A Hybrid Simulation Based Method for Post Seismic Structural Health Monitoring of Concrete Bridges

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This dissertation is dedicated to my parents and sister for their never ending love, prayers and supports.
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SUMMARY

The present study reports on development of a hybrid simulation based method for postearthquake structural health monitoring of concrete bridges. Linking between filed and laboratory facilities through a limited number of sensors, the proposed method provides the possibility to monitor the behavior of the most vulnerable components of a bridge structure more comprehensively.

To investigate the feasibility of the introduced methodology, an experimental program was planned to perform the hybrid simulations on 1/10, 1/8 and 1/6 scaled models of a two-span bridge subjected to progressively increasing amplitudes of seismic motions. Hybrid simulation is an experimental testing methodology performed on a hybrid model which is composed of scaled physical and numerical components of a structural system. The experimental and analytical portions incorporated into a single model by compelling the displacement compatibility and the force equilibrium at the common nodes. All the components of the bridge structure studied in this research, except one column, were simulated numerically in the analytical part of the hybrid models. The physical part of the hybrid simulations consists of well confined reinforced concrete columns instrumented with an array of surface adhered and embedded fiber optic sensors along the diameter of the circular cross section in the plastic hinge zone. The results of shaking table tests on a 1/4 scaled model of the bridge were used to study the accuracy and reliability of different scales of the hybrid simulations. The data obtained from the sensors were employed for analysis of damage in the bridge columns. The developed embedded sensors along with the external sensors were used to picture the redistribution of strains in the cross section of plastic hinge area.
Performance assessment of bridge structures equipped by structural health monitoring systems requires the development of deterministic methodologies producing quantitative information on different damage states. A novel methodology is developed to evaluate the energy dissipation in a reinforced concrete column with circular cross section based on the curvatures measured in the plastic hinge area. Dissipated energy is a sensitive quantity to monitor the progression of damage in structures. Integrating the introduced method in a normalized dissipated energy index provides the possibility to detect and quantify minor and moderate damages which are barely visible during visual inspection.
I. Introduction

1.1 Background

Bridges form crucial links in the transportation network especially in the aftermath of earthquakes. The bridges built in the seismic zones experience various types of ground motions and shaking intensities in their life time. Currently the post-earthquake damage evaluation and capacity assessment of the bridges are based on visual inspection. In order to properly assess the condition of the bridges it is essential to develop methodologies that go beyond visual inspections (Buckle, 1994). Consequently, structural health monitoring (SHM) is emerging in response to the need for post-disaster evaluation of the bridge structures. The advances in communication, computer and sensor technology provided the possibility to remotely monitor and evaluate structural performance in real time. To accomplish performance assessment of bridge structures based on the data from SHM, deterministic methodologies producing quantitative information on different damage states of the bridge structures should be developed. These methods will be necessary to establish repair strategies, and to make timely decisions pertaining to the lifelines and traffic patterns (Ansari, 2005). This aim can be achieved through defining engineering limit states which may be expressed by limiting values of quantities such as maximum strains or damage indices. The maximum strains can be continuously monitored in columns instrumented by appropriate sensors. However monitoring of damage indices incorporating forces or hysteresis energy is not always practical. There are difficulties in accurate monitoring of forces in bridges especially when they are subjected to reversed multidirectional dynamic motions during earthquakes. Thus, to avoid instrumentation for monitoring the imposed seismic forces it is more practical to introduce damage indices solely based on strain and curvature measurements. Each damage classification can be related to one or more engineering
limit states and evaluation of engineering limit states leads to assessment of the performance levels (Lehman et al. 2004) (Fig. 1.1).

Since the advancement of present-generation performance-based seismic design procedures is widely recognized as a forward step to develop resilient and loss-resistant infrastructures, SHM based structural assessment should be consistent with the principles in the performance-based seismic design. There are three sources attempting to form the foundation for performance-based design concepts: SEAOC Vision 2000 (SEAOC, 1995); ATC 40 (ATC 40, 1996); and FEMA 273 and 274 (FEMA 273 and FEMA 274, 1996). Among those, SEAOC Vision 2000 introduces a five-level performance evaluation procedure to translate the performance descriptions into engineering limit states. Hose et al (1999) determined the relationship between qualitative description of the damages in reinforced concrete bridge columns and quantification of performance parameters through the five-level performance evaluation framework.

In modern construction, the seismic performance of reinforced concrete bridge structures depends on the behavior of ductile hinge regions in the columns (Iranmanesh et al. 2008). Current design codes allow for the piers to go through inelastic deformations in order to dissipate
energy. This approach renders the columns vulnerable to large lateral displacements causing damage in the plastic hinge zones (Elnashai et al. 1989). The progression of damages starts from hair-line cracks, continues to full development of local mechanism and finally ends to buckling of steel longitudinal reinforcement and crushing of concrete core. This research focuses on the small and moderate damages and tries to quantify these damages in terms of an energy-based damage index measurable by the SHM sensors. In small and moderate states, the damage is pronounced in the form of invisible or barely visible defects such as residual crack openings, internal cracks, voids as well as de-bonding between steel and concrete.

1.2 Hybrid Simulation

Past experimental studies on the seismic response of bridges have focused on the component performance due to lack of available testing facilities that could accommodate large-scale system testing. The shake table system has made it possible to address this shortcoming. Shaking table tests provide comprehensive data on the dynamic behavior of a bridge structure to a specific ground motion in terms of global and localized damages on the response. However investigation of damage on a bridge through a shake table test is very complex and expensive due to the need for building a complete structural system with active or passive gravity load setup and limited capacity and size of most of the shaking tables.

As an alternative, hybrid simulation which combines the experimentation and computation can be used to predict the system response. The hybrid simulation method provides the capability to subdivide a large structure into subassemblies. The well understood and linear parts of the structure as well as mass and damping ratios can be modeled reliably using finite element models (Stojadinovic et al. 2006). The highly nonlinear or less understood parts of the structure can be simulated physically in the laboratory. Thus, hybrid simulation can be
considered as a modern form of system testing, which does not encounter the problems of imposing proper boundary conditions and modeling gravity loads as in shaking table tests. The experimental part of the research presented in this thesis, includes hybrid simulations on scaled models of a two-span bridge formerly tested during a shaking-table test program. The numerical part of the hybrid models includes all the components of the bridge structure except one column simulated physically in the laboratory. The physical portion of the hybrid simulations consists of a well confined reinforced concrete column instrumented with an array of fiber optic sensors along the diameter of the circular cross section in the plastic hinge zone. The tests on the hybrid models of the bridge were carried out in three scales of 1/10, 1/8 and 1/6 under earthquake excitations with peak ground accelerations increasing from 0.075g to 1.66g.

1.3 Objectives

The primary objective of this research is to develop a hybrid simulation based method for post earthquake structural health monitoring of concrete bridges. Integrating the advances in communication technology and hybrid simulation, the introduced methodology is a forward step towards utilizing the laboratory facilities in simulation based monitoring of structures. By implementing this approach, the extensive arrays of sensors required for health monitoring of a complex structure reduces to a limited number of accelerometers transferring the information of motions experienced by the structure. Bridging between field level and laboratory level, the proposed method makes it possible to monitor complex behavior components of a large structure more sophisticatedly. The architecture of the method is presented through a flowchart in Figure 1.2. The idea of the proposed structural health monitoring method is to instrument a bridge in the field with a few numbers of sensors (accelerometers) at the ground level of the bridge. The
seismic motions experienced by the bridge following each earthquake, are measured by the sensors and transmitted to a monitoring station in the laboratory. The monitoring station consists of the scaled hybrid model of the bridge in which the physical portion includes the most vulnerable parts of the bridge and the numerical part incorporates the rest of the bridge structure. The physical portion of the hybrid model is instrumented comprehensively with an array of sensors to acquire the damage status of the bridge structure. The seismic motions transmitted from the sensors in the field to the monitoring station are scaled down accordingly and applied to the bridge structure.

Figure 1.2: a hybrid simulation based method for post earthquake structural health monitoring of concrete bridges
In order to prove the feasibility of the proposed method, the components of the diagram in Fig 1.2 are studied in different chapters of the presented thesis. Development of the analytical model for determining the nonlinear response of the bridge model is discussed in chapter 4. The results pertaining to validation of the scaled hybrid models in compare to shaking table tests are presented in chapter 6 of this dissertation. One of the objectives of this thesis is to investigate the accuracy and reliability of different scales of hybrid simulation in reproducing the response of the shaking table tests. The hybrid models were developed in three scales of 1/6, 1/8 and 1/10 in order to obtain the minimum scale appropriate for mimicking the results of the shaking-table test and simulating the damage progress in the physical part of the models.

Post-earthquake analysis of damage on the scaled reinforced concrete columns including development of an energy-based damage index measurable by the structural health monitoring sensors is detailed in Chapter 7. This study focuses on small and moderate damages which are invisible or barely visible externally. The initiation and progress of damage based on the post-event redistribution of strains along the cross section of the columns in the plastic hinge area is investigated in Chapter 7.
III. Literature review

Structural health monitoring can be described as the process of determining the structural integrity by tracing the damage via an array of sensors and evaluating the performance through assessing certain engineering limit states (Chang et. al., 2003). The present dissertation reports on introduction of a structural health monitoring method for performance assessment of concrete bridges by the means of hybrid simulation procedure. The so called global and local health monitoring methods are based on the measurement of dynamic properties of the structure such as natural frequencies and mode shapes or variations in current state of structure in terms of strains and displacements (Lestari et. al., 2007). However in concrete members experiencing the repeated number of cycles as in seismic motions, dissipated energy can be selected as the key parameter sufficiently sensitive to different states of damage (Laskar et. al., 2009). Structural health monitoring of concrete bridges can take the advantage of energy dissipation for post earthquake damage assessment, provided the measurement challenges have been overcome (Bassam et. al. 2011). This chapter first reviews briefly the structural health monitoring methods for bridges, and then discusses damage indices based on hysteretic (or dissipated energy). Finally a summary on the history of development of hybrid simulation method followed by some case studies by other researchers will be presented.

2.1. Structural health monitoring methods for the bridges

Structural health monitoring methods are mostly carried out based on dynamic properties of a structure such as natural frequencies, mode shapes or damping ratios. The change in the properties of a structure such as stiffness, mass and damping causes the change in the dynamic properties. Therefore the idea is to detect the modal properties (frequencies, mode shapes and modal damping) and correlate them to the damage experienced by the structure. Frequency-
based and mode shape-based methods (Valdiver, 1977; Shi et al., 2000; Kim and Stubs, 2003; Wang and Qiao, 2007), modal strain energy-based methods (Cornwell et al, 1999) and modal flexibility-based methods (Pandy and Biswas, 1994; Wang and Qiao, 2007) are among damage detection methodologies using modal parameters. In most cases, these methods have been employed on simple structures rather than civil infrastructures. Some of the aforementioned methods have been applied to bridge structures (Farrar et al, 1994; Salawu, 1997, Xu and Wu, 2007). However a limited number of studies have been conducted on application of these methods on concrete bridges experienced earthquake excitations.

Chen et al (2008) tried to justify the methods that identify structural component stiffness degradation by pre- and post-event low amplitude vibration measurements, based on a linear time-invariant (LTI) system model. Johnson et al (2008) identified six modes of a bridge specimen tested under progressive seismic motions followed by transverse and longitudinal white-noise excitations. They used peak picking methods to identify five predominant periods from the Fourier transform of the acceleration histories. The system damping was estimated by using a logarithmic decrement method as well as ARX algorithm. Soyoz et al (2010) used the structural parameter values identified based on vibration measurement and updated in Bayesian sense for the estimation of the reliability of a bridge structure after an earthquake event. In their study, a large-scale shaking table test of a three-bent concrete bridge model was performed in order to verify the proposed reliability estimation method.

2.2. Damage assessment based on dissipated energy

Most of the damage assessment methods quantify the damage in terms of damage index which is a quantity pertaining to structural response parameters such as displacement, force or energy dissipation. The time history damage indices quantify the damage based on a structural
response parameter during a seismic event. A damage index proposed by Powell et. al. (1988) utilizes the displacement ductility ratio to assess the damage:

\[
DI_{\mu} = \frac{\delta_m - \delta_y}{\delta_u - \delta_y} = \frac{\mu - 1}{\mu_u - 1} \leq 1
\]  

(2.1)

Where \( \delta_m \) and \( \delta_y \) are the maximum displacement and yield displacement respectively and \( \delta_u \) is the maximum lateral displacement capacity under monotonically increasing lateral loading. \( \mu \) and \( \mu_u \) are the maximum and ultimate displacement ductility ratios under monotonically increasing deformations and are defined in equations 2.3 and 2.4:

\[
\mu = \frac{\delta_m}{\delta_y}
\]  

(2.2)

\[
\mu_u = \frac{\delta_u}{\delta_y}
\]  

(2.3)

However accumulation of damage reduces the ultimate displacement capacity and cyclic loading decreases \( DI_{\mu} \). The parameter which can consider the effect of cyclic loading history and accumulation of damage is dissipated energy. The seismic input energy to structural system (\( E_i \)) is expressed as:

\[
E_i = E_h + E_k + E_s + E_\xi
\]  

(2.4)

Where \( E_h \), \( E_k \), \( E_s \) and \( E_\xi \) are irrecoverable hysteretic energy, kinetic energy, recoverable elastic strain energy and viscous damping energy. Among these terms of energy, the dissipated (or hysteretic) energy (\( E_h \)) incorporates the cumulative effects of repeated inelastic cycles and is usually related to the structural damage. When the structure behaves in the elastic regime, \( E_h \) is zero. A general damage index based on the normalized dissipated energy can be written (Cosenza et al. 1993) as:

\[
DI_H = \frac{E_h}{E_{h-u}}
\]  

(2.5)
Where, $E_{h-u}$ is the hysteretic energy capacity of the system under monotonic increase of loading.

