PEER Structural Performance Database

User's Manual

(Version 1.2)

Michael Berry

University of Washington

Myles Parrish

University of Washington

Marc Eberhard

University of Washington

Pacific Earthquake Engineering Research Center

University of California, Berkeley

January 2004

(Rev. July 2013)

Table of Contents

LIST OF FIGURES ii

LIST OF TABLES iii

ACKNOWLEDGEMENTS iv

CHAPTER 1: INTRODUCTION 1

CHAPTER 2: COLUMN PROPERTIES 3

2.1 MATERIAL PROPERTIES 3
2.2 COLUMN GEOMETRY 4
2.3 CONFINEMENT DETAILS 6
2.4 TEST CONFIGURATION 6

CHAPTER 3: TEST RESULTS 9

3.1 FAILURE CLASSIFICATION 9
3.2 FORCE-DISPLACEMENT DATA 10
3.3 EFFECT OF AXIAL LOAD 11
3.4 OBSERVED DAMAGE 14

CHAPTER 4: CHARACTERISTICS OF AVAILABLE DATA 16

4.1 DISTRIBUTION OF KEY COLUMN PROPERTIES 16
4.2 CALCULATED ACI NOMINAL FLEXURAL CAPACITY 20

REFERENCES 21

APPENDIX A: SUMMARY OF TESTS OF RECTANGULAR-REINFORCED COLUMNS 22

APPENDIX B: SUMMARY OF TESTS OF SPIRAL-REINFORCED COLUMNS 27

APPENDIX C: STRUCTURE OF XML DATA 30

APPENDIX D: COLUMN TEST REFERENCES 33

List of Figures

Figure 2.1: Confinement Types ... 7
Figure 2.2: Column Test Configurations ... 8
Figure 3.1: Failure Classification Flowchart ... 10
Figure 3.3: Definition of Displacement Preceding Damage State ... 15
Figure 4.1: Distribution of Column Depth ... 17
Figure 4.2: Distribution of Column Aspect Ratio ... 18
Figure 4.3: Distribution of Axial-Load Ratio ... 18
Figure 4.4: Distribution of Longitudinal-Reinforcement Ratio ... 19
Figure 4.5: Distribution of Transverse-Reinforcement Ratio ... 20

List of Tables

Table 2.1: Material Properties.. 4
Table 2.2: Column Geometry 5
Table 2.3: Cross-Section Classifications 5
Table 2.4: Confinement Details 6
Table 2.5: Confinement Details 7
Table 3.1: Failure Mode Codes.. 9
Table 4.1: Column Property Statistics 17
Table 4.2: Summary of Calculated Flexural Capacities 20
Table A.1: Summary of Tests of Rectangular-Reinforced Columns............................... 23
Table B.1: Summary of Tests of Spiral-Reinforced Columns.. 27

Acknowledgements

The database described in this report builds on the work of Dr. Andrew Taylor, Dr. William Stone and other researchers at the National Institute of Standards and Technology (NIST) (Taylor and Stone, 1993, Taylor et al., 1997). The data provided by these researchers formed the core of this database.

As part of their MSCE thesis research at the University of Washington, Amit Mookerjee (1999), Myles Parrish (2001), Haili Camarillo (2003) and Michael Berry (2003) expanded that database and developed the University of Washington website (http://www.ce.washington.edu/~peera1/). The authors greatly appreciate the contributions of Debra Bartling. Working under the supervision of Professor Jack Moehle, Ms. Bartling created the searchable website located at http://nisee.berkeley.edu/spd/.

It would have been impossible to assemble this database without the generous assistance of numerous researchers who donated their time and data. Appendix D of this report contains a list of the test references. These references provide many details that are not included in the column database, and wherever possible, they should be cited directly, in addition to this report.

Support of this work was provided primarily by the Earthquake Engineering Research Centers Program of the National Science Foundation, under Award Number EEC-9701568 through the Pacific Earthquake Engineering Research Center (PEER).

Chapter 1: Introduction

The PEER Structural Performance Database has been assembled to provide researchers with the data needed to evaluate and develop seismic performance models for reinforced concrete columns. This database builds on previous work at the National Institute of Standards and Technology (NIST). The original NIST database described 107 tests of rectangular-reinforced columns and 92 tests of spiral-reinforced concrete columns. For each test, the NIST database provided a reference, digital top force-displacement histories, key material properties, as well as a description of the test geometry. The data was available from two reports and accompanying floppy discs (Taylor and Stone 1993; Taylor et al. 1997).

With the support of the Pacific Earthquake Engineering Research Center (PEER), University of Washington researchers added new tests to the database and expanded the information available for each test. As of January 2004, the database described 274 tests of rectangular-reinforced columns and 160 tests of spiral-reinforced columns. The database now provides additional details of the tests, including the P-Δ configuration and the maximum column deflection imposed before reaching various damage states. The database is available on the World Wide Web from the University of Washington (*http://www.ce.washington.edu/~peera1*) and from PEER (*http://nisee.berkeley.edu/spd/*). The PEER website allows users to search for column tests with particular ranges of attributes. In a few cases, the PEER website provides additional information not available at the UW website, such as drawings and photographs. As of September 2008, The University of Washington website contains 1306 tests of rectangular-reinforced columns and 183 tests of spiral-reinforced columns

This report documents the database. Chapter 2 provides the definitions used to describe the column material properties, geometry and reinforcing details, as well as the test configuration. Chapter 3 documents the reporting of the test results, including the failure classification, force-deflection histories, axial loads and observed damage. Chapter 4 provides a list of the tests in the database, statistical distributions of key column characteristics, and maximum recorded moments and shears. This chapter also provides statistics on the nominal flexural moment capacities, as calculated by the procedures of the American Concrete Institute's Building Code Requirements for Structural Concrete (ACI 318-02).

Chapter 2: Column Properties

Key test properties are available from the UW website in Lotus .wk1 format. The same properties are available from the PEER website in tab-deliminated and xml formats. The column properties provided in the database are defined in this chapter. The properties are organized in terms of material properties (Section 2.1), column geometry (Section 2.2), confinement details (Section 2.3) and test configuration (Section 2.4). For a few tests, the PEER website also provides key drawings and pictures.

2.1 Material Properties

The material properties provided for each column test in the database are listed in Table 2.1. This table includes the notation used to designate the column properties. The column titled *Column Type* in this table designates whether each property is provided only for the rectangular-reinforced columns (R), only for the spiral-reinforced columns (S) or for both types of columns (R, S).

Table 2.1: Material Properties

Material	Notation	Description of Property	Column Type
Concrete	f'c	Characteristic compressive strength of concrete (MPa)	R, S
Longitudinal Reinforcement	fyl	Yield stress of longitudinal reinforcement (MPa)	S
	fsu long.	Ultimate steel strength for longitudinal reinforcement (MPa)	S
	fyl Corner	Yield stress of longitudinal corner bars (MPa)	R
	fyl Inerm.	Yield stress of longitudinal intermediate bars (MPa)	R
	fsu Corner	Ultimate steel strength of longitudinal corner bars (MPa)	R
	fsu Interm.	Ultimate steel strength of longitudinal intermediate bars (MPa)	R
Transverse Reinforcement	fyt	Yield stress of transverse reinforcement (MPa)	R, S
	fsu trans.	Ultimate steel strength for transverse reinforcement(MPa)	R, S

2.2 Column Geometry

The column database describes important geometric properties of each column. These geometric properties and the corresponding notation are listed in Table 2.2. All rectangular-reinforced columns had rectangular cross-sections, but the spiral-reinforced columns had three cross-section shapes (octagonal, circular and square). These shapes were assigned the codes listed in Table 2.3.

Table 2.2: Column Geometry

	Notation	Description	Column Type
Overall Column Dimensions	H or D	Column Depth (mm)	R, S
	B	Column Width (mm)	R
	Area (A_g)	Cross-sectional area of column (mm^2)	R, S
	L	Length of equivalent cantilever (mm)	R, S
Longitudinal Reinforcement	Total # Bars	Number of longitudinal reinforcing bars	R, S
	Bar Dia.	Diameter of longitudinal reinforcement bars (mm)	S
	Bar Dia. Corner	Diameter of longitudinal corner bars (mm)	R
	Bar Dia. Interm.	Diameter of longitudinal intermediate bars (mm)	R
	Lsplice	Length of longitudinal reinforcement splice	R, S
	Reinf. Ratio	Longitudinal reinforcement ratio (calculated).	R, S
Transverse Reinforcement	Bar Dia.	Diameter of transverse reinforcement (mm)	R, S
	Spacing	Spacing of transverse reinforcement (mm)	R, S
	Vol. Trans	Volumetric transverse reinforcement ratio (reported)	R, S
	Nv	Number of transverse shear bars in cross section	R, S
	Clear Cover (Rect)	Distance from outer surface of column to outer edge of transverse reinforcement (mm)	R
	Clear Cover (Spiral)	Distance from outer surface of column to center of transverse reinforcement (mm)	S

Table 2.3: Cross-Section Classifications

Notation	Cross-Section Shape	Code
O	Octagonal	2
C	Circular	0
S	Square	3

2.3 Confinement Details

The configurations of the lateral-reinforcement in the rectangular columns were categorized into nine classifications. The nine classifications are defined in Table 2.3 and illustrated in Figure 2.1.

Table 2.4: Confinement Details

Notation	Description	Confinement Code
I	Interlocking ties	1
R	Rectangular ties (around perimeter)	2
RI	Rectangular and Interlocking ties	4
RU	Rectangular ties and U-bars	8
RJ	Rectangular ties with J-hooks	6
RD	Rectangular and Diagonal ties	3
RO	Rectangular and Octagonal ties	7
RIJ	Rectangular and Interlocking ties, with J-hooks	5
UJ	U-bars with J-hooks	9

2.4 Test Configuration

To compare column behavior consistently for a wide range of testing configurations, the test configurations and force-deflection data were reduced to the case of an equivalent cantilever column (Fig. 2.2a). Test configurations considered in the column database included cantilever (Fig. 2.2a), double-curvature (Fig. 2.2b), double-ended (Fig. 2.2c), hammerhead (Fig. 2.2d), and flexible-base (Fig. 2.2e). The corresponding configuration codes are provided in Table 2.5.

The definition of the equivalent cantilever length, L, for each column configuration is provided in Fig. 2.2. For each configuration, L_{meas} was defined as the distance from the elevation at which lateral column displacements were measured to the column base. For the majority of column tests, L_{meas} was equal to L. In other words, the top displacement was measured at the elevation at which the lateral force was applied.

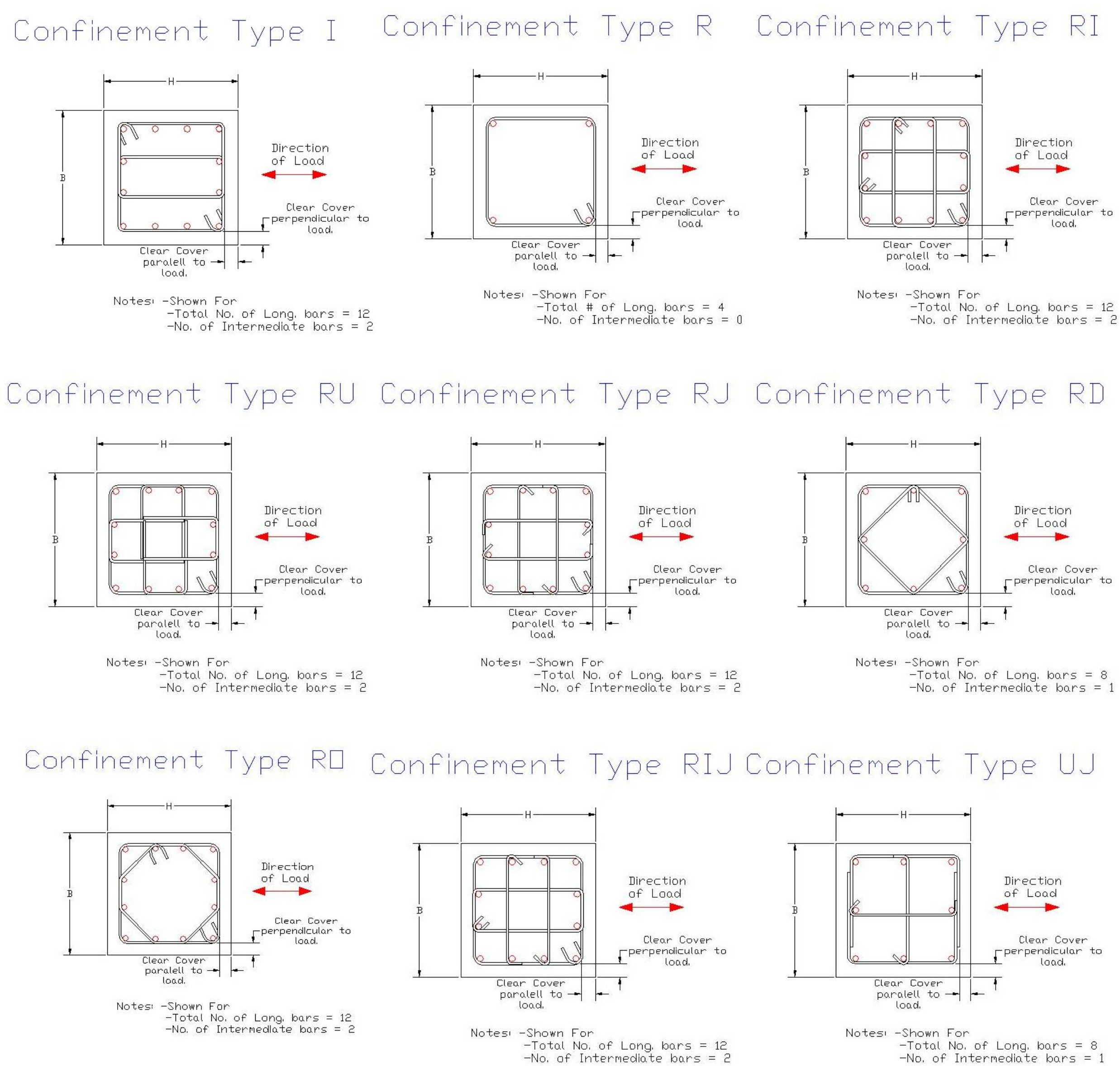

Figure 2.1: Confinement Types

Table 2.5: Confinement Details

Description	Confinement Code
Cantilever	C
Double-Curvature	DC
Double-Ended	DE
Flexible Base	CFB
Hammerhead	HH

