainflow cycle counting procedure. Statistical analysis was used to quantify maximum and cumulative story drift as well as maximum column axial load demand, for development of a loading sequence for experimental testing.
3.6.1 Buildings for Column Demand Study

3-story and 7-story prototype buckling-restrained braced frame buildings, designed for a typical Los Angeles site, were used for loading sequence development. The 7-story building was a design example from Steel TIPS (Lopez and Sabelli 2004). Figure 3.7 shows a plan view and dimensions of the 7-story buildings. BRBFs in the north-south direction (2 braced bays per floor) were modeled as part of this study. Gravity loads used in the design of both the 3-story and 7-story buildings are given in Table 3.3. The 3-story building was specifically designed for this study using the same plan dimensions and gravity loading as the 7-story Steel Tips building, except that the 3-story design used 10 psf partition wall loading, whereas the 7-story design used 20 psf. Elevation views, dimensions, and member sizes for the two frames are provided in Figures 3.8 and 3.9.

3.6.2 Buckling-Restrained Braced Frame Models

3.6.2.1 Geometry

Models of the 3-story and 7-story BRBF were developed and analyzed with the nonlinear structural analysis program DRAIN-2DX (Prakash et al. 1992). Beam and column centerline dimensions were used to define model geometry. Beam-to-column connections with buckling-restrained braces (BRBs) framing in were modeled as fixed connections to account for the rigidity provided by the gusset plate connections. Panel zones were not explicitly modeled and panel zone deformations were neglected.
3.6.2.2 Modelining Techniques

Beams and columns were modeled with inelastic beam-column elements. The axial force-moment yield surface for the beam-column elements is shown in Figure 3.10. A yield stress of 55 ksi and a post-yield stiffness equal to 5% of the elastic stiffness were used for all beam and column members.

BRBs were modeled with inelastic truss elements. It is common to specify a brace core plate yield stress of 42 ksi (±4 ksi) based on project specific tensile coupon testing (Lopez and Sabelli 2004). A brace tensile yield stress of 42 ksi was therefore used in the model. The compressive yield stress was 110% of the tensile yield stress, and the post-yield stiffness equal to 3% of the elastic stiffness, as is typical of BRB component test results (Black et al. 2002).

Gravity loads used in the models are given in Table 3.3. The International Building Code (IBC) gravity load combination of $1.2D + 0.5L$ (ICC 2006) was applied to the model frames during the static pushover and time-history analyses. Gravity loads for the half of the structure associated with each frame, but not directly acting on it, were applied to a $P$-$\Delta$ column. Rayleigh damping (mass and stiffness proportional damping) was used for all elements. Damping coefficients were based on 2% damping in the first and second modes for each frame.

3.6.2.3 Modal and Pushover Analysis

Fundamental natural periods determined from eigenvalue analysis of both model frames are given in Table 3.4. Also provided are the approximate first mode
periods calculated using the empirical formula [Equation 12.8-7 in Minimum Design Loads for Buildings and Other Structures, ASCE 7 (ASCE 2005)].

A pushover analysis was performed for both model frames. The base shear was distributed over the height of the building as specified in the IBC. Figures 3.11(a) and 3.11(b) shows the base shear versus roof drift relationship and sequence of yielding for the 3-story BRBF. A plot of column axial load versus story drift ratio is provided in Figure 3.11(c). Note that the column gravity load offset was equal to $0.12P_n$, where $P_n$ equals the nominal axial compression strength. Figure 3.11(d) shows the axial load-moment ($P$-$M$) interaction curve for the bottom-story column at the base up to the point where the yield surface was reached at 2.5% story drift. Figure 3.12 provides similar plots for the 7-story BRBF pushover analysis. The $P$-$M$ interaction curve [Figure 3.12(d)] is shown up to 2.2% story drift when the yield surface was reached. Column gravity load offset in this case was equal to $0.13P_n$. In both cases the column axial load versus story drift ratio shows an essentially bi-linear relationship similar to BRB demand.

### 3.6.3 Time-History Analysis

#### 3.6.3.1 Earthquake Records

A suite of twenty large magnitude, small distance (LMSR) Los Angeles ground motion records were used for loading protocol development. These records have been used recently for other loading sequence studies (Krawinkler et al. 2003, Medina 2003, Richards and Uang 2006). The ground motion records are herein referred to as P01 to P20. Table 3.5 provides information on the event, source, peak
ground acceleration, and duration of each record. Un-scaled acceleration time-histories are shown in Figure 3.13.

The typical LA site acceleration response spectrum for 5% damping, with $S_{DS} = 0.64$ and $S_{DI} = 1.1$ (see Figure 3.14), was adjusted to a 2% damping spectrum using the scaling procedure in FEMA 356 (FEMA 2000a). The 5% damping spectrum values were divided by 0.8 to obtain the 2% damping spectrum. Both the 2% and 5% damping acceleration response spectra are shown in Figure 3.15.

Scale factors for the earthquake records were calculated to set the 2% damping spectral acceleration of each record equal to the 2% damping design spectral acceleration, at the fundamental natural period of each frame. Each ground motion record was scaled differently for each frame. Scale factors are provided in Table 3.6. Figures 3.16 and 3.17 show the response spectra for the scaled records along with the design spectrum. The vertical lines on these plots indicate the fundamental natural period of the frames. Figure 3.18 shows the average response spectra for each frame for the scaled earthquake records along with the design spectrum.

### 3.6.3.2 Analysis and Data Reduction

Both model frames were subjected to each of the 20 specifically scaled ground motion records. Story drift ratio (SDR), column, and BRB time-history responses were extracted from the analysis results. Typical results are provided in Appendix A for the 3-story BRBF subjected to records P09 and P14 and the 7-story BRBF for records P08 and P19. The figures show story drift ratio, bottom-story column axial
load, and end moment time-histories along with the axial load-moment ($P-M$) interaction for one bottom-story column and BRB hysteretic response for each brace.

For loading protocol development, story drift ratio time-histories were converted into series of cycles using a simplified rainflow cycle counting procedure (Krawinkler et al. 2001, Richards and Uang 2003). This process resulted in symmetric cycles defined by their range (change in peak-to-peak values from time-history) and ordered with decreasing range. Figures 3.19 and 3.20 provide story drift ratio rainflow cycle counting results for both frames and each of the 20 ground motions.

3.6.4 Column Demands

3.6.4.1 Story Drift Demand Parameters

The moment resisting frame (MRF) connection loading protocol in the AISC Seismic Provisions (AISC 2005b) is an interstory drift angle based sequence. Similarly, the eccentrically braced frame (EBF) link-to-column connection protocol is a link rotation angle based sequence. Unlike the MRF and EBF loading protocols, for testing of columns under high axial load and drift demand a dual parameter (i.e., axial load and story drift ratio) loading sequence is required. The development of this braced frame column testing protocol follows the same basic framework as was used in development of the steel moment frame connection loading sequence (Krawinkler et al. 2000) and the EBF link-to-column connection loading sequence (Richards and Uang 2006).
Damage to steel members under cyclic loading is assumed to be described by the cumulative damage model given by Eq. 3.1 (Krawinkler 1996). This cumulative damage model has previously been assumed in development of loading protocols for cyclic testing of beam-to-column moment frame connections and EBF link-to-column connections (Krawinkler et al. 2000, Richards and Uang 2006) and was assumed for development of the column loading sequence.

\[ D = C_p \sum_{i=1}^{N} (\Delta \delta_i)^{c_p} \]  

(3.1)

where
- \( D \) = total damage,
- \( C_p \) = structural performance parameter,
- \( N \) = number of damaging cycles,
- \( \Delta \delta_i \) = deformation range of cycle \( i \), and
- \( c_p \) = another structural performance parameter that is usually greater than 1.

For this damage model the important parameters are number and distribution of damaging cycles, maximum deformation, and cumulative deformation.

**Number of Significant Cycles**

The majority of story drift cycles that a column experiences during an earthquake are very small and do not significantly contribute to cumulative damage. But for almost all excursions greater than the yield drift, column axial load is close to its maximum value. Thus, large elastic cycles and inelastic cycles need to be considered in loading protocol development. Cycles with ranges greater than one-half of the elastic range are considered to be damaging (Krawinkler et al. 2000). Columns are expected to remain elastic for all but a few excursions during even a significant earthquake. A loading sequence developed based on cycles greater than one-half the elastic range of the column would be unrealistically short. Demand on
braced frame columns is directly related to demand imposed by the braces, which in the case of BRBs, are expected to undergo moderate inelastic deformation during a significant earthquake. The range of significant (damaging) story drift ratio cycles has been selected to correspond to one-half that expected at yield of the prototype building BRBs. A typical BRB axial yield displacement of 0.25 in. (Newell et al. 2005) would correspond to a story drift ratio at yield of 0.002 rad. for the prototype frames (chevron bracing configuration and brace inclination angle equal to about 45°). The elastic *range* (defined as peak-to-peak response) is, therefore, 0.004 rad. and one-half this range is 0.002 rad. Story drift ratio cycles with a range greater than 0.002 rad. are considered to be significant.

*Number of “Large” Cycles*

Large story drift ratio cycles are considered to be those with a range greater than 0.005 rad. These cycles contribute the most to column damage.

*Cumulative Range*

Damage to steel members under cyclic loading is assumed to be described by the cumulative damage model given by Eq. 3.1. Therefore, the sum of significant cycle ranges is an important measure of cumulative demand and, combined with experimental results, can provide an indication of when failure of a particular member may occur.

*Maximum Drift Range and Maximum Drift*

The maximum drift range is the largest symmetric cycle coming from the rainflow counting procedure (first cycle in the ordered cycles). The maximum drift is the largest excursion from the time-history results. This maximum drift should
correspond to the point in the developed loading sequence where the cumulative demand is reached.

### 3.6.4.2 Demand from Time-History Analysis

Story drift ratio demand parameters for the 3-story BRBF under each ground motion record are shown in Figure 3.21. Story drift ratio demand parameters for the 7-story BRBF are shown in Figure 3.22. Column axial load demands for the 3-story and 7 story BRBFs are shown in Figures 3.23 and 3.24, respectively. Note that the axial load shown is a combination of gravity and seismic loads. Percentile values of demand parameters, shown in part (b) of Figures 3.21 to 3.24, are based on a lognormal distribution fit to the data shown in the corresponding part (a) figure. For loading protocol development, the number of significant cycles should be represented on average, so 50th percentile values are of interest. All other demand parameters should be represented conservatively in a loading protocol, so 90th percentile values are of interest (Krawinkler et al. 2000). Tables 3.7 and 3.8 summarize story drift ratio and column axial load demand parameters for the 3-story and 7-story frames. The 90th percentile values of column compressive axial load indicate that the target loads (e.g., 0.35\(P_{yn}\), 0.55\(P_{yn}\), 0.75\(P_{yn}\)) for this project were within a reasonable range.

### 3.6.5 Braced Frame Column Testing Loading Sequence

A proposed loading sequence for braced frame column testing subject to combined axial load and story drift ratio demand was developed, based on the results of the time-history analysis described above. The loading sequence is prescribed in
terms of story drift ratio (see Table 3.9). Note that the initial step in the experimental protocol was application of gravity (compressive) load.

It was observed from the time-history analysis that for all excursions larger than the yield drift the column axial load approached a constant maximum value. Therefore, column axial loads for the testing sequence were determined based on the elastic-perfectly plastic column axial load versus story drift ratio relationship shown in Figure 3.25. Calculation of column axial loads at the protocol drift levels are based on reaching the target column compressive axial load, $P_T$, (e.g., $0.35P_{y_n}$, $0.55P_{y_n}$, $0.75P_{y_n}$) at 0.002 rad. story drift ratio (yield drift) and axial loads for all excursions larger than the yield drift were equal to the maximum level for that specimen. The story drift and column axial load were in phase (see Figure 3.26) to represent realistic frame action.

Figure 3.27 shows the cumulative density functions (CDFs) from the bottom-story column drift cycle data and discrete CDFs for the proposed loading sequence. The CDF indicates the percentage of cycles having a range less than some given range. The discrete CDFs of the proposed loading sequence are below the time-history analysis data CDFs, indicating that the protocol is conservative and contains a greater percentage of large amplitude cycles as compared with the data.

### 3.6.6 Modified Loading Sequence

The loading history for the test specimens was developed as described above. Three different loading schemes were used during testing to overcome difficulties encountered in simultaneously controlling the SRMD input longitudinal and lateral
displacements to achieve both the column axial load and story drift targets. Table 3.10 provides the specimen testing order and loading scheme (A, B, or C) used for each specimen. For all loading schemes the story drift ratio combined with a column clear length of 15 ft was used to compute the input lateral displacements for the SRMD platen. The target column axial load was used to calculate a target axial displacement based on specimen axial stiffness and a known SRMD system stiffness in the longitudinal direction.

Loading Scheme A (Specimens W14×132-35 and W14×176-35) was based on an earlier version of the loading sequence than presented in this chapter. BRBs were assumed to yield at 0.003 rad. story drift ratio and a bilinear column axial load versus story drift relationship (see Figure 3.28) was used to calculate column axial loads. This calculation was based on reaching the target column compressive axial load, $P_T$, $(0.35P_{y_n})$ at 6% story drift. Table 3.11 provides the Loading Scheme A story drift ratio sequence. The loading sequences for Specimens W14×132-35 and W14×176-35 are shown in Figures 3.29 and 3.30, respectively. Axial load and drift were in-phase to represent realistic frame action. Note that Specimen W14×132-35 was tested to 6% drift and Specimen W14×176-35 was tested to 10% drift. It was observed during the testing and data analysis of these two specimens that for large cycles, greater than 2% drift, the column axial load targets were not achieved. This was in part due to the significant column axial displacement component from applied lateral displacement (drift) and in part due to softening of column response from yielding and local buckling.
Loading Scheme B (Specimens W14×132-55) was based on the loading sequence presented in Section 3.6.5 (see Table 3.9). Axial load and drift were simultaneously applied in-phase with each other. Loading Scheme B was different from Loading Scheme A in that it accounted for the significant column axial displacement component from drift in calculation of the applied longitudinal displacements. However, the column axial load targets for large drift cycles were still not achieved due to softening of column response from yielding and local buckling and, therefore, Loading Scheme C was developed.

Loading Scheme C (all remaining specimens) was identical to Loading Scheme B through the 1.5% drift cycles. Calculation of column axial loads at the prescribed drift levels were based on reaching the target column compressive axial load, $P_T$, (e.g., $0.35P_{yn}$, $0.55P_{yn}$, $0.75P_{yn}$) at 0.002 rad. story drift ratio (yield drift) and axial loads for all excursions larger than the yield drift were equal to the maximum level for that specimen. The story drift and column axial load were in-phase (see Figure 3.26) to represent realistic frame action. For cycles at 2% drift and beyond the drift component of displacement was applied first, followed by application of longitudinal displacement until the target axial load was achieved. This part of the loading scheme is illustrated in Figure 3.31. By loading in this manner the tests could still be safely conducted in displacement control and the combined target axial loads and story drifts could be achieved.

Figure 3.32 shows typical axial load-moment ($P-M$) interaction and end moment versus drift response for the 4% drift cycle of Specimen W14×132-75 (Loading Scheme C). The labeled points on the graphs correspond to points from the
loading sequence shown in Figure 3.32(a) and indicate the direction of motion in these responses. The trends observed from this single cycle are typical of those observed for this form of the loading sequence. Points 2 and 5 correspond to points where the target values of axial load and drift were reached. These points will be indicated as discrete points in the results presented in Chapter 4. In particular, point 5 (target compressive axial load and drift) was of primary interest for defining the test specimen interstory drift capacity (to be defined in Chapter 4) under high axial load.

3.7 Summary and Conclusions

Since a prescribed loading sequence for cyclic testing of braced frame columns did not exist one was developed based on the results of nonlinear time-history analysis of 3-story and 7-story BRBF prototype buildings, subjected to a suite of 20 earthquake ground motion records. Bottom-story drift ratio time-histories were processed using a rainflow cycle counting procedure. Statistical analysis was used to quantify maximum and cumulative column demands and a combined axial load and story drift loading sequence for experimental testing was developed. The increasing amplitude loading sequence was controlled by story drift ratio, and column axial loads were determined based on idealized brace demand on the columns. Time-history analysis indicated that for all cycles beyond the yield displacement of the BRBFs (approximately 0.002 rad.) bottom-story column axial load approached a constant maximum value. Loading sequence column axial loads were therefore calculated assuming elastic-perfectly plastic axial load versus drift behavior. Story drift and axial load were in-phase to represent realistic frame action.
Table 3.1 Test Matrix

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>Width-to-Thickness Ratio</th>
<th>Gravity Load (0.15P_n)</th>
<th>Total Column Axial Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>b/2t_f h/t_w</td>
<td></td>
<td></td>
</tr>
<tr>
<td>W14×132-35</td>
<td>7.15 17.7</td>
<td>246 kips</td>
<td>0.35P yn = 679 kips</td>
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<tr>
<td>W14×132-55</td>
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<td>0.55P yn = 1067 kips</td>
</tr>
<tr>
<td>W14×132-75</td>
<td></td>
<td></td>
<td>0.75P yn = 1455 kips</td>
</tr>
<tr>
<td>W14×176-35</td>
<td>5.97 13.7</td>
<td>336 kips</td>
<td>0.35P yn = 907 kips</td>
</tr>
<tr>
<td>W14×176-55</td>
<td></td>
<td></td>
<td>0.55P yn = 1425 kips</td>
</tr>
<tr>
<td>W14×176-75</td>
<td></td>
<td></td>
<td>0.75P yn = 1943 kips</td>
</tr>
<tr>
<td>W14×233-35</td>
<td>4.62 10.7</td>
<td>446 kips</td>
<td>0.35P yn = 1199 kips</td>
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<tr>
<td>W14×233-55</td>
<td></td>
<td></td>
<td>0.55P yn = 1884 kips</td>
</tr>
<tr>
<td>W14×370-35</td>
<td>3.10 6.89</td>
<td>718 kips</td>
<td>0.35P yn = 1908 kips</td>
</tr>
</tbody>
</table>

Table 3.2 Steel Mechanical Properties

<table>
<thead>
<tr>
<th>Member</th>
<th>Steel Grade</th>
<th>Yield Strength&lt;sup&gt;a&lt;/sup&gt; (ksi)</th>
<th>Tensile Strength&lt;sup&gt;a&lt;/sup&gt; (ksi)</th>
<th>Elongation&lt;sup&gt;a,b&lt;/sup&gt; (%)</th>
<th>Heat No.</th>
<th>Steel Mill</th>
</tr>
</thead>
<tbody>
<tr>
<td>W14×132</td>
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<td>68.7 (55.5) 55.9</td>
<td>77.9 (71.5) 79.0</td>
<td>36 (24) 37</td>
<td>249171</td>
<td>Nucor-Yamato</td>
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<tr>
<td>Flange</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Web</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W14×176</td>
<td>A992</td>
<td>59.4 (56.5) 58.3</td>
<td>80.5 (75.5) 79.7</td>
<td>46 (23.5) 32</td>
<td>238477</td>
<td>Nucor-Yamato</td>
</tr>
<tr>
<td>Flange</td>
<td></td>
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<tr>
<td>Web</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>W14×233</td>
<td>A992</td>
<td>61.0 (60.0) 59.7</td>
<td>85.6 (77.0) 85.3</td>
<td>38.5 (24.5) 42</td>
<td>251442</td>
<td>Nucor-Yamato</td>
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</tr>
<tr>
<td>W14×370</td>
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<td>60.0 (54.5) 55.0</td>
<td>81.1 (72.5) 76.0</td>
<td>44 (26) 40</td>
<td>251259</td>
<td>Nucor-Yamato</td>
</tr>
<tr>
<td>Flange</td>
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<td></td>
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<tr>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

<sup>a</sup>Values in parentheses are based on Certified Mill Test Reports, others from testing by TSI.