The seismic structural damage can be expressed as a combination of damage induced by both excessive deformations and that caused by repeated inelastic cycles (Park and Ang 1985):

$$D_{PA} = \frac{\delta}{\delta_u} + \frac{BE_h}{Q_y\delta_u} \leq 1$$  \hspace{1cm} (2.6)

$Q_y$ is the yield strength and $\beta$ is a constant which depends on structural characteristics of the column (Kunnath et al., 1990) and (Bozorgnia and Bertero, 2001).

It is confirmed by several researchers that the dissipated energy is the best quantity reflecting the effect of cyclic loading history on a structure (Chai et al. 1995) and (Hindi et al. 2001). In the present research it will be demonstrated that the dissipated energy is sufficiently sensitive to major states of damage before failure. However computation of the dissipated energy is a challenging issue in both analytical approach and field practice (Priestly, 2000). Other than the nonlinear finite element method, there are empirical equations proposed to estimate the amount of dissipated energy. In general, the amount of dissipated energy in a reinforced concrete member depends on several parameters such as: reinforcement arrangement, percentage of longitudinal reinforcement and geometry and size of the cross section. Therefore, the empirical equations which predict the dissipated energy of the structures based on structural and material type regardless of the design parameters are not sufficiently accurate (Lin et al. 2003). Application of dissipated energy in structural health monitoring as an indicator of damage, requires measurement of the forces which is not practical in the field. Hence, it is essential to develop a method for calculation of dissipated energy independent from force measurement to achieve field-deployable damage indices.
2.3. Development of hybrid simulation

Hybrid simulation originated from pseudo-dynamic testing method developed in early 1970s (Takanashi et al 1975). In the beginning the pseudo-dynamic tests included just one experimental assembly on which the displacement history imposed based on an on-line computer analysis incorporating the measured responses and reaction forces of the tested part (Mahin et al. 1989, Shing et al. 1996). In order to acquire the restoring force required to solve the equation of motion, the pseudo-dynamic tests used a ramp and hold loading procedure (Hanson and McClamroch 1984). In the pseudo-dynamic test method it was not necessary to apply real-time loading history to simulate the earthquake motion. Thus, the experimental conditions for large structures could be more under-controlled in comparison to shaking table tests (Kim and Lee 1995). However the inaccuracies pertaining to experimental errors and load rates were disclosed by comparing the results of pseudo-dynamic and shaking table tests (Yamazaki et al 1989).

In order to mitigate the experimental errors, Nakashima (1987) and Mahin et al (1989) recognized the necessity to improve the control of actuators. Mahin et al (1989) indentified that the computed displacements may be falsely imposed on the specimen due to inappropriate test setup and insufficient servo-control. Magonette (2001) reported on development of the continuous pseudo-dynamic testing and its application on large scale specimen. He demonstrated that the continuous pseudo-dynamic testing is successful in improvement on force measurements by compensating for force relaxation during the hold phase of the loading procedure. The inaccuracies regarding the load rates were insignificant in compare to other uncertainties in the pseudo-dynamic testing method especially in large scale specimens.

Substructuring can be considered as the most important feature in hybrid simulation. Through this technique, the test structure can be divided to two distinct subassemblies, numerical and
experimental (Wagg and Stoten 2001). One or more experimental subassemblies can be used in this technique. The physical elements can be highly nonlinear, complex behavior or more vulnerable. Numerical portion comprise of one or more than one analytical elements with well understood behavior. Another interesting feature utilizing the substructuring technique in hybrid simulation is that the experimental and numerical subassemblies can be geographically distributed (Mosqueda 2006). A complete description about fundamentals, components and procedure of hybrid simulation will be presented in chapter 5.

2.3. Hybrid simulation case studies

Yang et al (2008) conducted a hybrid simulation to study the response of a steel suspended-zipper-frame. In the hybrid model developed by Yang et al, the first-story inverted-V-braced sub-assembly, which was expected to experience more inelastic displacements, was simulated physically in the laboratory. The rest of the two dimensional frame, which was supposed to undergo less inelastic displacements, was modeled by nonlinear finite element. Figure 2.1 presents the substructuring arrangement employed by Yang et al. (2008). Comparison of results based on the hybrid simulation and pure analytical analysis indicated that the hybrid method is an accurate simulation for a complex structure. The ground motion used in Yang et al’s (2008) research was a modified version of LA22 ground motion recorded at JMA station during the 1995 Kobe earthquake. In addition, OpenFresco (Schellenberg et al. 2009), a new middleware that enables interaction between OpenSees and the control system, was used for the first time to perform a hybrid simulation of a complex structure. Further explanation about the Openfresco and Opensees will be presented in Chapter 4 and 5.
Lin et al (2010) performed a series of hybrid tests on a three-story single-bay full-scale buckling-restrained braced frame (BRBF) at the Taiwan National Center for Research on Earthquake Engineering. One of the objectives of their research was to verify the capability of a newly developed software framework for quasi-static structural testing. They used Opensees and Openfresco (Shellenberg et al 2009) to perform pseudo dynamic hybrid simulation. It was demonstrated that the hybrid experimental responses of the specimen were acceptably predicted by finite element programs such as Risa 3D and OpenSees.

In another study, hybrid simulations were performed to predict the response of a typical California highway overpass bridge under earthquake excitations followed by a heavy truck load
(Terzic and Stojadinovic 2010). The prototype was a five-span reinforced concrete bridge with single-column-bents. The physical portion of the hybrid model comprised a single column. The analytical and experimental portions of the hybrid model were interacting through four degrees of freedom: 2 lateral displacement and 2 rotations. Figure 2.2 shows the test setup designed for the experimental part of the hybrid simulation.

![Figure 2.2: Experimental setup for hybrid simulation of a five-span bridge (Terzic and Stojadinovic 2010)](image)

The remainder of the bridge structure was simulated numerically in OpenSees. The two subassemblies were interacting through the OpenFresco framework. The comparison between the results obtained from hybrid simulation and calibrated analytical models implied a close agreement of response quantities.

Survey of literature indicates that the effects of laboratory model scales on the results in hybrid testing have not been studied. Moreover, the method has not been used for application in structural health monitoring. The hybrid simulation presented in this dissertation is the first application of the method with the objective of structural health monitoring. The effect of scaling on reliability of scaled hybrid models of a complex structure is studied. Moreover, the results of
the hybrid simulations of a bridge were compared to the output of a pure experimental simulation on three shaking tables.
III. Fiber Optic Sensors

Developing a robust structural health monitoring system requires implementation of sensors capable of sensing the primary quantities pertaining to the performance of structures. Over the past three decades, fiber optic sensors have been evolved remarkably in various fields of aerospace, mechanical, biomedical and civil engineering. Notable characteristics such as: resistance against harsh environment, capability for embedment in different materials and immunity to electromagnetic fields, make the optical fibers a significant solution for challenging problems in sensing technology (Ansari, 2007). Fiber optics can be applied in structural health monitoring of civil systems by concentrating on measurement of strains, displacements, accelerations and vibration modes (Ansari, 2002). The practical implementation of fiber optic sensors in civil structural systems is comprehensively reviewed in another manuscript (Ansari, 2007). The most commonly used type of fiber optic sensors in the field of civil-structural engineering is Fiber Bragg Grating (FBG) sensor. The measurement mechanism in FBG sensors is based on relating the changes in the wavelength of light to the measurand of interest. A detailed discussion on FBG sensors is presented elsewhere (Ansari, 2008).

In modern design of reinforced concrete columns with circular section, three distinct parts of a cross section, namely, concrete cover, longitudinal steel reinforcement and concrete core influence on the behavior of column (Berry 2007). In the research presented here, three types of FBG sensors were employed to monitor the strains and deformations within different parts of the cross section in the plastic hinge area of reinforced concrete columns. The schematic in Figure 3.1 depicts the arrangement of sensors across the diameter of cross section and along the height of plastic hinge zone. The arrangements of sensors across the diameter of cross
sections and along the height of the columns are presented in detail for different scaled columns in chapter 6 of this dissertation. An arch-shaped sensor assembly was applied to

Figure 3.1: Schematic arrangement of sensors in across the cross section and along the plastic hinge height

monitor the deformations on the surface of concrete cover. This sensor is capable of withstanding large deformation reversals and measuring crack opening displacements (i.e. 10 mm). The detailing of this type of sensor can be accessed in (Bassam et al. 2011). Figure 3.2 shows a photo of two arch-shaped sensors adhered on the surface of one of the columns during a course of an experiment. The strains on the longitudinal reinforcements were measured by FBG sensors directly mounted on the surface of steel bars. Single FBG sensing elements were
pretensioned and then adhered on the surface of steel bars to capture the localized strains. The fragile FBG sensing elements were guarded by a special type of adhesive stretch tape used for strain gauge protection inside the concrete. The optical lead wires were also needed to be protected inside the concrete by three layers of shrinkage tubes to be immune from damages at ingress/egress locations. Figure 3.3 depicts a photo of FBG sensors installed on the surface of steel longitudinal reinforcement. The most challenging part for sensing of deformations across the column cross section was concrete core. In order to monitor the deformations inside the concrete core of the columns an embedded sensor assembly was developed. In the next sections of this chapter the design and calibration of the embedded sensor will be discussed. Installation of embedded sensors inside the concrete core of columns was performed during the construction of columns and will be explained in chapter 6.

3.1 Design of embedded sensor

One of the significant attributes of optical fibers is the capability for embedment within materials during their production (Ansari 2007). The geometric compatibility with the host material provides the possibility for fiber optic sensors to be integrated in a protective material without jeopardizing its sensitivity. Both sensing element and sensor lead-lines need to be protected inside the concrete. Pouring of concrete inside the forms, compacting and vibrating of fresh concrete and shrinkage of concrete following setting are all amongst the situations which can fracture the fragile fiber optic material. A limited number of studies are available in the literature on protection of fiber optic sensor in concrete (Leng et al 2006). The common protection procedure for these sensors includes a metallic tube which encapsulates the sensing body. Two shear keys defining the gauge length of sensor are attached to ends of the tube. Through this method the overall deformation over a length between the two end flanges of the
sensor is measured. The properties of the encapsulating material can significantly affect the performance of the sensor. The strain transfer should be effectively accomplished between protection tube and both sensing element as well as concrete (Kesavan et al 2010). However the metallic

Figure 3.2: Arch-shaped sensors on the surface of a 1/6 scaled column
encapsulation brings some shortcomings in sensing of deformation inside the concrete. In order to measure the deformations inside the concrete elements under a bending regime especially in regions with high curvature demands as plastic hinge zones, it is more desirable to utilize embedded sensors which are less stiff than concrete and flexible enough to capture the deformed shape of internal fibers of column. In addition, when the embedded sensor experiences large amounts of deformations, the protection material goes beyond the yielding limit and behaves in a plastic manner. Although the strain compatibility is still valid under the plastic regime, the strain transmission from sensing element to surrounding concrete can be disturbed due to localized residual deformations in the protection tube wall. To overcome the mentioned shortcomings, a special type of plastic tube from Polyether ether keton (PEEK) material was used as the protection cover of fiber optic embedded sensor developed for this research. PEEK is an organic thermoplastic polymer with excellent mechanical and chemical resistance properties applied in engineering applications. At the temperature of 23 °C, the tangent modulus as well as yield stress
of PEEK material are 4.10 GPa and 107.1 MPa respectively (Kemmish 2010). The maximum strains experienced by concrete core of the columns after full development of plastic hinge are still less than yield strain of PEEK material. Figure 3.4 shows a photo of manufactured sensor. The components of designed sensor are detailed in Figure 3.5. The steel shear keys are responsible for providing the bond required for transferring the deformation between the concrete and the embedded sensor. Each shear key is connected to a threaded steel tube with two nuts. The strain transfer between PEEK tube and threaded steel tube was accomplished by a high performance epoxy for hard to glue plastics. The bare fiber optic was pre-tensioned and then encapsulated within the PEEK tube through fiber optic connector epoxy injected along the length of PEEK tube.
Figure 3.4: Fiber optic embedded sensor

Figure 3.5: Details of fiber optic embedded sensor
3.2 Calibration of embedded sensor

In FBG sensors, the strain measurement is based on the shift in the central wavelength of the Bragg gratings in the optical fiber. The wavelength, at which the optical fiber signal is reflected, is defined by Bragg grating pitch (Measures, 2001). The relationship between the reflected wavelength and the grating period is:

\[ \lambda_B = 2n_e\Lambda \]  \hspace{1cm} (3.1)

Where, \( \lambda_B \) is the reflected wavelength, \( n_e \) is the refractive index and \( \Lambda \) is the period of grating in the optical fiber. Once the fiber is stretched, the shift in the Bragg grating’s pitch results in a proportionate change in wavelength. The relationship between strain, \( \varepsilon \), and the change in wavelength, \( \Delta\lambda \), is derived through calibration and defined by gauge factor, GF:

\[ \varepsilon = \frac{\Delta\lambda/\lambda_B}{GF} \]  \hspace{1cm} (3.2)

An optical interrogation instrument dynamically scans wavelength spectrum and detects the individual FBG sensors along the length of an optical fiber, each with a specific Bragg wavelength. For the calibration of embedded sensors, a specific setup capable of stretching the sensor by the means of a multi-meter was manufactured. The photo in Figure 3.6 shows the calibration setup containing a sensor. Figure 3.7 depicts a schematic of the test setup. In the calibration process, the displacement of sensor over the gauge length was correlated to the FBG shift of wavelength to acquire the sensor gauge factor. To calculate the average strain over the gauge length of each sensor, the displacement was divided by the gauge length. Equation 3.3 and 3.4 show the calibration equations of the sensor in terms of displacement and strain:

\[ \Delta L = \frac{\Delta\lambda}{GF_D} \] \hspace{1cm} (3.3)
\[ \epsilon = \frac{\Delta \lambda}{G_{FS}} \]  

(3.4)

\( G_{FD} \) and \( G_{FS} \) are the displacement and strain gauge factors which correlate the shift in the wavelength of sensor \((\Delta \lambda)\) to displacement and strain.

