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Research--Application for the Development of An Innovative Retrofit Scheme

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A Hybrid Pseudodynamic Testing Platform for Structural Engineering Research—Application for the Development of An Innovative Retrofit Scheme

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ABSTRACT

The objective of this study is to develop an integrated hybrid Pseudodynamic (PSD) testing platform that can be utilized to evaluate the behavior of subassemblies, building, bridges, and various infrastructures under earthquake, dynamic or static loads, avoiding some of the problems associated with shake table tests or pure computer modeling methods. Explicit and implicit algorithms that have been employed or proposed by other researchers in PSD testing are reviewed and evaluated in this study in terms of their numerical stability, accuracy, and error-propagation characteristics. The improved alpha-method is employed in the current testing platform owing to its characteristics of unconditional stability, sufficient accuracy, and having been verified experimentally.

A pre-testing simulation computer program is developed in this study. In this pre-testing simulation program, the restoring force of the virtual experimental substructure is calculated using ABAQUS instead of reading the value from physical tests. The pre-testing simulations for six representative cases were performed, showing that simulation results were consistent with the analytical solutions obtained through the finite element analysis program ABAQUS.

The current testing platform is developed in an object-oriented C++ framework architecture. The entire testing platform was verified experimentally by performing a series of physical tests of SDOF and MDOF systems. Two computers were used in the tests of MDOF systems. The two computers were used to control the physical test of the experimental specimen, and numerical modeling of the analytical substructure. A solution for data exchange between the two computers was developed and was verified.
experimentally. A series of PSD tests were performed to investigate the effects of the convergence tolerance, initial elastic stiffness, and selection of reduction factor. The test results indicate that the effects due to these parameters are related to the input excitation energy. The larger the input energy is, the smaller are such effects. A start-restart procedure is embedded in the current testing platform to handle unexpected situations that could occur during tests.

A 3-story special concentrically braced frame was tested utilizing the PSD testing platform developed in this study. The current testing platform was further verified experimentally by performing the PSD testing of a realistic building with special buckling-enhanced braces developed as part of this study. The special buckling-enhanced brace consists of an inner core steel double angle and outer pipes strapped to the angle with industrial steel straps. The braces in the first story of the building were tested physically while the rest of the building was modeled using ABAQUS. The buckling capacity of the special buckling-enhanced braces was found to be nearly 33% larger than that of standard double angle braces.
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TABLE OF CONTENTS

ABSTRACT ..................................................................................I

ACKNOWLEDGEMENTS ..............................................................IV

LIST OF CONTENTS ..................................................................V

LIST OF FIGURES .......................................................................XI

LIST OF TABLES .........................................................................XIX

NOTATIONS ................................................................................XX

Chapter 1 INTRODUCTION ..........................................................1

1.1 Background of Research .......................................................1

1.2 Literature Review .................................................................8

1.2.1 Development of PSD Testing Method .................................8

1.2.2 Applications of PSD Testing Method .................................12

1.3 Objective of Research ..........................................................14

1.4 Organization and Scope of Dissertation ...............................17

Chapter 2 NUMERICAL METHODS FOR HYBRID PSEUDODYNAMIC

TESTING ..................................................................................21

2.1 Introduction ..........................................................................21

2.2 Numerical Algorithms ..........................................................23

2.2.1 Explicit Methods .............................................................24

2.2.1.1 Central Difference Method ..........................................24
2.2.1.2 Newmark-β Method.................................................................26
2.2.1.3 Operator-Splitting (OS) Time Algorithm.................................28
2.2.1.4 Predictor-Corrector Algorithm..............................................31
2.2.2 Implicit Integration Method.....................................................32
  2.2.2.1 Alpha-Method.................................................................33
  2.2.2.2 Improved Alpha-Method...................................................34
2.3 Conclusions and Remarks..........................................................40

Chapter 3 DEVELOPMENT OF PRE-TESTING SIMULATION COMPUTER PROGRAM.................................................................42
  3.1 Introduction..............................................................................42
  3.2 Substructuring Technique.......................................................45
  3.3 Pre-testing Simulation Program Framework..................................48
    3.3.1 Controller Module............................................................51
    3.3.2 Virtual Experimental Module............................................52
    3.3.3 Numerical/Analytical Module............................................53
  3.4 Simulation Examples..............................................................54
    3.4.1 Case 1: Elastic Cantilever Column (SDOF).............................55
    3.4.2 Case 2: Elastic Frame with Hinge (Two DOFs).......................56
    3.4.3 Case 3: Elastic Frame without Hinge (Three DOFs)...............58
    3.4.4 Case 4: Nonlinear Cantilever Column (Nonlinear SDOF).........60
    3.4.5 Case 5: Nonlinear Frame with Hinge (Nonlinear Two DOFs)....61
    3.4.6 Case 6: Nonlinear Frame without Hinge (Nonlinear Three DOFs)63
3. 5 Conclusions and Remarks.................................................................64

Chapter 4 DEVELOPMENT OF AN INTEGRATED HYBRID PSEUDODYNAMIC TESTING PLATFORM...........................................78

4.1 Introduction....................................................................................78

4.2 Equipment Used in PSD Testing....................................................83

4.3 PSD Tests......................................................................................87

4.3.1 SDOF System.............................................................................87

4.3.1.1 Test Results for SDOF System..................................................87

4.3.1.2 Test Results of SDOF Cases under Harmonic Excitations........89

4.3.1.2.1 Effect of Corrector Stiffness...............................................89

4.3.1.2.2 Effect of Convergence Tolerance.........................................91

4.3.1.2.3 Effect of Reduction Factor................................................91

4.3.1.2.4 Summary for SDOF Cases under Harmonic Excitation...........92

4.3.1.3 Test Results of SDOF System under El Centro Excitation...........93

4.4 PSD Tests for MDOF Systems.......................................................94

4.4.1 Solution for Data Exchange between Computers..........................94

4.4.2 Test Setup for MDOF System.....................................................97

4.4.3 Test Results of MDOF System...................................................99

4.4.3.1 MDOF System with Elastic Analytical Substructure under Harmonic Excitations......................................................99

4.4.3.1.1 Effect of Corrector Stiffness.................................................99

4.4.3.1.2 Effect of Tolerance..........................................................100
Chapter 5 FULL-SCALE PSEUDODYNAMIC TESTS OF SPECIAL
CONCENTRICALLY BRACED FRAMES WITH BUCKLING-ENHANCED BRACES

5.1 Introduction

5.2 PSD Tests of Concentrically Braced Frames with Conventional Braces

5.2.1 Equivalent Static Force Design Procedure of Braced Frame

5.2.2 Test Set-up

5.2.3 Tests Results of Braced Frame with Conventional Braces

5.3 PSD Tests of Braced Frames with Buckling-Enhanced Braces

5.3.1 Reduced Design Spectrum and Excitation

5.3.2 Layout of Braced Frame with Buckling-Enhanced Braces

5.3.3 Details of Buckling-Enhanced Braces

5.3.4 Implementation of PSD Tests

5.3.5 Test Results of Braced Frame with Buckling-Enhanced Braces

5.3.5.1 Test Results under Reduced Earthquake Excitation

5.3.5.2 Deformation of Braces

5.3.5.3 Test Results for Building under Double Reduced Design Excitation

5.3.5.4 Test Results of Frame with Braces Enhanced with Composite Pipes under Reduced Design Excitation

5.3.5.5 Tests Results of Frame under 200% Reduced Design Excitation

5.4 Simplified Model of Buckling-enhanced Braces

5.5 Conclusions and Remarks
Chapter 6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary of Research

6.2 Conclusions and Remarks

6.3 Recommendations

REFERENCES
LIST OF FIGURES

Figure 1.1 Procedure for hybrid PSD testing...............................................................20
Figure 3.1.a Hybrid PSD testing framework..............................................................67
Figure 3.1.b PSD Pre-testing Simulation Framework..................................................67
Figure 3.2.a Model for cantilever column.................................................................68
Figure 3.2.b Ground excitation for cantilever column..............................................68
Figure 3.3 Displacement of cantilever steel column subjected to ground excitation......69
Figure 3.4.a Models of substructures for frame with hinge on left corner...............70
Figure 3.4.b Ground excitation for frame with hinge on left corner.........................70
Figure 3.5.a Horizontal displacement of frame with hinge on left corner.................71
Figure 3.5.b Rotation on right corner of frame with hinge on left corner...............71
Figure 3.6.a Experimental and analytical substructure models for frame without hinge..72
Figure 3.6.b Ground excitation for frame without hinges........................................72
Figure 3.7(a) Horizontal displacement for frame without hinge...............................73
Figure 3.7.b Rotation on left corner for frame without hinge.................................73
Figure 3.7.c Rotation on right corner for frame without hinge...............................74
Figure 3.8 Deformation model for nonlinear material.............................................74
Figure 3.9 Response for nonlinear cantilever column.............................................75
Figure 3.10.a Horizontal displacement for nonlinear frame with hinge on left corner....75
Figure 3.10.b Rotation on right corner for nonlinear frame with hinge on left corner...76
Figure 3.11.a Horizontal displacement for nonlinear frame without hinge...............76
Figure 3.11.b Rotation on left corner for nonlinear frame without hinge.................77
Figure 3.11.c Rotation on right corner for nonlinear frame without hinge………………77
Figure 4.1 Imposition of displacement to experimental specimen………………..……117
Figure 4.2.a Test setup for PSD testing of simply supported beam…………………..……117
Figure 4.2.b Structural model for PSD testing of simply supported beam……………….……118
Figure 4.3 Test results of SDOF system using different corrector stiffness……………….……118
Figure 4.4 Tests results of SDOF system using different tolerances…………………..……119
Figure 4.5 Test results of SDOF system using different reduction factors…………………..……119
Figure 4.6 Test and simulation results of simple beam under harmonic excitation…….……120
Figure 4.7 Tests results of SDOF system under El Centro excitation………………..……120
Figure 4.8 Data exchange solution between computers for hybrid PSD testing………………121
Figure 4.9.a Structures for hybrid PSD tests of MDOF systems…………………..……122
Figure 4.9.b Setup for experimental substructure……………………………………….……123
Figure 4.10.a Displacement of MDOF system with elastic analytical substructure under
harmonic excitation varying corrector stiffness…………………………………….……124
Figure 4.10.b Rotation of MDOF system with elastic analytical substructure under
harmonic excitation varying corrector stiffness…………………………………….……124
Figure 4.11.a Displacement of MDOF system with elastic analytical substructure under
harmonic excitation with different tolerances…………………………………….……125
Figure 4.11.b Rotation of MDOF with elastic analytical substructure under harmonic
excitation with different tolerances…………………………………….……125
Figure 4.12.a Displacement of MDOF system with elastic analytical substructure under
harmonic excitation using different reduction factors………………………….……126
Figure 4.12.b Rotation of MDOF system with elastic analytical substructure under harmonic excitation using different reduction factors……………….….126

Figure 4.13.a Tests results of displacement for MDOF system with elastic analytical substructure under harmonic excitation…………………………………127

Figure 4.13.b Tests results of rotation for MDOF system with elastic analytical substructure under harmonic excitation………………………….…..….127

Figure 4.14.a Displacement of MDOF with elastic analytical substructure under earthquakes using different tolerances…………………………………….…..128

Figure 4.14.b Rotation of MDOF with elastic analytical substructure under earthquakes using different tolerances…………………………………….…..128

Figure 4.15.a Displacement of MDOF system with elastic analytical substructure under earthquakes using different reduction factors……………………129

Figure 4.15.b Rotation of MDOF system with elastic analytical substructure under earthquakes using different reduction factors……………………129

Figure 4.16.a Displacement for MDOF system with elastic analytical substructure under earthquakes using different corrector stiffness……………….…..130

Figure 4.16.b Rotation of MDOF system with elastic analytical substructure under earthquakes using different corrector stiffness……………….…..130

Figure 4.17.a Displacement for MDOF with elastic substructure under earthquakes….131

Figure 4.17.b Rotation for MDOF with elastic substructure under earthquakes………..131

Figure 4.18 Deformation model for nonlinear material…………………………………….…..132

Figure 4.19.a Displacement of MDOF system with nonlinear substructure using different tolerances………………………………………………………….…..132
Figure 4.19.b Rotation of MDOF system with nonlinear substructure using different tolerances…………………………………………………………………………………133

Figure 4.19.c Restoring force versus displacement of shear frame using different tolerances……………………………………………………………………………………………………………………..…….133

Figure 4.20.a Displacement for MDOF system with nonlinear substructure using different reduction factors…………………………………………………………………………………………………………………………………………..…….134

Figure 4.20.b Rotation for MDOF system with nonlinear substructure using different reduction factors…………………………………………………………………………………………………………………………………………..…….134

Figure 4.20.c Response of nonlinear analytical substructure using different reduction factors…………………………………………………………………………………………………………………………………………..…….135

Figure 4.21.a Displacement for MDOF system with nonlinear substructure using different corrector stiffness…………………………………………………………………………………………………………………………………………..…….135

Figure 4.21.b Rotation for MDOF system with nonlinear substructure using different corrector stiffness…………………………………………………………………………………………………………………………………………..…….136

Figure 4.21.c Response of nonlinear analytical substructure using different corrector stiffness……………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………………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Figure 4.23 Status of computers after termination of test………………………………………..138

Figure 4.24.a Displacements obtained from complete test and start-restart test……….139
Figure 4.24.b Rotations obtained from complete test and start-restart test............139
Figure 4.25 Combinations of El Centro and harmonic excitations.....................140
Figure 4.26.a Displacement of MDOF system under combined excitations..........140
Figure 4.25.b Rotation of MDOF system under combined excitations...............141
Figure 5.1 Typical buckling-restrained brace.............................................173
Figure 5.2 Design spectrum......................................................................173
Figure 5.3 Computer model of 3-Story SAC braced frame.........................174
Figure 5.4.a Structural model of 3-story SAC building...............................174
Figure 5.4.b Analytical substructure...........................................................175
Figure 5.4.c Experimental substructure......................................................175
Figure 5.5 Connection detail at ends of double angle brace.........................176
Figure 5.6.a Top connection detail of brace.................................................177
Figure 5.6.b Top connection of brace..........................................................178
Figure 5.7.a Bottom connection detail of brace............................................178
Figure 5.7.b Floor beams and actuators.......................................................179
Figure 5.7.c Bottom pin connection of brace..............................................179
Figure 5.8 Earthquake excitation...............................................................180
Figure 5.9 Comparison of design spectrum and generated spectrum.............180
Figure 5.10 Restoring forces versus displacements of braces.......................181
Figure 5.11.a Deformations of tension braces during test............................181
Figure 5.11.b Deformations of compression braces during test....................182
Figure 5.12 Reduced design spectrum......................................................182
Figure 5.13 Reduced earthquake excitation..............................................183
Figure 5.14 Comparison of reduced design spectrum and generated spectrum.............183
Figure 5.15 Structural model of braced frame with buckling-enhanced braces............184
Figure 5.16.a Typical buckling-enhanced brace.......................................................184
Figure 5.16.b Schematic cross section of buckling-enhanced braces.......................184
Figure 5.16.c Connection of angle at end of braces.................................................185
Figure 5.17.a Horizontal displacement on the first story........................................185
Figure 5.17.b Inter-story drift between the first and second stories.........................186
Figure 5.17.c Inter-story drift between the second and third stories........................186
Figure 5.18.a Locations of strain gages along length of brace A............................187
Figure 5.18.b Locations of strain gages on cross section of brace A........................187
Figure 5.18.c Buckling shape of brace during test..................................................188
Figure 5.19 Strains measured in brace A...............................................................188
Figure 5.20 Strain measured in pipe of brace A......................................................189
Figure 5.21 Locations of strain gages along length of brace B...............................189
Figure 5.22 Strains measured in brace B...............................................................190
Figure 5.23 Restoring force versus displacement of braces under reduced excitation...190
Figure 5.24.a Displacement on the first story.........................................................191
Figure 5.24.b Inter-story drift between the first and second stories........................191
Figure 5.24.c Inter-story drift between the second and third stories........................192
Figure 5.25 Strains measured in brace A in 200% excitation case..............................192
Figure 5.26 Strain measured in pipe of brace A in 200% reduced design excitation.....193
Figure 5.27 Strains measured in brace B for case of 200% reduced design excitation...193
Figure 5.28 Restoring force versus displacement of braces for case of 200% reduced design excitation………………………………………………………194

Figure 5.29 Schematic cross section of braces with composite pipes…………………..194

Figure 5.30.a Displacement of the first story of building with braces enhanced with composite pipes………………………………………………………..195

Figure 5.30.b Inter-story drift between the second and first stories of building with braces enhanced with composite pipes……………………………………195

Figure 5.30.c Inter-story drift between the third and second stories of building with braces enhanced with composite pipes…………………………………..196

Figure 5.31.a Comparison of displacement on the first floor of buildings with different braces………………………………………………………………196

Figure 5.31.b Comparison of inter-story drifts between the first and second stories of buildings with different braces…………………………………………197

Figure 5.31.c Comparison of inter-story drifts between the second and third stories of buildings with different braces………………………………………..197

Figure 5.32.a Strains of the angles in brace A enhanced with composite pipes for reduced design excitation case………………………………………………198

Figure 5.32.b Strain of the pipe in brace A enhanced with composite pipes for reduced design excitation case………………………………………………198

Figure 5.33.a Strain of the angles in brace B enhanced with composite pipes for reduced design excitation case………………………………………………199

Figure 5.33.b Strain of the pipes in brace B enhanced with composite pipes for reduced design excitation case………………………………………………199

XVII
Figure 5.34  Restoring force versus displacement for braces enhanced with hollow and composite pipes for reduced design excitation case......................200

Figure 5.35.a Displacement of the first story of building with braces enhanced with composite pipes.................................................................200

Figure 5.35.b Inter-story drift between the second and first stories of building with braces enhanced with composite pipes............................................201

Figure 5.35.c Inter-story drift between the third and second stories of building with braces enhanced with composite pipes.................................201

Figure 5.36.a Comparison of displacement on the first story of building with braces enhanced with composite pipes.................................202

Figure 5.36.b Comparison of inter-story drift between the second and first stories of building with braces enhanced with composite pipes..............202

Figure 5.36.c Comparison of inter-story drift between the third and second stories of building with braces enhanced with composite pipes......................203

Figure 5.37 Strains measured in angles of brace \( A \) in 200% design excitation case......203

Figure 5.38.a Strains measured in angles of brace \( B \) in 200% design excitation case...204

Figure 5.38.b Strain of composite pipe in brace \( B \) in 200% design excitation case......204

Figure 5.39 Restoring force versus displacement for braces with hollow and composite pipes in 200% reduced design excitation case.................................205

Figure 5.40 Forces of two free bodies..........................................................206

Figure 5.41 Two free Bodies with equivalent forces........................................206

Figure 5.42 Structural model for buckling-enhanced braces...............................206
LIST OF TABLES

Table 1.1 Existing PSD testing sites in the U.S.................................20
Table 3.1 Steps for substructuring PSD tests.................................66
Table 4.1 Parameters used in tests of SDOF cases...........................115
Table 4.2 Parameters used in tests of MDOF for SB harmonic cases.....115
Table 4.3 Parameters used in tests of MDOF for SB-EC cases..........116
Table 4.4 Parameters for cases with nonlinear analytical substructures.116
Table 5.1 Test results of concrete cylinders.................................172
NOTATIONS

Chapters 1-3

\( r^e \)  Restoring force of experimental substructure

\( r^a \)  Restoring force of analytical substructure

\( \ddot{d}^e \)  Target displacement of experimental substructure

\( \ddot{d}^a \)  Target displacement of analytical substructure

\( M \)  Mass

\( C \)  Damping

\( R \)  Restoring force

\( F \)  External excitation force

\( K \)  Stiffness

\( \ddot{d}(t) \)  Time-dependent acceleration

\( \dot{d}(t) \)  Time-dependent velocity

\( d(t) \)  Time-dependent displacement

\( T_n \)  Structural periods

\( \Delta t \)  Integration time interval

\( D \)  Displacement

\( v \)  Velocity

\( a \)  Acceleration

\( K_i \)  Initial elastic stiffness

\( f_i \)  External excitation at time step \( i \)

\( R_A \)  Restoring force at the DOFs of the analytical substructure
$R_E$ Restoring force at the DOFs of the experimental substructure

$R_I$ Restoring force at the interface DOFs of two substructures

$F_A$ External excitation force at the DOFs of the analytical substructure

$F_E$ External excitation force at the DOFs of the experimental substructure

$F_I$ External excitation force at the interface DOFs of two substructures

$V_A$ Velocity at the DOFs of the analytical substructure

$V_E$ Velocity at the DOFs of the experimental substructure

$V_I$ Velocity at the DOFs of the interface of substructures

$a_A$ Acceleration at the DOFs of the analytical substructure

$a_E$ Acceleration at the DOFs of the experimental substructure

$a_I$ Acceleration at the interface DOFs of two substructures

$d^C$ Command displacement

$d^{EX}$ External displacement

$L$ Length

$E$ Elastic Young’s modules

$\omega$ Natural frequency

$TOL$ Tolerance

$K^E$ Stiffness of the experimental substructure

$K^a$ Stiffness of the analytical substructure

$d_1$ Displacement of the first DOF

$d_2$ Displacement of the second DOF

$d_3$ Displacement of the third DOF

$\alpha$ Parameter in numerical methods
\( \beta \) Parameter in numerical methods
\( \gamma \) Parameter in numerical methods

**Chapter 4**

\( t^c_n \) Starting time for computation phase at the \( n^{th} \) time step

\( t^L_n \) Starting time for loading phase at the \( n^{th} \) time step

\( t^h_n \) Starting time for holding phase at the \( n^{th} \) time step

\( \Delta t^c \) Period of computation phase

\( \Delta t^L \) Period of loading phase

\( \Delta t^h \) Period of hold phase

\( E \) Elastic Young’s modulus

\( d_n \) Displacement at the \( n^{th} \) time step

\( d_{n+1} \) Displacement at the \( (n+1)^{th} \) time step

\( P(t) \) Time-dependent external excitation force applied to structures

\( Po \) Constant force applied to structures

\( M \) Mass

\( V \) Velocity

\( a \) Acceleration

\( d_{i+1} \) Displacement at the \( (i+1)^{th} \) step

\( d^k_{i+1} \) Displacement at the \( k^{th} \) iteration of the time step \( i+1 \)

\( \Delta d^k_{i+1} \) Displacement increment at the \( k^{th} \) iteration of the time step \( i+1 \)

\( r_{i+1} \) Restoring force at the time step \( i+1 \)
$r_{i+1}^k$  Restoring force at the $k^{th}$ iteration of the time step $i+1$

$K$  Stiffness parameter

$K_i$  Initial elastic stiffness

$TOL$  Tolerance

$RF$  Reduction factor

$E(t)$  North-south component of El Centro earthquake excitation

**Chapter 5**

$R$  Response modification coefficient

$\Omega_o$  System over-strength factor

$C_d$  Deflection amplification factor

$P_{cr}$  Critical buckling load of braces

$I$  Importance factor
Chapter 1 Introduction

1.1 Background of Research

Model-based simulation has been utilized for decades to evaluate the linear or nonlinear behavior of structures, but the confidence placed on the theoretical simulation is severely limited by the uncertainties associated with the simplifications in the modeling process. This situation may become worse for some special nonlinear structures or subassemblies such as composite shear walls and partially restrained beam-to-column connections where the nonlinear behavior of structures or subassemblies are quite complex and the related modeling procedure is very challenging.

For these reasons, physical testing is generally considered as the most direct and reliable approach to evaluate the response of these kinds of complex nonlinear structures under the static or dynamic loads. Usually, a shake table can be used as a direct means for physical testing of structures under dynamic loading. One problem related to this physical testing approach is that testing specimens often have to be scaled to a small size because of the limited size and capacity of commonly available shake tables. Uncertainties are placed on the dynamic response of structures because of the limited scalability of testing specimens. In addition, the behavior of the investigated subassemblies or regions, such as beam-to-column connections, may be difficult to observe during fast-moving shake table tests. Ironically, these subassemblies or structures usually play critical roles in ensuring the safety of the whole structure. A comprehensive and accurate understanding of their structural response in a large-scale or even full-scale test under various load cases is necessary.
On the other hand, thanks to the availability of a large body of experimental data and verified theoretical models, some simple structures such as steel trusses and frames can be modeled rather accurately in finite element programs. In order to address the shortcomings mentioned above while taking advantage of established techniques, one innovative testing approach, namely the hybrid pseudo-dynamic (PSD) testing, has been recently proposed in order to take advantage of the merits of both the finite element modeling for simple structures and the physical testing for special complicated subassemblies to study the static or dynamic behavior of large-scale or even full-scale structures (Takanashi et al., 1975).

The PSD testing is an integrated experimental-numerical hybrid testing procedure. With respect to its numerical aspects, it is similar to a standard step-by-step transient structural analysis in that finite element techniques are utilized to calculate the response of structures under an external excitation history, which is discretized into a series of time steps. Subassemblies such as steel trusses and frames that can be modeled with confidence are evaluated in a standard finite element program such as ABAQUS (Abaqus, 2003). With respect to its experimental procedure, large-scale or even full-scale subassemblies are tested physically due to their complicated nonlinear behavior and/or because of uncertainties associated with their behavior. Hydraulic actuators are used to impose the target displacements to the experimental subassemblies as in conventional quasi-static tests. At the same time, the restoring forces developed by the experimental subassemblies are read from the load cell of actuators and sent to the controlling program for the PSD testing.
The integrated numerical-experimental procedure for the PSD testing method is further illustrated in Figure 1.1, where $d^e$ and $d^a$ are the target displacements of the experimental and analytical substructures, respectively, and $r^e$ and $r^a$ are respectively the restoring forces developed in the experimental and analytical substructures. A typical PSD testing platform consists of three main elements: the PSD controlling program, the experimental testing engine, and the simulation engine. In PSD testing, the whole structure is divided into an experimental substructure and an analytical/numerical substructure. With the given initial displacements and velocities, the governing equation of motion of the whole structure is solved numerically using the PSD controlling program to determine the displacement at the end of the first time step for both experimental and numerical substructures. Once the displacements are determined, the displacement of each degree of freedom for the experimental substructure is quasi-statically imposed to the experimental specimens using hydraulic actuators and then the restoring forces of the specimens are measured and sent back to the PSD controlling program. Meanwhile, the displacement of each degree of freedom for the numerical subassemblies is applied in a standard finite element program such as ABAQUS, and then the restoring forces of the numerical subassemblies are computed by ABAQUS and sent back to the PSD controlling program. In the PSD controlling program, the restoring forces from the experimental and the numerical subassemblies are incorporated into the governing equation of motion of the whole structure using the substructuring technique, and then the governing equation of motion of the whole structure is solved again numerically to determine the displacement for both the experimental and simulation subassemblies for the next step. This process is repeated iteratively until the complete dynamic response is
evaluated. It can be noted that hybrid PSD testing of structures occurs in a slowed-down timeframe. This implies that local damage, crack occurrence and other behaviors in areas of special interest can be observed during the tests. It can also be noted that the load is quasi-statically applied to the experimental specimens. This implies that potentially a much larger load can be applied to the specimens than in shake table tests where the load is applied through fast-moving actuators. This makes it possible to test large-scale or even full-scale specimens.

Hybrid PSD testing integrates the strengths of both the modeling simulation and physical testing methods. The merits of the hybrid testing approach can be further summarized as follows:

1. Reducing uncertainties associated with limited scalability of shake table tests. In hybrid PSD testing, the load is quasi-statically applied to the experimental specimen and therefore a big force can be applied to large-scale specimens through hydraulic actuators. On the contrary, in general shake table tests, the dynamic loading is applied to the structure through the shake table and the size of the structure is therefore limited by the capacity of the available shake table.

2. Mitigating errors related to simplified theoretical modeling of complex nonlinear structures or subassemblies. It is not a trivial task to accurately model complex nonlinear structures. Usually, to model a nonlinear structure, a series of assumptions have to be made to simplify the modeling procedure at the cost of modeling accuracy and reliability. Physical tests of nonlinear structures performed using the hybrid PSD testing approach provide a direct and reliable means to evaluate behaviors of complex nonlinear structures or subassemblies.
3. Obtaining first-hand experimental data for some special nonlinear complex structural subassemblies such as composite shear walls and partially restrained beam-to-column connections. It is difficult to accurately model these types of special structural subassemblies. Their structural behaviors under dynamic or earthquake loading are hard to observe in fast shake table tests. The structural performances such as the failure pattern and initiation of cracks in a special structural region such as beam-to-column connections can be closely investigated in PSD testing since it is a slowed-down testing procedure where loads are quasi-statically applied to the experimental specimens over a greatly extended timescale.

4. Saving construction/fabrication costs and time for tests. In conventional testing methods, the construction or fabrication of a whole structure is necessary and it is an expensive and time-consuming process for a physical test. In hybrid PSD testing, only a limited portion of the structure is constructed or fabricated owing to the fact that only a special part of the structure, instead of the whole structure, is tested physically.

Although the PSD hybrid testing approach has attracted more and more attention in evaluating the dynamic performance of structures as an innovative method, some challenges related to the testing software and hardware are still to be tackled. They are summarized as follows:

1. Experimental errors can be induced by the finite resolution of measuring and control devices. In hybrid PSD testing, the excitation history is divided into a series of time steps. The target displacements applied to the experimental and
analytical substructures at each step are dependent on the displacements and restoring forces obtained in previous steps. The measuring errors occurred at any step may accumulate and propagate to the next step as pointed out by Shing and Mahin (1987) and Shing and Manivannan (1990). However, the measuring and control errors are unavoidable in any physical tests. They usually result from systematic or random experimental errors because of the finite resolution of measuring and control devices. If a tangible error occurs at each step during the test, it may contaminate the test result and even cause the failure of a test. To mitigate the error effect caused by control and measuring devices, an effective means is to improve the resolution of the testing hardware and therefore to make sure that the errors related to the hardware can be limited.