<sup>b</sup>Certified Mill Test Report elongation in parentheses based on 8 in. gage length, others based on 2 in. gage length.
Table 3.3 Gravity Loads Used for Design and Analysis

(a) Roof Loads

<table>
<thead>
<tr>
<th>Element</th>
<th>Loading (psf)</th>
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<tbody>
<tr>
<td>Roofing and Insulation</td>
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<tr>
<td>Steel and Concrete Deck</td>
<td>47.0</td>
</tr>
<tr>
<td>Steel Framing and Fireproofing</td>
<td>8.0</td>
</tr>
<tr>
<td>Ceiling</td>
<td>3.0</td>
</tr>
<tr>
<td>Mechanical/Electrical</td>
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<tr>
<td>Curtain Wall</td>
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</table>

(b) Floor Loads

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<thead>
<tr>
<th>Element</th>
<th>Loading (psf)</th>
</tr>
</thead>
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<tr>
<td>Steel and Concrete Deck</td>
<td>47.0</td>
</tr>
<tr>
<td>Steel Framing and Fireproofing</td>
<td>8.0</td>
</tr>
<tr>
<td>Partition Walls</td>
<td>10.0&lt;sup&gt;a&lt;/sup&gt;, 20.0&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>Ceiling</td>
<td>3.0</td>
</tr>
<tr>
<td>Mechanical/Electrical</td>
<td>2.0</td>
</tr>
<tr>
<td>Curtain Wall</td>
<td>15.0</td>
</tr>
</tbody>
</table>

<sup>a</sup>3-Story  
<sup>b</sup>7-Story (Lopez and Sabelli 2004)

Table 3.4 Predicted Natural Periods

<table>
<thead>
<tr>
<th>Frame</th>
<th>$T_1$ (sec.)</th>
<th>$T_2$ (sec.)</th>
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</thead>
<tbody>
<tr>
<td>3-Story BRBF</td>
<td>0.510</td>
<td>0.207</td>
</tr>
<tr>
<td></td>
<td>(0.300)&lt;sup&gt;a&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>7-Story BRBF</td>
<td>0.909</td>
<td>0.328</td>
</tr>
<tr>
<td></td>
<td>(0.550)</td>
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</tr>
</tbody>
</table>

<sup>a</sup>Values in parentheses calculated using $T_a=(0.02)h_a^{0.34}$ per ASCE 7-05
<table>
<thead>
<tr>
<th>Name</th>
<th>Event</th>
<th>Station</th>
<th>R (km)</th>
<th>PGA (g)</th>
<th>Duration (sec.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P01</td>
<td></td>
<td>Agnews State Hospital</td>
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<td>0.172</td>
<td>40.0</td>
</tr>
<tr>
<td>P02</td>
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<td>Capitola</td>
<td>14.5</td>
<td>0.443</td>
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<tr>
<td>P03</td>
<td>Loma Prieta (1989)</td>
<td>Gilroy Array #3</td>
<td>14.4</td>
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<td>39.9</td>
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<tr>
<td>P04</td>
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<td>Gilroy Array #4</td>
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<td>P06</td>
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<td>Hollister City Hall</td>
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<tr>
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<tr>
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<td>Sunnyvale-Colton Ave.</td>
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<td>39.3</td>
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<tr>
<td>P09</td>
<td>Northridge (1994)</td>
<td>Canoga Park-Topanga Can.</td>
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<td>P10</td>
<td></td>
<td>LA-N Faring Rd.</td>
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<td>P11</td>
<td></td>
<td>LA-Fletcher Dr.</td>
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<td>P12</td>
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<td>Glendae-Las Palmas</td>
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<td>0.206</td>
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<td>P13</td>
<td></td>
<td>LA-Hollywood Store FF</td>
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<td>La Crescenta-New York</td>
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<tr>
<td>P16</td>
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<td>P18</td>
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<td>Plaster City</td>
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<td>Westmoreland Fire Station</td>
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<td>0.172</td>
<td>40.0</td>
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Table 3.6 Ground Motion Scaling Factors

<table>
<thead>
<tr>
<th>Name</th>
<th>Scaling Factor</th>
<th>3-Story BRBF</th>
<th>7-Story BRBF</th>
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<tbody>
<tr>
<td>P01</td>
<td>2.940</td>
<td>2.899</td>
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</tr>
<tr>
<td>P02</td>
<td>1.253</td>
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<td>P03</td>
<td>1.461</td>
<td>2.166</td>
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<td>P04</td>
<td>1.195</td>
<td>2.182</td>
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<td>P05</td>
<td>2.313</td>
<td>5.715</td>
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<td>P18</td>
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<td>1.691</td>
<td>5.028</td>
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<td>P20</td>
<td>2.593</td>
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Table 3.7 Column Story Drift Demand

(a) 3-Story BRBF
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<th>Analysis</th>
<th>Protocol</th>
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<tbody>
<tr>
<td>Number of Significant Cycles</td>
<td>31</td>
<td>36</td>
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<tr>
<td>Number of Large Cycles</td>
<td>11</td>
<td>18</td>
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<tr>
<td>Cumulative Demand</td>
<td>0.22 rad.</td>
<td>0.28 rad.</td>
</tr>
<tr>
<td>Maximum Cycle Range</td>
<td>0.023 rad.</td>
<td>0.03 rad.</td>
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<tr>
<td>Maximum Cycle</td>
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(b) 7-Story BRBF
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<th>Analysis</th>
<th>Protocol</th>
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</thead>
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<td>Number of Significant Cycles</td>
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<tr>
<td>Number of Large Cycles</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>Cumulative Demand</td>
<td>0.20 rad.</td>
<td>0.28 rad.</td>
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<tr>
<td>Maximum Cycle Range</td>
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<td>0.03 rad.</td>
</tr>
<tr>
<td>Maximum Cycle</td>
<td>0.013 rad.</td>
<td>0.015 rad.</td>
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Table 3.8 Column Axial Load Demand

(a) 3-Story BRBF
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<th>Column Axial Load</th>
</tr>
</thead>
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(b) 7-Story BRBF
<table>
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<td>Minimum</td>
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<td>40.7</td>
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Table 3.9 Proposed Story Drift Ratio Loading Sequence

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<th>Number of Cycles</th>
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<tr>
<td>0</td>
<td>Apply column axial gravity load</td>
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<tr>
<td>1</td>
<td>0.001</td>
<td>6</td>
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<td>8</td>
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</tr>
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<td>10(^a)</td>
<td>0.02</td>
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</table>

\(^a\)Continue with increments in SDR of 0.01, and perform one cycle at each step

Table 3.10 Test Sequence and Loading Scheme

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<tr>
<th>Specimen Designation</th>
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<td>3</td>
<td>B</td>
</tr>
<tr>
<td>W14×132-75</td>
<td>5</td>
<td>C</td>
</tr>
<tr>
<td>W14×176-35</td>
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<td>A</td>
</tr>
<tr>
<td>W14×176-55</td>
<td>4</td>
<td>C</td>
</tr>
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<td>W14×176-75</td>
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<td>W14×233-35</td>
<td>7</td>
<td>C</td>
</tr>
<tr>
<td>W14×233-55</td>
<td>8</td>
<td>C</td>
</tr>
<tr>
<td>W14×370-35</td>
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<td>C</td>
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Table 3.11 Loading Scheme A: Story Drift Ratio Loading Sequence

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<th>Story Drift Ratio</th>
<th>Number of Cycles</th>
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</thead>
<tbody>
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<td>0</td>
<td>Apply column axial gravity load</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.001</td>
<td>6</td>
</tr>
<tr>
<td>2</td>
<td>0.002</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>0.003</td>
<td>6</td>
</tr>
<tr>
<td>4</td>
<td>0.004</td>
<td>6</td>
</tr>
<tr>
<td>5</td>
<td>0.005</td>
<td>4</td>
</tr>
<tr>
<td>6</td>
<td>0.0075</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>0.01</td>
<td>2</td>
</tr>
<tr>
<td>8</td>
<td>0.015</td>
<td>2</td>
</tr>
<tr>
<td>9(^a)</td>
<td>0.02</td>
<td>1</td>
</tr>
</tbody>
</table>

\(^a\)Continue with increments in SDR of 0.01, and perform one cycle at each step
Figure 3.1 SRMD Test Facility

(a) Schematic View

(b) Overview of Specimen

Figure 3.1 SRMD Test Facility
(a) Overall Dimensions

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Figure 3.2 Specimen Details
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(a) Pushover Response

(b) Sequence of Yielding

Figure 3.11 3-Story BRBF: Pushover Analysis Results
Figure 3.11 3-Story BRBF: Pushover Analysis Results (cont.)

(c) Bottom-Story Column Axial Load versus Story Drift Ratio

(d) Bottom-Story Column Axial Load-Moment Interaction
Figure 3.12 7-Story BRBF: Pushover Analysis Results

(a) Pushover Response

(b) Sequence of Yielding

Figure 3.12 7-Story BRBF: Pushover Analysis Results
Figure 3.12 7-Story BRBF: Pushover Analysis Results (cont.)

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Figure 3.13 Acceleration Time Histories of Un-scaled Ground Motions (cont.)
Figure 3.14 IBC Design Acceleration Response Spectrum (5% Damping)

Figure 3.15 Design Acceleration Response Spectra for 2% and 5% Damping
Figure 3.16 3-Story BRBF: Scaled 2% Damped Acceleration Response Spectra
Figure 3.16 3-Story BRBF: Scaled 2% Damped Acceleration Response Spectra (cont.)
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Figure 3.20 7-Story BRBF: Bottom-Story Drift Ratio Rainflow Counting Cycles (cont.)
(a) Data for Each Record         (b) Percentile Values

Figure 3.21 3-Story BRBF: Story Drift Ratio Demand
Figure 3.22 7-Story BRBF: Story Drift Ratio Demand
Figure 3.23 3-Story BRBF: Column Axial Load Demand

Figure 3.24 7-Story BRBF: Column Axial Load Demand
Figure 3.25 Elastic-Perfectly Plastic Column Axial Load versus Story Drift Ratio Relation

(a) Seismic Load

(b) Combined Seismic and Gravity Load
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Figure 3.27 Comparison of Bottom-Story Drift Ratio CDFs

(a) 3-Story BRBF

(b) 7-Story BRBF
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(a) Seismic Load

(b) Combined Seismic and Gravity Load
Figure 3.29 Specimen W14×132-35: Loading Sequence

(a) Lateral Deformation

(b) Axial Load
Figure 3.30 Specimen W14×176-35: Loading Sequence

(a) Lateral Deformation

(b) Axial Load
Figure 3.31 Sequence for Loading Scheme C (2% Drift and Beyond)

0-1 Sweep arc to target drift
1-2 Increase longitudinal displacement until target longitudinal load is reached
2-3 Return to original set-point
3-4 Sweep arc to target drift
4-5 Increase longitudinal displacement until target longitudinal load is reached
Figure 3.32 Specimen W14×132-75: 4% Drift Cycle
4 EXPERIMENTAL RESULTS

4.1 Introduction

This chapter presents a summary of experimental results for the tested W14 column specimens. Appendix B provides detailed results for each of the nine specimens. Figures are included in Appendix B which show the progression of yielding, flange local buckling, and overall deformation with increasing drift. Also included, where appropriate, are figures showing specimen fracture. Plots of $P-M$ interaction and moment versus drift response illustrate specimen global behavior.

Table 4.1 provides a summary of the maximum drift imposed on each specimen and well as a description of any specimen fractures. Specimens W14×132-75, W14×176-35, W14×176-55, and W14×176-75 eventually experienced partial or complete fracture of the column flange through the haunch bolt hole net section (see Figure 4.1). Specimens W14×233-35 and W14×370-35 fractured near the column flange to base plate complete joint penetration groove welded joint (see Figure 4.2). With the exception of Specimen W14×370-35, which failed at 3% drift, the fracture of the other specimens occurred at 8% drift or larger, which was well beyond the expected seismic drift demand on these columns. It should also be noted that the flange net section fracture failure mode was associated with the haunches used as part of the test setup and not a typical detail used in building construction.

The specimen end moment was calculated from Eq. 4.1 with the assumption that the inflection point (double curvature bending) was in the middle of the column length.
\[ M = \frac{1}{2} (VH - P_L \Delta) \]

where
\( H \) = column clear length,
\( M \) = end moment,
\( P_L \) = measured longitudinal force of the platen (positive for tension),
\( V \) = measured lateral force of the platen, and
\( \Delta \) = imposed lateral displacement of the platen.

Figure 4.3 shows column lateral displacement profiles at 2% drift for the five specimens tested using Loading Scheme C. These measured profiles support the assumption that the inflection point was located in the middle of the column length.

### 4.2 Flange Yielding

For all specimens only minor yielding was observed at a story drift of 1.5%, the maximum expected drift from the nonlinear time-history analysis described in Chapter 3. Figure 4.4 shows typical example photos of the progression of flange yielding from 2% to 10% drift for Specimen W14×132-75. The extent of flange yielding, as evidenced by flaking of the whitewash, measured after testing is shown in Figure 4.5 for all specimens. Combined compressive axial load and bending resulted in increased compressive stresses on one flange (referred to as the compression flange) and the tensile axial load and bending resulted in increased tensile stresses on the opposite flange. For this testing the compressive axial load was greater than the tensile axial load as a result of the gravity load offset. Consistent with this applied loading the compression flange yielded length was observed to be greater than the tension flange yielded length. For the 0.75\( P_{yn} \) specimens the compression flange yielded length was approximately two times the column depth. It is noted that this is...
two times the normally assumed plastic hinge length for moment frame beams (FEMA 2000b). These observations were supported by the measured strain profiles included in Appendix B.

4.3 Local Buckling

Flange local buckling (FLB) was observed for all specimens with the exception of Specimen W14×370-35 (which failed at 3% drift before any local buckling developed). FLB occurred in a symmetric mode (i.e. deformation of the flange on opposite sides of the web was in the same direction). No web local buckling (WLB) was evident for any of the specimens. During testing it was observed that FLB forming on the negative drift excursion would fully straighten on the following positive drift excursion and visa-versa. This cyclic straightening continued to occur until approximately 8% drift when the buckled amplitude would only partially straighten on the subsequent opposite drift excursion. Figure 4.6 shows photos of the progression of FLB for Specimen W14×132-75 taken at the compression target points from 4% to 10% drift. Figure 4.7 shows a comparison of FLB photos for the different specimens at 6% drift (compressive axial load) and Figure 4.8 at 8% drift. As anticipated, the amplitude of FLB was more significant for increasing flange width-thickness ratio and increasing axial load. The relatively small amplitude of FLB and absence of WLB at 6% drift (more than three times the maximum drift from the analysis of Chapter 3) provided an indication that column strength degradation due to local buckling is not expected to be of critical importance for the seismic design of
the tested column sections. This may not be the case, however, for deep columns which have higher web slenderness than the W14 sections tested.

4.4 Axial Load-Moment Interaction and End Moment versus Drift Response

The axial load-moment ($P-M$) interaction surface, shown in Figure 4.9, is provided along with the experimental and analytical results in the following chapters. Plastic moment capacity, $M_p$, was calculated as the plastic section modulus times the yield strength. Tension axial load capacity, $P_{ty}$, was calculated as the section area times the yield strength. Compression axial load capacity, $P_{cy}$, was calculated based on weak axis buckling with an effective length factor equal to 0.65 (fixed-fixed end conditions). The column length in these calculations was assumed to equal 16.5 ft (average of 18 ft total length and 15 ft clear length) to account for the fact that the haunches did not provide a perfectly fixed end condition. Chapter 4 presents $P-M$ interaction surfaces calculated using both nominal and actual material properties. Note that no LRFD strength reduction factors were applied in determination of the $P-M$ interaction surface ordinates.

Figures 4.10 to 4.13 show a comparison of the $P-M$ interaction for all tested specimens. The $P-M$ test data and the established $P-M$ interaction surface agreed well with each other. The cyclic overstrength factor ($\beta_o$) was determined by dividing length $a$ by length $b$, as shown in Figure 4.14. Normalized $P-M$ data points at 4% story drift are shown in Figure 4.15 and calculated $\beta_o$ values are provided in Table 4.2. The average value of $\beta_o$ at 4% story drift was 1.05. Overstrength tended to be higher for heavier sections with smaller width-thickness ratios.
The moment versus drift response for all specimens is compared in Figures 4.16 to 4.19. Recall that the discrete points on the figures indicate where the combined axial load and drift targets were reached. The points on the positive drift side that show a significantly reduced capacity occurred after partial or complete fracture of the specimen. The overall moment versus drift response was very similar for each specimen due to the sequence in which the load was applied. Recall that for 2% drift and beyond in Loading Scheme C lateral displacement was first applied until the target drift was reached and then axial load was increased to the target value. The points included in the moment versus drift plots represent the actual behavior of interest in this study. By comparing the results from, for example, Figures 4.17(b) and 4.17(c) the reduced moment capacity due to increased axial load is evident.

4.5 Plastic Rotation Capacity

Figure 4.20 shows an example plot of the end moment versus drift response envelope based on the combined axial load and drift target points on the compression side; that is, the points represent the end moment versus story drift relation under constant axial load. End moment has been normalized by the actual plastic moment, $M_{pa}$, calculated based on measured material properties. In this study, the interstory drift capacity ($IDC$) was defined as that when the peak end moment, $M_{max}$, has degraded by 10%. The horizontal line shown is at 90% of $M_{max}$. This 10% reduction from the peak moment resistance may be conservative, but given the critical function of columns to the overall stability of a structure, was deemed appropriate at this time. (For comparison, SMRF beam-to-column moment connections require the
measured flexural resistance of the connection to be at least 0.80$M_p$, or an allowable 20% reduction from $M_p$.

The story drift ratio in Figure 4.20 corresponds to $\theta$ in Figure 4.21(a). With a rigid-body rotation of this test configuration, Figure 4.21(b) shows that $\theta$ also represents the total end rotation of the column. $IDC$ also corresponds to the total end rotation capacity, $\theta_T$, of the column.

The plastic rotation capacity, $R_p$, is determined using equations 4.2 to 4.8 and $IDC$ as previously defined. Determination of the plastic moment capacity appropriately takes into account the reduction in capacity resulting from axial load. The stiffness used in calculation of the yield displacement includes contributions from both flexural and shear terms. Calculation of plastic rotation capacity will be further discussed in Chapter 6.

When $\frac{P}{P_{ya}} \geq 0.15$  

$$M_{pc} = 1.18 \left( 1 - \frac{P}{P_{ya}} \right) Z_x F_{ya}$$  \hspace{1cm} (4.2)

When $\frac{P}{P_{ya}} < 0.15$  

$$M_{pc} = Z_x F_{ya}$$  \hspace{1cm} (4.3)

$$K_\theta = \frac{K_b K_s}{K_b + K_s}$$  \hspace{1cm} (4.4)

$$K_b = \frac{6EI_x}{H^2}$$  \hspace{1cm} (4.5)

$$K_s = \frac{GA_w}{2}$$  \hspace{1cm} (4.6)

$$\theta_{yc} = \frac{M_{pc}}{K_\theta H}$$  \hspace{1cm} (4.7)
\[
R_p = \frac{\theta_p}{\theta_{yc}} = \frac{\theta_T - \theta_{yc}}{\theta_{yc}}
\]  

(4.8)

where
\begin{align*}
A_w &= \text{web area (}= d_{tw}), \\
E &= \text{modulus of elasticity}, \\
F_{ya} &= \text{actual yield stress}, \\
G &= \text{shear modulus}, \\
I_x &= \text{moment of inertia about the x-axis}, \\
H &= \text{column clear length}, \\
K_\theta &= \text{rotational stiffness}, \\
K_b &= \text{rotational flexural stiffness}, \\
K_s &= \text{rotational shear stiffness}, \\
M_{pc} &= \text{reduced plastic moment}, \\
P &= \text{axial load}, \\
P_{ya} &= \text{actual axial yield strength}, \\
R_p &= \text{plastic rotation capacity}, \\
Z_x &= \text{plastic section modulus about the x-axis}, \\
\theta_p &= \text{plastic rotation}, \\
\theta_T &= \text{total end rotation capacity or IDC}, \text{ and} \\
\theta_{yc} &= \text{yield rotation}. \\
\end{align*}

Figure 4.22 provides a comparison of end moment versus drift response for the compressive axial load target points for the five specimens tested using Loading Scheme C. The reduced moment capacity from increased axial load is evident from Figures 4.22(b) or 4.22(c). It was also observed that increasing the column section weight increased the story drift at which degradation in moment capacity occurred. This was due to the reduced influence of local buckling for the stockier column sections. The values of IDC or $\theta_T$ determined as described above are summarized in Table 4.3. Note that the $IDC$ or $\theta_T$ values are significantly larger than those reported in beam testing (FEMA 2000b). The higher deformation capacities of these tested W14 columns was mainly due to the low slenderness ratios in terms of local buckling (especially web local buckling) and lateral-torsional buckling.
4.6 Summary and Conclusions

Cyclic testing of nine full-scale W14 columns which were representative of those in bottom stories of multistory braced frames was conducted. These columns were subjected to different levels of axial force demand combined with story drift demand. The following conclusions can be made.

