MULTI-SITE NETWORK HYBRID TESTING OF A MULTI-SPAN BRIDGE SYSTEM

By

Ya-Ying Chang

B.A.Sc., University of British Columbia, 1998

M.A.Sc., University of British Columbia, 2000

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**MULTI-SITE NETWORK HYBRID TESTING OF
A MULTI-SPAN BRIDGE SYSTEM**

Submitted by

Ya-Ying Chang

In partial fulfillment of the requirements for the degree of
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Chair, Department of Civil and Environmental Engineering

Thesis Supervisor

Carleton University

Ottawa, Ontario, Canada

January 2008

.Dedicated to my father

Abstract

Experimental research is often carried out using a scaled-model test specimen due to the high cost of full-scale testing, the limitations of test equipments, and physical restrictions imposed by the size of laboratories. In order to allow testing of full-scale structures, which might not be possible otherwise to be conducted at any one individual laboratory, many research efforts have been focused on the development of hybrid experimental techniques in recent years. Because of the complexity and wide range of differences in the operation requirements of different testing equipment and facilities, there are the needs of a standardized platform and protocols for efficient set-up of collaborative network based multi-site hybrid experiments, and standardized protocol procedures for efficient data exchange. The aim of the current study is to establish a robust standardized procedure for laboratories at multiple sites to network their testing facilities in order to perform collaborative experiments. A collaborative research project is established between Canada and Taiwan to develop modules for control and instrumentation based on the use of standardized software interfaces. Pseudo-dynamic tests on a four-pier bridge system were performed using the developed protocols. The sub-components of the prototype bridge system were tested at three remote sites over long distance. Two of the remote sites were in Taiwan and the third remote site was in Canada. Three double-skin concrete filled tubular (DSCFT) columns, which is an innovative design for bridge piers at high seismicity area, were constructed and tested at the participating laboratories. The fourth pier and the bridge decks were simulated by a numerical model. The Internet-based Simulation for Earthquake Engineering (ISEE) platform was employed for data

repository and communication among laboratories. The design of hybrid test set-up and procedures, and the test specimens are described. Three analytical models were constructed for predicting the nonlinear behaviour of the DSCFT columns, and two of the analytical models were implemented in the ISEE environment to calculate the nonlinear responses of the bridge during the experiment. Selected 2D plots of analytical and experimental results, 3D visualization of the deformed prototype bridge, and the videos of the test specimens are broadcasted through the World-Wide Web in real-time, which allows outside researchers to be able to participate in this experiment through tele-presence. This is the first cross-continent collaborative networked hybrid test on large scale specimens, which has successfully demonstrated the potentials and future direction of experimental research of large size structures and civil infrastructure systems.

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Nomenclature

$[a]$	acceleration matrix
A	Cross-sectional area
$[C]$	damping matrix
$[d]$	displacement matrix
D	Target displacement commands
f'_c	Specified compressive strength of concrete
f_y	Specified yield strength of steel
F	Restoring forces measured in the local coordinate system
$[F]$	Internal force matrix
$[K]$	Stiffness matrix
$[M]$	Mass matrix
N	Number of time steps
$[P]$	External force matrix
P	Applied constant axial load
P_n	Axial strength
R	Restoring forces in the global coordinate system
$[\tilde{r}_{n+1}]$	Measured restoring force matrix
Δt	Increment of time
T	Natural period

$[v]$	Velocity matrix
U	Elongation/ shortening of actuators

Acronyms and Abbreviations

AVS	Application Visualization System
BRB	Buckling Restrained Braced
CFT	Concrete filled tubular
CGM	Command Generation Module
CU	Carleton University
DAQ	Data acquisition
DSCFT	double-skinned concrete filled tube
EWS	Engineering Work Station
FCM	Facility Control Module
ISEE	Internet-based Simulation for Earthquake Engineering
KAIST	Korea Advanced Institute of Science & Technology
KOCED	Korea Construction Engineering Development Program
KU	Kyoto University
LVDT	Linear variable differential transformer
NCREE	National Center for Research on Earthquake Engineering
NEES	Network for Earthquake Engineering Simulation
NTU	National Taiwan University

OCU	Osaka City University
OpenSees	Open System for Earthquake Engineering Simulation
OS	Operator-split
PEER	Pacific Earthquake Engineering Research
PISA3D	Platform for Inelastic Structural Analysis
PNSE	Platform for Networked Structural Experiment
RCS	Reinforced concrete structure
SA	Simplified Approach
SF	Scale factor
SQL	Structured Query Language
UHS	Uniform Hazard Spectrum
WWW	World Wide Web

CHAPTER 1

1 INTRODUCTION

1.1 Introduction

Considering the continuing increase in complexity of modern structural systems and the many innovations in structural design, and despite the advances in computer modeling and analysis methods of complex structures in the past four decades, experimental research is still crucial and needed to verify new ideas and concepts, whether new design or new modeling and simulation techniques. Common experimental methods to obtain information of complex structural behaviour and to verify performance of complex Civil Engineering structures under seismic loading can be categorized into quasi-static loading test, pseudo-dynamic test, or full dynamic test using shake table.

In a quasi-static loading test, the test specimen is subjected to slowly changing prescribed forces or deformations imposed by hydraulic actuators at such a slow loading rate that the inertial effect within the structure can be ignored. This technique is usually used for observing the behaviour of structural components, connections, or the failure modes of structural members subjected to cycles of loading and unloading. However, the dynamic behaviour of the tested structural component under earthquake loading may not be accurately captured by quasi-static loading tests due to neglect of the inertia effect in the test. To account for the missing inertia effect in structures under dynamic loading conditions, the experimental techniques of pseudo-dynamic test was developed in which

quasi-static forces that also include the simulated inertia forces of the test structure are applied to the structure. Employing substructuring techniques in pseudo-dynamic test, only part of the test structure where the dynamic response or hysteretic behaviour is complex needs to be physically tested, while the remaining part of the structural system can be simulated by computer models, and the total dynamic responses of the entire system can be obtained by combining the results from the experimental and analytical substructures. In pseudo-dynamic tests, the strain rate effects are neglected because the loading is applied quasi-statically. Furthermore, the simulated inertial force effects in pseudo-dynamic tests are not exact because of approximation in representing the inertia effect by discrete parameter mode in the simulation, actuator response behaviour, and approximation of computer modules. However, the simulated dynamic response behaviour and deformations observed in test specimens of pseudo-dynamic tests can provide valuable insight on the dynamic behaviour of the structural system during an actual earthquake. In full dynamic shaking table test, structures are subjected to actual earthquake acceleration records to obtain the most accurate reproduction of the dynamic behaviour of the tested structures. The inertial effects and structure assembly issues are well represented. However, in shake table tests, the size of the test structure is limited by the size and capacity of the shake table facility.

Experimental research is often carried out using a scaled-model test specimen due to the high cost of full-scale testing, the limitations of test equipments, and physical restrictions imposed by the space and facilities of laboratories, which may limit the validity of applying the research results to real structures in the field. In order to be able to conduct

experiments of large size specimens to give better insights and understanding of the realistic behaviour of structures under extreme loading conditions, which might not be possible otherwise to be conducted at any one individual laboratory, the concept of network hybrid testing at multiple sites is developed. In a multi-site test, the prototype test structure is divided into substructures or sub-components, each of which can be tested at a different test site. The testing of all the sub-components is carried out concurrently in real-time to form a single experiment of the prototype system. This substructure testing technique is also known as “Hybrid Testing,” which consists of physical and experimental substructures, and virtual computational substructures. The physical substructures are usually the nonlinear subcomponents that are difficult to model accurately, and the virtual computational substructures are the remaining structural components which can be accurately represented by numerically models. This technique is particularly suitable for testing and analysing specific regions of interest within a structure or system. The concept of Internet-based multiple site virtual laboratory testing facilitates more efficient utilization of test facilities at different laboratories by giving researchers around the world the flexibility and means to conduct large scale experimental research through sharing of equipment and experimental data on a real-time basis. Similar concepts of Internet-based virtual laboratory testing at multiple sites are currently being developed and implemented by researchers in many countries, such as the research programs in Japan (Watanabe *et al.*, 1996; Nagata *et al.*, 1996; Sugiura *et al.*, 1998; Ohtani *et al.*, 2002), the Internet-based Simulation for Earthquake Engineering (ISEE) (Yang *et al.*, 2003; Wang *et al.*, 2003) in Taiwan, the Network for Earthquake Engineering Simulation (NEES) Project (Rietherman, 2004) in US, and the Korea

Construction Engineering Development (KOCED) Program (Kim, 2004) in Korea, which all aim to revolutionize earthquake engineering research by providing researchers at different laboratories anywhere around the world the means to collaborate and to conduct networked collaborative hybrid experiments, which include components of physical testing, computer modeling, and simulation of substructure components at multiple sites.

1.2 Objectives and Scopes of the Study

Because of the complexity and wide range of differences in the operation requirements of different testing equipment and facilities, there are the needs of a standardized platform and protocols for efficient set-up of collaborative network based multi-site hybrid experiments, and standardized protocol procedures for efficient data exchange. This is an application of information technology in structural engineering research.

A collaborative experiment is established between Taiwan and Canada to develop a robust standardized procedure for laboratories geographically distributed around the world to pool and network together their testing facilities to perform collaborative experiments using the ISEE platform developed at National Center for Research on Earthquake Engineering (NCREE). A four-pier bridge system is chosen as the prototype structure for the hybrid collaborative test. The sub-components of the prototype bridge system are tested at three remote sites over long distance; two in Taiwan and the third in Canada. The loading and responses of the fourth pier are simulated by computer models. The test specimens are loaded pseudo-dynamically using input ground motions from the 1999 Chi-Chi earthquake and the simulated Uniform Hazard Spectrum (UHS) compatible

ground motion in Vancouver, Canada, scaled to the representative seismic hazard levels of 50%, 10%, and 2% probabilities of exceedance in 50 years. This is the first cross-continent collaborative networked hybrid test of large scale specimens. The aims of this collaborative experiment and research are as follows:

- (i) To develop procedure for easy set-up of network of participating test equipment and facilities, which includes the development of control modules and visualization modules are the participating remote sites.
- (ii) To evaluate the performance of the Internet-based Simulation for Earthquake Engineering (ISEE) platform for network hybrid testing using the developed standardized procedure. Network pseudo-dynamic tests of a five span bridge system with DSCFT piers is carried out using the ISEE platform.
- (iii) To study the dynamic behaviour of the innovative design of double-skin concrete filled tubular (DSCFT) columns under bi-lateral excitations.
- (iv) To evaluate the methods and techniques developed for numerical analyses and simulations in hybrid testing context. The PISA3D program developed by researchers in Taiwan (PISA3D, 2005) and the Simplified Approach method developed at Carleton University (Dailamani, 2002; Chang *et al.*, 2004; Deng 2006) are investigated.

1.3 Layout of Thesis

This thesis is organized into 7 Chapters and 5 Appendices. Motivations for the study, introductory materials, and objectives of this research are presented in the current chapter. The survey of relevant published literature is presented in Chapter 2. The test setup and

procedures of a network collaborative experiment are presented in Chapter 3. In Chapter 4, the design of the prototype bridge structure and the design of test specimens are presented. The analytical and experimental results are presented in Chapters 5 and 6, respectively. Conclusions and recommendations for further work are presented in Chapter 7. Appendix A contains the brief description of the developed Facility Control Modules. Details of the data communication procedures for a network pseudo-dynamic test are included in Appendix B. Design drawings of the test specimens are shown in Appendix C. The comparisons of analytical force-time histories obtained from the three analytical models are shown in Appendix D, and the comparisons between the analytical and experimental force-time histories are shown in Appendix E.

CHAPTER 2

2 REVIEW OF LITERATURES

It is difficult to perform tests on large scale structural systems due to the limitation of testing facilities and test implementation costs. In recent years, significant research efforts have been devoted to develop the technology for collaborative structural experiments, which allows two or more laboratories at different geographical locations to jointly conduct one single coordinated experiment. The following is an overview of some existing ongoing research reported in the literature on the preceding issues.

2.1 Researches in Japan and Korea

A remote parallel control and computing system through internet for pseudo-dynamic testing was initially proposed and developed in Japan (Watanabe *et al.*, 1996; Nagata *et al.*, 1996; Sugar *et al.*, 1998). The remote pseudo-dynamic testing system consisted of a system server, which carried out response analysis of the testing structural components; several local computers for control of testing equipment; and structural testing systems of test equipment (such as actuators) and data acquisition system, and the test specimens. The system server and local computers were linked by internet. The configuration of remote parallel pseudo-dynamic testing system proposed by Watanabe et al (1996) is shown in Figure 2.1. The testing system consisted of an Engineering Work Station (EWS), two local computers and two structural testing systems. The EWS operated under UNIX system was the system server for carrying out analytical analyses during pseudo-dynamic experiments. The two local computers operated under the Window NT

system retrieved the imposed displacement information on the test specimens from the system server, and then deposited the forces measured from the actuators back to the system server. Since different operating systems were used by the system server and the local control computers, Samba, a free application software, was used for the sharing of data and information between EWS and local computers. An Application Visualization System (AVS) was setup to monitor the progress of the dynamic response analysis and to visualize the progress on numerical information of the experiment. To evaluate the feasibility of this proposed network configuration for pseudo-dynamic testing, a prototype system of a single span elevated bridge consists of steel and RC piers was tested (Sugiura *et al.*, 1998). The input excitation was a ground acceleration record from the 1995 Kobe earthquake.

A similar network configuration has also been proposed and developed by Yun *et al.* (2000). In this study, the prototype structure was a continuous concrete box girder bridge with base-isolators at the tops of four concrete piers. This study was carried out by connecting four computer systems located separately at Korea Advanced Institute of Science & Technology (KAIST), Yosu University, and Changwon University. In the pseudo-dynamic test, the base-isolators were numerically simulated with assumed hysteretic characteristics. All four computers were operated under Windows 98. The structural analysis program and data transmission protocol program were written in Visual C++. The network configuration of the experiment is shown in Figure 2.2. The selected ground motion for the pseudo-dynamic test was the 1940 El Centro N-S Earthquake record with the maximum peak ground acceleration scaled to 0.2g. Three

different test setup scenarios were considered to compare the efficiency of data communication time. The simulation test was performed by (i) four different computers located at three separate laboratories; (ii) four computers all located at KAIST; (iii) one computer located at KAIST. The simulation test results show that it took 35 minutes for 4000 iterations between four computers located 250 km apart connected through internet. Following the initial separate research, a collaboration project was established to investigate the feasibility of online testing between KAIST and Kyoto University (KU) using the network configuration developed at KU (Watanabe *et al.*, 2001). The configuration of the network communication for this experiment is shown in Figure 2.3. The prototype test structure was a three spans bridge with continuous girder. The input excitation was the 1994 Hanshin-Awaji (Kobe) earthquake. In the pseudo-dynamic experiment, the equation of motion was solved by the central difference method at a time increment of 0.01 second for 1000 time steps. The test results show that the average elapsed time for each step is 22 seconds.

To test the efficiencies of the network configurations developed at KU, shown in Figure 2.1, and at KAIST, shown in Figure 2.2, a collaborative research project was established between KU, KAIST, and Osaka City University (OCU) (Park *et al.*, 2005). The prototype structure selected in this study was a base-isolated bridge with four continuous spans. The base-isolators on top of piers 1 and 2 were tested at KAIST and KU, respectively, while the base-isolator on pier 3 was numerically simulated at OCU. In this study, the El Centro earthquake record was selected as the input ground motion with the

maximum peak ground acceleration adjusted to 0.2 g. Four sets of tests were performed to test the efficiencies of the network configurations are listed below:

(1) Base-isolators were physically tested at KAIST and KU while the third base-isolator was numerically simulated at OCU using the network configuration proposed by Watanabe et al. (1996). The average elapsed time per step is 25.1 seconds.

(2) All three base-isolators were numerically simulated at KAIST, KU, and OCU using the network configuration proposed by Watanabe et al. (1996). The average elapsed time per time step is 15.0 seconds.

(3) Base-isolator was physically tested at only KAIST while the other two base-isolators were numerically simulated at KU and OCU using the network configuration proposed by Yun et al. (2000). The average elapsed time per time step is 1.02 seconds.

(4) All three base-isolators were numerically simulated at KAIST, KU, and OCU using the network configuration proposed by Yun et al. (2000). The average elapsed time per time step is 0.25 seconds.

The studies by Japanese and Korean researches are the earliest implementation and application of network pseudo-dynamic experiment techniques. These researches have shown that testing facilities scattered around the world can be interconnected and used interactively by remote control through Internet. However, there are areas which require further research, such as real-time sharing of experimental data on Internet, standardized data management, and the influence of network performance on the accuracy of the experiment.

2.2 Network for Earthquake Engineering (NEES) Project in USA

The Network for Earthquake Engineering Simulation (NEES) program was launched in 1999 by the National Science Foundation (NSF) in USA (NEES, 2004a). The goal of the NEES program is to provide a networked, US national simulation resource that includes geographically-distributed, shared-use experimental research equipment sites built and operated to advance earthquake engineering research and education through collaborative and integrated experimentation, theory, data achieving, and model-based simulation. The NEES program includes the following three core components:

(1) NEES Equipment Sites: A network of 15 experimental facilities forms a large-scale, shared-use facility for conducting earthquake engineering research. These facilities include shaking tables, large-scale laboratories, field experimentation and monitoring facilities, a tsunami wave basin, and geotechnical centrifuges.

(2) NEES IT Infrastructure: An IT infrastructure named NEESgrid is developed to link the NEES equipment sites providing access to computing resources and research equipment (NEES, 2006). Technically, the NEESgrid is a collection of software services, which include the centralized data repository and management, tools for simulation and structural analysis, real-time data viewer, etc., designed for the NEES project. The information technology infrastructure, NEESgrid, is led by the University of Illinois, Urbana-Champaign.

(3) NEES Consortium, Inc. (NEESinc): It is a community-based consortium which represents the interests of the earthquake engineering research, education, and practice communities. It manages the information technology infrastructure of the NEES program (NEES, 2004b).

As part of the NEES program, a multi-site hybrid simulation test was carried out to demonstrate the tele-operation capacities of the NEESgrid for conducting geographically distributed testing (Mosqueda *et al.*, 2005). The prototype structure for this study was a six span bridge with five columns, which were physically modeled in the participating laboratories. The experimental results show that the elapsed time for each step was approximately 0.66 seconds on average. It was also found that there were a few steps with substantial delays for approximately 23 seconds, which may resulting from the loss of data packets during network communication.

The NEES infrastructure is well developed with softwares for organizing and sharing data, numerical simulation and analyses, tele-communication and tele-presence cameras for remote viewing, and platform for streaming sensor data. The hybrid simulation test conducted by Mosqueda *et al.* (2005) has proven the feasibility and efficiency of the NEES infrastructure. However, these developed softwares may not be compatible with the control of test equipment at other institutions.