The calibration tests were repeated in three cycles to check against any hysteresis effects. The hysteresis effect in the calibration plot of a sensor can be due to either movement of sensor’s components against each other or intrinsic characteristic of FBG, both of which were overcome in the designed sensors. For all practical purposes the loading and unloading possessed a common path assuring inexistence of hysteresis.

![Figure 3.6: The calibration setup for embedded sensor](image)

![Figure 3.7: A schematic of the calibration setup for embedded sensor](image)
To accomplish the objectives of this research, 18 embedded sensors were manufactured and calibrated for use in the nine scaled columns. Figure 3.8 presents a calibration graph including three cycles of loading and unloading for both low and high range of displacement demands. Figure 3.9 shows another plot for the same calibration test correlating the strains associated with the displacement of sensor over the gauge length to the change in FBG wavelength. Figures 3.10 through 3.26 present the strain-calibration graphs for the other 17 sensors. The strain and displacement gauge factors pertaining to all 18 sensors are presented in Table 3.1.
Figure 3.9: Calibration of embedded sensor for strain (Sensor 1)

\[ y = 849.64x - 1E+06 \]
\[ R^2 = 0.9981 \]

Figure 3.10: Calibration of embedded sensor for strain (Sensor 2)

\[ y = 843.32x - 1E+06 \]
\[ R^2 = 0.9972 \]
Figure 3.11: Calibration of embedded sensor for strain (Sensor 3)

Figure 3.12: Calibration of embedded sensor for strain (Sensor 4)
Figure 3.13: Calibration of embedded sensor for strain (Sensor 5)

\[ y = 1097.5x - 2E+06 \]
\[ R^2 = 0.9987 \]

Figure 3.14: Calibration of embedded sensor for strain (Sensor 6)

\[ y = 867.4x - 1E+06 \]
\[ R^2 = 0.9984 \]
Figure 3.15: Calibration of embedded sensor for strain (Sensor 7)

\[ y = 716.7x - 1E+06 \]
\[ R^2 = 0.9975 \]

Figure 3.16: Calibration of embedded sensor for strain (Sensor 8)

\[ y = 807.43x - 1E+06 \]
\[ R^2 = 0.9974 \]
Figure 3.17: Calibration of embedded sensor for strain (Sensor 9)

Figure 3.18: Calibration of embedded sensor for strain (Sensor 10)
Figure 3.19: Calibration of embedded sensor for strain (Sensor 11)

Figure 3.20: Calibration of embedded sensor for strain (Sensor 12)
Figure 3.21: Calibration of embedded sensor for strain (Sensor 13)

\[ y = 923.98x - 1 \times 10^6 \]
\[ R^2 = 0.9966 \]

Figure 3.22: Calibration of embedded sensor for strain (Sensor 14)

\[ y = 1025.5x - 2 \times 10^6 \]
\[ R^2 = 0.9976 \]
Figure 3.23: Calibration of embedded sensor for strain (Sensor 15)

\[ y = 837.07x - 1E+06 \]
\[ R^2 = 0.9972 \]

Figure 3.24: Calibration of embedded sensor for strain (Sensor 16)

\[ y = 892.17x - 1E+06 \]
\[ R^2 = 0.9977 \]
Figure 3.25: Calibration of embedded sensor for strain (Sensor 17)

\[ y = 960.35x - 1 \times 10^6 \]

\[ R^2 = 0.9973 \]

Figure 3.26: Calibration of embedded sensor for strain (Sensor 18)

\[ y = 831.6x - 1 \times 10^6 \]

\[ R^2 = 0.997 \]
Table 3.1: The displacement and strain gauge factors for 18 sensors

<table>
<thead>
<tr>
<th>Sensor</th>
<th>Strain Gauge Factor</th>
<th>Displacement Gauge Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00118</td>
<td>196.16</td>
</tr>
<tr>
<td>2</td>
<td>0.00119</td>
<td>197.63</td>
</tr>
<tr>
<td>3</td>
<td>0.00097</td>
<td>162.36</td>
</tr>
<tr>
<td>4</td>
<td>0.00080</td>
<td>133.64</td>
</tr>
<tr>
<td>5</td>
<td>0.00091</td>
<td>151.86</td>
</tr>
<tr>
<td>6</td>
<td>0.00115</td>
<td>192.15</td>
</tr>
<tr>
<td>7</td>
<td>0.00140</td>
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<tr>
<td>8</td>
<td>0.00124</td>
<td>206.42</td>
</tr>
<tr>
<td>9</td>
<td>0.00091</td>
<td>152.17</td>
</tr>
<tr>
<td>10</td>
<td>0.00109</td>
<td>181.76</td>
</tr>
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<td>11</td>
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<tr>
<td>14</td>
<td>0.00098</td>
<td>162.52</td>
</tr>
<tr>
<td>15</td>
<td>0.00119</td>
<td>199.11</td>
</tr>
<tr>
<td>16</td>
<td>0.00112</td>
<td>186.81</td>
</tr>
<tr>
<td>17</td>
<td>0.00104</td>
<td>173.55</td>
</tr>
<tr>
<td>18</td>
<td>0.00120</td>
<td>200.42</td>
</tr>
</tbody>
</table>
IV. Nonlinear Earthquake Response Modeling of a 2-span Bridge

A quarter scale model of a two-span reinforced concrete bridge was tested utilizing the multiple shaking table system at University of Nevada, Reno. The tests were part of a multi-university project using the network for earthquake engineering simulation (NEES) facilities. The bridge was tested from the pre-yield state to failure. OpenSees (Open System for Earthquake Engineering Simulation) program was employed to conduct analytical modeling for determining the nonlinear response of the bridge model. In order to validate the finite element simulation, the response of the bents as measured by the wire transducers were compared with the displacements obtained from simulation. Throughout all the events, the simulated results match the actual response rather well.

4.1. Background

A quarter-scale reinforced concrete bridge model with two spans supported on three two-column piers was tested to failure using the shake table system at University of Nevada Reno (Johnson et al. 2008). These experiments were part of a larger project with the objective of manifesting the capacities of the network for earthquake engineering simulation (NEES) system for studying the effects of soil-foundation-structure interaction on bridges (Wood et al. 2004). As illustrated in Figure 4.1, the columns of the bridge structure are with different heights. The differences in the heights of the columns cause the stiffness irregularities in the structure of the bridge. The length scale of 1/4 was chosen on the basis of capacity limits of the test setup. Figure 4.2 shows the dimensions of the slab and bents as well as the column reinforcement details of the shake-table specimen. The total length of the specimen was 67.3 ft. The bents had clear heights of 6, 8 and 5 ft respectively. To prevent complete collapse after the columns failure, safety frames were installed as illustrated in Figure 4.1.
The bridge was tested under both low-amplitude (pre-yielding of columns) and high-amplitude earthquake motions. The simulation of bridge in this chapter was carried out based on the high-amplitude motions.

4.2. High-Amplitude Shake Table Tests

The bridge model was subjected to both low and high-amplitude motions. The motions were based on the components of the Century City Country Club North record from the 1994 Northridge, California earthquake. The system response remained elastic under low amplitude motions and all the columns stayed below their yielding state. The analytical modeling presented in this thesis is solely based on the high-amplitude motions exciting the bridge model in the transverse direction from preyield to failure state. High-amplitude motions were ramped in steps from Test 12 (0.075g peak ground acceleration) to Test 19 (1.66g peak ground acceleration). The maximum amplitudes of input earthquake motions are listed in table 4.1 for test 12 to test 19.
Figure 4.2: Slab dimensions, bent dimensions, and column rebar for the shake table specimen (Johnson et al. 2008)
Table 4.1: Bridge input earthquake amplitudes

<table>
<thead>
<tr>
<th>Event</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>0.075</td>
</tr>
<tr>
<td>13</td>
<td>0.15</td>
</tr>
<tr>
<td>14</td>
<td>0.25</td>
</tr>
<tr>
<td>15</td>
<td>0.5</td>
</tr>
<tr>
<td>16</td>
<td>0.75</td>
</tr>
<tr>
<td>17</td>
<td>1.00</td>
</tr>
<tr>
<td>18</td>
<td>1.33</td>
</tr>
<tr>
<td>19</td>
<td>1.66</td>
</tr>
</tbody>
</table>

4.3. Analytical Modeling

In this thesis, the object oriented finite element software framework OpenSees and the graphical user interface OpenSees Navigator was used to simulate the nonlinear response of the shake table. The programs were developed at the University of California, Berkeley, primarily to support earthquake simulations (Mazzoni et al. 2007).

The measured shake table motions from tests 12 to 19 (high-amplitude tests) were input to the finite element model. A broad range of structural nonlinearity was experienced by the bridge model under the progressing seismic events. In this simulation, nonlinear fiber elements with distributed plasticity were employed to duplicate the behavior of columns.

4.3.1. Fiber-based Analysis

The nonlinear fiber-based beam-column element with distributed plasticity was used to duplicate the flexural response of concrete columns. In the fiber-section analysis, unidirectional...
steel and concrete fibers are defined to simulate a flexural member. Material properties of steel and concrete are specified in the direction of the column length. Since the flexure-shear interaction is not integrated in the formulation of the element, the fiber section assumption may not be appropriate for modeling response of shear-dominant members. However, fiber analysis still remains the most precise and frugal method to simulate seismic behavior of concrete columns (Spacone et al. 1996a, 1996b).

The direct stiffness method which solves the equilibrium equation of the global system to obtain the nodal displacements, is generally used in the fiber analysis. By interpolating the element nodal deformations at the integration points along the element length, the section deformation is determined. Based on the assumption that plane sections remain plane (Euler-Bernoulli theory of slender beam-columns), the strain in each fiber is obtained. The calculated fiber strains are used to update the fiber stress and stiffness in accordance with the fiber material model.

4.3. 2. Configuration of Model

A schema of nodes and elements as it appears in OpenSees navigator interface are shown in Fig. 4.3. All the imposed scaling masses were assumed to be lumped on the deck nodes. Basically due to scaling effects, the axial loads in the modeled bridge columns were smaller than the ones in the prototype bridge. Scaling masses were placed on the top of the deck to provide required axial forces in the columns (Bazant 2005). It is assumed that the column bases are rigidly attached to the foundation such that they can be modeled as fixed connections. All the superstructure elements were considered uncracked as per design requirements. Hence, the post tensioned deck and the cap beams were modeled with linear elastic elements. The linear
elastic elements applied for this purpose were considered to have gross section properties using the unconfined concrete properties at the 28-day cylinder strength.

4.3. 3. Nonlinear Elements

Force-based fiber-section elements with distributed plasticity referred to as Nonlinear-Beam-Column elements in OpenSees were used to model the bridge columns. Three different fiber-element types representing concrete core, steel reinforcement and concrete cover were used in the column fiber-section. As illustrated in Figure 4.4 a fiber-section with 8 slices, 7 layers of core, and 2 layers of cover was chosen for the analysis.
4.3. 4. **Material models**

The uniaxial material model selected for concrete in this analysis referred to as the Kent-Scott-Park model with degrading linear unloading/reloading stiffness on the basis of the work reported by Karsan and Jirsa (1969). The typical cyclic behavior of the concrete material employed for simulation in OpenSees is shown in Figure 4.5. The 28 day compressive strength of the concrete used for construction of the bridge was 5 ksi (34.5 MPa) as reported by Johnson et al. 2008. The average strains at peak stress and at the ultimate strength of the 28 day concrete were 0.002 and 0.006, respectively. For the confined concrete the material model parameters were calculated based on Mander’s model (Mander et al, 1988), with 6.56 ksi (45.2 MPa) peak stress at 0.005 strain and 5.1 ksi (35.1 MPa) stress at the ultimate strain of 0.0169.
The Giuffré-Menegotto-Pinto Model with Isotropic Strain Hardening was selected for cyclic response of steel in the plastic regime. This material model was employed to build a uniaxial steel material behavior with isotropic strain hardening. This model also has the capability for transition from elastic to plastic regimes. Figure 4.6 shows the stress-strain curve for this material model with strain hardening (Mazzoni et al. 2007). The reinforcing steel bars were modeled using a bi-linear curve with an initial slope of 29000 ksi (199810 MPa), yielding stress of 68 ksi (469 MPa), and the hardening slope of 212 ksi (1461 MPa). The tensile and compressive behavior of these bars was considered symmetrical.
4.3. 5. Zero-length section element and Bar stress vs slip model

A zero-length section element includes one section corresponding to one integration point, which determines the force-deformation response of the element. The zero-length section element available in OpenSees can be used to model the end rotations of the members.

A model characterizing the stress versus end slip response of the steel rebar was introduced by Zhao and Sritharan (2007) based on the measured response of columns and pull out test data. The model imitates the strain penetration effects using the zero-length section element available in OpenSees. This model is used for simulating bond slip along a portion of the anchorage length due to strain penetration effects in fully anchored steel reinforcement bars, which is mostly the case for column longitudinal bars anchored into footings (Mazzoni 2007).

The function embodied in Figure 4.7 depicts the envelope of the bar stress versus the slip response at the end of the flexural member. The slip at the point that the bar stress reaches the yield ($s_y$) and ultimate strengths ($s_u$) are obtained from equations 4.1 and 4.2, respectively:

![Figure 4.6: Hysteresis and strain hardening of steel (Mazzoni 2007)](image)
\[ s_y = 0.1 \left( \frac{d_b}{4} \frac{f_y}{\sqrt{f_c'}} (2\alpha + 1) \right) \frac{1}{u} + 0.0134 \]  
(4.1)

\[ s_u = 35s_y \]  
(4.2)

Where \( d_b \) (in) is steel bar diameter; \( f_y \) (ksi) and \( f_c' \) (ksi) are yield strength of steel bar and compressive strength of concrete; and \( \alpha \) is a parameter used in the local bond-slip relation and can be taken as 0.4 in accordance with CEB-FIP Model Code 90 (FIB Task Group 5.2 2000).