2. Numerical errors may be introduced because of the approximate formulations employed in the numerical integration schemes. In hybrid PSD testing, the governing equation of motion is solved by employing different numerical integration algorithms. Since the governing equation of motion of nonlinear structures is a nonlinear differential equation, approximate expressions of the velocity, acceleration, and displacement have to be used. This is why the stability and accuracy characteristics of numerical integration algorithms have been extensively investigated since the finite element method was introduced to the engineering field around 50 years ago. If conditionally stable explicit numerical integration algorithms are employed, a small integration step size has to be chosen in order to ensure numerical stability. This implies that an excitation history has to be divided into a large number of small time steps. The increase of time steps
implies more chances to induce experimental errors in the test. If unconditionally stable implicit numerical algorithms are employed, the stability of the numerical simulation can be ensured unconditionally and a bigger time interval can be chosen to reduce the chances of introducing experimental errors in the test. However, one problem associated with the implicit algorithms is that the solution procedure of implicit algorithms needs to use iterative procedures and the instantaneous stiffness of the structure is required by the iterative procedures. Unfortunately, the instantaneous stiffness of structures almost cannot be measured during the test and the initial elastic stiffness has to be utilized. This may induce numerical errors during the test while replacing the instantaneous stiffness with the initial elastic stiffness.

3. Limitations posed in testing of some special structures. In PSD testing, structures are discretized into many elements so that the dynamic equilibrium equation of the structure can be constructed and the implicit or explicit numerical algorithms can be used to solve the governing equation of motion. For some structures that require a fairly refined spatial distribution of finite elements in order to accurately and realistically capture their dynamic behaviors, such as concrete dams which have a fairly uniform mass distribution, the test may require a large number of actuators as pointed by Shing et al. (1996). This will significantly hamper the realistic application of this method in testing of this special kind of structures. In addition, the behavior of some structures, for which the nonlinear response is sensitive to strain rate, can be reproduced only with real-time PSD testing. In PSD testing, hydraulic actuators are used to quasi-statically apply the target
displacement to the corresponding degree-of-freedom in an extended timescale. For some structural materials which are sensitive to strain rate, their nonlinear behavior measured in an extended timescale may be different from the situation under the realistic earthquakes.

1.2 Literature Review

1.2.1 Development of PSD Testing Method

Takanashi et al. (1975) presented a computer-controlled testing procedure to evaluate the dynamic performance of nonlinear structures without using a shake table. In their test, the restoring forces of structures were measured from the load cells of hydraulic actuators, instead of from simulations as in the conventional theoretical modeling methods. To eliminate the necessity of measuring the instantaneous tangent stiffness of structures, which is very difficult to perform during the test, the central difference algorithm was employed to solve the governing equation of motion. Because of the simplicity of this explicit algorithm, almost all ensuing on-line computer-controlled tests performed in Japan employed this numerical algorithm, as summarized by Takanashi et al. (1987).

Since the inception of the PSD testing method, a variety of numerical algorithms have been developed to improve the efficiency and accuracy of this innovative testing method. Mahin and Shing (1985) performed the first PSD test in the U.S. using the explicit Newmark algorithm (Newmark, 1959) by taking the Newmark parameters $\beta=0$ and $\gamma=0.5$. Considerable further efforts have been made to evaluate the stability and accuracy characteristics of this testing method (Shing and Mahin, 1987a; Shing and Mahin, 1987b;
Mahin et al., 1989; Shing and Manivannan, 1990; and Peek and Yi, 1990a, b). At the early development stage, explicit integration algorithms were usually employed in tests mainly because of two reasons that hindered the application of implicit algorithms in PSD testing: 1) the instantaneous tangent stiffness of structures is required by the iterative procedure in the implicit algorithms, but it is not a trivial task to perform the measurement of stiffness during the test and may not even be possible; and 2) the iterative procedure related to the implicit algorithms may induce experimental errors because of the loading reversals/hysteresis for nonlinear structures which are usually sensitive to the deformation history.

However, some problems associated with explicit numerical algorithms also introduced tough challenges. The explicit algorithms are conditionally stable and therefore pose a severe limitation on the integration step size, especially for stiff multi-degree-of-freedom (MDOF) systems where the fundamental natural period is usually quite small. It can be further explained as follows: as a small integration time step is chosen in order to meet the conditional stability requirement, the displacement increment at each time step may become relatively small and therefore the finite resolution of transducers used to control the actuators may prevent such small displacement increments from being applied correctly. In addition, the explicit algorithms usually do not introduce a positive numerical damping to suppress the experimental errors.

To avoid the conditional stability problem associated with explicit numerical algorithms, Thewalt and Mahin (1987) presented an implementation procedure for implicit integration algorithms. The implicit alpha-method (Hilber et al., 1977; Hilber et al., 1978; and Hughes, 1983) was employed in their test. To avoid the loading
 reversals/hysteresis problem related to the numerical iterative procedures, an analogue electronic device was used in their test to continuously update the instantaneous restoring force developed in the experimental specimen, instead of using the numerical iterative procedure. Shing and Manivannan (1990) further compared the implicit alpha-method with the central difference and Newmark explicit algorithm in terms of numerical accuracy, stability and error-propagation characteristics, revealing that the implicit alpha-method had a much smaller error-propagation effect than both the Newmark explicit algorithm and the central difference method.

Based on the implicit alpha-method, Shing et al. (1990a, b) proposed an improved procedure to mitigate the effect of loading reversals/hysteresis by introducing a reduction factor, without using the analogue electronic device as in the test performed by Thewalt and Mahin (1987). Shing et al. (1991) further evaluated the stability and accuracy characteristics of this improved implicit alpha-method in terms of error propagation and numerical convergence characteristics, demonstrating that the improved alpha-method had smaller error propagation effects than both the Newmark explicit algorithm and the central difference method.

With the development of the PSD testing method, more and more numerical algorithms have been employed or proposed. The operator-splitting (OS) algorithm (Hughes and Liu, 1978; and Hughes et al., 1979) was employed by Nakashima et al. (1990) in the test of a 3-story base-isolated building frame. This implicit-explicit algorithm was originally proposed by Hughes and Liu (1978) to handle some special finite element simulation situations where different families of finite elements were required to model different regions of a structure. In the test performed by Nakashima et
al. (1990), the base isolator was physically tested, and the remainder of the 3-story frame was modeled numerically. Combescure and Pegon (1997) further reviewed this explicit-implicit algorithm and pointed out that the numerical stability of this algorithm would be impaired if it is used to test some structures with severe stiffness softening. This algorithm was employed by Hashah et al. (2004) to test a steel frame which did not experience significant stiffness softening. Wang et al. (2001) further pointed out that this algorithm lacks sufficient accuracy when the ratio of the numerical integration time step over the fundamental natural period is larger than or equal to 0.05, implying that this algorithm suffers from the limitation of choosing the time step size.

In addition, Zhang et al. (2005) proposed the predictor-corrector explicit algorithm whereas this algorithm has not been employed in any tests so far. They proposed a complicated procedure to improve the stability and accuracy characteristics of this conditionally stable algorithm. Based on the alpha-method, Bonelli and Bursi (2004) developed a testing procedure for the generalized-alpha algorithm and they performed some physical tests to investigate the accuracy and stability characteristics of this algorithm, indicating that the accuracy and stability characteristics of this algorithm are dependent on the chosen factors in the formulations. This algorithm has not been employed by any other researchers so far due to the possible reason that it involves too many complicated formulations and therefore it is not easy to perform in tests.

In the U.S., PSD testing platforms have been built in four laboratories nationwide as indicated in Table 1.1 (Hashash et al., 2004; Shing et al., 2004; Ricle et al., 2004; and Stojadinovic et al., 2006). The conditionally stable explicit numerical methods are employed in these PSD testing platforms. As stated previously, explicit algorithms will
encounter numerical stability problem when stiff MDOF systems are tested. With respect to the analytical engine, the finite element program OPENSEES (OPENSEES, 2004) is used to evaluate the analytical substructure in the platforms developed at University of California at Berkeley and University of Colorado. Unfortunately, the start-restart procedure has not been incorporated into OPENSEES to the author’s best knowledge. However, the start-restart procedure is a necessary tool to evaluate general nonlinear materials in a multiple-step analysis. The PSD testing platform developed at the University of Illinois at Urbana-Champaign (UIUC) uses ABAQUS as the analytical engine. ABAQUS possesses the start-restart capability and therefore this testing platform is capable to handle general nonlinear analytical substructures. All of these platforms can deal with situations with multiple degrees of freedom. In the testing platform reported in this report, ABAQUS is used as the analytical engine and therefore it can handle general nonlinear analytical substructures. Different from that developed at UIUC, the unconditionally stable Alpha-method is incorporated, making the reported testing platform capable of testing stiff MDOF systems without any numerical stability problems.

1.2.2 Applications of PSD Testing Method

In the early development stages of PSD testing, this innovative testing method was usually used for testing of simple structures or buildings. Seki et al. (1988) tested a one-bay two-story steel concentrically braced frame to investigate the earthquake response of this kind of common structures, indicating that the shear yielding of the brace shear panel can dissipate significant energy through nonlinear deformation. Muto (1988) tested a one-
bay three-story braced frame in the scale of 1/5 to compare the performances of different braces. In their test, three kinds of braces were tested while the dimensions and sizes of the frame were remained the same. Sattary and Wright (1984) tested a full-scale seven-story building using the PSD testing method at the Building Research Institute in Japan. A full-scale five-story masonry building was physically tested by Seible (1994) using the PSD testing method. Because this test is greatly slowed-down with the aid of PSD testing, the cracks propagation patterns in walls were easily marked and responses at each floor were successfully recorded.

All these computer-controlled tests were performed to investigate entire structures or buildings. Tests of entire structural models are often expensive and require special large-scale testing facilities. Even so, such tests are generally unable to account for some factors such as soil-structure interaction (Mahin et al., 1989). The motivation and objective to develop the PSD testing method basically are to test the special subassemblies of interest of the structure, instead of the entire structure. Dermitzakis and Mahin (1985) introduced the substructuring technique into the PSD testing. In their test, the experimental substructure, which was a column, was physically tested while the remainder of the steel frame was modeled. The application of the substructuring technique in PSD testing opened a viable way to test large-scale or even full-scale structures without using special large-scale testing facilities. With the aid of substructuring techniques, Shing et al. (1994) tested a concentrically braced frame where the first story of a three-story building was physically tested and the second and third stories of the building were numerically modeled. The experimental results showed that the PSD testing method can provide reliable testing results for performance evaluation of
structures under earthquake loads. Chung et al. (1999) tested a one-bay 3-story base-isolated building where the superstructure of the building was modeled numerically and the base isolator was tested physically using two actuators. In their test, the seismic performance of the building with base-isolators was evaluated. The test results obtained through the PSD testing method were further compared against those obtained through shake table tests, indicating that they were in good agreement. A similar test was performed by Kim and Lee (1995) to investigate a base-isolated liquid storage tank. The effects of base isolators in reducing the earthquake response of this special kind of structures were evaluated in their PSD test. Kwon et al. (2005) presented a PSD test of a four-span bridge. There were three piers in this bridge and two ends were supported by rollers (simply supported boundary conditions). In their test, the deck and the middle pier of the bridge were modeled using a standard finite element program, and the other two piers were physically tested. This bridge was a part of the Santa Monica Freeway (I-10) that was damaged in 1994 Northridge earthquake. Their PSD test results were consistent with the simulation.

1.3 Objectives of Research

The objective of this research is to build a hybrid pseudodynamic (PSD) testing platform at the University of Cincinnati Large-Scale Test Facility. To demonstrate the capability of the testing platform developed in the current study, three sets of tests were performed. The first set of tests was performed to investigate a simply supported beam, which represented a single-degree-of-freedom (SDOF) system. The second sets of tests involved a simple multi-degree-of-freedom (MDOF) case where a subassembly of the
entire structure was tested physically, and at the same time the remainder of the whole structure was modeled in the standard finite element program ABAQUS. The third set was performed for the PSD test of a realistic complex concentrically braced frame. The objectives of the present research are further detailed as follows:

1. A suitable numerical algorithm will be selected. As an initial step to develop hybrid PSD testing capability at the University of Cincinnati, an extensive literature review needs to be conducted to evaluate the numerical algorithms that have been employed or proposed by other researchers so far. Subsequently, a suitable algorithm will be chosen and implemented in the current study. The selected numerical algorithm should possess characteristics of unconditional stability, sufficient accuracy, ease of performing in tests, as well as it should have been verified experimentally. The implementation procedure of the selected numerical algorithm will be embedded in the testing platform that will be developed in the current study.

2. A pre-testing simulation computer program will be developed. In the pre-testing simulation program, the experimental substructure will be modeled in the finite element program ABAQUS, instead of being tested physically. Pre-testing simulations prior to the test can provide the expected response of structures and thus help outline the testing process and equipment required, as well as convergence tolerances and expected test loads. By comparing the pre-testing simulation results with those obtained through ABAQUS, the entire numerical procedure embedded in the testing platform can be verified. Furthermore, by performing pre-testing simulations, the communication between the PSD
controlling program and the finite element program ABAQUS can be checked prior to real tests.

3. An object-oriented PSD testing platform will be developed. This testing platform will be developed in an object-oriented C++ framework where each objective will be materialized through an individual program element. All program elements will be verified by performing a series of tests. Firstly, this testing platform will be employed to test the simplest case, i.e., a simple steel beam which represents a SDOF system. By performing this series of tests, the communication between the testing computer where the testing platform is installed and the MTS will be verified experimentally. Secondly, the capability of the testing platform will be further demonstrated by testing a steel frame where the experimental substructure is a simply supported beam, and the remainder of the complete steel frame is the analytical substructure which will be modeled in ABAQUS. The steel frame represents a MDOF system. By performing the second set of tests, the entire testing platform will be completely verified experimentally, including the communication between the testing computer and the simulation computer where ABAQUS will be installed. Parametric investigations will be performed to evaluate the accuracy, stability and reliability of the testing platform developed in the current study.

4. A realistic complex system will be tested utilizing the PSD testing platform developed in the current study. The testing platform will be utilized to test a complex concentrically braced frame. In this test, the experimental substructure will consist of two braces which will experience complex nonlinear deformation,
and the analytical substructure will be the remainder of the steel braced frame. Two braces that will be tested are supposed to be in the first story of the frame. When the frame is subjected to the ground motion, one brace will be under tension and the other under compression. The PSD testing platform will be comprehensively checked by performing this test.

5. The performance of special braces will be evaluated using the PSD testing platform. A special retrofit system will be developed in the current study to increase the buckling capacity of braces. This special brace consists of a core steel angle and an outer pipe strapped to the angle. During the earthquake, only the core angle will resist the tensile load when the brace is under tension while both the core angle and the outer strapped pipe will resist the compression load when the brace is under compression. The seismic performance of the 4-bay 3-story braced frame including this special type of braces will be evaluated by analyzing the test results obtained through the PSD tests. To further increase the buckling capacity of this special brace, the outer strapped pipe will be filled with concrete. Some recommendations for furthering the present research will be given based on the lessons and experiences from the current study.

1.4 Organization and Scope of Dissertation

This dissertation presents the development of hybrid PSD testing capability at the University of Cincinnati Large-Scale Test Facility. Chapter 1 presents the research background and a general outline of the current research. The general concepts of PSD testing and objectives of the current research are introduced. Chapter 2 mainly reviews
the numerical algorithms that have been employed or proposed so far by other researchers. Explicit and implicit algorithms are compared and evaluated in terms of the numerical stability and accuracy and error-propagation characteristics. The advantages and disadvantages of different algorithms are summarized and an algorithm suitable for the present research is then selected. The development of a pre-testing simulation computer program is described in Chapter 3. Pre-testing simulations for different linear and nonlinear structures are presented and further compared with the solutions obtained through the finite element program ABAQUS. Chapter 4 presents the development of the object-oriented PSD testing platform. The capability of the testing platform is comprehensively checked experimentally by performing PSD tests of SDOF and MDOF systems, respectively. The SDOF system is a simply supported steel beam. The MDOF system is a steel frame where the experimental substructure is a simply supported beam and the rest of the frame is the analytical substructure. To make the testing platform more general, the physical testing of the experimental substructure is controlled by a computer called the testing engine, and the simulation of the analytical substructure is controlled by another computer called the simulation engine. The communication between the testing and simulation engines is checked in the tests of the MDOF system. A series of tests are performed to investigate the effects induced by different tolerances and initial stiffness used during the test. To handle some unexpected situations during the test, a special procedure, namely the Start-Restart procedure, is developed in the current testing platform. This special procedure can be used to handle situations when the structure may experience unexpected responses. To further demonstrate the generality of the current testing platform, cases where the external excitation consists of irregular earthquake
ground motions and harmonic wave are tested utilizing the Start-Restart procedure developed in the current study. The application of the substructuring technique in hybrid PSD testing is also presented in Chapter 4. Chapter 5 presents the test of a realistic complex steel braced frame where a special type of braces is used to resist the lateral earthquake load in the first story of the building. The objective to develop this special type of braces is to increase the buckling capacity of braces and therefore reduce the inter-story drifts of buildings. The performance of this special type of braces is evaluated based on the test results obtained through the PSD testing. Two designs of the brace are evaluated in the test. In the first design, a double angle brace is stiffened with hollow pipes, while in the second design the pipes strapped to the double angle brace are filled with concrete in order to further increase the buckling capacity of braces.
Table 1.1 Existing PSD Testing Sites in the U. S.

<table>
<thead>
<tr>
<th>Algorithm</th>
<th>Analytical Engine</th>
<th>Real-time or Not</th>
</tr>
</thead>
<tbody>
<tr>
<td>UC Berkeley Explicit Newmark</td>
<td>OPENSEES</td>
<td>Yes</td>
</tr>
<tr>
<td>University of Illinois Explicit</td>
<td>ABAQUS</td>
<td>No</td>
</tr>
<tr>
<td>Operator-Splitting (OS)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>University of Colorado Alpha-</td>
<td>OPENSEES</td>
<td>Yes</td>
</tr>
<tr>
<td>method</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lehigh University Alpha-method</td>
<td>Standard matrix</td>
<td>No</td>
</tr>
<tr>
<td>University</td>
<td>method</td>
<td></td>
</tr>
</tbody>
</table>

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Integrated PSD Testing Platform

<table>
<thead>
<tr>
<th>PSD Controlling Program</th>
<th>Experimental Testing Engine (MTS Controller)</th>
<th>Numerical Modeling Engine (ABAQUS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$r^e$: restoring force of experimental substructure</td>
<td>$d^e$: displacement of experimental substructure</td>
<td>$r^a$: restoring force of analytical substructure</td>
</tr>
<tr>
<td>$d^e$: displacement of experimental substructure</td>
<td>$r^a$: restoring force of analytical substructure</td>
<td>$d^a$: displacement of analytical substructure</td>
</tr>
</tbody>
</table>

Figure 1.1 Procedure for hybrid PSD testing
Chapter 2  

Numerical Methods for Hybrid Pseudodynamic Testing  

2.1 Introduction  

The finite element method has been extensively used to simulate the response of structures under static and dynamic loads in the fields of civil, aerospace and mechanical engineering, etc. The fundamental idea of the finite element method is to divide a structure into many small elements, lump the mass on the nodal degrees of freedom, and then compute the response of each element by solving the governing equation of the whole structure. For nonlinear structures, their stiffness varies with the deformation and therefore the governing equation of the motion is a nonlinear differential equation. For linear structures, when the external loads are not regular harmonic excitations, it is not a trivial task to obtain direct and accurate solutions for the response of structures under dynamic loading.  

Implicit or explicit numerical algorithms have been used to compute the transient response of structures by dividing the transient analysis of structures into a series of time steps and approximating the formulations of displacement, velocity and acceleration at each time step. The accuracy and stability characteristics associated with these numerical algorithms have been investigated in terms of numerical errors induced by the approximations of formulations and structural models. The simulation errors induced by the approximations related to the structural model can be limited by sufficiently refining the finite element distribution in a structure. In the mean time, the simulation accuracy and stability in the finite element method are dependent on the numerical algorithms.
employed because different numerical algorithms employ different approximate formulations for velocity, displacement and acceleration at each time step. Generally, the implicit algorithms have characteristics of unconditional stability whereas the explicit algorithms are often conditionally stable. However, implicit algorithms usually involve more computation efforts than explicit algorithms because of the iterative procedures required in implicit algorithms.

In hybrid experimental-numerical PSD testing, the structure is divided into some elements and the mass of the structure is lumped in the nodal degrees of freedom as in the finite element method. Differently from the simulation-based modeling method, the restoring force of the experimental substructure in PSD testing is acquired from the physical tests although the restoring force of the analytical substructure is calculated with the finite element programs as in the simulation method. The restoring forces of the experimental and simulation substructures are seamlessly integrated into the governing equation of motion of the whole structure using the substructuring technique. Different numerical algorithms can be used to solve the governing equation of motion as in the finite element simulation method. The numerical algorithms developed for the finite element method in the past 50 years can be utilized in hybrid PSD testing, but the accuracy and stability characteristics associated with these numerical algorithms need to be evaluated because some experimental errors are introduced in the physical test of the experimental substructure. In the mean time, since the hybrid PSD testing involves both numerical simulation and physical test, the concerns on the accuracy and stability characteristics of the PSD testing are related not only to the numerical algorithms, but also to the experimental errors introduced during the physical test (Shing and Mahin,
1987; Shing and Manivannan, 1990; and Shing et al., 1991). The characteristics of stability and accuracy associated with these numerical algorithms are reviewed in this chapter.

### 2.2 Numerical Algorithms

The governing equation of motion for a system under dynamic or earthquake loads can be written as

$$ M\ddot{d}(t) + C\dot{d}(t) + R(d(t)) = F(t) $$

where $M$ and $C$ are the symmetric and positive mass and damping matrices, respectively, the superposed dot denotes time differentiation, $\ddot{d}(t)$, $\dot{d}(t)$ and $d(t)$ are respectively the time-dependent acceleration, velocity and displacement, $R(d(t))$ is the restoring force and $F(t)$ is the external excitation force.

For an elastic system, the restoring force, $R(d(t))$, can be directly written as $R(d(t)) = K d(t)$, where $K$ is the linear stiffness of the system. The governing equation of motion for the system therefore can be reduced to a linear second order differential equation. Since the external excitation during an earthquake generally varies irregularly, the direct solution for the governing equation of motion of an elastic system under earthquakes is almost impossible. For a nonlinear system where the structural stiffness varies with deformation, the governing equation of motion becomes a nonlinear second order differential equation, and thus it becomes a challenge to solve the governing equation of motion for a nonlinear system under earthquakes.

It is not a trivial task to build reasonable structural models for some nonlinear systems such as partially restrained beam-to-column connections and composite shear walls. This
implies that the simulation-based modeling method encounters challenges in evaluation of these special kinds of structures under earthquake loading. On the other hand, the restoring forces of these very complicated structures or subassemblies can be acquired by direct measurements through physical tests and they can be substituted into governing equation (2.1). In hybrid PSD testing, the restoring forces of the nonlinear experimental substructures are measured experimentally. The numerical algorithms are employed to solve the governing equation of motion to get the response of structures at each time step. The applications of these numerical algorithms in PSD testing are discussed in the following sections. The merits and weaknesses of different algorithms are reviewed in terms of computation accuracy and stability.

2.2.1 Explicit Methods

2.2.1.1 Central Difference Method

The central difference method is one of the simplest explicit methods that have been used for decades in the finite element method. Taking constant time intervals, $\Delta t_i = \Delta t$, the velocity and acceleration at time step $i$ can be respectively written as:

$$\dot{d}_i = \frac{d_{i+1} - d_{i-1}}{2\Delta t} \tag{2.2}$$

$$\ddot{d}_i = \frac{d_{i+1} - 2d_i + d_{i-1}}{(\Delta t)^2} \tag{2.3}$$

Substituting equations (2.2) and (2.3) into the governing equation (2.1) gives

$$M \frac{d_{i+1} - 2d_i + d_{i-1}}{(\Delta t)^2} + C \frac{d_{i+1} - d_{i-1}}{2\Delta t} + R_i(t) = F_i(t) \tag{2.4}$$

Equation (2.4) can be rewritten as:
\[ \hat{k} d_{i+1} = \hat{f}_i \]  
\hspace{1cm} (2.5)

where:

\[ \hat{k} = \frac{M}{(\Delta t)^2} + \frac{C}{2\Delta t} \]  
\hspace{1cm} (2.6)

\[ \hat{f}_i = \frac{2M}{(\Delta t)^2}d_i - \left[ \frac{M}{(\Delta t)^2} - \frac{C}{2\Delta t} \right]d_{i-1} - R_i + F_i \]  
\hspace{1cm} (2.7)

In view of equations (2.5)-(2.7), the displacement at time step \( i+1 \) can be obtained as

\[ d_{i+1} = \hat{f}_i / \hat{k} \]  
\hspace{1cm} (2.8)

In view of equations (2.6-2.8), it can be noted that the displacement at time step \( i+1 \) is dependent only on the state of the structure at time step \( i \). The central difference method is therefore usually classified as one of the explicit methods. Owing to its characteristic that the displacement can be explicitly obtained based on the previous step solutions, it is suitable for implementation in experimental research. The central difference method was employed in the earliest PSD tests conducted by Takanashi and Nakashima (1975). This method was also used by some ensuing Asian and European researchers in their early-stage PSD tests (Tsai et al., 1994; Magonette and Negro, 1998; Molina et al., 1998; and Darby et al., 1999). The application of this explicit integration method in PSD testing is limited by its characteristic of conditional stability. To ensure the computation conditional stability, the time interval should satisfy the requirement:

\[ \Delta t \leq \frac{T_n}{\pi} \]  
\hspace{1cm} (2.9)

where \( T_n \) is the \( n^{th} \) period of the structure, and \( \Delta t \) denotes the constant time interval.
To meet this stability requirement, generally, it is not a big issue for a soft SDOF system. However, it is rather stringent for a stiff MDOF system where the fundamental structural time period is usually very small. If a small time interval is chosen, the displacement increments at some steps become correspondingly small, and therefore the hydraulic actuator almost cannot move at all due to the finite resolution of actuators. This may be one of the reasons why the early PSD tests (Takanashi and Nakashima, 1975; Magonette and Negro, 1998; Molina et al., 1998; and Darby et al., 1999) were conducted with only simple structures such as one or two-story shear frames when the central difference method was employed in their tests.

2.2.1.2 Newmark-β Method

To obtain the transient response of structures, Newmark (1959) presented a family of time-step algorithms based on the discretized governing equation of motion at time step $i+1$. The governing equation of motion and approximations of displacement and velocity can be written as follows:

$$ Ma_{i+1} + Cv_{i+1} + r_{i+1} = f_{i+1} $$  \hspace{1cm} (2.10)

$$ d_{i+1} = d_i + \Delta t v_i + (\Delta t)^2 \left[ \frac{1}{2} - \beta \right] a_i + \beta a_{i+1} $$  \hspace{1cm} (2.11)

$$ v_{i+1} = v_i + (\Delta t)(1 - \gamma) a_i + (\Delta t) \gamma a_{i+1} $$  \hspace{1cm} (2.12)

where $a_i$, $v_i$ and $d_i$ are the acceleration, velocity and displacement at time step $i$, respectively.

The parameters $\beta$ and $\gamma$ define the variation of acceleration over the time step and determine the stability and accuracy characteristics of this method (Chopra, 2001).
Setting parameters $\beta=1/4$ and $\gamma=1/2$, the variation of acceleration over the time step is constant and this specific Newmark’s method is usually called the constant average acceleration method. Taking parameters $\beta=1/6$ and $\gamma=1/2$, the acceleration varies with time linearly and the Newmark’s method is specifically called the linear acceleration method. The constant average acceleration method with parameters $\beta=1/4$ and $\gamma=1/2$ is an unconditionally stable implicit algorithm. The linear acceleration method with parameters $\beta=1/6$ and $\gamma=1/2$ is stable when meeting the requirement $(\Delta t / T_n) \leq 0.551$, but numerical errors may be induced while using this method for simulation of nonlinear structures (Chopra, 2001).

Taking the parameter $\beta=0$ gives an explicit formulation of Newmark’s method since the displacement at time step $i+1$ expressed in equation (2.11) is dependent only on the state of time step $i$. Explicit algorithms do not involve iterative procedures and generally are easier to perform in physical tests than the implicit algorithms. Shing and Mahin (1983) employed the Newmark’s explicit method in the PSD test of a MDOF system taking parameters $\beta=0$ and $\gamma=1/2$. The numerical formulations for Newmark’s method used in the integration procedure are:

\[
Ma_{i+1} + Cv_{i+1} + r_{i+1} = f_{i+1} \tag{2.13}
\]

\[
d_{i+1} = d_i + \Delta tv_i + \frac{1}{2}(\Delta t)^2 a_i \tag{2.14}
\]

\[
v_{i+1} = v_i + \frac{1}{2}\Delta t(a_i + a_{i+1}) \tag{2.15}
\]

The time step to ensure the simulation stability can be written as

\[
\frac{\Delta t}{T_n} \leq \frac{1}{\pi \sqrt{2}} \frac{1}{\sqrt{\gamma - 2\beta}} = \frac{1}{\pi} \tag{2.16}
\]
The explicit Newmark’s method has the same order of accuracy as the central difference method (Newmark, 1959). According to a series of evaluations conducted by Mahin and Shing (1985), Shing and Mahin (1985) and Shing and Mahin (1987) regarding the stability and accuracy characteristics of this explicit algorithm if it is employed in PSD testing, this explicit integration method cannot induce favorable numerical energy dissipation which is important in mitigating the experimental errors, and it can distort the time periods of the structure as the time step approaches the stability limit given in equation (2.16). Thus, the time steps need to be carefully selected to meet the stability requirements and avoid the frequency distortion. For this reason, the PSD tests of some MDOF systems with widely dispersed frequencies may pose challenges if using this integration method (Shing and Mahin, 1987).