2.3 Internet-based Simulation for Earthquake Engineering (ISEE) in Taiwan

As the outcome of extensive research efforts on hybrid structure testing, a computer platform, Internet-based Simulation for Earthquake Engineering (ISEE), has been developed by researchers at NCREE (Hsu, 2002; Yang *et al.*, 2003; Wang *et al.*, 2003). This computer platform consists of software and hardware components specially developed for cooperative experimental structural testing and research linked through the

Internet environment. Two approaches, the Database Approach and the Application Protocol Approach, have been employed to provide different setup solutions for network communications as well as collaborative framework in the ISEE environment.

2.3.1 Database Approach

The ISEE framework for the Database Approach consists of three major components: the Data Center, the Analysis Engine, and Facility Controllers (Hsu, 2002; Yang *et al.*, 2002). The Data Center is a database server which also serves as a data exchange hub between the Analysis Engine and the Facility Controllers. The Analysis Engine simulates and computes the interactions expected between the sub-components tested at participating laboratories according to the information received from the remote sites. The Facility Controller is the software bridging the Data Center and the experimental facilities in each participating laboratory. The communications between the Data Center, the Analysis Engine, and the Facility Controller are developed and incorporated using C++ programs. The network configuration of the Database Approach is shown in Figure 2.4.

Several collaborative network pseudo-dynamic tests have been performed to investigate the feasibility and efficiency of the ISEE environment using the Database Approach (Tsai *et al.*, 2003). First, network pseudo-dynamic tests were performed on a Buckling Restrained Braced (BRB) frame. In this study, the test specimen was located at NCREC, while the Analysis Engine was also located at NCREC (for the first test) and at Stanford University, USA (for the second test) (Tsai and Huang, 2002; Tsai *et al.*, 2003). The BRB frame was modeled as a 2D frame using OpenSees, an object-oriented simulation

framework developed by researchers at the Pacific Earthquake Engineering Research (PEER) Center for simulating the seismic responses of structural and geotechnical systems subjected to earthquakes (Mazzoni *et al.*, 2000; OpenSees, 2004). The integration time increment was 0.005 seconds for a total of 2000 steps. The experimental results show that the average elapsed times were 0.11 seconds and 0.95 seconds for numerical simulations of the BRB at NCREE with the Analysis Engine located at NCREE and at Stanford University, respectively.

A series of two-site sub-structural pseudo-dynamic tests were also performed on Double-Skin Concrete Filled Tubular (DSCFT) columns. The prototype structure was a four-pier bridge system. The bridge decks and the two end piers were numerical simulated by linear beam-column elements while the two central piers were physically tested at the National Taiwan University and at NCREE. The input excitations were ground motion acceleration records from the 1999 Chi-Chi earthquakes. With the Analysis Engine placed at NCREE, the average elapsed time per time step for the pseudo-dynamic test was 1.91 seconds. With the Analysis Engine placed at Stanford University, the average elapsed time per time step was 4.32 seconds.

A collaborative research was then established between NCREE and Stanford University to perform a full-scale composite moment frame using the ISEE environment. The test specimen was a three-story three-bay RCS frame of reinforced concrete columns and composite steel frame. This was a one-site experiment and the networking time was not recorded during the experiment.

2.3.2 Application Protocol Approach

An alternative platform for Networked Structural Experiments (PNSE) has been developed that can be used to link numerical programs and facility control programs using point-to-point TCP/IP connection (Wang *et al.*, 2003). The architecture of the PNSE is shown in Figure 2.5. The main components of the PNSE platform include the PNSE server, the Command Generation Module (CGM), and Facility Control Module (FCM). The PNSE server serves as the communication centre between the CGM and FCMs. The CGM is a numerical program that generates commands (i.e., displacements) to be imposed on the test specimens located at geographically distributed laboratories. The FCMs are the control programs for experimental facilities.

The platforms for Application Protocol Approach has also been applied on multi-site experiments (Tsai *et al.*, 2003; Wang *et al.* 2004; Yang *et al.* 2004; Tsai *et al.* 2004; Tsai *et al.* 2005). The test specimens used for this study were DSCFT columns tested at NCREC and NTU. Three network scenarios were considered in this study:

- (1) Test A: the server and the analysis program ran on the same computer at NCREC.
- (2) Test B: the server was located at NCREC and the analysis program at Stanford University.
- (3) Test C: the server and the analysis program were located at Stanford University.

Both the server program and the analysis program are written in C++ language. The testing equipment at NTU and NCREC are MTS 407 and MTS FlexTestIIIm, respectively.

The experimental results show that the average elapse time for the three tests were 0.663 seconds, 0.80 seconds, and 1.06 seconds, respectively.

Several pseudo-dynamic tests have been successfully performed using the ISEE environment. However, there are still some areas which require further research, such as the development of a compatible and standardized control software platform for reliable and robust control commands between the various facility controllers, and the optimization of execution of the platform.

2.4 Other Major Ongoing Research Projects

Other current major research projects on the development of techniques for collaborative research on hybrid test at multi-site are the E-Defense Network (ED-Net) of Japan (Ohtani *et al.*, 2002) and the Korea Construction Engineering Development (KOCED) Program (Kim, 2004) project in Korea.

The construction of a large scale networked 3-D Full-Scale Earthquake Testing Facility, known as E-Defense, was completed in January 2005. The E-Defense is a 20 m by 15 m 6-degrees-of-freedom shaking table. The maximum payload of the “E-Defense” shake table are 1200 tons subject to the motion with a maximum velocity of 200 cm/s and maximum displacement of ± 100 cm in the two horizontal directions, and a maximum velocity of 70 cm/s and maximum displacement of ± 70 cm in the vertical direction. For international collaboration and dissemination of research results, the E-Defense Network

(ED-net) has the facilities and capabilities of tele-observation and tele-operation through high performance Internet (Ohtani *et al.*, 2002).

The objectives and concepts of the KOCED program are similar to the NESS project. Twelve large scale testing facilities are to be constructed at major universities across Korea. These testing facilities will be connected using high performance information network. Digital data repository and high performance computing facility will also be integrated into this grid system (Kim, 2004).

2.5 Conclusions

Pseudo-dynamic test is commonly used in Earthquake Engineering research to evaluate the nonlinear response characteristics of complex structural members. To utilize the research resources, the concept of network pseudo-dynamic test, also known as network hybrid test, is developed to allow testing of sub-structural components of large scale structural systems at multiple sites concurrently. In recent years, significant research efforts have been devoted to develop the technology for collaborative structural experiments, which allows two or more laboratories at different geographical locations to jointly conduct one single coordinated experiment. Many of the related research programs mentioned in this chapter are multi-year research projects on upgrading the existing testing facilities. The development and implementation of network hybrid test systems have been carried out by researchers around the world. Various simulated network hybrid tests had been conducted by researchers at Japan, Korea, US, and Taiwan, which had validated the feasibility of network hybrid testing. However, because of the

complexity and wide range of differences in the operation requirements of different testing equipment and facilities, there are the needs of a standardized platform and protocols for efficient set-up of collaborative network based multi-site hybrid experiments, and standardized protocol procedures for efficient data exchange. Areas that still require further investigation include methods for online monitoring of the test procedures, real-time display and sharing of experimental data, network security, the efficiency of network testing.

Figure 2.1: Network configuration of the Kyoto Experiment (Sugiura *et al.*, 1998)

Figure 2.2: Network Configuration of the KAIST experiment (Yun *et al.*, 2000)

Figure 2.3: Network Configuration of the Kyoto-KAIST experiment (Yun *et al.*, 2000)

Figure 2.4: Network configuration of the Database Approach (Yang *et al.*, 2003)

Figure 2.5: Network configure of the Application Protocol Approach (Wang *et al.*, 2003)

CHAPTER 3

3 DESIGN OF TEST SET-UP AND PROCEDURES

3.1 Introduction

Pseudo-dynamic testing method is an efficient experiment approach for examining the dynamic behaviour or earthquake responses of complex structures with the capability to focus on obtaining experimental detailed information on the hysteretic behaviour of nonlinear structural members or regions of the test structure, while the remaining part of the structural system is numerically modeled and simulated. The concept of network hybrid test allows testing of multiple sub-structural components concurrently at multiple sites. In this research, the computer platform “Internet-based Simulation for Earthquake Engineering (ISEE),” which is developed by researchers at NCREE, is used to perform the network hybrid test. This computer platform consists of software and hardware components specially developed for cooperative experimental structural testing and research linked through the Internet environment. The ISEE framework consists of three major components: the Data Center, the Command Generate Module (CGM), and Facility Control Module (FCM). The Data Center is a database server which also serves as a data exchange hub between the CGM and the FCMs. The Command Generate Module simulates and computes the interactions expected between the sub-components tested at participating laboratories according to the information received from the remote sites. The Facility Control Module is the software bridging the Data Center and the experimental facilities in each participating laboratory.

3.2 ISEE Environment

As mentioned in the previous chapter, two approaches are developed by the researchers at NCREE for the network pseudo-dynamic test: the Database Approach and the Application Protocol Approach. The difference between the two approaches is the communication method as described in Chapter 2.3. Initially, both approaches would be commenced during the collaborative experiment; however, the implementation of the Application Protocol Approach was not complete at the time of the scheduled tests. Therefore, the Database Approach is selected as the network environment for the collaborative experiment in the present study. The Data Center and the Analysis Engine are located in Taiwan, while the three Facility Controllers are located at Carleton University (CU), National Center for Research on Earthquake Engineering (NCREE) in Taiwan, and National Taiwan University (NTU). The components of the Database Approach are described in the following sections.

3.2.1 Data Center

The Data Center serves as a data exchange hub and repository during an experiment in ISEE database approach. All the experiment data and analysis data are transmitted to the Data Center during and after a pseudo-dynamic test. The Data Center employs a database server to facilitate exchange of experimental or analysis data among different ISEE components that may reside in laboratories at different geographical locations. The Data Center also provides a WWW (World Wide Web) interface for researchers to setup required experimental parameters for collaborative network structural tests before the experiment starts. In addition, experimental and analytical data are collected and

processed, and information is distributed during the duration of the experiment. In this work, the Data Center employs the Microsoft SQL (Structured Query Language) Server database and the IIS web server. In addition, for the C++ programs of the Analysis Engine and the Facility Controllers to easily communicate with the Data Center, a C++ class, named SQLAccess, is developed (Hsu 2002) and incorporated into those C++ programs.

3.2.2 Analysis Engine (Command Generation Module)

For a network hybrid test, a complete structural model is implemented in the Analysis Engine. During a real-time network hybrid test, the responses of the prototype structure are obtained by discrete time interval and the responses are calculated in a step-by-step manner. At each time step, the selected discrete integration algorithm computes a displacement step, which is subsequently imposed on the structure by means of computer controlled servo-hydraulic actuators. Once the structure is deformed, the resulting restoring forces are measured. Based on the measured restoring forces, the current damping, and applied forces, the resulting new acceleration is calculated. A new displacement step can then be calculated and the next step has thus commenced.

To evaluate the methods and techniques developed for numerical analyses, two different analysis programs are used during this collaborative experiment: OpenSees (Open System for Earthquake Engineering Simulation) (OpenSees, 2004) and PISA3D (Platform of Inelastic Structural Analysis for 3D Systems) (Tsai and Lin, 2003; PISA3D, 2005).

3.2.3 Facility Controllers (Facility Control Modules)

Accurate remote controlling of the load applying actuators at multiple sites is one of the challenges for conducting a network collaborative pseudo-dynamic experiment. In this study, the three participating laboratories use different models of servo-controllers. The controllers at Carleton, NCREE, and NTU are MTS 458, MTS FlexTestIIIm, and MTS 407, respectively, and hence, a compatible and standardized control software platform is highly desired for reliable and robust control commands between the various facility controllers at different participating sites in a collaborative test. Presently, different commercial software solutions are available for interface control and instrumentations of tests. Two commonly used software packages are the National Instrumentation LabVIEW and the Hewlett-Packard VEE environment. In this research, LabVIEW is selected as the controlling software platform to integrate testing equipment at various participating sites into a unified test platform for the cooperative structural test. In addition, this software platform also provides remote control of the data acquisition instrumentations through a computer network. The key design considerations of the controlling software platform for long distance multiple site collaborative hybrid testing are listed as follows

- 1) Module for equipment control: The primary function of this module is to generate the commands for accurate and robust control of actuators and data acquisition across different manufactures' equipment and standardized software for equipment control operating under different computer operating systems, such as Windows, Linux, UNIX, and Open Source.
- 2) Module for real time display: Because of the requirement of the software platform to be reliable and robust for long distance multiple site hybrid tests, accurate

timing and synchronization of commands and data communication are critically important. Time delay due to heavy network communication and the impact on time sensitive test are critical considerations which must be considered in the design of the module.

- 3) Module for data storage and management: Test data at different sites may be stored locally using different data formats. In order to facilitate the exchange of information during the test and later for test data interpretation and research, data harmonization and the adoption of a standardized data exchange format will enhance the portability the software design.
- 4) Module for network and security issues: The primary function of this module is to network participating laboratories together for the purpose to conduct collaborative hybrid test at multiple sites. To prevent unauthorized interruption of the collaborative multiple site hybrid test, access to the collaborative hybrid test network is authorized to researchers at the participating laboratories. Other issues such as network failure or temporally suspension of the collaborative hybrid test due to unexpected damage of test specimen need to be considered in the design of the module.
- 5) Module for instant messaging: A module built for instant communication between participating researchers at different laboratories when unforeseen circumstances arise during the test.

Each of the module mentioned above are developed as sub-VIs (virtual instrument), which are similar to sub-routines that can be called in the program. A set of control

modules specifically for the test equipment at NTU were developed by Mr. S. J. Wang at NCREE. These control modules are adopted and modified at Carleton University. Figure 3.1 shows the control panel of the Facility Controller Modules. A brief description of the developed modules and the data communication procedures can be found in Appendices A and B, respectively.

The general procedures of the Facility Controller during a network experiment are:

Step 1: Define experimental parameters

(1) Related to network security

- Data Center/ SQL host name
- Experimental ID and Facility Controller ID
- Number of commands to be exchanged during the experiment

(2) Other parameters

- Number of time steps and the size of the time step

Step 2: Define experimental setup

(1) Control setup

- Control mode (internal measurements of the actuators or external measuring devices)
- Number of actuators to be used during the experiment
- Number of external measurements (such as LVDTs)

(2) Other setup parameters

- Length of actuators

- Length of external measurements
- Length of the test specimen

Step 3: Data management

- Define file path/ file directory
- Define the save type (overwrite or append)
- Select the files to be saved

Step 4: Select the experimental options

- Experiment state (a new test or a resume test)
- Whether or not to perform coordinate transformation
- Whether or not to return control commands
- Whether or not to display real-time plots during the progress of the experiment
- Whether or not to show the signal readings during the progress of the experiment
- Whether or not to hold the control at the end of each defined control loop

Step 5: Setup the equipment control

- If analog signals are required for the control of testing equipment, a Analog-to-Digital DAQ card is required to convert the signals
- Define the analog input and output channels
- Define the ranges of the testing equipment
- Define the control method and other parameters

Step 6: Establish network connection with the Data Center and wait for all other participating laboratories to be connected to the Data Center. The experiment proceeds once all of the participating laboratories are connected to the Data Center. In the main time, zero voltage commands are sent to the control system before switching to external equipment control for safety concerns.

Step 7: The target displacement commands, also referred to as critical signals, are received from the Data Center. The Facility Command Modules calculate the incremental actuator commands and these incremental actuator commands are imposed on the test specimen until the target displacement commands are reached.

Step 8: The restoring forces are measured, recorded, and then sent back to the Data Center. Return to Step 7 and wait for the commands for the next time step. The procedures are repeated until the end of the time series defined in Step 1.

3.2.4 Visualization Module

Visualization Modules are implemented in the ISEE environment to present experimental and analytical data are presented in graphical format during the progress of an experiment. At present, both 2D and 3D visualization modules are implemented in the ISEE platform (Tsai, K.C. and Chuang, 2005), shown in Figure 3.2. The 2D module presents numeric experimental data such as the time history of displacements or forces or the force-

displacement hysteresis in 2D plots. The 3D module renders the deformation of the structure using the 3D computer graphics approach. These Visualization Modules instantaneously process the data during the progress of the experiment to present and distribute the experimental results in graphical forms through the Internet, allowing researchers and engineers at remote locations to participate in the test by obtaining information on the structural behavior of the test specimens in near real-time as the experiment progresses.

3.2.5 Camera Module

During a pseudo-dynamic test, varying loads are applied quasi-statically; therefore, it is possible to take high-resolution still images with cameras set at desired time interval for further research and investigation on the structural behavior and failure modes of the bridge and sub-components. A camera module is a remote controlled camera which takes still pictures by remote commands or preset rules. At Carleton University, two cameras are set-up to take still images of the test specimen at every 10 seconds (as shown in Figure 3.3). At NCREC, cameras are set to capture a single image after the facilities complete the commands of each time step.

Other than digital pictures, video images are streamed to web servers during the progress of the hybrid test allowing researchers and engineers to participate in the experiment at anywhere around the world. Video showing the dynamic response of the specimen (or a part of the specimen) can be made by editing a series of still images taken from a fixed viewpoint. Shown in Figure 3.4 is a screen captured image of the Video Module at

Carleton University. Compared to the Video Module, the Camera Module captures higher resolution but only in still images.

3.3 Test Procedures

In the implementation of hybrid structural testing here, a full scale numerical model of the prototype structure is implemented in the Analysis Engine to calculate the dynamic responses of the entire system using implicit-explicit integration scheme. At each time step, the Analysis Engine computes displacements of the structure. The displacement responses are then sent to the Data Center for data storage and plotting. These displacements responses are sent to the respective participating laboratories to be imposed on the test specimen by means of hydraulic actuators through the control of Facility Controller. The force required to impose these displacements are measured and are fed-back to the Data Center and the Analysis Engine. Based on the measured restoring forces, the current damping, and the current applied forces, the resulting new accelerations are calculated. The steps are repeated until the end of time series.

In this study, the Newmark-based Operator-Split (OS) method is employed for time integration of the network pseudo-dynamic experiment. The Newmark's time-integration method can be interpreted as an iteration loop of a series of response prediction steps and correction steps. The concept of the OS method is to separate the damping and stiffness matrices into implicit and explicit parts (Hughes and Liu, 1978; Hughes, 1987). The only difference between the Newmark's method and the OS method is that the latter one considers the non-linear effects only on the unbalanced force in Equation 3.1. In a

network pseudo-dynamic experiment, the explicit part of the algorithm denotes the nonlinear part of the damping and stiffness matrices of the specimens, and the implicit part is the rest of the damping and stiffness matrices. The numerical integrations proposed by Hughes contain implicit Newmark algorithms and explicit predictor-corrector algorithms. To ensure unconditional stability of the integration scheme, the Newmark's average acceleration time integration method ($\beta = 0.25$; $\gamma = 0.5$) is used (Hughes, 1987). The accuracy of the numerical integration of equations of motion depends on the selection of time step. A general rule of thumb for the selection of time step is that Δt (time step) should be less than T/π , where T is the fundamental mode period of the structural system (Humar, 1990).