1. Subjecting the columns to different levels of axial force demand (35%, 55%, and 75% of nominal axial yield strength), interstory drift capacities \((IDC)\) of approximately 0.07 rad. to 0.09 rad. were observed assuming a 10% reduction from maximum moment resistance was used to define the \(IDC\).

2. Defining the plastic rotation capacity \((R_p)\) as the ratio between the plastic component of the column end rotation and the yield rotation, reduced appropriately to account for the column axial load effect, the \(IDC\) values correspond to an \(R_p\) value ranging from approximately 9 to 19.

3. No global buckling was observed in all test specimens. Flange local buckling was the dominant buckling mode. Web local buckling was not obvious, probably due to the inherent low width-thickness ratios of the tested W14 sections. The stabilizing effect from the stocky web to delay the onset of flange local buckling significantly contributed to the large deformation capacity mentioned earlier.

4. The test data agreed well with the established \(P-M\) interaction surface. An average cyclic overstrength of 1.05 was observed at 4% story drift between the test data and the \(P-M\) interaction surface based on actual material properties.
Table 4.1 Testing Summary

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>Loading Scheme</th>
<th>Max. Drift (rad.)</th>
<th>Notes</th>
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<tbody>
<tr>
<td>W14×132-35</td>
<td>A</td>
<td>0.06</td>
<td>No fracture</td>
</tr>
<tr>
<td>W14×132-55</td>
<td>B</td>
<td>0.08</td>
<td>No fracture</td>
</tr>
<tr>
<td>W14×132-75</td>
<td>C</td>
<td>0.10</td>
<td>Complete flange net section fracture</td>
</tr>
<tr>
<td>W14×176-35</td>
<td>A</td>
<td>0.10</td>
<td>Partial flange net section fracture</td>
</tr>
<tr>
<td>W14×176-55</td>
<td>C</td>
<td>0.10</td>
<td>Complete flange net section fracture</td>
</tr>
<tr>
<td>W14×176-75</td>
<td>C</td>
<td>0.10</td>
<td>Complete flange net section fracture</td>
</tr>
<tr>
<td>W14×233-35</td>
<td>C</td>
<td>0.08</td>
<td>Fracture of flange to base plate welded joint</td>
</tr>
<tr>
<td>W14×233-55</td>
<td>C</td>
<td>0.08</td>
<td>No fracture</td>
</tr>
<tr>
<td>W14×370-35</td>
<td>C</td>
<td>0.02</td>
<td>Fracture of flange to base plate welded joint</td>
</tr>
</tbody>
</table>

Table 4.2 Cyclic Overstrength

<table>
<thead>
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<th>Specimen Designation</th>
<th>$\beta_o$</th>
</tr>
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<tbody>
<tr>
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<td>0.98</td>
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</tr>
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<td>W14×176-55</td>
<td>1.17</td>
</tr>
<tr>
<td>W14×176-75</td>
<td>1.13</td>
</tr>
<tr>
<td>W14×233-35</td>
<td>1.01</td>
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<tr>
<td>W14×233-55</td>
<td>1.07</td>
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Table 4.3 Interstory Drift Capacity

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>$IDC$ or $\theta_T$ (rad.)</th>
<th>$\theta_{yc}$ (rad.)</th>
<th>$R_p$</th>
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<tbody>
<tr>
<td>W14×132-75</td>
<td>0.0725</td>
<td>0.00539</td>
<td>12.5</td>
</tr>
<tr>
<td>W14×176-55</td>
<td>0.0912</td>
<td>0.00627</td>
<td>13.5</td>
</tr>
<tr>
<td>W14×176-75</td>
<td>0.0868</td>
<td>0.00430</td>
<td>19.2</td>
</tr>
<tr>
<td>W14×233-35</td>
<td>0.08</td>
<td>0.00810</td>
<td>8.9</td>
</tr>
<tr>
<td>W14×233-55</td>
<td>&gt;0.08</td>
<td>0.00624</td>
<td>&gt;11.8</td>
</tr>
</tbody>
</table>
Figure 4.1 Typical Column Net-Section Fracture (Specimen W14×132-75)

(a) Overall View of Fracture Location

(b) View of Fracture from Exterior Side of Flange (with Haunches Removed)

(c) View of Fracture from Interior Side of Flange (with Haunches Removed)
(a) Overall View of Fracture Location

(b) View of Fracture from Exterior Side of Flange

(c) View of Fracture from Interior Side of Flange

Figure 4.2 Typical Column to Base Plate Weld Fracture (Specimen W14×233-35)
Figure 4.3 Lateral Displacement Profiles at 2% Drift
Figure 4.4 Specimen W14×132-75: Yielding Pattern
Figure 4.4 Specimen W14×132-75: Yielding Pattern (cont.)
Figure 4.5 Extent of Flange Yielding

(a) W14×132-35 (6% Drift)

(b) W14×132-55 (8% Drift)

(c) W14×132-75 (10% Drift)
Figure 4.5 Extent of Flange Yielding (cont.)
(g) W14×233-35 (8% Drift)

(h) W14×233-55 (8% Drift)

(i) W14×370-35 (2% Drift)

Figure 4.5 Extent of Flange Yielding (cont.)
Figure 4.6 Specimen W14×132-75: Flange Local Buckling (West End)
Figure 4.7 Comparison of Flange Local Buckling at 6% Drift
Figure 4.8 Comparison of Flange Local Buckling at 8% Drift
Figure 4.9 $P-M$ Interaction Curve for Column Specimens
Figure 4.10 Specimen W14×132: Comparison of $P$-$M$ Interaction
Figure 4.11 Specimen W14×176: Comparison of $P$-$M$ Interaction
Figure 4.12 Specimen W14×233: Comparison of P-M Interaction

Figure 4.13 Specimen W14×370-35: P-M Interaction
Figure 4.14 Definition of Cyclic Overstrength Factor

Figure 4.15 Maximum Response Points at 4% Drift
Figure 4.16 Specimen W14×132: Comparison of End Moment versus Drift Response
Figure 4.17 Specimen W14×176: Comparison of End Moment versus Drift Response
Figure 4.18 Specimen W14×233: Comparison of End Moment versus Drift Response

(a) 35% $P_{yn}$

(b) 55% $P_{yn}$

Figure 4.19 Specimen W14×370-35: End Moment versus Drift Response
Figure 4.20 Target Point End Moment versus Drift Response

Figure 4.21 Equivalence between Interstory Drift Angle and Member End Rotation
Figure 4.22 Comparison of Target Points End Moment versus Drift Response
5 FINITE ELEMENT ANALYSIS OF COLUMN MODELS

5.1 Introduction

In order to provide additional data for establishment of steel wide-flange column rotation capacity a finite element analysis parametric study was conducted. Finite element models were initially calibrated using experimental results from the tested W14 column specimens. The effects of residual stress and initial geometric imperfections were investigated and included in the analysis. Additionally, finite element analysis was used to investigate the behavior of W18 and W24 deep column sections, which were beyond the scope of experimental testing. These deep columns have larger web width-to-thickness ratios than the tested W14 sections and thus interaction of flange and web local buckling may result in earlier strength degradation as compared with stockier sections.

5.2 Modeling Techniques

The commercially available program Abaqus (Simulia 2008) was used for finite element analysis. Models predict global behavior, yielding, and strength degradation resulting from local buckling at large drifts. Standard shell elements were used in the models. The general-purpose shell element type used (Abaqus S4R) has four nodes with six degrees of freedom per node, 3 translational and 3 rotational. This element allows for transverse shear deformations, accounts for finite membrane strains, and will allow for changes in thickness, making it suitable for large-strain analysis.
The boundary conditions simulated those used for experimental testing of the steel column specimens. Only the 15 ft clear length of the specimens was modeled. A fixed connection was used at both ends of the column to simulate the restraint provided by the column base plate and the flange and web haunches (see Figure 3.3). Consistent with the test set-up no additional lateral bracing was provided. Figure 5.1 shows the column model boundary conditions and finite element mesh. At the fixed end of the model all six translational and rotational degrees of freedom were constrained to be zero. At the loaded end of the model the out-of-plane translational \( U_1 \) as shown in Figure 5.1(b)] and all three rotational degrees of freedom were constrained to be zero. Lateral drift displacement was applied in the \( U_2 \) direction and axial load was applied in the \( U_3 \) direction. A rigid constraint was imposed on edges at the column ends to prevent stress concentrations at the loading and reaction points.

Models were subjected to one of three loading sequences. The first loading sequence was a monotonic loading consisting of an initial application of the target axial load followed by lateral displacement to the target drift. Separate monotonic loading cases were run for both the tension and compression target axial loads. The second loading sequence was cyclic and consisted of using recorded axial load and lateral displacement from experimental testing as the input loading for simulation of experimental testing. The third loading sequence, which was consistent with the one developed in Chapter 3 for braced frame column testing, began with application of an initial axial gravity load followed by simultaneous, in-phase axial load and lateral displacement. For this loading sequence the axial load and drift targets were achieved simultaneously.
5.3 Material Properties

A992 steel was specified for the test specimen column sections. In the analysis, it was assumed that the yield strength for all the steel material was 54 ksi. An elastic modulus of 29,000 ksi and a Poisson’s ratio of 0.3 were specified for the elastic material properties. The plasticity in the models was based on a von Mises yield surface and associated flow rule. The plastic hardening was defined by a nonlinear kinematic hardening law.

Data from cyclic coupon testing performed at Lehigh University (Kaufmann et al. 2001) was used to determine appropriate values for the plasticity material model parameters (Richards, 2004). Steel C from the Lehigh study was selected as the prototype material. Steel C had a yield strength of 54 ksi and an ultimate strength of 72 ksi under monotonic testing, which was judged to be representative of the A992 steel members used for laboratory specimen fabrication. Figure 5.2 shows results of cyclic coupon testing of the prototype material performed at ±4% strain. The shape of the stabilized curve was used to determine the parameters that define the plasticity model.

5.4 Initial Residual Stress

5.4.1 Typical Residual Stress Distribution

Residual stress exists in a non-loaded wide-flange steel section and primarily results from uneven cooling after hot rolling of the section. This residual stress was inherently included in the tested column specimens. The magnitude and distribution of residual stress for typical sections has been previously established using the method
of sectioning, in which residual stresses are determined by the change in length of thin strips cut from a short length of column (Huber and Beedle 1954, Tebedge et al. 1973, Galambos 1998). Early research on column behavior established that the presence of residual stress decreased the initial yield load of the section (Huber and Beedle 1954, Beedle and Tall 1960, Bjorhovde 1972, Galambos 1998). Nakashima (1994) observed that the ductility of columns was not significantly affected by residual stress. Models W14×132, W14×370, W24×131, and W24×279 were used to investigate the effect of residual stress on the performance of the column specimens subjected to combined high axial load and lateral drift.

5.4.2 Modeling of Residual Stress

A simplified linear residual stress distribution is shown in Figure 5.3(a). This self-equilibrating residual stress distribution was applied to the model as an initial stress condition as shown in the axial stress contour plot in Figure 5.3(b). Experimental evaluation of the magnitude of initial residual stress in as-rolled wide-flange column sections has indicated a maximum residual stress level of \(1/3 F_y\) (Salmon and Johnson 1996). For this study the maximum residual stress was assumed to be 18 ksi \((1/3 \times 55 \text{ ksi})\). A maximum residual stress of 12 ksi \((1/3 \times 36 \text{ ksi})\) was also investigated in the monotonic analysis to enable comparison of different residual stress levels and validate the residual stress modeling procedure.
5.4.3 Effect of Residual Stress on Monotonic and Cyclic Behavior

Figure 5.4 shows the Model W14×132 monotonic end moment versus drift response for 0%, 35%, and 75% $P_{3u}$ and axial load versus axial displacement for the case with no lateral displacement. Residual stress was observed to soften the transition from elastic to plastic behavior, but did not have a significant effect on the monotonic global behavior of this model. The reduction in first-yield load shown in Figure 5.4(d) corresponds to the appropriate 12 ksi and 18 ksi reduction for the respective models. Similar observations were made for Model W24×131 (see Figure 5.5).

Normal stress contours at 1% drift and 0% $P_{3u}$ are shown in Figures 5.6 and 5.7 for Models W14×132 and W24×131. These stress contours are nominally linear without residual stress, but with residual stress included are modified because of the initial compressive stress near the flange tips and tensile stress near the flange-to-web junction. Higher stresses were observed to propagate further from the ends of the column when residual stress was included. This has the potential to result in more significant local buckling because higher compressive stress acts over a longer length of the unsupported edge of the column flange.

Figures 5.8 to 5.15 provide plots of $P$-$M$ interaction and end moment versus drift response for Models W14×132, W14×370, W24×131, and W24×279 at 35% and 75% $P_{3u}$ both with and without residual stress. For the W14 models only minor deviations in behavior were observed and only at very high drift. For the W24 models the decrease in capacity with residual stresses included was more significant, especially for Model W24×131 (larger web width-to-thickness ratio). Residual stress
was included in models used to populate the analysis database due to these observed differences in behavior.

5.5 Initial Geometric Imperfections

5.5.1 Typical Initial Imperfections

Another factor that influences column capacity is initial geometric imperfections (or out-of-straightness). These imperfections result from tolerances in member straightness as produced by the rolling mill and from field erection tolerances. ASTM A6 limits the maximum permissible camber and sweep variation from straight to 1/8 in. per 10 ft (or $L/960$). As shown in Figure 5.16, camber is the out-of-straightness in the strong-axis direction and sweep in the weak-axis direction. This deformation is typically assumed to take the shape of a half-sine wave. Measurements of as-rolled sections have shown that the average out-of-straightness is less than the maximum permissible value and equal to approximately $L/1470$ (Bjorhovde 1972). The AISC Specification column strength equations have been developed using this typical value of $L/1470$. For consistency with previous work this study has also used $L/1470$ as the maximum camber and sweep out-of-straightness.

Column out-of-plumb results from field erection tolerances and can occur in both orthogonal directions as shown in Figure 5.16(c). The AISC Code of Standard Practice for Steel Buildings and Bridges (AISC 2005a) limits the maximum permissible deviation from plumb to $L/500$. This limiting value was used for
analysis, as it was assumed the as-built out-of-plumb would approach the maximum permissible.

To some extent camber, sweep, and out-of-plumb geometric imperfections are expected to occur simultaneously in building columns. While it is not anticipated that all three will occur simultaneously at or close to their maximum assumed values no statistical information is available to quantify the expected geometric imperfections. Therefore, for this study a combined geometric imperfection with camber \((L/1470)\), sweep \((L/1470)\), and out-of-plumb in both the strong- and weak-axis directions \((L/500)\) was considered. This may be conservative, but given the lack of field measurements and the critical function of columns was deemed appropriate.

5.5.2 Modeling of Initial Imperfections

Initial geometric imperfections were included in the models using the Abaqus imperfections feature. An initial analysis was conducted to produce the desired initial deformed shape. The column boundary conditions were modified to release the rotational degrees of freedom at each end of the column and displacements, consistent with those shown in Figure 5.16(d), were applied at the column mid-height and load end. This deformed shape was then used as the initial column model geometry for cyclic analysis. Figure 5.17 shows the initial geometric imperfections for Model W14×132 about the strong-axis (combined camber and out-of-plumb) and weak-axis (combined sweep and out-of-plumb). The magnitudes of initial geometric imperfections were equivalent for all column sections of the same length because the imperfections were only dependent on length.
5.5.3 Effect of Initial Imperfections on Cyclic Behavior

Figures 5.18 to 5.25 provide plots of $P$-$M$ interaction and end moment versus drift response for Models W14×132, W14×370, W24×131, and W24×279 at 35% and 75% $P_{yn}$ both with and without initial geometric imperfections. A moderate reduction in rotation capacity was observed for Model W14×132 when initial geometric imperfections were considered. Only minimal deviation in response is shown for Model W14×370. The effect of initial geometric imperfections was observed to be more significant for the W24 models. This observation is contrary to previous research which reported that geometric imperfections did not significantly affect ductility (Nakashima 1994). However, this previous work did not include deep columns in their study. Initial geometric imperfections were included in models used to populate the analysis database due to these observed differences in behavior.

5.6 Comparison of W14 Column Specimens and Models

Before performing a parametric study to further expand the database of available results, models of the tested column specimens were calibrated using the results of the experimental testing discussed in Chapter 4.

5.6.1 Monotonic Loading Sequence Results

Models of the test specimens were initially subjected to monotonic loading to provide a simple approximation of cyclic behavior before performing a more computationally intensive cyclic analysis. End moment versus drift response (see Figure 5.26) was in general agreement with behavior observed during testing. An
increase in the axial load level caused a corresponding decrease in moment capacity. Also, consistent with observations from the experimental results, Figures 5.26(a) and 5.26(b) show a softening of the moment versus drift response at large negative drifts for Models W14×132 and W14×176. This softening behavior was not observed for Specimens W14×233 and was not observed in the monotonic response of Models W14×233 or W14×370.

5.6.2 Comparison of Specimen and Model Results

Models were subjected to a loading sequence using recorded axial load and lateral displacement from experimental testing as the input loading for analysis. A comparison of test specimen and model P-M interaction is provided in Figures 5.27 to 5.30. The P-M interaction surface shown is based on actual material properties. In general, good agreement was observed between the specimen and model responses. Models did tend to slightly over predict the moment as compared with experimental results. Figures 5.31 to 5.34 compare the experimental and analytically predicted end moment versus drift response. Discrete points shown on these figures represent points where the combined axial load and drift targets were achieved. The predicted end moment versus drift response was observed to degrade faster for Models W14×132 and W14×176 than was observed in experimental testing. The model data shown includes the effect of residual stress and initial geometric imperfections. Analysis results without initial geometric imperfections reasonably predicted the response of the test specimens. This provides an indication that the initial geometric imperfections for the tested specimens were not as severe as those assumed for
analysis. Good agreement was observed between the specimen and model end moment versus drift response for the W14×233 section, even with residual stress and initial imperfections included in the model. This can be attributed to the reduced effect of these initial conditions for the stockier section. Specimen W14×370-35 failed at 3% drift and therefore comparison of specimen and model response at larger drifts was not possible.

Figure 5.35 shows axial stress contour plots and the model deformed shapes at 5% drift (compressive axial load) for Model W14×132. The stress contours indicate that the length of column flange in tension, as a result of bending stress from lateral drift, decreased with increasing axial load. This increased the flange local buckling propensity of the section because a longer flange length was in compression. The figure also shows increasing amplitude of flange local buckling with increasing axial load. Figure 5.36 shows that the amplitude of flange local buckling was observed to decrease as the section weight increased. For Model W14×233 and Model W14×370 only very minor local buckling was observed, even at very large drifts. These observations from analysis results were consistent with those from experimental testing.

5.6.3 Comparison of Results for Two Cyclic Loading Sequences

Figures 5.37 to 5.40 show a comparison of P-M interaction for models loaded with the loading history from test data and the proposed loading history (in-phase axial load and lateral displacement). The P-M interaction surface shown is based on modeled material properties. Both loading sequences were observed to result in very
similar $P-M$ interaction response. The moment versus drift for the two loading sequences is shown in Figures 5.41 to 5.44. At first the results from the two different loading sequences did not appear to be well correlated. However, when the discrete points on the loading history from test plots were compared with the proposed loading history response plots reasonable agreement was observed. The correlation in response between models subjected to the two different cyclic loading histories provided an indication that Loading Scheme C, used for experimental testing, was a reasonable approximation of the more ideal in-phase axial load and lateral displacement loading sequence. The correlation also enabled extrapolation to other axial load levels and other column sections using the confidence gained in the modeling techniques to reasonably predict experimental test results.