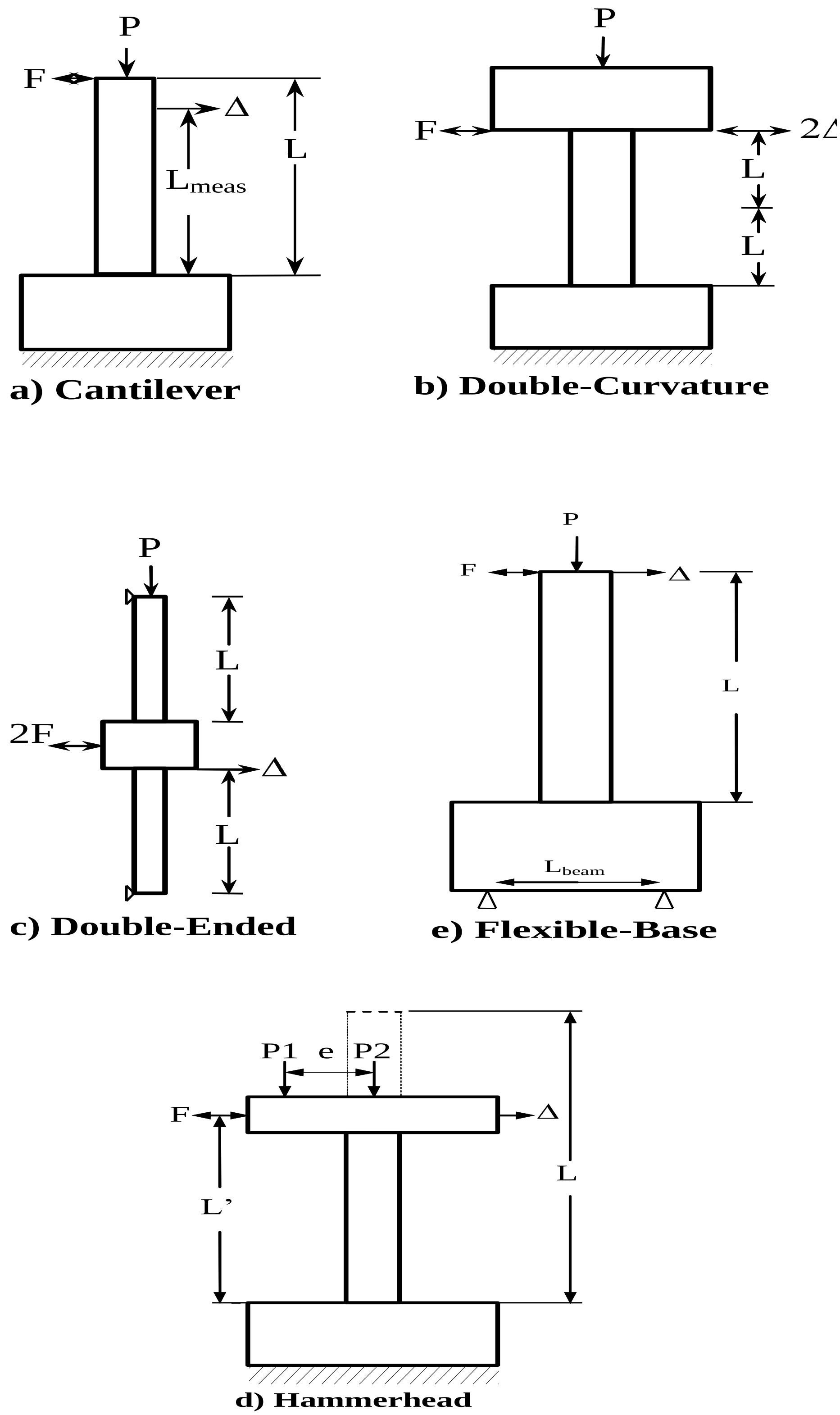

Figure 2.2: Column Test Configurations

Chapter 3: Test Results

3.1 Failure Classification

The nominal column failure mode was classified as flexure-critical, flexure-shear-critical, or shear-critical, according to the following criteria (see Figure 2.5). If no shear damage was reported by the experimenter, the column was classified as flexure-critical. If shear damage was reported, the absolute maximum effective force (F_{eff}), was compared with the calculated force corresponding to a maximum strain of 0.004 ($F_{0.004}$). The failure displacement ductility at the 80% effective force, μ_{fail}, was also considered. If the maximum effective force was less than 95% of the ideal force ($F_{eff} < 0.95*F_{0.004}$) or if the failure displacement ductility was less than or equal to 2 ($\mu_{fail} \leq 2$), the column was classified as shear-critical. Otherwise, the column was classified as flexure-shear-critical.

Table 3.1: Failure Mode Codes

Failure	Code
Flexure	1
Shear	2
Flexure-Shear	3

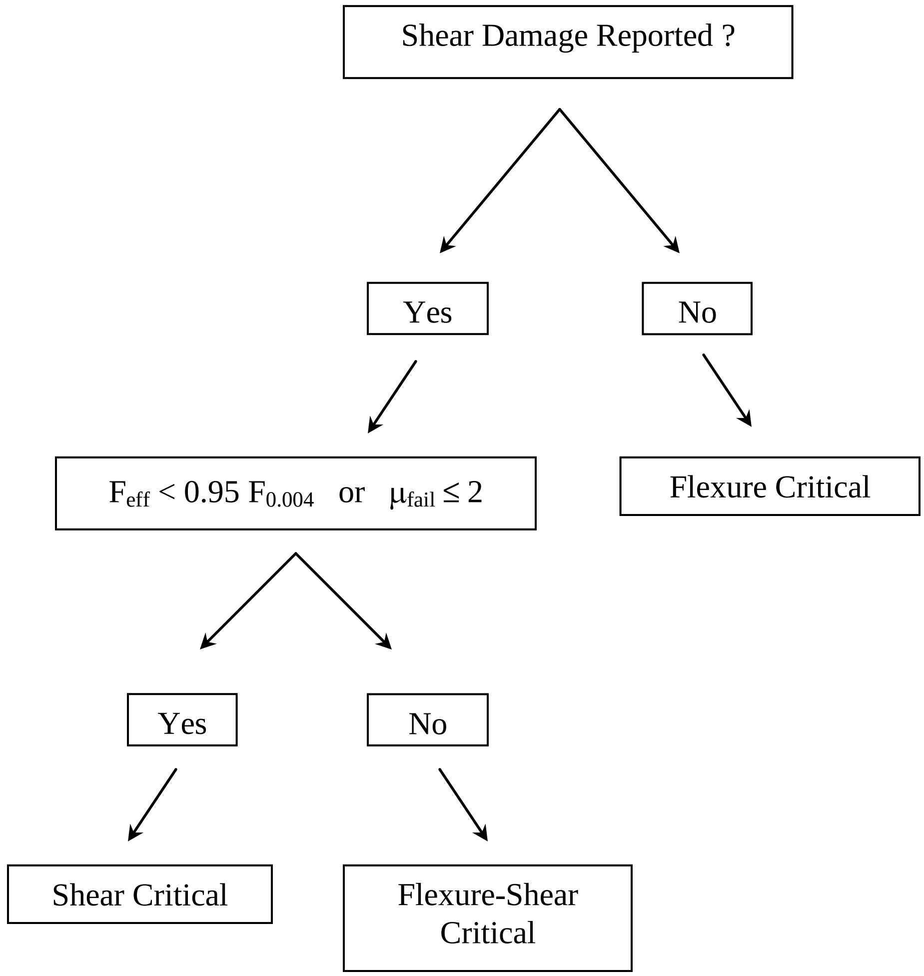

Figure 3.1: Failure Classification Flowchart

3.2 Force-Displacement Data

The force-deflection histories provided by the test researchers were modified as little as possible. Where necessary, units were converted to kN and mm, and depending on the test configuration, factors of 1/2 were introduced into the force or deflection history so that all columns could be treated as cantilevers (Section 2.4). In some cases, the histories provided were the histories of actuator force, and in some cases, the original researchers modified the histories to reflect P-Δ effects. These effects may be significant, particularly for tests with high axial loads and large displacements.

The database provides force-displacement histories in tab-delimited (.txt) formats, which can be imported into many applications. The first row of each lateral force-displacement file contains the test name. The number of force-displacement data points is noted in the second row. Subsequent rows contain top-displacement values (mm) in the first column, lateral-load values (kN) in the second column, and where available, axial-load values in the third column (kN). All transverse force-displacement histories are reported in terms of an equivalent cantilever column, regardless of the test configuration (Section 2.4).

3.3 Effect of Axial Load

To account for P-Δ effects, column forces provided in the database need to be resolved into their vertical and horizontal components. The vertical component can be approximated as P, the axial load provided in the database. The horizontal component of the vertical actuator needs to be added to (or subtracted from) the force applied by the horizontal actuator to obtain the net horizontal force.

To allow researchers to take into account P-Δ effects, the database identifies four types of lateral force-displacement histories (illustrated in Figure 3.2):

- **Type I:** Force-deflection data provided by the researcher was in the form of effective force (F_{eff}) versus deflection (Δ) at L_{meas}. In this case, the net horizontal force (F_H) can be determined according to the following equation:

$$F_H = F_{eff} - P \Delta / L_{meas} \qquad \text{Equation 3.1}$$

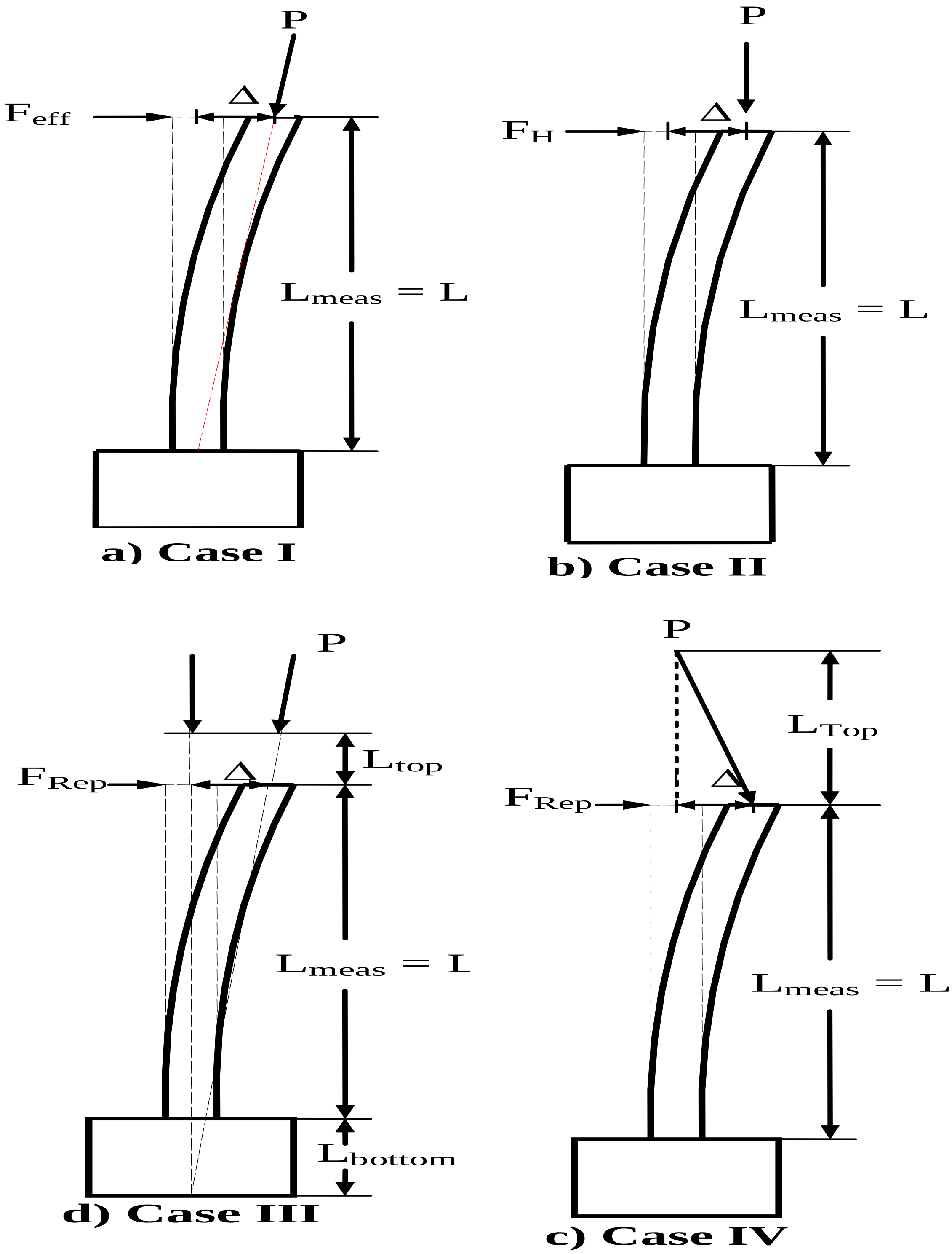

Figure 3.2: P-D Correction

- **Type II:** Force-deflection data was provided by the researcher in the form of net horizontal force (F_H) versus deflection (Δ) at L_{meas}.