The general procedures of numerical integrations during a network hybrid test are:

Step 1. Based on the equation of motion and the initial conditions ($[d_0]$ and $[v_0]$) of the structure, the response of the structure is determined at the discrete time interval, t_n . The expected displacements, $[d_{n+1}]$, and velocities, $[v_{n+1}]$, induced by the input forces can be calculated by the following equations (Hughes, 1987):

$$[M][\ddot{a}_{n+1}] + [C][\dot{v}_{n+1}] + [K][d_{n+1}] = [F_{n+1}] \quad (3.1)$$

$$[d_{n+1}] = [d_n] + \Delta t[\dot{v}_n] + \frac{\Delta t^2}{2}\{(1 - 2\beta)[\ddot{a}_n]\} \quad (3.2)$$

$$[\dot{v}_{n+1}] = [\dot{v}_n] + \Delta t\{(1 - \gamma)[\ddot{a}_n]\} \quad (3.3)$$

$$[\ddot{a}_0] = [M]^{-1}([F_0] - [C][\dot{v}_0] - [K][d_0]) \quad (3.4)$$

where $[M]$, $[C]$, $[K]$, $[F_{n+1}]$ are the mass, viscous damping, stiffness, and excitation force matrices, respectively. The subscript n stands for the count of the discrete time instance; for $n = 0, 1, 2, \dots, N$, where N is the total number of time steps. $[d_{n+1}]$, $[v_{n+1}]$, and $[a_{n+1}]$ are the incremental displacement, velocity, and acceleration of the structural components, respectively.

Step 2. The displacements obtained in Step 1 are sent as displacement commands to the Data Center and the Facility Controllers at the participating laboratories. Then the target displacements $[d_{n+1}]$ are imposed on the test specimens by means of actuators through the Facility Command Modules. Once the target displacements are reached, the corresponding resisting forces $[\tilde{r}_{n+1}]$ are measured, recorded, and then are sent back to the Analysis Engine through the Data Center.

Step 3. Calculate the incremental vector of accelerations $[a_{n+1}]$ based on the measured resisting forces and the input forces by the following equation:

$$[a_{n+1}] = \{[M] + \gamma\Delta t[C^I] + \beta\Delta t^2[K^I]\}^{-1}([P_{n+1}] - [\tilde{f}_{n+1}]) \quad (3.5)$$

where $[C^I]$ and $[K^I]$ denotes the implicit part of damping and stiffness matrices, which are assigned as the initial damping and stiffness matrices, respectively. $[P_{n+1}]$ and $[\tilde{f}_{n+1}]$ are the external force and internal force vectors, respectively. The $[\tilde{f}_{n+1}]$ vector is assembled from inertia forces, damping forces, and stiffness resisting forces of each element, where

the stiffness resisting forces of test specimen elements are replaced by the measured restoring force vector $[\tilde{r}_{n+1}]$.

Step 4. Calculate the displacement, velocity vectors from the acceleration vector obtained in Step 3. Return to Step 2 for the next time step.

When loads are applied in two orthogonal directions to a test specimen, the true displacements of the test specimen are different from the internal displacements of the actuators, as illustrated in Figure 3.5. Therefore, force and displacement transformations are required during the experiment for true forces and displacements. Assuming that the applied lateral forces go through the mass centre, of the specimen and the axial load is constantly applied perpendicular to the top surface of the column, the transformations of displacements and forces can be expressed as the following equations:

$$U_x = \sqrt{((L_x + D_x)^2) + D_y^2} - L_x \quad (3.6)$$

$$U_y = \sqrt{((L_y + D_y)^2) + D_x^2} - L_y \quad (3.7)$$

$$R_x = F_x \times \frac{(L_x + D_x)}{\sqrt{((L_x + D_x)^2) + D_y^2}} + F_y \times \frac{D_x}{\sqrt{((L_y + D_y)^2) + D_x^2}} - P \times \frac{D_x}{\sqrt{D_x^2 + D_y^2 + L^2}} \quad (3.8)$$

$$R_y = F_y \times \frac{(L_y + D_y)}{\sqrt{((L_y + D_y)^2) + D_x^2}} + F_x \times \frac{D_y}{\sqrt{((L_x + D_x)^2) + D_y^2}} - P \times \frac{D_y}{\sqrt{D_x^2 + D_y^2 + L^2}} \quad (3.9)$$

where L_x and L_y are the lengths of actuators in the X and Y directions, respectively. The length of an actuator is measured as the distance between the two swivels. D_x and D_y are the target displacements command calculated by the Analysis Engine. U_x and U_y are the elongation / shortening of the actuators. P is the applied constant axial load. F_x and F_y are the resisting forces measured from the actuators. R_x and R_y are the resisting force components in the global coordinate system.

3.4 Summary

The Database Approach of the ISEE platform is employed in this network collaborative experiment between Carleton University, NCREE, and NTU. The Database Center and the Analysis Engine reside at NCREE, while the Facility Controllers are located at CU, NCREE, and NTU Laboratories. Within a network pseudo-dynamic experiment, a numerical model of the test structure is implemented in the Analysis Engine. A discrete integration scheme is used to compute the displacement responses at the interface with the physical substructures subjected to earthquake loadings. These displacement responses are imposed on the physical substructure by actuators through the control of Facility Controllers in the laboratories. The corresponding forces required to impose these displacements are then fed-back into the model for predicting the structural responses for the next time step. The accuracy and stability of the integration scheme depends on the fundamental frequency of the structure. A general rule for selecting the appropriate time step is to set Δt (time step) less than T/π .

A world wide web page containing the experimental information, real-time display of experimental and analytical data, and video feed of the experiment is setup for the network hybrid test.

Figure 3.1: Facility control modules

Figure 3.2: Visualization module in the ISEE environment

Figure 3.3: Camera module settings at Carleton University

Figure 3.4: Video module at Carleton University

Figure 3.5: Plan view of test setup

CHAPTER 4

4 DESIGN OF THE BRIDGE SYSTEM AND SPECIMENS

The seismicity of eastern Taiwan is similar to the seismicity of southwestern British Columbia, and therefore a four-pier bridge system is designed for a high seismic location either in Taiwan or in Vancouver, Canada for this research. The seismic design provisions of the Taiwan Highway Bridge Design Code (Ministry of Transportation and Communication 1995) and the Canadian Highway Bridge Design Code (CHBDC, 2000) are followed for the design of the prototype bridge structure.

4.1 Prototype Bridge System

The prototype bridge structure is illustrated in Figure 4.1. It is assumed to be designed for a highly active seismic location either in Taiwan or in western Canada. The prototype bridge is a five-span bridge and the deck width is 9.75 m, which is commonly found in 2-lane bridge configurations in Taiwan. Each span is 40 m long. The bridge piers are 10 m, 15 m, 15 m, and 12 m for piers 1, 2, 3, and 4, respectively. Piers of 10 m, 12 m, and 15 m in height are typically found in highway bridges, and therefore, are used for the asymmetrical prototype bridge system. The bridge piers are assumed to be fully fixed at the base and pinned at connections to the superstructure in both transverse and longitudinal directions. The bridge system considered in this study is assumed to have a cap beam on top of the column. Each cap beam is assumed to be rigid and is connected to the steel girder using a simple connection providing free support rotation. The bridge deck is assumed to have 230 mm thick reinforced concrete slab built composite with steel

girder spaced at 2.6m. The slab is assumed to have a 1 m overhang on both sides of the deck. The pavement is assumed to be 80 mm thick. The dimensions of the superstructure for this prototype bridge are shown in Figure 4.2. The weight of superstructure including deck and steel girder is estimated to be 19.5 tonf/m. The bridge piers are double-skinned concrete filled tubular (DSCFT) columns, which consist of two concentric circular thin tubes with concrete filled in between. The weight of DSCFT columns P1, P2, P3, P4 are 13.2, 19.2, 19.2 and 19.2 tonf/m, respectively.

The prototype bridge is designed according to the Taiwan bridge seismic provision (DOT, 2000) and the National Building Code of Canada (NBCC 2005). The bridge structure is assumed to be located either in Chiayi City or in Vancouver with Soil Type I (hard rock site), and the occupancy importance factor I is 1.0. Design load combination is 1.0DL+1.0EQ. Full composite action between the slab and the steel girder is assumed in the calculation of the stiffness of the superstructure. The theoretical flexural stiffness adopted here for the DSCFT columns follows the definition given for concrete filled tubular (CFT) columns (Hsu and Lin, 1997; Zhong, 1994; Tsai *et al.*, 2002), which is expressed in Equation 4.1.

$$EI = (0.6625 + 0.9375 \alpha)(E_s I_s + \beta E_c I_c) \quad (4.1)$$

where $\alpha = A_s/A_c$. The elastic modulus of the core concrete is estimated from $E_c = 4730\sqrt{f'_c}$ (MPa), where f'_c is the average compressive concrete strength at 28 days. The elastic modulus for steel tube E_s is taken as 200 GPa. I_s and I_c are the moment of inertia of steel and concrete, respectively. According to the experimental results from Lin

(2003), β is roughly equal to 0.12. The axial strength of the DSCFT columns is calculated based on the AISC-LRFD design code (1994) for CFT columns. Since there is no reinforcing bar within the DSCFT column, the equation for calculating the axial strength, P_n , is modified and can be obtained as follows

$$P_n = A_s F_{cr} \quad (4.2)$$

$$F_{cr} = \left(0.658^{\lambda_c^2}\right) F_{my} \quad \text{for } \lambda_c \leq 1.5 \quad (4.2a)$$

$$F_{my} = f_y + 0.85 f'_c \left(\frac{A_c}{A_s} \right) \quad (4.2b)$$

$$\lambda_c^2 = \left(\frac{KL}{\pi r_m} \right)^2 \left(\frac{F_{my}}{E_m} \right) \quad (4.2c)$$

$$E_m = E_s + 0.4 E_c \frac{A_c}{A_s} \quad (4.2d)$$

where KL = effective simply supported column length and r_m = radius of gyration of steel tube.

The diameters and thickness of the DSCFT columns are designed according to the axial load and moment requirements (P-M interaction diagrams). The embedded DSCFT-to-foundation connection (Wei, 2002) is adopted in the present test specimen. The stiffness and strength of the superstructure and piers are listed in Table 4.1. The details of the full-scale test specimens are listed in Table 4.2. The dimensions and estimated resistance of DSCFT-to-foundation connections are shown Table 4.3.

From past studies of damage in bridges, it has been shown that the crushing of concrete or failure of the structure usually occurred at the piers; therefore, it is assumed that the slabs remain elastic under cyclic loading. As a result, the slabs can be simulated by beam elements in the numerical model. Only the structural members that are expected to behave nonlinearly, such as bridge piers, are constructed and tested in the laboratories.

4.2 Scaled Test Specimens

The test specimens are designed to comply with the similitude requirements following the Buckingham's π theorem (Harris and Sabnis, 1999) for a direct reduced-scale model of the prototype bridge shown in Figure 4.1. Different scaling factors are used in the design of the sub-component test specimens at the multiple test sites based on the available testing equipment and physical size of each participating laboratory sites in this international pseudo-dynamic experiment. For this research, the scale factors (SF) for displacement are chosen to be 1/5 for Carleton University and the NTU Laboratories and 2/5 for the NCREE Laboratory. The scaling criteria is to keep the Poisson's ratio unity. A design criterion of column section is to limit the column axial stress less than 10% sectional axial load capacity. The heights of the test specimens at CU, NCREE, and NTU are 2 m, 6 m, and 3 m, respectively. Table 4.4 gives a summary of the scale factors adopted and dimensions of the test specimens to be tested at the participating laboratories. Figures 4.3 to 4.5 show the test specimens constructed at CU, NCREE and NTU laboratories. The relationship between the prototype structure and the test specimens are as follows (Harris and Sabnis, 1999)

$$D_s = SF \times D_p \quad (4.3)$$

$$R_s = SF^2 \times R_p \quad (4.4)$$

where D and R denote the displacement and restoring forces. The subscripts s and p indicate the scaled specimen and prototype structure, respectively.

4.3 Instrumentations

4.3.1 Strain Gauges

Strain gauges were installed on the DSCFT column to measure the strain of the DS tubes at different levels during the tests. The schematic drawings of the strain gauges are shown in Figures 4.6 to 4.10. Six strain gauges were installed on the anchor bolts of the embedded anchoring ring system to measure the tensile strains of the anchor bolts during the tests. The strain gauges used in this experiment are Showa 5mm strain gauges.

In order to achieve the optimum bonding between the gauges and the steel tubes, the locations where the gauges were to be installed were grinded to ensure a smooth and clean surface. Following the bonding of the strain gauges, they were sealed and covered with water-resistant latex material and further protected with a silicone overlay, shown in Figure 4.11. All strain gauges were also tested after the casting of the footing of the test specimen and found to be functioning except of strain gauge No. 29. The locations of the strain gauges are listed in Table 4.5.

4.3.2 Potentiometer Instrumentation

On the test specimen at CU laboratory, three linear variable differential transformers (LVDT) were installed at 250 mm (the expected yield line), 1700 mm, and 2000 mm measured from the top of the footing on the faces opposite to the actuators in both longitudinal and transverse directions. The top two LVDTs were used to measure the absolute displacements of the test specimen during the experiment. The supporting racks of the LVDTs were anchored on the top of the footing to eliminate the errors in measuring the displacement of the test specimen during the experiment due to slippage and rocking of the footing. One end of the LVDT was fixed on the supporting rack while the other end was pinned to the test specimen. The locations of the LVDTs are shown in Figure 4.12.

4.4 Material Properties

DSCFT columns were formed using two round hollow structural sections and filled with concrete between the tubes. Some advantages of this design are the reduction in the amount of materials, increased confinement of core concrete, and no formwork required for casting concrete. The materials used in the construction of the double-skinned tubular column are described in the following sections.

4.4.1 Double-Skinned Tubular Column

The double-skinned tubular column was manufactured from steel sheets that were rolled and then welded together. A base plate with stiffeners was welded to the base of the DS

tubes. Design considerations and procedures of the base plate and stiffeners can be found in Wei (2002) and are not presented here.

The designed thicknesses of the steel tubes were 3 mm and 4 mm of A36 steel plates for the inner and outer tubes, respectively. Due to the limitations on supplies, the actual plate thicknesses of the test specimen at CU were 3.5 mm and 4.9 mm in average for the inner and outer tubes, respectively. Five steel coupons were fabricated from each sheet plate for uniaxial tensile tests, which were carried out at room temperature, following the ASTM standard to verify the material properties. The dimensions of the steel coupons fabricated at CU are shown in Figure 4.13. The average yield strength of the steel tubes are 224.1 MPa and 253.4 MPa for the inner and outer steel tubes, respectively. The uniaxial test results of the steel coupons are summarized in Table 4.6.

4.4.2 Concrete

The concrete ordered for the fabrication of the footing and for the in-filled concrete has a specified nominal 28 days compressive strength of 30MPa, 100mm slump, 10mm maximum aggregate size, and no air entrainment. Super-plasticizer was added to cement mixture before casting to increase its workability. The concrete footing was cast by two pours. During the first pour, cement mixture was pour to the bottom of the anchoring ring system (about 35 cm above the ground), which is a part of the embedded column-to-foundation connection. One week after the first pour of concrete, the double-skinned column was installed and cement mixture was pour to the top of the footing formwork and between the steel tubes. The slump tests of the first pour are 150 mm without super-

plasticizer and 190 mm with super-plasticizer. For the second pour of the concrete footing and the infill of the DSCFT column, the slump test results are 150 mm without super-plasticizer and 250 mm with super-plasticizer. Two sets of tests were performed on the standard concrete cylinders: first set of cylinder testing at 28 days, and second set of cylinder testing at the day of hybrid testing experiment. 18 concrete cylinders were tested to obtain the average compressive strength of concrete at CU. The average 28 days compressive concrete strength for the three test specimens are 40.1 MPa, 27.5 MPa, and 27.5 MPa at CU, NCREE, and NTU, respectively. The material properties of the three test specimens are summarized in Table 4.6.

4.4.3 Reinforcing Steel

The reinforcing steel used in for the fabrication of the footing was grade 400. The main reinforcing steel bars were No. 25M deformed bars, as shown in Figure 4.14, and the ties were No. 20M deformed bars. The circular reinforcing cage around the embedded DSCFT columns is formed using No. 20M deformed bars with No. 15M stirrups, which are shown in Figure 4.15. All reinforcing bars had 135-degree bend at two ends with an extension of no less than 6 times of the diameter of the reinforcing bars. The configurations of the reinforcing bars, ties, and stirrups are shown in Figure 4.16.

4.5 Fabrication and Assemblage of Test Specimen

The main reinforcing box was composed from rectangular and U-shape reinforcing bars as shown in Figure 4.17. Because of the shape of reinforcing bars, the order of the installation of reinforcing bars was important. The first rectangular reinforcing bar was

place in the transverse direction. The second rectangular reinforcing bar was slide into the first rectangular reinforcing bar through the opening at the bend, and then the second rectangular reinforcing bar turned to the longitudinal direction. Slide seven rectangular reinforcing bars one by one through the opening of the reinforcing bar in the longitudinal direction, and then turned 90 degrees to the transverse direction. Repeat the same procedure for the other seven rectangular reinforcing bars and placed them in the longitudinal direction. The U-shaped reinforcing bars were then hooked to the rectangular reinforcing cage in both the transverse and longitudinal directions. Circular reinforcing cage was then inserted into the rectangular cage. Once the circular reinforcing cage was in place, the rectangular reinforcing bars and the U-shaped reinforcing bars were then rearranged to the desired spacing and then tied together. The anchoring ring system, as shown in Figure 4.18, was installed after the reinforcing cage had positioned into the formwork.

The first casting of the concrete was poured up to 35 cm measured from the base of the footing, as shown in Figure 4.19. The double-skinned (DS) column was installed after one week of the first casting, shown in Figure 4.20. After the DS column was installed, concrete mixture was poured to the top of the footing, between the DS column and up to the top of the loading block, and inside the inner tube of the DS column up to 30 cm above the top of the footing.

4.6 Test Set-up

At Carleton University, two actuators were setup to impose deformations on the DSCFT column under bi-lateral input ground motions. The load capacities of the two actuators were 110 kips (489.3 kN) and 150 kips (667.25 kN) in the X and Y-direction, respectively. The X-direction is the longitudinal direction of the roadway and the Y-direction is in the transverse direction of the deck. The stroke capacities of the two actuators were ± 5 inches (127.5 mm) and ± 3 inches (76.5 mm) in the X and Y-direction, respectively. The actuators were supported by two large strong supporting frames with tie-backs for additional security, as shown in Figure 4.21. The footing was anchored to the strong floor with four anchoring bolts, which had a diameter of 2-1/2 inches each. Due to the difficulty of making threads at the ends of anchor bolts, heavy duty washers are welded to one end of the anchor bolts as shown in Figure 4.22. Post jacks were used to crank apart the legs of supporting frames to reduce the amount of possible sliding errors (Figure 4.23). Anti-sliding devices, shown in Figure 4.24, were also installed at the four faces of the footing to further reduce or even eliminate sliding of the footing during the experiment. Four structural angles were installed at the four corners of the footing as reference points for survey and photo images.