2.2.1.3 Operator-Splitting (OS) Time Algorithm

Hughes et al. (1979) presented a family of time integration numerical algorithms. The governing equation of motion and approximations of velocity and displacement were expressed as follows:

\[ M \ddot{a}_{i+1} + (1 + \alpha)Cv_{i+1} - \alpha Cv_i + (1 + \alpha)r_{i+1} - \alpha r_i = (1 + \alpha)f_{i+1} - \alpha f_i \]

\[
\hat{d}_{i+1} = d_i + \Delta t v_i + \frac{(\Delta t)^2}{2}(1 - 2\beta)a_i \quad (2.18.a)
\]

\[
\hat{v}_{i+1} = v_i + \Delta t(1 - \gamma)a_i \quad (2.18.b)
\]

\[
\hat{d}_{i+1} = \hat{d}_{i+1} + (\Delta t)^2 \beta a_{i+1} \quad (2.19)
\]

\[
\hat{v}_{i+1} = \hat{v}_{i+1} + \Delta t \gamma a_{i+1} \quad (2.20)
\]
Expressions in equations (2.18.a-b) are explicit steps and usually called the predictor steps. Expressions in equations (2.19-2.20) are implicit steps and usually called the corrector steps. Parameters in the family of formulations are suggested to select within the range

\[ 1/3 \leq \alpha \leq 0, \gamma = \left( \frac{1}{2} - \alpha \right) \text{ and } \beta = \frac{(1 - \alpha)^2}{4} \] (2.21)

This is essentially the same as the implicit algorithm presented by Hilber et al. (1983) whereas the displacement and velocity are divided into two steps, namely the predictor-corrector steps. Hughes and Liu (1983) further proofed the numerical stability of this algorithm and named it the implicit-explicit algorithm.

Based on the implicit-explicit algorithm originally proposed by Hughes et al. (1979), Nakashima and Kaminosono (1990) successfully implemented this algorithm in PSD testing. To overcome loading reversal/hystereses problems associated with the implicit iterative procedure occurred in the implicit-explicit algorithm, Nakashima and Kaminosono (1990) modified this algorithm by approximating the restoring force, and this implicit-explicit method was named the operator-splitting method. The approximation of the restoring force was expressed as:

\[ r_{n+1}(d_{n+1}) = K^I d_{n+1} + \left[ r_{n+1}(\hat{d}_{n+1}) - K^I \hat{d}_{n+1} \right] \] (2.22)

where \( K^I \) is the initial elastic stiffness of the structure, \( \hat{d}_{n+1} \) is the predictor displacement at time step \( i+1 \), and \( r_{n+1} \) denotes the restoring force corresponding to the predictor displacement \( \hat{d}_{n+1} \). It can be noted that: 1) this expression is accurate if the structure
remains linear; and 2) if the structure experiences significant stiffness softening, the accuracy of this approximation procedure needs to be carefully evaluated.

Substituting equation (2.22) into the governing equation of motion (2.17) gives

\[ \hat{M} a_{n+1} = \hat{f} \]  \hspace{1cm} (2.23)

where the pseudo-mass matrix \( \hat{M} \) is

\[ \hat{M} = M + \Delta t \gamma (1 + \alpha) C + \beta (\Delta t)^2 (1 + \alpha) K' \]  \hspace{1cm} (2.24)

and the pseudo-force vector \( \hat{f}_{n+1} \) is

\[ \hat{f}_{i+1} = (1 + \alpha) f_{i+1} - \alpha f_i + \alpha r_i - (1 + \alpha) \hat{r}_{i+1} + \alpha C \hat{v}_{i+1} - (1 + \alpha) C \hat{v}_{i+1} + \alpha (\Delta t \gamma C + \beta (\Delta t)^2 K') a_n \]  \hspace{1cm} (2.25)

Therefore, the acceleration at time step \( i+1 \) can be obtained by substituting equations (2.23)-(2.25) into equation (2.17), and subsequently the displacement and velocity can be obtained explicitly in view of equations (2.19) and (2.20), respectively. It can be noted that this algorithm becomes explicit by approximately simplifying the nonlinear restoring force. The unconditional stability characteristic of this algorithm can be ensured if the structure remains elastic while it becomes conditionally stable if the structure experiences a significant stiffness softening as pointed out by Combescure and Pegon (1997).

Advantages of this algorithm are summarized as follows: 1) it is an explicit method and it is easy to perform in the test. As argued by Zhang et al. (2005), the explicit algorithms can help overcome the iterative problem which may affect the data exchange between distributed computers; and 2) the conditional stability characteristic can be achieved by performing adequate trials of the integration step size if the structure does
not experience significant stiffness softening during tests (Combescure and Pegon, 1997). The disadvantage of this algorithm is that the numerical accuracy and stability cannot be ensured if the structure experiences significant stiffness softening during the test.

### 2.2.1.4 Predictor-Corrector Algorithm

Hughes and Liu (1978a, b) presented the explicit predictor-corrector algorithm. The governing equation of motion and approximations of velocity and displacement were written as:

\[
Ma_{n+1} + C\dot{v}_{n+1} + K\ddot{d}_{n+1} = f_{n+1}
\]

\[
\dot{d}_{n+1} = d_n + \Delta t v_n + \frac{\Delta t^2}{2}(1 - 2\beta)a_n
\]

\[
\dot{v}_{n+1} = v_n + \Delta t(1 - \gamma)a_n
\]

\[
d_{n+1} = \dot{d}_{n+1} + \Delta t^2 \beta a_{n+1}
\]

\[
v_{n+1} = \dot{v}_{n+1} + \Delta t \gamma a_{n+1}
\]

where \(\dot{d}_{n+1}\) and \(\dot{v}_{n+1}\) are respectively called the predictor expressions for displacement and velocity, and \(d_{n+1}\) and \(v_{n+1}\) are called the corrector expressions for displacement and velocity, respectively.

As pointed out by Hughes and Liu (1978a), there are no unconditionally stable predictor-corrector schemes since they are all explicit multi-step methods. Zhang et al. (2005) performed some theoretical studies regarding the application of this algorithm in PSD testing although the implementation of this scheme in physical tests has not been published based on the author’s best knowledge.
2.2.2 Implicit Integration Method

The implicit algorithms are often superior to the explicit algorithms in terms of numerical stability and accuracy. However, their applications in PSD testing can be hindered by several practical problems: 1) the implicit algorithms generally require iterative procedures to solve the governing equation of motion. This causes not only computation inefficiency, but may induce undesirable loading reversals/hysteresis for a nonlinear structure whose stiffness is dependent on the deformation history; and 2) the accuracy and stability of tests may be impaired by the lack of reliable information of the instantaneous tangent stiffness of structures which is required by the iterative procedure. Moreover, it is difficult to measure the instantaneous tangent stiffness of structures during the test.

However, the conditional stability characteristic of the explicit algorithms poses stringent requirements on the selection of the integration step size. Especially, for the PSD tests of stiff MDOF systems or structural subassemblies which are parts of large structural systems, the integration step size needs to be very small in order to ensure computation stability. The smaller the selected integration step size is, the more time is required to complete a given excitation. It implies that computation and testing efficiency will be impaired. It is not a trivial task to choose a suitable time step size to ensure the requirements of the computation efficiency and stability for the explicit algorithms at the same time. The unconditional stability characteristic of implicit algorithms makes them superior to explicit algorithms in overcoming these challenges and become favorable choices for the PSD testing of stiff MDOF structures. The implicit schemes that have
been recently proposed or employed so far in PSD testing are reviewed in the next sections.

2.2.2.1 Implicit Alpha-Method

To avoid the limit of the conditional stability for explicit integration algorithms, Thewalt and Mahin (1987) employed the implicit alpha-method (Hilber et al., 1977; and Hilber and Hughes, 1978) in PSD tests because of its desirable numerical energy dissipation and unconditional stability characteristics. The alpha-method possesses the desirable characteristic of decreasing the higher mode responses induced by the experimental errors and this is very important in capturing the accurate responses of structures as pointed out by Mahin et al. (1989). The governing equation of motion and the formulations for the displacement and velocity can be written as:

\[ Ma_{i+1} + Cv_{i+1} + (1 + \alpha)r_{i+1} - \alpha r_i = f_{i+1} \]  \hspace{1cm} (2.31)

\[ d_{i+1} = d_i + (\Delta t)v_i + (\Delta t)^2[\left(\frac{1}{2} - \beta\right)a_i + \beta a_{i+1}] \]  \hspace{1cm} (2.32)

\[ v_{i+1} = v_i + (\Delta t)[(1 - \gamma)a_i + \gamma a_{i+1}] \]  \hspace{1cm} (2.33)

The scalar parameters are suggested to be within the range:

\[ 1/3 \leq \alpha \leq 0, \quad \gamma = \left(\frac{1}{2} - \alpha\right) \quad \text{and} \quad \beta = \frac{(1 - \alpha)^2}{4} \]  \hspace{1cm} (2.34)

To simplify the implementation procedure in their test, Thewalt and Mahin (1987) assumed the viscous damping of the structure to be zero. Rearranging equation (2.31) gives

\[ a_{i+1} = M^{-1}[f_{i+1} - (1 + \alpha)r_{i+1} + \alpha r_i] \]  \hspace{1cm} (2.35)
Substituting equation (2.35) into equation (2.32) and then rearranging equation (2.32) gives

\[ d_{i+1} = d_i + (\Delta t)v_i + (\Delta t)^2 \left( \frac{1}{2} - \beta \right) a_i + (\Delta t)^2 \beta \dot{M}^{-1} \left[ f_{i+1} + \alpha \xi_i \right] - (\Delta t)^2 \beta M^{-1} (1 + \alpha) r_{i+1} \]  

(2.36)

As the first successful application of the implicit scheme in the PSD test of nonlinear MDOF structures without using numerical iterations, Thewalt and Mahin (1987) used an analogue electronic device in the displacement control loop to correct the predictor displacements based on the continuous feedback of the instantaneous restoring force developed by the experimental specimens. The feedback-based control loop was embedded into the testing implementation procedure so that the target displacement can be continuously updated with the measured restoring force until the measured restoring force and target displacement satisfied the dynamic equilibrium equation (2.31). The target displacement was then corrected using the most latest measured restoring force, in view of equations (2.32) and (2.36).

This method was proved to be reliable and superior to the explicit schemes in terms of numerical stability and energy dissipation. Nevertheless, it is more difficult to implement than explicit schemes because an additional feedback-based displacement control loop was required.

2.2.2.2 Improved Alpha-Method

Shing et al. (1990) presented the testing implementation procedure of the improved alpha-method (Hughes, 1983; and Hulbert and Hughes, 1987). This implementation procedure was successfully employed to test a stiff masonry structure where the
fundamental time period of the structure was very small, and it was very difficult to choose a suitable integration time step for explicit algorithms. Shing et al. (1991) further extensively reviewed the accuracy, stability, and error propagation characteristics of the improved alpha-method, revealing that this implicit scheme possesses more favorable energy dissipation characteristic than explicit numerical algorithms.

Similar to the scheme presented by Thewalt and Mahin (1987), this algorithm can be used to test most structures owing to its unconditional stability characteristic. Instead of using a feedback-based control loop, this improved alpha-method used the numerical iteration procedure where the initial stiffness of the structure is utilized because the instantaneous stiffness is difficult to acquire during tests. To avoid undesirable loading reversals/hysteresis induced by the iterative procedures, a displacement reduction factor that is less than unit is introduced to reduce the chance of displacement overshoot of actuators and to achieve a more or less uniform convergence for all degrees of freedom. Another important contribution made by Shing et al. (1991) is that a dual displacement control procedure was implemented, utilizing displacement transducers both internal and external to hydraulic actuators. The external transducers are used to check the convergence of numerical iterations and the internal transducers are used to control the position of actuators. The dual displacement control procedure was proved to be effective in reducing the positioning errors of actuators in tests where the reaction frames are not stiff enough to prevent the displacement difference measured by transducers external and internal to actuators.

The formulations of the improved alpha-method employed by Shing et al. (1991) are briefly reviewed below. The governing equation of motion for a system and formulations
for the displacement and velocity can be written as (Hughes, 1983; and Hulbert and Hughes, 1987):

\[
Ma_{i+1} + (1 + \alpha)Cv_{i+1} - \alpha Cv_i + (1 + \alpha)R_{i+1} - \alpha R_i = (1 + \alpha)F_{i+1} - \alpha F_i
\]  
(2.37)

\[
d_{i+1} = d_i + \Delta v_i + \Delta t^2 [(0.5 - \beta)a_i + \beta a_{i+1}]
\]  
(2.38)

\[
v_{i+1} = v_i + \Delta t[(1 - \gamma)a_i + \gamma a_{i+1}]
\]  
(2.39)

where \(M\) and \(C\) are respectively the mass and damping matrices of the structure, \(v_i\) and \(a_i\) are respectively the velocity and acceleration at time step \(i\), \(\Delta t\) is the time interval, \(R_i\) is the restoring force at time step \(i\), and \(F_i\) is the external force excitation at time step \(i\). To ensure unconditional computation stability, the parameters are recommended to be within the range:

\[-1/3 \leq \alpha \leq 0, \beta = (1 - \alpha)^2 / 4\text{ and } \gamma = 1/2 - \alpha\]  
(2.40)

As pointed out by Peek and Yi (1990a, b), the improved alpha-method (Hughes, 1983; and Hulbert and Hughes, 1987) differs from the earlier version of alpha-method (Hilber et al., 1977; and Hilber and Hughes, 1978) in that the improved method includes a slight modification of the right-hand side of the governing equation with the external excitation as well as the possibility of using the viscous damping. Moreover, the improved alpha-method preserves the second-order accuracy while the earlier alpha-method possesses only the first-order accuracy, implying that the earlier alpha-method used by Thewalt and Mahin (1987) does not possess the second-order accuracy, except for initial value problems in which the external excitation is zero (Peek and Yi, 1990a, b).

Rearranging equations (2.37)-(2.39) gives

\[
\dot{M}d_{i+1} = \dot{M}d_{i+1} - \Delta t^2 \beta(1 + \alpha)R_{i+1}
\]  
(2.41)

where
\[ \Delta \ddot{d}_{i+1} = \dot{M}[d_i + \Delta v_i + \Delta t^2 (0.5 - \beta) a_i] + \Delta t^2 \beta[(1 + \alpha) F_{i+1} - \alpha F_i - C v_i - (1 + \alpha)(1 - \gamma) \Delta \dot{t} C a_i + \alpha r_i] \]  
\quad (2.42)

\[ \dot{M} = M + (1 + \alpha) \gamma \Delta t C \]  
\quad (2.43)

\( \Delta \ddot{d}_{i+1} \) in equation (2.42) can be determined explicitly since it is dependent only on the state of the structure at time step \( i \). Nevertheless, the displacement \( d_{i+1} \) expressed in equation (2.41) is dependent on the previous step solutions as well as the current restoring force \( R_{i+1} \). It means that the displacement \( d_{i+1} \) expressed in equation (2.41) has to be evaluated using an iterative procedure. Therefore, equation (2.41) can be written as:

\[ \Delta \ddot{d}_{i+1} = \dot{M} \ddot{d}_{i+1} - \Delta t^2 \beta (1 + \alpha) R_{i+1}^k + \dot{M} e_{i+1}^k \]  
\quad (2.44)

where \( e_{i+1}^k, d_{i+1}^k \) and \( R_{i+1}^k \) are the simulation divergence, displacement and restoring force in the \( k^{th} \) iteration at the \( i+1 \) time step.

Subtracting equation (2.44) from equation (2.41) gives

\[ \Delta \ddot{d}_{i+1}^k = -\Delta t^2 \beta (1 + \alpha) R_{i+1}^k + \dot{M} e_{i+1}^k \]  
\quad (2.45)

where

\[ \Delta d_{i+1}^k = d_{i+1} - d_{i+1}^k \]  
\quad (2.46-a)

\[ \Delta R_{i+1}^k = R_{i+1} - R_{i+1}^k \]  
\quad (2.46-b)

Provided the instantaneous stiffness for the displacement increment is \( K \), then

\[ \Delta R_{i+1}^k = K \Delta d_{i+1}^k \]  
\quad (2.47)

Substituting equation (2.47) into equation (2.45) gives:

\[ \overline{K} \Delta d_{i+1}^k = -\dot{M} e_{i+1}^k \]  
\quad (2.48)

where
\[ \bar{K} = \dot{M} + \Delta t^2 \beta (1 + \alpha) K \] (2.49)

It is almost impossible to accurately measure the instantaneous stiffness of structures during tests. Therefore, the instantaneous stiffness parameter \( K \) in equation (2.49) was replaced with the initial elastic stiffness \( K^I \) by Shing et al. (1991). If a structure remains linearly elastic, the necessity for iterations naturally disappears. As pointed out by Shing et al. (1991), the numerical convergence expressed in equation (2.48) can be always satisfied, even if both damping and mass at certain degrees of freedom are zero or if the tangent stiffness becomes negative at some locations, as long as the structure exhibits a softening behavior and \( \dot{M} + \Delta t^2 \beta (1 + \alpha) K^a \) is positive definite, where \( K^a \) is the actual tangent stiffness of the structure. Physically, the numerical convergence of this scheme can be explained as follows: the equation (2.48) can be considered as a dynamic spring system subjected to the external force \( -\dot{M}e_{i+1}^k \), and the mass and stiffness of this system are \( \dot{M} \) and \( \bar{K} \), respectively. If the stiffness related to the restoring force of this system remains larger than or equals to the instantaneous stiffness, the system can always come back to equilibrium because it is subjected to a restoring force equal or larger than the required one.

Three unique characteristics of the PSD testing implementation procedure proposed by Shing et al. (1991) are summarized as follows:

1) The initial elastic stiffness is used. As discussed above, the initial stiffness \( K^I \) is used in the iteration procedure to avoid the challenge of measuring the instantaneous stiffness of structures during tests. The initial elastic stiffness is higher than the actual tangent stiffness \( K^a \) for most nonlinear structures with stiffness softening behaviors. This helps ensure iteration convergence while the
governing equation of motion is solved using the improved alpha-method. On the other hand, for elastic structures, the stiffness remains constant and the initial stiffness always equals to the tangent one. Therefore, the use of the initial elastic stiffness is reasonable for most linear or nonlinear structures.

2) The reduction factor is introduced. To reduce experimental errors induced by the displacement overshooting of actuators, a reduction factor less than unit is introduced when the displacement increment is applied to the experimental specimen through actuators. The overshooting of actuators can lead to loading reversal/hysteresis in tests of nonlinear structures whose behaviors are dependent on the deformation history. The introduction of the reduction factor can not only help reduce the experimental errors, but also ensure a more or less uniform convergence for all degrees of freedom.

3) The residual convergence errors are partly corrected. The convergence errors cannot be totally avoided because they result from the finite resolution of control and measuring devices. Actually, if the convergence tolerance is too small, the cost of control and measuring devices will dramatically increase in order to have small finite resolutions. In the mean time, the smaller is the convergence tolerance, the more time is required to complete a given excitation. This is inefficient for most tests if too small tolerances are selected. It is an economical way to correct the convergence errors during tests by introducing a numerical correction procedure. Otherwise, the convergence errors can accumulate and propagate to the following steps and subsequently contaminate test results. The procedure to correct the residual convergence errors proposed by Shing et al.
(1991) is to modify the displacement increment at each time step based on the initial stiffness of the structure and the displacement increment of the latest time step. It should be noted that this correction procedure is still an approximate procedure since the initial stiffness, not the instantaneous tangent stiffness, is utilized in the correction procedure.

2.3 Conclusions and Remarks

Based on reviews of the explicit and implicit algorithms that have been employed or proposed so far by other researchers in the past two decades, it may be appropriate to conclude that:

1) Implicit algorithms are usually superior to explicit algorithms in terms of numerical stability and accuracy. The computation efficiency may be impaired by the iteration procedure required by the implicit algorithms. Owing to their unconditional stability characteristic, implicit algorithms can be employed to test most linear and nonlinear structures and the integration time step can be chosen to be relatively larger than explicit algorithms. Explicit algorithms are easier to implement in tests than implicit algorithms. Due to the characteristic of conditional stability, explicit algorithms present challenges if they are employed to test stiff MDOF systems since the time step needs to meet some requirements to ensure computation stability and accuracy.

2) The improved alpha-method presented by Shing et al. (1991) can overcome some of the problems induced by the iterative procedure required by the implicit algorithms while retaining the characteristics of sufficient accuracy and
unconditional stability. The reliability of this algorithm has been verified experimentally. The correction procedure employed in this algorithm can mitigate the convergence errors resulted from the finite resolution of measuring and control devices. This algorithm can be employed to test most linear and nonlinear structures without significantly reducing computation accuracy. Therefore, the improved alpha-method presented by Shing et al. (1991) is a good choice for the undergoing PSD tests in the current study.
Chapter 3

Development of Pre-testing Simulation Program for

Pseudodynamic Testing

3.1 Introduction

The hybrid PSD testing approach has attracted more and more attention as an effective means in evaluating the seismic performance of complicated structures since it was proposed by Takanashi and Nakashima (1975). It can be used to overcome many difficulties associated with conventional shake table tests where the size and weight of structures or buildings are limited by the capacity of the available shake tables. Moreover, it can be utilized to avoid many uncertainties associated with the model-based simulation method where approximate assumptions have to be made in order to build finite element models for complicated structures or subassemblies. With the aid of PSD testing, the physical experimental tests of large-scale and even full-scale structures become viable. This achievement is made possible through the use of hybrid PSD testing where only the structures or subassemblies that are difficult to be modeled are tested physically and the remainder of the structure is simulated numerically. The physical test and numerical simulation are seamlessly integrated into the dynamic equilibrium equation of the whole structure utilizing the substructuring technique.

In PSD testing, the dynamic equilibrium equation of the whole structure is solved by using different numerical algorithms to get the target displacement for each degree of freedom at each time step. Different explicit and implicit numerical algorithms and state-
of-the-art hardware architectures have been proposed to improve the stability and accuracy of hybrid PSD testing. Important issues, important to researchers, include verification of the numerical algorithms employed in PSD testing in terms of computation stability, accuracy and reliability; and prediction of the possible experimental errors that occur during physical tests. Generally, the implicit algorithms are superior to explicit algorithms in terms of numerical accuracy and stability. However, implicit algorithms are usually more difficult to implement for physical tests than explicit algorithms because of the iterative procedure required by the implicit algorithms. On the other hand, for the explicit algorithms, a very small integration time step has to be chosen for testing of stiff MDOF systems in order to meet the stability requirements posed by the explicit algorithms because of their conditional stability characteristic. It can be noted that all these issues related to the numerical algorithms should be evaluated before physical tests so that a suitable numerical algorithm can be selected. In addition, the accumulation and propagation of experimental errors induced by the finite resolution of control and measuring devices are a concern since the experimental errors may induce the spurious higher-mode responses and subsequently result in instability or failure of tests as pointed by Mahin et al. (1989).

It is usually expensive and time-consuming to perform physical tests. To avoid unexpected failure of tests, an important task for researchers is to design the testing setup and predict the test results before physical tests. In hybrid PSD testing, the whole test involves different control and measuring devices and software, the finite element modeling program, and the computer programs used for communication between computers. It becomes more important to design the testing setup and check the computer
programs before physical tests. Generally, pre-testing simulations can be conducted to check the stability and accuracy characteristics of the numerical algorithms employed in the PSD testing platforms and predict the response of structures during physical tests. Hashash et al. (2004) recently developed a pre-testing simulation computer program with MATLAB (Mathworks, 2003) where the operator-splitting (OS) integration algorithm was employed. The computation stability and error propagation characteristics of the OS algorithm were extensively evaluated by Combescure and Pegon (1997), indicating that the computation will become conditionally stable if the structure undergoes severe stiffness degradation. Stiffness degradation is an unavoidable phenomenon for many nonlinear structures such as concrete structures and beam-to-column connections. Furthermore, for the OS algorithm, its computation accuracy characteristic becomes questionable if the integration step size cannot meet a strict limit requirement as pointed out by Wang et al. (2001). To attain unconditional stability of computations in PSD tests, Shing et al., 1991 proposed the improved alpha-method by introducing a reduction factor at each iterative step during tests. This is an implicit method with unconditional stable characteristics while solving the nonlinear dynamic equilibrium equation of complicated nonlinear structures. In addition, the implicit alpha-method possesses the characteristic of second-order accuracy (Hughes, 1983). The reliability of this algorithm when it is utilized in PSD testing was further verified by Shing et al. (1991) experimentally.

A pre-testing simulation computer program for the PSD tests employing the implicit improved alpha-method has not been published according to the author’s best knowledge. In the current study, a C++ computer program framework for hybrid PSD pre-testing simulations is presented. This object-oriented pre-testing simulation framework can be
used to evaluate the stability and accuracy characteristics of the numerical algorithm employed, verify the testing platform developed in the current study, determine the required resolution of control and measuring devices, and predict the response of structures. This C++ object-oriented pre-testing simulation program will be embedded in the PSD testing platform developed in the current study as an important tool for predicting the response of structures before real physical tests.

3.2 Substructuring Technique

At the early stage of development of the PSD testing method, the entire structure or building was usually tested to verify this innovative computer-controlled testing concept (Takenashi and Nakashima, 1987; and Mahin et al., 1989). For instance, Sattary and Wright (1984) tested a full-scale seven story building using the PSD testing method at the Building Research Institute in Japan. Obviously, very large-scale testing facilities were used in their test, and it was rather expensive to test this kind of large-scale complete structures. However, it is more often the case that the critical nonlinear structural members of a structure are located in special concentrated small areas such as beam-to-column connections, composite coupling beams, and nonlinear buckled braces. In PSD testing, with the aid of the substructuring technique, it is possible to test only the nonlinear critical areas physically and model the rest of the structure numerically, instead of testing the whole system.

In the substructuring technique, the whole structure is divided into two substructures: the nonlinear complicated subassemblies or structures of special interest that need to be tested physically are partitioned into the experimental substructure, and the surrounding
portion that can be modeled numerically with confidence is partitioned into the analytical/numerical substructure. To accurately represent the whole structure, the responses of these two substructures need to be seamlessly integrated in parallel into the governing equation of motion at the interface between them. To overcome this challenge, an interaction controlling loop is designed using substructuring techniques as shown in Figure 3.1.a. In PSD testing, the displacements of the interface degrees of freedom are computed utilizing a numerical algorithm and the displacement command for the experimental substructure is then sent to the experimental controller. The experimental controller generates signals to drive actuators to impose these displacements to the experimental substructure while the restoring forces developed by the experimental specimens are read by the load cell of actuators and are sent back to the numerical algorithm module. It is crucial that the numerical algorithm embedded in the numerical module should possess the characteristics of unconditionally stability and high-order accuracy. The improved alpha-method with characteristics of unconditional stability and second-order accuracy is employed in the current research. The application of the substructuring technique in PSD testing is further explained below by providing detailed formulation expressions.

In order to ensure favorable numerical damping during the simulation, the parameter alpha is introduced in the alpha-method. The governing equation of motion is (Hughes, 1983)

\[
Ma_{i+1} + (1 + \alpha) Cv_{i+1} - \alpha Cv_i + (1 + \alpha) R_{i+1} - \alpha R_i = (1 + \alpha) F_{i+1} - \alpha F_i
\]  

(3.1)

where \(M\) and \(C\) are respectively the symmetric and positive mass and damping matrices; \(a_i, v_i, R_i\) and \(F_i\) are respectively the acceleration, velocity, restoring force and external
excitation at time step \( i \), and \( \alpha \) is a constant parameter that should be taken within the range \(-1/3 \leq \alpha \leq 0\) to ensure the stability of computation.

By partitioning the mass, damping, restoring force, and external stiffness matrices into the analytical and experimental substructures, the general dynamic equilibrium equation (3.1) can be rewritten as

\[
\begin{bmatrix}
M_{AA} & M_{AI} & 0 \\
M_{IA} & M_{II} & M_{IE} \\
0 & M_{EI} & M_{EE}
\end{bmatrix}
\begin{bmatrix}
a_A \\
a_I \\
a_E
\end{bmatrix}
+ (1 + \alpha)
\begin{bmatrix}
C_{AA} & C_{AI} & 0 \\
C_{IA} & C_{II} & C_{IE} \\
0 & C_{EI} & C_{EE}
\end{bmatrix}
\begin{bmatrix}
v_A \\
v_I \\
v_E
\end{bmatrix}
= \alpha
\begin{bmatrix}
\alpha a_A \\
\alpha a_I \\
\alpha a_E
\end{bmatrix}
+ \begin{bmatrix}
\alpha v_A \\
\alpha v_I \\
\alpha v_E
\end{bmatrix}

(3.2)
\]

where subscripts \( A \) and \( E \) denote the degrees of freedom within the analytical and experimental substructures, respectively, and \( I \) denotes the interface nodal degrees of freedom.