$$F_H = F_{Rep} \qquad \text{Equation 3.2}$$

- **Type III:** Force data provided by the researcher represents the lateral load applied by the horizontal actuator. However, the axial load is not applied at the same elevation as the lateral force, or the line of action of the axial load does not pass through the column base. In this case, the horizontal component (P_H) of the vertical load actuator was subtracted from the reported force, F_{Rep}, to get the net horizontal force (F_H).

$$\alpha = \tan^{-1}\left[\frac{\Delta\left(\frac{L+L_{top}}{L}\right)}{L+L_{bot}+L_{top}}\right] \qquad \text{Equation 3.3}$$

$$P_H = P \cdot \sin\alpha \qquad \text{Equation 3.4}$$

$$F_H = F_{\text{Re}\,p} - P_H \qquad \text{Equation 3.5}$$

- **Type IV:** Force data provided by the researcher represents the lateral load applied by the horizontal actuator, but the top of the vertical actuator does not translate. In this case, the horizontal component of the vertical load actuator needs to be added to the reported force, F_{Rep}, to get the net horizontal force (F_H).

$$F_H = F_{Rep} + P\,\Delta / L_{Top} \qquad \text{Equation 3.6}$$

For all load configurations, the contributions of the net horizontal force and the gravity (vertical) load to the total base moment can then be determined as follows:

$$M_{base} = F_H \cdot L + P \cdot \Delta \cdot \left(\frac{L_{top}+L}{L_{meas}}\right) \qquad \text{Equation 3.3}$$

F_H : net horizontal force (Column Shear)

L : shear span length

P : gravity (vertical) load

Δ : measured displacement at cantilever elevation L_{meas}

L_{top} : distance from elevation at which lateral force was applied to elevation at which gravity (vertical) load is applied.

L_{meas} : elevation at which lateral column displacement was measured

The effective force can then be defined as:

$$F_{eff} = M_{base} / L \qquad \text{Equation 3.4}$$

3.4 Observed Damage

The maximum recorded column deflections prior to observing a particular level of damage, Δ_{Damage}, (illustrated in Figure 3.3) are provided for column tests in which the deflection was documented in the research reports.

The damage deformations, Δ_{Damage}, are provided for the seven damage states defined below. Not all damage levels were reported for each test.

- Onset of spalling, defined as the first observation of spalling.
- Onset of significant spalling, defined by the reported observation of “significant spalling” or “considerable spalling.” Alternatively, if spall heights could be determined, significant spalling was defined as a spall height equal to at least 10% of the cross-section depth.
- Onset of bar buckling, defined as the observation of the first sign of longitudinal bar buckling.
- Longitudinal bar fracture, defined as the observation of the first sign of a longitudinal bar fracturing.

- Transverse reinforcement fracture, defined as the observation of the first sign of the transverse reinforcement fracturing, or becoming untied.
- Loss of axial-load capacity, defined as the observation of loss of axial-load carrying capacity of the column.
- Column failure (reported for 49 tests), defined for the purpose of this database, as the first occurrence of one of the following events: buckling of a longitudinal bar, fracture of transverse reinforcement, fracture of a longitudinal bar, or loss of axial-load capacity.

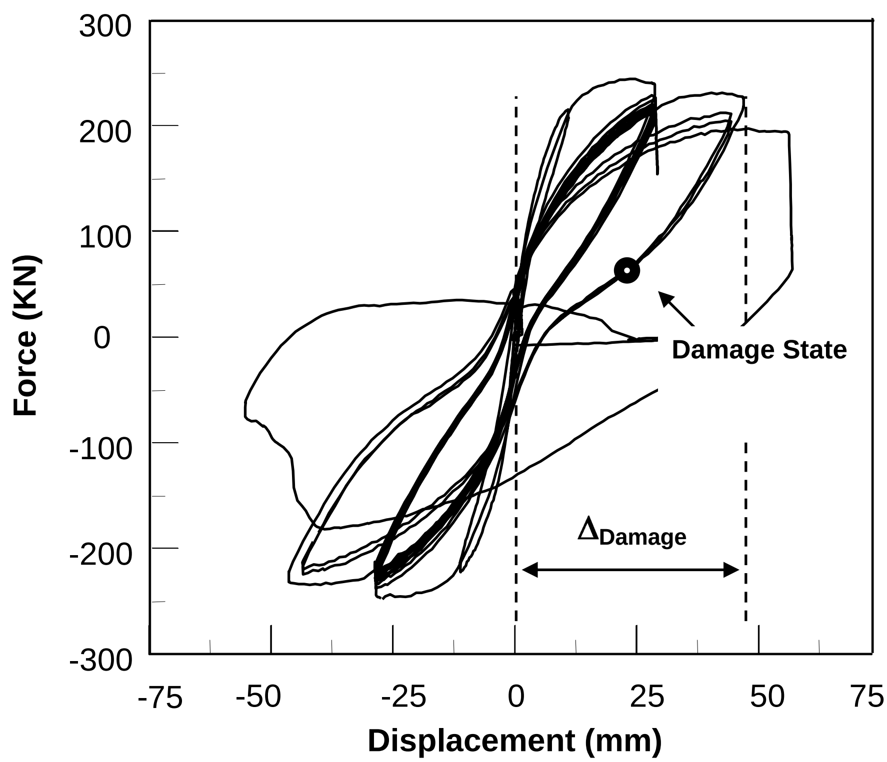

Figure 3.2: Definition of Displacement Preceding Damage State

Chapter 4: Characteristics of Available Data

The data available in the PEER structural performance database is summarized in this chapter. The distributions of key column properties (depth, aspect ratio, axial-load ratio, longitudinal reinforcement ratio and transverse reinforcement ratio) are examined for both rectangular-reinforced and spiral-reinforced columns. In addition, the chapter provides a list of the 404 tests included in the database (as of January 2004), along with comments and key test results (e.g., maximum moment and shear resisted by the column). The chapter also reports the nominal moment capacity of the columns.

The tests included in the PEER database are listed in Appendices A and B. The appendices also include (for each column test) comments about the data, the maximum moment and shear resisted by the column, the ratio of measured maximum moment to the nominal ACI moment (ACI 318-02), the ideal yield displacement as described by Berry (2003), and the failure classification (Chapter 3.1). References for each column test are provided in Appendix D.

4.1 Distribution of Key Column Properties

Table 4.1 provides the means and coefficients of variations (CoV) of key column properties for 274 rectangular-reinforced columns and 160 spiral-reinforced columns. Statistics are provided for the column depth, aspect ratio, axial-load ratio, longitudinal reinforcement ratio (ρ_l) and transverse reinforcement ratio (ρ_s).

Table 4.1: Column Property Statistics

	Rectangular-Reinforced (274 tests)			Spiral-Reinforced (160 tests)		
Column Property	Mean	Std	CoV	Mean	Std	CoV
Depth (mm)	319	117	0.37	399	174	0.44
Aspect Ratio	3.58	1.46	0.41	3.44	2.01	0.59
Axial-Load Ratio	0.27	0.19	0.70	0.14	0.14	1.01
ρ_l (%)	2.39	0.96	0.40	2.66	1.03	0.39
ρ_s (%)	2.01	1.22	0.61	1.00	0.74	0.74

The distribution of column depth is illustrated in Figure 4.1 The rectangular-reinforced data is approximately normally distributed about its mean value of 319 mm. Approximately 80% of the rectangular-reinforced columns had a depth between 200 and 500 mm. The spiral-reinforced data does not have a normal distribution.

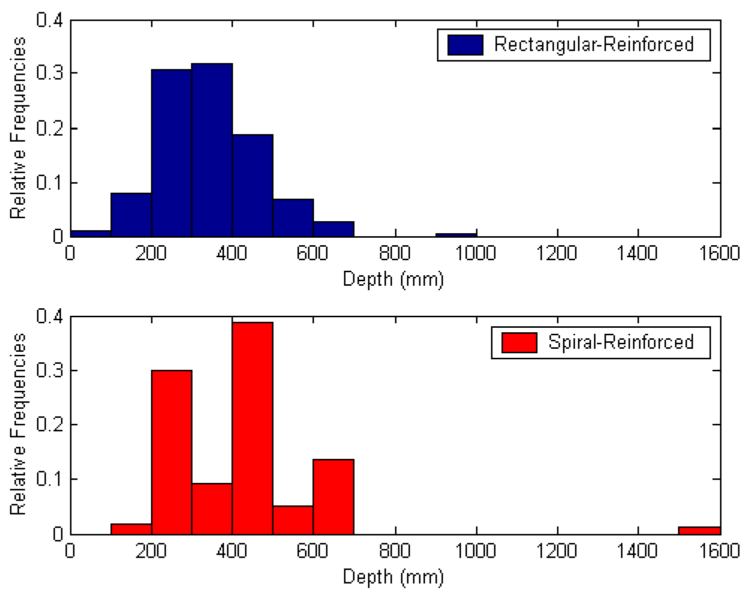

Figure 4.1: Distribution of Column Depth

The distributions of the column aspect ratio are illustrated in Figure 4.2. The rectangular-reinforced data was approximately normally distributed about its mean value of 3.6 with a skew towards the lower aspect ratios. The spiral-reinforced data was weighted toward the lower aspect ratios, with 49% of the spiral-reinforced columns having an aspect ratio between 1 and 3.

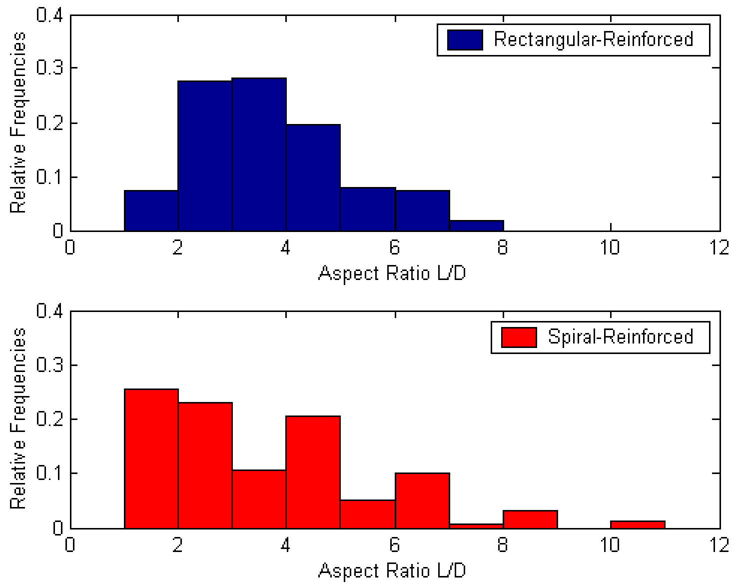

Figure 4.2: Distribution of Column Aspect Ratio

The distributions of the axial-load ratio are illustrated in Figure 4.3. Both the rectangular-reinforced and spiral-reinforced columns had distributions weighted towards the lower axial-loads ratios. In particular 65% of the rectangular-reinforced and 85% of the spiral-reinforced columns had an axial load between 0 and 0.3.

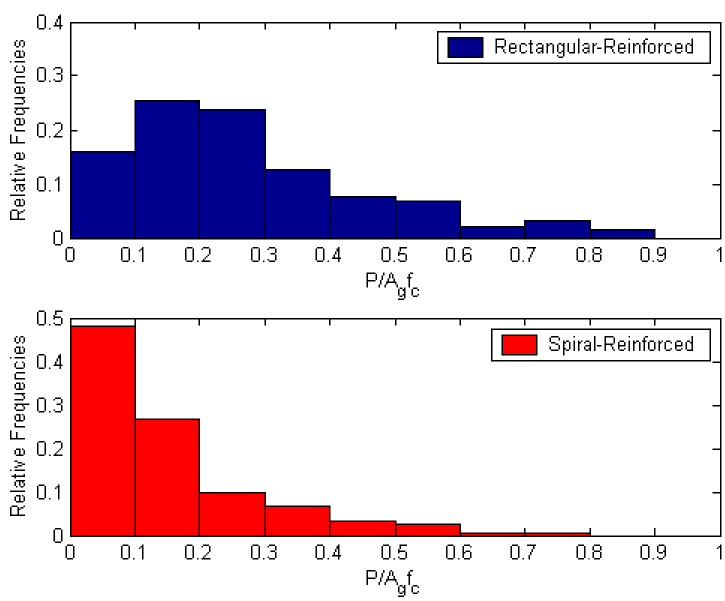

Figure 4.3: Distribution of Axial-Load Ratio

The distributions of the longitudinal-reinforcement ratio are shown in Figure 4.4. The rectangular-reinforced data was approximately normally distributed about its mean value of 2.39%, with a skew toward the lower reinforcement ratios. The spiral-reinforced data was not distributed normally.