5.7 Parametric Study

5.7.1 Introduction

Typically, column sections have a depth that is approximately equal to their flange width. Certain W8, W10, W12, and W14 sections fit into this category (of these the W12 and W14 sections are most commonly used). In moment frames, deep columns are sometimes used to optimize steel weight while meeting code specified drift limits. The web slenderness of a deep column is significantly greater than that of a comparable typical column section for a given moment of inertia about the strong-axis. Interaction of flange and web local buckling has been shown to affect the rotation capacity of moment frame beams (Uang and Fan 2001, Okazaki et al. 2006) and is also assumed to affect column rotation capacity. Figure 5.45 plots the web
width-to-thickness ratio \((h/t_w)\) versus flange width-to-thickness ratio \((b_f/2t_f)\) for W8 to W27 sections that might be used as column sections. The W14 columns tested were in general very stocky sections compared with some of the deeper sections. Figure 5.46(b) shows that the flange slenderness for the W14×132 section was very close to the limiting value of 7.2 (AISC 2005b) but the web slenderness was less than half of the allowable value for a column with an axial load ratio, \(P/P_{y,n}\), equal to 0.75. In order to investigate the performance of columns with web slenderness closer to the AISC Seismic Provisions limits, W18 and W24 deep column sections were modeled and subjected to the loading sequence developed for braced frame columns. Figure 5.46(c) and (d) shows that the web width-to-thickness ratios for the deep column sections were approximately double those of the W12 and W14 column sections and closer to the current seismically compact limit.

5.7.2 Motivation for Including Deep Columns

As previously discussed, braced frames act like a vertically cantilevered truss with the braces controlling the overall lateral drift of the building and the columns carrying predominately axial loads. The high efficiency of BRBs has reduced the area of steel required for the braces in BRBFs and designers have noted an increase in expected story drift as compared with a SCBF designed using the same code based approach. Moment frames on the other hand resist lateral loads by flexural behavior of rigidly connected beams and columns. Designers sometimes use deep columns to increase the lateral stiffness of a moment frame without increasing the overall weight of the structure. For example, the moment of inertia for a W14×370 is 5440 in\(^4\) and
for a W24×176 is 5680 in⁴. The deeper W24×176 section has a similar moment of inertia but is approximately 1/2 the weight of the W14×370 section, resulting in a significant reduction in total steel weight and lower structural cost. One potential strategy to reduce BRBF drift demands is use of a dual braced frame and moment frame system with deep columns. However, previous research has shown significant interaction of flange and web local buckling modes for moment resisting frame beams. The behavior of deep columns under significant flexural and axial demands is uncertain. Therefore, in addition to typical W12 and W14 column sections the behavior of W18 and W24 deep columns under combined axial load and drift demand was investigated as part of this parametric study.

5.7.3 Column Sections Investigated

The matrix of wide-flange section models investigated as part of this parametric study is given in Table 5.1. In addition to the four tested W14 sections, models of W12, W18, and W24 sections (four per nominal column depth) were included in this study. Sections for investigation were selected such that the lightest section had a flange width-to-thickness ratio close to the AISC seismically compact limit and the heaviest section had a flange width-to-thickness ratio of approximately 3. The remaining two sections (per nominal column depth) were selected for uniform distribution of width-to-thickness ratios between the largest and smallest sections.

Column lengths of 12 ft, 15 ft, and 18 ft were modeled such that the column member slenderness ($K_L/r_o$) could be varied independently of change in section size.
Models included the initial residual stresses shown in Figure 5.3 and initial geometric imperfections shown in Figure 5.16.

The first step in the loading sequence was application of a column axial gravity load equal to $0.15P_n$ (where $P_n$ was based on out-of-plane buckling with $K = 1.0$ and $L = 15$ ft). Models were then subjected to the in-phase, increasing amplitude combined axial load and story drift sequence that was developed for braced frame column testing. Target axial loads of 10%, 20%, 35%, 55%, 75%, and 90% $P_{yn}$ were investigated for the W12 and W14 models. For the W18 and W24 models the target axial loads were 20%, 35%, 55%, and 75% $P_{yn}$.

### 5.7.4 Finite Element Analysis Results

Tables 5.2 to 5.5 provide yield rotation, interstory drift capacity, and plastic rotation capacity for the W12, W14, W18, and W24 models respectively. Calculation of these quantities was briefly described in Chapter 4 and will be discussed in greater detail in Chapter 6, where this data will be utilized for development of rotation capacity predictive models.

A comparison of the parametric study analysis results is provided in Section 5.8. Appendix C provides additional plots of $P-M$ interaction, end moment versus drift, and constant-axial-load end moment versus drift response. For select models the local buckling deformed shape at 5% drift (compressive axial load) is also provided.
5.8 Comparison of Finite Element Model Results

5.8.1 Effect of Axial Load and Section Weight

Figure 5.47 shows end moment versus drift and constant-axial-load end moment versus drift response for Model W14×176 \((L = 15 \text{ ft})\) at 20%, 35%, 55%, and 75% \(P_{yn}\). Constant-axial-load end moment versus drift envelopes are based on the combined axial load and drift target points on the compression side; that is, these represent the end moment versus story drift relation under constant axial load (equal to the target axial load). Examination of this figure reveals that for a given column section: (1) \(IDC\) decreased with increasing axial load, and (2) moment capacity decreased with increasing axial load.

Similar plots of end moment versus drift and constant-axial-load end moment versus drift are shown in Figure 5.48 for the W14 series models at a target axial load of 35% \(P_{yn}\) and 15 ft column length. An increase in maximum moment is observed that is consistent with the increasing section size. Also, an increase in \(IDC\) is observed corresponding to the increase in section weight. As a section becomes stockier the onset of local buckling (and associated strength degradation) is delayed, thus increasing the \(IDC\) of the section. Comparison of model deformed shapes was in agreement with this observation. For a given nominal column depth (i.e., W14), the amplitude of flange and web local buckling increased with increasing axial load and also increased with decreasing section weights.
5.8.2 Deep Column Effect

Comparison of analysis results for the typical column sections (W12 and W14) and deep column sections (W18 and W24) revealed more significant flange and web local buckling for the deep column sections. This flange and web local buckling and interaction of buckling modes caused more rapid strength degradation than observed for the stocky W12 and W14 sections, resulting in a decreased IDC.

Figure 5.49 shows the local buckling deformation of Models W12×87-35, W12×230-35, W18×86-35, and W18×234-35 (L = 15 ft) at 6% drift (compressive axial load). The flange width-to-thickness ratios for the W12×87 and W18×86 sections were similar, as were the flange width-to-thickness ratios for the W12×230 and W18×234 sections. A comparison of the deformed shape of the two W12 models or the two W18 models clearly shows a decrease in the amplitude of both flange local buckling (FLB) and web local buckling (WLB) for the heavier section (i.e., the amplitude of FLB and WLB for W12×230-35 was less than W12×87-35). The effect of local buckling on degradation of end moment versus drift response is shown in Figure 5.50. Models W12×87-35, and W18×86-35 predict earlier and more rapid degradation in end moment versus drift response than the corresponding heavier column sections. The reliability of deep columns under combined high axial load and drift demand should be the subject of future experimental verification.

5.9 Summary and Conclusions

A finite element analysis parametric study of W12, W14, W18 and W24 column models has been conducted. These column models were subjected to
different levels of axial force demand combined with story drift demand. Models of the W14 test specimens were shown to reasonably predict experimental results. The following conclusions can be made.

1. Residual stress did not significantly affect the cyclic behavior of the W14 models. Minor reduction in capacity was observed for deep W24 column models, especially the lighter sections with larger width-to-thickness ratios. Residual stress was included as an initial stress condition in parametric study models.

2. The modeled camber, sweep, and out-of-plumb initial geometric imperfections were shown to potentially reduce plastic rotation capacity, especially for the deep column models. This observation was contrary to previous research which reported that geometric imperfections did not significantly affect ductility (Nakashima 1994). However, this previous work did not include deep columns in their study. Initial geometric imperfections were included in parametric study models.

3. For a given nominal column depth (i.e., W14) an increase in IDC was observed for increasing section weight. For stockier sections the onset of local buckling (and associated strength degradation) was delayed, thus increasing the IDC of the section.

4. The IDC of deep (W18 and W24) columns was reduced compared to the stockier typical column sections (W12 and W14). More significant flange and web local buckling and interaction of these local buckling modes was observed for the deep column models. The stocky web of typical column sections provides more restraint to the flange than that provided by the web of a deep column with a
similar flange width-to-thickness ratio, thus reducing the amplitude of local buckling.
Table 5.1 Model Matrix

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<th>Section</th>
<th>Width-to-Thickness Ratio</th>
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Table 5.2 W12 Series Models: Rotation Capacity

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<th>L=18 ft</th>
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<td>IDC (rad.)</td>
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Table 5.3 W14 Series Models: Rotation Capacity

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### Table 5.4 W18 Series Models: Rotation Capacity

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### Table 5.5 W24 Series Models: Rotation Capacity

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Figure 5.1 Model Boundary Conditions and Geometry

(a) Specimen Plan View

Loaded End:

\[ U_1 = 0 \]
\[ U_2 \neq 0 \]
\[ U_3 \neq 0 \]
\[ R_1 = R_2 = R_3 = 0 \]

Fixed End:

\[ U_1 = U_2 = U_3 = 0 \]
\[ R_1 = R_2 = R_3 = 0 \]
Figure 5.2 Cyclic Material Test Results (Kaufmann et al. 2001)
Figure 5.3 Residual Stress Distribution

(a) Assumed Residual Stress

(b) Normal Stress Contours
Figure 5.4 Model W14×132: Effect of Initial Residual Stress Distribution on Global Behavior
Figure 5.4 Model W14×132: Effect of Initial Residual Stress Distribution on Global Behavior (cont.)

(c) End Moment versus Drift (75% $P_{ya}$)

(d) Axial Load versus Axial Displacement (Drift = 0)
Figure 5.5 Model W24×131: Effect of Initial Residual Stress Distribution on Global Behavior
Figure 5.5 Model W24×131: Effect of Initial Residual Stress Distribution on Global Behavior (cont.)
Figure 5.6 Model W14×132: Effect of Initial Residual Stress Distribution on Normal Stress Contours (1% Drift, 0% $P_{yn}$)
Figure 5.7 Model W24×131: Effect of Initial Residual Stress Distribution on Normal Stress Contours (1% Drift, 0% $P_{yn}$)
Figure 5.8 Model W14×132-35: Response with and without Residual Stress
Figure 5.9 Model W14×132-75: Response with and without Residual Stress

(a) $P-M$ Interaction

(b) End Moment versus Drift Response
Figure 5.10 Model W14×370-35: Response with and without Residual Stress

(a) $P-M$ Interaction

(b) End Moment versus Drift Response
Figure 5.11 Model W14×370-75: Response with and without Residual Stress

(a) $P-M$ Interaction

(b) End Moment versus Drift Response
Figure 5.12 Model W24×131-35: Response with and without Residual Stress

(a) $P$-$M$ Interaction

(b) End Moment versus Drift Response
Figure 5.13 Model W24×131-75: Response with and without Residual Stress

(a) $P-M$ Interaction

(b) End Moment versus Drift Response
Figure 5.14 Model W24×279-35: Response with and without Residual Stress
Figure 5.15 Model W24×279-75: Response with and without Residual Stress

(a) \( P-M \) Interaction

(b) End Moment versus Drift Response
Figure 5.16 Initial Geometric Imperfections

(a) Camber

(b) Sweep

(c) Out-of-Plumb

(d) Combined
Figure 5.17 Model W14×132: Initial Geometric Imperfections (Amplified ×10)

(a) Strong-Axis

(b) Weak-Axis
Figure 5.18 W14×132-35: Effect of Combined Geometric Imperfections
Figure 5.19 W14×132-75: Effect of Combined Geometric Imperfections
Figure 5.20 W14×370-35: Effect of Combined Geometric Imperfections
Figure 5.21 W14×370-75: Effect of Combined Geometric Imperfections

(a) P-M Interaction

(b) End Moment versus Drift Response
Figure 5.22 W24×131-35: Effect of Combined Geometric Imperfections
Figure 5.23 W24×131-75: Effect of Combined Geometric Imperfections
Figure 5.24 W24×279-35: Effect of Combined Geometric Imperfections
Figure 5.25 W24×279-75: Effect of Combined Geometric Imperfections

(a) $P$-$M$ Interaction

(b) End Moment versus Drift Response
Figure 5.26 Comparison of W14 Column Models Monotonic End Moment versus Drift Response
Figure 5.27 Model W14×132: Comparison of Specimen and Model P-M Interaction
Figure 5.28 Model W14×176: Comparison of Specimen and Model $P$-$M$ Interaction
Figure 5.29 Model W14×233: Comparison of Specimen and Model P-M Interaction

(a) 35% \( P_{yn} \)

(b) 55% \( P_{yn} \)
Figure 5.30 Model W14×370-35: Comparison of Specimen and Model $P-M$ Interaction
Figure 5.31 Model W14×132: Comparison of Specimen and Model End Moment versus Drift Response
Figure 5.32 Model W14×176: Comparison of Specimen and Model End Moment versus Drift Response

(a) 35% $P_{yn}$

(b) 55% $P_{yn}$

(c) 75% $P_{yn}$
Figure 5.33 Model W14×233: Comparison of Specimen and Model End Moment versus Drift Response

(a) 35% $P_yn$

(b) 55% $P_yn$
Figure 5.34 Model W14×370-35: Comparison of Specimen and Model End Moment versus Drift Response
Figure 5.35 Model W14×132: Local Buckling and Normal Stress Contours at 5% Drift
Figure 5.36 Comparison of Local Buckling at 5% Drift (Amplified ×2)
Figure 5.37 Model W14×132: Comparison of P-M Interaction

(a) 35% $P_{yn}$

(b) 55% $P_{yn}$

(c) 75% $P_{yn}$
Figure 5.38 Model W14×176: Comparison of $P-M$ Interaction
Loading History from Test

![Graph](image1)

Proposed Loading History

![Graph](image2)

(a) 35% $P_{yn}$

![Graph](image3)

(b) 55% $P_{yn}$

![Graph](image4)

(c) 75% $P_{yn}$

Figure 5.39 Model W14×233: Comparison of $P$-$M$ Interaction
Figure 5.40 Model W14×370: Comparison of P-M Interaction
Figure 5.41 Model W14×132: Comparison of End Moment versus Drift Response
Figure 5.42 Model W14×176: Comparison of End Moment versus Drift Response
Figure 5.43 Model W14×233: Comparison of End Moment versus Drift Response
Loading History from Test

Proposed Loading History

(a) 35% $P_{yn}$

(b) 55% $P_{yn}$

(c) 75% $P_{yn}$

Figure 5.44 Model W14×370: Comparison of End Moment versus Drift Response
Figure 5.45 Comparison of Width-to-Thickness Ratios for W8 to W27 Sections

(a) W8×31 to W8×67

(b) W10×49 to W10×112

(c) W12×66 to W12×336

(d) W14×90 to W14×730

(e) W18×76 to W18×311

(f) W21×101 to W21×201

(g) W24×104 to W24×370

(h) W27×146 to W27×539
Figure 5.46 Comparison of Width-to-Thickness Ratios with $\lambda_{ps}$ for Wide-Flange Sections Models
Figure 5.47 Model W14×176: Comparison of End Moment versus Drift Envelopes

(a) 20% $P_{y,n}$

(b) 35% $P_{y,n}$

(c) 55% $P_{y,n}$

(d) 75% $P_{y,n}$
Figure 5.48 Model Comparison of End Moment versus Drift Envelopes for Increasing Section Weight (35% $P_{yn}$)
Figure 5.49 Comparison of Local Buckling Deformation at 6% Drift
Figure 5.50 Comparison of End Moment versus Drift Envelopes

(a) Model W12×87-35

(b) Model W12×230-35

(c) Model W18×86-35

(d) Model W18×234-35
6 LIMITS ON ALLOWABLE CAPACITY

6.1 Introduction

Columns are generally designed to remain elastic under loads resulting from the design earthquake ground motion. However, when nonlinear time-history analysis has been used to evaluate new designs and existing buildings, combined high column axial load and inelastic rotation demand, resulting from story drift, has been observed. Experimental testing of full-scale W14 columns and a finite element analysis parametric study has evaluated deformation capacity of column members with varying levels of axial load and rotation demand. This chapter will develop column deformation capacity predictive models and discuss other design criteria appropriate for application to the evaluation of columns in new and existing buildings.

6.2 ASCE 41 Criteria

Little guidance is available in the literature concerning the cyclic plastic rotation capacity (ratio of the inelastic rotation attained to the idealized elastic rotation at first yield) of steel columns subjected to combined axial and flexural loading. Sections 5.4.2.4.2 and 5.4.2.4.3 of ASCE 41 (ASCE 2007) do provide plastic rotation capacities for moment frame columns subjected to combined loading, which have been used by designers for braced frame columns in the absence of other guidelines. These acceptance criteria for nonlinear analysis procedures are summarized in Table 6.1. According to ASCE 41, columns with a required axial strength \( P_r \) to available axial strength \( P_c \) ratio of 0.5 or less are considered to be deformation-controlled elements with a plastic rotation capacity dependent on axial load ratio, flange and web...
width-to-thickness ratios, and performance objective. Columns with an axial load ratio greater than 0.5 are considered to be force-controlled members for combined axial and flexural loading and shall satisfy Eq. 6.1.

\[
\frac{P_r}{P_c} + \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \leq 1.0
\]  

(6.1)

where

- \(M_{cx}\) = available flexural strength about x-axis,
- \(M_{cy}\) = available flexural strength about y-axis,
- \(M_{rx}\) = required flexural strength about x-axis,
- \(M_{ry}\) = required flexural strength about y-axis,
- \(P_c\) = available axial strength, and
- \(P_r\) = required axial strength.

The ASCE 41 criteria, therefore, assume that columns loaded above one-half their available axial strength possess no plastic rotation capacity.

Figure 6.1 shows the ASCE 41 criteria for the collapse prevention limit state and flange and web width-to-thickness ratios satisfying the AISC *Seismic Provisions* requirements, along with experimental data points. Plastic rotation capacity, \(R_p\) for the experimental data was calculated per ASCE 41 using Eqs. 6.2 and 6.3.

\[
R_p = \frac{\theta_p}{\theta_{yc}} = \frac{\theta_T - \theta_{yc}}{\theta_{yc}}
\]

(6.2)

\[
\theta_{yc} = \frac{Z_x F_{ya} H}{6EI_x} \left(1 - \frac{P}{P_{ya}} \right)
\]

(6.3)

where

- \(E\) = modulus of elasticity,
- \(F_{ya}\) = actual yield stress,
- \(I_x\) = moment of inertia about x-axis,
- \(H\) = column clear length,
- \(P\) = axial load,
- \(P_{ya}\) = actual axial yield strength,
- \(R_p\) = plastic rotation capacity,
- \(Z_x\) = plastic section modulus about x-axis,
\[ \theta_p = \text{plastic rotation}, \]
\[ \theta_T = \text{total end rotation capacity or } IDC, \text{ and} \]
\[ \theta_{yc} = \text{yield rotation}. \]

Figure 6.2 shows that \( \theta_T \) is equal to the total rotation corresponding to when \( M \) has reduced to \( 0.9M_{\text{max}} \) (the interstory drift capacity). For the experimental data points the yield rotation, \( \theta_{yc} \) was calculated based on measured material properties, which was consistent with the ASCE 41 approach of using expected strengths (\( F_{ye} \) and \( P_{ye} \)). The expected strength for different steel grades is based on the mean value of actual yield stress determined from mechanical testing. This yield stress is more representative of expected performance than the nominal design values. For A992 steel wide-flange sections \( F_{yn} = 50 \text{ ksi} \) and \( F_{ye} = 55 \text{ ksi} \).