In PSD testing, the restoring force developed by the experimental substructure is acquired from direct experimental measurement, and the restoring force developed by the analytical substructure is computed through standard finite element program (it is ABAQUS in the current study). It is therefore necessary to obtain the restoring forces of two substructures separately and then seamlessly integrate them in parallel into the restoring force matrix at the interface degrees of freedom between the two substructures in the dynamic equilibrium equation. To illustrate this integration procedure, the restoring force can be expressed as
\[
\begin{bmatrix}
K_{AA} & K_{AI} & 0 \\
K_{IA} & K_{II} + K_{IE} & K_{IE} \\
0 & K_{EI} & K_{EE}
\end{bmatrix}
\begin{bmatrix}
d_A \\
d_I \\
d_E
\end{bmatrix}
= \begin{bmatrix}
R_A \\
R_I^A + R_I^E \\
R_E
\end{bmatrix}
\]

where

\[
R_I^A = K_{IA}d_A + K_{II}^A d_I
\]

\[
R_I^E = K_{II}^E d_I + K_{IE}^E d_E
\]

where \( K \) is the element stiffness, and \( d \) is the displacement at the nodal degrees of freedom, the superscripts \( A \) and \( E \) denote components from the analytical and experimental substructures, respectively, and \( I \) denotes the interface nodal degrees of freedom.

The selected numerical algorithm for the PSD testing is the alpha-method. The implementation of this algorithm for PSD testing was described in Chapter 2 whereas the substructuring technique is described in the current section. For convenience of description, implementation steps utilizing the substructuring technique in PSD testing are summarized in Table 3.1.

### 3.3 Pre-testing Simulation Program Framework

Prior to conducting physical tests, it is desirable to perform simulations to predict the experimental results, verify the PSD testing control programs, and evaluate the potential experimental errors so that suitable hardware equipment can be selected. In the implementation procedure using the improved alpha-method presented by Shing et al. (1991), a dual displacement control loop was employed to curb the positioning error of actuators resulted from the deformation of the reaction frame. The transducer external to
the actuator attached to the experimental specimen is to measure the actual displacement of the experimental specimen, while the transducer internal to the actuator is used to control the position of the actuator. Therefore, there are two displacement parameters related to the experimental substructure: 1) The command displacement \(d^C\) is computed by the controlling module at each iteration time step, and it is the target displacement for the experimental specimen. The command displacement is measured by the transducer internal to the actuator, and it equals the actual displacement of the actuator. Note that the displacement internal to the actuator is essentially the same as the command displacement for the experimental specimen, and 2) The external displacement \(d^{EX}\) is measured by the transducer external to the actuator, and it is the actual displacement of the specimen since it is directly measured by the transducer attached to the experimental specimen. The external displacement could be different from the target displacement because of secondary effects such as the deformation of the reaction frame.

In the pre-testing simulation, the secondary effects are assumed to be sufficiently small. For example, if the stiffness of the reaction frame relative to the experimental specimen is large enough, the deformation of the reaction frame will become relatively negligible. These two displacements will essentially be identical. Therefore, the pre-testing simulation becomes possible if it is assumed that the command displacement \(d^C\) computed by the PSD testing controller module equals to the actual displacement of the specimen \(d^{EX}\) measured by the transducers external to actuators. In the pre-testing simulation program, the restoring force developed by the experimental substructure can be computed through the standard finite element program, such as ABAQUS, by imposing the command displacement \(d^C\) to the experimental substructure, instead of
performing physical tests. It should be noted this is an approximate procedure because the experimental substructures are usually difficult to model and this is the motivation why the hybrid PSD testing is being developed in the first place. As stated previously, this approximate pre-testing simulation is to predict the approximate responses of structures, not attempting to get real results. It also helps to select suitable testing equipment for real physical tests.

The program framework for the pre-testing simulation is illustrated in Figure 3.1.b. Comparing with Figure 3.1.a, it can be noted that the MTS controller is replaced with ABAQUS and the restoring force of the virtual experimental substructure will be calculated using ABAQUS, instead of being measured by the load cells. The program can be divided into three modules according to their functions, namely (a) the controller module, (b) the virtual experimental module, and (c) the numerical/analytical module. Each module performs different tasks. The controller module solves the governing equation of motion of the structure, and generates the displacement for each degree of freedom at each time step. The virtual experimental module mimics the experimental substructure where approximate assumptions are made so that the experimental substructure can be modeled. The analytical module models the analytical substructure. It can be noted that with the exception of the virtual experimental module, the other two modules are the same as in the real PSD testing. Details of these three modules are further described below.

The program is implemented in an object-oriented C++ framework following the implementation steps of the improved alpha-method shown in Table 3.1. To make the program more general so that it can be used to mimic the real PSD testing procedure, the
controller module was installed in a computer, and the virtual experimental and analytical modules were installed in another computer. These modules were installed in different computers in order to check the communication protocols. The C++ programs used for the communication between the computers were embedded in the controller module and analytical module. Because C++ is a low level computer program language, the current object-oriented program is potentially compatible with most finite element programs which are used in the analytical module to model the analytical substructure.

3.3.1 Controller Module

In PSD testing, the entire structure is divided into experimental and numerical substructures. Using substructuring techniques, the responses of these two substructures are integrated into the dynamic equilibrium equation at the interface degrees of freedom. The controller module is developed using the low level C++ programming language in the current research. This controller module acts as a coordinator between the experimental and numerical substructures. That is, it computes the displacements employing numerical algorithms at each time step and sends the displacement commands to hydraulic actuators and ABAQUS. The computed displacements for the degrees of freedom of the numerical substructure are sent to ABAQUS, and then ABAQUS applies the displacements to the numerical substructure. The command signal of the target displacements for the degrees of freedom of the experimental substructure is sent to hydraulic actuators, and then the actuators impose the target displacement to the experimental substructure. In this pre-testing simulation program, because the experimental substructure is replaced by an approximate model, namely the virtual
experimental substructure which is modeled in the finite element program, the command
displacement for the experimental substructure is imposed to the virtual experimental
substructure through ABAQUS, instead of applying it to the real experimental
substructure as in actual tests.

3.3.2 Virtual Experimental Module

In PSD testing, the experimental module sends the restoring force of the experimental
substructure measured by the load cell of the actuator to the controller module after the
actuator imposes the target displacement to the experimental specimen. This task is
performed in the pre-testing simulation program by imposing the command displacement
to the virtual experimental specimen in the finite element program ABAQUS. ABAQUS
will then compute the restoring force of the specimen and send this restoring force back
to the controller module so that the dynamic equilibrium equation can be solved
numerically for the next iteration time step.

Two observations should be made: 1) the virtual experimental module in the pre-
testing simulation procedure, as shown in Figure 3.1.b, is performed through the finite
element program ABAQUS, instead of through the physical tests; and 2) modeling of the
experimental specimen predicts an approximate response of structures so that suitable
measuring and control devices can be selected for performing physical tests. This
modeling procedure is not accurate. Otherwise, the expensive and time-consuming
physical tests would not be needed.
3.3.3 Numerical/Analytical Module

The numerical module is to compute the restoring force developed by the analytical substructure. In the current simulation program, the standard finite element program ABAQUS is used. The numerical module computes the restoring force of the numerical/analytical substructure at each iteration time step based on the displacement obtained from the controller module and then sends this restoring force back to the controller module for the computation of displacements for the next iteration time step. It can be noted that the analytical module in the pre-testing simulation program is the same as that in actual hybrid PSD tests. This is also the reason why the pre-testing simulation program is developed. By performing the pre-testing simulation, the analytical module can be checked before actual physical tests, and the communication between the analytical module and the controller module can be verified.

In the pre-testing simulation program, the analytical module receives the target displacement for the analytical substructure generated by the controller module and then generates an ABAQUS input file for the analytical substructure. After the ABAQUS input file is generated, the finite element program ABAQUS is called and the restoring force of the analytical substructure is calculated. Once the restoring force is output by ABAQUS, it is sent to the controller module so that the dynamic equilibrium equation of the entire structure can be solved and the target displacement of each degree of freedom for the next iteration time step can be generated.
3.4 Simulation Examples

In real physical tests, experimental errors cannot be avoided because of the finite resolution of measuring and controlling devices. In the current pre-testing simulation program, it is assumed that displacements of the experimental specimen are measured accurately without positioning and measuring errors. Therefore, the target command displacement computed by the controller module equals to the displacement measured by the transducers external and internal to actuators. The substructuring technique described in Section 3.2 is utilized to integrate the restoring forces of the virtual experimental substructure and the analytical substructure. The improved alpha-method described in Chapter 2 is employed to solve the governing equation of motion of the entire structure, and it is embedded in the controller module. If the controller module possesses desirable characteristics of unconditional stability and high-order accuracy, it is expected that the response of the structure should be reasonably consistent with the analytical solution obtained through a transient dynamic analysis using the standard finite element program ABAQUS. It should be noted that the result obtained directly through ABAQUS is called the analytical solution in this study, and it is regarded as the correct benchmark solution for the comparison purpose. The PSD pre-testing simulations are performed for linear and nonlinear structures. Examples 1 through 3 discuss linear cases, and examples 4 through 6 consider nonlinear cases. The time increment for all simulation cases is taken as $\Delta t = 0.01s$, and the structural damping for the simulation cases is assumed to be zero. A zero damping is assumed for each case in order to examine the resonance phenomenon when the elastic structure is subjected to a sinusoidal excitation with excitation frequency.
equal to its natural frequency. This approach provides an easier method for checking whether the pre-testing simulation is correct or not.

### 3.4.1 Case 1: Elastic Cantilever Column (SDOF)

Consider a cantilever elastic column, as shown in Figure 3.2.a, with a rectangular cross section $120 \times 120 \text{mm}$ and height $L = 5,000 \text{mm}$. A concentrated mass $M = 100 \text{N-s}^2/\text{mm}$ is lumped at the column top. The elastic Young’s modulus of the column is $E = 200,000 \text{N/mm}^2$ (i.e., steel). The stiffness and natural frequency of the column as a SDOF system can be calculated as $K = 87.09 \text{N/mm}$ and $\omega = 0.933 \text{rad/s}$, respectively. The column is subjected to an external sinusoidal ground excitation with excitation frequency equal to the natural frequency of the system, as shown in Figure 3.2.b. The external excitation force is: $F(t) = -M \sin(\omega t) = -100 \sin(0.933t)$.

As shown in Figure 3.3, for an elastic SDOF case, the PSD pre-testing simulation result is in good agreement with the analytical solution obtained through the standard finite element program ABAQUS. The excellent correlation demonstrates that the dynamic equilibrium equation is solved accurately at each time step for this elastic simple structure. Hence, the implementation procedure of the numerical algorithm embedded in the current testing platform can be used for testing of SDOF elastic cases. In the current pre-testing simulation case, the tolerance is assumed to be 0.1 mm, indicating that a rather accurate experimental result is expected to get if the tolerance as big as $TOL = 0.1 \text{mm}$ is selected in the PSD tests. Note that the measuring and control errors that occur during testing are not reproduced in the simulation.
3.4.2 Case 2: Elastic Frame with Hinge (Two DOFs)

The pre-testing simulation is performed with reference to an elastic frame with a hinge on the left corner (Figure 3.4.a). It represents an elastic two-DOF system. The hinge was placed for ease of implementation in the laboratory, as discussed in Chapter 4. At the hinge, the rotational DOF is eliminated, and hence testing of the left column which represents the experimental substructure can be performed by using one actuator. The height and span of the frame are taken as 5,000\(\text{mm}\), and all of the members are assumed to have 120×120\(\text{mm}^2\) rectangular cross sections. The elastic Young’s modulus of the frame is \(E=200,000\text{N/mm}^2\) (i.e., steel). A concentrated mass, \(M=200\text{N-s}^2/\text{mm}\), is lumped on the top floor of the frame. As shown in Figure 3.4.a, the structure is divided into the experimental substructure (i.e., the left column) and the analytical substructure. The experimental substructure is a cantilever column. The numerical/analytical substructure is a frame consisting of a vertical column and a horizontal beam. Because the interface between two substructures is a hinge and there are no vertical loads, only one actuator will be needed for actual testing. The stiffness matrices for the experimental and analytical substructures and the entire complete structure were calculated by using standard techniques. The stiffness for the analytical/numerical substructure is

\[
[K]^a = \begin{bmatrix}
5.08E9 & 8.7E5 \\
8.7E5 & 348.36
\end{bmatrix}
\]

(3.6)

where the superscript \(a\) denotes the analytical substructure.

The stiffness for the experimental substructure which is a cantilever column, is

\[
[K]^E = 87.09
\]

(3.7)

where the superscript \(E\) denotes the experimental substructure.
Based on substructuring techniques described in section 3.2, equations (3.6) and (3.7) are incorporated to obtain the stiffness of the entire complete structure as

\[
K = \begin{bmatrix}
    k_{11}^a & k_{12}^a \\
    k_{21}^a & k_{22}^a + k_{22}^E
\end{bmatrix} = \begin{bmatrix}
    5.08E9 & 8.7E5 \\
    8.7E5 & 435.45
\end{bmatrix}
\]  

(3.8)

where subscripts 1 and 2 denote the horizontal and rotational degrees of freedom, respectively, and superscripts \(E\) and \(a\) denote the experimental and analytical substructures, respectively.

The frame is subjected to a sinusoidal external ground excitation with excitation frequency equal to the first natural frequency of the system, as shown in Figure 3.4.b. The external excitation force can be written as:

\[
F(t) = -(M)\sin(\omega t) = -\begin{bmatrix}
    0 \\
    200
\end{bmatrix} \sin(1.196t)
\]

(3.9)

where \(\omega\) is the frequency of the external excitation, and \(M\) is the mass matrix of the structure.

The tolerance for the horizontal DOF in the experimental substructure is assumed to be \(TOL=0.05\,mm\). Figure 3.5.a shows the horizontal displacement of the elastic frame obtained from the PSD pre-testing simulation and the analytical solution obtained through ABAQUS. This figure indicates that the results obtained by these two methods are in good agreement. The horizontal displacement reasonably displays resonance response characteristics when the structure is subjected to a sinusoidal excitation with a resonant frequency. As shown in Figure 3.5.b, the PSD pre-testing simulation result for the rotation on the right corner of the elastic frame is consistent with the analytical solution obtained through ABAQUS. The rotational deformation on the right corner of the frame increases with time and gradually becomes unbounded when it is subject to a
resonant excitation, reasonably displaying the resonant response for a structure with zero damping.

3.4.3 Case 3: Elastic Frame without Hinge (Three DOFs)

Consider a general elastic frame without hinges where the experimental substructure has two DOFs: one is the horizontal displacement DOF and the other is the rotational deformation DOF at the interface (see Figure 3.6.a). This case is a more common MDOF case than case 2 described in section 3.4.2. As shown in Figure 3.6.a, the structure is divided into two substructures. The experimental substructure is essentially a cantilever column with two DOFs. The analytical substructure consists of a vertical column and a horizontal beam with both the vertical and rotational DOFs at the left end. The material properties and dimensions of this frame are the same as those for case 2. Similar to the procedure used in case 2, the elastic stiffness matrices for the experimental and analytical substructures and the entire structure can be calculated.

The stiffness matrix of the analytical/numerical substructure is:

\[
K^a = \begin{bmatrix}
5.8E9 & 8.7E5 & 1.45E9 \\
8.7E5 & 348.36 & 0 \\
1.45E9 & 0 & 2.9E9 \\
\end{bmatrix}
\]  (3.10)

where the superscript \(a\) denotes the analytical substructure.

The stiffness matrix of the experimental substructure is:

\[
K^e = \begin{bmatrix}
348.36 & 8.7E5 \\
8.7E5 & 2.9E9 \\
\end{bmatrix}
\]  (3.11)

where the superscript \(E\) denotes the experimental substructure.
Based on substructuring techniques, equations (3.10) and (3.11) are incorporated to obtain the stiffness of the entire complete structure:

\[
K = \begin{bmatrix}
  k^a_{11} & k^a_{12} & k^a_{13} \\
  k^a_{21} & k^a_{22} + k^E_{22} & k^a_{23} + k^E_{23} \\
  k^a_{31} & k^a_{32} + k^E_{32} & k^a_{33} + k^E_{33}
\end{bmatrix} = \begin{bmatrix}
  5.8E9 & 8.7E5 & 1.45E9 \\
  8.7E5 & 696.73 & 8.7E5 \\
  1.45E9 & 8.7E5 & 5.8E9
\end{bmatrix} \tag{3.12}
\]

where subscripts 1 and 2 denote the rotational degrees of freedom at the right and left ends of the beam, respectively, the subscript 3 denotes the horizontal displacement degree of freedom, and superscripts \( E \) and \( a \) denote the experimental and analytical substructures, respectively.

The frame is subjected to a sinusoidal excitation with excitation frequency equal to the first natural frequency of the elastic frame structure, as shown in Figure 3.6.b. The external excitation force applied to the structure is:

\[
F(t) = -(M)sin(\omega t) = \begin{cases}
0 \\
200 \sin(1.56t)
\end{cases}
\tag{3.13}
\]

where \( \omega \) is the frequency of the external excitation, and \( M \) is the mass matrix of the structure.

Figure 3.7.a shows the horizontal displacement of the frame, indicating that it is in good agreement with the analytical solution obtained through ABAQUS. The rotational displacements on the left and right corners of the frame are shown in Figure 3.7.b and Figure 3.7.c, respectively. It can be noted that they are consistent with the analytical solutions, and rotations on both corners are the same because this is a symmetrical frame. It can be noted from Figures 3.7.a through c that the horizontal displacements and rotations at both corners are consistent with the analytical solution, displaying the
resonance characteristic when an elastic structure with zero damping is subjected to a resonant excitation.

3.4.4 Case 4: Nonlinear Cantilever Column (Nonlinear SDOF)

The nonlinear case discussed in this section is more general than the linear case described in Section 3.4.1, and it is representative of cases benefiting from PSD physical tests. The nonlinearity discussed herein is only a material nonlinearity and not a geometric nonlinearity. The hybrid PSD testing method will, nevertheless, be capable of accounting for all sources of nonlinearity in the experimental specimen. The nonlinear material deformation model shown in Figure 3.8 is employed for illustration purpose. The initial elastic Young’s modulus for the material shown in Figure 3.8 is $E=200,000 \, N/mm^2$ (i.e., steel). The isotropic material hardening ratio is assumed to be 0.1.

Consider a cantilever column with a rectangular cross section of $120\times120mm$ and height $L=5,000mm$. The dimensions of this column are the same as those for the elastic case discussed in Section 3.4.1 (see Figure 3.2.a). A large excitation force $F(t)= -100M\sin(\omega t)$ with a duration of 20 seconds is applied to the column to produce nonlinear deformations. The excitation frequency is taken as the initial elastic frequency of the structure, i.e., $\omega=\sqrt{\frac{k^I}{M}}=0.933\, rad/s$, where $K^I=87.09\, N/mm$ is the initial elastic stiffness of the cantilever column, $M=100\, N\cdot s^2/mm$ is the lumped mass. The duration of the excitation is 20 seconds, implying $F(t)=0$ for $t>20s$. The excitation is similar to that shown in Figure 3.2.b with the exception of the peak acceleration which is 100 times larger than that used for case 1. It should be noted that the stiffness of the nonlinear cantilever column varies with time, and the initial elastic stiffness is utilized in the
iterations as discussed in Chapter 2. The experimental tolerances for the horizontal and rotational DOFs are assumed to be \( TOL=0.1\,\text{mm} \) and \( TOL=0.0001 \), respectively.

Figure 3.9 shows that the displacement responses as computed through the pre-testing PSD simulation is in good agreement with the analytical simulation results obtained from ABAQUS, even when the tolerance is as large as \( TOL=0.1\,\text{mm} \). This may be due to two reasons: 1) the structure is a simple one which involves only one DOF; and 2) the amplitude of the structural response is relatively large (as a result of nonlinear deformations produced by a strong excitation because a strong excitation) and the experimental tolerance becomes relatively small compared with the displacement increment at each time step.

### 3.4.5 Case 5: Nonlinear Frame with Hinge

Consider a nonlinear frame with a hinge on the top left corner. The dimensions and sizes of the frame are the same as those used previously for case 2 (Figure 3.4). The nonlinear material deformation model shown in Figure 3.8 is also used for this frame. The initial elastic Young’s modulus of the material is \( E=200,000\,\text{N/mm}^2 \) (i.e., steel), and it is assumed that the isotropic hardening ratio of the material is 0.1. A concentrated mass \( M=200\,\text{N-s}^2/\text{mm} \), is lumped on the beam/floor. The experimental tolerance for the horizontal DOF is assumed to be \( TOL=0.05\,\text{mm} \). Note that the assumed tolerance in the current case is different from those in other cases. This is to demonstrate that all the pre-testing simulations are consistent with the analytical solutions even when different tolerances are assumed in different cases. The response amplitudes are also different for different cases. Generally, larger tolerances should be used for cases with larger
amplitudes. The structure is partitioned into experimental and analytical substructures as shown in Figure 3.4. The stiffness matrices of these nonlinear substructures vary with deformation. The initial elastic stiffness matrix of the entire structure, which is the same as that for case 2 (i.e., equation 3.8), is used for the iterations, as discussed in Chapter 2. For the sake of brevity, the initial structural stiffness is not repeated here.

The frame is subjected to a sinusoidal external excitation with an excitation frequency equal to the first natural frequency of the elastic structure. To demonstrate the nonlinearity of the structure, the peak amplitude of the external excitation is set to be 100 times larger than that for the elastic case shown in Figure 3.4.b. It is assumed that the excitation lasts 20 seconds. The expression of the excitation is written as

\[ F(t) = -100(M)\sin(\omega t) = -\left\{ \begin{array}{ll} 0 \\ 200 \end{array} \right\} (100) \sin(1.196t) \quad t \leq 20s \quad (3.14) \]

\[ F(t) = 0 \quad 20s < t < 30s \quad (3.15) \]

Figures 3.10.a-b show that the pre-testing simulation results for the horizontal displacement of the floor and the rotational deformation of the top right corner of the frame are in good agreements with the analytical solutions obtained through ABAQUS. Hence, the application of the initial elastic stiffness in the iterative procedures during the simulation does not appear to produce significant errors. This may be because the tolerance \( TOL = 0.05mm \) relative to the response amplitude is relatively small, corresponding to 1% of the maximum response amplitude. It can be noted that the errors induced by the introduction of the initial elastic stiffness can be limited by using advanced hardware with high resolution and so that relatively small tolerance can be applied during tests. The stability and accuracy characteristics of the improved alpha-
method employed in the current simulation are shown to be excellent according to the results shown in Figures 3.10.a and b.

3.4.6 Case 6: Nonlinear Frame without Hinge

Consider a nonlinear frame without hinges. The dimensions and sizes of the frame are the same as those for the elastic frame discussed in case 3 (Figure 3.6.a). The frame is divided into an analytical/numerical substructure, and an experimental substructure which has both the horizontal and rotational DOFs at the top of the cantilever column. The nonlinear material model shown in Figure 3.8 is once again used for this case. The stiffness of the structure varies with deformation. The initial elastic stiffness matrix of the entire structure, which is needed for the iterations during the PSD test, is essentially the same as that expressed in equation (3.12) for the elastic case (case 3). A concentrated mass $M=200N\cdot s^2/mm$, is lumped on the beam/floor of the frame. The tolerances for the horizontal and rotational DOFs are assumed to be $TOL=0.05\ mm$ and $TOL=0.0001$, respectively. The frame is subjected to a sinusoidal excitation with an excitation frequency equal to the first natural frequency of the elastic structure. The external excitation is assumed to last 20 seconds, written as

$$F(t) = \begin{cases} \begin{bmatrix} 0 \\ -200 \sin(1.56t) \\ 0 \end{bmatrix} & t \leq 20s \\ 0 & 20s < t \leq 30s \end{cases} \quad (3.16)$$

$$F(t)=0 \quad 20s < t \leq 30s \quad (3.17)$$

Figure 3.11.a shows the horizontal displacement of the frame, indicating that the pre-testing simulation result is consistent with the analytical solution obtained from ABAQUS. Figures 3.11.b and c show the rotations on the left and right corners of the
frame, respectively. Their simulation results are in good agreement with the analytical solutions. It can be noted that the rotational responses of the two corners are the same. This is simply because the frame is symmetrical.

As shown in Figures 3.11.a-c, the resonant response characteristic of the frame can be noted at the beginning of the excitation, but the response of the nonlinear frame does not display resonant characteristic after several cycles of excitation. This is because the frequency of the structure will change after the onset of nonlinearity and will not remain equal to the frequency of the excitation. In other words, nonlinearity of the structure provides damping effects.

3.5 Conclusions and Remarks

The pre-testing simulation program framework for PSD testing is presented and discussed in this chapter. This program consists of three components/modules that are seamlessly integrated using the substructuring technique. The pre-testing simulation results for six representative cases are compared with the analytical solutions obtained by ABAQUS. Comparisons of displacements and rotations show good agreements. Based on the results presented in this chapter, the following conclusions are made:

1) The improved alpha-method is a good choice for the PSD hybrid tests. Sufficiently accurate pre-testing simulation results are obtained through the computer program developed based on this algorithm. Pre-testing simulation results are consistent with the analytical solutions obtained by ABAQUS.

2) The hybrid PSD testing is a general testing approach for dynamic and earthquake research. It can be employed to test not only elastic structures, but also nonlinear
structures. It can be used to test either individual members or structural assemblies.

3) The communication protocol between the controller module and the analytical module which are installed in two different computers is verified by performing pre-testing simulations. The communication procedure used in the pre-testing simulation is the same as that in real PSD tests, implying that this communication procedure can be used for real PSD tests.
Table 3.1 Steps for substructuring PSD tests (Shing et al., 1994)

1. Evaluate $\hat{M}$ and $K^*$ by means of equation (2.43) and (2.49)
   Select the tolerance $\Delta d^n$ for each DOF $n$
   Set $i=0$ and initialize $d_{oA}, d_{oE}, d_{oI}, v_o, r_o$, and $a_o$

2. Input excitation $f_{i+1}$

3. Compute $\hat{M} \hat{d}_{i+1}$ by means of equation (2.2)

4. Set $k=0$

5. Set $r_{i+1}^{(k)} = r_i; d_{i+1}^C = d_i^C$, where $d_i^C = \begin{bmatrix} d_i^A & d_i^E & d_i^I \end{bmatrix}^T$
   Set $d_{EX}^{(k)} = d_i^{EX}$

6. Evaluate $\hat{M} e_{i+1}^R(k) = \hat{M} d_{EX}^{(k)} - \hat{M} d_{i+1} + \Delta t^2 \beta (1 + \alpha) r_{i+1}^{(k)}$

7. Solve $K^* \Delta d_{i+1}^{(k)} = -\hat{M} e_{i+1}^R(k)$ for $\Delta d_{i+1}^{(k)}$

8. If $|\Delta d_{i+1}^{(k)}| \leq \Delta d^n$, go to step 15

9. Evaluate $d_{i+1}^C = d_{i+1}^C + \theta \cdot \Delta d_{i+1}^{(k)}$, where $0 < \theta \leq 1$

10. Impose $d_{E,i+1}^{(k+1)}$ and $d_{E,i+1}^{(k+1)}$ on the test structure

11. Measure $r_{E,i+1}^{(k+1)}, d_{M,i+1}^{(k+1)}$, and $d_{M,i+1}^{(k+1)}$ developed by experimental substructures.

12. Using $d_{E,i+1}^{(k+1)}$ and $d_{M,i+1}^{(k+1)}$, evaluate $r_{d,i+1}^{(k+1)}$ and $i_{d,i+1}$

13. Assemble restoring force vector:
    $r_{i+1}^{(k+1)} = \begin{bmatrix} r_i^{(k+1)T} & r_i^{E(k+1)T} & r_i^{(k+1)T} \end{bmatrix}_{i+1}^T$
    Assemble displacement vector:
    $d_{i+1}^{M(k+1)} = \begin{bmatrix} d_i^{A(k+1)T} & d_i^{M(k+1)T} & d_i^{E(k+1)T} \end{bmatrix}_{i+1}^T$

14. Set $k=k+1$
   go to Step 6

15. $d_{i+1} = d_{i+1}^{M(k)} + \Delta d_{i+1}^{(k)}$, $r_{i+1} = r_{i+1}^{(k)} + K \Delta d_{i+1}^{(k)}$, $d_{i+1}^C = d_{i+1}^C^{(k)}$, $d_{i+1}^M = d_{i+1}^M^{(k)}$
    $a_{i+1} = [d_{i+1} - d_j - \Delta v_j - \Delta t^2 (0.5 - \beta) a_j] / (\Delta t^2 \beta)$,
    $v_{i+1} = v_j + \Delta [(1 - \gamma) a_j + \gamma a_{i+1}]$

16. Set $i=i+1$
   go to Step 2.
Figure 3.1.a  Hybrid PSD testing framework

Figure 3.1.b  PSD pre-testing simulation framework

\[ r^e: \text{restoring force of experimental substructure} \]
\[ d^e: \text{displacement of experimental substructure} \]
\[ r^a: \text{restoring force of analytical substructure} \]
\[ d^a: \text{displacement of analytical substructure} \]
Figure 3.2.a Model for cantilever column

Figure 3.2.b Ground excitation for cantilever column
Figure 3.3 Displacement of cantilever column subjected to ground excitation
Figure 3.4.a Models of substructures for frame with hinge on left corner

Note: Mass is not present in the experimental/analytical substructures because the forces are quasi-statically applied to the substructures in PSD testing. In other words, PSD testing “transfers” a dynamic analysis into a static case, in which the mass is not needed for the analysis.

