Seismic Demands in Precast Concrete Diaphragms

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by

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DEDICATION

To Meg
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\( a_m \) acceleration in model space
\( a_p \) acceleration in prototype space
\( b \) double tee width
\( b_w \) wall width
\( c \) neutral axis depth to extreme compression fiber
\( d \) distance from the diaphragm section centroid to chord reinforcement centroid
\( d_b \) reinforcing bar diameter
\( d_{dia} \) diaphragm depth
\( d_{m,1} \) maximum positive deformation from hysteresis response
\( d_{m,2} \) maximum negative deformation from hysteresis response
\( d_{p,1} \) positive plastic deformation from hysteresis response
\( d_{p,2} \) negative plastic deformation from hysteresis response
\( d_{ult,1} \) positive ultimate ductility factor to strength degradation for hysteresis input
\( d_{ult,2} \) negative ultimate ductility factor to strength degradation for hysteresis input
\( f'_c \) concrete strength
\( f_{pu} \) ultimate stress of a prestressing tendon
\( f_{si} \) initial stress of a prestressing tendon
\( g \) acceleration of gravity
\( h_r \)  height above the ground to the roof
\( h_i \)  height above the ground to the floor level \( i \)
\( h_n \)  height above the ground to the roof
\( k_m \)  stiffness in model space
\( k_p \)  stiffness in prototype space
\( l_p \)  wall plastic hinge length
\( l_w \)  wall length
\( l_m \)  unit length in model space
\( l_p \)  unit length in prototype space
\( l_p \)  plastic hinge length
\( m_m \)  mass in model space
\( m_p \)  mass in prototype space
\( n \)  number of stories
\( r \)  generic post-yield stiffness factor for hysteresis input
\( r_1 \)  positive post-yield stiffness factor for hysteresis input
\( r_2 \)  negative post-yield stiffness factor for hysteresis input
\( t_m \)  unit time in model space
\( t_p \)  unit time in prototype space
\( v_m \)  velocity in model space
\( v_p \)  velocity in prototype space
\( w_i \)  the weight tributary to level \( i \)
\( w_{px} \)  the weight tributary to the diaphragm at level \( x \)
$A_g$  concrete gross section area

$A_m$  unit area in model space

$A_p$  unit area in prototype space

$A_s$  steel area

$A_{s,chord}$  steel area of the chord reinforcement at either end of the diaphragm

$A_{sh}$  wall shear area in analytical modeling

AR  floor aspect ratio

$C_s$  base shear coefficient

$E_c$  modulus of elasticity for concrete

$E_s$  modulus of elasticity for steel

$F_a$  short-period site coefficient (at 0.2 second period)

$F_{cr}$  cracking force

$F_{cr,1}$  positive cracking force for hysteresis input

$F_{cr,2}$  negative cracking force for hysteresis input

$F_i$  the design force applied to level $i$

$F_m$  unit force in model space

$F_p$  unit force in prototype space

$F_{px}$  the diaphragm design force

$F_{resid}$  residual force factor for hysteresis input

$F_V$  long-period site coefficient (at 1.0 second period)

$F_{y,1}$  positive yield force for hysteresis input

$F_{y,2}$  negative yield force for hysteresis input
H  height above the ground to the roof
I  moment of inertia
$I$  the structural importance factor based on Occupancy Category
$I_{\text{eff}}$  diaphragm moment of inertia estimate accounting for the tension stiffening of concrete
$I_g$  gross section moment of inertia
$I_{lb}$  diaphragm moment of inertia based on the lower bound estimate
$I_o$  moment of inertia of the uncracked section for input to the analytical models
$K_1$  positive uncracked stiffness for hysteresis input
$K_2$  negative uncracked stiffness for hysteresis input
$K_{\text{neg}}$  negative stiffness factor for hysteresis input
$K_{u,1}$  positive unloading stiffness for hysteresis response
$K_{u,2}$  negative unloading stiffness for hysteresis response
$L$  floor span length
$M$  system overturning moment
$M_{cr}$  wall overturning moment at cracking
$M_{\text{dia, max}}$  maximum diaphragm midspan moment
$M_u$  wall design overturning moment
$M_{u,\text{dia}}$  diaphragm in-plane design moment
$M_{\text{wall}}$  wall overturning moment
$M_y$  wall overturning moment at idealized yield
$N$  axial load at the base of the wall
\( N_m \)  
wall axial load in model space