Significant plastic rotation capacity was observed for the tested W14 column sections (see Figure 6.1), even for axial load ratios greater than 0.5. Comparison of the ASCE 41 acceptance criteria with test data indicated that the ASCE 41 criteria and assumption that columns with an axial load ratio above 0.5 possess no plastic rotation capacity are overly conservative for the tested W14 column sections. Figure 6.3 shows a comparison of the ASCE 41 predicted \( R_p \) along with the finite element analysis database \( R_p \) values. Consistent with test data, significant plastic rotation capacity was observed for axial load ratios greater than 0.5. However, for axial load ratios less than 0.5 the analysis results included a significant number of data points with plastic rotation capacities less than that predicted by the current ASCE 41 criteria. Clearly the current approach does not adequately predict the range of rotation capacities expected based on the results of experimental testing and finite element analysis.
6.3 Determination of Plastic Rotation Capacity

6.3.1 Interstory Drift Capacity

As was described in Chapter 4 the interstory drift capacity, IDC was defined as that when the peak end moment, $M_{max}$ has degraded by 10%. This 10% reduction from the peak moment resistance may be conservative, but given the critical function of columns to the overall stability of a structure, was deemed appropriate at this time. (For comparison, SMRF beam-to-column moment connections require the measured flexural resistance of the connection to be at least $0.80M_p$, or an allowable 20% reduction from $M_p$.) IDC also corresponds to the total end rotation capacity, $\theta_T$ of the column.

6.3.2 Reduced Plastic Moment

Determination of the yield rotation, $\theta_{yc}$ required a definition of reduced plastic moment, $M_{pcs}$, that considered the reduction in moment capacity resulting from axial load. Figure 6.4 show three alternative $P$-$M$ interaction definitions. Eq. 6.5 describes the $P$-$M$ relation developed for plastic design at the section level or for short columns (ASCE 1971). This equation is also included in ASCE 41 for calculation of expected column flexural strength, but ASCE 41 uses Eq. 6.6 to calculate $M_{pc}$ for determination of $\theta_{yc}$. Eq. 6.7 is the AISC Specification $P$-$M$ relation at the section level. This equation is almost identical to the exact analytical solution for a W8×31 subjected to strong-axis bending (AISC 2005c).
When \( \frac{P}{P_{ya}} \geq 0.15 \),
\[
M_{pc} = 1.18 \left(1 - \frac{P}{P_{ya}}\right) Z_x F_{ya} \tag{6.5a}
\]
When \( \frac{P}{P_{ya}} < 0.15 \),
\[
M_{pc} = Z_x F_{ya} \tag{6.5b}
\]
\[
M_{pc} = \left(1 - \frac{P}{P_{ya}}\right) Z_x F_{ya} \tag{6.6}
\]
When \( \frac{P}{P_{ya}} \geq 0.20 \),
\[
M_{pc} = \frac{9}{8} \left(1 - \frac{P}{P_{ya}}\right) Z_x F_{ya} \tag{6.7a}
\]
When \( \frac{P}{P_{ya}} < 0.20 \),
\[
M_{pc} = \left(1 - \frac{P}{2P_{ya}}\right) Z_x F_{ya} \tag{6.7b}
\]

where

- \( F_{ya} \) = actual yield stress,
- \( M_{pc} \) = reduced plastic moment,
- \( P \) = axial load,
- \( P_{ya} \) = actual axial yield strength, and
- \( Z_x \) = plastic section modulus about x-axis.

For a given axial load, Eq. 6.6 tends to under-predict moment capacity as compared with Eqs. 6.5 and 6.7. An underestimate of \( M_{pc} \) may result in an underestimate of \( \theta_{yc} \) and a corresponding overestimate of \( R_p \). Therefore, the current ASCE 41 approach may provide a non-conservative prediction of \( \theta_{yc} \) and \( R_p \). Eqs. 6.5 and 6.7 provide a more realistic approximation of \( P-M \) interaction behavior. For consistency with equations already included in ASCE 41 this study has selected Eq. 6.5 to use for calculation of \( M_{pc} \). Use of this equation (as opposed to Eq. 6.6) is more consistent with expected behavior and reduces the potential for a non-conservative prediction of \( R_p \).
6.3.3 Yield Rotation

ASCE 41 currently uses Eq. 6.3 to calculate column yield rotation. This equation does not include additional deflection due to shear deformation or second-order effects. The $P$-$\Delta$ effect, discussed in Chapter 2, results in additional column bending moment and change in member stiffness due to the axial force acting on the column laterally displaced shape. Eq. 6.8 includes an amplification factor (braced term) to account for the change in member stiffness resulting from axial load (Weaver and Gere 1990).

$$\theta_{yc} = \frac{Z_x F_{ya} H}{6EI_x} \left( 1 - \frac{P}{P_{ya}} \right) \left[ 6\left[ 2 - 2\cos(\kappa H) - \kappa H \sin(\kappa H) \right] \right] \left( \kappa H \right)^2 \left[ 1 - \cos(\kappa H) \right]$$  \hspace{1cm} (6.8)

where

- $E$ = modulus of elasticity,
- $F_{ya}$ = actual yield stress,
- $H$ = column clear length,
- $I_x$ = moment of inertia about x-axis,
- $P$ = axial load,
- $P_{ya}$ = actual axial yield strength,
- $Z_x$ = plastic section modulus about x-axis,
- $\kappa$ = stiffness factor ($\sqrt{P/EI_x}$), and
- $\theta_{yc}$ = yield rotation.

Figure 6.5 plots the $\theta_{yc}$ amplification factor versus $\kappa H$ for an Euler-Bernouli beam. For $\kappa H$ less than 1.6 (maximum from model data) the amplification of $\theta_{yc}$ due to $P$-$\Delta$ effects was less than 5%. A comparison of initial rotational stiffness with increasing axial load for Model W14×132 is shown in Figure 6.6. The initial rotational stiffness decreased by only approximately 1.5% with an increase in axial load from 0% to 90% $P_{yn}$. It was deemed that for design purposes this amplification of $\theta_{yc}$ was not
significant enough to warrant the increased computational cost required to include this minor amplification.

The rotational stiffness used by ASCE 41 to calculate $\theta_{yc}$ (Eq. 6.9) is based on Euler-Bernoulli beam theory for double-curvature bending and neglects contributions from shear deformation. Figure 6.7 plots $\theta_{yc}$ calculated with Timoshenko beam theory (bending and shear deformation) versus $\theta_{yc}$ calculated for an Euler-Bernoulli beam (bending deformation only). Including shear deformation resulted in $\theta_{yc}$ values that were approximately 10% to 50% larger than those calculated considering bending deformation only. Due to the significant contribution of shear deformation to overall deformation it is critical to include the shear component in calculation of $\theta_{yc}$. For this study the stiffness used to compute $\theta_{yc}$ for double-curvature bending was calculated using Eqs. 6.9 to 6.11 and included contributions from both flexural and shear terms.

$$K_b = \frac{6EI_x}{H^2}$$  \hspace{1cm} (6.9)

$$K_s = \frac{GA_w}{2}$$  \hspace{1cm} (6.10)

$$K_\theta = \frac{K_bK_s}{K_b + K_s}$$  \hspace{1cm} (6.11)

where

$A_w$ = web area ($= d_t w$),
$E$ = modulus of elasticity,
$G$ = shear modulus,
$I_x$ = moment of inertia about the x-axis,
$H$ = column clear length,
$K_\theta$ = rotational stiffness,
$K_b$ = rotational flexural stiffness, and
$K_s$ = rotational shear stiffness.
Eqs. 6.9 to 6.11 are consistent with equations already included in ASCE 41 for calculation of eccentrically braced frame link beam stiffness.

### 6.3.4 Plastic Rotation Capacity

The plastic rotation capacity, $R_p$, was determined using Eqs. 6.2, 6.5, and 6.9 to 6.12 and the IDC (drift when $M$ reduced to 0.9$M_{max}$).

\[
\theta_{yc} = \frac{M_{pc}}{K_0H} \quad (6.12)
\]

\[
R_p = \frac{\theta_p}{\theta_{yc}} = \frac{\theta_T - \theta_{yc}}{\theta_{yc}} \quad (6.2)
\]

This definition of $R_p$ improves upon that of ASCE 41 by calculating $M_{pc}$ based on a more realistic $P$-$M$ interaction model and including the significant effect of shear deformation in the calculation of $\theta_{yc}$.

### 6.4 Comparison of Interstory Drift and Plastic Rotation Capacities

The IDC versus axial load ratio and $R_p$ versus axial load ratio data points for the W14 test specimens are shown in Figure 6.8. No clear trend in IDC versus axial load response was observed. An examination of Figure 6.8(b) indicates that for a given column section (i.e., W14×176 or W14×233) $R_p$ increases with increasing axial load. This observation is somewhat counterintuitive, however, it can be simply explained. The yield rotation of a column decreases with increasing axial load based on Eqs. 6.5, and 6.9 to 6.12. $R_p$ is equal to the plastic component of column end rotation divided by $\theta_{yc}$ (Eq. 6.2). For the experimental data points shown in Figure
6.8, $\theta_{yc}$ decreased at a faster rate than IDC decreased with increasing axial load. Therefore, the net effect is an increase in $R_p$ with increasing axial load.

Figures 6.9 to 6.13 provide plots of IDC versus axial load ratio and $R_p$ versus axial load ratio for Models W12, W14, W18, W24, and the combined results, respectively. As expected, for a given column section IDC decreased with increasing axial load. $R_p$ increased with increasing axial load, as discussed above. This effect was less pronounced for the deep column models.

For the column models investigated, IDC ranged from 0.01 rad. to 0.135 rad. and $R_p$ from approximately 1.6 to 48. Nakashima (1994) investigated the variation of ductility capacity of steel beam-columns using a database of 224 small-scale, H-shaped beam-column specimens with axial load ratios up to 0.65. Using a value of $0.95M_{max}$ to define the IDC and calculating $R_p$ in a manner similar to ASCE 41, values of $R_p$ from approximately 1 to 20 were observed. Using an alternate definition of IDC equal to $0.8M_{max}$, $R_p$ values up to approximately 50 were observed. These rotation capacities were in a similar range to the ones observed in this experimental and analytical program.

### 6.5 Development of Plastic Rotation and Interstory Drift Capacity Models

#### 6.5.1 Slenderness Parameter Interaction

Previous analytical research and observations from experimental testing have shown that the rotation capacity of steel wide-flange members is influenced by the interaction of flange local buckling, web local buckling and lateral-torsional buckling. The finite element analysis results for deep column sections, described in Chapter 5,
clearly illustrated this interaction of flange and web local buckling modes. Kemp (1996) developed an effective slenderness ratio considering flange, web, and lateral slenderness and observed that the lateral slenderness had a dominant influence on rotation capacity.

Uang and Fan (2000) investigated cyclic stability criteria for reduced beam section (RBS) steel moment connections. A nonlinear model considering the interaction of flange, web, and lateral slenderness parameters was developed based on the experimental results of 55 full-scale RBS moment connection test specimens. Regression analysis indicated a strong influence of web local buckling and weak influence of lateral-torsional buckling on plastic rotation capacity.

Since this previous work was focused on rotation capacity of beams, an axial load parameter was not included in the slenderness parameter interaction models. To develop a rotation capacity model for columns with axial load and rotation demand the important dependence on axial load ratio must be considered. Uang and Fan’s model was modified to include axial load dependence and model parameters determined using regression analysis. The database used to determine model parameters was from the results of the finite elemental analysis described in Chapter 5. Modeled axial load ratios were representative of those expected for braced frame columns. The histograms of flange, web, and lateral slenderness, shown in Figure 6.14, indicate that models represented a practical range of slenderness parameters.
6.5.2 Nonlinear Plastic Rotation Capacity Model

Eq. 6.13 represents a nonlinear model including the key parameters (independent variables) that affect plastic rotation capacity. A model of similar form to Eq 6.13 has previously been shown to reasonably predict rotation capacity of steel moment frame beams (Uang and Fan 2000). Multiplicative models of this form are common in science and engineering applications (Freund et al. 2006). Eq. 6.13 can be linearized by taking the logarithm of both sides of the equation as given by Eq. 6.14. The constant $C$ and exponents $\alpha$, $\beta$, $\gamma$, and $\delta$ were determined from regression of this linearized model.

$$R_p = C \left( \frac{b_f}{2t_f} \right)^\alpha \left( \frac{h}{t_w} \right)^\beta \left( \frac{L_b}{r_y} \right)^\gamma \left( \frac{P}{P_{ya}} \right)^\delta$$  \hspace{1cm} (6.13)

$$\log(R_p) = \log(C) + \alpha \log \left( \frac{b_f}{2t_f} \right) + \beta \log \left( \frac{h}{t_w} \right) + \gamma \log \left( \frac{L_b}{r_y} \right) + \delta \log \left( \frac{P}{P_{ya}} \right)$$  \hspace{1cm} (6.14)

where

- $C$ = constant to be determined from regression,
- $L_b$ = unbraced length,
- $P$ = axial load,
- $P_{ya}$ = actual axial yield strength,
- $R_p$ = plastic rotation capacity,
- $b_f$ = flange width,
- $h$ = clear distance between flanges less the fillet or corner radius for rolled shapes,
- $r_y$ = radius of gyration about y-axis,
- $t_f$ = flange thickness,
- $t_w$ = web thickness,
- $\alpha$ = exponent to be determined from regression,
- $\beta$ = exponent to be determined from regression,
- $\gamma$ = exponent to be determined from regression, and
- $\delta$ = exponent to be determined from regression.

For a properly defined model the response variable ($R_p$) is a function of one or more independent variables ($b_f/2t_f$, $h/t_w$, $L_b/r_y$, and $P/P_{ya}$). Figure 6.15(a) shows that
for Models W12 and W14 the parameters $b_f/2t_f$ and $h/t_w$ are linearly dependent and, therefore, one of the terms in the regression model was redundant. For Models W12 and W14 the $h/t_w$ term, was removed from the nonlinear model and regression of Eq. 6.15a was performed. The $b_f/2t_f$ term was retained because it was observed to be the

\[
R_p = C \left( \frac{b_f}{2t_f} \right)^{\alpha} \left( \frac{L_b}{r_y} \right)^{\beta} \left( \frac{P}{P_{ya}} \right)^{\delta} \tag{6.15a}
\]

\[
R_p = C \left( \frac{h}{t_w} \right)^{\alpha} \left( \frac{L_b}{r_y} \right)^{\beta} \left( \frac{P}{P_{ya}} \right)^{\delta} \tag{6.15b}
\]

Models W18 and W24 showed a similar linear dependence between $b_f/2t_f$ and $h/t_w$ [see Figure 6.15(b)]; however, Eq. 6.15b provided a better prediction of rotation capacity of these deep columns. Figure 6.15(c) shows no linear relationship between $b_f/2t_f$ and $h/t_w$ when considering Models W12, W14, W18, and W24. Eq. 6.13 was used to model the rotation capacity when the combined data set was analyzed.

Two options were considered to model the axial load parameter: axial load normalized by actual axial compressive strength ($P/P_{na}$) or axial load normalized by actual axial yield strength ($P/P_{ya}$). Neither axial load parameter showed any significant improvement in model prediction capabilities over the other. The term $P/P_{ya}$ was selected to model axial load dependence because it is computationally simpler.
6.5.3 Regression Analysis of Plastic Rotation Capacity

Regression analysis of plastic rotation capacity for a data set consisting of W12 and W14 column model analysis results for axial loads of 10%, 20%, 35%, 55%, 75%, and 90% $P_{yn}$ was conducted to determine the unknown constant and exponents. A least-squares regression resulted in Eq. 6.16 with a coefficient of determination, $r^2$, equal to 0.83. (An $r^2$ of 1 indicates a perfect correlation in actual and predicted response.)

$$R_p = 3034 \left( \frac{b_f}{2t_f} \right)^{-1.34} \left( \frac{L_b}{r_y} \right)^{-0.84} \left( \frac{P}{P_{ya}} \right)^{0.42}$$

(6.16)

The magnitude of the exponent for $b_f/2t_f$ is larger than that for $L_b/r_y$, indicating that the effect of local buckling is somewhat more significant than global column slenderness.

Figure 6.16(a) shows a plot of predicted $R_p$ versus actual $R_p$ (from finite element analysis). The dashed one-to-one line would represent a perfect correlation. Some significant deviations in predicted versus actual response were observed at higher levels of $R_p$.

A regression analysis was also performed on a data set consisting of Model W12 and W14 analysis results for axial loads of 20%, 35%, 55%, and 75% $P_{yn}$ resulting in Eq. 6.17 with an $r^2$ equal to 0.94.

$$R_p = 2760 \left( \frac{b_f}{2t_f} \right)^{-1.41} \left( \frac{L_b}{r_y} \right)^{-0.80} \left( \frac{P}{P_{ya}} \right)^{0.53}$$

(6.17)

Eliminating 10% and 90% $P_{yn}$ from the considered axial load for determination of regression coefficients is justified as follows. Axial loads of only 10% $P_{yn}$ do not
make efficient use of column member capacity and are not expected for braced frame columns. For many of the W12 and W14 column models, an axial load equal to 90% $P_{yn}$ exceeded the design compressive strength of the member. Therefore, 90% $P_{yn}$ is also an unrealistic axial load level, since for design the column axial load demand would not be allowed to exceed the compression capacity of the column. Figure 6.16(b) shows excellent correlation of predicted and actual $R_p$, especially below 10. This range is of practical interest because sections satisfying the AISC Seismic Provision seismically compact width-to-thickness ratio limits are expected to possess plastic rotation capacities of 5 or 6.

A separate regression analysis was performed for the W18 and W24 column model data set for axial loads of 20%, 35%, 55%, and 75% $P_{yn}$. Predicted $R_p$ is given by Eq. 6.18 and resulted in an $r^2$ of 0.91.

$$R_p = 50759 \left( \frac{h}{t_w} \right)^{-1.76} \left( \frac{L_b}{r_y} \right)^{-0.80} \left( \frac{P}{P_{ya}} \right)^{0.30} \quad (6.18)$$

The magnitude of the exponent for $h/t_w$ is larger than that for $L_b/r_y$ indicating that the effect of local buckling is more dominant than global column slenderness. Figure 6.17 plots the predicted $R_p$ versus actual $R_p$. Again, excellent correlation was observed for $R_p$ below 10.

A regression analysis considering all column models (W12, W14, W18, and W24) for axial loads of 20%, 35%, 55%, and 75% $P_{yn}$ resulted in Eq. 6.19 with an $r^2$ equal to 0.92.
\[ R_p = 2607 \left( \frac{b_f}{2t_f} \right)^{-1.10} \left( \frac{h}{t_w} \right)^{-0.45} \left( \frac{L_b}{r_y} \right)^{-0.66} \left( \frac{P}{P_{ya}} \right)^{0.41} \]  

(6.19)

The exponent for \( b_f/2t_f \) is more than twice the exponent for \( h/t_w \) indicating that the influence of flange local buckling is dominant over the influence from web local buckling. This may be affected by the linear relation between \( b_f/2t_f \) and \( h/t_w \) for the typical and deep column models. Predicted \( R_p \) versus actual \( R_p \) is shown in Figure 6.18. As expected for this data set with both typical and deep column sections somewhat greater dispersion was observed in the data.