Figure 3.4.b Ground excitation for elastic frame with hinge on left corner
Figure 3.5.a Horizontal displacement of frame with hinge on left corner

Figure 3.5.b Rotation on right corner of frame with hinge on left corner
Figure 3.6.a Experimental and analytical substructure models for frame without hinges

Figure 3.6.b Ground excitation for frame without hinges
Figure 3.7(a) Horizontal displacement for frame without hinge

Figure 3.7.b Rotation on left corner for frame without hinge
Figure 3.7.c Rotation on right corner for frame without hinge

Figure 3.8 Deformation model for nonlinear material
Figure 3.9 Response for nonlinear cantilever column

Figure 3.10.a Horizontal displacement for nonlinear frame with hinge on left corner
Figure 3.10.b Rotation on right corner for nonlinear frame with hinge on left corner

Figure 3.11.a Horizontal displacement for nonlinear frame without hinge
Figure 3.11.b Rotation on left corner for nonlinear frame without hinge

Figure 3.11.c Rotation on right corner for nonlinear frame without hinge
Chapter 4 Development of an Integrated Hybrid Pseudodynamic Testing Platform

4.1 Introduction

Physical tests are usually performed to investigate the behavior of complicated nonlinear structures in order to get direct and reliable results when the computer modeling is not adequate. The two main methods currently used to test structures, under the influence of dynamic loading, are shake tables and the pseudodynamic testing method (Darby et al., 1999). Small-scale models of structures have to be used in shake table tests because of the limited size and capacity of the available shake tables, and therefore it is often difficult to investigate the behavior of full-scale structures. The hybrid pseudodynamic testing (PSD) approach can be used to avoid the limitations of shake table tests for evaluating the performance of large-scale structures under dynamic or earthquake loads. PSD testing combines the merits of both the computer modeling and physical testing methods. A large-scale critical nonlinear structural element that cannot be modeled satisfactorily, namely the experimental substructure, is tested physically, while the remainder of structure that can be modeled with confidence, namely the analytical substructure, is modeled in a finite element program.

In PSD testing, the mass, damping, and external excitation matrices of both the analytical and experimental substructures are assumed to be known in the dynamic equilibrium equation of motion. Therefore, the restoring force matrix is the only unknown that needs to be acquired in order to solve the governing equation of motion.
The restoring force of the analytical substructure can be calculated using finite element programs such as ABAQUS (Abaqus, 2003), but the restoring force of the experimental substructure is not easy to acquire satisfactorily using a finite element program due to the uncertainties associated with the material and geometric nonlinearities, and therefore it needs to be acquired by performing physical tests. To obtain the restoring force of the experimental substructure, actuators are used to apply target displacements to the experimental specimen through an electronic controller (i.e., the MTS (MTS, 2003)), and the restoring force of the experimental specimen can be recorded from the load cell of actuators during the test. The restoring force matrices of the analytical and experimental substructures can be seamlessly integrated in parallel into the dynamic equilibrium equation of the whole structure using the substructuring technique (Dermitzakis and Mahin, 1985).

Once the restoring force matrix of the whole structure is acquired, different implicit or explicit numerical integration algorithms can be employed to solve the dynamic equilibrium equation. The implicit alpha-method (Hilber et al., 1977 and 1978) with characteristics of unconditional stability and second-order accuracy is employed in the current study. The unconditional stability characteristic of the alpha-method implies that the numerical integration step size can be selected arbitrarily while retaining computation stability for both linear and nonlinear structures. On the contrary, if an explicit algorithm is used, especially for a stiff MDOF system, a very small time step size has to be chosen in order to ensure stability of numerical simulations. This will pose a very stringent requirement on the resolution of control and measuring devices since the displacement increment becomes very small provided a small time step size is chosen.
Shing et al. (1991) presented an implementation procedure of the implicit alpha-method and they verified it experimentally although the test was conducted for an overall structure without including an analytical substructure in their test. The implicit alpha-method was further employed to test a braced steel frame by Shing et al. (1994) where the analytical substructure was assumed to remain elastic during the test. Because the restoring force of the elastic analytical substructure could be calculated easily by multiplying the elastic stiffness by the displacement, Shing et al. (1994) did not use any finite element program in their test to evaluate the analytical substructure. This may be the reason why a communication procedure between the PSD controlling program and the finite element programs such as ABAQUS was not developed in their test. However, for a general case, the analytical substructure may be nonlinear and the restoring force of a nonlinear analytical substructure has to be evaluated using a finite element program. It might be appropriate to point out that the testing platform developed by Shing et al. (1994) in a Fortran program framework can be employed to test some structures where analytical substructures are elastic, but it cannot be used for tests of general cases where the analytical substructure is nonlinear.

To date, the hybrid PSD testing approach is divided into two categories, namely the conventional and real-time testing, in terms of the loading speed of actuators during the test. In real-time PSD testing, actuators move as fast as the ground motion of realistic earthquakes. On the contrary, in conventional PSD testing, the target displacement is quasi-statically applied to the experimental specimen over a greatly expanded timescale. For both conventional and real-time PSD testing, a necessary task is to develop a suitable testing platform to seamlessly integrate the analytical and experimental substructures.
during the test and control the entire testing process. Recently, the University of California at Berkeley, University of Colorado, and University of Buffalo developed a testing platform with MATLAB/Simulink for real-time PSD testing (Mosqueda et al., 2005). In this real-time testing platform, unfortunately, the nonlinear analytical substructure cannot be evaluated because to date no finite element program can be so fast to evaluate the stiffness of a nonlinear structure in only several milliseconds. Another severe problem associated with real-time PSD testing is that the entire test setup is prone to shaking during the test due to the fast motion of actuators (Wu et al., 2006). It is not a trivial task to overcome these severe problems associated with real-time PSD testing in the near future. Therefore, the current study focuses on the conventional hybrid PSD testing, not real-time ones.

The University of Illinois at Urbana-Champaign (UIUC) has developed a testing platform with MATLAB (Mathworks, 2005) for conventional PSD testing and it has been employed to test a two-span concrete bridge, showing reasonable test results (Hashash et al., 2004). The purpose of this chapter is to present a C++ object-oriented testing platform for conventional PSD testing. The current testing platform possesses three basic features as that developed by UIUC: 1) a numerical algorithm is embedded in the testing platform to solve the governing equation of motion to generate the target displacement for both experimental and analytical substructures. This numerical algorithm needs to satisfy the computation accuracy and stability requirements. Note that the numerical algorithm employed in the UIUC testing platform was the conditionally stable implicit-explicit operator-splitting method (Hughes and Liu, 1978; and Hughes et al., 1979) whereas the unconditionally stable implicit alpha-method is utilized in the current study; 2) a
communication procedure is embedded to send target displacements of the analytical substructure to another computer where the finite element programs such as ABAQUS are installed, and to acquire the restoring force of the analytical substructure from the finite element programs; and 3) a communication procedure is embedded to drive actuators to apply target displacements to the experimental specimen through an electronic controller (i.e., the MTS controller) and read the restoring force of the experimental specimen from the load cell of actuators during the test.

The current object-oriented testing platform has its several unique features: 1) it can be maintained, modified, and upgraded rather easily with the development of PSD testing techniques. This testing platform is developed in an object-oriented C++ framework. Each objective is achieved in a separate individual element. When an individual element needs to be modified, others can remain untouched; 2) it can easily interface with other software used for simulation and physical testing such as ABAQUS and the MTS controller, and other data acquisition hardware/systems owing to the high compatibility of C++ language (Stroustrup, 2000). C++ is a low level computer language which is extensively used for data exchange between different software in various engineering fields. Also, C++ is one of the most popular computer languages used in the hardware control field. The driving programs of many automatic hardware systems for physical testing are developed with C++; and 3) tests can be easily operated and monitored using this testing platform. Owing to the direct Start, Hold, and Stop human-machine interfaces, tests can be easily controlled. The Hold option allows test operators to temporarily hold the test, to for example observe the response of experimental specimens, check the test fixtures, and perform preliminary analysis of measured experimental data,
etc. The Stop option allows one to terminate a test if unexpected severe situations happen during the test. One can restart a terminated test from the exact location where it was terminated.

To verify the testing platform developed in the current study, two sets of tests were performed at the University of Cincinnati Large-Scale Test Facility. The first set of tests was performed to test a simple steel beam, representing a single-degree-of-freedom (SDOF) system. To further demonstrate the capabilities of the current hybrid PSD testing platform, the second set of tests was performed to test a multi-degree-of-freedom (MDOF) system consisting of an experimental substructure and an analytical substructure. The experimental substructure was a simple beam which was essentially the same as that used in the first set of tests. The cases where the analytical substructure was linear or nonlinear were extensively evaluated. Parametric studies were performed to evaluate the stability and accuracy characteristics of the current testing platform. The embedded Hold and Stop human-machine interfaces which were developed to facilitate the operation of current testing platform were verified experimentally. The Start-Restart procedure which was developed to handle severe unexpected situations during tests was demonstrated by testing a case where the external ground motion consisted of an earthquake ground motion followed by a regular harmonic excitation.

4.2 Equipment Used in PSD Testing

In conventional hybrid PSD testing, the loads are quasi-statically applied to the experimental specimens over a greatly expanded timescale. Therefore, equipment used
for static tests in most structural testing facilities can be utilized to conduct hybrid PSD
tests. The pieces of equipment used to verify the current testing platform include:

1) A servo-hydraulic system consisting of servo-valves and pressurized hydraulic oil
supply. This system is used to drive actuators. The servo-hydraulic system
receives an analog command signal of the target displacement from the MTS
controller. The command signal information includes the target displacement, the
moving speed of actuators and control channel numbers. The servo-hydraulic
system quasi-statically applies the target displacement to the experimental
specimen based on the command signal information.

2) A testing computer used to control the MTS system. In this testing computer, the
object-oriented C++ PSD controlling program and the MTS controller were
installed. The MTS controller can obtain the target displacement command from
the PSD controlling program through C++ dynamic link libraries (DLL). The
analog command signal of target displacements is generated by the MTS
controller, and then sent to actuators powered by the servo-hydraulic system.

3) A simulation computer used to calculate the restoring force of the analytical
substructure. In this computer, a simulation engine is installed which can
exchange data with the testing computer and drive ABAQUS to compute the
restoring force of the analytical substructure.

4) One hydraulic actuator with the capacity of 890KN and stroke of ±152.4mm.
Generally, a ramp-and-hold loading procedure is used in the MTS system to apply
the target displacement to the experimental specimen (MTS, 2003). This loading
procedure which was theoretically investigated by Peek and Yi (1990a and b) is
briefly reviewed here. In conventional PSD testing, each typical time step for the 
application of displacement to actuators is divided into three phases, namely the 
computation phase, loading phase, and holding phase, as shown in Figure 4.1. 
They are explained as follows: a) computation phase for the nth iterative time step 
begins at time $t = t^c_n$, where $t$ is the experimental time, and $C$ denotes the 
computation phase. The period of computation phase is denoted as $\Delta t^c$. During 
this period, the target displacement is computed and the command signal is sent to 
the actuator; b) loading phase begins at time $t^L_n$, where $L$ denotes the loading 
phase. The period of this phase is denoted as $\Delta t^L$. During this phase, the 
displacement is incremented gradually and smoothly until it reaches the target 
displacement. The incremental function employed in the current study was linear 
whereas other functions are available to achieve the application of the 
displacement. The application speed of actuators can be selected before the test. A 
speed of 0.05mm per second was used in the current study; c) hold phase begins at 
time $t^h_n$ in which $h$ denotes the hold period. It is the time at which the command 
signal reaches the target displacement. The period of this phase is denoted as $\Delta t^h$. 
Because of the finite actuator response time, the motion of actuators does not 
cease at time $t = t^h_n$. At the end of the hold phase, the displacements and resisting 
forges at each degree of freedom in experimental substructure are measured and 
then sent back to the controller for the computation of the displacement for the 
next iteration step. The hold period is supposed to be very small for real-time PSD 
testing. This will pose a stringent requirement on the testing system since the
stress of the actuator’s cylinder and oil-supply system may not be relaxed within a very small time period.

5) A load cell mounted on the actuator is used to measure the restoring forces of the experimental specimen. The load cell measures the force simultaneously while the displacement is applied to the experimental specimen. Once the target displacement is reached, the load recorded by the load cell is acquired by the MTS controller, and then is sent to the PSD controlling program through dynamic link libraries for the calculation of the displacement for the next step.

6) A displacement transducer is used for position control. Note that only the actuator’s internal displacement transducer was used in the current study although a dual displacement control loop was used by Shing et al. (1991). The purpose of using the displacement transducer external to the actuator in their tests was to reduce the secondary effect induced by the deformation of the reaction frame. In the current study, the deformation of the reaction frame was negligible since the experimental specimen was significantly more flexible than the reaction frame. Therefore, the displacement measured by the internal transducer was regarded to be equal to the command target displacement.

7) A steel reaction frame was used to support the actuator. As shown in Figure 4.2.a, the reaction frame consists of a 244cm long W24×117 horizontal beam bolted to two 518cm high W12×72 vertical columns. The bases of both vertical columns are welded to 244cm long W14×90 beams that are post-tensioned to the strong floor.
4.3 PSD Tests

4.3.1 SDOF System

4.3.1.1 Test Setup for SDOF System

The first set of tests was performed with a simply supported steel beam, representing a single-degree-of-freedom (SDOF) system. The experimental specimen was a W8x48 steel beam with length of 2286 mm. As shown in Figures 4.2.a-b, an actuator was mounted at mid-span of the beam to apply the inertial force of the lumped mass. For convenience of description, a coordinate system is constructed for the SDOF system, as shown in Figure 4.2.b. The origin is located at the left end of the beam and the horizontal axis extends along the beam. Note that the displacement is negative while the beam deforms downward since the upward deformation is considered positive in the current study. The external excitation force is

\[ P(t) = P_o + M a(t) \]  \hspace{1cm} (4.1)

where \( M \) is the lumped mass, \( a(t) \) is the acceleration of the mass, and \( P_o \) is a constant force used to push the beam to ensure that the ends of the beam remain in contact with the supports.

It should be noted that the constant component \( P_o \) of the external force \( P(t) \) can also help ensure that the actuator starts from a nonzero initial location. The design of a nonzero initial location is useful to mitigate experimental errors during the test. It can be explained as follows: if the test starts at a zero displacement, the displacement increment during the first several steps may be very small and the actuator may not respond to the command displacements. The resulting experimental errors may be accumulated and propagated to next steps during the test (Shing and Mahin, 1987). In addition, it becomes
easier to perform the test by keeping the beam pushed so that it can be simply placed on two steel end rollers without using any fixity bolts while the simply supported boundary condition is satisfied. Special and expensive fixture would be needed to resist uplift forces while allowing the degrees of freedom of simple supports.

It may be appropriate to note that the current study proposes an innovative way to represent a SDOF system in PSD testing. Shing et al. (1991), Darby et al. (1999) and Stojadinovic et al. (2005) used the cantilever column to mimic a SDOF system. It was a difficult task to sufficiently satisfy the clamped boundary condition of the cantilever column in their tests. Essentially, the cantilever column base is supported by a rotational and horizontal spring. With respect to the rotational deformation, for a full-scale experimental specimen, it is very challenging to fix the base satisfactorily to make the rotational stiffness at the base approach infinite (it works like a rotational spring). Similarly, the horizontal displacement at the base is hard to be avoided for a full-scale or large-scale experimental specimen. To satisfy the clamped boundary condition at the column base, in the test presented by Shing et al (1991), the column was welded with an end plate which was connected to the ground floor using post-tensioned bolts. As reported by Shing et al (1991), the experimental stiffness was still substantially lower than an ideal fixed-base column because of the imperfect base fixity.

In order to demonstrate that the current testing platform can be used for general excitation cases, the SDOF system was subjected to two types of external excitation: a harmonic excitation and a realistic earthquake. The El Centro NS component of 1940 Imperial Valley Earthquake was selected in the tests. Parameters used for each test are provided in Table 4.1. The tests were performed to investigate the effects of the corrector
stiffness \((K)\), convergence tolerance, and reduction factor. It should be noted that the corrector stiffness herein means the stiffness parameter used in the procedure to correct the convergence error at the end of each time step as described in step 15 of Table 3.1. Theoretically, the instantaneous stiffness should be used as the corrector stiffness. However, in PSD testing, the instantaneous stiffness is almost impossible to measure during tests. Therefore, the initial elastic stiffness or a stiffness that is slightly larger than the initial stiffness is used in the corrector procedure.

4.3.1.2 Test Results of SDOF Cases under Harmonic Excitations

For the harmonic excitation cases, the lumped mass was assumed to be 4.9 \(KN \cdot s^2/mm\). Before performing PSD tests, the initial elastic stiffness of the beam was measured experimentally as 37\(KN/mm\). A 5\% viscous damping was assumed. The external excitation was \(P(t) = -148 - 7.84 \sin(1.15t)\). The sign of minus for the constant force component means to push the beam downward all the time during the test in order to ensure that the ends of the beam remain in contact with the supports. It also should be noted that the downward deformation of the beam is defined to be negative during the test, as shown in Figure 4.2.b.

4.3.1.2.1 Effect of Corrector Stiffness

In order to mitigate the residual convergence error, Shing et al. (1991) introduced a correction procedure as described in Step 15 of Table 3.1: \(d_{i+1} = d_{i+1}^k + \Delta d_{i+1}^k\), and \(r_{i+1} = r_{i+1}^k + K\Delta d_{i+1}^k\), where \(d_{i+1}\) and \(r_{i+1}\) are respectively the displacement and restoring force at time step \(i+1\), \(d_{i+1}^k\), \(r_{i+1}^k\) and \(\Delta d_{i+1}^k\) are respectively the displacement, restoring
force, and residual convergence error at the $k^{th}$ iteration of the time step $i+1$, $K$ is the corrector stiffness used in the correction procedure. Ideally, the corrector stiffness $K$ should be updated using the instantaneous stiffness at each time step during the test. However, due to the difficulty of measuring the instantaneous stiffness during the test, the initial stiffness $K_i$ is usually employed. The measurement of the initial stiffness $K_i$ may not be correct completely. Moreover, the corrector stiffness $K$ employed in the test is not the instantaneous stiffness, and it may be different from the actual initial elastic stiffness $K_i$. Therefore, this induces a potential source of experimental errors. To investigate the possible errors induced by the application of the corrector stiffness $K$, different corrector stiffness values were used in the tests as summarized in Table 4.1.

Consider cases SOD-Harm-01 and 07 where the corrector stiffness employed in tests were 37 and 74 $KN/mm$, respectively, while the reduction factor and convergence tolerance remained one and 0.02 $mm$, respectively (see Table 4.1). Figure 4.3 shows that there are very slight differences between test results of these two cases. As shown in Figure 4.3, if the corrector stiffness is two times (74 $KN/mm$) of the initial stiffness (37$KN/mm$), test results are still reasonable and are almost the same as the case where the initial stiffness (37$KN/mm$) was employed. Hence, the use of the corrector stiffness larger than the actual ones (here it is 2 times) does not induce significant errors for the current SDOF cases. This may be because of the small tolerance ($TOL= 0.02 mm$) employed in the tests relative to the response amplitudes ($d_{max}=6mm$).
4.3.1.2.2 Effect of Convergence Tolerance

The convergence tolerance is dependent on the finite resolution of control and measuring devices. If a large tolerance is chosen, the iteration numbers can be reduced and therefore the testing time can be reduced if the test accuracy requirement can be satisfied. However, if the tolerance is too large, some local characteristics of the structural response cannot be captured. On the other hand, if a small tolerance is chosen, a large number of iterations are required to reach the dynamic equilibrium at each step. This may result in excessive testing time beyond practical limits, and too small tolerance may even result in loading reversals/hysteresis which harms the test accuracy. Another problem associated with the small tolerance is that it poses a stringent requirement on the resolution of test equipment, and thus the test setup can become prohibitively expensive. Therefore, the convergence tolerance should be large enough while satisfying the accuracy requirement.

Consider cases of SOD-Harm-01 and 03 where the tolerances were 0.02 and 0.04\,mm, respectively, while the corrector stiffness and reduction factor were maintained as 37\,KN/mm and one, respectively. Figure 4.4 shows that the responses of two cases are almost the same, indicating that the increase of tolerance from 0.02 to 0.04\,mm does not induce significant errors. It means that a tolerance as large as 0.04\,mm is still acceptable for the current SDOF system.

4.3.1.2.3 Effect of Reduction Factor

The reduction factor can be used to reduce the chance of loading reversals/hysteresis and help achieve a more or less uniform convergence for all degrees of freedom for
MDOF systems as proposed by Shing et al., 1991. Consider cases of SOD-Harm-01 and 05 where the reduction factors were 1.0 and 0.85, respectively, while the corrector stiffness and convergence tolerance were maintained equal to $37\text{KN/mm}$ and 0.02$\text{mm}$, respectively. Comparison of these two cases can reveal the effect of the reduction factor on test results when the initial stiffness is used in the correction procedure and the tolerance is as small as $TOL=0.02\text{mm}$. Figure 4.5 shows that the effect of the reduction factor is negligible if the tolerance is small and the initial stiffness is measured accurately. This may be because the system was a SDOF system and therefore it always achieved the intended uniform convergence. In addition, this may be because this system remained elastic during tests and therefore the loading reversal problem did not exist at all. In other words, the reduction factor did not have any effect on the results for the current SDOF elastic system.

4.3.1.2.4 Summary for SDOF Cases under Harmonic Excitation

Figure 4.6 shows the test results of all harmonic SDOF cases shown in Table 4.1. As shown in Figure 4.6 there is no appreciable difference between these cases, indicating that parameters including the corrector stiffness, tolerance, and reduction factor do not have significant effects on the test results in the current tests. This may be because: 1) the current system, a simple beam, was the simplest structure; 2) the tolerance employed in all tests was relatively small, ranging from 0.02 to 0.04$\text{mm}$, and therefore bound the experimental errors induced by these parameters; and 3) the system remained elastic and the loading reversal did not induce errors during tests.
The test results of all harmonic excitation SDOF cases presented in Table 4.1 are further evaluated by comparing them with numerical simulation results. The simulation results were obtained using both the pre-testing simulation program and ABAQUS, respectively. Note that the simulation solutions obtained from the pre-testing simulation program and ABAQUS are identical. In the numerical simulations reported herein, the stiffness of the beam is the measured value obtained from the experimental results. The results obtained through numerical simulations are called the analytical solutions herein. Figure 4.6 shows that all the test results are consistent with the analytical solution (simulation results), indicating that the test results of all cases are acceptable.

4.3.1.3 Test Results of SDOF System under El Centro Excitation

To further demonstrate the capability of the PSD testing platform developed in the current study, tests were performed when the system was subjected to realistic earthquake loads. The test setup and structural model are shown in Figures 4.2.a and b, respectively. A mass of 15.78KN-s²/mm was lumped at the mid-span of the beam, and a 2% viscous damping was assumed. An excitation force $P(t) = -148-ME(t)$ was applied to the SDOF system, where $M$ is the lumped mass and $E(t)$ is the NS component of El Centro excitation with a scale factor of 1%.

Consider cases of SOD-ElCentro-01 and 02 (see Table 4.1) for which the values of corrector stiffness are 37 and 45KN/mm, respectively, while the tolerance and reduction factor respectively remained the same as 0.02mm and one. Figure 4.7 shows that the tests results for the two cases are almost the same, indicating that the corrector stiffness does not induce appreciable experimental errors when the tolerance is as small as 0.02mm.
4.4 PSD Tests for MDOF Systems

4.4.1 Solution for Data Exchange between Computers

In the hybrid PSD tests of MDOF systems, the entire structure is divided into experimental and analytical substructures. The experimental substructure is physically tested and the analytical substructure is evaluated utilizing the finite element program ABAQUS. In the current study, the computer with the PSD controlling program and MTS controller is called the testing engine. The other computer with the finite element program ABAQUS and a C++ computer program for data exchange between computers is called the simulation engine. The testing engine was used to generate the target displacement for each degree of freedom of the structure and drive the actuators (by using the MTS controller) to perform the physical test of the experimental specimen. The simulation computer was utilized to calculate the restoring force of the analytical substructure by using ABAQUS. These two engines were linked through the network. The data exchange solution between two computers is illustrated in Figure 4.8. The implementation procedures/steps are further explained as follows:

1) The simulation engine is started. The simulation engine sets the status of the displacement of the analytical substructure as “Not Ready”, and then keeps checking the status of the displacement till the status of the displacement of the analytical substructure is changed to “Ready”. The statuses of both the displacement and the restoring force of the analytical substructure are recorded in a file (here it is called Flag-1). It should be noted that the simulation engine is started before the testing engine. This is to allow the test operators to focus on the physical experimental activities during the test and do not need to pay attention to
the simulation engine any more once the PSD test is started. This can also allow test operators to have a more flexible schedule. For example, one can start the simulation engine on the first day while doing the physical test on the second day.

2) The testing engine is started. The PSD controlling program in the testing engine generates the target displacement of the analytical substructure by solving the governing equation of motion, and then sends the target displacement of the analytical substructure to the simulation engine and updates the displacement status of the analytical substructure to “Ready”. Once the testing engine finishes sending the displacement of the analytical substructure to the simulation engine, the status of the restoring force of the analytical substructure is set to “Not Ready”, and it begins to check the status of the restoring force of the analytical substructure till it becomes “Ready”. The statuses of both the displacement and the restoring force of the analytical substructure are saved in the file, Flag-2, which is sent to the simulation engine. In addition, the target displacement of the analytical substructure is put into an ABAQUS input file, and then this ABAQUS input file is sent to the simulation engine.

3) The simulation engine calls the finite element program ABAQUS once the displacement status of the analytical substructure becomes “Ready”, and then the restoring force of the analytical substructure is computed through ABAQUS based on the ABAQUS input file received from the testing engine. Once ABAQUS finishes computing the restoring force of the analytical substructure, the status of the restoring force is updated to “Ready”, and at the same time the displacement status of the analytical substructures is changed to “Not Ready”.
The statuses of both the displacement and the restoring force are recorded in a file (here it is called Flag-1) and sent to the testing engine. In addition, the ABAQUS output file is sent to the testing engine once ABAQUS finishes computing the restoring force of the analytical substructure.

4) After the status of the restoring force of the analytical substructure becomes “Ready”, the PSD controlling program in the testing engine reads the restoring force of the analytical substructure from the ABAQUS output file received from the simulation engine, and then formulates this restoring force into the governing equation of motion using the substructuring technique. After the governing equation of motion is formulated, the PSD controlling program generates the displacement of the analytical substructure by solving the governing equation of motion. Once the displacement of the analytical substructure is generated, the displacement status of the analytical substructure is updated to “Ready” and the status of the restoring force of the analytical substructure is updated to “Not Ready”. The statuses of both the displacement and the restoring force are updated in the file, Flag-2, and sent to the simulation engine. In addition, the testing engine generates an ABAQUS input file based on the target displacement of the analytical substructure, and then this ABAQUS input file is sent to the simulation engine.

5) Once the file, Flag-2, is received by the simulation engine, it starts ABAQUS and computes the restoring force of the analytical substructure based on the ABAQUS input file received from the testing engine as in step 3. After the simulation finishes the computation of the restoring force of the analytical substructure, the

96
statuses of the restoring and displacement of the analytical substructure are updated in the file, Flag-1. In addition, the ABAQUS output file which includes the restoring force of the analytical substructure is sent to the testing engine as in step 3. The test will be performed recursively till all the time steps are evaluated.

### 4.4.2 Test Setup for MDOF System

For the hybrid PSD tests of MDOF systems in the current study, the experimental substructure was a simply supported beam and the analytical substructure was a frame consisting of a horizontal beam and a vertical column that was clamped at its base, as shown in Figure 4.9.a. The connection between the experimental and the analytical substructures was a hinge in the middle span of the simply supported beam so that there was only one horizontal displacement degree of freedom at the interface. Therefore, the experimental substructure was subjected to only the horizontal force in the middle span when the entire structure was subjected to ground motions.

It should be noted that the global horizontal force due to the ground excitation applied to the experimental substructure (the simple beam shown in Figure 4.9.a) was transformed into a local force that is perpendicular to the axis of the beam during the test in the laboratory. In other words, the excitation force was applied to the simple beam in the transverse direction during the test in the laboratory as shown in Figure 4.9.b. By designing the input excitation, the simple beam can be pushed during the entire test as what was done in the SDOF case. Essentially, the testing setup for the experimental substructure was the same as that for the SDOF case shown in Figure 4.2. Although the simply supported beam used in the MDOF cases was the same as in SDOF cases, for the
MDOF cases, the beam was a part of the entire structure whereas for the SDOF cases it represented the entire structure.

Since the experimental specimen was still a simple beam and the experimental setup was essentially the same as the SDOF cases, the same test setup discussed in Section 4.3.1.1 was used. However, two computers were used for the hybrid tests of MDOF systems. One was used as the testing engine to install the PSD controlling program and the MTS controller to apply the target displacements to the experimental specimen (the simple beam). The other one was used as the simulation engine to operate ABAQUS to evaluate the analytical substructure. The data exchange between the two computers was achieved through the implementation procedure described previously in Section 4.4.1.