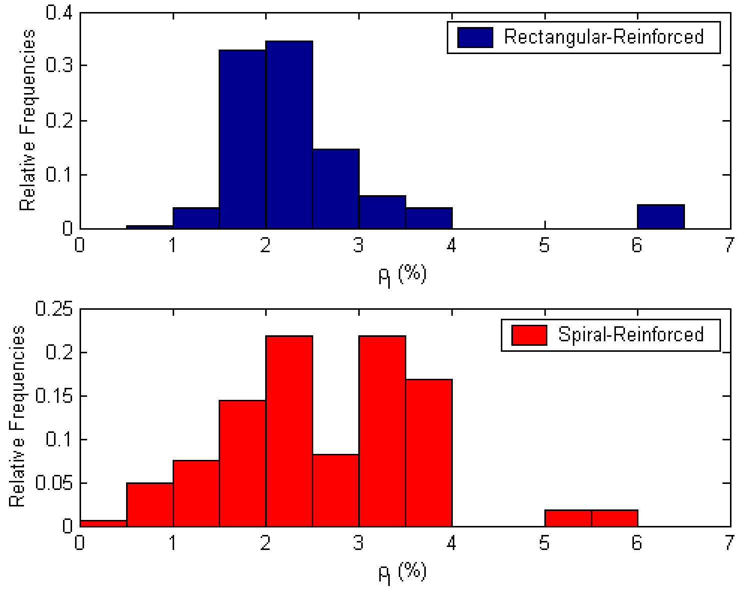

Figure 4.4: Distribution of Longitudinal-Reinforcement Ratio

The distributions of transverse reinforcement ratio are presented in Figure 4.5. The rectangular-reinforced data is weighted around its mean value of 2%, but cannot be characterized easily by a distribution. In comparison with the rectangular columns, the spiral-reinforced data tends to have low transverse reinforcement ratios. Nearly 50% of the spiral-reinforced columns had a transverse reinforcement ratio between 0.5% and 1.0%.

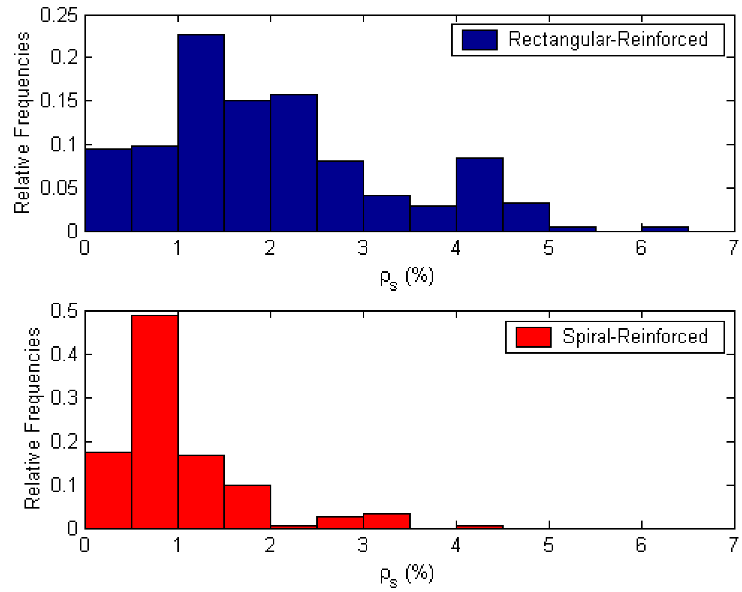

Figure 4.5: Distribution of Transverse-Reinforcement Ratio

4.2 Calculated ACI Nominal Flexural Capacity

To provide an example of the use of the database and to help interpret the column data, the nominal flexural capacity (ACI 2002) was calculated for each column in the database. The calculated moment capacities are provided in Tables A.1 and B.1. In addition, the mean and coefficient variation of the ratio of measured maximum moment to nominal ACI flexural capacity are provided in Table 4.2. Berry and Eberhard (2004) provides other examples of how the PEER database can be used to evaluate and develop performance models.

Table 4.2: Summary of Calculated Flexural Capacities

	Failure Mode	# of Tests	M_{max}/M_{ACI}	
			Mean	Cov
Rectangular-Reinforced	Flexure	214	1.19	0.15
	Shear	10	0.85	0.24
	Flexure-Shear	44	1.25	0.28
Spiral-Reinforced	Flexure	87	1.25	0.12
	Shear	26	0.81	0.20
	Flexure-Shear	36	1.17	0.12

References

American Concrete Institute (ACI 318-02), "Building Code Requirements for Structural Concrete", 2002.

Berry, M. P., and Eberhard, M. O. (2004). "A Practical Performance Model for Bar Buckling." *J. Struct. Eng.*, under review.

Berry, M. P., and Eberhard, M. O. (2003). "Performance Models for Flexural Damage in Reinforced Concrete Columns." *Pacific Earthquake Engineering Research Center Report 2003/??*, University of California, Berkeley, California.

Camarillo, H. (2003). "Evaluation of Shear Strength Methodologies for Reinforced Concrete Columns." Master's Thesis, Dept. of Civil and Environmental Engineering, University of Washington, Seattle.

Mookerjee, A. (1999). "Reliability of Performance Estimates of Spiral and Hoop-Reinforced Concrete Columns." Master's Thesis, Dept. of Civil and Environmental Engineering, University of Washington, Seattle.

Parrish, M. (2001). "Accuracy of Seismic Performance Methodologies for Rectangular Reinforced Concrete Columns." Master's Thesis, Dept. of Civil and Environmental Engineering, University of Washington, Seattle.

Taylor, A.W., Kuo, C., Wellenius, K. and Chung, D. (1997). A Summary of Cyclic Lateral-Load Tests on Rectangular Reinforced Concrete Columns, National Institute of Standards and Technology, Report NISTIR 5984.

Taylor, A.W. and Stone, W.C. (1993). A Summary of Cyclic Lateral-Load Tests of Spiral Reinforced Concrete Columns, National Institute of Standards and Technology, Report NISTIR 5285.

Appendix A: Summary of Tests of Rectangular-Reinforced Columns

This appendix provides a brief summary of the reinforced concrete column tests described by the PEER Structural Performance Database. The maximum column moment (M_{max}) listed in Tables A.1 and B.1, was computed from the test data, including P-Δ effects. The ACI nominal flexural capacity (M_{ACI}) was calculated following the provisions of ACI's Building Code Requirements for Structural Concrete (ACI 318-02). The nominal displacement at yield was computed following the procedure described by Berry and Eberhard (2003). The failure mode was defined in Chapter 3.1 (Table 3.1).

Table A.1: Summary of Tests of Rectangular-Reinforced Columns

Test Number	Reference	Column Designation	Comments	M_{MAX} (kN-m)	M_{MAX} / M_{ACI}	V_{MAX} (kN)	Δ_y (mm)	Failure Mode
1	Gill et al. (1979)	No. 1		838	1.22	657	7.12	1
2	Gill et al. (1979)	No. 2		953	1.05	764	7.13	1
3	Gill et al. (1979)	No. 3		817	1.20	642	4.85	1
4	Gill et al. (1979)	No. 4		903	1.44	697	4.09	1
5	Ghee et al. (1981)	No. 3	bar spacing 120mm-75mm-120mm	337	1.24	192	9.61	1
6	Ghee et al. (1981)	No. 4	bar spacing 120mm-75mm-120mm	298	1.14	169	12.19	1
7	Soesian. et al. (1986)	No. 1		354	1.17	200	10.36	1
8	Soesian. et al. (1986)	No. 2		481	1.18	279	9.16	1
9	Soesian. et al. (1986)	No. 3		474	1.17	277	8.77	1
10	Soesian. et al. (1986)	No. 4		457	1.19	265	9.59	1
11	Zahn et al. (1986)	No. 7		398	1.35	213	11.88	1
12	Zahn et al. (1986)	No. 8		554	1.44	269	10.27	1
13	Watson and Park (1989)	No. 5		535	1.42	292	8.09	1
14	Watson and Park (1989)	No. 6		528	1.41	295	6.19	1
15	Watson and Park (1989)	No. 7		525	1.75	293	4.02	1
16	Watson and Park (1989)	No. 8		523	1.81	295	4.16	1
17	Watson and Park (1989)	No. 9		602	2.08	310	4.76	1
18	Tanaka and Park (1990)	No. 1	Transverse reinforcement hoops were welded	290	1.14	167	13.81	1
19	Tanaka and Park (1990)	No. 2		291	1.14	168	13.04	1
20	Tanaka and Park (1990)	No. 3		302	1.18	175	11.37	1
21	Tanaka and Park (1990)	No. 4		292	1.15	170	12.35	1
22	Tanaka and Park (1990)	No. 5		704	1.15	386	13.55	1
23	Tanaka and Park (1990)	No. 6		717	1.17	409	11.96	1
24	Tanaka and Park (1990)	No. 7		1070	1.34	588	9.68	1
25	Tanaka and Park (1990)	No. 8		1090	1.36	619	8.39	1
26	Park and Paulay (1990)	No. 9		727	1.21	393	10.71	1
27	Arakawa et al. (1982)	No. 102	flexible support	60	1.15	153	3.78	1
28	Nagasaka (1982)	HPRC10-63	cover scaled from drawing	27	1.02	87	2.74	3
29	Nagasaka (1982)	HPRC19-32	cover scaled from drawing	34	1.08	111	1.94	3
30	Ohno and Nishioka (1984)	L1	cover scaled from drawing	201	1.30	119	9.49	1
31	Ohno and Nishioka (1984)	L2	cover scaled from drawing	186	1.20	111	9.05	1
32	Ohno and Nishioka (1984)	L3	cover scaled from drawing	172	1.11	104	9.79	1
33	Ohue et al. (1985)	2D16RS	cover scaled from drawing, splitting-bond failure	41	1.08	98	4.55	3
34	Ohue et al. (1985)	4D13RS	cover scaled from drawing, splitting-bond failure	44	1.01	108	4.59	3
35	Zhou et al. (1985)	No. 806	L = 80mm, L/H = 1	3	1.18	27	2.15	3
36	Zhou et al. (1985)	No. 1007	L = 80mm, L/H = 1	3	1.54	31	1.88	3
37	Zhou et al. (1985)	No. 1309	L = 80mm, L/H = 1	2	2.78	23	1.19	3
38	Imai and Yamamoto (1986)	No. 1	cover scaled from drawing	392	0.95	471	4.16	3
39	Zhou et al. (1987)	No.104-08		13	1.15	79	0.84	2
40	Zhou et al. (1987)	No. 114-08	rectangular spiraled ties	15	1.27	87	1.31	2
41	Zhou et al. (1987)	No. 124-08	rectangular spiraled ties	18	1.59	108	1.88	3
42	Zhou et al. (1987)	No. 204-08		21	1.79	65	0.71	3
43	Zhou et al. (1987)	No. 214-08	rectangular spiraled ties	20	1.71	59	1.55	1
44	Zhou et al. (1987)	No. 223-09	rectangular spiraled ties	22	2.22	59	1.39	3
45	Zhou et al. (1987)	No. 302-07		25	1.55	48	1.68	3
46	Zhou et al. (1987)	No. 312-07	rectangular spiraled ties	26	1.66	51	1.38	3
47	Zhou et al. (1987)	No. 322-07	rectangular spiraled ties	25	1.57	48	2.61	3
48	Kanda et al. (1987)	85STC-1		60	1.20	76	4.38	1
49	Kanda et al. (1987)	85STC-2		62	1.26	80	3.74	1
50	Kanda et al. (1987)	85STC-3		60	1.20	76	4.38	1
51	Kanda et al. (1987)	85PDC-1		66	1.33	85	3.82	1
52	Kanda et al. (1987)	85PDC-2		59	1.19	75	4.02	1
53	Kanda et al. (1987)	85PDC-3		59	1.19	75	4.25	1
54	Arakawa et al. (1989)	OA2		29	0.86	129	1.33	2
55	Arakawa et al. (1989)	OA5		30	0.80	132	0.73	2
56	Muguruma et al. (1989)	AL-1		132	1.35	239	2.51	1
57	Muguruma et al. (1989)	AH-1		154	1.58	244	2.61	1
58	Muguruma et al. (1989)	AL-2		137	1.63	242	1.92	1
59	Muguruma et al. (1989)	AH-2		161	1.91	247	1.78	1
60	Muguruma et al. (1989)	BL-1		136	1.16	241	2.89	1
61	Muguruma et al. (1989)	BH-1		147	1.26	246	2.78	1
62	Muguruma et al. (1989)	BL-2		161	1.32	283	2.36	1
63	Muguruma et al. (1989)	BH-2		173	1.42	288	2.37	1
64	Ono et al. (1989)	CA025C		40	1.13	130	2.13	3
65	Ono et al. (1989)	CA060C		42	1.32	134	1.10	3
66	Sakai et al. (1990)	B1		196	1.04	375	2.42	1
67	Sakai et al. (1990)	B2		203	1.08	371	2.28	1
68	Sakai et al. (1990)	B3		214	1.14	406	2.52	1
69	Sakai et al. (1990)	B4		195	1.04	375	2.46	1
70	Sakai et al. (1990)	B5		203	1.08	387	2.24	1