Figure 4.7: Steel bar stress versus slip for completely anchored reinforcing bars into footings (Zhao et al. 2005)
4.3. 6. Input seismic ground motions

Input ground motions to the bridge pertained to the recorded Northridge earthquake accelerations. The ground motions were applied in a number of successively increasing steps in amplitude. These motions were applied to the three bridge bases through shaking tables simultaneously in the transverse direction.

4.4. Validation of the finite element model

Validation of the finite element simulation was accomplished on the basis of post-seismic analysis of the results from the shake-table test. Displacement response of the bents as measured by the wire transducers and the simulated results are compared in Figures 4.9 to 4.16 pertaining to seismic events 12 to 19. For all practical purposes the simulated results matched the actual response rather well. Peak maximum and minimum relative displacements for Tests 12–19 are listed in Table 4.2 to 4.4. In addition, the tables show the percent difference between the analytical and experimental peak maximum and minimum displacements for each bent. In overall, as much as the input amplitudes of motions increase, the differences between maximum and minimum displacements of the bents from shaking table tests and simulations become higher. This can be attributed to the shortcomings of the numerical material models in predicting the behavior of concrete columns in the states close to failure. Discussion regarding the results of the nonlinear finite element analysis in comparison to scaled hybrid simulations will be presented in chapter 6.
Figure 4.9: Comparison of transverse displacement from finite element analysis and experiment

(0.075g PGA, Event 12)
Figure 4.10: Comparison of transverse displacement from finite element analysis and experiment
(0.15g PGA, Event 13)
Figure 4.11: Comparison of transverse displacement from finite element analysis and experiment

(0.25g PGA, Event 14)
Figure 4.12: Comparison of transverse displacement from finite element analysis and experiment

(0.5g PGA, Event 15)
Figure 4.13: Comparison of transverse displacement from finite element analysis and experiment

(0.75g PGA, Event 16)
Figure 4.14: Comparison of transverse displacement from finite element analysis and experiment

(1.00g PGA, Event 17)
Figure 4.15: Comparison of transverse displacement from finite element analysis and experiment

(1.33g PGA, Event 18)
Figure 4.16: Comparison of transverse displacement from finite element analysis and experiment

(1.66g PGA, Event 19)

Table 4.2: Comparison of maximum and minimum bent displacements measured from shaking table test and calculated from finite element analysis (Bent 1, scale 1/4)

<table>
<thead>
<tr>
<th>Event</th>
<th>Finite Element</th>
<th>Shaking Table</th>
<th>% Difference from Shaking Table</th>
<th>Finite Element</th>
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<td>34.70</td>
<td>-3.70</td>
<td>-2.74</td>
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</table>

Average | 15.07 | 18.26 |
Table 4.3: Comparison of maximum and minimum bent displacements measured from shaking table test and calculated from finite element analysis (Bent 2, scale 1/4)

<table>
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<td>Average</td>
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</table>

Table 4.4: Comparison of maximum and minimum bent displacements measured from shaking table test and calculated from finite element analysis (Bent 3, scale 1/4)

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</table>
V. Hybrid Simulation

Hybrid simulation is an experimental testing methodology performed on a hybrid model which can be used for simulating the behavior of structures under dynamic loads. A hybrid model is composed of scaled experimental and computational components of a structure incorporated into a single model by enforcing the displacement compatibility and the force equilibrium at the common nodes. One or more portions of the structure which may be highly nonlinear, numerically difficult to model, or have uncertain properties are tested experimentally in one or more laboratories and the remainder of the structure is simulated in one or more computers (Schellenberg et al. 2009).

The dynamic response of the hybrid model under excitations is computed during a hybrid simulation in the time domain using a step-by-step integration procedure. Thus, hybrid simulation can be considered an experimental method, where the loadings applied on the specimens are determined during the run of an experiment. Hybrid simulation can also be viewed as a standard finite element analysis with time-stepping solution procedure, where the numerical model includes some experimental subassemblies. Hence, hybrid simulation is a form of system testing which overcomes the issues on accommodating proper boundary conditions that exist in classical shaking table tests.

5.1. Introduction

Currently, there are several methods to perform experimental testing for assessing the behavior of structural systems under earthquake. The first methodology is the quasi-static testing technique, in which a predefined history of loads or displacements is applied on the tested structure by actuators.

The effect of changes in boundary conditions, material properties, loading rates, and other factors can be studied by applying a load or displacement history on a specimen. These
tests are rather easy and economical to conduct. However the applied load patterns are generally inadequate to mimic the alternating force distribution that a structure experiences during a real seismic event.

The second category of laboratory testing methods for structures is shaking table tests. The shaking table tests simulate the actual conditions that exist during a particular earthquake precisely. The inertial and energy dissipation characteristics of a tested structure, geometric nonlinearities, localized yielding and damage as well as the component failures are considered in the dynamic response of structures on the shaking tables. A complete structural system constructed in detail following the rules of dynamic similitude is generally required for shaking table tests. Moreover, controlling the real time interaction of the structural components with side boundaries is particularly challenging. In addition, the size, weight, and strength of specimens are restricted by limited capacity and size of shaking tables. As a result, the actuality of many shaking table tests is disputable due to reduced scales and simplifications of the specimens (Elkhouraibi and Mosalam 2007).

The third method is hybrid simulation. Hybrid simulation is an experimental testing technique in which a simulation is carried out based on a step-by-step solution procedure for a hybrid model. A hybrid model simulates a structural system in both numerical and physical components.

In conventional numerical simulation, the entire structure is analyzed computationally. However, in hybrid simulation method the forces related to the stiffness or mass of one or more than one component of a structural system are obtained from a laboratory test. In hybrid simulation, the physical parts of the hybrid model are examined in one or more laboratories, whereas the numerical components are analyzed in one or more computers at the same time.
These tests can be executed quasi-statically using conventional computer-controlled actuators, since dynamic aspects of the tests are manipulated numerically. Thus, hybrid simulation can be considered as either a classical nonlinear finite element analysis in which the physical properties of some elements are derived experimentally, or as an actuator-based testing procedure in which the loading is defined during the run of an experiment.

There are several advantages obtained by using hybrid simulation in comparison to quasi-static and shaking table test methods. Hybrid simulation makes it possible to study the behavior of a structure subjected to various loading patterns by providing the capability of defining the loading analytically. Wind and blast loads, traffic patterns, hydrodynamic loading conditions and seismic events can be simulated by including them in the finite element part of the hybrid model. In hybrid simulation the well understood parts which can be modeled reliably by finite element method are simulated numerically. Thus the physical portion of the simulation reduces the highly nonlinear components or the parts which are difficult to simulate analytically. Therefore the limitations on weight, size and strength of the physical assemblies are much less than shaking table tests. Since hybrid simulation can be carried out on prolonged time-scales, the equipments frequently accessible at existing testing facilities such as actuators, hydraulic power-supply and servo-valves are adequate for testing (Stojandinovic et al 2006). All the mentioned attributes of hybrid simulation make it an economical approach for executing structural system testing in laboratories. Slow tests in hybrid simulations provide a more comprehensive understanding on the behavior of a structure in terms of initiation and progress of damage along the duration of a simulation. In order to exploit the facilities in different laboratories, experimental and numerical substructures can be geographically distributed. Soil-structure interactions as well as geometric nonlinearities can be embodied inside the analytical part of the hybrid model.
5.2. Procedure and elements of hybrid simulation

Several software and hardware components are necessary to perform a hybrid simulation. Figure 5.1 illustrates how these components interact during a hybrid simulation. The components are as follows:

1. A finite element model of a structure in a computer which discretize a structural dynamic problem spatially and timely.
2. A specimen supported by a test-setup which represents the physical portion of the structure
3. Static or dynamic actuators connected to a controller in order to apply the incremental response determined by time-stepping integration methods to the physical part of the structure.
4. The data acquisition system including instruments such as load cells, LVDTs (linear voltage displacement transformer), and accelerometers as a means of measuring the response of the specimen and returning the data to the time-stepping algorithm to promote the solution to the next step of analysis

The flowchart in Figure 5.2 depicts hybrid simulation testing procedure which utilizes finite element method with direct integration analysis and linear equilibrium solution algorithm computers (Schellenberg et al 2009). In each time step of the analysis, the first action is to determine the new trial displacements and to increment the forces and analysis time. Subsequently the new trial displacements are sent to both analytical and experimental parts. The analytical portion reserves the new trial displacements in order to determine the effective or unbalanced forces. In the experimental portion, the actuators impose the new trial displacements to the physical parts.
The second action in the hybrid simulation analysis is to solve the system in the current time step for the new trial displacements and incremented forces. In this step of analysis, first the effective stiffness matrices are assembled based on both analytical and experimental portions. The initial stiffness matrices of experimental subassemblies are determined analytically prior to the start of the analysis. However there are some methods which calculate the tangent stiffness matrices of the experimental subassemblies based on the force and displacement measurements.

Figure 5.1: Components of hybrid simulation
For Each Time Step:

- **Analytical Part**
  - Set Trial Displacement
  - Calculate Tangent or initial Stiffness
  - Calculate Resisting Forces

- **Hybrid Simulation Analysis**
  - Start the direct integration analysis
  - Calculate new trial displacement and Update Domain (Set new displacements, increment loads and time by \( \Delta t \))
  - Solve the current time step
  - Form effective Tangent, initial or mixed Stiffness \( K \) in \( KU=F \)
  - Form the unbalanced force \( F \) in \( KU=F \)
  - Solve \( KU=F \) for \( U \)
  - Update the response at \( t+\Delta t \)
  - Finish the solution

- **Experimental Part**
  - Command the Actuator to impose the trial displacement
  - Return Initial or Tangent Stiffness if available
  - Measure the response when the target is reached

Figure 5.2: Procedure flowchart of hybrid simulation computers (Schellenberg et al 2009)
In the next step, the unbalanced or effective force vector should be assembled. The analytical portion of hybrid simulation uses the stored trial displacements to calculate and return its contribution to the global unbalanced (or effective) force vector in the finite element analysis. From the other side, the experimental subassemblies send the measured resisting forces to the global unbalanced (or effective) force vector. Afterwards the equilibrium solution algorithm solves the linear system of equations consisting of globally assembled effective stiffness matrix and the force vector. Finally the achieved displacements are utilized to update the structural responses for the next time step.

5.3. Slow versus rapid hybrid simulation

Based on the speed of execution, hybrid simulation can be classified into two categories of slow and fast. Slow hybrid simulation tests are applied for the structures in which the physical part does not display rate-dependent behavior. It is necessary to perform the slow tests with the continuous motion of actuators to prevent from force relaxation impediments.

Fast hybrid simulation tests demand dynamic actuators with high capacity accumulators and hydraulic pumping systems. Considering the fact that accurately controlling these actuators is more difficult because of inertial force feedbacks, the inertial and damping force produced by the physical part of the hybrid model in fast tests need to be considered correctly.

The equations of motion undertake different forms for slow and fast hybrid simulations. In the slow tests where the effect of inertial forces in physical portion of the hybrid model is ignored, the equations for the structural dynamic problem can be written as follow:

\[ M\ddot{U}_{i+1} + C\dot{U}_{i+1} + F^A(\dot{U}_{i+1}, \ddot{U}_{i+1}) + F^E(U_{i+1}) = F_{i+1} \]  

Where \( M \) is the mass matrix of the structure, \( C \) is the viscous damping matrix, \( F^A \) is the resisting force vector of analytical portion, \( F^E \) is the resisting force of experimental subassemblies and
F_{i+1} is the vector of externally applied nodal forces. In this equation, the mass and viscous damping matrices of all the numerical and experimental elements are assembled numerically. Since the test is being executed slowly, the resisting force in experimental subassemblies does not incorporate any terms related to inertial or viscous damping.

On the other hand, when hybrid simulations are carried out rapidly, it is crucial to account for viscous damping and inertial forces produced in the physical portion of the hybrid model in the resisting force of experimental subassemblies. Therefore the resisting force of experimental subassemblies $F^E_r$ needs to be modified as follow:

$$F^E_r (U_{i+1}) = F^E_{r,i+1} - M^E \ddot{U}^E_{i+1} - C^E \dot{U}^E_{i+1}$$  \hspace{1cm} (5.2)

The measured resisting forces are stored in the vector $F^E_{r,i+1}$ which includes the dynamic effects and is assembled from the experimental part of the hybrid model. $M^E$ and $C^E$ are the mass and viscous damping matrices respectively. They are assembled from the experimental part of the hybrid model, and $\ddot{U}^E_{i+1}$ and $\dot{U}^E_{i+1}$ are the accelerations and velocities respectively. They are measured by the data acquisition system and assembled from the experimental portion of the hybrid model.

5.4. Hybrid simulation Middleware (OpenFresco)

In hybrid simulation, the experimental framework should provide the means to communicate with the finite element code. The Open-Source Framework for Experimental Setup and Control, OpenFresco, used in this research, is the software providing additional functions for the finite element to interact with laboratory controllers and data acquisition systems computers (Schellenberg and Mahin 2006).

This software framework provides some means to represent the experimental portions of the hybrid model. The software produces input commands (displacement, velocities,
accelerations and forces) for the controllers and data acquisition systems and convert the measured signals (displacement, velocities, accelerations and forces) back into proper forms for the finite element software. In addition this software is capable of interacting with a wide range of controllers and data acquisition systems as well as a variety of finite element software in geographically distributed locations.

Figure 5.3 depicts the components of the OpenFresco software which make it capable of providing a bridge to communicate between finite element software and the experimental equipments.

![Figure 5.3: Components of OpenFresco Software (Schellenberg et al 2009)](image)

5.4.1 Experimental Element

Experimental element class represents specimens which are physically tested in a laboratory. Experimental element works within the finite element analysis part of the hybrid
simulation and provides force vector, mass matrix, stiffness matrix and damping matrix for a given displacement state. Experimental elements need to transform the nodal displacements from the global coordinate system to appropriate quantities for imposing on the physical specimen and transform the measured displacements back to the global coordinate system.