Constant axial load was applied on the test specimen during the experiment by post-tensioning a dywidag bar that ran through the centre of the test specimen. One end of the dywidag bar was locked at the base of the footing and the other end of the bar was stretched by using a hollow jack, which provided the constant compression force, placed on the top of the specimen. The axial force was measured using a pancake shaped hollow

load cell, which was placed between the hollow jack and the top surface of the DSCFT column, shown in Figure 4.25.

4.7 Summary

A four-pier bridge system was selected as the prototype structure for this Canada-Taiwan network hybrid test. Three DSCFT columns were designed and fabricated at CU, NCREC, and NTU. The scaling factors were 1/5, 2/5, and 1/5 for the test specimens at CU (P1), NCREC (P2), and NTU (P3), respectively, and the corresponding column heights were 2 m, 6m, and 3 m. All columns were embedded in a foundation detailed to develop the full plastic moment from the composite column (Tsai and Wei, 2002; Wei, 2002). The average compressive strength of concrete at 28 days are 40 MPa, 27.5 MPa, and 27.5 MPa for P1, P2, and P3, respectively.

Total of 36 strain gauges installed on the test specimen at CU to measure the shear strain of the DS tubes during the tests. Six strain gauges were installed on the anchor bolts of the embedded anchoring ring system to measure the tensile strains of the anchor bolts during the tests. Six LVDTs were installed to measure the absolute displacements of the test specimen during the experiment.

Two actuators were set up to impose loads in both longitudinal and transverse directions. Post jacks and anti-sliding devices were installed to reduce rocking and sliding of the footing during an experiment. A constant axial load was applied on the test specimen by

post-tensioning a dywiday bar through the centre of the specimen to simulate the weight of the superstructure.

Table 4.1 Stiffness and strength of the superstructure and piers

	Superstructure	Piers			
		P1	P2	P3	P4
EI (MN-m ²)	65340.0 (strong-axis)	23492.2	50920.9	50920.9	50920.9
EA (MN)	414520	92566.1	137228.1	137228.1	137228.1
J (m ⁴)	5.96	0.115	0.243	0.243	0.243
E _m (MPa)	N.A.	409947.1	397187.1	397187.1	397187.1
F _{my} (MPa)	248.2	741.7	711.7	711.7	711.7

Table 4.2 Details of full-scale DSCFT bridge piers

Piers	P1	P2	P3	P4
Column height	10 m	15 m	15 m	12 m
Axial Load(kN)	1.02E+04	1.04E+04	1.04E+04	1.09E+04
Lateral force requirement (kN)	2.55E+03	2.18E+03	2.12E+03	2.80E+03
Moment requirement (kN-m)	25520	32730	31860	33600
Diameter of external tube	2.5 m	3.0 m	3.0 m	3.0 m
Diameter of internal tube	1.5 m	1.8 m	1.8 m	1.8 m
Thickness of external tube	2cm	2.5cm	2.5cm	2.5cm
Thickness of internal tube	1.5cm	2cm	2cm	2cm
Axial load capacity P0 (kN)	116854	131040	131040	131040
Moment capacity (kN-m)	57599	95277	95277	95277
Aspect ratio of axial load (%)	8.74%	7.95%	8.91%	8.35%
Aspect ratio of moment (%)	44.3%	34.4%	55.2%	35.3%

Table 4.3 Design details of DSCFT-to-foundation connections

Specimen Location	h^1 (m)	h_e^2 (m)	M_{base} (kN-m)	$M_{bearing}$	$M_{overhang}$	M_{bolt}	$\frac{M_{bearing}}{M_{base}}$	$\frac{M_{overhang}}{M_{base}}$	$\frac{M_{bolt}}{M_{base}}$	Amount of bolts	Strength Ratio
P1 (CU)	1.0	0.5	511	591	151	340	1.16	0.30	0.67	10-A325 M16	2.13
P2 (NCREE)	1.72	0.96	7644	5070	1008	5243	0.66	0.13	0.69	10-A325 M16	1.48
P3(NTU)	1.0	0.5	961	684	296	454	0.71	0.31	0.47	24-A325 M32	1.49

¹ Height of the footing.

² Depth of the embedded footing-to-column connection.

Table 4.4 Dimensions of test specimens

	Pier 1	Pier 2	Pier 3	Pier 4
Location	Carleton	NCREE	NTU	Simulated
Scale Factor	20%	40%	20%	100%
Height of the Prototype Structure	10 m	15 m	15 m	12 m
Original outer tube diameter	2.5 m	3.0 m	3.0 m	3.0 m
Original inner tube diameter	1.5 m	1.8 m	1.8 m	1.8 m
Scaled Specimen Height	2 m	6m	3 m	-
Scaled outer tube diameter	0.5 m	1.2 m	0.6 m	-
Scaled inner tube diameter	0.3 m	0.72 m	0.36 m	-
Scaled outer tube thickness	4 mm	10 mm	5 mm	-
Scaled inner tube thickness	3 mm	8 mm	4 mm	-
Stress scale	100%	100%	100%	100%
Force scale	4%	16%	4%	100%
Moment Scale	0.8%	6.4 %	0.8 %	100%
Designed axial load capacity (kN) ³	4674	27346	6838	-
Scaled axial load applied (kN)	408.5	2174	609	-

³ Test specimens are designed by Dr. M.L. Lin of NCREE.

Table 4.5 Location of strain gauges

Box #	Channel # in Box	HP3497A Channel #	Strain Gauge #	Location ⁴
1	0	0	12	Embedded
	1	1	20	75 mm
	2	2	28	175 mm
	3	3	11	Embedded
	4	4	19	75 mm
	5	5	27	175 mm
	6	6	35	400 mm
	7	7	39	1050 mm
	8	8	26	75 mm
	9	9	34	175 mm
		10 (power supply)		
2	0	20	13	Embedded
	1	21	21	75 mm
	2	22	30	175 mm
	3	23	36	400 mm
	4	24	40	1050 mm
	5	25	22	75 mm
	6	26	14	Embedded
	7	27	29 (damaged)	
	8	28	15	Embedded
	9	39	23	75 mm
		30 (power supply)		

⁴ Locations of the strain gauges are measured from the top of the footing.

3	0	40	10	Strain gauges on the inner surface of the inner tube
	1	41	9	
	2	42	8	
	3	43	7	
	4	44	6	Strain gauges on the steel bolts (embedded)
	5	45	5	
	6	46	4	
	7	47	3	
	8	48	2	
	9	49	1	
		50 (power supply)		
4	0	60	31	175 mm
	1	61	37	400 mm
	2	62	41	1050 mm
	3	63	16	Embedded
	4	64	24	75 mm
	5	65	32	175 mm
	6	66	42	1050 mm
	7	67	38	400 mm
	8	68	33	175 mm
	9	69	25	75 mm
		70 (power supply)		
5	0	80	17	75 mm
	1	81	18	75 mm
		90 (power supply)		

Table 4.6 Material test results

Pier No.		P1		P2		P3	
		outer	inner	outer	inner	outer	inner
Steel tubes	f_y (MPa)	253.4	224.1	314.9	364.9	302.1	233.5
	f_u (MPa)	442.9	363.9	489.0	493.2	385.0	335.7
Concrete	f'_c	40.1		27.5		27.5

Figure 4.1: Prototype bridge system

Figure 4.2: Dimensions of the superstructure of the prototype system

Figure 4.3: Test specimen at Carleton University

Figure 4.4: Test specimen at NCREE

Figure 4.5: Test specimen at NTU

Unit: mm

Figure 4.6: Locations of strain gauges on the DS column at CU (Elevation View)

Figure 4.7: Locations of strain gauges on the DS column at CU (Plan View)

Front view of the Anchoring
Ring System
Unit:mm

Strain Gauge Location
(Threaded rods)

Figure 4.8: Schematic drawing of strain gauges on the anchoring ring system

Figure 4.9: Instrumentation layout of DSCFT column at NCREE

Figure 4.10: Instrumentation layout of DSCFT column at NTU

Figure 4.11: Installed strain gauges at CU

Figure 4.12: Schematic drawing of the LVDTs (at CU)

(a) steel coupon of outer steel tube

(b) steel coupon of inner steel tube

Figure 4.13: Details of steel coupon cut from the sheet plates for the inner and outer steel tubes

Figure 4.14: Rectangular reinforcing bars

Figure 4.15: Circular reinforcing cage

Unit: mm

Types of Rebar	Rebar Size	Quantity
#7 (or 25M)	16
#7 (or 25M)	8
#5 (or 15M)	3
#6 (or 20M)	18
#4 (or 15M)	8

Figure 4.16: Reinforcing bars schedule

Figure 4.17: Complete reinforcing cage

Figure 4.18: Anchoring ring system

Figure 4.19: First casting of the test specimen

Figure 4.20: DS column installed before the 2nd pour of concrete

Figure 4.21: Supporting frames at Carleton University

Figure 4.22: Anchor bolts

Figure 4.23: Post jack system

Figure 4.24: Anti-sliding device

Figure 4.25: Axial loading system

CHAPTER 5

5 ANALYTICAL MODELS AND SIMULATION RESULTS

5.1 Introduction

Nonlinear static and dynamic analyses are performed using OpenSees, an object-oriented simulation framework developed by researchers at the Pacific Earthquake Engineering Research (PEER) Center for simulating the seismic responses of structural and geotechnical system subjected to earthquakes (Mazzoni *et al.*, 2000), and PISA3D (Tsai and Lin, 2003), developed by researchers at NCREE, prior to the test to calculate the natural frequencies and periods of the test bridge, to predict the possible peak responses of the piers during the test, and to verify the force and stroke limitations of the actuators are not exceeded. The material parameters are based on tension coupon tests of steel tubes and concrete compressive tests of the specimen material at the participating laboratories. To investigate the versatility and accuracy of the ISEE environment, both OpenSees and PISA3D models are used to simulate the responses of the bridge by the Analysis Engine during the collaborative experiment.

5.2 Analytical models

5.2.1 PISA 3D model

From previous experimental studies by researchers at NCREE, it is shown that the hysteretic behavior of DSCFT column is similar to CFT column modeled by using PISA3D (Tsai *et al.* 2004). In this study, DSCFT columns are modeled using three-

parameter degrading beam-column elements, which consider strength deterioration, stiffness degradation, and pinching behaviour of the concrete (Tsai and Lin, 2003; Lin and Tsai, 2006), as shown in Figure 5.1. The steel tubes of the DSCFT columns are modeled as strain hardening material as shown in Figure 5.2. The superstructure of the bridge system is modeled using beam-column elements, which the internal moments are assumed linearly distributed over the element and the internal shear is constant over the entire length of the element. In the numerical model of the bridge, each span is divided into four elements and the superstructure mass is lumped at each nodal point connecting the beam. The connections between the superstructures and the bridge piers are modeled using elastic material. Second order effects or geometrical axial load effects are considered in the analysis. Details of the development of the PISA3D model and discussion on the numerical simulation results obtained by PISA3D are provided by NCREE research team and can be found in the reference by Weng *et al.* (2006).

5.2.2 OpenSees model

Open System for Earthquake Engineering Simulation framework (OpenSees) is a software framework in development at the University of California, Berkeley (Mazzoni *et al.*, 2000) for simulating the seismic response of structural and geotechnical system subjected to earthquakes. It has a library of different material and structural element modeling modules and computational platforms which can be selected and assembled into specific analysis tools for advanced analysis of complex structural system. In the OpenSees model, the superstructure is modeled as a beam-column element. Each span is divided into four segments and the superstructure mass is lumped at each nodal point connecting the beam. The location of each lumped mass point is elevated to the level of

the centre of gravity of the bridge deck using a vertical rigid link element. Each cap beam of bridge pier is also modeled as a rigid link element connecting to the bridge pier. The deck and the piers are modeled in OpenSees by displacement-based beam-column elements with five integration points. Each integration point represents a nonlinear section moment-curvature status. Figure 5.3 shows the structural model for the prototype bridge system. The concrete and steel are modeled using the FEDEAS steel and concrete material models (Filippou 1996) implemented in OpenSees. The concrete model adopted in the analysis considers tensile strength and linear tension softening behaviour of concrete material as shown in Figure 5.4. The steel model is the Giuffre-Menegotto-Pinto model with isotropic strain hardening as shown in Figure 5.5.

5.2.3 OpenSees model using Simplified Approach

DSCFT is an innovative structural system particularly suitable for construction of tall bridge piers in seismic regions. Since the DSCFT is a complex composite structural system, it is difficult to analyze using general analysis methods. A simplified method developed at Carleton University for prediction of the nonlinear hysteretic responses of composite structural members is applied here to analyze the behaviour of the DSCFT columns to obtain insights of their behaviour (Dailamani, 2002; Chang *et al.*, 2004; Deng, 2006). In the simplified method, a composite structural member is modeled by an equivalent member of homogeneous nonlinear material with a derived stress-strain relationship, which satisfies the requirement that the equivalent member has the same moment-curvature behaviour as the original member. Selected states in the behavior of the composite structural members are analyzed explicitly to establish directly points on the specified stress-strain relationship of the nonlinear homogeneous material model of

the equivalent member by strain energy and strain compatibility in the derivation of the specified stress-strain relationship of the nonlinear equivalent member material. Typical states of the composite structural member section considered in the formulation are cracking of concrete, yielding of reinforcing steel, peak stress in concrete, reinforcing steel hardening, residual stress in confined concrete, and ultimate fracture of reinforcing steel. The equivalent stress-strain relationship derived using the Simplified Method is shown in Figure 5.6.

To illustrate the procedure for the derivation of the stress-strain relationship of the equivalent member section, a rectangular reinforced concrete beam member is analyzed as an example here. The procedure can be extended to the analysis of any reinforced concrete or concrete-steel composite structural members of arbitrary shapes with different reinforcing details based on the same approach.

Assuming the strain at the top surface of the rectangular member section at a particular state is ε_{top} , and the neutral axis is located at a depth of c measured from the top of the member section as shown in Figure 5.6, the curvature ϕ of the section can be expressed by Equation 5.1.

$$\phi = \frac{\varepsilon_{top}}{h - c} \quad (5.1)$$

where h is the member section height

Using the concrete constitutive model by the modified Kent and Park and the steel material model, the neutral axis is determined by an iterative process based on equilibrium of the internal resultant forces of the member section. The internal force F and moment M of the section, can be obtained by integrating the stress distributions on concrete and steel over the member section area A as follows

$$F = \int \sigma dA \quad (5.2)$$

$$M = \int \sigma y dA \quad (5.3)$$

where y is the moment arm of the stress in the member section.

As discussed earlier, an equivalent member section shown in Figure 5.7 (a) is assumed to be composed of a uniform homogeneous material. The characteristic of this material is that the stress-strain relationship of the equivalent section, as shown in Figure 5.7 (f), should lead to the same moment-curvature behaviour as the original member section, as shown in Figure 5.7 (d) and 5.7 (g). In the formulation adopted here, the equivalent section is defined to have the same total area of the original member section. In the derivation, the equivalent section is derived by having the same curvature as the original member section. The homogeneous material is assumed to have the same stress-strain behaviour in both tension and compression in the derivation of the equivalent section as shown in Figure 5.7 (b).

Typical states of the reinforced concrete member section considered in the formulation are cracking of concrete, yielding of reinforcing steel, peak stress in concrete, reinforcing steel hardening, residual stress in confined concrete, and ultimate fracture of reinforcing steel, as illustrated in Figure 5.7. To obtain a more accurate and complete equivalent stress-strain relationship for the analyzed reinforced structural member, additional states in the behaviour of the reinforced structural member can be analyzed.

5.3 Selection of Input Excitations

The design level of seismic hazard is commonly presented in the form of a uniform hazard spectrum (UHS), which is an envelope of response spectra at specified probability level. The seismic hazard is described by spectral-acceleration values at periods of 0.2, 0.5, 1.0 and 2.0 seconds (Atkinson and Beresnev, 1998).

5.3.1 Selection of ground motion time histories

In general, recorded ground motion time histories are not the same as or compatible with the design spectrum. Therefore, the time history of recorded ground motion needs to be spectrally modified so that its response spectrum is compatible with the design response spectrum.

A commonly accepted method of scaling ground motions for non-linear time history analysis is based on matching as closely as possible the response spectrum of the scaled motion and the target spectrum over a given range of periods. Shome et al. (1998) proposed scaling to match the spectral acceleration to the target spectral acceleration at the first-mode period of a structure. To simulate the bi-lateral seismic responses of the

multi-span bridge, a series of bi-directional ground motions is used in this experiment. Scale input ground motion is important because the damage of a structure is frequently associated with a resonance between the frequency of the ground motion and the natural frequency of the structure.

Based on the scaling method proposed by Shome *et al.* (1998), the earthquake records are scaled to match the spectral responses at the fundamental period, $S_a(T_1)$, of the prototype structure in the direction considered. In this study here, the guidance of UBC (UBC, 1997) and International Building Code 2000 provisions (ICC, 2000) on scaling of earthquake records is followed. For each pair of (bi-lateral) horizontal ground motion components, the square root of the sum of the squares (SRSS) of the 5 percent-damped site specific spectrum of the scaled horizontal components is constructed. The motion is scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5%-damped spectrum for the design earthquake for periods between $0.2T$ seconds and $1.5T$ seconds, where T is the fundamental period of the building.

Two pairs of earthquake records from the 1999 Chi-Chi earthquakes and from the UHS compatible ground-motion time histories for Canada (Atkinson and Beresnev, 1998) are chosen as the input ground motion for this experiment. In the selection of the ground accelerations, records from the 1999 Chi-Chi earthquake in Taiwan and other North America earthquakes have been considered as possible input ground motions for the test. The final selected input ground motions for the test are the 1999 Chi-Chi earthquake motions recorded at Station CHY024 and an artificial earthquake record compatible with

the uniform hazard spectrum specified for Vancouver, Canada. The design spectra of spectral accelerations for 10% probability of exceedance in 50 years are normalized spectral accelerations with a factor $C = 1.25$, seismic zone factor $Z = 0.33$, and seismic force reduction factor $F_u = 2.0$ in the seismic design code of Taiwan (MOTC 1995; MOTC 2001). The spectral accelerations for Vancouver representing 5% damped seismic hazard spectrum at 2%/50 year probability are 0.94g, 0.64g, 0.33g, 0.17g for fundamental periods of 0.2 second, 0.5 second, 1 second and 2 seconds, respectively.

The earthquake records are scaled to match the spectral acceleration at the first vibration period of the bridge to specified earthquake hazard levels. The test specimens are tested by subjecting to increasingly higher seismic demands which represent incipient damage level earthquakes (50% probability of exceedance in 50 years), design earthquake (10% probability of exceedance in 50 years), the maximum considered level earthquake (2% probability of exceedance in 50 years) and then another earthquake record of 10% probability of exceedance in 50 years to represent an aftershock earthquake.