Table A.1: Continued

Test Number	Reference	Column Designation	Comments	M_{MAX} (kN-m)	M_{MAX} / M_{ACI}	V_{MAX} (kN)	Δ_y (mm)	Failure Mode
71	Sakai et al. (1990)	B6		210	1.12	400	2.43	1
72	Sakai et al. (1990)	B7		182	0.99	352	1.68	1
73	Amitsu et al. (1991)	CB060C	bar spacing 74mm-37mm-37mm-74mm	170	1.28	506	1.20	3
74	Wight and Sozen (1973)	No. 40.033a(East)		88	1.07	94	10.46	3
75	Wight and Sozen (1973)	No. 40.033a(West)		90	1.10	98	10.15	3
76	Wight and Sozen (1973)	No. 40.048(East)		93	1.21	101	10.68	3
77	Wight and Sozen (1973)	No. 40.048(West)		88	1.15	95	11.00	3
78	Wight and Sozen (1973)	No. 40.033(East)		84	1.04	91	17.41	3
79	Wight and Sozen (1973)	No. 40.033(West)		93	1.16	101	12.87	3
80	Wight and Sozen (1973)	No. 25.033(East)		78	1.05	85	21.65	3
81	Wight and Sozen (1973)	No. 25.033(West)		83	1.11	91	12.94	3
82	Wight and Sozen (1973)	No. 40.067(East)		83	1.04	86	25.22	3
83	Wight and Sozen (1973)	No. 40.067(West)		89	1.11	92	12.91	3
84	Wight and Sozen (1973)	No. 40.147(East)		107	1.34	112	12.15	3
85	Wight and Sozen (1973)	No. 40.147(West)		102	1.28	106	11.22	3
86	Wight and Sozen (1973)	No. 40.092(East)		104	1.29	108	9.67	3
87	Wight and Sozen (1973)	No. 40.092(West)		108	1.35	113	10.39	3
88	Atalay and Penzien (1975)	No. 1S1		112	1.21	62	14.11	1
89	Atalay and Penzien (1975)	No. 2S1		113	1.23	61	14.93	1
90	Atalay and Penzien (1975)	No. 3S1		105	1.13	57	15.74	1
91	Atalay and Penzien (1975)	No. 4S1		89	0.88	49	20.06	1
92	Atalay and Penzien (1975)	No. 5S1		139	1.10	74	18.92	1
93	Atalay and Penzien (1975)	No. 6S1		143	1.13	75	18.97	1
94	Atalay and Penzien (1975)	No. 9		148	1.07	79	18.15	1
95	Atalay and Penzien (1975)	No. 10		151	1.10	78	18.62	1
96	Atalay and Penzien (1975)	No. 11		144	1.05	77	15.22	1
97	Atalay and Penzien (1975)	No. 12		157	1.13	78	18.78	1
98	Umehara and Jirsa (1982)	CUS		150	0.66	324	4.45	2
99	Umehara and Jirsa (1982)	CUW		124	0.88	265	4.25	2
100	Umehara and Jirsa (1982)	2CUS		192	0.73	412	4.33	2
101	Bett et al. (1985)	No. 1-1		99	0.68	214	4.82	2
102	Azizinamini et al. (1988)	NC-2		648	1.22	443	10.65	1
103	Azizinamini et al. (1988)	NC-4		670	1.13	463	9.28	1
104	Saatcioglu and Ozcebe (1989)	U3		284	1.02	267	20.80	1
105	Saatcioglu and Ozcebe (1989)	U4		360	1.32	324	13.06	1
106	Saatcioglu and Ozcebe (1989)	U6		377	1.32	341	13.58	1
107	Saatcioglu and Ozcebe (1989)	U7		376	1.31	340	13.61	1
108	Galeota et al. (1996)	AA1		169	1.20	130	7.53	1
109	Galeota et al. (1996)	AA2		159	1.13	121	8.60	1
110	Galeota et al. (1996)	AA3		124	1.03	95	12.05	1
111	Galeota et al. (1996)	AA4		171	1.42	138	5.41	1
112	Galeota et al. (1996)	BA1		176	1.46	141	5.31	1
113	Galeota et al. (1996)	BA2		163	1.16	126	7.99	1
114	Galeota et al. (1996)	BA3		170	1.21	131	7.69	1
115	Galeota et al. (1996)	BA4		141	1.17	110	9.49	1
116	Galeota et al. (1996)	CA1		131	1.09	101	11.00	1
117	Galeota et al. (1996)	CA2		173	1.23	126	7.99	1
118	Galeota et al. (1996)	CA3		166	1.37	132	6.10	1
119	Galeota et al. (1996)	CA4		178	1.27	135	7.70	1
120	Galeota et al. (1996)	AB1		221	1.17	175	13.24	1
121	Galeota et al. (1996)	AB2		224	1.17	165	11.14	1
122	Galeota et al. (1996)	AB3		230	1.19	166	12.58	1
123	Galeota et al. (1996)	AB4		255	1.35	205	10.18	1
124	Galeota et al. (1996)	BB		206	1.09	158	14.97	1
125	Galeota et al. (1996)	BB1		246	1.31	195	9.33	1
126	Galeota et al. (1996)	BB4		240	1.25	175	10.92	1
127	Galeota et al. (1996)	BB4B		234	1.22	171	11.33	1
128	Galeota et al. (1996)	CB1		229	1.21	172	13.56	1
129	Galeota et al. (1996)	CB2		216	1.15	167	13.42	1
130	Galeota et al. (1996)	CB3		252	1.31	170	13.27	1
131	Galeota et al. (1996)	CB4		246	1.28	172	11.89	1
132	Wehbe et al. (1998)	A1	Irregular tie configuration #3 cross ties perpendicular to load.	860	1.32	337	23.58	1
133	Wehbe et al. (1998)	A2	Irregular tie configuration #3 cross ties perpendicular to load.	935	1.28	363	22.00	1
134	Wehbe et al. (1998)	B1	Irregular tie configuration #3 cross ties perpendicular to load.	887	1.35	346	27.31	1
135	Wehbe et al. (1998)	B2	Irregular tie configuration #3 cross ties perpendicular to load.	991	1.33	372	26.86	1
136	Lynn et al. (1998)	3CLH18		416	0.95	277	12.89	3
137	Lynn et al. (1998)	2CLH18		369	1.11	241	9.10	3
138	Lynn et al. (1998)	2CMH18		472	1.16	306	9.20	3

Table A.1: Continued

Test Number	Reference	Column Designation	Comments	M_{MAX} (kN-m)	M_{MAX} / M_{ACI}	V_{MAX} (kN)	Δ_y (mm)	Failure Mode
139	Lynn et al. (1998)	3CMH18		504	0.98	328	11.97	3
140	Lynn et al. (1998)	3CMD12		547	1.06	355	13.17	3
141	Lynn et al. (1998)	3SLH18		403	0.92	270	11.29	3
142	Lynn et al. (1998)	2SLH18		345	1.04	229	9.16	3
143	Lynn et al. (1998)	3SMD12		576	1.15	367	12.22	3
144	Xiao and Martirossyan (1998)	HC48L19T10-0.1P		179	1.16	324	6.22	1
145	Xiao and Martirossyan (1998)	HC48L19T10-0.2P		213	1.18	378	5.27	1
146	Xiao and Martirossyan (1998)	HC48L16T10-0.1P		147	1.07	276	5.18	1
147	Xiao and Martirossyan (1998)	HC48L16T10-0.2P		181	1.09	319	6.23	1
148	Xiao and Martirossyan (1998)	HC4-8L16-T6-0.1P		144	1.06	268	6.32	3
149	Xiao and Martirossyan (1998)	HC4-8L16-T6-0.2P		174	1.03	324	5.71	3
150	Sugano (1996)	UC10H	cover scaled from sketch	161	1.17	334	1.34	1
151	Sugano (1996)	UC15H	cover scaled from sketch	182	1.32	365	1.41	1
152	Sugano (1996)	UC20H	cover scaled from sketch	198	1.43	392	1.46	1
153	Sugano (1996)	UC15L	cover scaled from sketch	176	1.12	363	1.86	1
154	Sugano (1996)	UC20L	cover scaled from sketch	179	1.14	370	1.84	1
155	Nosho et al. 1996	No. 1		121	1.00	42	24.19	1
156	Bayrak and Sheikh (1996)	ES-1HT		290	1.07	124	6.62	1
157	Bayrak and Sheikh (1996)	AS-2HT		300	1.06	140	7.72	1
158	Bayrak and Sheikh (1996)	AS-3HT		296	1.09	135	6.48	1
159	Bayrak and Sheikh (1996)	AS-4HT		301	1.11	127	9.76	1
160	Bayrak and Sheikh (1996)	AS-5HT	Twice as stiff as 6HT and 7HT	349	0.95	173	4.72	1
161	Bayrak and Sheikh (1996)	AS-6HT		341	0.94	154	9.97	1
162	Bayrak and Sheikh (1996)	AS-7HT		316	0.86	144	10.22	1
163	Bayrak and Sheikh (1996)	ES-8HT		349	0.96	166	6.77	1
164	Saatcioglu and Grira (1999)	BG-1		320	1.33	169	9.99	1
165	Saatcioglu and Grira (1999)	BG2		307	1.27	165	9.64	1
166	Saatcioglu and Grira (1999)	BG-3		270	1.22	148	15.40	1
167	Saatcioglu and Grira (1999)	BG4		335	1.22	171	11.01	1
168	Saatcioglu and Grira (1999)	BG5	Welded grid transverse reinforcement	336	1.23	173	13.76	1
169	Saatcioglu and Grira (1999)	BG-6	Welded grid transverse reinforcement	364	1.32	188	11.31	1
170	Saatcioglu and Grira (1999)	BG-7	Welded grid transverse reinforcement	344	1.24	178	11.97	1
171	Saatcioglu and Grira (1999)	BG8	Welded grid transverse reinforcement	327	1.16	180	20.66	1
172	Saatcioglu and Grira (1999)	BG9	Welded grid transverse reinforcement	361	1.26	185	12.41	1
173	Saatcioglu and Grira (1999)	BG-10	Welded grid transverse reinforcement	344	1.21	177	13.62	1
174	Matamoros et al. (1999)	C10-05N		45	0.99	70	10.65	1
175	Matamoros et al. (1999)	C10-05S		43	0.96	68	10.12	1
176	Matamoros et al. (1999)	C10-10N		63	1.12	96	8.95	1
177	Matamoros et al. (1999)	C10-10S		62	1.09	93	9.06	1
178	Matamoros et al. (1999)	C10-20N		76	1.03	108	10.29	1
179	Matamoros et al. (1999)	C10-20S		72	1.08	102	9.26	1
180	Matamoros et al. (1999)	C5-00N		36	1.05	59	11.80	1
181	Matamoros et al. (1999)	C5-00S		36	1.01	58	12.41	1
182	Matamoros et al. (1999)	C5-20N		48	0.98	71	10.54	1
183	Matamoros et al. (1999)	C5-20S		46	0.96	69	10.87	1
184	Matamoros et al. (1999)	C5-40N		59	1.13	85	8.24	1
185	Matamoros et al. (1999)	C5-40S		59	1.16	85	8.10	1
186	Mo and Wang (2000)	C1-1		356	1.20	243	14.95	1
187	Mo and Wang (2000)	C1-2		383	1.21	258	14.79	1
188	Mo and Wang (2000)	C1-3		440	1.36	291	14.87	1
189	Mo and Wang (2000)	C2-1		353	1.20	241	16.71	1
190	Mo and Wang (2000)	C2-2		375	1.18	250	15.62	1
191	Mo and Wang (2000)	C2-3		446	1.37	294	13.45	1
192	Mo and Wang (2000)	C3-1	Atypical Transverse Reinforcement	334	1.12	228	17.91	1
193	Mo and Wang (2000)	C3-2	Atypical Transverse Reinforcement	372	1.18	248	17.89	1
194	Mo and Wang (2000)	C3-3	Atypical Transverse Reinforcement	432	1.32	286	15.43	1
195	Aboutaha and Machado (1999)	ORC1	Missing key steel properties	472		258		1
196	Aboutaha and Machado (1999)	ORC2	Missing key steel properties	710		308		1
197	Aboutaha and Machado (1999)	ORC3	Missing key steel properties	775		423		1
198	Aboutaha et al. (1999)	SC3		496	0.82	407	9.49	2
199	Aboutaha et al. (1999)	SC9		737	0.68	605	9.52	2
200	Thomsen and Wallace (1994)	A1		26	1.35	44	6.12	1
201	Thomsen and Wallace (1994)	A3		44	1.20	67	3.53	1
202	Thomsen and Wallace (1994)	B1		19	1.13	32	5.68	1
203	Thomsen and Wallace (1994)	B2		31	1.13	48	4.56	1
204	Thomsen and Wallace (1994)	B3		38	1.05	58	3.96	1
205	Thomsen and Wallace (1994)	C1		22	1.28	37	6.68	1
206	Thomsen and Wallace (1994)	C2		28	1.06	44	4.78	1
207	Thomsen and Wallace (1994)	C3		33	0.96	50	5.59	1
208	Thomsen and Wallace (1994)	D1		34	1.02	52	4.56	1
209	Thomsen and Wallace (1994)	D2		35	0.98	55	5.12	1
210	Thomsen and Wallace (1994)	D3		32	1.01	48	5.62	1