5.4.2 Experimental Site

Since, in reality, the laboratories for the experimental part of the hybrid simulations and computational sites may be in different locations, the experimental site class is designed in OpenFresco to make the different experimental and computational sites communicate to each other. The experimental site supports various communication protocols such as TCP/IP, UDP and etc and cryptographic protocols such as TLS or SSL.

5.4.3 Experimental Setup

Experimental setup represents the different configurations of the actuators in the laboratories. Experimental setup transforms the boundary conditions in the local coordinate system of the experimental element into the degrees of freedom of the actuators. In addition, the experimental setup transforms the measured quantities by the data acquisition system back to the degrees of freedom of the experimental element in the local coordinate system.

5.4.4 Experimental Control

Experimental control provides an interface in the hybrid simulation software framework which makes it capable of communicating with various controllers and data-acquisition systems. In fact, while the experimental setup is in charge of mapping the configuration of the transfer system with experimental element local coordinate system, the IT aspect of communication between the finite element software and the experimental transfer system is carried out by experimental control.
VI. Hybrid Simulation Based Experimental Program

Experimental program in this research consists of the hybrid simulations conducted to study the behavior of a two-span reinforced concrete bridge. The hybrid simulations were carried out in three different scales of 1/6, 1/8 and 1/10. Evaluation of these hybrid simulations was carried out by comparing the responses obtained from the hybrid simulations and shaking table tests. Comparison of the results indicates that the hybrid simulation approach used in this study can be used to accurately model the seismic response of a complex structural system such as a two-span reinforced concrete bridge. Details pertaining to the shake-table experiments conducted on a quarter-scale, two-span bridge system at the University of Nevada, Reno were discussed in chapter 5. In this chapter, development of the hybrid models, construction, instrumentation and test setup for the physical parts of the hybrid models as well as hybrid simulation test procedure, results and interpretation of the results are discussed.

6.1. Development of the hybrid models

Novel applications of hybrid simulation are presented in this chapter. The main purpose is to demonstrate and validate the features and the flexibility of the simulation framework. The tests were performed at University of Illinois at Chicago and utilized the implementation of details for hybrid simulation. The scaling factors were established based on laboratory restrictions and economic practicality. However it should be confirmed that the size effects are not pronounced in the test results. Three different scales of 1/6, 1/8 and 1/10 were used for the hybrid model in order to investigate the effect of scaling in reproducing the shaking table test results. It was envisioned that the optimal minimum scale appropriate for simulating the damage progress in the physical part of the models would have been discovered based on the results of this study.
6.1.1 Scaling and similitude Laws

While the physical testing constraints limit the feasibility of full-scale prototype testing, the application of reduced-scale models to simulate the behavior of full-scale prototypes is crucial. To achieve accurate correlation between scaled model and prototype behavior, the basic principles of similitude must be satisfied. The similitude theory and practical scale modeling for structural applications are reviewed to obtain a better understanding of proper scaled modeling techniques. Scaled simulations for the 1/6, 1/8 and 1/10-scale models were based on the 1/4-scale shaking table experiment conducted at University of Nevada Reno (Johnson et al. 2008). The structural model and the ground motions were scaled using similitude principles so that the proper response could be observed in the model domain.

Strain was selected as the dimensionless property which would be preserved between the scaled model and prototype:

\[ \varepsilon_m = \varepsilon_p \]  

(6.1)

where \( m \) and \( p \) stand for model and prototype respectively. Modulus of elasticity \( E \) for reinforced concrete in all scales remains the same. Therefore:

\[ E_m = E_p \]  

(6.2)

Using the stress-strain relationship:

\[ \sigma = E \varepsilon \]  

(6.3)

Results in the equality of stress \( \sigma \) in model and prototype:

\[ \sigma_m = \sigma_p \]  

(6.4)

Stress can be expressed in terms of force (\( F \)) and length (\( L \)):

\[ \sigma = \frac{F}{L^2} \]  

(6.5)

Thus:
By substituting Newton’s Second Law in equation (6.6) the following equations can be achieved:

\[
\frac{F_m}{L_m^2} = \frac{F_p}{L_p^2}
\]  

(6.6)

Or:

\[
\frac{M_m}{L_m T_m^2} = \frac{M_p}{L_p T_p^2}
\]  

(6.7)

\[
\frac{M_m}{M_p} = \frac{L_m T_m^2}{L_p T_p^2}
\]  

(6.8)

Where \(M\) and \(T\) are mass and time respectively. Scaling factors \((\lambda)\) for mass, length, and time, which are ratios of the model-scale value divided by the prototype-scale value can be related by equation (6.9):

\[
\lambda_m = \lambda_L \lambda_T^2
\]  

(6.9)

Acceleration is another quantity which was chosen to be preserved between the scaled model and the prototype. Therefore the time scale factor can be expressed in terms of the length scale factor:

\[
\lambda_T = \sqrt{\lambda_L}
\]  

(6.10)

The similitude laws for the scaling factor triplet (length, strain, acceleration) \((\lambda_L, \lambda_e, \lambda_A)\) are shown in Table 6.1. Modal analysis was performed on the scaled finite element models to validate the authenticity of scaled stiffness and mass matrices in predicting the scaled structural periods.

6.1.2 Components and procedure of hybrid simulation

As described in chapter four, principal components including software and hardware are necessary to perform a hybrid simulation. These components are depicted in Figure 6.1.
<table>
<thead>
<tr>
<th>Physical Quantity</th>
<th>Dimension</th>
<th>Scaling Factor with ($\lambda_L, \lambda_F = 1, \lambda_A = 1$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length, $l$</td>
<td>$L$</td>
<td>$\lambda_L$</td>
</tr>
<tr>
<td>Displacement, $d$</td>
<td>$L$</td>
<td>$\lambda_L$</td>
</tr>
<tr>
<td>Velocity, $v$</td>
<td>$LT^{-1}$</td>
<td>$\lambda_L^{-1}$</td>
</tr>
<tr>
<td>Acceleration, $a$</td>
<td>$LT^{-2}$</td>
<td>1</td>
</tr>
<tr>
<td>Force, $F$</td>
<td>$F$</td>
<td>$\lambda_L^{2}$</td>
</tr>
<tr>
<td>Time, $t$</td>
<td>$T$</td>
<td>$\lambda_L^{-2}$</td>
</tr>
<tr>
<td>Modulus, $E$</td>
<td>$FL^{-2}$</td>
<td>1</td>
</tr>
<tr>
<td>Stress, $\sigma$</td>
<td>$FL^{-2}$</td>
<td>1</td>
</tr>
<tr>
<td>Strain, $\epsilon$</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Mass, $m$</td>
<td>$FL^{-1}T^2$</td>
<td>$\lambda_L^{2}$</td>
</tr>
<tr>
<td>Damping, $c$</td>
<td>$FL^{-1}T$</td>
<td>$\lambda_L^{-3}$</td>
</tr>
<tr>
<td>Stiffness, $k$</td>
<td>$FL^{-1}$</td>
<td>$\lambda_L$</td>
</tr>
<tr>
<td>Period, $T$</td>
<td>$T$</td>
<td>$\lambda_L^{-1}$</td>
</tr>
<tr>
<td>Frequency, $f$</td>
<td>$T^{-1}$</td>
<td>$\lambda_L^{1}$</td>
</tr>
</tbody>
</table>
In the hybrid simulation procedure, a specimen representing the bridge column in the shortest bent of the two-span bridge was treated as the physical portion of a hybrid model. The rest of the bridge as well as the seismic ground motions were considered in the numerical part of the hybrid model. A portion of the bridge that experiences the most extensive damages under the transverse seismic loading was selected to be modeled experimentally. For the hybrid models in this research, a column in the shortest bent was chosen over the columns in the other bents because the shortest bent attracts larger seismic forces than the other bents for the same displacement.
demand. Figure 6.2 shows the finite element model including the experimental element distinguished with an encompassing oval. During the hybrid simulation test the bridge model was subjected to two sequences of loading: first the gravity load and second the recorded ground motion. In each integration time step, the displacements imposed at the top of the specimen were calculated based on the dynamics of the discrete model of the bridge structure. The controller commands the actuator to apply the calculated displacement. The load cell and displacement transducer (LVDT) measure the corresponding resisting force as well as displacement and then transfer the data to the data acquisition (DAQ) system. The DAQ system returns the measured force and displacement to the time-stepping solution algorithm to advance the solution to the next analysis step.

Figure 6.2 Hybrid model: finite element model including the experimental element
6.1.3 Physical part of the hybrid models

Concrete columns in three scales were designed and constructed as the experimental portion of the hybrid model. The geometry, dimensions and reinforcement of concrete columns are detailed in Figure 6.2. The columns were constructed 10 inch longer in order to accommodate the steel clamps for attachment of the column top to the actuators. The ratio of longitudinal reinforcement in the columns was 1.56% (Johnson et al. 2008). Eight number three (8#3) and six number three (6#3) steel rebars were used as the longitudinal reinforcement of columns for the 1/6 and 1/8 scaled models. The amount of reinforcement for the 1/8 scaled model was larger than the 1.56%. This was to satisfy the ACI 318 limits for reinforcement of compression members. According to ACI 318 (10.9.2) minimum number of longitudinal bars in compression members shall be 6 for bars enclosed by spirals (ACI 2005). The longitudinal reinforcement in the 1/10 scale column consists of 6 steel bars with diameter of 0.25 in. The transverse reinforcement was designed using the NCHRP 12-49 (ATC/MCEER 2001). The lateral reinforcement volumetric ratio of 0.48, 0.40 and 0.53 was provided for 1/10, 1/8 and 1/6 scale columns respectively in order to prevent the longitudinal reinforcement from local and global buckling. The confinement was not a determining criterion for the spiral reinforcement due to lack of axial forces in the columns. For 1/6 and 1/8 scaled columns steel wire with diameter of 0.19 in was applied to supply the spirals. In 1/10 scale column the spiral reinforcement was provide by steel wire with diameter of 0.09 inch. The dimensions and reinforcement of columns are presented in Table 6.2. The bases of columns were attached to the square footings which were designed to resist the maximum probable moment and shear forces at the bottom of columns in the seismic tested events. The footings were fixed rigidly to the steel frame in the laboratory through the steel anchorage bars.
The material properties of the concrete, steel longitudinal reinforcement and the spiral wires are presented in Table 6.5. The maximum aggregate size of concrete in 1/6 and 1/8 scale columns was 3/8 inch while for the concrete of 1/10 scale columns aggregate with the maximum size of 3/16 inch was used. The concrete in both columns and foundations was designed to reach the compressive strength of 5000 psi after 28 days. Three 6 inch by 12 inch cylinder samples of concrete were cast from each column and foundation during the time of construction. The compressive strengths of cured concrete cylinders in the day of testing were in a range from 5000 psi to 7000 psi.
Figure 6.3: Reinforcement and dimensions of specimens (in inch)
Table 6.2: Dimensions and reinforcement of specimens

<table>
<thead>
<tr>
<th>Scale</th>
<th>Diameter</th>
<th>Clear Height</th>
<th>Longitudinal Reinforcement</th>
<th>Transverse Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/10</td>
<td>4.8</td>
<td>24</td>
<td>6 rebars diameter 0.25</td>
<td>wire diameter 0.09 @ 1.3</td>
</tr>
<tr>
<td>1/8</td>
<td>6</td>
<td>30</td>
<td>6 rebars diameter 3/8</td>
<td>wire diameter 0.19 @ 5.2</td>
</tr>
<tr>
<td>1/6</td>
<td>8</td>
<td>40</td>
<td>8 rebars diameter 3/8</td>
<td>wire diameter 0.19 @ 2.9</td>
</tr>
</tbody>
</table>

Table 6.3: Material properties of specimens

<table>
<thead>
<tr>
<th>Material</th>
<th>Specified (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>yield strength</td>
</tr>
<tr>
<td>Longitudinal steel bars</td>
<td>68000</td>
</tr>
<tr>
<td>Spiral steel wires</td>
<td>70000</td>
</tr>
<tr>
<td>Concrete</td>
<td>5000-7000</td>
</tr>
<tr>
<td></td>
<td>compressive strength</td>
</tr>
</tbody>
</table>

6.1.4 Numerical part of the hybrid models

The numerical parts of the hybrid models consisted of all components of the bridge except one column in the shortest bent. The analytical portions of the hybrid simulations were modeled in OpenSees program. For all three scales of 1/6, 1/8 and 1/10 the numerical parts of the simulations were scaled down accordingly based on the introduced similitude laws. The details of the numerical parts of the simulations are the same as nonlinear finite element model of the bridge described in chapter 4.

6.1.5 Communication between numerical part and experimental part of the hybrid models

As detailed in chapter 5, OpenFresco (Open-Source Framework for Experimental Setup and Control) middleware was used in this research to bridge between the finite element program (OpenSees) and the experimental control (Schellenberg and Mahin 2006). The testing facility at University of Illinois at Chicago utilizes the MTS-TestStar controller for large-scale structural
experiments. OpenFresco provides an experimental control object in order to communicate with the MTS controller software which is MTS-795. At this step the controller software MTS-793 requires an additional programming interface to connect with the experimental control object of OpenFresco. The Computer Simulation Interface (CSI) which is the MTS 793 high level programming interface in C++ and Visual Basic provides the possibility for MTS 793 to interact with OpenFresco.