The dimension and size of the simply supported W8x48 steel beam were the same as in the SDOF cases and are not repeated here. The analytical substructure was the frame, as shown in Figure 4.9, with both the height of the vertical column and the span of the horizontal beam being 1143 mm, and both members had a rectangular cross section with dimensions of 95.2 x 95.2 mm. The mass was lumped in the middle span of the horizontal beam of the analytical substructure as shown in Figure 4.9. The initial stiffness of the entire system can be obtained using the substructuring technique, written as

\[
K_I = \begin{bmatrix}
K_{11}^E + K_{11}^a & K_{12}^a \\
K_{21}^a & K_{22}^a
\end{bmatrix}
= \begin{bmatrix}
37 + 11.52 & 6583.5 \\
6583.5 & 8.7E6
\end{bmatrix}
= \begin{bmatrix}
48.52 & 6583.5 \\
6583.5 & 8.7E6
\end{bmatrix}
\]  \tag{4.2}

where superscripts 1 and 2 denote the horizontal and rotational degrees of freedom, respectively, \( E \) and \( a \) denote the experimental and analytical substructures, respectively.

The element \( K_{11}^E = 37 \text{KN/mm} \) in the stiffness matrix included in equation (4.2) was
obtained by calibrating the experimental data of SDOF cases and the other elements in the stiffness matrix were calculated by modeling the analytical substructure in ABAQUS.

4.4.3 Test Results of MDOF System

4.4.3.1 MDOF System with Elastic Analytical Substructure under Harmonic Excitations

The lumped mass was assumed to be 10 $KN\cdot s^2/mm$ and the viscous damping ratio was taken as 5%. The external force $P(t) = -P_0 - M\sin(\omega t) = -115.2 - 10\sin(5t)$ was applied to the system. As stated previously, the minus sign means the force was applied to the simple beam downward all the time during the physical tests. It should be noted that the constant coefficient of $P_0 = -115.2$ KN was designed in order to ensure that the beam would be pushed down during the entire test. To investigate the effects of corrector stiffness, convergence tolerance, and reduction factor, parametric studies were performed in accordance with the cases shown in Table 4.2.

4.4.3.1.1 Effect of Corrector Stiffness

The effect of the corrector stiffness on the results was investigated when the structure was subjected to harmonic excitation. Consider SB-Harm-01 and 02 where the values of corrector stiffness were $K_i$ and 1.2 $K_i$, respectively, while the tolerance and reduction factor were respectively the same as $0.2mm$ and one. The initial stiffness matrix $K_i$ is the matrix shown in equation (4.1).

As shown in Figures 4.10.a-b, the test results for these two cases are very close, implying that the variation of the corrector stiffness, i.e., the corrector stiffness being 1.2
times the initial elastic stiffness \((K=1.2K_i)\), does not induce significant errors in the test results. It seems that a 20% variation of the corrector stiffness is acceptable as long as the tolerance remains equal to 0.2\(mm\), which corresponds to 5% of the maximum response amplitude \((4mm)\).

**4.4.3.1.2 Effect of Tolerance**

The effect of the convergence tolerance is investigated by comparing the test results for SB-Harm-02 and 03 where the tolerances were respectively 0.12 and 0.2\(mm\) while the corrector stiffness and reduction factor in both cases were set to 1.2\(K_i\) and one, respectively.

As shown in Figures 4.11.a-b, the variation of the tolerance does not induce appreciable differences in the test results when the corrector stiffness and reduction factor are the same. This may be because the tolerances used in the tests were relatively small compared with the response amplitude, corresponding to 3-5% of the maximum response amplitude.

**4.4.3.1.3 Effect of Reduction Factor**

The effect of the reduction factor is investigated by comparing the test results from SB-Harm-02 and 04 where the reduction factors were respectively one and 0.8 while the corrector stiffness and tolerance were kept the same as 1.2\(K_i\) and 0.2\(mm\), respectively.

As shown in Figures 4.12.a-b, the test results of these two cases are almost the same, indicating that the influence of the reduction factor is negligible. This may be because the structure remained elastic during the test. As explained previously in the SDOF cases,
Shing et al. (1991) proposed to use the reduction factor during tests in order to mitigate the experimental errors induced by load reversal/hysteresis for nonlinear structures. However, the current structure remained elastic during tests; thus, the test results from these two cases with different reduction factors are almost the same, as shown in Figures 4.12.a-b.

4.4.3.1.4 Summary of MDOF System with Elastic Analytical Substructure under Harmonic Excitation

The test results of all the cases plotted in Figures 4.13.a-b indicate that the differences of the test results from various cases are quite slight. It can be further noted that, within the time period $t= [0, 4]$ seconds, the test results for different cases are almost the same even when the corrector stiffness varied from $K_i$ to $1.2 K_i$, or the tolerance shifted from $0.12$ to $0.2 mm$. This may be because the effects induced by the corrector stiffness or the convergence errors are relatively small compared with the external input excitation in terms of energy during this period. With the input excitation energy decreasing during the period $t= [4, 15]$ seconds, the error effects induced by the stiffness and convergence tolerance become a little more pronounced although it is still rather small.

The test results of all the cases shown in Table 4.2 are compared with the simulation results which are obtained using the pre-testing simulation program and ABAQUS. The simulation results obtained through the pre-testing simulation program and ABAQUS are essentially identical, and therefore they are called the analytical solution herein. The stiffness of the experimental substructure (the simple beam) is the measured value
obtained from the experimental tests. As shown in Figures 4.13.a-b, test results are consistent with the numerical simulations (analytical solution).

4.4.3.2 MDOF System with Elastic Analytical Substructure under Earthquakes

The hybrid PSD testing platform was employed to test MDOF systems subjected to realistic earthquake loads. External excitation force $P(t) = -115.2 - ME(t)$ was used, where $M = 0.75 KN-s^2/mm$ is the lumped mass and $E(t)$ is the earthquake excitation corresponding to 1.5% NS component of El Centro excitation. The minus sign means the force was applied downward to the simple beam so that the simple beam can be pushed down during tests. A 2% viscous damping ratio was assumed. Four tests were conducted to evaluate the effects of the parameters listed in Table 4.3.

4.4.3.2.1 Effect of Tolerance

To investigate the effect of the value of tolerance, tests were performed for SB-EC-EL-01 and 04 where the tolerances were respectively 0.2 and 0.3 mm while the corrector stiffness and reduction factor were taken as $K_t$ and one, respectively. As shown in Figures 4.14.a-b, the effect of tolerance on test results is small in general, although such an effect is more appreciable during the time period $t = [8, 12]$ than during the other time periods.

As shown in Figures 4.14.a-b, the response amplitude during time period $t = [8, 12]$ is smaller than during the other time periods. This means that the external excitation input energy during time period $t = [8, 12]$ is smaller than during the other time periods. It implies that the effect of the tolerance decreases as the external excitation input energy increase. The larger the external input energy for a time period, the smaller is the effect of the tolerance.
4.4.3.2.2 Effect of Reduction Factor

The effect of the reduction factor is evaluated by comparing SB-EC-EL-02 and 04 where the values of reduction factor were respectively one and 0.8 while the stiffness and tolerance were the same as \( K=K_i \), and \( TOL=0.3\text{mm} \), respectively. Figures 4.15.a-b show that the difference between the two cases during time period \( t=[8, 12] \) is more noticeable than during the other time periods. The difference between the two test results is negligible during the other time periods.

Interestingly, a similar phenomenon is also noted in the above cases where tolerances are different. It may imply that the reduction factor works in a similar way as the tolerance. It can be further explained as follows: during the test, the displacement increment for each iteration is reduced by introducing a reduction factor \((RF)\) as described in step 9 of Table 3.1 (Shing et al., 1994):  
\[
d_{i+1}^{k+1} = d_{i+1}^{k} + RF \times \Delta d_{i+1}^{k},
\]
where \(d_{i+1}^{k+1}\) and \(d_{i+1}^{k}\) are the displacements for the \((k+1)\)th and \(k\)th iterations at time step \(i+1\), respectively, and \(\Delta d_{i+1}^{k}\) is the displacement increment calculated in step 7 of Table 3.1. Because \(\Delta d_{i+1}^{k}\) is approximately equal to the tolerance \((TOL)\) as indicated in step 8 of Table 3.1, it can be noted that the term \(RF \times \Delta d_{i+1}^{k}\) functions essentially as employing a tolerance smaller than \(\Delta d_{i+1}^{k}\) as long as a reduction factor \((RF)\) less than unit is used. Therefore, the introduction of the reduction factor (less than unit) in SB-EC-EL-02 makes the displacement increment for each iteration step smaller than in SB-EC-EL-04.
4.4.3.2.3 Effect of Corrector Stiffness

Consider cases SB-EC-EL-03 and 04 where the corrector stiffness were respectively $1.2k_i$ and $k_i$ while the tolerance and reduction factor were kept the same as $0.3mm$ and one, respectively.

As shown in Figures 4.16.a-b, the difference between the two test results is generally quite small, although such a difference during time period $t= [8, 12]$ is more noticeable than during the other time periods. This may be due to the fact that the external excitation energy during time period $t= [8, 12]$ is smaller than during other time periods, as shown in Figures 4.16.a-b.

4.4.3.2.4 Summary of MDOF System with Elastic Analytical Substructure under Earthquakes

Figures 4.17.a-b show the test results of all the cases shown in Table 4.3. The test results are further evaluated by comparing them with the simulation results (analytical solution). The simulation results are obtained using the pre-testing simulation program and ABAQUS. Figures 4.17.a-b show that the test results are consistent with the simulation results.

As shown in Figures 4.16.a-b, all the test results are almost the same except for SB-EC-EL-03 where the corrector stiffness, $1.2K_i$, is 20% larger than the actual stiffness. This implies that the effects due to selection of the values of tolerance and reduction factors are negligible whereas the effect induced by the correct stiffness is more appreciable. It can be noted from Figures 4.17.a-b that the effect induced by the tolerance, the corrector stiffness and reduction factor increases with the decrease of the
relative input excitation energy. This phenomenon can be clearly noted during the time period \( t = [8, 12] \) where the external excitation energy is relatively smaller than during other time periods.

### 4.4.3.3 MDOF System with Nonlinear Analytical Substructure under Earthquakes

The hybrid PSD testing platform developed in the current study was utilized to test the MDOF system with a nonlinear analytical substructure. The dimensions of the entire structure were the same as those for the case where the analytical substructure was elastic, as shown in Figure 4.9. The material of the analytical substructure was assumed to be nonlinear with the elastic Young’s modulus \( E = 200 \text{ GPa} \), yielding stress \( \sigma_y = 0.01 \text{ KN/mm}^2 \), and hardening ratio \( \gamma = 0.1 \), as shown in Figure 4.18. An external excitation force \( P(t) = -115.2 - ME(t) \) was applied to the system, where \( M = 0.75 \text{ KN-S}^2/\text{mm} \) is the lumped mass and \( E(t) \) is the earthquake excitation corresponding to 1.5% NS component of El Centro earthquake. The minus sign means the force was applied downward to the simple beam during tests. The viscous damping ratio was taken as 2%. Parameters used for the different cases are shown in Table 4.4.

#### 4.4.3.3.1 Effect of Tolerance

Consider cases of SB-EC-NL-01 and 02 where tolerances were respectively 0.2 and 0.3 mm while the values of reduction factor and corrector stiffness for both cases were taken as one and \( K_i \), respectively. Figures 4.19.a-b show that the difference between the two cases are negligible, indicating that the test result is not sensitive to the variation of
the tolerance if the reduction factor is one and the stiffness equals the initial elastic stiffness.

Figure 4.19.c shows the relation between shear force and horizontal displacement of the nonlinear analytical substructure which is essentially a shear frame. As shown in Figure 4.19.c, the responses of the shear frame in both cases are almost the same, but the response of SB-EC-NL-02 shifts slightly to the right of that for SB-EC-NL-01. This may be because a bigger tolerance in the former case leads to slightly smaller displacement amplitudes as shown in Figure 4.17.a.

4.4.3.3.2 Effect of Reduction Factor

Consider cases SB-EC-NL-02 and 03 where the reduction factors were respectively taken as one and 0.8, while the tolerance and stiffness are the same in both cases as 0.3mm and $K_i$, respectively. Figures 4.20.a-b show that the responses of these two cases are almost the same whereas the response for SB-EC-NL-03 has very slightly larger amplitudes. Figure 4.20.c shows the shear force versus the horizontal displacement for the nonlinear shear frame (analytical substructure). As shown in Figure 4.20.c, the responses of the shear frame in the two cases are almost the same with the exception of a slight shift to the left for case SB-EC-NL-03. This is because the displacement for SB-EC-NL-03 is slightly larger than in SB-EC-NL-02 case, as shown in Figure 4.20.a

4.4.3.3.3 Effect of Corrector Stiffness

The effect of the corrector stiffness on the test results is investigated by comparing the test results of SB-EC-NL-01 and 04 where the values of corrector stiffness employed
were respectively $K_i$ and $1.2K_i$ while the tolerance and reduction factor were kept the same in both cases as 0.2mm and one, respectively.

As shown in Figures 4.21.a-b, the two test results are almost the same during most time periods, except for the time period $t = [10, 12.5]$ when the external input excitation energy is relatively small. Recall that a similar phenomenon was noted for the MDOF systems with the elastic analytical substructure. It may be appropriate to conclude that errors induced by the corrector stiffness can become appreciable when the external input excitation energy is small, whether the analytical substructure is elastic or nonlinear.

As shown in Figure 4.21.c, the responses of the shear frame (nonlinear analytical substructure) in the two cases are almost the same whereas the response of SB-EC-NL-04 shifts slightly to the right. This is because the displacement in the SB-EC-NL-04 case is slightly smaller as shown in Figure 4.21.a. Recall that the same phenomenon is noted for the MDOF cases with the elastic analytical substructure. Therefore, it seems that the introduction of a corrector stiffness larger than the initial elastic stiffness induces a positive damping, whether the analytical substructure is linear or nonlinear. Actually, this is expected. In PSD testing, the corrector stiffness is introduced to avoid displacement overshooting which can introduce a negative damping and thus make the response diverge.

### 4.4.3.3.4 Summary of Test Results for MDOF with Nonlinear Substructure

Figures 4.22.a-c show the test results of MDOF systems with the nonlinear analytical substructure. As shown in Figures 4.22.a-c, similar to the MDOF cases with the elastic analytical substructure, the difference between the test results is negligible during most
time periods, indicating the effect of tolerance, corrector stiffness, and reduction factor is small in PSD testing. It can also be observed that the effects induced by these parameters are negligible by comparing the analytical solution, as shown in Figures 4.22.a-c. The test results are further compared with the numerical solutions (analytical solution) which are obtained using the pre-testing simulation program and ABAQUS. As shown in Figures 4.22.a-c, the test results are consistent with the analytical solution (simulation results).

4.5 Procedures to Handle Unexpected Situations

Some unexpected situations may happen during physical tests due to some uncertainties associated with the experimental specimens despite pre-test simulations that can provide some information to predict the response of structures. For example, it may be found during the test that the test fixture may have to be strengthened and/or stiffened, displacement transducers with larger strokes may have to be installed, etc. Under these situations, the test has to be stopped or paused so that these unexpected issues can be resolved before continuing the test. If a short time is needed to fix the unexpected problems or visually document the response of specimens, a Pause procedure can be used to temporarily hold the test. If a long time is required, a Restart procedure needs to be embedded in the testing platform so that the test operators can terminate a test and restart the test later from the exact location where it is terminated before. It should be noted that if the experimental substructure is nonlinear, the termination of a test will change the deformation history of the structure. This is really a compromise. To save a costly test which is unfortunately facing an unexpected situation and has to be terminated, the best
way is to shut down the experimental part so that the test can be continued later on. Terminating a test is the last choice for test operators under a special severe situation. Actually, this is also one of the reasons why the Hold/Pause procedure is developed in the current testing platform so that a test does not need to be terminated if the unexpected situation is not so severe that the continuity of the test is impossible.

4.5.1 Pause/Hold Procedure

Pausing a test (here it is called Hold), instead of terminating it, is an effective means to handle some unexpected situations during tests so that test operators have a short time to check and fix unexpected problems. This option also offers an opportunity to hold the test while the visual behavior of the test specimen is documented. During the pause period, the PSD controlling program, the MTS controller, and the data exchange between the testing and simulation computers are all paused. A Hold/Pause procedure is embedded in the current testing platform. This procedure was verified experimentally. The physical tests were continued after the tests were paused to check the test setup.

4.5.2 Start-Restart Procedure

Terminating a test is a viable means to handle severe unexpected situations so that test operators have enough time to fix the problems. For example, it may be found during the test that the range of displacement transducer is smaller than that required. Under this situation, the test has to be terminated so that the transducer can be replaced so that the test can be completed in the second phase. The first phase before the termination of the test herein is called the Start phase. The second phase is called the Restart phase. A Start-
Restart procedure was embedded in the current testing platform and a hybrid PSD test was conducted to check this procedure experimentally. The structure employed in the test was the same as that shown in Figure 4.9, consisting of the experimental and analytical substructures. The material of the analytical substructure is nonlinear with elastic modulus of 200$\text{GPa}$, yield stress 0.01$\text{KN/mm}^2$, and hardening ratio of 0.1 as shown in Figure 4.18. The lumped mass was assumed to be $M=0.75\text{KN-mm/s}^2$. The force applied to the system was $P(t) = -115.2\cdot ME(t)$, where $E(t)$ is the ground motion with the scale of 1.5% El Centro earthquake excitation. The time step chosen in the test was $\Delta t=0.02$ second. The Start-Restart procedure was verified experimentally as follows:

1) The first step was to perform a complete hybrid test without any stop during the test and this complete test can be considered as the benchmark for the purpose of comparison. It should be noted that the experimental specimen (here it is the simple beam) should be deformed within the elastic range so that the experimental specimen used in the followed second phase (restart phase) remains in the same state as in the benchmark test.

2) Secondly, the same test was run again with the same experimental specimen and the analytical substructure as in the benchmark test. However, the test was terminated at an intermediate time $t=0.8$ second of the excitation. This is called the Start phase. To make sure the test was terminated completely, the actuator was unloaded to zero position and the MTS system was completely shut down. The entire testing computer was completely shut down including the PSD controlling program and the MTS controller. The communication between the testing and simulation computers stopped. The status of the entire hybrid testing system after
the test was terminated is demonstrated in Figure 4.23. It should be noted that the simulation computer was still on (not shut down), waiting for the displacement of the analytical substructure for the next time step. The simulation computer was still checking the status of the displacement of the analytical substructure which should be generated by the testing computer until it was updated by the data sent from the testing computer. The simulation computer was kept running to ensure that the test can be restarted from the exact location for the nonlinear analytical substructure.

3) The third step was to restart the test. This is called the Restart phase. In order to make sure that the test was restarted from the exact location where it was terminated previously, the testing computer was restarted and it retrieved the data of the displacement, velocity, acceleration, and restoring force from the file saved before the test was terminated. The governing equation of motion was then solved by the PSD controlling program in the testing computer so that the displacement of the analytical substructure was sent to the simulation computer. Once the simulation computer received the displacement from the testing computer, it drove ABAQUS to evaluate the restoring force of the analytical substructure and then sent the restoring force of the analytical substructure to the PSD controlling program in the testing program. This loop was implemented iteratively till the entire test was completed. It should be noted that the ABAQUS output files which contain the entire load and deformation history of the analytical substructure were kept in the simulation computer all the time so that the evaluation of the analytical substructure was restarted from the exact position where it was terminated. This is
very important for structural evaluation of a nonlinear analytical substructure. Otherwise, the load-deformation history of the nonlinear analytical substructure will be lost.

The test results of the displacement and rotation of the structure are shown in Figures 4.24.a-b, respectively. Figures 4.24.a-b show that test results obtained through the start-restart procedure is consistent with that for the benchmark test. It verifies that the Start-Restart procedure presented in the current study is an acceptable solution to save a costly test that might have to be terminated because of some unexpected situations.

4.5.3 Further Applications of Start-Restart Procedure

Typically, in shake table tests or PSD tests, the excitation motion is a specific one. To tackle this problem, the start-restart procedure developed in the current study can be employed to test a case where the system is subjected to combined excitations. For example, the excitation consists of two phases: the motion is a realistic earthquake excitation during the first phase, and it is a harmonic excitation during the second phase. A test was performed to demonstrate how to handle this situation in PSD testing using the start-restart procedure developed in the current study.

The entire structural system considered was the same as that used to verify the Start-Restart procedure discussed previously, as shown in Figure 4.9. The excitation consisted of two phases: 1) Phase I: \( P(t) = -115.2 - MgE(t), \ t = [0, 5] \); and 2) Phase II: \( P(t) = -115.2 - 0.1MgS\sin(3.8t), \ t = [5, 10] \), where \( E(t) \) is the El Centro earthquake excitation, and \( g \) is the gravity acceleration. The complete excitation is shown in Figure 4.25.
The first step was to run a complete test where external excitations for phases I and II were combined into an individual excitation input file and this complete test is considered as the benchmark. The period for this complete test is 10 seconds, \( t = [0, 10] \). The second step was to rerun the test with excitation of phase I in which the test was terminated at time \( t=5 \) second. The third step was to restart the test beginning at time \( t=5 \) second and to apply the excitation for phase II. Figures 4.26.a-b show that the test results used the Start-Restart procedure are the same as those obtained through a complete benchmark test.

### 4.6 Conclusions and Remarks

The purpose of this chapter is to present a hybrid PSD testing platform developed at the University of Cincinnati Large-Scale Test Facility. This testing platform was developed in an object-oriented C++ framework architecture. The feature of the object-oriented architecture facilitates modification and upgrading as new PSD testing techniques are developed. Due to its capability for interfacing with general finite element programs such as ABAQUS, it can be used to test general structures where both the experimental and analytical substructures are nonlinear.

The complete testing platform was experimentally verified by performing two sets of physical tests including SDOF and MDOF systems. The test results show that this testing platform can provide reliable results for the evaluation of dynamic or earthquake performance of structures. The accuracy and stability of this testing platform are ensured with the use of the alpha-method integration algorithm which has characteristics of unconditional stability and high-order accuracy. Hold/Pause and Start-Restart procedures
embedded in the current testing platform can be used to handle different unexpected situations at different times, and to allow multiple ground motions applied sequentially.
### Table 4.1 Parameters used in tests of SDOF cases

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Corrector Stiffness $K$ (KN/mm)</th>
<th>Tolerance ($Tol$) (mm)</th>
<th>Reduction Factor ($RF$)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>SOD-Harm-03</td>
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<td>0.04</td>
<td>1.0</td>
</tr>
<tr>
<td>SOD-Harm-04</td>
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<td>0.04</td>
<td>1.0</td>
</tr>
<tr>
<td>SOD-Harm-05</td>
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<td>0.85</td>
</tr>
<tr>
<td>SOD-Harm-06</td>
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<td>SOD-ElCentro-01</td>
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<tr>
<td>SOD-ElCentro-02</td>
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<td>1.0</td>
</tr>
</tbody>
</table>

### Table 4.2 Parameters used in tests of MDOF for SB harmonic cases

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<th>Tolerance ($Tol$) (mm)</th>
<th>Reduction Factor ($RF$)</th>
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<td>1.0</td>
</tr>
<tr>
<td>SB-Harm-02</td>
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<td>1.0</td>
</tr>
<tr>
<td>SB-Harm-03</td>
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</tr>
<tr>
<td>SB-Harm-04</td>
<td>$1.2 \ K_i$</td>
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</table>
Table 4.3 Parameters used in tests of MDOF for SB-EC cases

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<th>Corrector Stiffness</th>
<th>Tolerance (mm)</th>
<th>Reduction Factor (RF)</th>
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</thead>
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<td>$K_i$</td>
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</tr>
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<td>$K_i$</td>
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<td>0.8</td>
</tr>
<tr>
<td>SB-EC-EL-03</td>
<td>1.2 $K_i$</td>
<td>0.3</td>
<td>1.0</td>
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<tr>
<td>SB-EC-EL-04</td>
<td>$K_i$</td>
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<td>1.0</td>
</tr>
</tbody>
</table>

Table 4.4 Parameters for cases with nonlinear analytical substructures

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Corrector Stiffness</th>
<th>Tolerance (mm)</th>
<th>Reduction Factor (RF)</th>
</tr>
</thead>
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</tr>
<tr>
<td>SB-EC-NL-03</td>
<td>$K_i$</td>
<td>0.3</td>
<td>0.8</td>
</tr>
<tr>
<td>SB-EC-NL-04</td>
<td>1.2$K_i$</td>
<td>0.2</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Figure 4.1 Imposition of displacement to experimental specimen ( Peek and Yi, 1990a)

Figure 4.2.a Test setup for PSD testing of simply supported beam
Figure 4.2.b Structural model for PSD testing of simply supported beam

\[ P(t) = P_o + M a(t) \]

Figure 4.3 Test results of SDOF system using different corrector stiffness

\[ M_p(t) = P_o + Ma(t) \]
Figure 4.4 Tests results of SDOF system using different tolerances

Figure 4.5 Test results of SDOF system using different reduction factors
Figure 4.6 Test and simulation results of simple beam under harmonic excitation

Figure 4.7 Tests results of SDOF system under El Centro excitation
Figure 4.8 Data exchange solution between computers for hybrid PSD testing

Simulation Engine

Flag-2 (Displ. Not Ready, Restoring Force Ready)
ABAQUS Output File

Run ABAQUS

Flag-1 (Displ. Ready, Restoring Force Not Ready)
ABAQUS Input file

Testing Engine

Flag-2 (Displ. Not Ready, Restoring Force Ready)
ABAQUS Output File

Run C++ PSD
Controlling Program

Flag-1 (Displ. Ready, Restoring Force Not Ready)
ABAQUS Input file
Figure 4.9.a Structures for hybrid PSD tests of MDOF systems
Figure 4.9.b Setup for experimental substructure
Figure 4.10.a Displacement of MDOF system with elastic analytical substructure under harmonic excitation varying corrector stiffness

Figure 4.10.b Rotation of MDOF system with elastic analytical substructure under harmonic excitation varying corrector stiffness
Figure 4.11.a Displacement of MDOF system with elastic analytical substructure under harmonic excitation with different tolerance

Figure 4.11.b Rotation of MDOF system with elastic analytical substructure under harmonic excitation with different tolerance
Figure 4.12.a Displacement of MDOF system with elastic analytical substructure under harmonic excitation using different reduction factors

Figure 4.12.b Rotation of MDOF system with elastic analytical substructure under harmonic excitation using different reduction factors
Figure 4.13.a Tests results of displacement for MDOF system with elastic analytical substructure under harmonic excitation

Figure 4.13.b Tests results of rotation for MDOF system with elastic analytical substructure under harmonic excitation
Figure 4.14.a Displacement of MDOF with elastic analytical substructure under earthquakes using different tolerances

Figure 4.14.b Rotation of MDOF with elastic analytical substructure under earthquakes using different tolerances
Figure 4.15.a Displacement of MDOF system with elastic analytical substructure under earthquakes using different reduction factors

Figure 4.15.b Rotation of MDOF system with elastic analytical substructure under earthquakes using different reduction factors
Figure 4.16.a Displacement for MDOF system with elastic analytical substructure under earthquakes using different corrector stiffness

Figure 4.16.b Rotation of MDOF system with elastic analytical substructure under earthquakes using different corrector stiffness
Figure 4.17.a Displacement for MDOF with elastic substructure under earthquakes

Figure 4.17.b Rotation for MDOF with elastic substructure under earthquakes
Figure 4.18 Deformation model for nonlinear material

Figure 4.19.a Displacement of MDOF system with nonlinear substructure using different tolerances
Figure 4.19.b Rotation of MDOF system with nonlinear substructure using different tolerances

Figure 4.19.c Restoring force versus displacement of shear frame using different tolerances
Figure 4.20.a Displacement of MDOF system with nonlinear substructure using different reduction factors

Figure 4.20.b Rotation for cases with nonlinear substructure using different reduction factors
Figure 4.20.c Response of nonlinear analytical substructure using different reduction factors

Figure 4.21.a Displacement for MDOF with nonlinear substructure using different corrector stiffness
Figure 4.21.b Rotation for MDOF with nonlinear substructure using different corrector stiffness

Figure 4.21.c Response of nonlinear analytical substructure using different corrector stiffness
Figure 4.22.a Test and simulation results of displacement for MDOF system with nonlinear substructure

Figure 4.22.b Test and simulation results of rotation for MDOF system with nonlinear substructure
Figure 4.22.c Test and simulation results of response of nonlinear shear frame

![Graph showing shear force vs. displacement for different conditions.]

1. Keep latest ABAQUS output file saved before the test is terminated
2. Wait for target displacement of analytical substructure

Communication stopped

Simulation Computer

1. Shut down C++ PSD controlling program
2. Unload actuator to zero position
3. Shut down MTS

Testing Computer

Figure 4.23 Status of computers after termination of test
Figure 4.24.a Displacements obtained from complete test and start-restart test

Figure 4.24.b Rotations obtained from complete test and start-restart test
Figure 4.25 Combination of El Centro and harmonic excitations

Figure 4.26.a Displacement of MDOF system under combined excitations
Figure 4.25.b Rotation of MDOF system under combined excitations
Appendix 4.A

Experimental Design of Steel Beam Specimens

A simple steel beam is used to easily check the many aspects of the developed PSD testing platform. It is necessary to keep the beam elastic at all times so that it can be used repeatedly. Various aspects of design of this beam are summarized in the following.