Table A.1: Continued

Test Number	Reference	Column Designation	Comments	M_{MAX} (kN-m)	M_{MAX} / M_{ACI}	V_{MAX} (kN)	Δ_y (mm)	Failure Mode
211	Sezen and Moehle (19??)	No. 1		464	1.22	303	13.94	3
212	Sezen and Moehle (19??)	No. 2		477	1.29	301	6.97	3
213	Sezen and Moehle (19??)	No. 4		454	1.17	295	15.94	3
214	Legeron & Paultre (2000)	No. 1006015		237	0.95	100	27.89	1
215	Legeron & Paultre (2000)	No. 1006025		334	1.02	130	20.29	1
216	Legeron & Paultre (2000)	No. 1006040		344	0.99	123	21.76	1
217	Legeron & Paultre (2000)	No. 10013015		217	0.85	90	28.80	1
218	Legeron & Paultre (2000)	No. 10013025		349	1.06	140	18.76	1
219	Legeron & Paultre (2000)	No. 10013040		388	1.06	150	16.06	1
220	Paultre et al. (2001)	No. 806040		334	1.15	130	15.78	1
221	Paultre et al. (2001)	No. 1206040		426	1.12	156	16.16	1
222	Paultre et al. (2001)	No. 1005540		396	1.04	143	18.98	1
223	Paultre et al. (2001)	No. 1008040		372	1.02	136	21.03	1
224	Paultre et al. (2001)	No. 1005552		403	1.15	150	13.59	1
225	Paultre et al. (2001)	No. 1006052		417	1.12	150	15.72	1
226	Pujol (2002)	No. 10-2-3N		79	1.05	113	6.67	1
227	Pujol (2002)	No. 10-2-3S		79	1.05	113	7.29	1
228	Pujol (2002)	No. 10-3-1.5N		80	1.08	112	6.82	1
229	Pujol (2002)	No. 10-3-1.5S		80	1.07	112	6.59	1
230	Pujol (2002)	No. 10-3-3N		80	1.09	112	7.00	1
231	Pujol (2002)	No. 10-3-3S		80	1.09	112	6.57	1
232	Pujol (2002)	No. 10-3-2.25N		81	1.12	114	6.69	1
233	Pujol (2002)	No. 10-3-2.25S		81	1.11	114	6.84	1
234	Pujol (2002)	No. 20-3-1.5N	Missing FD data, L, Lmeas, Ltop					1
235	Pujol (2002)	No. 20-3-1.5S	Missing FD data, L, Lmeas, Ltop					1
236	Pujol (2002)	No. 20-3-3N		93	1.07	130	6.51	1
237	Pujol (2002)	No. 20-3-3S		93	1.08	130	6.84	1
238	Pujol (2002)	No. 10-2-2.25N		81	1.09	116	6.31	1
239	Pujol (2002)	No. 10-2-2.25S		81	1.09	116	6.21	1
240	Pujol (2002)	No. 10-1-2.25N		83	1.12	117	6.28	1
241	Pujol (2002)	No. 10-1-2.25S		83	1.12	117	6.48	1
242	Kono and Watanabe (2002)	D1N30		139	1.32	201	3.43	1
243	Kono and Watanabe (2002)	D1N60		124	1.36	186	2.58	1
244	Arai, et. al. (2002)	L1D60		1560	1.39	1239	5.70	1
245	Arai, et. al. (2002)	L1N60		1680	1.50	1339	3.80	1
246	Arai, et. al. (2002)	L1N6B		1590	1.89	1201	3.69	1
247	Takemura and Kawashima (1997)	Test 1 (JSCE-4)	Axial Load = 0.027 Agf'c	189	1.03	150	7.80	1
248	Takemura and Kawashima (1997)	Test 2 (JSCE-5)	Axial Load = 0.027 Agf'c	184	1.01	146	10.90	1
249	Takemura and Kawashima (1997)	Test 3 (JSCE-6)	Axial Load = 0.027 Agf'c	191	1.07	149	8.34	1
250	Takemura and Kawashima (1997)	Test 4 (JSCE-7)	Axial Load = 0.027 Agf'c	196	1.11	154	8.43	1
251	Takemura and Kawashima (1997)	Test 5 (JSCE-8)	Axial Load = 0.027 Agf'c	199	1.07	156	7.42	1
252	Takemura and Kawashima (1997)	Test 6 (JSCE-9)	Axial Load = 0.027 Agf'c	209	1.14	159	7.81	1
253	Xiao and Yun (2002)	No.FHC1-0.2	F-D Envelope only	1390	1.32	724	14.79	1
254	Xiao and Yun (2002)	No.FHC2-0.34	F-D Envelope only	1550	1.35	798	11.10	1
255	Xiao and Yun (2002)	No.FHC3-0.22	F-D Envelope only	1440	1.33	761	13.81	1
256	Xiao and Yun (2002)	No.FHC4-0.33	F-D Envelope only	1540	1.33	800	11.36	1
257	Xiao and Yun (2002)	No.FHC5-0.2	F-D Envelope only	1380	1.27	723	14.05	1
258	Xiao and Yun (2002)	No.FHC6-0.2	F-D Envelope only	1350	1.24	712	15.95	1
259	Bayrak (1998)	RS- 9HT	F-D Envelope only	387	1.25	174	12.94	1
260	Bayrak (1998)	RS-10HT	F-D Envelope only	388	1.31	179	9.36	1
261	Bayrak (1998)	RS-11HT	2 Sizes of Transverse Reinforcement, F-D Envelope only	349		190		1
262	Bayrak (1998)	RS-12HT	F-D Envelope only	348	1.13	162	10.50	1
263	Bayrak (1998)	RS-13HT	F-D Envelope only	430	0.99	188	11.32	1
264	Bayrak (1998)	RS-14HT	F-D Envelope only	414	0.96	154	19.52	1
265	Bayrak (1998)	RS-15HT	F-D Envelope only	323	1.22	142	19.79	1
266	Bayrak (1998)	RS-16HT	F-D Envelope only	309	1.17	140	16.22	1
267	Bayrak (1998)	RS-17HT	F-D Envelope only	395	1.22	171	20.14	1
268	Bayrak (1998)	RS-18HT	F-D Envelope only	355	1.14	159	11.64	1
269	Bayrak (1998)	RS-19HT	F-D Envelope only	385	1.26	173	11.52	1
270	Bayrak (1998)	RS-20HT	F-D Envelope only	410	1.28	190	14.40	1
271	Bayrak (1998)	WRS-21HT	F-D Envelope only	254	1.00	84	19.33	1
272	Bayrak (1998)	WRS-22HT	F-D Envelope only	262	1.05	93	22.06	1
273	Bayrak (1998)	WRS-23HT	F-D Envelope only	230	1.10	88	21.52	1
274	Bayrak (1998)	WRS-24HT	F-D Envelope only	236	1.14	88	17.82	1

Appendix B: Summary of Tests of Spiral-Reinforced Columns

Table B.1: Summary of Tests of Spiral-Reinforced Columns

Test Number	Reference	Column Designation	Comments	M_{MAX} (kN-m)	M_{MAX} / M_{ACI}	V_{MAX} (kN)	Δ_y (mm)	Failure Mode
1	Davey (1975)	No. 1	Specimen had pier cap,eccentric vertical load	527	1.29	180	14.09	1
2	Davey (1975)	No. 2	Specimen had pier cap,eccentric vertical load	600	1.40	334	10.73	1
3	Davey (1975)	No. 3	Specimen had pier cap,eccentric vertical load	485	1.14	142	17.78	1
4	Munro et al. (1976)	No. 1	Specimen had pier cap,not tested to failure	365	1.13	133	21.51	1
5	Ng et al. (1978)	No. 2	Specimen had pier cap	49	1.19	36	0.00	NA
6	Ng et al. (1978)	No. 3	Specimen had pier cap	72	1.23	61	7.11	1
7	Ghee et. al. (1981)	No. 1		250	1.20	139	8.82	1
8	Ghee et. al. (1981)	No. 2		303	1.38	163	8.92	1
9	Potangaroa et al. (1979)	No. 1	tested C108 to m=8	887	1.20	687	5.76	1
10	Potangaroa et al. (1979)	No. 3		933	1.29	729	4.53	3
11	Potangaroa et al. (1979)	No. 4	tested to m=8	1000	1.15	781	7.18	1
12	Potangaroa et al. (1979)	No. 5A	tested to m=8	1060	1.24	812	5.14	1
13	Potangaroa et al. (1979)	No. 5B	5a tested again under increased axial load	1124		937		
14	Ghee et al. (1985)	No. 1	Flexible base, axial load=0	256	1.03	321	9.01	3
15	Ghee et al. (1985)	No. 2	Flexible base, axial load=0	175	0.97	219	6.12	3
16	Ghee et al. (1985)	No. 3	Flexible base, axial load=0	276	1.13	276	10.07	3
17	Ghee et al. (1985)	No. 4	Flexible base, axial load=0	231	0.98	289	10.21	2
18	Ghee et al. (1985)	No. 5	Flexible base, axial load=0	265	1.10	331	9.52	3
19	Ghee et al. (1985)	No. 6	Flexible base, axial load=0	235	0.99	392	6.77	2
20	Ghee et al. (1985)	No. 7	Flexible base, axial load=0	225	0.92	281	8.19	2
21	Ghee et al. (1985)	No. 8	Flexible base	377	1.33	445	7.39	3
22	Ghee et al. (1985)	No. 9	Flexible base	401	1.39	364	14.22	1
23	Ghee et al. (1985)	No. 10	Flexible base	371	1.29	437	7.01	3
24	Ghee et al. (1985)	No. 11	Flexible base	339	1.17	407	6.98	3
25	Ghee et al. (1985)	No. 12	Flexible base, axial load lowered after m=1.5	321	1.22	526	4.88	3
26	Ghee et al. (1985)	No. 13	Flexible base	365	1.29	436	8.50	3
27	Ghee et al. (1985)	No. 14	Flexible base,axial load=0	253	1.07	316	8.64	3
28	Ghee et al. (1985)	No. 15	Flexible base,axial load=0	184	1.17	230	5.65	3
29	Ghee et al. (1985)	No. 16	Flexible base	287	1.04	352	9.52	3
30	Ghee et al. (1985)	No. 17	Flexible base	320	1.16	312	10.37	3
31	Ghee et al. (1985)	No. 18	Flexible base	309	1.11	505	5.49	2
32	Ghee et al. (1985)	No. 19	Flexible base	266	0.96	437	4.59	2
33	Ghee et al. (1985)	No. 20	Flexible base	351	1.11	487	5.76	2
34	Ghee et al. (1985)	No. 21	Flexible base,loaded monotonically up to m=6,axial load=0	216	0.90	271	5.57	2
35	Ghee et al. (1985)	No. 22	Flexible base, axial load=0	228	0.97	285	4.95	2
36	Ghee et al. (1985)	No. 23	Flexible base, axial load=0	266	1.13	333	8.76	3
37	Ghee et al. (1985)	No. 24	Flexible base, axial load=0	272	1.14	341	7.36	3
38	Ghee et al. (1985)	No. 25	Flexible base, axial load=0, no spiral reinf.	144	0.75	239	3.22	2
39	Zahn et. al. (1986)	No. 5	f'c not reported for test day. 28-day strength reported	240	1.10	142	9.59	1
40	Zahn et. al. (1986)	No. 6	f'c not reported for test day. 28-day strength reported	324	1.52	175	6.45	1
41	Watson (1989)	No 10		393	1.44	212	8.08	1
42	Watson (1989)	No 11		394	1.79	207	6.35	1
43	Wong et al. (1990)	No. 1	Axial load reduced after failure	394	1.32	461	5.87	1
44	Wong et al. (1990)	No. 2	Axial load reduced after failure	412	1.29	489	3.78	3
45	Wong et al. (1990)	No. 3		499	1.59	579	4.32	1
46	Petrovski and Ristic (1984)	M1E1	P-Delta Code Missing					1
47	Petrovski and Ristic (1984)	M1E2	P-Delta Code Missing					1
48	Petrovski and Ristic (1984)	M2E1	P-Delta Code Missing					3
49	Petrovski and Ristic (1984)	M2E2	P-Delta Code Missing					3
50	Lim et al. (1990)	Con1		22	1.02	14	30.54	1
51	Lim et al. (1990)	Con2		24	1.11	37	11.02	1
52	Lim et al. (1990)	Con3		24	1.15	36	10.43	1
53	Stone and Cheok (1989)	Flexure		13300	1.15	1289	109.63	1
54	Stone and Cheok (1989)	Shear		14500	1.29	2968	41.27	1
55	Cheok and Stone (1986)	N1		50	1.11	59	7.39	1
56	Cheok and Stone (1986)	N2		63	1.30	73	6.16	1
57	Cheok and Stone (1986)	N3		57	1.25	32	16.10	1
58	Cheok and Stone (1986)	N4		51	1.13	63	4.89	1
59	Cheok and Stone (1986)	N5		64	1.30	77	6.31	1
60	Cheok and Stone (1986)	N6		52	1.16	30	14.35	1