### 6.5.4 Proposed Plastic Rotation Capacity Model

Due to the differences in behavior of deep column sections as compared with the typical column sections (earlier and more significant local buckling and decreased rotation capacity) separate plastic rotation capacity models are proposed for the two section types. Eqs. 6.20 and 6.21 provide reasonable predictions of plastic rotation capacity for typical and deep steel wide-flange or built-up columns subjected to double-curvature bending. These equations have been developed for columns with flange and web width-to-thickness ratios satisfying current AISC Seismic Provisions seismically compact limits and for steel with a nominal yield stress of 50 ksi.

\[ R_p = 2750 \left( \frac{b_f}{2t_f} \right)^{-1.40} \left( \frac{L_b}{r_y} \right)^{-0.80} \left( \frac{P}{P_{ye}} \right)^{0.55} \]  

(6.20)

\[ R_p = 6850 \left( \frac{b_f}{2t_f} \right)^{-0.80} \left( \frac{h}{t_w} \right)^{-0.95} \left( \frac{L_b}{r_y} \right)^{-0.65} \left( \frac{P}{P_{ye}} \right)^{0.30} \]  

(6.21)
The constants and exponents in Eqs. 6.20 and 6.21 have been slightly modified from those of Eqs. 6.17 and 6.18 for simplicity. Also, $P_{ya}$ was replaced by $P_{ye}$ for consistency with the current ASCE 41 approach for evaluation of existing buildings.

The applicability of these equations is limited to the range of parameters included in the parametric study described in Chapter 5. Column axial loads are limited to the range of $0.20 - 0.75 \frac{P}{P_{yn}}$. For the W12 and W14 models the column depth to flange width ratio ($d_c/b_f$) varied between 1.0 and 1.2. Figure 6.19 shows that for W8, W10, W12, and W14 sections typically used as columns this ratio is in the range of 0.95–1.25 (consistent with the range investigated in the parametric study). Use of Eq. 6.20 is limited to $d_c/b_f \leq 1.25$. The web to flange width-to-thickness ratio for the W12 and W14 models varied between 2.2 and 2.6. For the W8 to W14 sections, Figure 6.20 shows that this ratio is in the range of 2.0–2.7. This range is similar to that included in the parametric study and serves as the limiting range for use of Eq. 6.20. Global column slenderness ($L_b/r_y$) was in the range of 34 to 70 for the W12 and W14 column models. For the proposed column plastic rotation capacity model global column slenderness is limited to $L_b/r_y \leq 70$.

For the W18 and W24 models the column depth to flange width ratio ($d_c/b_f$) varied between 1.7 and 2.0. Figure 6.21 shows that for W18, W21, W24, and W27 sections this ratio is in the range of 1.7–2.1. Eq. 6.21 is appropriate for $d_c/b_f$ between 1.7 and 2.1. The web to flange width-to-thickness ratio for the investigated W18 and W24 sections varied between 4.6 and 6.0. Figure 6.22 shows that this ratio is in the range of 4.5–6.0 for the W18 to W27 sections and is the limiting range for use of Eq. 6.21. Global column slenderness ($L_b/r_y$) was in the range of 45 to 82 for the deep
column models. For use of Eq. 6.21 global column slenderness is limited to $L_b/\sigma_y \leq 80$.

Eqs. 6.20 and 6.21 are also appropriate for built-up sections with dimensions similar to wide-flange sections and for parameters within the ranges of applicability provided in Table 6.2. Figure 6.23 shows two built-up sections that fall outside the range of applicability for these capacity prediction models because the web to flange width-to-thickness ratios are vastly different than typical hot-rolled sections.

The plastic rotation capacity predicted by Eq. 6.20 or Eq. 6.21 can be compared to the plastic rotation demand determined from nonlinear time-history analysis to evaluate the adequacy of a given column in a new or existing building. Or with appropriate rearrangement of the equations, the limiting allowable axial load ratio for a given column section and plastic rotation could be determined. This approach may be useful for new design where plastic hinging is expected at column bases. The plastic rotation demand could be determined based on the building code specified drift limit or higher performance design criteria. The axial load ratio calculated considering amplified earthquake loads could then be compared with the predicted limiting axial load ratio to determine the adequacy of a given column.

6.5.5 Nonlinear Interstory Drift Capacity Model

Eq. 6.22 represents a nonlinear model (similar in form to Eq. 6.13) for prediction of $IDC$. Eq. 6.22 can be linearized by taking the logarithm of both sides of the equation as given by Eq. 6.23. The constant $B$ and exponents $\varepsilon$, $\zeta$, $\eta$, and $\iota$ were determined from regression of this linearized model.
\[ IDC = B \left( \frac{b_f}{2t_f} \right)^{\xi} \left( \frac{h}{t_w} \right)^{\zeta} \left( \frac{L_b}{r_y} \right)^{\eta} \left( \frac{P}{P_{ya}} \right)^{1} \]  
(6.22)

\[ \log(IDC) = \log(B) + \varepsilon \log \left( \frac{b_f}{2t_f} \right) + \zeta \log \left( \frac{h}{t_w} \right) + \eta \log \left( \frac{L_b}{r_y} \right) + \log \left( \frac{P}{P_{ya}} \right) \]  
(6.23)

Similar to the $R_p$ model, due to the linear dependence of $b_f/2t_f$ and $h/t_w$, regression analysis was performed for the typical and deep column model data using Eqs. 6.24a and 6.24b, respectively.

\[ IDC = B \left( \frac{b_f}{2t_f} \right)^{\xi} \left( \frac{L_b}{r_y} \right)^{\eta} \left( \frac{P}{P_{ya}} \right)^{1} \]  
(6.24a)

\[ IDC = B \left( \frac{h}{t_w} \right)^{\zeta} \left( \frac{L_b}{r_y} \right)^{\eta} \left( \frac{P}{P_{ya}} \right)^{1} \]  
(6.24b)

### 6.5.6 Regression Analysis of Interstory Drift Capacity

Regression analysis of interstory drift capacity for a data set consisting of Model W12 and W14 analysis results for axial loads of 20\%, 35\%, 55\%, 75\%, $P_{yn}$ was conducted to determine the unknown constant and exponents. A least-squares regression resulted in Eq. 6.25 with a coefficient of determination, $r^2$, equal to 0.94.

\[ IDC = 0.38 \left( \frac{b_f}{2t_f} \right)^{-1.18} \left( \frac{L_b}{r_y} \right)^{-0.05} \left( \frac{P}{P_{ya}} \right)^{-0.21} \]  
(6.25)

The magnitudes of the exponents indicate that the effect of local buckling is the most significant parameter affecting $IDC$. The magnitude of the exponent for $L_b/r_y$ is close to zero indicating that the influence of global column slenderness on $IDC$ was insignificant for the modeled sections. Figure 6.24 shows a plot of predicted $IDC$. 


versus actual IDC (from finite element analysis). Some significant deviations in predicted versus actual response were observed at higher levels of IDC.

A separate regression analysis was performed for the W18 and W24 column model data set. Predicted IDC is given by Eq 6.26 and resulted in an $r^2$ of 0.95.

$$IDC = 1.95 \left( \frac{h}{t_w} \right)^{-1.52} \left( \frac{L_b}{r_y} \right)^{0.06} \left( \frac{P}{P_{ya}} \right)^{-0.43}$$  \hspace{1cm} (6.26)

The value of the exponents for $L_b/r_y$ is close to zero, indicating that global column slenderness does not significantly affect IDC for the W18 and W24 column model data set. Figure 6.25 plots the predicted IDC versus actual IDC.

A regression analysis considering all column models for axial loads of 20%, 35%, 55%, and 75% $P_{yn}$ resulted in Eq. 6.27 with an $r^2$ equal to 0.94.

$$IDC = 0.53 \left( \frac{b_f}{2t_f} \right)^{-0.30} \left( \frac{h}{t_w} \right)^{-1.00} \left( \frac{L_b}{r_y} \right)^{0.11} \left( \frac{P}{P_{ya}} \right)^{-0.32}$$  \hspace{1cm} (6.27)

The exponent for $h/t_w$ is more than three times the exponent for $b_f/2t_f$ indicating that the influence of web local buckling is dominant over the influence from flange local buckling. This relative importance of flange and web local buckling for IDC is opposite that of $R_p$ (see Eq. 6.19) and provides further justification of the use of separate models for typical and deep column sections. Predicted IDC versus actual IDC is shown in Figure 6.26. As expected for this data set with both typical and deep column sections somewhat greater dispersion was observed in the data.
6.5.7 Proposed Interstory Drift Capacity Model

Similar to the plastic rotation capacity models, the use of separate models for typical and deep column sections is proposed because of observed differences in the behavior of the two column types. Eqs. 6.28 and 6.29 provide reasonable predictions of interstory capacity for typical and deep steel wide-flange or built-up columns, with parameters within the ranges of applicability provided in Table 6.2, subjected to double-curvature bending. These equations have been developed for columns with flange and web width-to-thickness ratios satisfying current AISC Seismic Provisions seismically compact limits and for steel with a nominal yield stress of 50 ksi. Eq. 6.28 is applicable for W8, W10, W12, and W14 typical column sections. Eq. 6.29 is appropriate for the majority of W18, W21, W24, and W27 deep column sections. These equations are also appropriate for built-up sections with dimensions similar to wide-flange sections.

\[
IDC = 0.30 \left( \frac{b_f}{2t_f} \right)^{-1.20} \left( \frac{P}{P_{ye}} \right)^{-0.20} \quad (6.28)
\]

\[
IDC = 2.50 \left( \frac{h}{t_w} \right)^{-1.50} \left( \frac{P}{P_{ye}} \right)^{-0.40} \quad (6.29)
\]

The constants and exponents in Eqs. 6.28 and 6.29 have been slightly modified from those of Eqs. 6.26 and 6.27 for simplicity.

The interstory drift capacity predicted by Eq. 6.28 or Eq. 6.29 can easily be compared to the interstory drift demand determined from nonlinear time-history analysis to evaluate the adequacy of a given column in a new or existing building. Use of interstory drift capacity for comparison with computer analysis results is more
straightforward than plastic rotation capacity. The definition of interstory drift is 
universal; whereas, use of plastic rotation capacity requires that the computer results 
are based on a definition of plastic rotation that is consistent with that used in this 
study.

6.6 Protected Zones

Previous experimental testing has shown that the performance of portions of 
members undergoing large inelastic straining is sensitive to discontinuities caused by 
welding, penetrations, or construction flaws. These discontinuities create a stress 
concentration and correspondingly increase the fracture potential of the member 
(FEMA 2000b). The AISC Seismic Provisions define certain regions of the SLRS as 
protected zones. Examples of these protected zones include MRF hinging zones, 
EBF links, and the ends and center of SCBF braces. No welded, bolted, screwed or 
shot-in attachments (for items such as exterior facades, partitions, duct work, etc.) are 
allowed within the protected zones. Additionally, any discontinuities created within 
the protected zones during fabrication or erection are required to be repaired using a 
method acceptable to the engineer of record. Currently there are no protected zone 
requirements for SLRS columns. The time-history analysis described in Chapter 3 
indicated that BRBF columns can be subject to inelastic demand near the column 
bases and often at intermediate stories. Also, for MRF it is expected that column 
plastic hinging will occur in the bottom-story. For the tested W14 columns a plastic 
hinge length of approximately two times the depth of the column \( d_c \) was observed 
(see Chapter 4). It is therefore recommended that for bottom-story SLRS columns a
protected zone be defined to extend $2d_c$ from the column base plate or end of gusset plate, as appropriate.

### 6.7 Column to Base Plate Connection

This research has been focused on member level behavior. The column to base plate connection has been assumed to have adequate strength and ductility to prevent failure of the column to base plate welded joint or base plate to foundation anchorage. The ends of test specimens were reinforced with re-usable haunches to limit demand on the column to base plate welded joint. Despite this strengthening, Specimen W14×370-35 failed at 3% drift due to fracture of column to base plate welded joint. Finite element analysis predicted the $IDC$ of Model W14×370-35 to equal 9.5% drift, significantly larger than the specimen drift at failure. While column members may possess adequate plastic rotation capacity at high axial loads the adequacy of the column to base plate connections must be evaluated for the expected level of demand. Use of notch-tough weld metal and other lessons learned after the 1994 Northridge earthquake should be considered in evaluating these connections. When column bases are embedded in reinforced concrete grade beams or basement walls the effect of concrete crushing due to high local bearing stresses should be considered.

### 6.8 Summary and Conclusions

This chapter has discussed the limitations of the current ASCE 41 predicted plastic rotation capacity in relation to the results of experimental testing and finite
element analysis conducted as part of this project. The ASCE 41 definitions for reduced plastic moment and yield rotation were modified to more accurately reflect expected behavior. Predictive models of column deformation capacity were developed considering the interaction of flange and web local buckling, global column slenderness, and axial load. Regression analysis was performed to determine unknown model coefficients and evaluate the relative significance of the key interaction parameters. The following conclusions and recommendations can be made.

1. The current ASCE 41 plastic rotation capacity does not adequately predict the rotation capacity observed from the experimental and finite element analysis data. ASCE 41 predicts no plastic rotation capacity above an axial load ratio of 0.5. However, tested specimens and finite element models exhibited significant rotation capacities at axial load ratios greater than 0.5. Additionally, at lower axial load ratios the plastic rotation capacity of some finite element models was less than that predicted by current ASCE 41 criteria.

2. The $P-M$ interaction relation used by ASCE 41 can underestimate the reduced plastic moment and yield rotation, resulting in non-conservative estimates of plastic rotation capacity. The $P-M$ interaction relation given by Eq. 6.5 has been used to calculate the reduced plastic moment because it is more consistent with expected behavior and reduces the potential for a non-conservative prediction of plastic rotation capacity.

3. The current ASCE 41 equation for calculation of yield rotation does not include consideration of shear deformation. It was shown that shear deformation can increase the yield rotation by approximately 10% to 50% for the range of column
models investigated. Therefore, shear deformation has been included in calculation of yield rotation in this study.

4. For a given column section, interstory drift capacity decreased with increasing axial load ratio. However, plastic rotation capacity increased with increasing axial load. This can be explained by the fact that yield rotation decreased with increasing axial load at a faster rate than the interstory drift capacity decreased with increasing axial load. This increase in plastic rotation capacity with increasing axial load was less pronounced for the deep column models.

5. Nonlinear models including the effect of flange, web, and lateral slenderness, and axial load ratio were evaluated using least-squares regression analysis of the database of finite element analysis results described in Chapter 5. For a data set including the W12 and W14 models or the W18 and W24 models a linear relation was observed between flange and web width-to-thickness ratios and therefore one of these terms was redundant and was removed from the regression model. For both typical and deep column sections the influence of local buckling was found to be more significant than that of global column slenderness.

6. Column deformation capacity models in terms of interstory drift or plastic rotation were developed that provide reasonable predictions for typical and deep steel wide-flange columns in double-curvature bending with flange and web width-to-thickness ratios satisfying current AISC Seismic Provisions seismically compact limits, for a range of axial loads from $0.20P_{yn}$ to $0.75P_{yn}$, and for steel with a nominal yield stress of 50 ksi. These models are appropriate for evaluation of column deformation capacity for new and existing buildings.
7. Protected zones are defined for portions of seismic load resisting systems subject to significant inelastic demand to prevent potential stress concentrations that increase fracture potential. Within a protected zone no welded, bolted, screwed or shot-in attachments are allowed. It is recommended that for bottom-story seismic load resisting system columns a protected zone be defined to extend two times the depth of the column from the column base plate or end of gusset plate, as appropriate.

8. This research has shown that at the member level, columns have significant plastic rotation capacity. Due consideration must also be given to column to base plate connections and base plate to foundation anchorage to ensure a complete and adequate load path for the expected forces.
Table 6.1 ASCE 41 Acceptance Criteria for Nonlinear Procedures

<table>
<thead>
<tr>
<th>Axial Load Ratio</th>
<th>Plastic Rotation (×θ_{yc} rad.)</th>
<th>Immediate Occupancy</th>
<th>Life-Safety</th>
<th>Collapse Prevention</th>
</tr>
</thead>
<tbody>
<tr>
<td>For ( P_r/P_c &lt; 0.20 )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A) ( \frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_y}} ) \text{ and } ( \frac{h}{t_w} \leq \frac{300}{\sqrt{F_y}} )</td>
<td></td>
<td>1</td>
<td>6</td>
<td>8</td>
</tr>
<tr>
<td>B) ( \frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_y}} ) \text{ or } ( \frac{h}{t_w} \geq \frac{460}{\sqrt{F_y}} )</td>
<td>0.25</td>
<td>2</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>For ( 0.2 &lt; P_r/P_c &lt; 0.50 )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A) ( \frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_y}} ) \text{ and } ( \frac{h}{t_w} \leq \frac{260}{\sqrt{F_y}} )</td>
<td>0.25</td>
<td>( 8 \left( 1 - \frac{5}{3} \frac{P_r}{P_c} \right) )</td>
<td>11 ( 1 - \frac{5}{3} \frac{P_r}{P_c} )</td>
<td></td>
</tr>
<tr>
<td>B) ( \frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_y}} ) \text{ or } ( \frac{h}{t_w} \geq \frac{400}{\sqrt{F_y}} )</td>
<td>0.25</td>
<td>0.5</td>
<td>0.8</td>
<td></td>
</tr>
<tr>
<td>For ( P_r/P_c &gt; 0.50 )</td>
<td></td>
<td>Force-Controlled (see Eq. 6.1)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 6.2 Range of Applicability for Predicted Column Deformation Capacity

<table>
<thead>
<tr>
<th>Equation</th>
<th>( P/P_{yn} )</th>
<th>( d_c/b_f )</th>
<th>( h/t_w )</th>
<th>( L_b/r_y )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eqs. 6.20 and 6.28</td>
<td>0.20–0.75</td>
<td>( \leq 1.25 )</td>
<td>2.0–2.7</td>
<td>( \leq 70 )</td>
</tr>
<tr>
<td>Eqs. 6.21 and 6.29</td>
<td>0.20–0.75</td>
<td>1.7–2.1</td>
<td>4.5–6.0</td>
<td>( \leq 80 )</td>
</tr>
</tbody>
</table>
Figure 6.1 Comparison of Test Data with ASCE 41 Acceptance Criteria (Collapse Prevention Limit State)

Figure 6.2 Definition of Plastic Rotation
Figure 6.3 Comparison of Analysis Data with ASCE 41 Acceptance Criteria (Collapse Prevention Limit State)

Figure 6.4 Comparison of $P$-$M$ Interaction Definitions
Figure 6.5 Yield Rotation Amplification with Axial Load (Euler-Bernoulli Beam)
Figure 6.6 Model W14×132: Comparison of Initial Rotational Stiffness
Figure 6.7 Comparison of Yield Rotation for Timoshenko and Euler-Bernoulli Beam Theories (W12, W14, W18, and W24 models)
Figure 6.8 Specimen W14 Series: Rotation Capacities versus Axial Load Ratio

(a) Interstory Drift Capacity

(b) Plastic Rotation Capacity
Figure 6.9 Model W12 Series: Rotation Capacities versus Axial Load Ratio
Figure 6.10 Model W14 Series: Rotation Capacities versus Axial Load Ratio

(a) Interstory Drift Capacity

(b) Plastic Rotation Capacity
(a) Interstory Drift Capacity

(b) Plastic Rotation Capacity

Figure 6.11 Model W18 Series: Rotation Capacities versus Axial Load Ratio
Figure 6.12 Model W24 Series: Rotation Capacities versus Axial Load Ratio

(a) Interstory Drift Capacity

(b) Plastic Rotation Capacity
Figure 6.13 Model W12, W14, W18 and W24 Series: Rotation Capacities versus Axial Load Ratio.