The original CHY024 and M65R40T4 acceleration time-histories are shown in Figure 5.8. The corresponding spectra representing the design earthquake and the maximum considered level earthquake hazard levels are shown in Figure 5.9. The response spectra of the selected Chi-Chi earthquake ground acceleration and the spectrum-compatible time history record for western Canadian earthquake are shown in Figure 5.10. The earthquake scenario for this experiment, including the earthquake intensity and sequence, is shown in Figure 5.11, which includes the three scaled earthquake ground accelerations

which represent 50% probability of exceedance in 50 years (M65R40T4), 10% probability of exceedance in 50 years (CHY024), and 2% probability of exceedance in 50 years (M65R40T4) in the longitudinal and transverse directions.

5.3.2 Bi-directional quasi-static cyclic loading

In addition, a cyclic load test was also carried out to investigate the ultimate collapse behaviour of the individual bridge pier to better understand the performance of the DSCFT column. A bi-directional loading pattern was used for the cyclic test at Carleton University Laboratory. The loading amplitudes are 0.25%, 0.375%, 0.5%, 0.75%, 1.0%, 1.5%, and 2% drift ratios of the test specimen for each one complete loading cycle, shown in Figure 5.12. The cyclic loading orbit is shown in Figure 5.13.

5.4 Analytical Results

In order to verify the demands of the testing equipment during the experiment, inelastic time-history analyses of the full-scale prototype bridge system were carried out before the collaborative experiment. Figure 5.14 shows the first four modes of the prototype structure based on the fiber layer model implemented in OpenSees. The first four modal periods are 0.91 second, 0.70 second, 0.62 second, and 0.62 second, respectively. The first three modes are vibrations in the direction perpendicular to the bridge roadway (Y-direction) and the fourth mode shows vibration in the direction parallel to the bridge longitudinal direction (X-direction). The bridge columns are evaluated under the input ground motions representing the incipient damage level earthquake (50% probability of exceedance in 50 years), the design earthquake (10% probability of exceedance in 50 years), and the maximum considered level earthquake (2% probability of exceedance in

50 years). The total length of the input ground motion is 115 seconds. Test A represents the incipient damage level earthquake, which is 15 seconds in length; Test B represents design earthquake, which is 45 seconds in length; Test C represents the maximum considered level earthquake, which is 15 seconds in length; Test D represents the aftershock earthquake, which is 40 seconds in length.

Figures 5.15 and 5.16 show the analytical hysteresis loops of the three models in the X and Y directions, respectively. It is observed that the analytical results from the three models agree reasonable well in the longitudinal direction of the bridge roadway (X-direction) for P1 and P4. The stiffnesses of the piers seem slightly lower in the OpenSees model than that of the other two models. In the direction perpendicular to the bridge roadway (Y-direction), the Simplified Approach (SA) model is stiffer than the other two models, which may be due to the uncertainty of the concrete confinement factor, k , used in the confined concrete model. From previous studies by Chang *et al.* (2004), the confinement provided by the double-skinned steel tubes is better than the conventional spiral confinement, and therefore, a higher confinement factor is adopted in the SA model.

Figures 5.17 to 5.32 show the analytical lateral bridge pier top displacements when the bridge is subjected to the input ground motion. The analytical displacement responses of the three models agree well in both X and Y-directions for Test A and Test B. It is noted that there are period elongations of the results obtained by the PISA3D model in the X-direction after 7.8 seconds of Test A, shown in Figures 17 to 20. As the intensity of the input ground motion increases, the discrepancy in analytical prediction becomes

noticeable because of the differences in damage models adopted in each analytical model. Permanent deformation of bridge piers is observed in the plots at the end of Test C, shown in Figures 5.25 to 5.28. The analytical responses by the PISA3D model for P4 in the Y-direction are noticeably different from the other two models.

The analytical base-shear forces when the bridge is subjected to the scaled ground motion representing different seismic hazard levels are presented in Appendix D. From the plots, it is noted that the estimated column base shear force is the lowest by the SA model in the X-direction. The analytical results from the OpenSees model and the SA model are in phase. Period elongation and amplitude decay of the PISA3D predictions at all four piers are observed after 7.8 seconds, which indicate the initiation of damage in bridge piers. Possible causes of discrepancy between the analytical results are different damage models and material models adopted in the three analytical models. In the Y-direction, the three analytical models give comparable results for P1, P2, and P3.

In summary, the analytical results from the three models agree reasonable well for all the earthquake scenarios. The estimated force and displacement requirements are 258 kN and 27.2 mm for P1 at CU, 898 kN and 65.6 mm for P2 at NCREC, and 240 kN and 27.2 mm for P3 at NTU, respectively, based on the force and displacement envelopes from the three analytical models described in Section 5.2. The estimated maximal displacements, drift ratio, and shear forces of the piers subjected to the input ground motions are summarized in Tables 5.1 and 5.2.

5.5 Summaries

Three analytical models of the full scale four-pier prototype bridge structure are analyzed to determine the demands of the testing equipment during the experiment. The four-pier bridge system is evaluated when subjected to bi-directional earthquake loads and cyclic pushover loading. The bridge piers are double-skinned concrete filled steel tubular piers ranging from 10 m to 15 m in height. Two pairs of earthquake records from the 1999 Chi-Chi earthquakes and from the UHS compatible ground-motion time histories for Canada are chosen as the input ground motion for this experiment. The ground motions are scaled to match the response spectra of the scaled motion and the target design spectra. The fundamental period of the prototype structure is estimated to be 0.91 seconds. The scaled peak ground acceleration is 0.74 g.

Overall, the analytical results obtained from the three models correlate well. The Simplified Approach model is stiffer than the other two analytical model due to a higher concrete confinement factor, k , adopted in the material models. The differences in the material models and damage models may be the possible causes of discrepancy in analytical predictions for Test C. From the force envelope of the three analytical models, the estimated peak column base-shear forces in the direction parallel to the bridge longitudinal axis (X-direction) are about 6459 kN, 4964 kN and 5113 kN for P1, P2, and P3, respectively. The estimated peak column base shear forces in the transverse direction to the bridge roadway (Y-direction) are 4243 kN, 5614 kN and 6001 kN for P1, P2, and P3, respectively. The corresponding reduce-scaled peak column base shear forces are about 258 kN, 794 kN and 205 kN in the X-direction for P1, P2, and P3 specimens,

respectively. The corresponding reduce-scaled peak column base shear forces in the Y-direction are 170 kN, 898 kN and 240 kN for P1, P2, and P3 specimens, respectively. From the displacement envelope of the three analytical models, the estimated peak lateral column top displacements are about 136 mm in the X direction at P1, and 164 mm in the Y direction at P2. The estimated peak column drift ratios are about 1.36% in the X-direction at P1 and 1.09% in the Y-direction at P2.

In addition, bi-directional cyclic pushover is performed after bridges piers have been exposed to the input ground motion to obtain the failure mechanism, the details of yield penetration, the estimation of plastic hinge zone length, and the hysteretic behavior of DSCFT bridge column member.

Table 5.1: Maximum analytical displacement responses of test specimens (full scale)

	P1 (at CU)		P2 (at NCREE)		P3 (at NTU)		P4 (Simulated)	
	X	Y	X	Y	X	Y	X	Y
PISA3D	135.8 mm	95.5 mm	135.5 mm	93.8 mm	135.4 mm	104.3 mm	130.3 mm	62.9 mm
OpenSees	127.7 mm	72.7 mm	125.2 mm	163.7 mm	126.8 mm	136.1 mm	122.6 mm	42.5 mm
Simplified Approach	116.7 mm	34.4 mm	114.6 mm	95.0 mm	115.9 mm	77.5 mm	112.6 mm	23.3 mm
Maximum	135.8 mm	95.5 mm	135.5 mm	163.7 mm	135.4 mm	136.1 mm	130.3 mm	62.9 mm
Drift Ratio	1.36%	0.96%	0.90%	1.09%	0.90%	0.91%	1.09%	0.42%

Table 5.2: Maximum analytical force responses of test specimens (full scale)

	P1 (at CU)		P2 (at NCREE)		P3 (at NTU)		P4 (Simulated)	
	X	Y	X	Y	X	Y	X	Y
PISA3D	5473.0 kN	4242.8 kN	4964.0 kN	5614.0 kN	4952.0 kN	6001.0 kN	7569.0 kN	4767.5 kN
OpenSees	4893.8 kN	3484.9 kN	4698.8 kN	4968.9 kN	5112.7 kN	5043.3 kN	9118.5 kN	4534.2 kN
Simplified Approach	4208.7 kN	2995.8 kN	3484.1 kN	5227.2 kN	3772.9 kN	5468.8 kN	7066.2 kN	4443.0 kN
Maximum	5473.0 kN	4242.8 kN	4964.0 kN	5614.0 kN	5112.7 kN	6001.0 kN	9118.5 kN	4767.5 kN

(a) States of degrading material

(b) Strength deterioration

(c) Stiffness degradation

(d) Pinching effects

Figure 5.1: PISA3D three-parameter concrete model

Figure 5.2: PISA3D hardening steel model

Figure 5.3: Structural model for the prototype bridge system

Figure 5.4: OpenSees concrete model

Figure 5.5: OpenSees steel model

Figure 5.6: Equivalent strain and stress profiles

(a) R.C. section	(b) Concrete model	(e) Equivalent	(f) Equivalent stress-strain relationship
Typical states of R.C. member 1. Cracking of concrete 2. Yielding of steel 3. Peak stress in concrete 4. Steel hardening 5. Residual stress in concrete 6. Ultimate fracture of steel
	(c) Steel model		(g) Equivalent moment curvature
Reinforced concrete member	(d) R.C. member moment-curvature	Equivalent member	

Figure 5.7: Simplified analysis model of reinforced concrete member section

Figure 5.8: Original ground accelerations used in test (before scaling)

Figure 5.9: Design spectra

Figure 5.10: Response spectra of the selected input ground motion

Figure 5.11: Earthquake scenario for the collaborative experiment

Figure 5.12: Cyclic loading pattern

Figure 5.13: Cyclic loading orbit

(a) The 1st mode ($T=0.91$ sec.)

(b) The 2nd mode ($T=0.70$ sec.)

(c) The 3rd mode ($T=0.62$ sec.)

(d) The 4th mode ($T=0.62$ sec.)

Figure 5.14: The first four vibration modes of the prototype bridge system

Figure 5.15: Analytical hysteresis loops of the three models in the longitudinal direction of the bridge (X-direction)

Figure 5.16: Analytical hysteresis loops of the three models in the transverse direction of the bridge (Y-direction))

Figure 5.17: Comparison of analytical displacement responses for P1 (Test A)

Figure 5.18: Comparison of analytical displacement responses for P2 (Test A)

Figure 5.19: Comparison of analytical displacement responses for P3 (Test A)

Figure 5.20: Comparison of analytical displacement responses for P4 (Test A)

Figure 5.21: Comparison of analytical displacement responses for P1 (Test B)

Figure 5.22: Comparison of analytical displacement responses for P2 (Test B)

Figure 5.23: Comparison of analytical displacement responses for P3 (Test B)

Figure 5.24: Comparison of analytical displacement responses for P4 (Test B)

Figure 5.25: Comparison of analytical displacement responses for P1 (Test C)

Figure 5.26: Comparison of analytical displacement responses for P2 (Test C)

Figure 5.27: Comparison of analytical displacement responses for P3 (Test C)

Figure 5.28: Comparison of analytical displacement responses for P4 (Test C)

Figure 5.29: Comparison of analytical displacement responses for P1 (Test D)

Figure 5.30: Comparison of analytical displacement responses for P2 (Test D)

Figure 5.31: Comparison of analytical displacement responses for P3 (Test D)

Figure 5.32: Comparison of analytical displacement responses for P4 (Test D)

CHAPTER 6

6 EXPERIMENTAL RESULTS

6.1 Introduction

A four-pier bridge system is chosen as the prototype structure for the hybrid collaborative test. The bridge piers are double-skinned concrete filled tubular (DSCFT) columns, which consist of two concentric circular thin steel tubes with concrete filled in between. From past studies of behaviour and performance of bridges during major earthquakes, it is recognized that damage of the bridge structure usually occurs at the piers. Therefore, bridge girders and decks are generally designed as elastic structural components and are modeled by linear beam elements in the numerical model. Similarly in experimental assessment of bridge structure performance, only structural members expected to behave non-linearly are needed to be tested in the laboratory, while those parts of the bridge that remain elastic during earthquakes can confidently be simulated by numerical models. In the present study, the earthquake responses of a four-pier bridge are investigated by networked hybrid testing. Three bridge piers are tested at three remote sites distributed over great distance: one in Canada and two in Taiwan. The loadings and responses of the fourth pier are simulated by computer models. Different scaling factors are used in the design of the sub-component test specimens at the multiple test sites based on the available testing equipment and physical size of each participating laboratory in this international collaborative pseudo-dynamic experiment. For this research, the scale factors for displacement are chosen to be 1/5 for the Carleton University site specimen

and the National Taiwan University site specimen, and 2/5 for the NCREE site specimen. The corresponding specimen heights are 2 m, 6 m, and 3 m for the specimens at CU, NCREE, and NTU, respectively. The test specimens are loaded pseudo-dynamically using input ground motions from the 1999 Chi-Chi earthquake and the simulated Cascadia earthquake in Vancouver, Canada; the input ground motions are scaled to the representative seismic hazard levels of 50%, 10%, and 2% probabilities of exceedance in 50 years.

For this collaborative experiment, there are a total of 7 network participating programs and 72 data channels of information and data which are exchanged among the participating laboratories during the progress of the experiment. The participating programs include the Data Center, the Analysis Engine; the Facility Control Module (FCM) at CU; FCM and Data Acquisition (DAQ) at NCREE; and FCM and DAQ at NTU. The DAQ system at CU was incorporated into the Facility Control Modules at CU. The data channels of the experiment include displacements and resisting forces, applied constant axial force, base moments, selected strains of steel bars, rotation angles, and sliding displacement, etc. The complete list of data channels is presented in Table 6.1.

A full scale numerical model of the prototype structure is implemented in the Analysis Engine to calculate its dynamic responses using Newmark-based implicit-explicit integration scheme with a time step of 0.02 seconds. The total duration of the input ground motion is 115 seconds. The network configuration of the network hybrid test is shown in Figure 6.1. An OpenSees-based (McKenna & Fenves, 2000) Command

Generation Module (CGM) was used to carry out the test for the first 75 seconds of the input ground-motion. A PISA3D-based (Tsai & Lin, 2003) CGM was used to carry out the last test, Test D, to validate the first use of PISA3D engine in the ISEE environment.

6.2 Experimental Results

The input ground motion for this collaborative experiment is composed of two scaled earthquake acceleration records: the CHY024 Chi-Chi Earthquake record and an UHS compatible record from Canada. The input ground motions are scaled to represent different seismic hazard levels.

Test A uses the scaled M65R40T4 UHS compatible record as the seismic hazard at the hazard level of 50% probability of exceedance in 50 years. This record is 15 seconds long and the scaled peak ground acceleration is 0.25 g in both X and Y directions.

Test B uses the Chi-Chi Earthquake motions recorded at station CHY024, which is scaled to represent the design earthquake in Taiwan and it is equivalent to the seismic hazard level that has 10% probability of exceedance in 50 years. Test B is 45 seconds long and the scaled peak ground acceleration is 0.39 g. The scaled CHY024EW record is used for the input ground motion in the X-direction and the scaled CHY024NS record is used for the input ground motion in the Y-direction.

Test C is the scaled M65R40T4 UHS compatible record to represent the seismic hazard level of 2% probability of exceedance in 50 years, which is the maximum design

earthquake in Canada. This input ground motion is 15 seconds in length and the scaled peak ground acceleration is 0.74 g in both X and Y directions.

Test D uses the same input ground motion as that in Test B, which now represents the aftershock earthquake that a structure may experience. The scaled CHY024NS record is used for the input ground motion in the X-direction and the scaled CHY024EW record is used for the input ground motion in the Y-direction. The input ground motion for Test D is 40 seconds in length and the scaled peak ground acceleration is 0.39 g.

At Carleton University, the loading of the test specimen is displacement controlled by external LVDTs. The error tolerance is set to be 0.1 mm. The network hybrid tests are set up for tele-presence from the other locations by broadcasting in real-time through the World Wide Web at <http://exp.ncree.org/dscft>. Table 6.2 lists the plots presented at the above link. The list of data collected at the Carleton University Laboratory is presented in Table 6.3. Table 6.4 presents the list of network pseudo-dynamic tests performed using the developed protocols.

6.2.1 Results of preliminary network testing

Preliminary tests are performed to investigate the network performance between the three participating laboratories using the ISEE environment. Listed below are the preliminary tests performed prior to the simultaneous networked testing of the three specimens:

- 1) Software testing is performed at three sites. All test specimens are numerically simulated.

- 2) Tests are performed on one test specimen while the other two sites are numerically simulated.
 - a. Test is performed at CU while NCREE and NTU are simulated.
 - b. Test is performed at NCREE while NTU and CU are simulated.
 - c. Test is performed at NTU while NCREE and CU are simulated.
- 3) Tests are performed on two test specimens while the third site is numerically simulated.
 - a. Test is performed at CU and NTU, while NCREE is simulated.
 - b. Test is performed at NCREE and NTU, while CU is simulated.

To ensure the test specimens do not exceed the elastic range, the input ground motions for the preliminary test are scaled down to 10% of the records for Test A to D that are described in Section 6.2. The average elapsed time per step for the three different scenarios ranges from 1.28 seconds to 3.28 seconds. From the preliminary results, it was concluded that the developed procedure and control modules can effectively carry out a collaborative multi-site experiment. The elapsed time, also referred to as time cost, per time step is expected to be about 3 to 4 seconds, depending on the magnitude of the input ground motion. As the displacement commands increase in amplitude, it is expected to take longer for the actuators to reach the target displacements. In addition, data acquisition systems that collect strain gauge data at all three sites are introduced to the multi-site pseudo-dynamic experiment, which also increases the time cost.

6.2.2 Results of multi-site hybrid test

For this international collaborative network hybrid experiment, the sub-structural components are loaded pseudo-dynamically at the participating laboratories using scaled input ground motions which represent different seismic hazard levels. The experimental force-displacement responses of the specimens are shown in Figures 6.2 to 6.5. A sudden decrease in force is observed during Test A at P1 in the X-direction, which is due to sliding of the footing in the direction of pulling during the test. As a result, an increase in pulling force is measured in the Y-direction at that instance as the actuator is pulling on the test specimen to maintain its position at the target displacement.