\( N_p \)  
wall axial load in prototype space

\( N_{walls} \)  
number of walls at each end of the structure

\( Pinch \)  
pinching factor for hysteresis input

\( PGA \)  
peak ground acceleration divided by the acceleration of gravity

\( R \)  
Response modification factor

\( S_1 \)  
mapped MCE, 5 percent damped, spectral response acceleration parameter at a period of 1-second [1]

\( S_a \)  
acceleration scale factor

\( S_F \)  
force scale factor

\( S_l \)  
length scale factor

\( S_t \)  
time scale factor

\( S_v \)  
velocity scale factor

\( S_{D1} \)  
design, 5 percent damped, spectral response acceleration parameter at a period of 1 second [1]

\( S_{DC} \)  
seismic design category

\( S_{DS} \)  
design, 5 percent damped, spectral response acceleration parameter at short periods [1]

\( S_{M1} \)  
the MCE, 5 percent damped, spectral response acceleration at a period of 1 second adjusted for site class effects [1]

\( S_{MS} \)  
the MCE, 5 percent damped, spectral response acceleration at short periods adjusted for site class effects [1]

\( S_S \)  
mapped MCE, 5 percent damped, spectral response acceleration parameter at short periods [1]
\( T_m \) period in model space
\( T_p \) period in prototype space
\( V \) system shear
\( V_{\text{dia}} \) diaphragm shear
\( V_{\text{wall}} \) wall shear
\( V_u \) wall design shear
\( W \) total seismic weight
\( W_{\text{trib}} \) tributary weight assigned to a wall

\( \alpha \) unloading stiffness parameter for hysteresis input
\( \beta \) deformation offset parameter for hysteresis input
\( \gamma \) location of the resultant lateral force normalized by the roof height
\( \varphi \) curvature
\( \kappa \) stiffness degradation factor for hysteresis input
\( \rho \) redundancy factor
\( \rho_1 \) longitudinal reinforcement ratio
\( \rho_{\text{sec}} \) general secant stiffness factor of yield stiffness to initial stiffness for hysteresis input
\( \rho_1 \) positive secant stiffness factor of yield stiffness to initial stiffness for hysteresis input
\( \rho_2 \) negative secant stiffness factor of yield stiffness to initial stiffness for hysteresis input
\( \rho_m \)  unit density in model space
\( \rho_p \)  unit density in prototype space
\( \Delta_{cr} \)  deformation at cracking
\( \Delta_y \)  deformation at yield
\( \eta_1 \)  first mode contribution factor
\( \eta_h \)  higher mode contribution factor
\( \lambda \)  overstrength factor
\( \phi_i^j \)  modal amplitude at level \( j \) due to mode \( i \)
\( \sigma_m \)  unit stress in model space
\( \sigma_p \)  unit stress in prototype space
\( \Gamma_i \)  modal participation factor
\( \overline{\Omega}_{FMR} \)  floor acceleration magnification factor based on Eqn. 6.8
\( \Omega_{M,\text{dia}} \)  floor acceleration magnification factor based on diaphragm moment
\( \Omega_{V,\text{dia}} \)  floor acceleration magnification factor based on diaphragm shear
\( \Omega_n \)  roof acceleration magnification factor
\( \Omega_o \)  Overstrength factor of the vertical lateral force resisting system
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ABSTRACT OF THE DISSERTATION

Seismic Demands in Precast Concrete Diaphragms

by

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University of California, San Diego, 2010
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Existing methods of analyzing lateral force demands on structures during seismic events do not fully contemplate the amplification caused by diaphragm flexibility. Moreover, commonly used code provisions do not conservatively anticipate floor demands generated in shear wall buildings. In buildings with floors that act as rigid diaphragms, elastic response necessitates the accurate estimation of design forces,
which is currently not provided with an equivalent lateral force analysis. In long span structures with perimeter walls, diaphragm flexibility may result in amplified floor demands. This amplification was addressed while investigating elastic diaphragm demands in precast concrete buildings.