6.1.6 Loading pattern

Two sequences of loadings were employed during the hybrid simulations on the scaled models of the bridge. First gravity loads were applied and then the followed by earthquake motions. High-amplitude motions were applied to the bottom of the base of the hybrid models in a number of successively increasing steps from Test 12 (0.075g peak ground acceleration) to Test 19 (1.66g peak ground acceleration). Table 6.4 presents the input earthquake amplitudes for the hybrid models. The time-history of accelerations applied to the base of the hybrid models are depicted in figures 6.4 through 6.11. The ground motions were scaled in time for each scale of hybrid models.
Table 6.4: Input earthquake amplitudes for the hybrid models

<table>
<thead>
<tr>
<th>Event</th>
<th>PGA (g)</th>
<th>Average maximum measured acceleration of shaking tables</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>0.075</td>
<td>0.076</td>
</tr>
<tr>
<td>13</td>
<td>0.15</td>
<td>0.14</td>
</tr>
<tr>
<td>14</td>
<td>0.25</td>
<td>0.19</td>
</tr>
<tr>
<td>15</td>
<td>0.5</td>
<td>0.56</td>
</tr>
<tr>
<td>16</td>
<td>0.75</td>
<td>0.92</td>
</tr>
<tr>
<td>17</td>
<td>1.00</td>
<td>0.85</td>
</tr>
<tr>
<td>18</td>
<td>1.33</td>
<td>1.40</td>
</tr>
<tr>
<td>19</td>
<td>1.66</td>
<td>1.72</td>
</tr>
</tbody>
</table>

Figure 6.4: Ground motion, transverse bridge direction, event 12
Figure 6.5: Ground motion, transverse bridge direction, event 13

Figure 6.6: Ground motion, transverse bridge direction, event 14
Figure 6.7: Ground motion, transverse bridge direction, event 15

Figure 6.8: Ground motion, transverse bridge direction, event 16
Figure 6.9: Ground motion, transverse bridge direction, event 17

Figure 6.10: Ground motion, transverse bridge direction, event 18
6.2 Instrumentation of reinforced concrete specimens

The lateral response of the concrete columns is dominated by the bending behavior. The plastic hinges form at the bottom of the columns after the columns undergo large lateral displacements. Most of the cracks are expected to form at the plastic hinge region. In order to investigate the damage progress in the plastic hinge region, three types of Fiber optic Bragg Grating (FBG) sensors were employed to monitor the displacements and strains in each distinct part of the reinforced concrete column section. The arch-shaped displacement sensors were adhered to the external surface of concrete cover while the internal crack sensors were embedded inside the columns cores. The FBG strain gauges were mounted on specific locations on the longitudinal reinforcement bars. The sensors were installed across the diameter of the circular section in the direction of the exerted loading to capture the redistribution of strains in the plastic
hinge area. Each pair of sensors was used to measure the column curvature in the plastic hinge region. Since the columns were loaded through a pin connection incapable of transferring bending moments, zero strains and consequently zero curvatures were expected at the top of columns. Two arch-shaped sensors were installed at the top of each column to ascertain this fact and monitor the probable curvatures. Figure 6.12 displays the locations of sensors along the height of columns for each scaled column. The sensor arrangements across the circular sections of scaled columns are detailed in Figure 6.15.
Figure 6.12: Sensors displayed along the height of columns for each scaled column

(The distances are all in inches)
Figure 6.13: Sensor arrangements across the circular sections of scaled columns

(The distances are all in inches)
6.3 Construction of reinforced concrete specimens

The construction procedure of the reinforced concrete specimens was similar to the actual practice in the field. Six steps were distinguishable in the construction process of specimens: construction of column and foundation reinforcement cages, instrumenting the column longitudinal bars with fiber optic bragg grating strain gauges, preparation of the foundation forms, casting the concrete of foundation, installing the internal crack sensors and casting the concrete to form the column.

All the steps of construction were performed at University of Illinois at Chicago laboratories. The steel reinforcement cages of columns and foundations were constructed separately. The spirals for all column sizes were manufactured in the machine shop from the steel wires. The fiber optic bragg gratings were attached to the steel longitudinal bars at certain locations and then protected by the VISHAY adhesive resin as well as strain gauge protection bandages (Figure 6.14). The spirals and the longitudinal rebars were connected together by tying wires. The foundation forms were made out of wood for different sizes of footings. There were four holes drilled on the bases of foundation forms in order to hold up the tubes which were used for keeping the passages of anchorage bolts open (Figure 6.15). The reinforcement cages were installed inside the foundation forms and lined up vertically by the means of leveling strings (Figure 6.16, 6.17). The concrete for the foundations was mixed and poured in the forms at the concrete mixing laboratory (Figure 6.18, 6.19). For each column, when the foundation concrete set, two internal crack sensors were installed in a pretensioned state inside the column cage (Figure 6.20). Sonotube concrete forms were placed over the column cages as a framework for casting concrete. Wooden frames with opening matching column diameters were manufactured to hold the column forms and straighten them vertically. The fiber optic cables of both strain
gauges and internal crack sensors were pulled out through the holes drilled on the sides of the sonotubes (Figure 6.21). The signals from fiber optic internal crack sensors were monitored prior and following casting of columns in order to ascertain the pretensioned state of the sensors (Figure 6.22).
Figure 6.14: Fiber optic bragg grating sensors on longitudinal rebars

Figure 6.15: Concrete forms and anchorage pipes
Figure 6.16: Reinforcement cages installed in the forms for 1/10 scaled column

Figure 6.17: Reinforcement cages installed in the forms for 1/8 and 1/6 scaled columns
Figure 6.18: Concrete of foundation (Scale 1/10)

Figure 6.19: Concrete of foundation (Scale 1/8 and 1/6)
Figure 6.20: Internal crack sensors installed in the reinforcement cage

Figure 6.21: Column forms, wooden frame and fiber optic pigtails
Figure 6.22: Post-construction control of internal crack sensors
6.4 Test Setup

A self-equilibrium steel reaction frame was designed and manufactured to provide the support for the concrete columns and the actuator applying the loadings to the top end of the columns (Figure 6.23). A 110 kip actuator with stroke capacity of 6 inches was used to apply the loads to the concrete columns. The actuator was driven by a 15 gpm Moog servo valve and the servo valve was attached in a line to a 3000 gmp hydraulic oil supply. A load cell with the capacity of ±110 kip was connecting the actuator to the specimens in order to measure the resisting forces. An LVDT (linear voltage differential transformer) with displacement range of ±3 inches was employed to measure the displacements at the top of the columns and to return the feedback required by the servo-hydraulic system and the hybrid models.

Figure 6.23: Experimental setup for concrete column tests
The actuator was moved along the height of the reaction frame in three different levels to accommodate testing of the three different column sizes due to the columns in different scales. Figure 6.24 displays the test setup and actuator positions for the three different sizes of reinforced concrete columns in this research.

![Actuator positions for the three different scales of columns](image)

In the bridge prototype, shaking table and finite element models, each column is interacting with the rest of the structure through six degrees of freedom, namely three displacements and three rotations (Figure 6.25). During the hybrid simulation, all these degrees of freedom could be controlled by the means of appropriate setup and actuators at the common point connecting between the physical and analytical portions of structure. However, for all practical purposes the transverse displacement (Uy) was chosen as the major degree of freedom to be controlled experimentally. Since the bridge model was loaded solely in the transverse direction (y-direction), the displacement in the longitudinal direction (Ux) and the rotation in the transverse direction (θy) as well as the tortional rotation (θz) could be neglected within acceptable limits. In order to consider the effects of rotation around the longitudinal axis and the displacements along the vertical direction in the global response of the structure two additional experimental elements were used in the hybrid model. Instead of communicating with experimental transfer system (controller and data acquisition system), the new experimental elements were controlled by a
simulation based experimental control object in OpenFresco. The control object is called SimUniaxialMaterial and provides the possibility to define a virtual load cell and actuator which are interacting based on a uni-axial material model in opensees.

Figure 6.25: Six degrees of freedom a through which each column interacting with the rest of structure
6.5 Test Results

The results pertaining to the validation of hybrid simulation are presented in this chapter. Post-seismic analysis of results from shaking table and hybrid simulation tests were used to validate the hybrid simulations. Displacement response of the bents as measured by the wire transducers and simulated with the hybrid models are compared for three different scales in Figures 6.26 to 6.79. Tables 6.5 to 6.13 present the maximum and minimum bent displacements during the seismic events experienced by each hybrid model in various scales. Moreover the percent differences between the maximum and minimum bent displacements obtained from hybrid simulations and the shaking table tests for each event are listed in Tables 6.5 to 6.15.

In order to investigate the effect of scaling in accuracy of hybrid simulations, an accuracy index (AI) was defined based on the average of percent differences between the maximum and minimum bent displacements from hybrid simulation and shaking table test. The averaging was carried out on all bents and hybrid models from the same scale so that a single accuracy index could represent each scale. Table 6.14 demonstrates the development of accuracy index for all the different scales of the hybrid models throughout the progressive seismic events. The last column in table 6.14 list the accuracy indices calculated from finite element analysis of the shaking table model for all seismic events. This column was added to the table in order to provide a basis to evaluate the scaled hybrid simulations in comparison to pure finite element analysis. The graph in Figure 6.80 illustrates the variation of the accuracy indices with respect to the increase in scale for progressive seismic events. As can be seen in this figure, the increase of scales generally increases the accuracy of hybrid models. Scale 1/6 mimics the shaking table data more successfully under both low amplitude and high amplitude motions. Scale 1/6 is even more accurate than the pure finite element analysis of the actual bridge size on the shaking table, in
high amplitude motions. This can be related to the capability of the experimental elements in hybrid models in predicting the response of the most vulnerable elements of the bridge models more realistically. Typical sensor data obtained from arch-shaped sensors, concrete embedded sensors and steel strain sensors pertaining to specimen 3 of 1/8 scaled columns in event 14 are plotted in Figure 6.81 to 6.86.
Figure 6.26: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 1, event 13

Figure 6.27: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 2, event 13
Figure 6.28: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 3, event 13

Figure 6.29: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 1, event 14
Figure 6.30: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 2, event 14

Figure 6.31: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 3, event 14
Figure 6.32: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 1, event 15

Figure 6.33: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 2, event 15
Figure 6.34: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 3, event 15

Figure 6.35: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 1, event 16
Figure 6.36: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 2, event 16

Figure 6.37: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 3, event 16
Figure 6.38: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 1, event 18

Figure 6.39: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 2, event 18
Figure 6.40: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 3, event 18

Figure 6.41: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 2, event 19
Figure 6.42: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/10, specimen 3, event 19

Figure 6.43: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 1, event 13
Figure 6.44: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 2, event 13

Figure 6.45: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 3, event 13
Figure 6.46: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 1, event 14

Figure 6.47: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 2, event 14
Figure 6.48: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 3, event 14

Figure 6.49: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 1, event 15
Figure 6.50: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 2, event 15

Figure 6.51: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 3, event 15
Figure 6.52: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 1, event 16

Figure 6.53: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 2, event 16
Figure 6.54: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 3, event 16

Figure 6.55: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 1, event 17
Figure 6.56: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 2, event 17

Figure 6.57: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 3, event 17
Figure 6.58: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 1, event 18

Figure 6.59: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 2, event 18
Figure 6.60: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 3, event 18

Figure 6.61: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 1, event 19
Figure 6.62: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 2, event 19

Figure 6.63: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/8, specimen 3, event 19
Figure 6.64: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 1, event 13

Figure 6.65: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 2, event 13
Figure 6.66: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 1, event 14

Figure 6.67: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 2, event 14
Figure 6.68: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 1, event 15

Figure 6.69: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 2, event 15
Figure 6.70: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 3, event 15

Figure 6.71: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 1, event 16
Figure 6.72: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 2, event 16

Figure 6.73: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 1, event 17
Figure 6.74: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 2, event 17

Figure 6.75: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 3, event 17
Figure 6.76: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 1, event 18

Figure 6.77: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 2, event 18
Figure 6.78: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 1, event 19

Figure 6.79: Comparison of transverse displacements of bents from Hybrid simulation and shaking table test, scale 1/6, specimen 3, event 19
Table 6.5: Comparison of maximum and minimum bent displacements measured from shaking table test and calculated from hybrid simulation (specimen 1, scale 1/10)

### Bent 1

<table>
<thead>
<tr>
<th>Event</th>
<th>Hybrid Simulation</th>
<th>Shaking Table</th>
<th>% Difference from Shaking Table</th>
<th>Hybrid Simulation</th>
<th>Shaking Table</th>
<th>% Difference from Shaking Table</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>0.19</td>
<td>0.24</td>
<td>19.52</td>
<td>-0.19</td>
<td>-0.25</td>
<td>21.74</td>
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<tr>
<td>14</td>
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<td>58.52</td>
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<td>-0.29</td>
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<tr>
<td>15</td>
<td>0.52</td>
<td>0.46</td>
<td>11.85</td>
<td>-0.35</td>
<td>-0.61</td>
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<tr>
<td>16</td>
<td>0.72</td>
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<td>-0.86</td>
<td>6.23</td>
</tr>
<tr>
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<td>-0.90</td>
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<td>-1.09</td>
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</tr>
<tr>
<td>Average</td>
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<td></td>
<td></td>
<td></td>
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</tr>
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### Bent 2

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<th>% Difference from Shaking Table</th>
<th>Hybrid Simulation</th>
<th>Shaking Table</th>
<th>% Difference from Shaking Table</th>
</tr>
</thead>
<tbody>
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<td>13</td>
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<td>19.07</td>
<td>-0.19</td>
<td>-0.17</td>
<td>11.79</td>
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Table 6.6: Comparison of maximum and minimum bent displacements measured from shaking table test and calculated from hybrid simulation (specimen 2, scale 1/10)

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Table 6.7: Comparison of maximum and minimum bent displacements measured from shaking table test and calculated from hybrid simulation (specimen 3, scale 1/10)

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Table 6.8: Comparison of maximum and minimum bent displacements measured from shaking table test and calculated from hybrid simulation (specimen 1, scale 1/8)

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Table 6.9: Comparison of maximum and minimum bent displacements measured from shaking table test and calculated from hybrid simulation (specimen 2, scale 1/8)

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Table 6.10: Comparison of maximum and minimum bent displacements measured from shaking table test and calculated from hybrid simulation (specimen 3, scale 1/8)

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Table 6.11: Comparison of maximum and minimum bent displacements measured from shaking table test and calculated from hybrid simulation (specimen 1, scale 1/6)

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Table 6.12: Comparison of maximum and minimum bent displacements measured from shaking table test and calculated from hybrid simulation (specimen 2, scale 1/6)