The comparisons of analytical and experimental hysteresis loops for the three test specimens at CU, NCREE, and NTU are shown in Figures 6.6 to 6.9. Under the incipient damage level earthquake and the design earthquake scenarios, the analytical results obtained by the OpenSees model and the PISA3D model agree reasonable well with the experiment results except for P2 and P3 in the direction perpendicular to the bridge roadway (Y-direction), where the PISA3D model is noticeably stiffer than the physical test specimens. At the incipient damage level earthquake scenario (Test A), the behaviour of the bridge piers depend mainly on the initial stiffness defined in the analytical model. Uncertainty in the selection of the parameter β for calculating the sectional properties, EI, of test specimens in Equation 4.1 is a possible reason of a stiffer PISA3D model than the actual test specimens. Similar observations are made when the bridge system is subjected to the design earthquake (Test B). At the maximum

considered seismic hazard level (Test C), both analytical models show damages in the bridge piers but the experimental results show signs of pinching only at P1 while P2 and P3 remain almost elastic as observed in the laboratories. The experimental results show minor decrease in strength of test specimens. No visual damage is observed during the experiment. The maximum force responses are 352.3 kN, 909.8 kN, and 203.6 kN in the X-direction for the test specimens at CU, NCREE, and NTU, respectively. In the Y-direction, the maximum force responses are 172.1 kN, 1052.6 kN, and 229.1 kN at CU, NCREE, and NTU, respectively. The maximum displacement responses are 21.82mm, 43.5 mm, and 21.9 mm in the X-direction for the test specimens at CU, NCREE, and NTU, respectively. In the Y-direction, the maximum displacement responses are 10.8 mm, 53.1 mm, and 26.9 mm at CU, NCREE, and NTU, respectively. The comparisons of the maximal force and displacement responses are presented in Table 6.5.

Comparisons are also made for force and displacement responses separately between the analytical and experimental results. Figures 6.10 to 6.17 show the comparisons of displacement responses. By comparing the displacement responses, it is observed that the OpenSees model slightly underestimate the strength of the three test specimens in both the X and Y directions for Test A and Test B; and the PISA 3D model overestimates the strength of the test specimens in the Y-direction for test specimens at NCREE (P2) and at NTU (P3). Some of the parameters set in the analytical model are based on the previous experimental work done at NCREE. It is possible that further studies are required to fully understand the nonlinear behaviour of the DSCFT columns to calibrate the parameters used in the PISA3D model. In the Y-direction, the analytical results from

the OpenSees model for P2 (at NCREE) and P3 (at NTU) start to shift out of phase with the experimental results after time of 8.0 seconds of the input ground motion.

The displacement loops of the four columns for Test C (2% probability of exceedance in 50 years) are shown in Figure 6.18. During Test C, a sudden slippage of the foundation occurred which caused the LVDT supporting rack to vibrate violently at CU Laboratory. This test was suspended until the post jacking system, shown in Figure 4.19, is installed. It was observed that the analytical predictions obtained from both the OpenSees model and the PISA3D model agree reasonable well with the experimental results in the X-direction. In the Y-direction, the analytical results from the OpenSees model agree well with the experimental results at P1 and P3. Figure 6.15 shows that higher displacement predictions are obtained by the OpenSees model than the experimental results. Analytical results from the PISA3D model agree well with the experimental results in the Y-direction at P2 and P3. The discrepancy between the analytical and experimental results is possibly caused by the damage models and material models adopted in the analytical models.

The PISA3D model was implemented in the Analysis Engine for Test D (aftershock earthquake). It was observed that the analytical results obtained from the OpenSees model agree well with the experimental results in the X-direction; however, the analytical results obtained from both the OpenSees and PISA3D models were very different from the experimental results in the Y-direction. After investigation of the PISA3D analytical model implemented for this experiment, it was found that there are a few errors in the

model input file. The nodal load applied at P1 in the Y-direction was omitted in the input file. Also, the sectional properties, EI, of P3 and P4 were not updated. These input errors caused the discrepancy between the experimental results and the analytical results. Similar observations were made between the analytical and experimental force-time histories, which are presented in Appendix E.

6.2.3 Results of cyclic tests

Bi-directional cyclic tests were performed on the test specimen at CU and NCREE after the real-time collaborative hybrid test. The network hybrid testing method is used for the cyclic test at CU. The displacements are received from the Analysis Engine through the Data Center and the restoring forces are measured once the target displacements are imposed on the test specimen. There is no integration scheme involved in this test. The loading patterns of the cyclic loadings are shown in Figure 6.191.

The cyclic test is suspended at tip displacement of 15 mm, the fourth cycles of the cyclic loading scheme, due to the failure of anchor bolts at welds and a visible crack at the base of the footing as shown in Figures 6.20 and 6.21, respectively. A Post-tensioning anchoring system was then implemented to fix the failure of anchor bolts. The post-tension force applied on each dywidag bar was 400 kN. Local buckling of the exterior steel tube at the base of the DSCFT column, shown in Figure 6.22, was observed when the test specimen was pushed to 2.6 % and 3.2% drift ratios in the X and Y directions, respectively. At the end of the cyclic test, the test specimen was pushed to the position of

the maximum strokes of the actuators, which is equivalent to a 3.75% drift ratio. The hysteresis behaviour of the test specimen under the cyclic loading is shown in Figure 6.23.

6.2.4 Network condition and elapsed time

Network stability and throughput performance are key factors for consideration in the setup of the network environment for carrying out a collaborative multi-site experiment. Since this experiment is carried out at the slow loading rate of a pseudo-dynamic test wherein the strain rate effect of load application on the material properties of the test specimens is ignored, the network performance is not as important as its stability. The average elapsed time per step for the four tests ranges from 3.05 seconds to 4.12 seconds. The average elapsed time per time step of the four seismic hazard levels is summarized in Table 6.4. Figure 6.24 shows the elapsed time of each time step for the four tests. The breakdown of network communication time per time step is shown in Figure 6.25. The Facility Control Time includes the actuator control time at Carleton University and the time to transfer actuator force responses to the Database Center. The Network time includes the data transmission time and the waiting time for the responses of the other participating laboratories, which correlate to the control time of other participating laboratories. At some instances, the network time alone takes up to 8.5 seconds which shows that the network performance throughout the experiment is not consistent. However, it is observed that the facility control is the main factor that influence the time cost. The Facility Control Time is found to be up to 16.0 seconds at one point and is found to be above 10 seconds at many instances. At Carleton University Laboratory, increasing in control time may be caused by the rocking of the foundation, slipping of the

foundation, rocking and slipping of the foundation, and deformation of the supporting frame.

6.3 Summary

Three double-skinned concrete filled tubular columns are constructed and tested at CU, NCREE, and NTU. The column heights are 2 m, 6 m, and 3 m, respectively. Network pseudo-dynamic tests are performed using the developed modules. The input ground motions compose of records from the Chi-Chi Earthquake and from Canadian UHS compatible ground motions; which are scaled to represent three different seismic hazard levels. The total length of the input ground motion is 115 seconds.

There were 7 network participating programs and 72 data channels involved in this collaborative multi-site experiment. The analytical and experimental results are broadcast on the World Wide Web on a real-time basis. Both OpenSees and PISA3D models were successfully implemented in the Analysis Engine to calculate the nonlinear responses of a structure during a network pseudo-dynamic experiment. A series of test were performed to investigate the network performance and the easiness of implementing the developed modules for facility control. The analytical predictions from the OpenSees model correlate well with the experimental results. The peak experimental shear forces are 352 kN, 910 kN, and 204 kN in the X-direcion of the test specimens at CU (P1), NCREE (P2), NTU (P3), respectively. In the Y-direction, the peak experimental shear forces are 172 kN, 1053kN, and 229 kN for P1, P2, and P3, respectively. The maximum drift ratios are 1.09% and 0.96% in the X and Y directions, respectively.

The average time cost per time step varies from 1.28 seconds, for network simulation between the three participating laboratories, and up to 4.12 seconds, for the case of seismic hazard level of 2% probability of exceedance in 50 years. The Facility Control Time takes about 50% to 60% of the time cost per time step. The Facility Control Time is influenced by the amplitude of the target displacements, the stiffness of the supporting frames, rocking and slipping of the foundation, and rocking and slipping of the supporting frames.

Table 6.1: List of data collected during the network experiment

CH	Symbol	Unit	Description
1	T	sec.	Time
2	P1BV _X	kN	P1 column base shear in the X-direction
3	P1BV _Y	kN	P1 column base shear in the Y-direction
4	DP1AXL	kN	P1 Additional Axial Load (Zero)
5	P1D _X	mm	P1 displacement in the X-direction
6	P1D _Y	mm	P1 displacement in the Y-direction
7	P1D _Z	mm	P1 axial displacement
8	P1DR _X	%	P1 drift ratio in the X-direction
9	P1DR _Y	%	P1 drift ratio in the Y-direction
10	P1DR _X	mm	Displacement measured of P1 in the X-direction
11	P1DR _Y	mm	Displacement measured of P1 in the Y-direction
12	CUF _X	kN	Force measured in the X-direction at CU
13	CUF _Y	kN	Force measured in the Y-direction at CU
14	P1M _X	kN-mm	P1 base moment in the X-direction
15	P1M _Y	kN-mm	P1 base moment in the Y-direction
16	P2BV _X	kN	P2 column base shear in the X-direction
17	P2BV _Y	kN	P2 column base shear in the Y-direction
18	DP2AXL	kN	P2 Additional Axial Load (Zero)
19	P2D _X	mm	P2 displacement in the X-direction
20	P2D _Y	mm	P2 displacement in the Y-direction
21	P2D _Z	mm	P2 axial displacement
22	P2 ϕ_X	deg	P2 Column base rotation in the X-direction
23	P2 ϕ_Y	deg	P2 Column base rotation in the Y-direction
24	P2 ϵ_1	-	P2 longitudinal strain at 1 st level
25	P2R1 ϵ_z	-	Axial strain of P2 steel rod No.1
26	P2R2 ϵ_z	-	Axial strain of P2 steel rod No.2

Table 6.1 (cont'd): List of data collected during the network experiment

CH	Symbol	Unit	Description
27	P2FD _X	mm	Foundation sliding displacement of P2 in the X-direction
28	P2FD _Y	mm	Foundation sliding displacement of P2 in the Y-direction
29	P2DR _X	%	P2 drift ratio in the X-direction
30	P2DR _Y	%	P2 drift ratio in the Y-direction
31	P2DR _X	mm	Displacement measured in the X-direction of P2
32	P2DR _Y	mm	Displacement measured in the Y-direction of P2
33	NCRF _X	kN	Force measured in the X-direction at NCREE
34	NCRF _Y	kN	Force measured in the Y-direction at NCREE
35	P2AF	kN	P2 Axial Load measured
36	P2CV _X	1/mm	P2 curvature in the X-direction
37	P2CV _Y	1/mm	P2 curvature in the Y-direction
38	P2M _X	kN-mm	P2 base moment in the X-direction
39	P2M _Y	kN-mm	P2 base moment in the Y-direction
40	P3BV _X	kN	P3 column base shear in the X-direction
41	P3BV _Y	kN	P3 column base shear in the Y-direction
42	DP3AXL	kN	P3 Additional Axial Load (Zero)
43	P3D _X	mm	P3 displacement in the X-direction
44	P3D _Y	mm	P3 displacement in the Y-direction
45	P3D _Z	mm	P3 axial displacement
46	P3 ϕ_X	deg	P3 Column base rotation in the X-direction
47	P3 ϕ_Y	deg	P3 Column base rotation in the Y-direction
48	P3 ϵ_1	-	P3 longitudinal strain at 1 st level
49	P3R1 ϵ_z	-	Axial strain of P3 steel rod No.1
50	P3R2 ϵ_z	-	Axial strain of P3 steel rod No.2

Table 6.1 (cont'd): List of data collected during the network experiment

CH	Symbol	Unit	Description
51	P3FD _X	mm	Foundation sliding displacement of P3 in the X-direction
52	P3FD _Y	mm	Foundation sliding displacement of P3 in the Y-direction
53	P3DR _X	%	P3 drift ratio in the X-direction
54	P3DR _Y	%	P3 drift ratio in the Y-direction
55	P3DR _X	mm	Displacement measured in the X-direction of P3
56	P3DR _Y	mm	Displacement measured in the Y-direction of P3
57	NTUF _X	kN	Force measured in the X-direction at NTU
58	NTUF _Y	kN	Force measured in the Y-direction at NTU
59	P3AF	kN	P3 Axial Load measured
60	P3CV _X	1/mm	P3 curvature in the X-direction
61	P3CV _Y	1/mm	P3 curvature in the Y-direction
62	P3M _X	kN-mm	P3 base moment in the X-direction
63	P3M _Y	kN-mm	P3 base moment in the Y-direction
64	P4BV _X	kN	P4 X-direction Column base shear
65	P4BV _Y	kN	P4 Y-direction Column base shear
66	P4D _X	mm	P4 X-direction displacement
67	P4D _Y	mm	P4 Y-direction displacement
68	P4D _Z	mm	P4 axial displacement
69	P4DR _X	%	P4 drift ratio in the X-direction
70	P4DR _Y	%	P4 Drift ratio in the Y-direction
71	P4M _X	kN-mm	P4 Base Moment in the X-direction
72	P4M _Y	kN-mm	P4 Base Moment Y-direction

Table 6.2: List of figures on the website

Plot	Figure title	X-axis	Y-axis
1	P1 column base shear in the X-direction (analytical and experimental) vs. time	Time	P1BV _x
2	P2 column base shear in the X-direction (analytical and experimental) vs. time	Time	P2BV _x
3	P3 column base shear in the X-direction (analytical and experimental) vs. time	Time	P3BV _x
4	P4 column base shear in the X-direction (analytical and experimental) vs. time	Time	P4BV _x
5	P1 column base shear in the Y-direction (analytical and experimental) vs. time	Time	P1BV _y
6	P2 column base shear in the Y-direction (analytical and experimental) vs. time	Time	P2BV _y
7	P3 column base shear in the Y-direction (analytical and experimental) vs. time	Time	P3BV _y
8	P4 column base shear in the Y-direction (analytical and experimental) vs. time	Time	P4BV _y
9	P1 displacement in the X-direction (analytical and experimental) vs. time	Time	P1D _x
10	P2 displacement in the X-direction (analytical and experimental) vs. time	Time	P2D _x
11	P3 displacement in the X-direction (analytical and experimental) vs. time	Time	P3D _x
12	P4 displacement in the X-direction (analytical and experimental) vs. time	Time	P4D _x
13	P1 displacement in the Y-direction (analytical and experimental) vs. time	Time	P1D _y

Table 6.2 (cont'd): List of figures on the website

Plot	Figure title	X-axis	Y-axis
14	P2 displacement in the Y-direction (analytical and experimental) vs. time	Time	P2D _Y
15	P3 displacement in the Y-direction (analytical and experimental) vs. time	Time	P3D _Y
16	P4 displacement in the Y-direction (analytical and experimental) vs. time	Time	P4D _Y
17	P1 column drift vs. column base shear in the X-direction	P1DR _X	P1BV _X
18	P2 column drift vs. column base shear in the X-direction	P2DR _X	P2BV _X
19	P3 column drift vs. column base shear in the X-direction	P3DR _X	P3BV _X
20	P4 column drift vs. column base shear in the X-direction	P4DR _X	P4BV _X
21	P1 column drift vs. column base shear in the Y-direction	P1DR _Y	P1BV _Y
22	P2 column drift vs. column base shear in the Y-direction	P2DR _Y	P2BV _Y
23	P3 column drift vs. column base shear in the Y-direction	P3DR _Y	P3BV _Y
24	P4 column drift vs. column base shear in the Y-direction	P4DR _Y	P4BV _Y
25	P1 displacement in the X-direction vs. displacement in the Y-direction	P1D _X	P1D _Y
26	P2 displacement in the X-direction vs. displacement in the Y-direction	P2D _X	P2D _Y
27	P3 displacement in the X-direction vs. displacement in the Y-direction	P3D _X	P3D _Y
28	P4 displacement in the X-direction vs. displacement in the Y-direction	P4D _X	P4D _Y
29	P2 curvature vs. column base moment (in the X-direction)	P2CV _X	P2BV _X
30	P3 curvature vs. column base moment (in the X-direction)	P3CV _X	P3BV _X
31	P2 curvature vs. column base moment (in the Y-direction)	P2CV _Y	P2BV _Y
32	P3 curvature vs. column base moment (in the Y-direction)	P3CV _Y	P3BV _Y

Table 6.3: List of channels at Carleton University

CH	Description	Symbol
1	Actuator Force in the X-direction	F_x
2	Actuator Force in the Y-direction	F_y
3	Axial Load	P_{1AF}
4	Strain gauge #1 (on steel rod)	e_1
5	Strain gauge #2 (on steel rod)	e_2
6	Strain gauge #3 (on steel rod)	e_3
7	Strain gauge #4 (on steel rod)	e_4
8	Strain gauge #5 (on steel rod)	e_5
9	Strain gauge #6 (on steel rod)	e_6
10	Strain gauge #7 (on the inside surface of the inner tube)	e_7
11	Strain gauge #8 (on the inside surface of the inner tube)	e_8
12	Strain gauge #9 (on the inside surface of the inner tube)	e_9
13	Strain gauge #10 (on the inside surface of the inner tube)	e_{10}
14	Strain gauge #11 (embedded, 400mm below the top surface of footing)	e_{11}
15	Strain gauge #12 (embedded)	e_{12}
16	Strain gauge #13 (embedded)	e_{13}
17	Strain gauge #14 (embedded)	e_{14}
18	Strain gauge #15 (embedded)	e_{15}
19	Strain gauge #16 (embedded)	e_{16}
20	Strain gauge #17 (embedded)	e_{17}
21	Strain gauge #18 (embedded)	e_{18}
22	Strain gauge #19 (75mm above the top surface of footing)	e_{19}
23	Strain gauge #20 (75mm above)	e_{20}
24	Strain gauge #21 (75mm above)	e_{21}
25	Strain gauge #22 (75mm above)	e_{22}

Table 6.3 (cont'd): List of channels at Carleton University

	Description	Symbol
26	Strain gauge #23 (75mm above)	e_{23}
27	Strain gauge #24 (75mm above)	e_{24}
28	Strain gauge #25 (75mm above)	e_{25}
29	Strain gauge #26 (75mm above)	e_{26}
30	Strain gauge #27 (175mm above the top surface of footing)	e_{27}
31	Strain gauge #28 (175mm)	e_{28}
32	Strain gauge #29 (175mm)	e_{29}
33	Strain gauge #30 (175mm)	e_{30}
34	Strain gauge #31 (175mm)	e_{31}
35	Strain gauge #32 (175mm)	e_{32}
36	Strain gauge #33 (175mm)	e_{33}
37	Strain gauge #34 (175mm)	e_{34}
38	Strain gauge #35 (400mm above the top surface of footing)	e_{35}
39	Strain gauge #36 (400mm)	e_{36}
40	Strain gauge #37 (400mm)	e_{37}
41	Strain gauge #38 (400mm)	e_{38}
42	Strain gauge #39 (1050mm above the top surface of footing)	e_{39}
43	Strain gauge #40 (1050mm)	e_{40}
44	Strain gauge #41 (1050mm)	e_{41}
45	Strain gauge #42 (1050mm)	e_{42}
46	LVDT #1 (2000mm above the top surface of the footing) - X-direction	ΔX_1
47	LVDT #2 (1700mm above the top surface of the footing) - X-direction	ΔX_2
48	LVDT #3 (325mm above the top surface of the footing) - X-direction	ΔX_3

Table 6.3 (cont'd): List of channels at Carleton University

	Description	Symbol
49	LVDT #4 (2000mm above the top surface of the footing)- Y-direction	ΔY_1
50	LVDT #5 (1700mm above the top surface of the footing) - Y-direction	ΔY_2
51	LVDT #6 (325mm above the top surface of the footing) - Y-direction	ΔY_3

Table 6.4: List of tests

Experiment Number ⁵	Sites ⁶	Notes ⁷	Average elapsed time per time step (sec)
304	NTU		1.86
306	NCREE		1.84
317	NCREE-NTU	Test 2A	2.48
318	NCREE-NTU	Test 2B,C,D	2.56
319	Software testing		1.28
320	CU-NTU	Test 3A	2.82
321	CU-NTU	Test 3C	3.28
322	CU	Test 4A	2.05
323	CU	Test 4B	2.13
324	CU-NCREE-NTU	Test 5A	3.30
325	CU-NCREE-NTU	Test 5B	3.05
326 + 327	CU-NCREE-NTU	Test 5C	4.12
328	CU-NCREE-NTU	Test 5D	3.54
329	NCREE cyclic	Test 7-2	N.A.
332 + 333 + 334	CU cyclic	Test 7-1	5.72

⁵ Experiment ID on the ISEE environment web site (<http://isee.ncree.gov.tw/ncree/>).