Consider a simple beam subjected to a harmonic excitation and a constant force. The governing equation of motion is

\[ m\ddot{u} + c\dot{u} + ku = P_0 \sin(\omega t) + P_1 \]  

(4.A.1)

The particular solution can be written as

\[ u_{p1} = C \sin(\omega t) + D \cos(\omega t) \]  

(4.A.2)

\[ u_{p2} = \frac{P_1}{k} = (u'_{st})_0 \]  

(4.A.3)

The complementary part of the solution is

\[ u_c = e^{-\xi\omega t} \left[ A \cos(\omega_D t) + B \sin(\omega_D t) \right] \]  

(4.A.4)

Therefore, the complete solution to the dynamic equation (4.A.1) is

\[ u(t) = u_{p1} + u_{p2} + u_c = C \sin(\omega t) + D \cos(\omega t) + e^{-\xi\omega t} \left[ A \cos(\omega_D t) + B \sin(\omega_D t) \right] + \frac{P_1}{k} \]  

(4.A.5)

where the constants \( C \) and \( D \) are

\[ C = \frac{P_0}{k} \frac{1 - (\omega/\omega_n)^2}{1 - (\omega/\omega_n)^2 + \left[ 2\xi(\omega/\omega_n) \right]^2} \]  

(4.A.6)

\[ D = \frac{P_0}{k} \frac{-2\xi(\omega/\omega_n)}{1 - (\omega/\omega_n)^2 + \left[ 2\xi(\omega/\omega_n) \right]^2} \]  

(4.A.7)
where the natural frequency is $\omega_n = \sqrt{\frac{k}{m}}$, viscous damping is $\zeta = C/(2m\omega_n)$, damping frequency is $\omega_D^2 = \omega_n^2(1 - \zeta^2)$ static displacement due to the harmonic force is $(U_{st})_o = P_o/k$, and the static displacement due to the constant force is $(u'_{st})_0 = P_I/k$. The two parameters $A$ and $B$ can be derived by knowing the initial conditions:

Velocity of this single degree of freedom system (SOD) can be written as

$$\dot{u}(t) =$$

$$-\zeta \omega_n e^{-\zeta \omega_D t} [A \cos(\omega_D t) + B \sin(\omega_D t)] + \omega_D e^{-\zeta \omega_D t} [-A \sin(\omega_D t) + B \cos(\omega_D t)] + C \omega \cos(\omega t) - D \omega \sin(\omega t)$$

(4.4.8)

For example, the solution for a case with $\dot{u}(0) = 0$, $u(0) = P_I/k$, $\zeta = 3\%$, $\omega/\omega_n = 0.45$,

$\omega_n/\omega_D = 1.00045$, and $\omega/\omega_D = 0.4502$ will be:

$$C = 1.25248 \frac{P_o}{k}, \quad D = -0.0424 \frac{P_o}{k}, \quad A = -D = 0.0424 \frac{P_o}{k},$$

and $B = -\left(\frac{\omega}{\omega_D}\right)C - \zeta \left(\frac{\omega}{\omega_D}\right)D$

$$B = -(0.4502)(1.25248)(P_o/k)-(0.03)(1.00045)(0.042)(P_o/k) = (0.56514)(P_o/k)$$

The complete solution for this special case can be written as

$$u(t) = \frac{P_o}{k} [1.25 \sin \omega t - 0.0424 \cos \omega t + e^{-\zeta \omega_D t} (0.0424 \cos \omega_D t + 0.565 \sin \omega_D t)] + P_I/k$$

Consider a W8x48 steel simply supported beam with the length of 2286mm. Per Manual of Steel Construction AISC LRFD 2003 (AISC, 2003), the properties of the steel beam are: cross sectional area $A = 9097mm^2$, moment of inertia $I = 76.6E6mm^4$ and the elastic modulus $E = 205GPa = 205KN/mm^2$. Assuming a mass $M = 4.41KN$-sec$^2$/mm is lumped at the mid-span of the beam, and the damping ratio of the beam is $\zeta = 3\%$. The
stiffness $K$, the natural frequency $\omega_n$ and the damping constant $C$ of the simply supported beam can be respectively calculated as:

$$K = \frac{48EI}{L^3} = \frac{(48)(205)(76.6)(10^5)}{(2286)^3} = 63.1KN/mm$$

$$\omega_n = \sqrt{\frac{k}{M}} = \sqrt{63.1/4.41} = 3.78rad/sec$$

$$C = 2M\omega_n\xi = 2(4.41)(3.78)(0.03) = 1.0KN-sec/mm$$

If an excitation $P(t) = P_0 + P_1 \sin(\omega t) = 112 + 44.8\sin(0.45\omega_n t)$ is applied at the mid-span of the beam, it can be noted that at the initial time $t=0$, the velocity and acceleration are zero, and the initial displacement of the beam should be $u(0)=P_0/k$.

Check 1. Solution Check

The analytical solution of the response of the beam described above is double checked by comparing it with the simulation result obtained through the PSD pre-testing simulation program developed in Chapter 3. As shown in Figure 4.A.1, the analytical solution is exactly consistent with the simulation result. It should be noted that this also serves as another check for the PSD pre-testing simulation program developed in Chapter 3.

Check 2. Elastic Check

The aforementioned calculated response of the simple beam is based on the elastic theory, i.e., the stiffness of the beam remains constant. To check the assumption that the beam remains elastic during the test, the maximum flexural stress of the beam needs to be checked. The top fiber stress on the cross section of the steel beam is
The maximum flexural stress is less than the yielding stress of 0.25KN/mm$^2$ and therefore it is acceptable. Alternatively, the maximum allowable restoring force can be checked as follows:

$$P_{\text{max}} = \frac{4\sigma_y S}{L} = \frac{(4)(0.25)(43.2)(25.4)^3}{2286} = 309.7\text{KN}$$

The yielding load is larger than the maximum applied force 185.92KN. Hence, the beam will remain within the elastic range.

**Check 3. Stability Check**

The lateral torsional stability of the beam is checked per Manual of Steel Construction LRFD 2003 (AISC, 2003). Consider the beam W8x48 with the length $L_b=2286\text{mm}$ and the yield stress $\sigma_y=0.25\text{KN/mm}^2$.

$$\frac{b_f}{2t_f} = 5.92 \leq 0.38\sqrt{\frac{E}{\sigma_y}} = 0.38\sqrt{205/0.25} = 10.9 \quad \text{O. K.}$$

$$\frac{h}{t_w} = 15.9 \leq 3.76\sqrt{\frac{E}{\sigma_y}} = 3.76\sqrt{205/0.25} = 107.7 \quad \text{O. K.}$$

where $b_f$ and $t_f$ are the width and thickness of the flange, respectively; $h$ is the height of web, and $t_w$ the thickness of the web.

Therefore, the compactness of the cross section is acceptable.

For a simply supported beam, the modification factor for moment gradient along the beam is $C_b = 1.32$. The critical length for torsional buckling can be calculated as

$$L_p = 1.76r_y\sqrt{\frac{E}{\sigma_y}} = (1.76)(52.8)\sqrt{205/0.25} = 2661\text{mm}$$
where $r_y$ is the radius of gyration with respect to the axis of buckling.

The critical length for torsional buckling is larger than the length 2286mm of the steel beam used in tests. Thus, lateral torsional buckling will not be an issue for the selected beam specimen.

Figure 4.A.1 Comparison of analytical solution and pre-testing simulation
Chapter 5

Full-Scale Pseudodynamic Tests of Special Concentrically Braced Frames with Buckling-Enhanced Braces

5.1 Introduction

Conventional concentrically braced frames have been extensively used to resist lateral seismic and wind loads because they are simple to construct and have relatively high stiffness. They are very effective in limiting inter-story drifts of buildings under earthquake loading. One problem associated with concentrically braced frames is the dramatic stiffness loss of compression braces when they reach their buckling capacities. Once the compression braces buckle, most of the lateral force applied to the building will be picked up by the tension braces only. A very large vertical force component resulted from the tension braces will emerge at the location of brace connections. This will pose a challenge for engineers in the design of conventional concentrically braced frames. The sudden and dramatic loss of elastic stiffness in critical structural members may lead to brittle damage in buildings. Such damage should be avoided in building designs in earthquake prone areas.

Another possible challenge associated with conventional braced frames is how to upgrade or retrofit braces in existing or damaged buildings (FEMA 273, 1997; and FEMA 351, 1997). For example, 1) the braces in an existing building may not be able to meet the more stringent seismic design requirements any more with the development of new building codes; 2) some braces are found to be inadequate because of mistakes made during design or construction stages; and 3) some braces need to be retrofitted due to
damage occurred in an earthquake event as reported by Tremblay et al. (1995, and 1996). To handle these situations, the common method is to replace braces with stronger braces. Also, if a stronger brace is needed, the connections will have to be upgraded as well. Obviously, this is quite an expensive and time-consuming method/procedure. Even more, it may be impossible to replace the braces in some cases where the temporary removal of braces may lead to collapse. Therefore, innovative and economical approaches to retrofit or upgrade braces are necessary in order to enhance the seismic performances of concentrically braced buildings/frames.

With the introduction of more complex and stringent guidelines for the design and construction of special concentrically braced steel frames following the Northridge earthquake, a rapid increase in the use of special concentrically braced frames has occurred (Mahin and Uriz, 2004). The buckling-restrained braced frame (BRBF) has been recently proposed in order to satisfy the seismic design requirements. As shown in Figure 5.1, a typical buckling-restrained brace consists of a core steel plate or cruciform plate capable of dissipating significant nonlinear deformation energy and an outer concrete encasement to restrain the global and local buckling of the core steel plate. The basic idea to develop buckling-restrained braces is to achieve the objective that the core steel plate works as the tension resistance member when the brace is under tension, while the lateral deformation of the core steel plate can be restrained by the outer concrete encasement when the brace is under compression. The nonlinear deformation of the core steel plate plays an important role in dissipating energy during the earthquake. The buckling-restrained brace seems promising in meeting seismic building design requirements and resisting strong lateral loads during earthquakes as reported by many
This is possible because of the enhanced buckling capacity and higher nonlinear energy dissipation capability of buckling-restrained braces. However, one concern related to the buckling-restrained brace is the required fabrication procedure. Fabrication of buckling-restrained braces is more time and labor-consuming than conventional braces. The repair or retrofit of the buckling-restrained braces seems challenging due to its complex composition of the core metal plate and outer concrete encasement. In addition, the buckling-restrained braced frames are still under development and many aspects related to their designs are still uncertain. These concerns are one of the reasons why the building code (AISC 341, 2005) requires that full-scale tests should be performed before this special type of braces is used.

To further enhance the effort to overcome the problems and challenges associated with the conventional braced frames and/or the buckling-restrained braced frames, the present study aims at developing a new type of brace with higher buckling capacity. This type of special braces is referred to as Buckling-Enhanced Brace herein. The basic motivations for developing this special type of braces are: 1) to enhance the buckling capacity of braces so that they have larger lateral resisting capacity than conventional braces; 2) to make the fabrication of braces easier than that for the buckling-restrained braces; and 3) to make upgrading and retrofitting of existing or damaged braces more economical and easier.

It can be noted that this typology of design is between the conventional brace and buckling-restrained brace. In other words, this type of special braces has a higher buckling capacity than conventional braces, and its fabrication and retrofitting/upgrading
procedures are much easier than that for the buckling-restrained braces whereas its ability to dissipate energy is less than that for the buckling-restrained braces.

This type of braces developed in this study consists of a core steel angle and an outer pipe/tube strapped to it using industrial packaging straps. The construction details of this special brace will be described in the following sections. For the buckling-restrained braces, when the brace is under tension the core metal plate can slide almost freely from the outer casing and therefore only the core metal plate resists the tension force. When the buckling-restrained braces are under compression, the lateral deformation of the core metal plate is restrained by the outer concrete encasement and therefore it can deform into a higher-mode buckling shape and subsequently the buckling capacity of the braces can be increased. In the design of the current special buckling-enhanced braces, the tension force will be picked up by the core steel angle only when the brace is under tension. When the brace is under compression, deformation of the core steel angle can be restrained by the outer pipe/tube wrapped to the core angle and therefore the buckling capacity of the brace can be increased. It can be noted that the tension stiffness of this type of braces remains equal to that of the core steel angle, but the buckling capacity of the brace is increased compared with the brace consisting of only the steel angle.

The structural characteristics of this new type of braces were evaluated experimentally in the current study in the context of a complete frame by investigating a 3-story FEMA/SAC office building which included the special braces in the first story. Computer modeling can be utilized to study the characteristics of structural systems. As for this special type of newly developed braces, it is very challenging to build a reasonable computer model because of the following reasons: 1) the interaction effect between the
core angle and the outer pipe is very complicated; and 2) the buckling deformation of the core angle under compression is uncertain and thus it is very difficult to build a reasonable computer model for a buckled brace. Therefore, physical tests were performed to evaluate the structural characteristics of this new type of braces, providing direct and reliable results. Typically, shake table tests can be performed to investigate the performance of structural members or buildings under earthquake loading. But the scalability of the experimental specimens may be limited by the capacity of the available shake tables. In addition, it is very expensive to construct a large-scale or full-scale model of a complete building.

To tackle these problems associated with computer modeling or shake table tests, the hybrid pseudodynamic testing (PSD) can be used as a viable alternative owing to its unique advantages. In hybrid PSD testing, only the complex structural members are physically tested while the rest of the structure is modeled in a finite element program. Because only the complex elements, not the entire structure or building, are constructed for physical tests, a large-scale and even a full-scale test becomes possible. Hybrid PSD testing has been successfully utilized to evaluate the seismic performances of large-scale and even full-scale structures and buildings (Mahin and Shing, 1985; Shing et al., 1994; Kim and Lee, 1995; etc.).

In this study, the object-oriented hybrid PSD testing platform that was developed previously in the current study was utilized to test the 3-story SAC office building. The details of this object-oriented PSD testing platform were described in Chapter 4 of this dissertation and thus only some basic ideas of PSD testing are briefly provided in this chapter. The braces expected to experience complicated nonlinear deformations after
buckling were tested physically, while the rest of the frame that remains elastic was modeled in the finite element program ABAQUS. In PSD testing, the experimental specimens that are tested physically are typically called the experimental substructure, and the remainder of the structure that is modeled in a finite element program is called the analytical/numerical substructure. In the current study, the newly developed special braces were tested experimentally and thus they were called the experimental substructure. The remainder of the building that was modeled in ABAQUS is called the analytical substructure. The restoring force of the experimental substructure was measured through the MTS controller and the restoring force of the analytical substructure was calculated using ABAQUS. The restoring forces of the two substructures were integrated into the governing equation of motion of the entire building using the substructuring technique which was presented in Chapter 2.

In the PSD tests of the 3-story braced frame presented in this chapter, the implicit numerical Alpha-method that was detailed in Chapters 2 and 3 was utilized to solve the governing equation of motion of the entire building. The stability and accuracy characteristics of this implicit numerical method embedded in the testing platform were verified experimentally through the single-degree-of-freedom (SDOF) and multi-degree-of-freedom (MDOF) systems, as discussed in Chapters 3 and 4. For the current complicated MDOF case of PSD tests, the object-oriented C++ testing platform was further verified experimentally. Note that in the current test, two separate experimental specimens (two braces) were physically tested, while only one individual experimental specimen had been physically tested in the previous PSD tests of the MDOF system presented in Chapter 4. The current PSD test of a realistic complex system can provide a
more comprehensive evaluation of the PSD testing platform developed in the current study.

The structural characteristics of the full-scale buckling-enhanced braces under tensile and compressive forces were evaluated experimentally in the PSD tests of a 3-story braced frame. Tests were performed for cases where the building was subjected to two levels of ground excitations. To investigate the response of this special type of braces under tension and compression, the deformations of the core steel angle and the outer pipe were monitored during the test. In this chapter, the buckling capacity of the conventional brace without the outer pipe is compared with that of the special buckling-enhanced braces. By comparing a case where the outer pipes were hollow with a case where pipes were filled with concrete, the stiffness effects of the outer pipes on the buckling capacity of the braces were studied experimentally.

5.2 PSD Tests of Braced Frames with Conventional Braces

5.2.1 Equivalent Static Force Design Procedure of Braced Frame

A 3-story office building was assumed to be located in Charleston, SC. The site was classified as site class D (stiff soil) and its mapped spectral response acceleration at short periods ($S_s$) and a second period ($S_f$) were 0.6g and 0.2g, respectively. The design spectrum for the site was generated as shown in Figure 5.2. The dimensions of the conventional concentrically braced frame were designed with a layout similar to the 3-story FEMA/SAC model building shown in Figure 5.3. The response modification coefficient, $R$, system over-strength factor, $\Omega_o$, and deflection amplification factor, $C_d$, for
this special braced frame were determined to be 6, 2 and 5, respectively, in accordance with building codes (ASCE 7, 2005; and IBC, 2000).

Following the equivalent static load design procedure (ASCE 7, 2005; and IBC, 2000), equivalent static forces were applied to the building, and then following the Load and Resistance Factor Design Specification for Structural Steel Building (AISC LRFD, 2003) the member sizes of the frame were designed. The beam-to-column connections on the first and second stories are assumed to be moment connections in the current design because the gusset plates at these locations are welded to the flanges of the columns, and therefore the rotations at these locations are considered to be rigidly restrained. As shown in Figure 5.3, the computer model of the 3-story SAC braced frame was built in the standard finite element program SAP2000 (SAP, 2000), and the member sizes of the frame were designed as follows: the columns were W12×96 except for the interior middle column which was W14×48; all beams were W24×104; and braces from the first to the third story were 2L5 ×5 ×7/8, 2L5 ×5 ×1/2, and 2L5 ×5 ×1/2, respectively.

5.2.2 Test Set-up

Compression braces will buckle under the earthquake loading. Modeling of buckling behavior of braces and particularly the innovative designs which are the focus of this chapter is difficult. The hybrid PSD testing platform developed in the current study offers a viable technique for the testing of the 3-story braced frame. In this PSD test, as shown in Figures 5.4.a-c, the braced frame was divided into experimental and analytical substructures. The experimental substructure consisted of the braces in the first story of the frame as shown in Figure 5.4.c, and the remainder of the frame was the analytical
substructure as shown in Figure 5.4.b. It should be noted that as the building is symmetrical; hence, only two braces out of the four braces in the first story will have to be tested physically. Clearly, one brace will be under tension and the other one will be under compression. For convenience of description, one brace is designated as brace $A$ and the other is labeled brace $B$, as shown in Figure 5.4.a.

Tests were performed at the University of Cincinnati Large-Scale Test Facility (UCLSTF). The PSD controlling program and the MTS controller were installed in a computer, called the testing computer. The PSD controlling program computed the target displacement for each degree of freedom of the entire frame. The finite element program ABAQUS was installed in another computer, called the simulation computer. The two computers were connected through a local network. The data exchange between the two computers was described in Section 4.4.1.

The braces used in the tests were A36 2L 2 ½ × 2 ½ × 5/16, instead of 2L5 ×5 ×7/8 to simulate design or construction errors, or a case in which the existing braces have to be upgraded because of code changes and/or building use changes. The stitch spacing of the braces was 940mm, and the stitch plates were 63.5 × 63.5mm per AISC LRFD-03. One 44.45 × 31.75mm A36 standard plate was welded to the angles at each end of braces in order to prevent block shear of the bolted connections, and to ensure the braces would undergo gross section yielding instead of the net section fracture, as shown in Figure 5.5. Hydraulic actuators were used to quasi-statically apply the target displacements to the two braces (the experimental specimens) through the MTS controller. The restoring forces of braces were read by load cells and sent back to the PSD controlling program. The hydraulic actuators had a maximum stroke of 203mm, a maximum load capacity of
670KN in compression, and 538KN in tension. The finite element program ABAQUS installed in the simulation computer was used to calculate the restoring force of the rest of the frame (the analytical substructure). The restoring forces of the experimental specimens and the analytical substructure were integrated using the substructuring technique described in Chapter 3. Note that there were two braced bays in the building, but only the braces in one bay in the first story of the building were tested physically because of the symmetry of the building. The measured restoring forces of the compression and tension braces were used as the restoring forces for both of the braced bays.

The 1.5m thick, 9.5m tall reaction wall in conjunction with the strong floor was used as the reaction “frame” for the full-scale experimental substructures (i.e., the two bays). As shown in Figures 5.6.a-b, two short A992 W14X159 steel beams were post-tensioned to the reaction wall in order to provide supports at the tops of the braces. As shown in Figures 5.7.a-c, two A992 W14X159 “floor beams” were post-tensioned to the strong floor. The actuators were bolted onto these two beams, and the braces were pinned to the actuators.

Wire potentiometers were used to measure the displacements of the braces. One wire potentiometer was secured to the base of the hydraulic actuator and interfaced with the bottom of the brace. A second was secured to the top connection and interfaced with the upper part of the brace, just below the connection. The command signal was the difference between the two potentiometers. The use of two displacement transducers will negate the effects of any slippage in the top or bottom connections on the displacement feedbacks. Strain gages were bonded to both angles at the quarter points and midheight,
as shown in Figures 5.18.a and b. In case of buckling-enhanced braces, two additional gages were bonded to the pipes at their midheight.

### 5.2.3 Tests Results of Braced Frame with Conventional Braces

A response spectrum compatible with the artificial ground motion was used in the tests. The design spectrum (Figure 5.2) was used as the basis for generating the artificial ground motion. The acceleration record of the resulting artificial ground motion is shown in Figure 5.8. Figure 5.9 shows that the acceleration spectrum reasonably matches the target design spectrum.

As stated previously, only two braces were tested physically because the building was symmetrical. Figure 5.10 shows the measured restoring force versus displacement responses of the two braces. As shown in Figure 5.10, the tension stiffness of the two braces was nearly the same equal to $40\text{KN/mm}$, and the buckling capacities of the two braces were almost the same as $75\text{KN}$. The stiffness of the braces became almost zero once they reached their buckling capacities. Figures 5.11.a-b show the measured displacement histories of the two braces. Both braces experienced large deformations after reaching their buckling capacities, which occurred within a short period of time.

### 5.3 PSD Tests of Braced Frames with Buckling-Enhanced Braces

#### 5.3.1 Reduced Design Spectrum and Excitation

Because of the capacity limitations of the actuators used in lab, the earthquake excitation should be smaller than the previous test. In this study, as shown in Figure 5.12, a reduced design spectrum was obtained by multiplying the design spectrum shown in
Figure 5.2 by the scalar quantity $I/R (I/R=1/6)$, where $I$ is the importance factor and $R$ is the response modification factor determined in accordance with the code of ASCE 7 (ASCE 7, 2005). Note that per ASCE 7 elastic design of buildings is performed using the excitation which is obtained by multiplying the design spectrum by the scalar quantity $I/R (I/R=1/6)$. Using the reduced design spectrum, an artificial earthquake record was generated. The response spectrum and acceleration record of the artificial excitation are shown in Figure 5.12 and 5.13, respectively. Once again, the spectrum for the artificial record reasonably matches the target design spectrum shown in Figure 5.14.

5.3.2 Layout of Braced Frame with Buckling-Enhanced Braces

To further reduce the displacement amplitude of the braces in the first story of the frame, the lateral stiffness of the frame was increased by adding braces in the interior bays of the first story (see Figure 5.15). In addition, all the braces in the first story of the building were replaced with the buckling-enhanced braces in order to increase the buckling capacity of the braces. It was expected that the use of special buckling-enhanced braces can increase the buckling capacity.

5.3.3 Details of Test Specimens (Buckling-Enhanced Braces)

Figure 5.16.a shows typical special buckling-enhanced braces before installation in the test setup. Figure 5.16.b shows the cross section of this special type of brace. As shown in Figures 5.16.a-b, the current special buckling-enhanced braces consist of a pair of core angles and two pipes that were wrapped together at ten locations along the length.
It should be noted that, as shown in Figure 5.16.c, only the angles were bolted to the connection at each end. The outer wrapped pipes are expected to slide along the angle when the brace is subjected to tension forces; hence, only the angles can resist tensile force. Under compressive loads, the lateral deformation of the angle is expected to be restrained by the outer wrapped pipes, leading to an increase in the buckling capacity.

The proposed buckling-enhanced brace is different from the buckling-restrained braces in that it is much easier to fabricate because it does not require encasing concrete as buckling-restrained braces do. In addition, buckling-enhanced braces are suitable for retrofit projects due to ease of attachment of outer pipes.

### 5.3.4 Implementation of PSD Tests

It is difficult to build a reasonable computer model for buckling-enhanced braces because 1) there are both interaction contact force effects and slight friction force transferring between the angle and pipe, both of which vary as a function of applied external forces; and 2) post buckling modeling of buckling-enhanced braces is not a trivial task, particularly by recognizing that test data do not exist to characterize the response of this type of innovative braces. Therefore, PSD tests were performed to evaluate the 3-story braced frame outfitted with special buckling-enhanced braces.

The braced frame shown in Figure 5.15 was divided into experimental and analytical substructures. The experimental substructure consisted of the buckling-enhanced braces in the first story of the 3-story SAC braced frame. The experimental substructure is marked in Figure 5.15 with dashed lines. The remainder of the 3-story frame was the analytical substructure which was modeled in the finite element program ABAQUS.
Since the sizes of all the braces in the first story were the same and the braced frame was symmetrical, only two braces (i.e., braces A and B shown in Figure 5.15) were tested physically. One brace is under tension and the other brace is under compression at any time. Hence, the experimental substructure in this test consisted of two individual specimens (i.e., the two braces). The floor diaphragms in the office building were assumed to be rigid. Note that because there were four identical bays, the measured restoring forces were applied to the four bays in the first story. The test setup used previously for the PSD tests of the frame with conventional braces was also utilized for the current test.

5.3.5 Test Results of Braced Frame with Buckling-Enhanced Braces

5.3.5.1 Test Results under Reduced Earthquake Excitation

The reduced earthquake excitation shown in Figure 5.13 was applied to the braced frame. Figures 5.17.a-c show the responses of the frame. As shown in Figures 5.17.a-c, the inter-story drifts in all three stories had almost the same shape although their amplitudes were different, implying that the first mode dominated the response of the frame.

5.3.5.2 Deformation of Braces

The strains measured at the quarter points and midheight of the angles are labeled as shown in Figure 5.18.a for ease of discussion. The theoretical buckling shape of a pinned-fixed brace is also shown in Figure 5.18.a using a dashed line. The strain gages were attached on the surface of the double angle, as shown in Figure 5.18.b. Figure 5.18.c
shows the observed buckled shape of the brace, which is consistent with the theoretical one shown in Figure 5.18.a.

Figure 5.19 shows the strains measured at locations A, B, and C (refer to Figure 5.18.a). When the brace was in tension, strains (positive strain means tension) at three different locations were almost exactly the same, indicating that the steel angle experienced a uniform deformation along the length. When the brace was under compression, as shown in Figure 5.19, the strains at locations B and C were almost the same. This response is expected because they are placed nearly symmetrically with respect to the buckle shape shown in Figure 5.18.a. It can also be noted from Figure 5.19 that when the brace was under compression, the strains measured at A are smaller than those measured at locations B and C. This trend can be explained with reference to the buckle shape shown in Figure 5.18.a. The compression strain at location A is primarily due to axial compression because the out-of-plane deformation around this location is nearly zero. At locations B and C, however, the out-of-plane deformation due to buckling produced additional compressive strains in addition to those from axial compression. Hence, the strains at these locations are larger than that at A.

Figure 5.20 shows the strain measured in the pipe at its midheight, i.e., at the same location as location B on the angles. The maximum tension strain in the outer pipe/tube was around 30µ strain which was only 10-15% of the corresponding maximum tension strain measured in the steel angle. Hence, the tension force was resisted mostly by the steel angle as designed. The maximum compression strain in the pipe was around 50µ strain which was around 15-17% of the corresponding maximum strain measured in the angle. This implies that: 1) the pipes can restrain the lateral deformation of the angles and
therefore increase the buckling capacity; and 2) a small part of the axial compression force was transferred to the pipe from the angle, which again is accordance with design objectives.

With respect to the other brace (i.e., brace $B$ shown in Figure 5.15), similarly, strain gages were attached to the steel angle at the quarter points and midheight as shown in Figure 5.21. The strain gage attached at the location of $D$ malfunctioned. Figure 5.22 shows the strains of the steel angle in brace $B$. As shown in Figure 5.22, the strains at locations $E$ and $F$ are almost the same since they were symmetrical with reference to buckled shape. The same phenomenon was also observed in brace $A$ as shown in Figure 5.19. This is simply because the structural behavior of braces $A$ and $B$ was the same although their deformations were opposite at each time step. A strain gage was also attached to one of the pipes at the midheight, but this gage did not unfortunately work.

Figure 5.23 shows the restoring force versus the axial displacement measured in both braces. The buckling capacity of braces can reach 100KN, whereas the conventional brace without using the outer wrapped pipe has the buckling capacity of 75KN as shown in Figure 5.10. This implies that a one third (33%) increase of the buckling capacity can be achieved by using the current special buckling-enhanced brace.

5.3.5.3 Test Results for Building under 200% Reduced Design Excitation

The second PSD test was performed to evaluate the response of the 3-story SAC office building under a stronger earthquake. In this test, the input ground motion was obtained by doubling the reduced excitation shown in Figure 5.13. Figures 5.24.a-c show the interstory drifts. The responses of the three floors were almost the same in shape
whereas their amplitudes were different. This phenomenon was also observed in the previous case where the reduced design excitation was applied to the building as shown in Figures 5.17.a-c.

Figure 5.25 shows the strains measured in the steel angle of brace \( A \) at locations designated as \( A, B, \) and \( C \) (see Figure 5.18.a). As shown in Figure 5.25, the strains measured at locations \( B \) and \( C \) were almost the same when the brace was under compression. This response is expected because they were nearly symmetrical with respect to the buckle shape, as shown in Figure 5.18.a. Figure 5.25 shows the strain measured at location \( A \) is almost the same as those measured at locations \( B \) and \( C \) when the brace was under tension. This phenomenon was also previously observed in the case where 100\% reduced excitation was applied to the building (see Figure 5.19).