Table B.1: Continued

Test Number	Reference	Column Designation	Comments	M_{MAX} (kN-m)	M_{MAX} / M_{ACI}	V_{MAX} (kN)	Δ_y (mm)	Failure Mode
61	Siryo (1975)	spbaa1	Possible joint rotation, confined with welded wire hoops, square cross-section	46	1.55	117	0.71	1
62	Siryo (1975)	ws21bs	square cross-section, L/D = 1	45	1.05	175	2.20	3
63	Siryo (1975)	ws22bs	square cross-section	53	1.14	102	2.51	1
64	Siryo (1975)	ws25bs	square cross-section, L/D = 1	46	1.04	182	2.56	3
65	Siryo (1975)	ws26bs	square cross-section	53	1.08	102	3.39	3
66	Siryo (1975)	ws27bs	square cross-section	76	1.22	146	3.45	1
67	Arakawa et al. (1987)	No. 1	axial load=0	53	0.67	176	1.49	2
68	Arakawa et al. (1987)	No. 2	axial load=0	61	0.77	204	1.43	2
69	Arakawa et al. (1987)	No. 3	no spiral reinforcement	48	0.54	158	1.17	2
70	Arakawa et al. (1987)	No. 4		58	0.64	191	1.02	2
71	Arakawa et al. (1987)	No. 6		68	0.77	223	1.18	2
72	Arakawa et al. (1987)	No. 8		64	0.71	211	1.06	2
73	Arakawa et al. (1987)	No. 9		68	0.62	226	1.61	2
74	Arakawa et al. (1987)	No. 10		76	1.10	251	1.68	3
75	Arakawa et al. (1987)	No. 11	no spiral reinforcement	57	0.59	187	1.03	2
76	Arakawa et al. (1987)	No. 12		58	0.61	189	1.05	2
77	Arakawa et al. (1987)	No. 13		72	0.73	234	0.86	2
78	Arakawa et al. (1987)	No. 14		84	0.85	274	0.88	2
79	Arakawa et al. (1987)	No. 15	axial load=0	76	0.94	168	4.13	3
80	Arakawa et al. (1987)	No. 16	axial load=0	79	0.99	176	3.54	1
81	Arakawa et al. (1987)	No. 17		74	0.82	245	1.17	2
82	Arakawa et al. (1987)	No. 18	no spiral reinforcement	59	0.66	131	1.93	2
83	Arakawa et al. (1987)	No. 19		84	0.93	184	1.68	2
84	Arakawa et al. (1987)	No. 20		96	1.08	209	3.30	1
85	Arakawa et al. (1987)	No. 21		93	1.04	151	4.92	1
86	Arakawa et al. (1987)	No. 22		77	0.94	168	1.59	2
87	Arakawa et al. (1987)	No. 23		96	1.00	209	3.25	3
88	Arakawa et al. (1987)	No. 24		70	0.71	230	1.05	2
89	Arakawa et al. (1987)	No. 25		91	0.93	196	1.60	2
90	Arakawa et al. (1987)	No. 26		104	1.07	167	4.80	1
91	Arakawa et al. (1987)	No. 27		79	0.97	171	3.19	3
92	Arakawa et al. (1987)	No. 28		103	0.98	224	3.04	3
93	Kunnath et. al. (1997)	No. A2		115	1.32	74	13.94	1
94	Kunnath et. al. (1997)	No. A3		120	1.37	75	12.77	1
95	Kunnath et. al. (1997)	No. A4		111	1.21	72	15.29	1
96	Kunnath et. al. (1997)	No. A5		123	1.34	77	16.84	1
97	Kunnath et. al. (1997)	No. A6		119	1.29	77	13.53	1
98	Kunnath et. al. (1997)	No. A7		120	1.32	79	11.14	1
99	Kunnath et. al. (1997)	No. A8		107	1.17	68	15.43	1
100	Kunnath et. al. (1997)	No. A9		114	1.26	75	11.87	1
101	Kunnath et. al. (1997)	No. A10		113	1.32	74	12.05	1
102	Kunnath et. al. (1997)	No. A11		103	1.21	68	12.70	1
103	Kunnath et. al. (1997)	No. A12		109	1.28	72	11.24	1
104	Priestley and Benzoni (1994)	NR1		365	1.25	393	2.35	3
105	Priestley and Benzoni (1994)	NR2		537	1.23	579	3.70	3
106	Kunnath et. al. (1997)	No. SRPH1		1300	1.30	285	39.88	1
107	Vu et al. (1998)	No. NH1		530	1.27	535	6.47	1
108	Vu et al. (1998)	No. NH2		254	1.32	296	7.11	3
109	Vu et al. (1998)	No. NH3		501	1.33	510	6.13	1
110	Vu et al. (1998)	No. NH4		870	1.45	905	10.11	3
111	Vu et al. (1998)	No. NH5		344	1.35	403	8.26	3
112	Vu et al. (1998)	No. NH6		975	1.61	957	7.77	1
113	Kowalsky et. al. (1999)	No. FL1	lightweight concrete	544	1.09	101	0.00	0
114	Kowalsky et. al. (1999)	No. FL2	lightweight concrete	639	1.23	124	0.00	0
115	Kowalsky et. al. (1999)	No. FL3	normal-weight concrete	611	1.20	117	60.92	1
116	Lehman and Moehle (2000)	No.415		708	1.24	269	17.60	1
117	Lehman and Moehle (2000)	No.815		745	1.30	130	64.81	1
118	Lehman and Moehle (2000)	No.1015		604	1.06	80	109.46	1
119	Lehman and Moehle (2000)	No.407		443	1.15	172	13.18	1
120	Lehman and Moehle (2000)	No.430	Longitudinal bars were bundled in two layers	1180	1.24	448	26.18	1
121	Calderone et. al. (2000)	No.328		1030	1.16	525	14.88	1
122	Calderone et. al. (2000)	No.828	0.9% hoop steel provided up to 3 feet from base. Remaining height has 0.45% hoop steel	975	1.10	172	83.04	1
123	Calderone et. al. (2000)	No.1028	0.9% hoop steel provided up to 4 feet from base. Remaining height has 0.45% hoop steel	1160	1.30	157	95.47	1
124	Sritharan et al. (1995)	IC1		737	1.26	387	0.00	0
125	Sritharan et al. (1995)	B105IC2		775	1.29	411	0.00	0
126	Sritharan et al. (1995)	IC3		815	1.34	433	0.00	0
127	Saatcioglu and Baingo (1999)	No.RC1		138	1.41	55	0.00	0
128	Saatcioglu and Baingo (1999)	No.RC2		132	1.39	53	0.00	0

Table B.1: Continued

Test Number	Reference	Column Designation	Comments	M_{MAX} (kN-m)	M_{MAX} / M_{ACI}	V_{MAX} (kN)	Δ_y (mm)	Failure Mode
129	Saatcioglu and Baingo (1999)	No.RC3		163	1.32	56	15.83	1
130	Saatcioglu and Baingo (1999)	No.RC4		162	1.31	55	12.67	1
131	Saatcioglu and Baingo (1999)	No.RC6		154	1.26	57	11.25	1
132	Saatcioglu and Baingo (1999)	No.RC7		139	1.20	59	17.08	1
133	Saatcioglu and Baingo (1999)	No.RC8	Column confined with individual circular hoops	158	1.28	55	13.47	1
134	Saatcioglu and Baingo (1999)	No.RC9	Specimen had no cover to longitudinal reinforcement	200	1.63	71	0.00	0
135	Nelson (2000)	Col1	Axial load varies	488	1.16	283	11.56	3
136	Nelson (2000)	Col2		456	1.17	279	9.74	1
137	Nelson (2000)	Col3		423	1.12	260	9.09	3
138	Nelson (2000)	Col4		415	1.12	252	9.48	3
139	Henry and Mahin (1999)	No. 415p		831	1.19	277	25.78	1
140	Henry and Mahin (1999)	No. 415s		716	1.20	259	23.62	1
141	Chai et. al. (1991)	No. 3	Retrofitted columns were tested also. Unknown concrete cover	889	1.13	207	29.11	1
142	Roeder et. al. (2001)	C1	fy not measured for spiral. Assumed to be 60 ksi.	230		117		1
143	Roeder et. al. (2001)	C2	fy not measured for spiral. Assumed to be 60 ksi.	218		111		1
144	Roeder et. al. (2001)	C3	PSC pile-wharf connection. fy not measured for spiral. Assumed to be 60 ksi. f'c not reported for test day. 28-day strength reported	269		137		1
145	Roeder et. al. (2001)	C4	PSC pile-wharf connection. fy not measured for spiral. Assumed to be 60 ksi. f'c not reported for test day. 28-day strength reported	360		167		1
146	Roeder et. al. (2001)	C5	PSC pile-wharf connection. fy not measured for spiral. Assumed to be 60 ksi. f'c not reported for test day. 28-day strength reported	377		177		1
147	Roeder et. al. (2001)	C6	PSC pile-wharf connection. fy not measured for spiral. Assumed to be 60 ksi. f'c not reported for test day. 28-day strength reported	371		176		1
148	Roeder et. al. (2001)	C7	PSC pile-wharf connection. fy not measured for spiral. Assumed to be 60 ksi. f'c not reported for test day. 28-day strength reported	376		171		1
149	Roeder et. al. (2001)	C8	PSC pile-wharf connection. fy not measured for spiral. Assumed to be 60 ksi. f'c not reported for test day. 28-day strength reported	393		182		1
150	Moyer and Kowalsky (2002)	No.1		372	1.09	144	37.91	1
151	Moyer and Kowalsky (2002)	No.2		388	1.15	152	41.12	1
152	Moyer and Kowalsky (2002)	No.3		462	1.38	180	36.72	1
153	Moyer and Kowalsky (2002)	No.4		382	1.11	150	40.39	1
154	Coffman et al. (1993)	Column 1	Column No. 1 was a spliced, reference column.	306	1.40	94	17.40	1
155	Hamilton (2002)	UCI1	Cross-section D	130	1.15	70	16.08	1
156	Hamilton (2002)	UCI2	Cross-section D	136	1.20	74	17.03	1
157	Hamilton (2002)	UCI3	Cross-section B, Monotonic	150	1.14	143	4.20	3
158	Hamilton (2002)	UCI4	Cross-section B	172	1.31	164	6.09	3
159	Hamilton (2002)	UCI5	Cross-section SD	178	1.54	170	4.76	3
160	Hamilton (2002)	UCI6	Cross-section D	182	1.62	98	12.88	1

Appendix C: Structure of xml Data

The structure of the xml files that describe each column test in the database (http://nisee.berkeley.edu/spd) is discussed in this appendix. The column data in the xml file is organized into 9 data structures (specimen, adminInfo, materialProperties, geometry, loading, longitudinalReinforcement, transverseReinforcement, failureType, damage and links). The organization of the key data structures (i.e., materialProperties, geometry, longitudinalReinforcement and transverseReinforcement) are summarized in Tables C.1 to C.5.

Table C.1: Organization of materialProperties Structure

Structure Fields	Structure Subfields	xml Notation	Description of Property	Column Type
		concreteStrength	Characteristic compressive strength of concrete (MPa)	R, S
longitudinalSteel		yieldStress	Yield stress of longitudinal reinforcement (MPa)	S
		strength	Ultimate steel strength for longitudinal reinforcement (MPa)	S
	corner	yieldStress	Yield stress of longitudinal corner bars (MPa)	R
		strength	Ultimate steel strength of longitudinal corner bars (MPa)	R
	intermediate	yieldStress	Yield stress of longitudinal intermediate bars (MPa)	R
		strength	Ultimate steel strength of longitudinal intermediate bars (MPa)	R
transverseSteel		yieldStress	Yield stress of transverse reinforcement (MPa)	R, S
		strength	Ultimate steel strength for transverse reinforcement(MPa)	R, S

Table C.2: Organization of geometry Structure

xml Notation	Description of Property	Column Type
depth	Column Depth (mm)	R, S
width	Column Width (mm)	R
lInflection	Length of equivalent cantilever (mm)	R, S
configuration	Test Configuration (Chapter 2.4)	R, S
lSplice	Length of longitudinal reinforcement splice	R, S
lMeasured	Distance to Deformation Measurement (Chapter 3.3)	R, S

Table C.3: Organization of longitudinalReinforcement Structure

Structure Fields	xml Notation	Description of Property	Column Type
	numberOfBars	Number of longitudinal reinforcing bars	R, S
	diameter	Diameter of longitudinal reinforcement bars (mm)	S
	diameter	Diameter of longitudinal corner bars (mm)	R
	diameterIntermediate	Diameter of longitudinal intermediate bars (mm)	R
	reinforcementRatio	Longitudinal reinforcement ratio (calculated).	R, S
	clearCover		
parallelToLoad	clearCover	Distance from the surface of column to outer edge of transverse reinforcement (mm), Parallel to the horizontal load.	R
	numberIntermediateBars	# of Intermediate Bars Parallel to Horizontal Load	R
perpendicularToLoad	clearCover	Distance from the surface of column to outer edge of transverse reinforcement (mm), Perpendicular to the horizontal load.	R
	numberIntermediateBars	# of Intermediate Bars Perpendicular to Horizontal Load	R

Table C.4: Organization of transverseReinforcement Structure

Structure Fields	xml Notation	Description of Property	Column Type
closeSpacing	barDiameter	Diameter of transverse reinforcement (mm)	R, S
	hoopSpacing	Spacing of transverse reinforcement (mm)	R, S
	volTransReinfRatio	Volumetric transverse reinforcement ratio (reported)	R, S
	numberShearLegs	Number of transverse shear bars in cross section	R, S
	type	Type of Confinement (Chapter 2.3)	R

Appendix D: Column Test References

Aboutaha, R.S., and Machado, R.I. (1999). "Seismic Resistance of Steel-Tubed High-Strength Reinforced-Concrete Columns." *J. Struct. Eng.*, 125(5), 485-494.

Aboutaha, R.S., Engelhardt, M.D., Jirsa, J.O.: and Kreger, M.E. (1999). "Rehabilitation of Shear Critical Concrete Columns by Use of Rectangular Steel Jackets." *ACI Struct. J.*, 96(1), 68-78.

Amitsu, S., Shirai, N., Adachi, H., and Ono, A. (1991). "Deformation of Reinforced Concrete Column with High or Fluctuating Axial Force." *Transactions of the Japan Concrete Institute,* 13.

Arai, Y., Hakim, B., Kono, S., Watanabe, F. (2002). "Damage Assessment of Reinforced Concrete Columns Under High Axial Loading." Personal Contact.

Arakawa, T., Arai, Y., Egashira, K., and Fujita, Y. (1982). "Effects of the Rate of Cyclic Loading on the Load-Carrying Capacity and Inelastic Behavior of Reinforced Concrete Columns." *Transactions of the Japan Concrete Institute,* 4.

Arakawa, T., Arai, Y., Mizoguchi, M., and Yoshida, M. (1989). "Shear Resisting Behavior of Short Reinforced Concrete Columns Under Biaxial Bending-Shear." *Transactions of the Japan Concrete Institute,* 11.

Arakawa, T., He, M., Arai, Y., and Mizoguchi, M. (1998). "Shear Resisting Behavior of Reinforced Concrete Columns with Spiral Hoops." *Transactions of the Japan Concrete Institute,* 10.

Atalay, M.B., and Penzien, J. (1975). "The Seismic Behavior of Critical Regions of Reinforced Concrete Components as Influenced by Moment, Shear and Axial Force." *Report No. EERC 75-19,* University of California, Berkeley, California.