Modal response spectrum analysis formulated the basis of the elastic design. A modified version of the modal first mode reduced method proposed by Rodríguez et al. (2002) was extended to shear wall buildings with diaphragm flexibility. This method conservatively estimates the design forces necessary for elastic diaphragm response at the design earthquake hazard. Diaphragm demands were assessed in a large scope analytical with the proposed design method. Results show this method sufficiently bounds demands for rigid floor systems or for systems whose flexibility is included.

An experimental program supported the analytical study providing in-plane diaphragm stiffness characteristics of a precast concrete building. A complete structural system was tested at 54% scale with unique diaphragm systems on each of its three floors. Hybrid rocking walls minimized residual drift and allowed customized nonlinear wall behavior at different hazards. Extensive testing and abundant instrumentation generated valuable information regarding the seismic performance of precast concrete systems. These results provided the necessary mechanism for analytical model validation and insight on precast concrete diaphragm behavior.
CHAPTER 1 INTRODUCTION

1.1 Background

Building structures resist lateral forces, such as earthquake induced inertia forces, through horizontal and vertical subsystems. Combined, these make up the structure’s lateral force resisting system (LFRS). Precast concrete buildings typically make efficient use of the floor or roof as the horizontal subsystem herein referred to as a diaphragm. Its role is to “tie” the structure’s vertical subsystem together [2], which subjects it to in-plane flexure and shear. This can be considered analogous to a horizontal deep beam [3], and in reinforced concrete diaphragms, the primary components in the analogy are the chord and collector. These resist tension-compression forces and shear forces, respectively. Alternatively, a strut and tie methodology can be used to develop load paths within the diaphragm [4].

To apportion the reinforcement within the diaphragm, a common assumption in the design process is to ignore diaphragm flexibility [3]. This rigid diaphragm assumption is satisfactory for most structural configurations. Long span buildings with vertical LFRS elements on their perimeter, however, may possess sufficient in-plane flexibility that this assumption is not justified. In cases where the assumption is not valid, the diaphragm flexibility must be considered in the distribution of forces to the vertical LFRS [1], [3]. Additionally, diaphragm flexibility changes the dynamic properties of a building as compared to the building when it is considered to have rigid diaphragms. This flexibility creates amplification and attenuation in the seismic demands.
To determine the lateral loads on structures due to earthquakes, three analysis procedures are permitted by code provisions [5], [1]. These are the Equivalent Lateral Force (ELF), Modal Response Spectrum (MRS) and Seismic Response History (SRH) procedures. Each of these is based on nonlinear response within the building’s vertical LFRS. Response modification factors, as indicated by the building code, are correlated with these vertical components’ abilities to reduce elastic forces by plastic deformation. This research focuses on the impact on diaphragm demands in buildings with shear walls as the vertical LFRS.

Plastic hinge formation in shear walls is responsible for the nonlinear deformation accommodating the inelastic design approach. Seismic performance objectives focus on the reinforcement details necessary for sustained plastic hinge formation at the base of the wall. However, there is no clear indication that nonlinear behavior is restricted to the walls. A diaphragm must have the strength and deformation capacity to ensure the intended inelastic deformation is developed in the vertical elements because the response modification factor is directly linked to vertical subsystem [3]. By directly relating the response modification factor to the vertical components, there is an implied expectation that elastic diaphragm behavior is required and, therefore, ensured by the design forces computed with any of the three permitted analysis procedures. Designers rely on the adequacy of these procedures to guarantee elastic behavior at the design forces.

If code provisions are interpreted to permit nonlinearity in the diaphragm, accurate elastic design forces are still necessary. For nonlinear diaphragm response, a ductility factor or a response modification factor independent from the value prescribed
for the vertical LFRS is required. Ductility can be related to the reduced elastic force demand, so accurate elastic diaphragm demands are required for either elastic or nonlinear design.

1.2 Literature review

1.2.1 Analytical investigations

Accounting for diaphragm flexibility in a building changes its assumed dynamic behavior. Prior to the 1994 Northridge earthquake two analytical investigations of diaphragm flexibility in shear wall buildings were conducted, but neither involved seismic demands. Jain [6] presented an analytical procedure for obtaining the dynamic properties of buildings with flexible diaphragms and end walls. However, seismic demands were not investigated. Similarly, Saffarini and Quadaimat [7] investigated the influence of a rigid floor assumption on floor deformations but made no comparison of capacity and demand from seismic loading. Their conclusions were that frame structures conform well to this assumption but error results from application of this assumption with shear wall buildings. The error was correlated with the ratio of in-plane diaphragm stiffness to the stiffness of the vertical LFRS.