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Table 6.13: Comparison of maximum and minimum bent displacements measured from shaking table test and calculated from hybrid simulation (specimen 3, scale 1/6)

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<th>Shaking Table</th>
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<td>Hybrid Simulation</td>
<td>Shaking Table</td>
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<th>% Difference from Shaking Table</th>
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<th>Shaking Table</th>
<th>% Difference from Shaking Table</th>
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Table 6.14: Development of the Accuracy Index (AI) for various scales and events

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Figure 6.80: Variation of the accuracy indices with respect to the increase in scale for progressive seismic events
Figure 6.81: Average strain on the left side of the concrete cover in plastic hinge area obtained from arch-shaped sensor, specimen 3, scale 1/8, Event 14

Figure 6.82: Average strain on the right side of the concrete cover in plastic hinge area obtained from arch-shaped sensor, specimen 3, scale 1/8, Event 14
Figure 6.83: Strain on the left longitudinal steel bar in plastic hinge area, specimen 3, scale 1/8, Event 14

Figure 6.84: Strain on the right longitudinal steel bar in plastic hinge area, specimen 3, scale 1/8, Event 14
Figure 6.85: Average strain in the left side of the concrete core in plastic hinge area obtained from concrete embedded sensor, specimen 3, scale 1/8, Event 14

Figure 6.86: Average strain in the right side of the concrete core in plastic hinge area obtained from concrete embedded sensor, specimen 3, scale 1/8, Event 14
VII. Damage Assessment Methodologies

In order to achieve the damage assessment of concrete bridge structures equipped by structural health monitoring systems, it is desirable to develop deterministic methodologies utilizing the sensors data to generate quantitative information on different damage states of a structure. To accomplish this aim, it is required to define engineering limit state which can be expressed by limiting values of quantities such as maximum strains or damage indices (Ansari 2005). Since the post earthquake damages in modern reinforced concrete bridges initiate and progress at ductile hinge regions in the columns, the damage assessment procedures introduced in this research are founded on the employment of data from few number of sensors installed in this zone (Iranmanesh et al. 2008). The concentration of this study is on small and moderate damage which are invisible or barely visible from outside. The first part of this chapter lists the sequence of damage experienced by different columns in the experimental program. In the second part, a novel methodology is developed to evaluate the energy dissipation in a reinforced concrete column with circular cross section based on the curvatures measured in the plastic hinge area. The introduced method provides the possibility to compute the energy-based damage indices independent from the force values. This is important in practical applications since measurement of force is not always a pragmatic possibility for bridges. The last part of this chapter attempts to investigate the commencement and progress of damage based on the post-event redistribution of strains along the cross section of the columns in the plastic hinge area.

7.1. Damage progression

The sequence of damage was similar for 1/6 and 1/8 scaled columns. In the order of first occurrence, cracking of concrete cover, yielding of longitudinal steel and spalling of concrete cover were the distinguishable damage states. The damage pattern in the 1/10 scaled column was
slightly different. In the 1/10 scaled columns the cracking in the concrete cover was accompanied by large crack opening at the bottom of column. Spalling of concrete was not observed in the sequence of damage. In fact the ultimate failure mode of 1/10 scale columns was tension-flexural while the ultimate failure mode of 1/6 and 1/8 scaled columns was compression-flexural (Park and Ang 1985). The sequence of damage in 1/6 and 1/8 scaled columns was similar to the columns in the shaking table test. These damage states are described in the next sections.

7.1.1 Cracking of cover concrete

The hairline cracks were observable following the first seismic event (Event 13). After propagation of cracks in the next events the distance between the cracks was approximately half of the column diameter. As the successive events were experienced by the columns, in general existing cracks propagate and new cracks form gradually (Figure 7.1 and 7.2).

![Figure 7.1: Distribution of cracks along the height of 1/8 scaled column](image)
Yielding of longitudinal steel reinforcement was measured by the means of fiber optic bragg grating strain sensors mounted on the surface of the extreme rebars in the cross section. In the circular cross sections, yielding initiates from the two extreme rebars in the cross section and then spreads to all adjacent bars. Therefore yielding phenomena causes a gradual softening in the force-displacement response of the column.

7.1.3 Spalling of concrete cover

Progress in height and width as the column undergoes larger displacement. For the 1/6 and 1/8 scaled columns spalling launches at event 18 (Figure 7.3) and spreads in event 19 (Figure 7.4). Spalling of concrete terminates the performance of arch shaped external sensors mounted on the surface of cover concrete.
Figure 7.3: Initiation of concrete spalling

Figure 7.4: Progress of concrete spalling
7.2. Evaluation of energy dissipation in concrete columns with circular section

One of the fundamental parameters in evaluating the post-earthquake capacity of reinforced concrete members is the energy dissipation. The amount of energy dissipation can be estimated by nonlinear finite element method or empirical equations in analytical problems (ATC-40 1996). In the experimental or field application the dissipated energy can be calculated provided the availability of measured force and displacement history. Park and Eom (2010) studied the mechanism of energy dissipation in reinforced concrete slender members and proposed a method to reevaluate the dissipated energy. According to their study, the energy dissipation in reinforced concrete members is primarily due to plastic behavior of steel reinforcement rather than concrete. In the research described here, the fundamental approach established by previous researchers for computing the dissipated energy in reinforced concrete columns with circular section is modified and expanded to render more precise results. The proposed formula is validated through the data provided by the sensors installed across the column circular section. The developed method is a field deployable technique for post seismic damage assessment of reinforced concrete bridges equipped by few sensors.

7.2.1. Development of method

The energy dissipated in cyclic behavior of a flexural dominated reinforced concrete column is principally as a result of plasticity in longitudinal steel bars. Figure 7.5 depicts the cyclic behavior of steel bars in plastic regime. The area enclosed by the cyclic stress-strain curve denotes the dissipated energy per volume of steel bar and can be approximated by a parallelogram encompassing the area. Thus the dissipated strain energy, $U$ by a longitudinal steel bar can be expressed as:

$$ U = 2R_b f_y (\varepsilon_1 - \varepsilon_2 - 2\varepsilon_y) \quad (7.1) $$
Figure 7.5: Dissipated energy density in longitudinal reinforcement bars

In which $R_b$ considers the Bauschinger effect and it is assumed to be 0.75 in the present research (Hoehler and Stanton 2006). $f_y$ and $\varepsilon_y$ represent the yield stress and strain of the longitudinal steel reinforcement bars. The maximum and minimum strains per cycle experienced by reinforcing bars are represented by $\varepsilon_1$ and $\varepsilon_2$ respectively. The conditional expression inside the brackets, $(\varepsilon_1 - \varepsilon_2 - 2\varepsilon_y)$, takes a zero value when $\varepsilon_1 - \varepsilon_2 - 2\varepsilon_y < 0$, which indicates no energy is dissipated in cycles where the strain in steel reinforcing bar is less than yield strain in tension and compression. By integrating the dissipation energy density, $U$ over the column cross section in a cyclic course of loading, the dissipated energy in the cross section $e$ can be computed as:

$$e = \int_{\text{steel area}} U \, dA$$  \hspace{1cm} (7.2)
To obtain the value of the expression $\varepsilon_1 - \varepsilon_2$ independent from the location of neutral axis in each cycle a simplified strain profile was suggested by Eom and Park (2010). Figure 7.6

![Diagram of strain profile](image)

Figure 7.6: Dissipated energy pattern in circular cross section of a reinforced concrete column

...demonstrates the details of assumptions for integration of strain energy density over a circular cross section. Based on the simplified strain profile the term $\varepsilon_1 - \varepsilon_2$ can be replaced by the expression $(\varphi^+_m + \varphi^-_m) \frac{D_s \sin \theta}{2}$. Assuming the steel reinforcing bars are distributed continuously in a circle with diameter of $D_s$, the area of steel bars per circumference unit length can be...
obtained as \( \frac{\rho D^2}{4D_s} \) where \( \rho \) is longitudinal reinforcement ratio and \( D \) is the diameter of the column.

Therefore the area of steel over an infinitesimal angle of \( d\theta \) is calculated as \( \frac{\rho D^2}{8} d\theta \). By taking into account all the established assumptions the dissipated energy of a circular cross section \( e \) can be expressed in the integral form of:

\[
e = 4 \int_{0}^{\pi/2} (2R_b f_y) \left( (\phi_m^+ + \phi_m^-) \frac{D_s \sin \theta}{2} - 2\varepsilon_y \right) \frac{\rho D^2}{8} d\theta 
\]

(7.3)

As explained earlier in this chapter, yielding of the longitudinal steel bars in columns with circular cross section is a gradual phenomena initiating from the two extreme bars in the section and then spreading to the adjacent bars. Thus, before the initiation of yielding, the expression \( (\phi_m^+ + \phi_m^-) \frac{D_s \sin \theta}{2} - 2\varepsilon_y \) takes the value of zero and following the start of yielding it returns non-zero values over a specific sector of the cross section. In order to compute the integral expression for cross sectional dissipated energy, Park and Eom assumed the complete yielding of steel reinforcement over the cross section and approximate the integral in equation 7.3 as:

\[
e \approx (4R_b f_y) \left( \frac{\rho D^2}{8} \right) \left( \phi_m^+ + \phi_m^- - 2\frac{\varepsilon y \pi}{D_s} \right) 
\]

(7.3)

Assumption of complete yielding of steel reinforcement over the cross section is not compatible with the fact that yielding of steel reinforcement is a gradual process (Lehman et al. 2004). According to the results of experimental tests on the columns, following moderate seismic excitations there is a considerable amount of dissipated energy while the longitudinal reinforcements are not fully yielded over the section.

In this research, an alternative approach is proposed to compute the integral in equation 7.3. As can be seen in Figure 7.6, the integral can be calculated within the lower limit of angle \( \alpha \) and upper limit of \( \pi/2 \).
\[
e_{II} = 4 \int_{\alpha}^{\pi/2} (2R_b f_y) ((\Phi^+ + \Phi^-) \frac{D_s \sin \theta}{2} - 2\varepsilon_y) \frac{D^2}{8} d\theta
\]

\[
= \left(4R_b f_y\right) \left(\frac{\rho D^2}{8}\right) (D_s) \cos \alpha \left(\Phi^+ + \Phi^- - 2 \frac{\varepsilon_y \pi}{D_s} \left(1 - 2\frac{\rho}{\varepsilon_y}\right)\right)
\]

(7.4)

\(\alpha\) is an angle which separates the yielded reinforcing steel bars from the rest of reinforcement in the section and can be expressed by the following equation:

\[
\alpha = \sin^{-1}\left(\frac{4\varepsilon_y}{(\Phi^+ + \Phi^-)D_s}\right)
\]

(7.5)

In slender reinforced concrete members most of the energy dissipates through the inelastic behaviors in the plastic hinge area, therefore the energy dissipated in the reinforced concrete columns can be obtained by integrating the cross sectional dissipated energy along the height of the plastic hinge. By assuming an average curvature in the plastic hinge area, the total energy dissipated by the cyclic behavior of the column can be estimated as:

\[
E = e \cdot L_p
\]

(7.6)

Where \(L_p\) is the length of plastic hinge which was chosen to be expressed by the following equation in this study (Berry and Eberhard 2007).

\[
L_p = \min \left(0.05L + \frac{0.1f_y d_b}{\sqrt{f_c}}, \frac{L}{4}\right)
\]

(7.7)

Where \(f_y\) and \(f_c\) are yield stress and compressive strength of reinforcing bars and concrete respectively, \(d_b\) represents the diameter of longitudinal reinforcing bars and \(L\) is the length of column.

7.2.2. Validation of method

The curvature is the inverse of the radius of the deflected shape of the column along its length and it can be obtained by the following relationship:
\[ \varphi = \frac{\varepsilon_a - \varepsilon_b}{d} \]  \hspace{1cm} (7.8)

Where \( \varepsilon_a \) and \( \varepsilon_b \) are the strains on the opposite sides of the column and \( d \) is the distance between the sensors which measure \( \varepsilon_a \) and \( \varepsilon_b \). The curvature data calculated from each pair of sensors installed at two opposite sides of the column cross section were used in computation of the dissipated energy with the proposed method. A computer program was developed in Matlab to compute the equations (7.4) to (7.7) on the successive cycles within the time-history seismic response of sensors. In order to validate the proposed method, the computed dissipated energy values through equations (7.4) to (7.7), were compared to the amount of energy dissipation acquired by calculating the area enclosed by the force-displacement hysteresis plots. Typical force-displacement hysteresis plots for specimen 3 of 1/8 scaled columns are shown in Figure 7.7. The graphs in Figures 7.8, 7.9 and 7.10 depict the hysteretic diagrams relating the moment at the bottom of the specimen 3 of 1/8 scaled columns and curvatures obtained from external, steel and internal sensors respectively.