⁶ Real test carried out at the sites are listed here. The remote sites not listed here are simulated by the developed control modules.

⁷ The alphabet letters indicate the seismic hazard levels. The duration of the input ground motion are described in Section 6.2.

Table 6.5: Comparisons of force and displacement responses

		P1		P2		P3	
		X	Y	X	Y	X	Y
Analytical force	Max	258.4	169.7	754.4	898.2	204.5	240.0
	Min	-213.0	-152.9	-794.3	-760.8	-198.1	-208.2
Experimental force	Max	352.3	172.1	909.8	1052.6	203.6	229.1
	Min	-313	-163.9	-830.2	-975.7	-153.6	-225.8
Error	Max	26.7%	1.4%	17.1%	14.7%	0.4%	12.0%
	Min	31.9%	6.7%	4.3%	11.8%	29.0%	42.3%
Analytical displacement	Max	27.2	12.6	52.8	65.5	26.6	27.2
	Min	-24.4	-14.6	-49.9	-63.7	-24.8	-26.9
Experimental displacement	Max	21.8	10.8	43.5	53.1	21.9	26.9
	Min	-19.2	-9.4	-38.9	-47.6	-19.5	-23.4
Error	Max	24.6%	16.2%	21.3%	23.4%	21.4%	1.3%
	Min	27.1%	55.7%	28.6%	33.8%	27.3%	15.4%

Figure 6.1: Network configuration

(a) P1 at Carleton University

(b) P2 at National Center for Research on Earthquake Engineering

(c) P3 at National Taiwan University

Figure 6.2: Experimental results of shear forces versus displacements for the 50% probability of exceedance in 50 years hazard level (Test A)

(a) P1 at Carleton University

(b) P2 at National Center for Research on Earthquake Engineering

(c) P3 at National Taiwan University

Figure 6.3: Experimental results of shear forces versus displacements for the 10% probability of exceedance in 50 years hazard level (Test B)

(a) P1 at Carleton University

(b) P2 at National Center for Research on Earthquake Engineering

(c) P3 at National Taiwan University

Figure 6.4: Experimental results of shear forces versus displacements for the 2% probability of exceedance in 50 years hazard level (TestC)

(a) P1 at Carleton University

(b) P2 at National Center for Research on Earthquake Engineering

(c) P3 at National Taiwan University

Figure 6.5: Experimental results of shear forces versus displacements for the 10% probability of exceedance in 50 years hazard level (aftershock, Test D)

(a) P1 at Carleton University

(b) P2 at National Center for Research on Earthquake Engineering

(c) P3 at National Taiwan University

Figure 6.6: Comparison of analytical and experimental results for Test A

(a) P1 at Carleton University

(b) P2 at National Center for Research on Earthquake Engineering

(c) P3 at National Taiwan University

Figure 6.7: Comparison of analytical and experimental results for Test B

(a) P1 at Carleton University

(b) P2 at National Center for Research on Earthquake Engineering

(c) P3 at National Taiwan University

Figure 6.8: Comparison of analytical and experimental results for Test C

(a) P1 at Carleton University

(b) P2 at National Center for Research on Earthquake Engineering

(c) P3 at National Taiwan University

Figure 6.9: Comparison of analytical and experimental results for Test D

Figure 6.10: Comparison of analytical and experimental displacement responses in the X-direction (Test A)

Figure 6.11: Comparison of analytical and experimental displacement responses in the Y-direction (Test A)

Figure 6.12: Comparison of analytical and experimental displacement responses in the X-direction (Test B)

Figure 6.13: Comparison of analytical and experimental displacement responses in the Y-direction (Test B)

Figure 6.14: Comparison of analytical and experimental displacement responses in the X-direction (Test C)

Figure 6.15: Comparison of analytical and experimental displacement responses in the Y-direction (Test C)

Figure 6.16: Comparison of analytical and experimental displacement responses in the X-direction (Test D)

Figure 6.17: Comparison of analytical and experimental displacement responses in the Y-direction (Test D)

(a) P1 at CU

(b) P2 at NCREC

(c) P3 at NTU

(d) P4 (simulated model)

Figure 6.18: Displacement loops of the bridge piers

(a) Cyclic loading pattern in the X-direction

(b) Cyclic loading pattern in the Y-direction

Figure 6.19: Cyclic loading patterns

Figure 6.20: Failure of anchor bolt at the weld

Figure 6.21: Crack at the base of the footing

Figure 6.22: Local buckling of the exterior steel tube during the cyclic loading

Figure 6.23: Hysteresis loop of test specimen under cyclic loading at Carleton University

Figure 6.24: Network communication time per time step

Figure 6.25: Breakdown of network communication time

CHAPTER 7

7 CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary

It is difficult to perform tests on large scale structural systems due to the physical size of large test specimens and limitation of facilities and the high cost. The substructure testing technique can overcome some of these difficulties. The advantage of this testing method is that only the portion of the structure of interest or importance, such as structural members that suffer damage or have complex nonlinearly behaviour are physically tested while the rest of the structure is computationally simulated. To overcome the limitation of test equipment and resources at any single laboratory, the concept of networked multi-site hybrid testing has been developed to allow the testing of large scale structural systems by sub-structuring the test structure into substructures and each substructure can be tested at a separate remote laboratory site coordinated with all the other geographically distributed multi-sites through the internet. Because of the complexity and differences in operating characteristics of different testing equipment and facilities, there is the need of a standardized platform for test equipment control and easy set-up of network based collaborative multi-site hybrid tests, and standardized protocol procedure for efficient data exchange. Efficient coordination and management of the instrumentation and data acquisition processes operating under different computing environments are important requirements in the design of collaborative multi-site hybrid tests. The collaborative research project between NCREE, National Taiwan University,

Taiwan and Carleton University, Canada was established to develop a standardized procedure for laboratories to join in Internet-based hybrid experimental research. A four-pier bridge system was chosen as the prototype structure for a collaborative experiment to evaluate the performance of the developed procedure implemented in the Internet-based simulation for earthquake engineering (ISEE) environment for cross-continent multi-site collaborative structural tests. Three physical substructures, the double-skin concrete filled tubular piers, were constructed and loaded pseudo-dynamically at the participating laboratories while the fourth pier was simulated by computer model.

Modules for control and instrumentation were developed using LabVIEW, and these control modules can be selected and assembled into specific control tool for different test equipment. PISA3D was successfully implemented in the Analysis Engine of the ISEE environment for this collaborative multi-site experiment. Three analytical models were constructed to predict the nonlinear behaviour of the DSCFT columns: a PISA3D model, an OpenSees fiber layer model, and a model using the Simplified Approach Method developed at Carleton University. The Simplified Method model correlates well with the OpenSees fiber layer model. The analytical hysteretic behaviour of the DSCFT column by the OpenSees fiber layer model agrees reasonable well with the experimental result. The first cross-continent network pseudo-dynamic experiment on large scale test specimens has been successfully carried out using the developed control modules. This is a new research field with potential significant impact on Earthquake Engineering Research. This research project successfully combined Information Technology and Structural Engineering research for large scale testing of a complex structural system.

7.2 Conclusions

The main conclusions of this research are outlined below:

- The developed control modules can be easily assembled into specific control tool for different test equipment at different remote site for a collaborative multi-site experiment
- For this collaborative multi-site experiment, the structural analysis programs, OpenSees and PISA3D, were implemented in the platform to carry out the time-history analysis during the network experiment, which demonstrated the versatility of the ISEE environment. Different analysis tools can be implemented into the ISEE environment for simulation and computation of the interactions expected between the sub-components tested at participating laboratories according to the information received from the remote sites.
- Network stability and performance are the key factors for a collaborative multi-site experiment. The network stability refers to the network connection and the network performance refers to the speed for data transmission. Since pseudo-dynamic experiment is loaded at a slow rate, the network performance is not as critical as the network stability. No interruption has occurred throughout the series of multi-site testing; however, the network performance is not as satisfactory as the network stability. Even though the average elapsed time per time step varies from 3.05 seconds to 4.12 seconds of the tests, it is found that the network communication time took above 5 seconds at some instances, and up to 8.5 seconds at one instance during the experiment. End-to-end lightpath

between laboratories can be used to increase the speed of data transfer, the security of the network, and network stability and performance.

- Three analytical models were constructed to estimate the possible peak responses of the sub-structural components tested at three remote sites: a PISA3D model, a fiber-layer OpenSees model, and an OpenSees model using the Simplified Approach developed at Carleton University. The Simplified Approach model correlates reasonable well with the OpenSees model; however, it is noted that the simplified approach model tends to become unstable after steel yields.
- The fiber-layer OpenSees model gives more accurate predictions comparing to the experimental results than the other two models. It is found that the PISA3D model overestimates the strength of the three test specimens.
- Local buckling of the external steel tube was observed of the test specimen at Carleton University. The local buckling occurred due to de-bonding between the steel tube and the filled concrete. No crack was observed on the confined concrete core at the cut section, 1 inch above the bulge, which shows that the double-skinned steel tubes are efficient for confinement.

7.3 Recommendations

Modules for equipment control developed for this collaborative multi-site experiment are written in LabVIEW. NEESgrid has also developed and implemented similar software tools and modules using LabVIEW for carrying out multi-site hybrid tests. Further research on the compatibility issues of collaborative research and framework for

coordination and implementation and sharing of distributed resources and facilities is necessary.

Large discrepancies between the analytical and experimental results are observed for the scenario of seismic hazard level equivalent to 2% probability of exceedance in 50 years. The PISA3D and Simplified Approach Method are both new analytical tools that can be improved on the precision of numerical modeling.

It is observed that the supporting frames at Carleton University Structure Laboratory deformed during the pseudo-dynamic experiment, which lead to reduction in the actuator travel range. To improve the structural engineering laboratory testing facility at Carleton University, a brace frame can be added to increase the strength of the supporting frame. Sliding and rocking of supporting frames and the specimen foundation are also observed during the experiment. After post-tensioning system is installed for the cyclic loading test, the sliding and rocking are eliminated. Post-tensioning anchor bolts of the supporting frame and the test specimen foundation can be used as a standard experimental setup procedure in the structural laboratory to prevent rocking and sliding during an experiment.

For reference in the construction and assemblage of the DSCFT test specimen at Carleton University, assemblage the reinforcing bars was difficult because of the shapes of the reinforcing bars with 135° bent at both ends. For the rectangular and circular reinforcing bars, it is recommended to use 90° bent for easier assemblage, and the reinforcing bars

can be welded at a few locations to increase the resistance against pull-out of reinforcing bars. It is also recommended to use 180° bent for the vertical reinforcing bars of the circular reinforcing cage to reduce the area required when inserting the circular reinforcing cage in to the main reinforcing cage.

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Appendix A: Brief Descriptions of LabVIEW Control Program

(1) Modules for Network Security

The screenshot shows a LabVIEW control panel with several input fields and a status indicator. At the top, there are two text boxes labeled 'Experimental Site' and 'Experimental Name'. Below these are two more text boxes labeled 'Date' and 'Time'. In the center, there is a circular indicator labeled 'IP connect'. At the bottom, there are two more text boxes labeled 'SQL Host Name' and 'SQL IP Address'.

Experiment Site & Name: inputs in these fields will be saved as the headings of the recorded data during the experiment (it will not affect the progress of the experiment if left empty).

Date & Time: Date and time of the experiment (this field is automatically generated).

SQL host Name: The corresponding IP Address of the Data Center (for example, 61.56.0.100 is the IP address of “isee.ncree.gov.tw”).

IP connect: Shows the status of the network connection with the Data Center (it is set to ping SQL IP Address every 5sec). Green light indicates that the connection is established, and red light indicates that the connection to the SQL host is failed.

SQL Database Login			
SQL Database Login			
SQL IP Address	61.56.0.100	Exp ID	239
		Lab ID	1
SQL Get CH			
Get CH Name	0	SQL Cmd1	SQL Cmd2
Get CH Num	2	Get CH Array	0
		1	2
SQL Put CH			
Put CH Name	2	SQL Force1	SQL Force2
OSIG Num	2		
OSIG Num	0		

SQL Database Login

SQL IP Address: SQL Server IP Address (61.56.0.100)

Exp ID & Lab ID: enter the corresponding Experiment ID and Lab ID set at the following website: “<http://isee.ncree.gov.tw/ncree/>” or “<http://ipcse.ncree.gov.tw/ncree/>”.

These information can be obtained under the “Condition” of each experiment.

SQL Get CH (Information to be received during the progress of the experiment)

Get CH Num: The number of channels that the corresponding laboratory expected to receive from the SQL host (for example, the Lab ID 1 is expected to receive two displacement commends during at each integration step, enter “2” in this field).

Get CH Name: name the channels to be received from the SQL host. These names will show up as the headings in the plots and in the data files saved during the experiment (for example, SQL Command1 and SQL Command2).

Get CH Array: indicates the corresponding get channel numbers in the SQL analysis array. Not an essential information for the FCM, but can be used as a reference for the user.

SQL Put CH (Feed-back information during the progress of the experiment)

Put CH Num: the number of channels/ data to be sent back to the SQL server (includes the total number of “Open Signals” and “Critical Signals”).

CSIG Num: the total number of “Critical Signal” channels (mainly, the number of restoring forces to be sent back to the SQL server).

OSIG Num: the total number of “Open Signal” channels (strain gauge, LVDT, etc.)

Put CH Name: name the channels to be sent to the SQL host. These names will show up as the headings in the plots and in the data files saved during the experiment (for this experiment: SQL Force1 and SQL Force2).

Put CH Array: indicates the corresponding “put channels” in the SQL analysis array. Not an essential information for the FCM, but can be used as a reference for the user.

OSIG Unit: the units for the corresponding “Open Signal” channels (this will be saved in the data files during the experiment).

Experimental Information					
Analysis Duration	50.00	Test Points	2500	Analysis Time Interval	0.02

Analysis Duration: the length of the input ground motion in seconds.

Test Points: the total number of integration points.

Analysis Time Interval: the integration time interval.

Note: Analysis Duration = “Test Points” multiply the “Analysis Time Interval.” An entry error in this field may result in early termination of the experiment or unable to finish the experiment.

Running State

Emergency Stop HOLD Cont.

Using Status: Not Connect Yet

Connect: Click on connect button to connect to the SQL Server after the setup options are defined.

Emergency Stop: To stop the test.

Hold: To put the test on hold. The test can be resumed by pressing this button again.

Cont.: Whether or not to continue the external control command. This feature is only used when the “Hold for 80%” or “Baby step” is selected.

(2) Modules for equipment setup

Experimental Setup

Command Control Mode: Ext. Tempo

Act Num: 2 Act Name: 0 Act 1 Act 2

Ext Tempo Num: 2 Ext Tempo Name: 0 Tempo 1 Tempo 2

Ext Loadcell Num: 1 Ext Loadcell Name: 0 Axial

Act Length (mm): 0 3230.00 3150.00 Specimen Length (mm): 3220.00

Tempo Length (mm): 0 1000.00 1000.00

Command Control Mode: internal control (based on the actuator displacements) or external control (used the external potentiometer).

Act Num: the number of actuators to be used during the experiment.

Act Name: naming the actuators (these will be saved as headings in the data files during the experiment. Act 1, in the x direction, and Act 2, in the y direction, are used for this experiment).

Ext Tempo Num: the number of external potentiometers to be used for the control of experiment (Temposonic or LVDT).

Ext Tempo Name: naming the ext. tempo (will be saved as headings in the data files during the experiment.).

Ext Loadcell Num: total number of external load cell.

Ext Loadcell Name: naming the external load cell (will be saved as headings in the data files during the experiment. For example, it is named “Axial” for this experiment).

Actuator Length (mm): the distance between the two hinges of an actuator. This is required in the calculation of coordinate transformation.

Specimen Length (mm): the distance between the top of footing to the central point of the actuator (after installation). This is required in the calculation of coordinate transformation.

Tempo Length (mm): the distance between the test specimen and the point of where the external potentiometer is fixed to the support. This is required in the calculation of coordinate transformation.

Experimental Option	
Resume the previous experiment?	No
Transfer the coordinate (Global <-> Local)?	No
Return the "critical response signals"?	Yes
Send the "open signals" acquired by the datalog?	No
Plot the chart during the experiment?	Yes
Show the "critical signal" data list during the experiment?	No
Show the "open signal" data list during the experiment?	No
Hold for each 80% step or baby step?	No

Resume the previous experiment? Whether or not to resume the test with the continuum of the input ground motion of the previous test. If the test specimen has been pushed beyond the elastic state, the current readings of the test specimen, which will be used as the initial conditions, will not be zero when starting the next test. Since the absolute zero position should not change during the course of the entire experiment, the user can either type in the initial conditions recorded at the beginning of the experiment or retrieve the data from the "Offset File." The following figure shows the setting of the initial conditions of the test specimen (i.e., the corresponding forces and displacements readings at the beginning of the experiment).

Transfer the coordinate (Global <-> Local)? It is an option to perform coordinate transformation between SQL Displacement/Force and Actuator Displacement/Force.

Return the "critical response signals"? ("Yes" – Pseudo-Dynamic Test; "No" - Quasi-Static Cyclic Test.)

Send the "open signals" acquired by the datalog? (Send Open Signals from the DAQ to the SQL Server)

Hold for each 80% or baby step? : Whether or not to hold the test when 80% of the target displacement (or at the end of each predefined step) is reached.