Diaphragm amplification due to flexibility was recorded in response to the 1984 Morgan Hill earthquake. Celbi et al. [8] observed midspan floor acceleration magnifications of 4.2 and 5.0 in the N-S and E-W directions of a single story gymnasium instrumented with accelerometers. Floor acceleration magnification is defined as the peak horizontal floor acceleration divided by the peak ground acceleration. Average floor acceleration magnifications of 1.4 and 1.7 were reported at
the diaphragm ends in the N-S and E-W directions, respectively. Although the
gymnasium roof was not a precast concrete diaphragm, the observations are consistent
with the expectation that diaphragm flexibility in structures with reinforced concrete
walls can result in significant floor acceleration magnifications.

The 1994 Northridge earthquake is often cited as the primary motivator for
investigating seismic performance of precast concrete buildings. The reconnaissance
report by Corley et al. [9] states that “Of the many hundreds of garages in the Los
Angeles area, the vast majority had little or no damage, and eight had partial or total
collapse.” Significant damage was observed in approximately 20 parking structures [9].
This earthquake highlighted the importance of adequate strength to ensure the intended
mode of nonlinear deformation develops and prompted significant research endeavors
to resolve the unacceptable performance.

Some concerns were quickly addressed. Design considerations such as welded
wire reinforcement details and shear capacity mechanisms were addressed by Wood et
al. [10] because of observed damage. An industry driven research effort [11] was
mounted to address concerns on precast concrete diaphragm design.

With the underlying premise that the ELF procedure underestimates seismic
demand in shear wall buildings [12], [13], [14], [15], the primary research focus has
been on the performance consequences of a design using it or a more appropriate design
force level and pattern. The primary consequences of a design to ELF estimations are
large demands on diaphragm ductility [14] and on lateral displacements of the vertical
components of the gravity load system [16], [17].
Fleischman and Farrow [13] observed that force demands can be larger than anticipated by current design procedures. Significant floor demands at lower levels exist, which is contrary to the force distribution assumed in the code provisions.

Fleischman et al. [16] found drift demands on gravity columns are larger than anticipated due to flexible diaphragm deflections under in-plane loading. They recommended elastic diaphragm design due to inadequate seismic response.

Lee et al. [17] concluded that significant modal correlation in buildings with flexible diaphragms due to the closely spaced modes. This renders a square root of the sum of the squares technique for modal analysis ineffective. They found the interstory drift demands on rigid diaphragm buildings with shear walls were acceptable, but that diaphragm flexibility has a significant modification to the structural dynamics resulting in unacceptable drift demands when using the ELF procedure. They proposed a method to predict interstory drifts in low-rise perimeter shear wall buildings.

Barron and Hueste [18] looked at the impact of diaphragm deformation on the structural response. They concluded that a rigid diaphragm assumption is adequate for design, but a flexible diaphragm model should be considered for floor aspect ratios greater than 3.0. Zheng and Oliva [19] developed a simplified deflection analysis procedure for untopped double tee systems designed for elastic behavior.

1.2.2 Diaphragm design recommendations

For design purposes or analytical investigations, flexural effective stiffness factors, the ratio of the mobilized moment of inertia to gross section moment of inertia, have been proposed. Nakaki [20] accounts for uniform and discrete crack patterns in her
formulation, which accounts for the web and chord reinforcement ratios as the primary variables. Values ranged from 0.05 to 0.4 for chord reinforcement ratios from 0.0 to 0.005 at minimum web reinforcement. The need for a stiffness formulation accounting for a discrete crack pattern is justified by the observation made by Wood et al. [10] of concentrated crack patterns that coincided with joints between precast units. Finite element analyses have produced stiffness factors in the same range [21].