(a) Interstory Drift Capacity

(b) Plastic Rotation Capacity
Figure 6.14 Slenderness Histograms

(a) Flange Slenderness

(b) Web Slenderness

(c) Column Slenderness

\[
\lambda_{ps} = 7.2
\]

\[
\lambda_{ps} = 40.4 \quad \left( \frac{P}{P_{yn}} = 0.75 \right)
\]
Figure 6.15 Relation Between Flange and Web Width-to-Thickness Ratios for Modeled Sections
Figure 6.16 Models W12 and W14: Predicted versus Actual Plastic Rotation Capacity

(a) 10%, 20%, 35%, 55%, 75%, and 90% $P_{yn}$

(b) 20%, 35%, 55%, and 75% $P_{yn}$
Figure 6.17 Models W18 and W24: Predicted versus Actual Plastic Rotation Capacity (Axial Load: 20%, 35%, 55%, and 75% $P_{yn}$)

Figure 6.18 Models W12, W14, W18, and W24: Predicted versus Actual Plastic Rotation Capacity (Axial Load: 20%, 35%, 55%, and 75% $P_{yn}$)
Figure 6.19 Column Depth to Flange Width Ratio for W8, W10, W12, and W14 Sections

Figure 6.20 Web to Flange Width-to-Thickness Ratio for W8, W10, W12, and W14 Sections

\[ \lambda_{ps} = 40.4 \ (P/P_{yn} = 0.75) \]
Figure 6.21 Column Depth to Flange Width Ratio for W18, W21, W24, and W27 Sections

Figure 6.22 Web to Flange Width-to-Thickness Ratio for W18, W21, W24, and W27 Sections
Figure 6.23 Built-up Sections not Similar to Hot-Rolled Sections

Figure 6.24 Models W12 and W14: Predicted versus Actual Interstory Drift Capacity
(Axial Load: 20%, 35%, 55%, and 75% $P_{yw}$)
Figure 6.25 Models W18 and W24: Predicted versus Actual Interstory Drift Capacity (Axial Load: 20%, 35%, 55%, and 75% \( P_{yn} \))

Figure 6.26 Models W12, W14, W18, and W24: Predicted versus Actual Interstory Drift Capacity (Axial Load: 20%, 35%, 55%, and 75% \( P_{yn} \))
7 SUMMARY AND CONCLUSIONS

7.1 Summary

Nonlinear time-history analysis has revealed that braced frame columns are sometimes subjected to combined high axial loads and large story drifts resulting in inelastic rotation (flexural) demand. This inelastic demand on column members is contrary to the current seismic design philosophy in which members other than the structural fuses (i.e., deformation-controlled elements) are designed to remain elastic under the expected demand. However, inelastic rotation has to be expected for reasonably sized columns with fixed-base connections. An experimental and analytical investigation has been conducted to evaluate the performance of steel wide-flange columns under combined axial load and large story drift.

The experimental program consisted of testing nine full-scale columns. ASTM A992 steel wide-flange sections typical of braced frame columns, representing a practical range of flange and web width-to-thickness ratios, and 15 ft story height were subjected to different levels of axial force demand (35%, 55%, and 75% of the nominal axial yield strength) combined with story drift demand. For testing purposes, a combined axial load and story drift loading sequence was developed because such a cyclic loading protocol for column testing is not available in building codes and specifications. This dual parameter cyclic loading sequence was based on the expected seismic demand quantified using nonlinear time-history analysis of prototype 3-story and 7-story buckling-restrained braced frames.

Subjecting the column specimens to different levels of axial force demand, interstory drift capacities of approximately 0.07 rad. to 0.09 rad. were observed
assuming a 10% reduction from maximum moment resistance was used to define the interstory drift capacity (\(IDC\)). Defining the plastic rotation capacity as the ratio between the plastic component of column end rotation and the yield rotation, reduced appropriately to account for the column axial load effect, the \(IDC\) values correspond to plastic rotation capacities ranging from approximately 9 to 20.

The test data agreed well with the established axial load-moment interaction surface. No global buckling was observed in all test specimens. Flange local buckling was the dominant buckling mode. Web local buckling was not obvious, probably due to the inherent low width-to-thickness ratios of the tested W14 sections. The stabilizing effect from the stocky web to delay the onset of flange local buckling significantly contributed to the large deformation capacity of the specimens.

A finite element analysis parametric study of W12, W14, W18 and W24 column models was conducted. These column models were subjected to different levels of axial force and drift demand using the developed braced frame loading sequence. Models of the W14 test specimens were shown to reasonably predict experimental results. The effect of residual stress and initial geometric imperfections were included in the models. For a given column section (i.e., W14×132) the amplitude of local buckling increased with increasing axial load and the \(IDC\) decreased with increasing axial load. For stockier sections the onset of local buckling (and associated strength degradation) was delayed, thus increasing the \(IDC\) of the section.

The current ASCE 41 plastic rotation capacity does not adequately predict the rotation capacity observed from the experimental and finite element analysis data.
ASCE 41 predicts no plastic rotation capacity above an axial load ratio of 0.5. However, tested specimens and finite element models exhibited significant rotation capacities at axial load ratios greater than 0.5. Additionally, at lower axial load ratios the plastic rotation capacity of some finite element models was less than that predicted by current ASCE 41 criteria. The ASCE 41 definition for yield rotation was modified to include the significant contribution from shear deformation. Plastic rotation and interstory drift capacity predictive models were developed considering the interaction of flange and web local buckling, column slenderness, and axial load ratio. Regression analysis was performed to determine unknown model coefficients and evaluate the relative significance of the key interaction parameters. These models are appropriate for evaluation of column plastic rotation and interstory drift capacities for new and existing buildings.

7.2 Conclusions

Based on the results of experimental and analytical investigation the following conclusions are presented regarding the seismic design and analysis of steel wide-flange columns under combined axial load and story drift demand.

1. For deep column models (W18 and W24) the plastic rotation capacity was reduced due to residual stress and initial geometric imperfections. This observation was contrary to previous research which reported that these factors did not significantly affect ductility (Nakashima 1994).

2. More significant flange and web local buckling was observed for the deep column models (W18 and W24) as compared with typical columns (W12 and W14).
This increased level of local buckling reduced the interstory drift capacity of the deep column models. The stabilizing effect from the stocky web to delay the onset of flange local buckling significantly contributed to the large deformation capacities of the W12 and W14 sections.

3. A comparison of ASCE 41 plastic rotation capacity criteria with experimental and finite element analysis data showed that ASCE 41 does not adequately predict column plastic rotation capacity. ASCE 41 predicts no plastic rotation capacity above an axial load ratio of 0.5. However, tested specimens and finite element models exhibited significant rotation capacities at axial load ratios greater than 0.5. Additionally, at lower axial load ratios the plastic rotation capacity of some finite element models was less than that predicted by ASCE 41 criteria.

4. The ASCE 41 definition of yield rotation does not include the contribution of shear deformation and can under-predict the actual yield rotation by as much as 50%. A modified definition of yield rotation was used in this study that was more consistent with expected behavior and reduced the potential for a non-conservative prediction of plastic rotation capacity.

5. Contrary to the trend predicted by ASCE 41, specimen and model plastic rotation capacities were observed to increase with increasing axial load. This can be explained by the definition of plastic rotation capacity and the fact that yield rotation decreased with increasing axial load at a faster rate than the interstory drift capacity decreased with increasing axial load.

6. Plastic rotation and interstory drift capacity models were developed that provide reasonable predictions for typical and deep steel wide-flange columns in double-
curvature bending with flange and web width-to-thickness ratios satisfying current AISC Seismic Provisions seismically compact limits. These models are appropriate for evaluation of column deformation capacity for the range of parameters included in the finite element analysis database. The range of applicability, in terms of axial load ratio, column depth to flange width ratio, web to flange width-to-thickness ratio, and column slenderness are presented in Chapter 6. These models are appropriate for evaluation of column rotation capacity for new and existing buildings.

7. It is recommended that a protected zone be defined for bottom-story seismic load resisting system columns, to extend two times the depth of the column from the column base plate or end of gusset plate, as appropriate.

7.3 Recommendations for Future Work

Results of finite element analysis showed that the interstory drift capacity of deep (W18 and W24) columns was reduced compared to the stockier typical column sections (W12 and W14). More significant flange and web local buckling and interaction of these local buckling modes were observed for the deep column models. Deep column sections should be subjected to an experimental program similar to the W14 column testing described in Chapters 3 and 4. This testing would provide experimental data for validating the deep column models described in Chapter 5.

This research has shown that at the member level, columns have significant plastic rotation capacity. Due consideration must also be given to column to base plate connections and base plate to foundation anchorage to ensure a complete and
adequate load path for the expected demand. Experimental and analytical research should be conducted to verify that typical column to base plate connections are capable of developing the expected plastic moment of the column.

7.4 Concluding Remarks

The development of new structural systems often refocuses attention on gaps in the existing knowledgebase. The buckling-restrained braced frame system has called to attention the potentially significant effect of columns with high axial loads combined with inelastic rotation demand. This concern is valid for concentrically braced frames, eccentrically braced frames, and moment resisting frames as well. Fortunately, the results of this experimental and analytical investigation have shown that columns meeting the width-to-thickness requirements for seismically compact sections possess large plastic rotation capacities, even at very large axial load levels.

This dissertation follows in a long line of those that continue to illustrate the importance of continued research on emerging and well established structural systems. This is especially true in light of the industry wide emphasis on higher performance of structures during and after extreme events. An ever increasing knowledgebase is required to accurately predict expected behavior and achieve performance goals.
APPENDIX A. TIME-HISTORY ANALYSIS RESULTS
A.1 3-Story BRBF

Figures A.1 to A.8 provide typical results for the 3-story BRBF subjected to ground motion records P09 and P14. The figures show story drift ratio, bottom-story column axial load, and end moment time-histories along with the axial load-moment \((P-M)\) interaction for one bottom-story column and BRB hysteretic response for each brace.
Figure A.1 3-Story BRBF: Story Drift Ratio Time-Histories (Record P09)
Figure A.2 3-Story BRBF: Column Response Time-Histories (Record P09)
Figure A.3 3-Story BRBF: $P-M$ Interaction (Record P09)
Figure A.4 3-Story BRBF: Brace Axial Load versus Axial Deformation Relation (Record P09)
Figure A.5 3-Story BRBF: Story Drift Ratio Time-Histories (Record P14)
Figure A.6 3-Story BRBF: Column Response Time-Histories (Record P14)

(a) Axial Load

(b) Moment at Column Base
Figure A.7 3-Story BRBF: $P$-$M$ Interaction (Record P14)
Figure A.8 3-Story BRBF: Brace Axial Load versus Axial Deformation Relation (Record P14)
A.2 7-Story BRBF

Figures A.9 to A.16 provide typical results for the 7-story BRBF for ground motion records P08 and P19. The figures show story drift ratio, bottom-story column axial load, and end moment time-histories along with the axial load-moment ($P-M$) interaction for one bottom-story column and BRB hysteretic response for each brace.
Figure A.9 7-Story BRBF: Story Drift Ratio Time-Histories (Record P08)
Figure A.10 7-Story BRBF: Column Response Time-Histories (Record P08)
Figure A.11 7-Story BRBF: $P-M$ Interaction (Record P08)
Figure A.12 7-Story BRBF: Brace Axial Load versus Axial Deformation Relation for Record P08
Figure A.12 7-Story BRBF: Brace Axial Load versus Axial Deformation Relationship for Record P08 (cont.)
Figure A.13 7-Story BRBF: Story Drift Ratio Time-Histories (Record P19)
Figure A.14 7-Story BRBF: Column Response Time-Histories (Record P19)
Figure A.15 7-Story BRBF: $P$-$M$ Interaction (Record P19)
Figure A.16 7-Story BRBF: Brace Axial Load versus Axial Deformation Relation for Record P19
Figure A.16 7-Story BRBF: Brace Axial Load versus Axial Deformation Relationship for Record P19 (cont.)
APPENDIX B. EXPERIMENTAL RESULTS
B.1 Specimen W14×132-35

Specimen W14×132-35 (0.35\(P_n\) axial load target) was tested on November 22, 2005 using Loading Scheme A. Lateral drifts of up to 6% (10.8 in.) were applied. For large cycles, greater than 2% drift, the column axial load targets were not achieved for the reasons described in Chapter 4. An axial compressive load offset of 0.15\(P_n\) (246 kips) was initially applied for the W14×132 specimens, followed by cyclic, increasing amplitude longitudinal and lateral displacement.

B.1.1 Observed Performance

Figure B.1 shows the progression of yielding at each end of the specimen. Flange local buckling, as shown in Figure B.2, was observed to develop at 5% drift (compression target axial load). Photos showing the specimen overall deformed configuration at 4% and 6% drift are shown in Figure B.3. No fracture of the test specimen was observed.

B.1.2 Recorded Response

The axial load-moment (\(P-M\)) interaction is shown in Figure B.4, along with the \(P-M\) interaction surfaces based on actual and nominal material properties. The horizontal line shown is at the target compressive axial load level of 0.35\(P_n\). Figure B.5 shows the end moment versus story drift relationship. Plots of longitudinal force versus lateral force, longitudinal force versus column axial displacement, and lateral force versus lateral displacement are provided in Figure B.6. The column axial
displacement shown in Figure B.6(b) was calculated from the average of displacement transducers L1 and L2 (see Figure 3.5).

B.2 Specimen W14×132-55

Specimen W14×132-55 (0.55P_{yn} axial load target) was tested on May 1, 2006 using Loading Scheme B. Lateral drifts of up to 8% (14.4 in.) were applied. For large cycles, greater than 2% drift, the column axial load targets were not achieved as discussed in Chapter 4.

B.2.1 Observed Performance

Figure B.7 shows the progression of yielding at each end of the specimen. The progression of flange local buckling (at compression target axial load) is shown Figure B.8. Photos showing the specimen overall deformed configuration at 4% and 8% drift are shown in Figure B.9. No fracture of the test specimen was observed.

B.2.2 Recorded Response

The recorded P-M interaction is shown in Figure B.10 and the end moment versus story drift response is shown in Figure B.11. Plots of longitudinal force versus lateral force, longitudinal force versus column axial displacement, and lateral force versus lateral displacement are provided in Figure B.12. The jagged response observed in these global response plots resulted from using manual control of the input displacements in the refinement of Loading Scheme C.
B.3 Specimen W14×132-75

Specimen W14×132-75 (0.75\(P_{yn}\) axial load target) was tested on May 11, 2006 using Loading Scheme C. Lateral drifts of up to 10% (18 in.) were applied.

B.3.1 Observed Performance

Figure B.13 shows the progression of yielding at each end of the specimen. The progression of flange local buckling is shown Figure B.14. Photos showing the specimen overall deformed configuration at large drifts are shown in Figure B.15. On the 10% drift tension excursion one column flange completely fractured through the bolt hole net section and propagated partially through the web (see Figure B.16). Despite fracture, the 10% drift cycle was completed with drift and axial load targets being achieved.

B.3.2 Recorded Response

The \(P-M\) interaction response is shown in Figure B.17. Figure B.18 shows the \(P-M\) interaction surface and \(P-M\) interaction for discrete points corresponding to points in the loading history (from 2% to 10% drift) when the target values of axial load and drift were achieved. These points correspond to points 2 and 5 of the loading sequence shown in Figure 3.31. The overall moment versus drift response is shown in Figure B.19 along with points corresponding to when the target values of axial load and drift were achieved. These points represent the response of primary interest in this study of columns under combined high axial load and drift demand. Figure B.20 provides a plot of the end moment versus drift response based on the target points on the compression side. The maximum moment (combined with the
target axial load) was reached at 6% drift. The horizontal line shown is at 90% of this maximum moment. Using a 10% reduction from $M_{\text{max}}$ to define the interstory drift capacity (IDC), the IDC for this specimen was 0.0725 rad. Additional force-displacement response plots are shown in Figure B.21.

Strain profiles across each column flange near the haunch tip are shown in Figure B.22 for 2% to 6% drift. (See Figure 3.6 for strain gage locations.) From a comparison of Figures B.22(a) and B.22(b) it is evident that combined axial and bending loading resulted in higher compressive strains in one flange (and the opposite flange on the other end of the specimen). This was consistent with observations from flaking of the whitewash. Figure B.23 provides flange strain profiles along the column length. The horizontal lines shown are at plus and minus the measured yield strain. These profiles confirm the observation that flange yielding was confined to the ends of the specimen.

### B.4 Specimen W14×176-35

Specimen W14×176-35 (0.35$P_{yn}$ axial load target) was tested on November 29, 2005 using Loading Scheme A. Lateral drifts of up to 10% (18 in.) were applied. For large cycles, greater than 2% drift, the column axial load targets were not achieved for the reasons described in Chapter 4. An axial compressive load offset of 0.15$P_{n}$ (335 kips) was initially applied for the W14×176 specimens.
B.4.1 Observed Performance

Figure B.24 shows the progression of yielding at each end of the specimen for 2% to 10% drift. Flange local buckling, as shown in Figure B.25, was observed to develop at 6% drift. Photos showing the specimen overall deformed configuration at large drifts are shown in Figure B.26. After testing a partial fracture of the column flange was observed, as shown in Figure B.27.

B.4.2 Recorded Response

The $P$-$M$ interaction and moment versus drift response are shown in Figures B.28 and B.29 respectively. Plots of longitudinal force versus lateral force, longitudinal force versus column axial displacement, and lateral force versus lateral displacement are provided in Figure B.30.

B.5 Specimen W14×176-55

Specimen W14×176-55 (0.55$P_{yu}$ axial load target) was tested on May 8, 2006 using Loading Scheme C. Lateral drifts of up to 10% (18 in.) were applied.

B.5.1 Observed Performance

Figure B.31 shows the progression of yielding at each end of the specimen. The progression of flange local buckling is shown in Figure B.32. Photos showing the specimen overall deformed configuration at large drifts are shown in Figure B.33. Near the end of the 10% drift cycle the column flange completely fractured through the bolt hole net section and propagated partially through the web (see Figure B.34).
B.5.2 Recorded Response

Figure B.35 shows the overall $P-M$ interaction and Figure B.36 shows the $P-M$ interaction for points where the axial load and drift target points were achieved. Figure B.37 shows the overall moment versus drift response along with the points where the targets were reached. The plot of the end moment versus drift response based on the target points on the compression side, shown in Figure B.38, indicates the drift capacity for this specimen was 0.0912 rad., based on an allowable 10% reduction in moment from $M_{max}$. Figure B.39 provides additional force-displacement response plots. Strain profiles across the column flange and along the column length are shown in Figures B.40 and B.41, respectively.

B.6 Specimen W14×176-75

Specimen W14×176-75 (0.75$P_{yn}$ axial load target) was tested on May 15, 2006 using Loading Scheme C. Lateral drifts of up to 10% (18 in.) were applied.

B.6.1 Observed Performance

Figure B.42 shows the progression of yielding at each end of the specimen. Increasing amplitude of flange local buckling from 4% to 10% drift is shown in Figure B.43. Photos showing the specimen overall deformed configuration at large drifts are shown in Figure B.44. On the 10% drift cycle one column flange completely fractured through the bolt hole net section and propagated partially through the web (see Figure B.45). Despite the fracture, the 10% drift cycle was completed with drift and axial load targets being achieved.
B.6.2 Recorded Response

The recorded $P-M$ response is shown in Figure B.46, along with the $P-M$ interaction yield surface. Figure B.47 shows the $P-M$ interaction for the target points. Notice on the tension side that the moment was essentially reduced to zero at 10% drift after fracture of the column flange but the target axial load was still achieved. Figure B.48 shows the overall and target point moment versus drift response. A drift capacity of 0.0868 rad. was observed from Figure B.49, which shows the end moment versus drift response based on the target points on the compression side. Longitudinal and lateral force versus displacement response is shown in Figure B.50. Strain profiles across the column flange and along the column length are shown in Figures B.51 and B.52, respectively.

B.7 Specimen W14×233-35

Specimen W14×233-35 (0.35$P_{yn}$ axial load target) was tested on May 18, 2006 using Loading Scheme C. Lateral drifts of up to 8% (14.4 in.) were applied. An axial compressive load offset of 0.15$P_{n}$ (446 kips) was initially applied for the W14×233 specimens.

B.7.1 Observed Performance

Figure B.53 shows the yielding pattern at each end of the specimen for 2% to 8% drift. The progression of flange local buckling is shown in Figure B.54. Photos showing the specimen overall deformed configuration at large drift are shown in Figure B.55. On the 9% drift tension excursion the specimen experienced a “divot”
pull out fracture of the flange to base plate welded joint (see Figure B.56) and testing was suspended.

**B.7.2 Recorded Response**

Figure B.57 shows the $P-M$ interaction and Figure B.58 shows the $P-M$ interaction for points where the axial load and drift target points were achieved. Figure B.59 shows the overall moment versus drift response along with the points where the targets were reached. The plot of the end moment versus drift response based on the target points on the compression side, shown in Figure B.60, indicates the drift capacity for this specimen was 0.08 rad., as limited by fracture of the column flange to base plate welded joint on the 9% drift tension excursion. Figure B.61 provides additional force-displacement response plots. Strain profiles across the column flange and along the column length are shown in Figures B.62 and B.63, respectively.