(3) Modules for data management

File Path & Save Type

File Directory: f:\network\2006\ Save Type: Overwrite

Save the CSIG Test file I? Yes

Save the CSIG Test file II? No

Save the OSIG Test file? No

Save the Time file? Yes

Save the Offset file? Yes

Time Interval: 1.00

File Directory: it has to be an existing folder created in the directory.

Save Type: the two options are Overwrite or Append.

Note: the data saved in the CSIG Test file I contain the only the “Critical Signals” during the experiment (i.e., the forces and displacements for the control). In the CSIG Test file II, all “Critical Signals” are saved during the experiment. It is suggested to save the following files: CSIG I, OSIG (if there are predefined “Open Signals”), “Time” and “Offset.”

(4) Modules for equipment control

Limit Detect

Limit Detect: []

Cmd Upper Limit: 0 5.00 5.00

Cmd Lower Limit: 0 -5.00 -5.00

Limit Detect: set Limits of the target commands (After coordinate transformation). (On : turn on limit detection; Off : turn off limit detection).

Cmd Upper Limit: upper limit of the expected target command during the experiment (after coordinate transformation).

Cmd Lower Limit: lower limit of the expected target command during the experiment (after coordinate transformation).

If the “Command” goes over either the lower limit or the upper limit, the following message will appear on the screen: “At least one command triggered the limit detect. Do you want to keep the experiment running and correct the limit boundaries?” Selecting “YES” will allow the user to change the upper and lower limits, and then the experiment can proceed as usual. Select “NO”, the experiment is terminated but the hydraulic pumps are still “ON.”

Control Parameter & Algorithm

Analog I/O Channels	
Command Output Channels	0,1
Act Disp Input Channels	0,1
Act Force Input Channels	2,3
Ext Tempo Channels	4,5
Ext Loadcell Input Channels	6

NI-DAQ	NI Device
☒ On	1

NI-DAQ:

On: Using the readings from DAQ Card to proceed with the test or simulation. A NI-DAQ card has to be installed when this option is selected, otherwise, an error message will appear on the screen.

Off: The response of the test specimen (force-displacement relationship) is predefined for simulation test.

NI Device: Enter the Device Number set in the Measurement & Automation for E-series NI-DAQ card. Command Output Channels, Act Disp Input Channels, Act Force Input Channels, Ext Tempo Channels, Ext Loadcell Input Channels: The corresponding channel numbers on the BNC Channel Box.

For the M and S-series, the physical channels are selected instead of device number.

Control Parameter & Algorithm

Voltage & Range		
Act Disp Voltage (+/-V)		
0	10.00	10.00
Act Disp Range (+/-mm)		
0	125.00	125.00
Act Force Voltage (+/-V)		
0	10.00	10.00
Act Force Range (+/-kN)		
0	125.00	125.00
Ext Tempo Voltage (+/-V)		
0	10.00	10.00
Ext Tempo Range (+/-mm)		
0	100.00	100.00
Ext Loadcell Voltage (+/-V)		
0	5.00	0.00
Ext Loadcell Range (+/-kN)		
0	900.00	0.00

Voltage (+/-V): The voltage range of the measuring devices.

Range (+/- Engineering Unit): The corresponding physical range of the measuring devices.

Control Parameter & Algorithm

Simulation/Real Test

Act Force Fbk Type
Multiply the Stiffness

Specimen Stiffness (kN/mm)
0 10.00 10.00

Ext Loadcell Fbk Type
Designate the Value

Ext Loadcell (kN)
0 0.00 0.00

Act Force Fbk Type: The two available options are “Real Fbk Signal” and “Multiply the Stiffness.”

- (1) Real Fbk Signal: get the readings of forces from the MTS system
- (2) Multiply the Stiffness: the forces are calculated by the displacement commands multiply by a constant stiffness.

Specimen Stiffness (kN/mm): Predefined stiffness of the test specimen when “Multiply the Stiffness” is selected as the Act Force Fbk Type.

Ext Loadcell Fbk Type:

- (1) Real Fbk Signal: get readings from the actual measuring devices.
- (2) Designate the Value: using a predefined value (mainly for simulation test of the program).

Ext Loadcell (kN): The axial load to be applied on the test specimen during the experiment.

Control Parameter & Algorithm

Ramp Control Mode	
segments/step 5	Control Algorithm 80%
sec/segment 0.10	Ramp Points 100
Error Tolerance (mm) 0.20	Ramp Rate (mm/sec) 5.00
Cycles Limit 100	Baby Step (mm/step) 1.00
Ending Control Mode Stay at level	

segment/step: The displacement command is divided into the number of segments to be executed by actuators at each step.

Error Tolerance: To check whether or not the difference between the command and the feedback signal is within the error tolerance set.

Cycles Limit: Set the maximum number of cycles for each time step. If the error tolerance is not achieved within the cyclic limit, the program will terminate the current loop and proceed to the next step of execution.

Control Algorithm: This option is for external control only. The user can select either to use 80% control (for each target displacement, the actuators will push to the 80% of difference between the target displacement and the measured position of the test specimen) or the Baby Step Control (predefined increment of displacement to be executed at each step to reach the target displacement).

Ramp Points: number of ramps to be executed for each step to reach the target displacement. To avoid overshooting of actuators during the test, it is advised to set the

ramp point between 20 and 50. If the ramp point is set too high, it will increase the execution time for each step.

Ramp Rate (mm/sec): The control speed for each rate in mm/sec.

Baby Step: If the “Baby Step Control” is selected as the Control Algorithm. Define the displacement to be executed at each step.

Ending Control Mode: Defines the position of actuators at the end of each test.

Stay at level: Hold the test specimen at the last position of execution of the test.

Taper to Zero: Move the test specimen back to the initial zero position.

It is suggest to choose the “Stay at level” to avoid danger.

Control Parameter & Algorithm

Digital I/O & Datalog Trigger		
Trigger	Trigger Voltage (V)	Pulse Time (sec)
<input type="button" value="Off"/>	<input type="text" value="5"/>	<input type="text" value="0.10"/>
Digital Output Channels (Trigger)		Dwell Time (sec)
<input type="text" value="0"/>		<input type="text" value="0.50"/>

Trigger: Using trigger for DAQ to collect “Open Signals.”

Digital Output Channels (Trigger): The digital output channel on the BNC Channel Box.

Dwell Time (sec): The minimum time to scan all channels.

(5) Modules for real-time display

(a) Displays the progress of the experiment

Elapsed Time (sec): the total elapsed time of the test.

Progress Percentage (%): the progress of the test in percentage (this is calculated based on the Analysis Duration, Test Points, and Analysis Time Interval entered in the fields described in the previous sections).

(b) Displays the information of control channles

Digital: shows the commands and feedbacks of all the channels for control during the test.

(c) Displays the offset information

The Offset window displays a table of offset information for various channels. At the top, there are buttons for 'Offset' and 'Offset Lock' (which is currently locked). Below the buttons, the table is organized into two rows of eight columns each. The first row shows the current offset values for 'Act Disp (mm)', 'Act Disp (Volt)', 'Act Force (kN)', 'Act Force (Volt)', 'Ext Tempo (mm)', 'Ext Tempo (Volt)', 'Ext Loadcell (kN)', and 'Ext Loadcell (Volt)'. The second row shows the 'Offset' values for the same channels, all of which are currently set to 0.00.

Act Disp (mm)	Act Disp (Volt)	Act Force (kN)	Act Force (Volt)	Ext Tempo (mm)	Ext Tempo (Volt)	Ext Loadcell (kN)	Ext Loadcell (Volt)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Offset: Offsets the initial readings from all channels to zero at the beginning of the test.

The original readings are saved in the “Offset File.”

Offset Lock: Once the network connection is established and the test is in process, the “Offset” button will be locked.

(d) Real-time plots of the selected channels

Chart: Plots the selected channels during a test.

Channel: Currently, the channels defined in the program are the SQL commands (6 channels), displacement of actuators (6 channels), external potentiometers (4 channels) and external load cells for axial loads (4 channels).

Mode: None, Time, Cmd (SQL command), Displacement, Cmd & Fbk Error (difference between SQL command and feedback), Force.

Unit: None, sec., mm, kN.

(e) Error Chart (errors between commands and actual measurements)

Channel: Act 1~6 (6 channels for actuators), Ext Tempo 1~4 (4 channels for external measuring devices)

Time Interval: The time interval to refresh the plots.

Appendix B: Data Communication Loops of the Control Program

Main Data Communication Loops								
Path (stack case sequence number)								Description
.0								Definition of all variables in program
1								Allocate default values for all variables.
2	0							Define of the full ranges of all necessary channels, and calculate the A/D conversion ratios.
	1							Transmit zero signals to command channels (for control safety).
	2							Define paths of files.
	3							Save information of experimental site in log file.
	4							Save column and row headers in CSIG and OSIG tables. Save column headers in CSIG I, CSIG II and OSIG files.
	5							Save column header in time file.
	6							Save column header in offset file.
	7							Boolean value for recording

								time history charts (for comparison of errors between commands and feedbacks).
	8							Boolean value for resuming the previous test (True) or starting a new test (False).
		0						Dialog message for user to choose a previous offset file.
		1						Allocate an offset file.
		2						Save offset values of actuator displacements and forces.
		3						Save offset values of external measuring devices.
		4						Lock the functionality of offset button.
	9							Boolean value for ready or not to connect to SQL server.
3	0							Read IP address, experiment ID and lab ID for the connection with SQL database server.
	1							Set the number and array for get channels and put channels.
	2							Connect to SQL server.
	3							Check with SQL server if

							all clients have connected to the server.
	4	0					Save current offset values in offset file and Lock the functionality of offset button (for control safety).
		Seq.1 NIDAQ (True)	0				Define the full ranges of all necessary channels and calculate the A/D conversion ratios.
			1				Set initial zeros. Transmit zero signals to AO command channels (for control safety at the beginning of the experiment).
			2~5				Read AI channels minus initial offset values.
			6				Set the previous readings as Initial Act Cmd, Prior Act Cmd, and Prior Cmd
		NIDAQ (False)	0				Initial conditions (zeros all commands and readings)
			1				Boolean value for saving signals in CSIG II file.
	5	0					Using status to show the process of the experiment.
		1	0				Record the current time as Check initial time (T1).
			1				Internet time (T1-T10) =

								Check initial time (T1) – Put end time (T10)
			2					Check for updates of the SQL database on website.
			3					Record the current time as Checkend time (T2).
			4					Internet time (T2-T1) = Checkend time (T2) – Check initial time (T1)
		2	0					Get SQL Cmd from SQL server.
			1					Record the current time as Get end time (T3).
			2					Get time (T3-T2) = Get end time (T3) – Check end time (T2)
		3						Coordinate transformation from the global SQL Cmd to local Cmd.
		4						Limit detection of Cmd.
		5	0					Record the current time as Hold initial time (T4).
			1					Check if the HOLD button is pushed or not.
			2					Record the current time as Hold end time (T5).
			3					Hold time (T5-T4) = Hold end time (T5) – Hold initial time (T4)

		6						Control and data processing.
		NIDAQ (True)	0					Record the current time as Control initial time (T6).
			1					Command Control Mode (Case 0: Internal displacement control; Case 1: External temp. control)
				Case 0	0			Calculate the stroke per segment.
					1	0		Calculate the increment command for actuators. (segment/step)
						1		Calculate the actuator commands (Act Cmd 1) for AO channels.
						2~5		Read AI channels minus initial offset values.
						6		Prior Act Cmd = Act Cmd = Act Cmd 1
					2			Prior Cmd (next step) = Cmd (current step)
				Case 1				Control Algorithm (Case 0: 80% control; Case 1: Baby step control)
					Case 0	0		Calculate the 80% command for comparing with Tempo reading.
						1	0	Calculate the increment command for actuators.

								(segment/step)
							1	Calculate the actuator commands (Act Cmd 1) for AO channels.
							2~5	Read AI channels minus initial offset values.
							6	Prior Act Cmd = Act Cmd = Act Cmd 1
							7	Boolean value for holding the test at each 80% step or not.
						2		Check if either the error tolerance or the cycle limit is satisfied.
					Case 1	0		Calculate the baby step command for comparing with Tempo reading.
						1	0	Calculate the actuator commands (Act Cmd 1) for AO channels.
							1~4	Read AI channels minus initial offset values.
							5	Prior Act Cmd = Act Cmd = Act Cmd 1
							6	Boolean value for holding at each baby step or not.
						2		Check if the error tolerance is satisfied.
			2					Record the current time as Ramp end time (T7).

			3					Ramp time (T7-T6) = Ramp end time (T7) – Control initial time (T6)
				0				Act Cmd 1 for AO channels.
				1~4				Read AI channels minus initial offset values.
				5				Check if either the error tolerance or the cycle limit is reached. Record the iteration number.
			5					Act Disp = Act Disp 1 Act Force = Act Force 1 Ext Tempo = Ext Tempo 1 Ext Loadcell = Ext Loadcell 1
			6					Record the current time as Iteration end time (T8).
			7					Iteration time (T8-T7) = Iteration end time (T8) – Ramp end time (T7) Control time (T8-T6) = Iteration end time (T8) – Control initial time (T6)
		NIDAQ (False)						Cmd = Prior Cmd = Prior Act Cmd = Act Cmd = Act Cmd 1= Act Disp 1 = Ext Tempo = Ext Tempo 1 Act Force = Act Force 1 = Cmd x Specimen Stiffness

								Ext Loadcell= Ext Loadcell 1
		7	0					Coordinate transformation from local Act Force to global SQL Force.
			1					Record the current time as Put initial time (T9).
			2					Put SQL force to SQL database.
			3					Record the current time as Put end time (T10).
			4					Put time (T10-T9) = Put end time (T10) – Put initial time (T9)
		8	0					Calculate the Disp Error between Cmd and Act Disp/Ext Tempo.
			1					Build a data array which contains elapsed time, current time, all readings, command, SOL put and get channel signals.
		9						Trigger data logger.
		10						Acquire the OSIG readings from the DAQ program. (Reserved type)
		11						Record all necessary signals in CSIG Data Table.
		12						Record all necessary signals in OSIG Data

							Table.
		13					Chart 1 plot.
		14					Chart 2 plot.
		15					Save the built data array in CSIG I file.
		16					Save the OSIG readings in CSIG I file.
		17	0				Record the current time as End time and calculate the next time step.
			1				Elapsed time = End time – Initial elapsed time
			2				Progress Percentage = Current Time/Analysis Duration
		18	0				Record the current time as Process end time (T11).
			1				Calculate the processing time.
		19					Save the time data in Time file.
	6	0					Using status to show that the experiment is completed.
		1					Ending Control Mode (Case 0: Stay at level; Case 1: Taper to zero)
		NIDAQ (True)	Case 0				Nothing to do and stay at level.

			Case 1	0				Transmit a command of Initial Act Cmd to AO channels.
				1~4				Read AI channels minus initial offset values.
		NIDAQ (False)	Case 0					Nothing to do and stay at level.
			Case 1					Zero signals for all commands and feedback channels.

Connection Detection Loop	
Path	Description
	Make sure if the connection between FCM and SQL server can work.

CSIG file II Loop			
Path		Description	
Case True	Seq. 0		Boolean value for starting or not to save CSIG signals in each time interval.
	Seq. 1		Command Control Mode.
	Seq. 2	Seq. 0	Build a data array which contains current time, all readings, commands, SOL put and get channel signals.
		Seq. 1	Save the built data array in CSIG II file.
Case False			Nothing to do.

Time History Chart Loop			
Path			Description
Case True	Seq. 0		Boolean value for starting or not to record time history charts. (For comparison of errors between commands and feedbacks)
	Seq. 1		Set the properties of charts. (such as offsets, scales, and ...)
	Seq. 2	Seq. 0	Command Control Mode. (0: Act Cmd 1; 1: Prior Cmd)
		Seq. 1	Plot the chart 1 contains commands, feedbacks, and errors.
		Seq. 2	Plot the chart 2 contains commands, feedbacks, and errors.

Offset Loop			
Path			Description
Seq. 0			Boolean value (Offset, NI-DAQ, and Offset Lock)
	Case True		Trigger the functionality of Offset. (Disp, Force, Tempo, and Loadcell)
	Case False		Command Control Mode. (0: Act Cmd 1; 1: Prior Cmd)
Seq. 1	Case True	Seq. 0~3	Read AI channels minus initial offset values. (Including Act disp and force, external tempsonic and loadcell)
	Case False		Nothing to do.

Appendix C: Design Drawings of Test Specimens

Figure C. 1: Elevation view of the test specimen at CU (P1)

Figure C.2: Elevation view of the DS of test specimen at CU (P1)

Figure C.3: Design drawing of reinforcement for the footing of the test specimen at CU (P1)

Figure C.4: Design drawing of the DS Column of the test specimen at NCREE (P2)

Unit : mm

Figure C.5: Design drawing of the reinforcement for the footing of the test specimen at NCREE (P2)

Appendix D: Analytical Results of Force-Time Histories

Figure D.1: Comparison of analytical base shear force responses for P1 (Test A)

Figure D.2: Comparison of analytical base shear force responses for P2 (Test A)

Figure D.3: Comparison of analytical base shear force responses for P3 (Test A)

Figure D.4: Comparison of analytical base shear force responses for P4 (Test A)

Figure D.5: Comparison of analytical base shear force responses for P1 (Test B)

Figure D.6: Comparison of analytical base shear force responses for P2 (Test B)

Figure D.7: Comparison of analytical base shear force responses for P3 (Test B)

Figure D.8: Comparison of analytical base shear force responses for P4 (Test B)

Figure D.9: Comparison of analytical base shear force responses for P1 (Test C)

Figure D.10: Comparison of analytical base shear force responses for P2 (Test C)

Figure D.11: Comparison of analytical base shear force responses for P3 (Test C)

Figure D.12: Comparison of analytical base shear force responses for P4 (Test C)

Figure D.13: Comparison of analytical base shear force responses for P1 (Test D)

Figure D.14: Comparison of analytical base shear force responses for P2 (Test D)

Figure D.15: Comparison of analytical base shear force responses for P3 (Test D)

Figure D.16: Comparison of analytical base shear force responses for P4 (Test D)

Appendix E: Analytical vs Experimental Force-Time Histories

Figure E.1: Comparison of analytical and experimental force responses of P1 (Test A)

Figure E.2: Comparison of analytical and experimental force responses of P2 (Test A)

Figure E.3: Comparison of analytical and experimental force responses of P3 (Test A)

Figure E.4: Comparison of analytical and experimental force responses of P1 (Test B)

Figure E.5: Comparison of analytical and experimental force responses of P2 (Test B)

Figure E.6: Comparison of analytical and experimental force responses of P3 (Test B)

Figure E.7: Comparison of analytical and experimental force responses of P1 (Test C)

Figure E.8: Comparison of analytical and experimental force responses of P2 (Test C)

Figure E.9: Comparison of analytical and experimental force responses of P3 (Test C)

Figure E.10: Comparison of analytical and experimental force responses of P1 (Test D)

Figure E.11: Comparison of analytical and experimental force responses of P2 (Test D)

Figure E.12: Comparison of analytical and experimental force responses of P3 (Test D)