Figure 5.26 shows the strain measured at the midpoint of the pipe in brace \( A \). In comparison to Figure 5.20, which shows the strain measured at the same location of the pipe for the case where the building was subjected to 100\% reduced design excitation, it is noted that the strains in both cases are almost the same although the strain in the case with double (200\%) design excitation is a little more uniform than the former case. This phenomenon may be explained as follows: the axial deformation of the pipe is essentially caused by the friction force transferred from the angles, and such a friction force cannot increase any more when the deformation of the pipe reaches a maximum value. This maximum value was apparently about the deformation measured in the case of 100\% reduced earthquake excitation.

With respect to the other brace (brace \( B \) shown in Figure 5.15), Figure 5.27 shows the strains measured at locations \( D, E \) and \( F \) (see Figure 5.21). The strains at locations \( E \) and
F were almost the same because they were nearly symmetrical with respect to the buckle shape. The strain measured at location D was the same as the other two strains (strains at locations E and F) when the brace was under tension; however, it is much smaller than the other two strains when the brace was under compression. This behavior was also observed, and was previously explained for the case with 100% reduced excitation. The strain gauge attached to the pipe of brace B did not unfortunately work.

Figure 5.28 shows the restoring force versus displacement of the braces. The buckling capacity of both braces was nearly 100KN, corresponding to an increase of 25KN in comparison to the conventional brace (without the wrapped pipes) which had a buckling capacity of 75KN, as shown in Figure 5.10. In other words, the special buckling-enhanced brace result in a 33% increase in the buckling capacity.

5.3.5.4 Test Results of Frame with Braces Enhanced with Composite Pipes under Reduced Design Excitation

To increase the stiffness of the outer pipes in the brace, they were filled with concrete as shown in Figure 5.29. Superplastizier was added to the concrete mix in order to facilitate placement of the concrete inside the pipes. The test results of concrete cylinders are provided in Table 5.1. It should be noted that the cylinders No.3 did not have any meaningful strength apparently because of excessive use of superplastizier. The average strengths from specimens No.1 and No.2 are nearly equal to the design strength of 35MPa. The strength of specimen No.3 is representative of the concrete poured into the pipes in brace B, and the strength of specimens No.1 and 2 represents the concrete poured into the pipes in brace A.
Hybrid PSD testing was identical to those discussed in the preceding sections. Figures 5.30.a-c show the test results for the responses of the building under the reduced design excitation. The responses of the three floors were the same as far as their shapes are concerned, which may be explained because of predominance of the first mode.

The responses of the building for the case with composite pipes and hollow pipes are compared in Figures 5.31.a-c. The differences between the two cases are rather insignificant because the tension stiffness of the braces, which played a dominant role, was the same for the composite and hollow pipes.

Figure 5.32.a shows the strains in the angle in brace A measured at locations B and C (see Figure 5.18.a). The strain gauge attached at location A did not work. The strains at locations B and C were almost the same. As explained previously, this behavior is attributed to symmetry of the locations B and C with reference to the buckle shape (see Figure 5.18.a).

Figure 5.32.b shows the strain measured in the composite pipe of brace A. Compared with Figure 5.32.a, it can be noted that the maximum tensile strain in the pipe was around 15-20% of that measured in the angle. Hence, the tension force in the brace was mostly resisted by the angle, as intended in the design. The same phenomenon was observed in the case with hollow pipes.

The strains of the angle in brace B are shown in Figures 5.33.a. It can be observed that, similar to the case where the outer pipes were hollow, the tensile strains at the instrumented locations (D, E and F, see Figure 5.21) were nearly the same, while the compressive strains at locations E and F were larger than that measured at location D. Figure 5.33.b shows the strain of the outer composite pipe. The tensile strain of the pipe
was around 15-20% of that of the angle, while the compressive strain of the pipe was nearly 20-30% of that of the angle.

Figure 5.34 shows the restoring force versus displacement of the braces in the first story of the building (braces $A$ and $B$). The buckling capacity of the braces with composite pipes is slightly larger than that of braces with hollow pipes, indicating that the concrete inside the pipe did not significantly increase the buckling capacity of braces. The small increase may be because the elastic modulus of the concrete is much smaller than steel, and therefore the concrete inside the pipe cannot appreciably contribute to the lateral stiffness of the braces. The lack of good quality concrete placed in the pipes attached to brace $B$ further reduced the participation of the concrete infill. Additional discussion regarding the expected benefits of filling the pipes with concrete are provided in Section 5.4.

### 5.3.5.5 Tests Results of Frame under 200% Reduced Design Excitation

A test was performed for the case when the building was subjected to 200% the reduced design excitation. The excitation was obtained by doubling the reduced design excitation shown in Figure 5.13. Figures 5.35.a-c show the displacement histories of each of the stories. The responses of the three stories exhibit same shapes apparently because of predominance of the first mode.

The test results for the case with hollow and composite pipes used to strengthen the braces are compared in Figures 5.36.a-c. The responses for the two cases are almost the same. A similar phenomenon was also observed in the case when the building was subjected to the 100% reduced design excitation. As explained previously, the similarity
can be explained with reference to the dominance of the tensile stiffness of the braces, which was nearly identical in both cases. In other words, the compression braces in the first story of the building played a minor role in the overall response of the selected building, and a slight increase in the buckling capacity of the compression braces did not significantly impact the lateral displacement of the building.

Figure 5.37 shows the strains in the angle of brace $A$ measured at locations $A$, $B$, and $C$ (see Figure 5.18.a for locations of the strain gages) are the same when the brace was under tension. The strain measured at location $A$ was smaller than the strains measured at locations $B$ and $C$ when the brace was under compression. As explained previously, this is because the locations $B$ and $C$ were almost symmetrical with reference to the buckle shape. A strain gauge attached at midpoint of the composite pipe in brace $A$ malfunctioned.

Figure 5.38.a shows the strains in the angle of brace $B$ at locations $D$, $E$, and $F$ (see Figure 5.21). All the strains were the same when the brace was under tension. The strain measured at location $D$ was smaller than the strains at locations $E$ and $F$ when the brace was under compression. The same phenomenon was observed in the other tests, and is attributed to out-of-plane deformation of the brace at locations $E$ and $F$.

Figure 5.39 shows the measured restoring force versus displacement for the braces with hollow and composite pipes for the 200% reduced design excitation. The buckling capacity of the braces with composite pipes was very slightly larger than that for the braces with hollow pipes. As discussed previously, the concrete infill did not significantly increase the stiffness of the pipes. This phenomenon will be further explained in Section 5.4.
5.4 Simplified Model for Buckling-Enhanced Braces

As explained previously, it is very difficult to accurately model the special buckling-enhanced braces developed in this study. However, a simplified model is proposed herein to further understand the behavior of this special type of braces.

The free bodies of the steel angle and pipe, which are the main components in a buckling-enhanced brace, are shown in Figure 5.40. When the brace is under compression, a surface friction force will be produced because of the relative movement between the two members. The surface friction forces can be transferred to an equivalent axial force and a bending moment applied at the two ends of the pipe and angle, as shown in Figure 5.41.

In the test, one end of the angle was essentially clamped and the other end was pinned, as illustrated previously in Figure 5.18.a. The displacement and rotation at the two ends of the pipes were the same as those of the steel angle; hence, the boundary conditions of the pipes may be also viewed as clamped and pinned connections. Therefore, the free bodies of the steel and pipe can be regarded as clamped-pinned beam-columns, as shown in Figure 5.42. The buckling capacity of the buckling-enhanced brace can be calculated based on the structural model depicted in Figure 5.42. The buckling capacity of a clamped-pinned beam-column can be calculated as:

\[ P_{cr} = \frac{2.05 \pi^2 EI}{L^2} \]  

(5.1)

where \( E \) is the elastic Young’s modulus, and \( I \) and \( L \) are the inertia moment and length of the beam/column, respectively.

The lengths of the pipe and angle are 7132mm. The moments of inertia of an individual hollow pipe and steel angle are 2.75E5mm\(^4\) and 3.63E5mm\(^4\), respectively. The
elastic Young’s modulus of steel is 205KN/mm² (205GPa). Note that there are two steel pipes and angles in each brace. Using Equation (5.1), the buckling capacity of the angles and pipes are computed from Equations (5.2) and (5.3), respectively.

\[
P_{cr1} = 2 \times \left(2.05 \times \pi^2 \times 205 \times 3.63 \times 10^5\right)/(7132)^2 = 59.2\text{KN} \tag{5.2}
\]

\[
P_{cr2} = 2 \times \left(2.05 \times \pi^2 \times 205 \times 2.75 \times 10^5\right)/(7132)^2 = 44.8\text{KN} \tag{5.3}
\]

Therefore, the total buckling capacity of the brace is:

\[
P_{c} = P_{cr1} + P_{cr2} = 104\text{KN} \tag{5.4}
\]

The calculated buckling capacity based on the simplified structural model shown in Figure 5.42 is almost exactly the same as the measured value, which is around 95-100KN (see Figures 5.23 and 5.28).

The structural model for the special buckling-enhanced brace developed above is further used to investigate the buckling capacity of the braces with composite pipes. If the strength of the concrete inside pipes is taken as 35MPa, the transferred moment of inertia of the concrete infill is, \(I_c = 5.24\text{E}4\text{mm}^4\). The moment of inertia of the composite pipe is \(I_{con} = I_s + I_c = (2.75 + 0.524) \times 10^5 = 3.27\text{E}5\text{mm}^4\), where \(I_c\) is the moment of inertia of the pipe. Note that there are two pipes in each brace. Using Equation (5.1), the buckling capacity contributed by the composite pipes is:

\[
P_{cr3} = 2 \times \left(2.05 \times \pi^2 \times 205 \times 3.27 \times 10^5\right)/(7132)^2 = 53.3\text{KN} \tag{5.5}
\]

Hence, the buckling capacity of a special buckling-enhanced brace with composite pipes is:

\[
P_{c}^\prime = P_{cr3} + P_{cr1} = 53.3 + 59.2 = 112.5\text{KN} \tag{5.6}
\]

Note that 59.2KN in this equation is the buckling capacity of the double angles. Comparing equations (5.4) and (5.6), it can be observed that at most 8% increase in the
buckling capacity may be expected. However, it is believed that the concrete inside the pipes was not fully consolidated in some regions of the pipes, and the concrete capacity inside the pipes of brace B was very low (Table 5.1). The small amount of possible increase (8% under ideal conditions) implies the fact that the measured buckling capacity of the braces with hollow pipes is nearly the same as that for composite pipes should be expected. The use of larger pipes (with larger moments of inertia) is expected to have a more significant influence on increasing the buckling capacity regardless of whether the pipes are hollow or filled.

5.5 Conclusions and Remarks

This chapter presents the full-scale PSD tests of a 3-story special concentrically braced frame with buckling-enhanced braces. The object-oriented PSD testing platform developed previously in the current study was utilized to test the performances of special buckling-enhanced braces in the context of a complete frame. Two separate experimental specimens were tested physically, whereas only one experimental specimen was tested in the PSD tests of the MDOF system demonstrated in Chapter 4. By performing PSD tests of a realistic complex structure (which was a 3-story SAC frame), the PSD testing platform was further verified experimentally. These tests demonstrated that the current testing platform can reliably be used for hybrid PSD testing of complex structures.

The special buckling-enhanced brace developed in the current study consists of a core double angle and outer pipes that are strapped to the angles with industrial steel straps placed at discrete locations along the brace length. The strapped pipes are intended to increase the buckling capacity. Special buckling-enhanced braces with hollow pipes and
concrete filled pipes (i.e., composite pipes) were also tested. The test results revealed that the buckling capacity of braces can be increased by using outer pipes to restrain the lateral deformation of the core steel angles. The additional buckling capacity will help reduce the building drifts. It is expected that the developed special braces can be utilized for retrofitting existing or damaged braced building. For example, the braces in buildings may need to be upgraded to account for design and/or construction errors, or the existing braces are found to be inadequate to meet the requirements of new building codes. A one third (33%) increase in the buckling capacity can be achieved according to the test results obtained in this study. A simple analytical model was developed to adequately compute the capacity of buckling-enhanced braces. The experimental data and analytical results do not suggest a significant increase by using composite pipes. Using the PSD testing capability developed in this study, additional tests need to be conducted for further design of buckling-enhanced braces.
Table 5.1 Test results of concrete cylinders
(30 days strength)

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Strength (MPa)</th>
<th>Average Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>36.27</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>33.58</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>35.65</td>
<td>35.2</td>
</tr>
<tr>
<td>2</td>
<td>34.47</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>33.00</td>
<td>33.5</td>
</tr>
<tr>
<td>2</td>
<td>32.90</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1.03</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1.38</td>
<td>1.2</td>
</tr>
<tr>
<td>3</td>
<td>1.10</td>
<td></td>
</tr>
</tbody>
</table>
Figure 5.1 Typical buckling-restrained brace (Black et al., 2004)

Figure 5.2 Design spectrum
Figure 5.3 Computer model of 3-Story SAC braced frame

Figure 5.4.a Structural model of 3-story SAC building
Figure 5.4.b Analytical substructure

Figure 5.4.c Experimental substructure
Figure 5.5 Connection detail at ends of double angle brace
Figure 5.6.a Top connection detail of brace
Figure 5.6.b Top connection of brace

Figure 5.7.a Bottom connection detail of brace
Figure 5.7.b Floor beams and actuators

Figure 5.7.c Bottom pin connection of brace
Figure 5.8 Earthquake excitation

Figure 5.9 Comparison of design spectrum and generated spectrum
Figure 5.10 Restoring forces versus displacements of braces.

Figure 5.11.a Deformation of tension brace during test.
Figure 5.11.b Deformation of compression brace during test

Figure 5.12 Reduced design spectrum
Figure 5.13 Reduced earthquake excitation

Figure 5.14 Comparison of reduced design spectrum and generated spectrum
Figure 5.15 Structural model of braced frame with buckling-enhanced braces

Figure 5.16.a Typical buckling-enhanced brace

Figure 5.16.b Schematic cross section of buckling-enhanced braces
Figure 5.16.c Connection of angle at end of braces

Figure 5.17.a Horizontal displacement of the first story
Figure 5.17.b Inter-story drift between the first and second stories

Figure 5.17.c Inter-story drift between the second and third stories
Figure 5.18.a Locations of strain gages along length of brace A

Figure 5.18.b Locations of strain gages on cross section of brace A
Figure 5.18.c Buckling shape of brace during test

Figure 5.19 Strains measured in brace A
Figure 5.20 Strain measured in pipe of brace $A$

Figure 5.21 Locations of strain gages along length of brace $B$
Figure 5.22 Strains measured in brace B

Figure 5.23 Restoring force versus displacement of braces under reduced excitation
Figure 5.24.a Displacement of the first story

Figure 5.24.b Inter-story drift between the first and second stories
Figure 5.24.c Inter-story drift between the second and third stories

Figure 5.25 Strains measured in brace $A$ in 200% reduced design excitation
Figure 5.26 Strain measured in pipe of brace A in 200% reduced design excitation

Figure 5.27 Strains measured in brace B for case of 200% reduced design excitation
Figure 5.28 Restoring force versus displacement of braces for case of 200% reduced design excitation

Figure 5.29 Schematic cross section of braces with composite pipes
Figure 5.30.a Displacement of the first story of building with braces enhanced with composite pipes

Figure 5.30.b Inter-story drift between the second and first stories of building with braces enhanced with composite pipes
Figure 5.30.c Inter-story drift between the third and second stories of building with braces enhanced with composite pipes

Figure 5.31.a Comparison of displacement of the first floor of buildings with different braces
Figure 5.31.b Comparison of inter-story drifts between the first and second stories of buildings with different braces

Figure 5.31.c Comparison of inter-story drifts between the second and third stories of buildings with different braces
Figure 5.32.a Strains of the angles in brace A enhanced with composite pipes for reduced design excitation case

Figure 5.32.b Strain of the pipe in brace A enhanced with composite pipes for reduced design excitation case
Figure 5.33.a Strains of the angles in brace $B$ enhanced with composite pipes for reduced design excitation case

Figure 5.33.b Strain of the pipes in brace $B$ enhanced with composite pipes for reduced design excitation case
Figure 5.34 Restoring force versus displacement for braces enhanced with hollow and composite pipes for reduced design excitation case

Figure 5.35.a Displacement of the first story of building with braces enhanced with composite pipes
Figure 5.35.b Inter-story drift between the second and first stories of building with braces enhanced with composite pipes.

Figure 5.35.c Inter-story drift between the third and second stories of building with braces enhanced with composite pipes.
Figure 5.36.a Comparison of displacement of the first story of building with braces enhanced with composite pipes

Figure 5.36.b Comparison of inter-story drift between the second and first stories of building with braces enhanced with composite pipes
Figure 5.36.c Comparison of inter-story drift between the third and second stories of building with braces enhanced with composite pipes

Figure 5.37 Strains measured in the angles of brace $A$ in 200% design excitation case
Figure 5.38.a Strains measured in angles of brace B in 200% reduced design excitation case

Figure 5.38.b Strain of composite pipe in brace B in 200% reduced design excitation case
Figure 5.39 Restoring force versus displacement for braces with hollow and composite pipes in 200% reduced design excitation case
Figure 5.40 Forces of two free bodies

Figure 5.41 Two free Bodies with equivalent forces

Figure 5.42 Structural model for buckling-enhanced braces
Chapter 6
Conclusions and Recommendations

6.1 Summary of Research

Hybrid Pseudodynamic (PSD) testing is an experimental-numerical integrated testing approach that can be used to avoid some of the problems associated with shake table tests or pure model-based simulation methods. In PSD testing, only the complicated components or subassemblies that cannot be modeled satisfactorily are tested physically, and the remainder of structures is modeled. The aim of this study is to develop hybrid PSD testing capability at the University of Cincinnati Large-Scale Test Facility. This aim is achieved through the development of an integrated testing platform in an object-oriented C++ framework architecture. It is expected that this testing platform can be used to evaluate the performances of structural subassemblies, buildings, bridges, and various infrastructure systems under earthquake, dynamic, or static loads.

In PSD testing, different numerical algorithms are used to solve the governing equation of motion at each time step in order to get the target displacement for each degree of freedom of the entire structure. Chapter 2 reviews different numerical algorithms that have been employed or proposed by other researchers. Explicit numerical algorithms that have been employed in PSD testing include the finite difference method, Newmark's explicit method (Newmark, 1959), and operator-splitting method (Nakashima and Kaminosono, 1990). Implicit algorithms that have been utilized by researchers include the alpha-method (Hilber et al., 1977; and Thewalt and Mahin, 1987), and the improved alpha-method (Shing et al, 1991). The explicit and implicit algorithms are
reviewed and evaluated in Chapter 2 in terms of their numerical stability and accuracy and error-propagation characteristics. Generally, unconditionally stable implicit algorithms are superior to conditionally stable explicit algorithms in terms of numerical stability and accuracy, but the computation efficiency of implicit algorithms may be impaired by the required iteration procedure. Owing to their characteristic of unconditional stability, implicit numerical algorithms can be employed for testing of most linear and nonlinear structures while the integration time step can be chosen arbitrarily and their numerical stability and accuracy can be retained. Explicit numerical algorithms are usually easier to perform in tests than implicit algorithms. However, because of their characteristic of conditional stability, explicit numerical algorithms may encounter challenges if they are employed to test stiff MDOF systems where the selected integration time step needs to meet rather tough requirements in order to ensure computational stability and accuracy since the first natural period of stiff MDOF systems is quite small.

In order to avoid problems associated with the explicit algorithms and the alpha-method employed by Thewalt and Mahin (1987), where an analogue electronic device in the displacement control loop was used to correct the predictor displacements based on the continuous feedback of the instantaneous restoring force developed by the experimental specimens, Shing et al. (1991) developed an implementation procedure of the improved alpha-method. Instead of using a feedback-based control loop used by Thewalt and Mahin (1987), Shing et al. (1991) used a numerical iteration procedure where the initial elastic stiffness of structures was utilized because the instantaneous stiffness of structures is difficult to acquire during tests. This approach can eliminate the
need for measuring the instantaneous stiffness of structures during the test. Shing et al. (1991) verified the improved alpha-method experimentally by performing tests of a cantilever column (a SDOF system), and Shing et al. (1994) further experimentally demonstrated the tests of a braced steel frame employing the improved alpha-method. The characteristics of unconditional stability, sufficient accuracy, ease of performing in test, and having been verified experimentally make the improved alpha-method a reliable numerical algorithm. The improved alpha-method is implemented in the testing platform developed in the current study.

Chapter 3 presents a pre-testing simulation computer program developed in this study. The pre-testing simulation program consists of three components/modules that are seamlessly integrated using substructuring techniques. These three components/modules are the PSD controlling module, the virtual experimental substructure module, and the numerical/analytical substructure module. In this pre-testing simulation program, the restoring force of the virtual experimental substructure is calculated in ABAQUS instead of reading the values from the physical test. The other two components, i.e., the PSD controlling module and the numerical/analytical module, are the same as those in actual hybrid PSD tests involving physical experimental substructures. The pre-testing simulations were performed for three typical structures: 1) a cantilever column (SDOF); 2) a one-story frame with a hinge on the left top corner where there is only one degree of freedom at the interface between the two substructures; and 3) a one-story frame without hinges where there are two degrees of freedom at the interface between the two substructures. For each of these three typical structures, pre-testing simulations were performed for linear and nonlinear cases. The pre-testing simulation results for these six
representative cases were compared against the analytical solutions obtained through the finite element program ABAQUS, showing good agreements. The numerical algorithm employed in the current testing platform (i.e., the improved alpha-method) is validated numerically by performing pre-testing simulations of these six representative cases. This pre-testing simulation program can be used to predict the response of structures in part to determine the resolution of the hardware required for a particular test.

The object-oriented PSD testing platform was experimentally demonstrated through a series of physical tests as presented in Chapter 4. Because this testing platform is developed in an object-oriented framework architecture, it can be modified and upgraded easily with the development of new PSD testing algorithms. Moreover, because C++ is a low level programming language, this testing platform can easily interface/communicate with most popular measuring and controlling hardware and software. Note that a quite large number of software for hardware control in engineering fields is developed using C++ computer language. The complete testing platform was experimentally verified by performing a series of physical tests of the SDOF and MDOF systems.

Chapter 4 presents the PSD tests of SDOF and MDOF systems. In the tests of the SDOF system, the experimental substructure was also the entire system and therefore the analytical substructure was not included. A simple steel beam was used to represent a SDOF system. A constant force was assumed to be applied to the simple beam. This force mitigated the error accumulation and propagation during the test which may be induced because of a nonzero initial displacement, and helped the beam to stay in contact with its supports. In implicit numerical algorithms, the instantaneous stiffness of structures is replaced with the initial elastic stiffness in the iteration procedure. The
selection of an appropriate convergence tolerance is important because of the finite resolution of the hardware crucial to PSD testing, i.e., load cells and displacement transducers. To investigate the effects of the convergence tolerance and the initial elastic stiffness, PSD tests were performed for the SDOF system (the simple beam) by selecting different parameters. The test results did not suggest significant changes as a result of varying these parameters possibly because of the simplicity of the system that remained elastic.

In the PSD tests of MDOF systems, the complete structure is divided into experimental and analytical substructures. The restoring force of the experimental substructure is acquired from the physical tests while that of the analytical substructure is calculated using the finite element program ABAQUS. The experimental substructure in this MDOF case was a simple beam. Essentially, it was the same as that used in the tests of the SDOF case. The analytical substructure was a frame consisting of a vertical column and a horizontal beam. The experimental and analytical substructures were connected through a pin at their interface. Therefore, there was only one degree of freedom at the interface between the two substructures, and accordingly only one hydraulic actuator was needed for the tests. Two computers were used in the test. The first one, called the testing engine, was used to control the physical testing of the experimental specimen and to acquire the restoring force of the experimental substructure. The second one, called the simulation engine, was used to install the finite element program ABAQUS and operate ABAQUS to calculate the restoring force of the analytical substructure. A solution for data exchange between the two computers was developed and was verified experimentally as part of the tests of the MDOF system. A
series of PSD tests were performed to investigate the effects of the convergence tolerance, initial elastic stiffness, and reduction factor selected. The test results show that the effects due to these parameters were related to the input excitation energy. The larger was the input energy, the smaller were such effects. To further validate the current testing platform, a set of tests was performed to test the MDOF system where the analytical substructure was nonlinear. The test results show that the current testing platform can also provide direct and reliable evaluations for nonlinear structures.

A start-restart procedure is embedded in the testing platform developed to handle some unexpected situations during tests. To mimic a real termination of the test, the MTS system (including the pumps and the manifold system) and the testing engine were shut down. The test was then restarted from the exact location where it was terminated before. A test performed to demonstrate this special procedure indicates that the test result obtained through the start-restart procedure was consistent with that obtained through a complete test without any stop.

Chapter 5 presents the application of hybrid PSD testing in the evaluation of a special concentrically braced frame under earthquake loads. This task was not only useful to further verify the testing platform developed in the current study, but it also provided an opportunity to evaluate the performances of a realistic building with a special type of buckling-enhanced braces. A braced frame (which was based on a 3-story FEMA/SAC office building) was selected for PSD testing. The braces in the first story of the building were tested physically while the rest of the building was modeled in ABAQUS. The special buckling-enhanced brace consists of a core steel double angle and outer pipes strapped to the angles with industrial steel straps. The new type of braces are designed to
achieve the objective that only the core steel angles resist the tension force. The outer pipes are strapped to the inner steel angles in an attempt to restrain the lateral deformation of the angle when the brace is under compression and hence to increase the buckling capacity. The performance of this special type of braces was evaluated by using the hybrid PSD testing platform developed in the current study. The buckling capacity of the new type of braces was approximately 33% larger than the capacity of standard double angle braces. In accordance to the design objectives, the measured strain history of the braces indicates that the tension force was mostly resisted by the core steel angle.

6.2 Conclusions

An integrated object-oriented PSD testing platform was developed in this study. This platform can be widely used for evaluating response of various structures under earthquake, dynamic, or static loads. The platform was verified experimentally through PSD testing of SDOF and MDOF systems. Based on these tests, the following conclusions are drawn:

1) Hybrid PSD testing is a general testing approach that can be used to evaluate the behavior of structural subassemblies, buildings, bridges, and various infrastructures under dynamic, earthquake, or static loads. This test method can provide direct and reliable test results for linear and nonlinear structures, and tackle the problems associated with shake table tests and stand-alone computer modeling methods.

2) The improved alpha-method is a suitable numerical algorithm that can be employed in hybrid PSD testing. Because of its unconditional stable characteristic, the integration time step can be chosen arbitrarily while the computation stability and sufficient
accuracy can be retained during the test. The numerical stability and accuracy characteristics of the improved alpha-method are validated in the current study by performing pre-testing simulations of six representative linear and nonlinear cases. Pre-testing simulation results are consistent with the analytical solutions obtained through ABAQUS.

3) The substructuring technique can be used to seamlessly interface experimental and analytical substructures. The implementation of the substructuring technique in PSD testing makes it possible to test large-scale or even full-scale subassemblies of an entire structure, instead of testing of reduced scales of the complete structure. The substructuring technique was successfully embedded in the current PSD testing platform. Therefore, the current testing platform can be utilized to conduct hybrid PSD testing.

4) Experimental errors associated with PSD testing can be mitigated by using advanced hardware. Advanced measuring and controlling devices have small resolution. Therefore, if advanced hardware is available in the laboratory, a small tolerance can be chosen in PSD tests and thus the experimental errors induced by the measuring and controlling errors can be curbed.

5) The start-restart procedure embedded in the current testing platform can be used to handle unexpected situations that might arise during tests. This procedure was demonstrated by terminating a test and then restarting the test from the exact location where it was terminated before. The test results obtained from the test using the start-restart procedure are consistent with those obtained from the test without any interruptions. Hence, the start-restart procedure can be used to resume a test if it is
terminated unexpectedly. It can also used to handle cases in which the system is subjected to one more than one input excitation.

6) The hybrid PSD testing platform developed in the current study can effectively be utilized to evaluate realistic structures. The platform can easily handle multiple experimental substructures.

7) The tension force applied to the special buckling-enhanced braces is mainly resisted by the core inner double angles. The tensile stiffness of this type of braces is nearly equal to that for standard double angle braces.

8) The special buckling-enhanced brace developed in this study is effective in increasing the buckling capacity of braces, and therefore it can be used to retrofit or upgrade existing or damaged buildings. The outer pipes in the buckling-enhanced braces can restrain the lateral deformation of the inner steel angles and thus increase the buckling capacity of braces. The inter-story drifts of the building can be reduced using the buckling-enhanced braces.

6.3 Recommendations

To further the current study, the following recommendations are made for future studies:

1) Other numerical algorithms should be added to the developed testing platform such that a variety of structures (i.e., stiff or flexible) can be tested.

2) Advanced hardware should be added to reduce the experimental errors, and hence to enhance the robustness of the developed PSD platform.
3) Wireless networks for PSD testing can be developed. In PSD testing, the communication between different measuring and controlling equipment is necessary. If a wireless network can be developed, it becomes more flexible to set up these measuring and control systems during the test.

4) More physical tests of buckling-enhanced braces should be conducted. Improved design and detailing techniques for buckling-enhanced braces need to be developed to further increase their buckling capacity. The application of this special type of braces in other buildings such as eccentrically braced frames should be pursued through the use of hybrid PSD testing platform developed in the current study.
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