**B.8 Specimen W14×233-55**

Specimen W14×233-55 ($0.55P_{yn}$ axial load target) was tested on May 22, 2006 using Loading Scheme C. Lateral drifts of up to 10% (18 in.) were applied. Testing of the specimen was suspended just prior to achieving the 9% drift target compression axial load due to a mechanical issue with the SRMD machine.
B.8.1 Observed Performance

Figure B.64 shows the yielding pattern at each end of the specimen. The progression of flange local buckling is shown Figure B.65. Photos showing the specimen overall deformed configuration at large drift are shown in Figure B.66. No fracture of the test specimen was observed.

B.8.2 Recorded Response

The recorded $P-M$ interaction is shown in Figure B.67, along with the $P-M$ interaction yield surface. Figure B.68 shows the $P-M$ interaction for the target points. Figure B.69 shows the overall and target point moment versus drift response. A drift capacity of at least 0.08 rad. was observed from Figure B.70, which shows the end moment versus drift response based on the target points on the compression side. A higher allowable drift would have likely been observed if there had not been a mechanical issue with the SRMD machine. Longitudinal and lateral force versus displacement response is shown in Figure B.71. Strain profiles across the column flange and along the column length are shown in Figures B.72 and B.73, respectively.

B.9 Specimen W14×370-35

Specimen W14×370-35 ($0.35P_{yn}$ axial load target) was tested on May 26, 2006 using Loading Scheme C. Lateral drifts of up to 2% (3.6 in.) were applied. An axial compressive load offset of $0.15P_{n}$ (718 kips) was initially applied, followed by cyclic testing.
B.9.1 Observed Performance

Figure B.74 shows the specimen before testing. The yielding pattern at 2% drift is shown in Figure B.75; only very minor yielding was observed. On the 3% drift tension excursion the specimen experienced a fracture of the column flange to base plate welded joint (see Figure B.76). Note that no local buckling was observed during testing.

B.9.2 Recorded Response

The $P-M$ interaction and moment versus drift response are shown in Figures B.77 and B.78, respectively. Plots of longitudinal force versus lateral force, longitudinal force versus column axial displacement, and lateral force versus lateral displacement are provided in Figure B.79.
(a) 2% Drift

(b) 4% Drift

(c) 6% Drift

Figure B.1 Specimen W14×132-35: Yielding Pattern
Figure B.2 Specimen W14×132-35: Flange Local Buckling (West End)
Figure B.3 Specimen W14×132-35: Overall Deformed Configuration
Figure B.4 Specimen W14×132-35: $P$-$M$ Interaction

Figure B.5 Specimen W14×132-35: End Moment versus Drift Response
Figure B.6 Specimen W14×132-35: Force-Displacement Response
Figure B.7 Specimen W14×132-55: Yielding Pattern
East End

West End

(d) 8% Drift

Figure B.7 Specimen W14×132-55: Yielding Pattern (cont.)
Figure B.8 Specimen W14×132-55: Flange Local Buckling (West End)
Figure B.9 Specimen W14×132-55: Overall Deformed Configuration
Figure B.10 Specimen W14×132-55: P-M Interaction

Figure B.11 Specimen W14×132-55: End Moment versus Drift Response
Figure B.12 Specimen W14×132-55: Force-Displacement Response
Figure B.13 Specimen W14×132-75: Yielding Pattern

(a) 2% Drift

(b) 4% Drift

(c) 6% Drift
Figure B.13 Specimen W14×132-75: Yielding Pattern (cont.)

(d) 8% Drift

(e) 10% Drift
Figure B.14 Specimen W14×132-75: Flange Local Buckling (West End)
Figure B.15 Specimen W14×132-75: Overall Deformed Configuration
Figure B.15 Specimen W14×132-75: Overall Deformed Configuration (cont.)
(a) Overall View of Fracture Location

(b) View of Fracture from Exterior Side of Flange (with Haunches Removed)

(c) View of Fracture from Interior Side of Flange (with Haunches Removed)

Figure B.16 Specimen W14×132-75: Column Fracture (10% Drift)
Figure B.17 Specimen W14×132-75: P-M Interaction

Figure B.18 Specimen W14×132-75: P-M Interaction (Target Points)
Figure B.19 Specimen W14×132-75: End Moment versus Drift Response

Figure B.20 Specimen W14×132-75: Compression Side Target Points End Moment versus Drift Response
Figure B.21 Specimen W14×132-75: Force-Displacement Response

(a) Longitudinal Force versus Lateral Force

(b) Longitudinal Force versus Column Axial Displacement

(c) Lateral Force versus Lateral Displacement
Figure B.22 Specimen W14×132-75: Strain Profiles Across Column Flange (Negative Drift)

Figure B.23 Specimen W14×132-75: Strain Profiles Along Column Length

(a) Strain Gages S1, S2, and S3
(b) Strain Gages S4, S5, and S6
Figure B.24 Specimen W14×176-35: Yielding Pattern
Figure B.24 Specimen W14×176-35: Yielding Pattern (cont.)
(a) 4% Drift

(b) 6% Drift

(c) 8% Drift

(d) 10% Drift

Figure B.25 Specimen W14×176-35: Flange Local Buckling (West End)
Figure B.26 Specimen W14×176-35: Overall Deformed Configuration

- (a) 4% Drift
- (b) 6% Drift
Figure B.26 Specimen W14×176-35: Overall Deformed Configuration (cont.)
Figure B.27 Specimen W14×176-35: Partial Column Flange Fracture (10% Drift)
Figure B.28 Specimen W14×176-35: P-M Interaction

Figure B.29 Specimen W14×176-35: End Moment versus Drift Response
Figure B.30 Specimen W14×176-35: Force-Displacement Response
Figure B.31 Specimen W14×176-55: Yielding Pattern
East End

West End

(d) 8% Drift

(e) 10% Drift

Figure B.31 Specimen W14×176-55: Yielding Pattern (cont.)
Figure B.32 Specimen W14×176-55: Flange Local Buckling (West End)
Figure B.33 Specimen W14×176-55: Overall Deformed Configuration
Figure B.33 Specimen W14×176-55: Overall Deformed Configuration (cont.)

(c) 8% Drift

(d) 10% Drift
(a) Overall View of Fracture Location

(b) View of Fracture from Exterior Side of Flange

(c) View of Fracture from Interior Side of Flange

Figure B.34 Specimen W14×176-55: Column Fracture (10% Drift)
Figure B.35 Specimen W14×176-55: P-M Interaction

Figure B.36 Specimen W14×176-55: P-M Interaction (Target Points)
Figure B.37 Specimen W14×176-55: End Moment versus Drift Response

Figure B.38 Specimen W14×176-55: Compression Side Target Points End Moment versus Drift Response

IDC=0.0912

0.9M_{max}
(a) Longitudinal Force versus Lateral Force

(b) Longitudinal Force versus Column Axial Displacement

(c) Lateral Force versus Lateral Displacement

Figure B.39 Specimen W14×176-55: Force-Displacement Response
Figure B.40 Specimen W14×176-55: Strain Profiles Across Column Flange (Negative Drift)

Figure B.41 Specimen W14×176-55: Strain Profiles Along Column Length
(a) 2% Drift

(b) 4% Drift

(c) 6% Drift

Figure B.42 Specimen W14×176-75: Yielding Pattern
(d) 8% Drift

(e) 10% Drift

Figure B.42 Specimen W14×176-75: Yielding Pattern (cont.)
Figure B.43 Specimen W14×176-75: Flange Local Buckling (West End)
Figure B.44 Specimen W14×176-75: Overall Deformed Configuration
Figure B.44 Specimen W14×176-75: Overall Deformed Configuration (cont.)

(c) 8% Drift

(d) 10% Drift
Figure B.45 Specimen W14×176-75: Column Fracture (10% Drift)

(a) Overall View of Fracture Location

(b) View of Fracture from Exterior Side of Flange

(c) View of Fracture from Interior Side of Flange
Figure B.46 Specimen W14×176-75: P-M Interaction

Figure B.47 Specimen W14×176-75: P-M Interaction (Target Points)
Figure B.48 Specimen W14×176-75: End Moment versus Drift Response

Figure B.49 Specimen W14×176-75: Compression Side Target Points End Moment versus Drift Response
Figure B.50 Specimen W14×176-75: Force-Displacement Response
Figure B.51 Specimen W14×176-75: Strain Profiles Across Column Flange
(Negative Drift)

(a) Strain Gages S1, S2, and S3
(b) Strain Gages S4, S5, and S6

Figure B.52 Specimen W14×176-75: Strain Profiles Along Column Length

(a) Positive Drift
(b) Negative Drift
Figure B.53 Specimen W14×233-35: Yielding Pattern
(d) 8\% Drift

Figure B.53 Specimen W14×233-35: Yielding Pattern (cont.)
(a) 4% Drift

(b) 6% Drift

(c) 8% Drift

Figure B.54 Specimen W14×233-35: Flange Local Buckling (West End)
Figure B.55 Specimen W14×233-35: Overall Deformed Configuration

(a) 4% Drift

(b) 6% Drift

Tension Excursion

Compression Excursion
Figure B.55 Specimen W14×233-35: Overall Deformed Configuration (cont.)

(c) 8% Drift
Figure B.56 Specimen W14×233-35: Column to Base Plate Weld Fracture (9% Drift)
Figure B.57 Specimen W14×233-35: P-M Interaction

Figure B.58 Specimen W14×233-35: P-M Interaction (Target Points)
Figure B.59 Specimen W14×233-35: End Moment versus Drift Response

Figure B.60 Specimen W14×233-35: Compression Side Target Points End Moment versus Drift Response
(a) Longitudinal Force versus Lateral Force

(b) Longitudinal Force versus Column Axial Displacement

(c) Lateral Force versus Lateral Displacement

Figure B.61 Specimen W14×233-35: Force-Displacement Response
Figure B.62 Specimen W14×233-35: Strain Profiles Across Column Flange (Negative Drift)

Figure B.63 Specimen W14×233-35: Strain Profiles Along Column Length

(a) Strain Gages S1, S2, and S3
(b) Strain Gages S4, S5, and S6
Figure B.64 Specimen W14×233-55: Yielding Pattern
(d) 8% Drift

Figure B.64 Specimen W14×233-55: Yielding Pattern (cont.)
Figure B.65 Specimen W14×233-55: Flange Local Buckling (West End)
Figure B.66 Specimen W14×233-55: Overall Deformed Configuration

(a) 4% Drift

(b) 6% Drift
Figure B.66 Specimen W14×233-55: Overall Deformed Configuration (cont.)
Figure B.67 Specimen W14×233-55: *P-M* Interaction

Figure B.68 Specimen W14×233-55: *P-M* Interaction (Target Points)
Figure B.69 Specimen W14×233-55: End Moment versus Drift Response

Figure B.70 Specimen W14×233-55: Compression Side Target Points End Moment versus Drift Response
Figure B.71 Specimen W14×233-55: Force-Displacement Response
(a) Strain Gages S1, S2, and S3

(b) Strain Gages S4, S5, and S6

Figure B.72 Specimen W14×233-55: Strain Profiles Across Column Flange (Negative Drift)

(a) Positive Drift

(b) Negative Drift

Figure B.73 Specimen W14×233-55: Strain Profiles Along Column Length
Figure B.74 Specimen W14×370-35: Before Testing

East End

West End

Figure B.75 Specimen W14×370-35: Yielding Pattern at 2% Drift
(a) Overall View of Fracture Location

(b) View of Fracture from Exterior Side of Flange (with Haunches Removed)

(c) View of Fracture Location 1 (with Haunches Removed)

(d) View of Fracture Location 2 (with Haunches Removed)

Figure B.76 Specimen W14×370-35: Column to Base Plate Weld Fracture (3% Drift)
Figure B.77 Specimen W14×370-35: P-M Interaction

Figure B.78 Specimen W14×370-35: End Moment versus Drift Response
Figure B.79 Specimen W14×370-35: Force-Displacement Response
APPENDIX C. FINITE ELEMENT ANALYSIS RESULTS
C.1 Comparison of Predicted Response and Local Buckling

Figures C.1 to C.16 provide plots of $P$-$M$ interaction, end moment versus drift, constant-axial-load end moment versus drift and the predicted local buckling deformation at 5% drift (compression target axial load) for Models W12×87, W12×230, W14×132, W14×370, W18×86, W18×234, W24×131, and W24×279 at 35% and 75% $P_{yn}$ with a column length equal to 15 ft. Where response is not shown up to 10% drift, analysis was terminated due to severe local buckling and associated computational instability. Comparison of model results shows that for a given nominal column depth (i.e., W14), the amplitude of flange and web local buckling increased with increasing axial load. Also, the buckled amplitude increased with decreasing section weight. Comparison of model results for the typical column sections (W12 and W14) and deep column sections (W18 and W24) revealed more significant flange and web local buckling for the deep column sections. This flange and web local buckling and interaction of buckling modes caused more rapid strength degradation than observed for the stockier W12 and W14 sections, resulting in decreased interstory drift capacity ($IDC$).

C.2 Comparison of Predicted Response with Variation in Axial Load

Plots of $P$-$M$ interaction, end moment versus drift, and constant-axial-load end moment versus drift for all modeled sections with a column length equal to 15 ft are shown in Figures C.17 to C.32. Comparison of these figures reveals that: (1) for a given column section, $IDC$ decreased with increasing axial load; and (2) for a given
nominal column depth and axial load ratio, \( IDC \) increased with increasing section weight.
Figure C.1 Model W12×87-35: Predicted Response

(a) $P-M$ Interaction

(b) End Moment versus Drift

(c) Constant-Axial-Load End Moment versus Drift

(d) Local Buckling Deformation at 5% Drift
Figure C.2 Model W12×87-75: Predicted Response
Figure C.3 Model W12×230-35: Predicted Response
Figure C.4 Model W12×230-75: Predicted Response
(a) $P-M$ Interaction

(b) End Moment versus Drift

(c) Constant-Axial-Load End Moment versus Drift

(d) Local Buckling Deformation at 5% Drift

Figure C.5 Model W14×132-35: Predicted Response
Figure C.6 Model W14×132-75: Predicted Response

(a) $P-M$ Interaction

(b) End Moment versus Drift

(c) Constant-Axial-Load End Moment versus Drift

(d) Local Buckling Deformation at 5% Drift
(a) $P$-$M$ Interaction

(b) End Moment versus Drift

(c) Constant-Axial-Load End Moment versus Drift

(d) Local Buckling Deformation at 5% Drift

Figure C.7 Model W14×370-35: Predicted Response
(a) $P$-$M$ Interaction

(b) End Moment versus Drift

(c) Constant-Axial-Load End Moment versus Drift

(d) Local Buckling Deformation at 5\% Drift

Figure C.8 Model W14×370-75: Predicted Response
Figure C.9 Model W18×86-35: Predicted Response
(a) $P-M$ Interaction

(b) End Moment versus Drift

(c) Constant-Axial-Load End Moment versus Drift

(d) Local Buckling Deformation at 5% Drift

Figure C.10 Model W18×86-75: Predicted Response
(a) $P$-$M$ Interaction

(b) End Moment versus Drift

(c) Constant-Axial-Load End Moment versus Drift

(d) Local Buckling Deformation at 5% Drift

Figure C.11 Model W18×234-35: Predicted Response
Figure C.12 Model W18×234-75: Predicted Response

(a) \( P-M \) Interaction

(b) End Moment versus Drift

(c) Constant-Axial-Load End Moment versus Drift

(d) Local Buckling Deformation at 5% Drift
Figure C.13 Model W24×131-35: Predicted Response

(a) $P-M$ Interaction

(b) End Moment versus Drift

(c) Constant-Axial-Load End Moment versus Drift

(d) Local Buckling Deformation at 5% Drift

Figure C.13 Model W24×131-35: Predicted Response
Figure C.14 Model W24×131-75: Predicted Response
(a) $P-M$ Interaction

(b) End Moment versus Drift

(c) Constant-Axial-Load End Moment versus Drift

(d) Local Buckling Deformation at 5% Drift

Figure C.15 Model W24×279-35: Predicted Response
Figure C.16 Model W24×279-75: Predicted Response

(a) P-M Interaction

(b) End Moment versus Drift

(c) Constant-Axial-Load End Moment versus Drift

(d) Local Buckling Deformation at 5% Drift
Figure C.17 Model W12×87: Global Response
Figure C.17 Model W12×87: Global Response (cont.)

(c) 35% $P_{yn}$

(d) 55% $P_{yn}$
Figure C.17 Model W12×87: Global Response (cont.)
Figure C.18 Model W12×106: Global Response

(a) 10% $P_{yn}$

(b) 20% $P_{yn}$
Figure C.18 Model W12×106: Global Response (cont.)

(c) 35% $P_y$  
(d) 55% $P_y$
Figure C.18 Model W12×106: Global Response (cont.)
Figure C.19 Model W12×152: Global Response
Figure C.19 Model W12×152: Global Response (cont.)
(e) 75% $P_{yn}$

(f) 90% $P_{yn}$

Figure C.19 Model W12×152: Global Response (cont.)
Figure C.20 Model W12×230: Global Response
Figure C.20 Model W12×230: Global Response (cont.)

(c) 35% $P_{yn}$

(d) 55% $P_{yn}$
Figure C.20 Model W12×230: Global Response (cont.)

(e) 75% $P_{yn}$

(f) 90% $P_{yn}$
Figure C.21 Model W14×132: Global Response

(a) 10% $P_{yn}$

(b) 20% $P_{yn}$
Figure C.21 Model W14×132: Global Response (cont.)

(c) 35% $P_{yn}$

(d) 55% $P_{yn}$
Figure C.21 Model W14×132: Global Response (cont.)

(e) 75% $P_{yn}$  

(f) 90% $P_{yn}$
Figure C.22 Model W14×176: Global Response
Figure C.22 Model W14×176: Global Response (cont.)

(c) 35% $P_{yn}$

(d) 55% $P_{yn}$
Figure C.22 Model W14×176: Global Response (cont.)
Figure C.23 Model W14×233: Global Response
Figure C.23 Model W14×233: Global Response (cont.)

(c) 35% $P_{yn}$

(d) 55% $P_{yn}$
Figure C.23 Model W14×233: Global Response (cont.)

(e) 75% $P_{yn}$

(f) 90% $P_{yn}$
Figure C.24 Model W14×370: Global Response
Figure C.24 Model W14×370: Global Response (cont.)

\[
\text{(c) 35\% } P_{yn}
\]

\[
\text{(d) 55\% } P_{yn}
\]
Figure C.24 Model W14×370: Global Response (cont.)

(e) 75% $P_{yn}$

(f) 90% $P_{yn}$
Figure C.25 Model W18×86: Global Response
Figure C.25 Model W18×86: Global Response (cont.)

(c) 55% $P_{yn}$

(d) 75% $P_{yn}$
Figure C.26 Model W18×119: Global Response
(c) 55% $P_{yn}$

(d) 75% $P_{yn}$

Figure C.26 Model W18×119: Global Response (cont.)
Figure C.27 Model W18×158: Global Response
Figure C.27 Model W18×158: Global Response (cont.)

(c) 55% $P_{yn}$

(d) 75% $P_{yn}$
Figure C.28 Model W18×234: Global Response

(a) 20% $P_{yn}$

(b) 35% $P_{yn}$
Figure C.28 Model W18×234: Global Response (cont.)

(c) 55% $P_{yn}$

(d) 75% $P_{yn}$
Figure C.29 Model W24×131: Global Response
Figure C.29 Model W24×131: Global Response (cont.)
Figure C.30 Model W24×162: Global Response
Figure C.30 Model W24×162: Global Response (cont.)
Figure C.31 Model W24×207: Global Response
Figure C.31 Model W24x207: Global Response (cont.)
Figure C.32 Model W24×279: Global Response

(a) 20% $P_{yn}$

(b) 35% $P_{yn}$
Figure C.32 Model W24×279: Global Response (cont.)

(c) 55% $P_{yn}$

(d) 75% $P_{yn}$
REFERENCES