The hysteretic dissipated energy calculated based on the measured force and displacement data is denoted by \( E_I \) and the energy dissipation computed by the proposed method is termed as \( E_{II} \). In Figure 7.27 \( E_I \) for the seismic events experienced by the 1/8 scaled columns are plotted against \( E_{II} \) values which were computed based on the curvatures measured by external sensors. The slope of linear-regression trend line in Fig 7.27 is 1.0028 with R-squared value of 0.96, indicating a one to one correspondence between \( E_I \) and \( E_{II} \). This analysis proves that the proposed method is appropriate for predicting the dissipation of energy in a column with circular cross section during a seismic course of loading with a high precision. Correlation of \( E_I \) and \( E_{II} \) was studied for the 1/6 scaled columns in Figure 7.28. For the 1/6 scaled columns, the number of data points are less due to fewer number of tests and improper
sensor data from some runs. The developed approach can be exclusively applied for flexure-dominated columns as 1/6 and 1/8 scaled specimen in the present research (Park and Eom 2010). Thus, the proposed method cannot be used for 1/10 scaled columns whose behavior is influenced by bond-slip failure mechanism.
Figure 7.7: Force-displacement hysteresis digrams for specimen 3, scale 1/8, Event 13

Figure 7.8: Force-displacement hysteresis digrams for specimen 3, scale 1/8, Event 14
Figure 7.9: Force-displacement hysteresis digrams for specimen 3, scale 1/8, Event 15

Figure 7.10: Force-displacement hysteresis digrams for specimen 3, scale 1/8, Event 16
Figure 7.11: Force-displacement hysteresis diagrams for specimen 3, scale 1/8, Event 17

Figure 7.12: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8 (external sensors), Event 13
Figure 7.13: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8 (external sensors),

Event 14

Figure 7.14: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8 (external sensors),

Event 15
Figure 7.15: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8 (external sensors),

Event 16

Figure 7.16: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8 (external sensors),

Event 17
Figure 7.17: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8 (steel sensors), event 13

Figure 7.18: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8 (steel sensors), event 14
Figure 7.19: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8 (steel sensors),

event 15

Figure 7.20: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8 (steel sensors),

event 16
Figure 7.21: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8 (steel sensors), event 17

Figure 7.22: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8 (concrete embedded sensors), event 13
Figure 7.23: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8
(concrete embedded sensors), event 14

Figure 7.24: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8
(concrete embedded sensors), event 15
Figure 7.25: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8 (concrete embedded sensors), event 16

Figure 7.26: Moment-curvature hysteresis diagrams for specimen 3, scale 1/8 (concrete embedded sensors), event 17
Figure 7.27: Correlation of $E_I$ and $E_{II}$ (Scale 1/8)

Figure 7.28: Correlation of $E_I$ and $E_{II}$ (Scale 1/8)
7.2.3 Analysis of damage based on dissipated energy

Hose et al (1999) specified the association between qualitative description of the damages in reinforced concrete bridge columns and quantification of performance parameters through the five-level performance evaluation framework (Table 7.1). The concentration of this study is on quantification of minor and moderate damages. It is crucial to distinguish the performance level of a bridge when the damages are invisible or barely visible by the visual inspection. The results of experiments in this research reveal that the dissipated energy is a sensitive quantity to minor and moderate damages and can be used as a clarifying indicator. Figure 7.29 presents the finite element push-over analysis and force-displacement envelope pertaining to successive seismic events experienced by three 1/8 scaled columns. All the data points except one point are located within the damage-level range of I (No damage), II (Minor damage) or III (Moderate damage). More advanced damage models take into account the accumulated damage resulted from cyclic loading by including a dissipated energy term in the damage index equations. As discussed in chapter 2, the method developed by Park and Ang (1985) is one of such methods where the damage index is defined as:

\[
dI_{PA} = \frac{\delta m}{\delta u} + \frac{\beta E}{Q_y \delta u} \leq 1
\]  

(7.9)

In this expression, \( Q_y \) = yield strength; \( E \) = irrecoverable hysteretic energy, and \( \beta \) is a constant which depends on structural characteristics of the column (Kunnath et al., 1990) (Bozorgnia and Bertero, 2001). The major advantage of Park and Ang method is its simplicity and extensive calibrations against experimentally observed seismic damage.
Table 7.1: Bridge performance and damage assessment (Hose et al 1999)

<table>
<thead>
<tr>
<th>Level</th>
<th>Damage Classification</th>
<th>Performance Level</th>
<th>Qualitative Performance Description</th>
<th>Quantitative Performance Description</th>
<th>Socio-economic Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>NO</td>
<td>CRACKING</td>
<td>Onset of hairline cracks</td>
<td>Cracks barely visible.</td>
<td>FULLY OPERATIONAL</td>
</tr>
<tr>
<td>II</td>
<td>MINOR</td>
<td>YIELDING</td>
<td>Theoretical first yield of longitudinal reinforcement.</td>
<td>Crack widths &lt; 1mm.</td>
<td>OPERATIONAL</td>
</tr>
<tr>
<td>III</td>
<td>MODERATE</td>
<td>INITIATION OF LOCAL MECHANISM</td>
<td>Initiation of inelastic deformation. Onset of concrete spalling. Development of diagonal cracks</td>
<td>Crack widths 1-2mm. Length of spalled region &gt; 1/10 cross-section depth.</td>
<td>LIFE SAFETY</td>
</tr>
<tr>
<td>IV</td>
<td>MAJOR</td>
<td>FULL DEVELOPMENT OF LOCAL MECHANISM</td>
<td>Wide crack widths/spalling over full local mechanism region.</td>
<td>Crack widths &gt; 2mm. Diagonal cracks extend over 2/3 cross-section depth. Length of spalled region &gt; 1/2 cross-section depth.</td>
<td>NEAR COLLAPSE</td>
</tr>
<tr>
<td>V</td>
<td>LOCAL FAILURE / COLLAPSE</td>
<td>STRENGTH DEGRADATION</td>
<td>Buckling of main reinforcement. Rupture of transverse reinforcement. Crushing of core concrete.</td>
<td>Crack widths &gt; 2mm in concrete core. Measurable dilation &gt; 5% of original member dimension.</td>
<td>COLLAPSE</td>
</tr>
</tbody>
</table>

The second term in the equation proposed by Park and Ang includes the dissipated energy \( E \) normalized by monotonic energy capacity of the column \( (Q_y \delta_u) \). The normalized dissipated energy term which characterizes the cyclic effect in Park and Ang index is expressed as an independent damage index in Equation 7.10:
\[ DI_{NE} = \frac{\beta E}{Q_y\delta u} \quad (7.10) \]

\( DI_{NE} \) was calculated for every specimen per each seismic event. The graphs in Figure 7.30 pertains to the normalized dissipated energy index acquired based on the measured force-displacement hysteresis as well as proposed method against the maximum displacement per event for 1/8 scaled columns. As can be seen in Figure 7.30 the normalized dissipated energy index is sensitive to the change in the damage classification from minor to moderate. The analysis of damage was repeated for the 1/6 and 1/10 scaled columns in Figures 7.31 to 7.34.
Figure 7.29: Push-over analysis and force-displacement envelope (1/8 scaled columns)

Figure 7.30: Normalized dissipated energy index versus maximum displacement of events (1/8 scaled columns)
Figure 7.31: Push-over analysis and force-displacement envelope (1/6 scaled columns)

Figure 7.32: Normalized dissipated energy index versus maximum displacement of events (1/6 scaled columns)
Figure 7.33: Push-over analysis and force-displacement envelope (1/10 scaled columns)

Figure 7.34: Normalized dissipated energy index versus maximum displacement of events (1/10 scaled columns)
7.3. Strain analysis across the column cross section

Bernoulli assumption states that plane cross-sections remain plane after deformations. In case this hypothesis stays valid, the distribution of strains across the diameter of column cross section will stay linear. However when the columns are subjected to seismic events with progressive amplitudes, in the process of initiation of local mechanism and formation of plastic hinge, the strains redistribute due to several reasons. Slippage of steel longitudinal bars is one of the sources of redistribution of strains across the cross section. The strain penetration occurs along longitudinal reinforcing bars that are fully anchored into connecting concrete members and causes the slippage of steel bars along a part of anchoring length (Zhao and Sritharan 2007). The strain penetration and the consequent rotation at the end of column significantly affect the localized strains and curvatures in the plastic hinge region. Propagation of cracks along the height and development of the cracks depth inside the cross section are the other causes which shift the distribution of strains. In the study presented here, the strain distribution across the diameter of cross section was captured through an array of fiber optic Bragg Grating sensors. Fiber optic sensors monitored the deformations in concrete cover, longitudinal steel bars and concrete core at the two opposite sides of the cross section. Figure 7.35 illustrates the convention established for direction of loading during the experiments. The distribution of strains associated to two extreme states of loading during each seismic event is presented for all the scaled specimens in Figures 7.36 to 7.51. All the specimens follow a fairly similar pattern in redistribution of strains. In most of the cases the strains in longitudinal steel do not increase in the same rate as strains in concrete cover and core. This can be primarily attributed to the slippage of steel bars due to strain penetration. The other common phenomena observable in the redistribution of strains is that the rate of increase in strains of concrete core is less than the
strains of concrete cover especially during high amplitude motions. This observation can be interpreted as the gradual growth in width of existing cracks and the increment in number of cracks passing through the gauge length of external arch sensors. The yield strain of steel bars in both tension and compression states as well as the compressive strain corresponding to the crushing of concrete cover are highlighted in all graphs. The compressive strains of cover concrete in 1/10 scaled columns never reach the crushing limit which is compatible with the tension-flexure failure behavior observed from 1/10 scaled columns.
Figure 7.35: Convention established for direction load and displacement
Figure 7.36: Strain distribution across the column cross section at maximum displacement of top
(Scale 1/8- Specimen 3)

Figure 7.37: Strain distribution across the column cross section at minimum displacement of top
(Scale 1/8- Specimen 3)
Figure 7.38: Strain distribution across the column cross section at maximum displacement of top
(Scale 1/8- Specimen 2)

Figure 7.39: Strain distribution across the column cross section at minimum displacement of top
(Scale 1/8- Specimen 2)
Figure 7.40: Strain distribution across the column cross section at maximum displacement of top (Scale 1/8- Specimen 1)

Figure 7.41: Strain distribution across the column cross section at minimum displacement of top (Scale 1/8- Specimen 1)
Figure 7.42: Strain distribution across the column cross section at maximum displacement of top (Scale 1/10- Specimen 1)

Figure 7.43: Strain distribution across the column cross section at minimum displacement of top (Scale 1/10- Specimen 1)
Figure 7.44: Strain distribution across the column cross section at maximum displacement of top
(Scale 1/10- Specimen 2)

Figure 7.45: Strain distribution across the column cross section at minimum displacement of top
(Scale 1/10- Specimen 2)
Figure 7.46: Strain distribution across the column cross section at maximum displacement of top
(Scale 1/10- Specimen 3)

Figure 7.47: Strain distribution across the column cross section at minimum displacement of top
(Scale 1/10- Specimen 3)
Figure 7.48: Strain distribution across the column cross section at maximum displacement of top

(Scale 1/6- Specimen 2)
Figure 7.49: Strain distribution across the column cross section at minimum displacement of top
(Scale 1/6- Specimen 2)
Figure 7.50: Strain distribution across the column cross section at maximum displacement of top

(Scale 1/6- Specimen 1 and 3)

Figure 7.51: Strain distribution across the column cross section at minimum displacement of top

(Scale 1/6- Specimen 1 and 3)
VIII. Conclusion

The scope of investigation in this study included the development of a hybrid simulation based method for post earthquake damage assessment of concrete bridges. The experimental portion of the research involved 1/10, 1/8 and 1/6 scaled hybrid simulations on a two span concrete bridge using progressively increasing amplitudes of the 1994 Northridge earthquake. One of the objectives of the research was to investigate the reliability of different scales of hybrid simulation in mimicking the response of the shaking table tests. Based on the analyzed results, 1/6 and 1/8 scaled hybrid models generally predicted the results of shaking table test more accurately. In high amplitude motions, the 1/6 scaled hybrid model is even more accurate than the non-scaled finite element analysis which can be attributed to more realistically simulation of the shortest bent of the bridge by physical elements. The research presented in this thesis, is the first attempt in the state of the art for application of hybrid simulation in the field of structural health monitoring with concentration on studying the effect of scaling.

Furthermore, a novel fiber optic embedded sensor was designed and manufactured to capture the internal deformations in the concrete core of the columns. The developed sensors along with the external sensor as well as the fiber optic sensors mounted on the steel longitudinal reinforcements were employed to study the post-event redistribution of strains across the cross section of columns in the plastic hinge area. The applied embedded sensors were successful to measure the tensile and compressive strains in the concrete core of the columns, in a consistent pattern with external sensors. The slippage of steel bars and propagation of cracks were among the phenomena pronounced in the graphs related to redistribution of strains.

A new method for computing the dissipated energy in reinforced concrete columns with circular section was introduced. The development of the method was founded on the fact that the Yielding of the longitudinal steel bars in columns with circular cross section is a gradual
phenomena initiating from the two extreme bars in the section and then spreading to the adjacent bars. The proposed method was solely based on the amount of monitored curvatures at the plastic hinge zone and independent from force and displacement measurements. The normalized dissipated energy computed from both force-displacement hysteresis and the introduced method was utilized as a damage analysis tool sensitive to minor and moderate damages. Based on the proposed method and observed damage levels for the different scales of columns investigated in this research, a generalized damage scale concentrating on barely visible damages can be suggested as in Table 8.1.
Table 8.1: Proposed engineering limit states for bridge performance assessment

<table>
<thead>
<tr>
<th>Level</th>
<th>Damage Classification</th>
<th>Performance Level</th>
<th>Qualitative Performance Description</th>
<th>Quantitative Performance Description</th>
<th>Socio-economic Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>NO</td>
<td>CRACKING</td>
<td>Onset of hairline cracks</td>
<td>$0 &lt; DI_{NE} \leq 0.05$</td>
<td>FULLY OPERATIONAL</td>
</tr>
<tr>
<td>II</td>
<td>MINOR</td>
<td>YIELDING</td>
<td>Theoretical first yield of longitudinal reinforcement.</td>
<td>$0.05 &lt; DI_{NE} \leq 0.15$</td>
<td>OPERATIONAL</td>
</tr>
<tr>
<td>III</td>
<td>MODERATE</td>
<td>INITIATION OF LOCAL MECHANISM</td>
<td>Initiation of inelastic deformation. Onset of concrete spalling. Development of diagonal cracks</td>
<td>$0.15 &lt; DI_{NE} \leq 0.35$</td>
<td>LIFE SAFETY</td>
</tr>
<tr>
<td>IV</td>
<td>MAJOR</td>
<td>FULL DEVELOPMENT OF LOCAL MECHANISM</td>
<td>Wide crack widths/spalling over full local mechanism region.</td>
<td>$0.35 &lt; DI_{NE}$</td>
<td>NEAR COLLAPSE</td>
</tr>
</tbody>
</table>
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