Azizinamini, A., Johal, L. S., Hanson, N. W., Musser, D. W., and Corley, W. G. (1988). "Effects of Transverse Reinforcement on Seismic Performance of Columns – A Partial Parametric Investigation." *Project No. CR-9617,* Construction Technology Laboratories, Skokie, Illinois.

Bayrak, O., and Sheikh, S. (1996). "Confinement Steel Requirements for High Strength Concrete Columns." *Proc. 11th World Conference on Earthquake Engineering*, Aculpulco, Mexico, Paper No. 463.

Bayrak O. (1998). "Seismic Performance of Rectilinearly Confined High Strength Concrete Columns." Doctoral Dissertation. Dept. of Civil Engineering, Univ. of Toronto, Canada.

Bett, B. J., Klingner, R. E., and Jirsa, J. O. (1985). "Behavior of Strengthened and Repaired Reinforced Concrete Columns Under Cyclic Deformations." *PMFSEL Report No. 85-3,* Department of Civil Engineering, University of Texas at Austin.

Calderone, A.J., Lehman, D.E., and Moehle, J.P. (2000). "Behavior of Reinforced Concrete Bridge Columns Having Varying Aspect Ratios and Varying Lengths of Confinement."

Pacific Earthquake Engineering Research Center Report 2000/08, University of California, Berkeley, California.

Chai, Y., Priestley, M., and Seible, F. (1991). "Seismic Retrofit of Circular Bridge Columns for Enhanced Flexural Performance." *ACI Struct. J.*, 88(5), 572-584.

Cheok, G.S., and Stone, W. C. (1986). "Behavior of 1/6-Scale Model Bridge Columns Subjected to Cycle Inelastic Loading." *NBSIR 86-3494*, U.S. National Institute of Standards and Technology, Gaithersburg, Maryland.

Coffman, H. L., Marsh, M. L., Brown, C. B. (1993). "Seismic Durability of Retrofitted Reinforced-Concrete Columns." *J. Struct. Eng.*, 119(5), 1643-1661.

Davey, B.E. (1975). "Reinforced Concrete Bridge Piers under Seismic Loading." Master of Engineering Report. Civil Engineering Department, University of Canterbury, Christchurch, New Zealand.

Galeota, D.: Giammatteo, M.M., Marino, R. (1996). "Seismic Resistance of High Strength Concrete Columns." *Procedings of the Eleventh World Conference on Earthquake Engineering,* Disc 3, Paper No. 1390.

Ghee, A. B., Priestley, M.J.N., and Park, R. (1981). "Ductility of Reinforced Concrete Bridge Piers under Seismic Loading." *Report 81-3*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Ghee, A., Priestley, M.J.N., and Paulay, T. (1989). "Seismic Shear Strength of Circular Reinforced Concrete Columns." *ACI Struct. J.*, 86(1), 45-59.

Gill, W. D., Park, R., and Priestley, M.J.N. (1979). "Ductility of Rectangular Reinforced Concrete Columns With Axial Load." *Report 79-1*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Henry, L., and Mahin, S. A. (1999). "Study of Buckling of Longitudinal Bars in Reinforced Concrete Bridge Columns." Report to the California Department of Transportation.

Imai, H., and Yamamoto, Y. (1986). "A Study on Causes of Earthquake Damage of Izumi High School Due to Miyagi-Ken-Oki Earthquake in 1978." *Transactions of the Japan Concrete Institute,* 8.

Kanda, M., Shirai, N., Adachi, H., and Sato, T. (1988). "Analytical Study on Elasto-Plastic Hysteretic Behaviors of Reinforced Concrete Members." *Transactions of the Japan Concrete Institute,* 10.

Kono, S., and Watanabe, F. (2000). "Damage Evaluation of Reinforced Concrete Columns Under Multiaxial Cyclic Loadings." *The Second U.S.-Japan Workshop on Performance-Based Earthquake Engineering Methodology for Reinfoced Concrete Building Structures*, Sapporo, Japan.

Kowalsky, M.J., Priestley, M.J.N., and Seible, F. (1999). "Shear and Flexural Behavior of Lightweight Concrete Bridge Columns in Seismic Regions." *ACI Struct. J.*, 96(1),136-148.

Kunnath, S., El-Bahy, A., Taylor, A., and Stone, W. (1997). "Cumulative Seismic Damage of Reinforced Concrete Bridge Piers." *Technical Report NCEER-97-0006*, National Center for Earthquake Engineering Research, Buffalo, New York.

Legeron, F., and Paultre, P. (2000). “Behavior of High-Strength Concrete Columns under Cyclic Flexure and Constant Axial Load.” *ACI Struct. J.*, 97(4), 591-601.

Lehman, D.E., and Moehle, J.P. (2000). "Seismic Performance of Well-Confined Concrete Bridge Columns." *Pacific Earthquake Engineering Research Center Report 1998/01*, University of California, Berkeley, California.

Lim, K. Y., McLean, D. I., and Henley, E. H. (1991). "Moment-Reducing Hinge Details for the Bases of Bridge Columns." *Transportation Research Record, No. 1275*, Transportation Research Board, Washington, D.C..

Lynn, A. (1999). “Seismic Evaluation of Existing Reinforced Concrete Building Colums.” Ph.D. Thesis, University of California at Berkeley.

Matamoros, A.B. (1999). “Study of Drift Limits for High-Strength Concrete Columns.” Department of Civil Engineering, University of Illinois at Urbana-Champaign.

Mo, Y.L., and Wang, S.J. (2000). “Seismic Behavior of RC Columns with Various Tie Configurations.” *J. Struct. Eng.*, 126(10), 1122-1130.

Moyer, M., and Kowalsky, M. (2001). “Influence of Tension Strain on Buckling of Reinforcement in RC Bridge Columns.” Department of Civil Engineering, North Carolina State University, Raleigh, North Carolina.

Muguruma, H., Watanabe, F., and Komuro, T. (1989). “Applicability of High Strength Concrete to Reinforced Concrete Ductile Column.” *Transactions of the Japan Concrete Institute*, 11.

Munro, I.R.M., Park, R., and Priestley, M.J.N. (1976). “Seismic Behaviour of Reinforced Concrete Bridge Piers.” *Report 76-9*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Nagasaka, T. (1982). “Effectiveness of Steel Fiber as Web Reinforcement in Reinforced Concrete Columns.” Transactions *of the Japan Concrete Institute*, 4.

Ng K. H., Priestley, M.J.N., and Park, R. (1978). “Seismic Behaviour of Circular Reinforced Concrete Bridge Piers.” *Report 78-14*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Nosho, K., Stanton, J., and MacRae, G. (1996). “Retrofit of Rectangular Reinforced Concrete Columns using Tonen Forca Tow Sheet Carbon Fiber Wrapping.” *Report No. SGEM 96-2*, Department of Civil Engineering, University of Washington, Seattle.

Nelson, Jared M. (2000). "Damage Model Calibration for Reinforced Concrete Columns." Master's Thesis, Department of Civil and Environmental Engineering, University of Washington, Seattle.

Ohno, T., and Nishioka, T. (1984). “An Experimental Study on Energy Absorption Capacity of Columns in Reinforced Concrete Structures.” *Proceedings of the JSCE, Structural Engineering/Earthquake Engineering*, 1(2).

Ohue, M., Morimoto, H., Fujii, S., and Morita, S. (1985). “The Behavior of R.C. Short Columns Failing in Splitting Bond-Shear Under Dynamic Lateral Loading.” *Transactions of the Japan Concrete Institute*, 7.

Ono, A., Shirai, N., Adachi, H., and Sakamaki, Y. (1989). “Elasto-Plastic Behavior of Reinforced Concrete Column With Fluctuating Axial Force.” *Transactions of the Japan Concrete Institute*, 11.

Park, R., and Paulay, T. (1990). “Use of Interlocking Spirals for Transverse Reinforcement in Bridge Columns.” *Strength and Ductility of Concrete Substructures of Bridges, RRU (Road Research Unit) Bulletin 84*, 1, 77-92.

Paultre, P., Legeron, F., and Mongeau, D. (2001). “Influence of Concrete Strength and Transverse Reinforcement Yield Strength on Behavior of High-Strength Concrete Columns.” *ACI Struct. J.*, 98(4), 490-501.

Petrovski, J., and Ristic, D. (1984). “Reversed Cyclic Loading Test of Bridge Column Models.” *Report IZIIZ 84-164*, Institute of Earthquake Engineering and Engineering Seismology.

Pontangaroa, R.T., Priestley, M.J.N., and Park, R. (1979). “Ductility of Spirally Reinforced Concrete Columns Under Seismic Loading.” *Report 79-8*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Priestley, M.J.N., and Benzoni, G. (1996). “Seismic Performance of Circular Columns with Low Longitudinal Reinforcement Ratios.” *ACI Struct. J.*, 93(4), 474-485.

Roeder C., Soderstom, and Graph (2001). "Seismic Performance of Pile-Wharf Connections", Pacific Earthquake Engineering Research Center, PEER.

Pujol, S. (2002). "Drift Capacity of Reinforced Concrete Columns Subjected to Displacement Reversals." Thesis, Purdue University.

Saatcioglu, M., and Ozcebe, G. (1989). “Response of Reinforced Concrete Columns to Simulated Seismic Loading.” *ACI Struct. J.*, 86(1), 3-12.

Saatcioglu, M., and Baingo, D. (1999). “Circular High-Strength Concrete Columns Under Simulated Seismic Loading.” *J. Struct. Eng.*, 125(3), 272-280.

Saatcioglu, M., and Grira, M. (1999). “Confinement of Reinforced Concrete Columns with Welded Reinforcement Grids.” *ACI Struct. J.*, 96(1), 29-39.

Sakai, Y., Hibi, J., Otani, S., and Aoyama, H. (1990). “Experimental Study on Flexural Behavior of Reinforced Concrete Columns Using High-Strength Concrete.” *Transactions of the Japan Concrete Institute*, 12.

Siryo K. K. (1975). "A seismic Analysis of Building Structural Members: a List of Experimental Results on Deformation Ability of Reinforced Concrete Columns Under Large Deflection (No.2)." Building Research Institute, Ministry of Construction, Japan.

Soesianawati, M.T., Park, R., and Priestley, M.J.N. (1986). “Limited Ductility Design of Reinforced Concrete Columns.” *Report 86-10*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Sritharan, S., Priestley, M.J.N., Seible F. (1996). "Seismic Response of Column/Cap Beam Tee Connections with Cap Beam Prestressing." *Structural Systems Research Project, Report No. ssrp-96/09*, University of California, San Diego.

Stone, W. C., and Cheok, G. S. (1989). “Inelastic Behavior of Full-Scale Bridge Columns Subjected to Cyclic Loading.” *NIST Building Science Series 166,* U.S. National Institute of Standards and Technology, Gaithersburg, Maryland.

Sugano, S. (1996). “Seismic Behavior of Reinforced Concrete Columns Which used Ultra-High-Strength Concrete.” *Eleventh World Conference on Earthquake Engineering, Paper No. 1383.*

Tanaka, H., and Park, R. (1990). “Effect of Lateral Confining Reinforcement on the Ductile Behavior of Reinforced Concrete Columns.” *Report 90-2*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Thomsen, J., and Wallace, J. (1994). “Lateral Load Behavior of Reinforced Concrete Columns Constructed Using High-Strength Materials.” *ACI Struct. J.*, 91(5), 605-615.

Umehara, H., and Jirsa, J.O. (1982). “Shear Strength and Deterioration of Short Reinforced Concrete Columns Under Cyclic Deformations.” *PMFSEL Report No. 82-3,* Department of Civil Engineering, University of Texas at Austin.

Vu, Nganha D., Priestley, M.J.N., Seible, F., and Benzoni, G. (1998). “Seismic Response of Well Confined Circular Reinforced Concrete Columns with Low Aspect Ratios.” *Proceedings of the 5th Caltrans Seismic Research Workshop,* Sacramento, California.

Watson, S. (1989). “Design of Reinforced Concrete Frames of Limited Ductility.” *Report 89-4,* Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Watson, S., and Park, R. (1989). “Design of Reinforced Concrete Frames of Limited Ductility.” *Report 89-4,* Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Wehbe, N., and Saiidi, M.S., and Sanders, D. (1998). “Confinement of Rectangular Bridge Columns for Modrate Seismic Areas.” *National Center for Earthquake Engineering Research (NCEER) Bulletin,* 12(1).

Wight, J.K., and Sozen, M.A. (1973). “Shear Strength Decay in Reinforced Concrete Columns Subjected to Large Deflection Reversals.” *Structural Research Series No. 403,* Civil Engineering Studies, University of Illinois, Urbana-Champaign.

Wong, Y.L., Paulay, T., and Priestley, M.J.N. (1990). “Squat Circular Bridge Piers Under Multi-Directional Seismic Attack.” *Report 90-4,* Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Xiao, Y., and Martirossyan, A. (1998). “Seismic Performance of High-Strength Concrete Columns.” *J. of Struct. Eng.,* 124(3), 241-251.

Xiao Y., and Yun., H. W. (2002). “Experimental Studies on Full-Scale High-Strength Concrete Columns.” ACI Struct. J., 99(2), 199-207.

Zahn, F.A., Park, R., and Priestley, M.J.N. (1986). “Design of Reinforced Bridge Columns for Strength and Ductility.” *Report 86-7*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Zhou, X., Higashi, Y., Jiang, W., and Shimizu, Y. (1985). “Behavior of Reinforced Concrete Column Under High Axial Load.” *Transactions of the Japan Concrete Institute*, 7.

Zhou, X., Satoh, T., Jiang, W., Ono, A., and Shimizo, Y. (1987). "Behavior of Reinforced Concrete Short Column Under High Axial Load." *Transactions of the Japan Concrete Institute,* 9.