For beneficial attributes or to avoid unfavorable performance in nonlinear response, elastic diaphragm designs have most often been recommended. However, a nonlinear design procedure was outlined in detail by Englekirk [22]. He recognizes higher mode contributions to the floor acceleration, and estimates a diaphragm ductility factor based on the level of detailing provided and an assumed diaphragm damping ratio. Based on analytical findings, Lee et al. [21] recommended linear and nonlinear design force profiles. For the nonlinear design proposal, the vertical profile of lateral loads accounts for the $0.4S_{DSI}$ from the first floor to the midheight of the building. From the midheight to the roof diaphragm a linear variation from the $0.4S_{DSI}$ to $0.8S_{DSI}$ is recommended. Their recommendation for strength is independent of diaphragm flexibility.

The simplicity of the ELF procedure makes it a preferred choice for building designers. Therefore, elastic diaphragm design recommendations have included a force amplification factor applied to the ELF procedure diaphragm forces. Nakaki [20] recommended a diaphragm amplification factor equal to the vertical LFRS overstrength factor, $\Omega_0$. 
Fleischman et al. [14] studied the interrelation of diaphragm flexibility and strength in perimeter wall LFRS structures. They observed that diaphragm force amplification is not present in frame structures. They found large ductility demands are present in nonlinear diaphragms. A recommend design can be obtained through the relationships that they developed between diaphragm overstrength, ductility, and flexibility.

Lee et al. [21] proposed two vertical design force profiles for elastic design. One profile option is equal design strength up the height of the building at a floor acceleration of $1.2S_{DSI}$, or three times the peak ground acceleration. Their second design profile varies linearly from $1.2S_{DSI}$ at the first story to $0.6S_{DSI}$ at the midheight and varies linearly from there to $1.2S_{DSI}$ at the roof diaphragm.

Fleischman and Wan [23] looked parametrically at the influence shear overstrength factors have on the diaphragm performance. Their work resulted in shear overstrength factors and required deformation capacities in the chord reinforcement for diaphragm ductility factors at different aspect ratios. Their conclusions included that web connectors can provide significant strength and stiffness to the diaphragm’s flexural behavior and nonductile shear failure will occur for systems without sufficient shear overstrength,

For the design of shear wall structures with rigid diaphragms, Rodríguez et al. [12] proposed a simplified MRS analysis termed the modal “first mode reduced” (FMR) method. A square root sum of the squares (SRSS) approach was used for combining the modal accelerations. The modal FMR method was shown to adequately estimate the
design floor horizontal accelerations resulting from analytical investigations [12] and from experimental shake table tests on small-scale reinforced concrete buildings [24].

While the ELF procedure and rigid diaphragm assumption were found to be inadequate for shear wall buildings other LFRS, specifically reinforced concrete frame systems, may be adequately designed using this method [7], [14]. This suggests that the main design flaw is related to the LFRS rather than a flexible diaphragm condition.

Further support for this concept was provided by Rodríguez et al. [12] who found acceleration demands in buildings with rigid diaphragms in excess of those estimated through the ELF procedure. Therefore, shear wall response not diaphragm flexibility is a likely primary cause for larger than expected diaphragm forces. The extent to which amplification results from diaphragm flexibility is the contribution made by this research.

1.2.3 Experimental programs
1.2.3.1 Precast concrete buildings
1.2.3.1.1 System tests

Only two precast concrete structural systems have been tested experimentally. These were tested under pseudo-dynamic or quasi-static cyclic lateral loading conditions. Priestley et al. [25] tested a 60% scale five-story building as part of the Precast Seismic Structural Systems (PRESSS) research program. Rodríguez and Blandon [26] tested a 50% scale two-story building. The primary focus of each of these tests was on system behavior not on diaphragm performance. However, both test structures observed cracking between floor units. The PRESSS building had good
performance between floor panels despite nonlinear deformation in connectors between double tee units. Rodríguez and Blandon observed wwr fracture in a critical shear region of the diaphragm next to the wall.

1.2.3.1.2 Double tee connector tests

For test setup considerations, double tee flange-to-flange connector tests [27], [28], [29], [30], [31], [32], [33] are conducted on panels with single connectors rather than actual double-tee units. Typically monotonic tension and shear behaviors are obtained along with tests to quantify cyclic response. These component tests are a critical first step in characterizing joint behavior, which allows strength and effective stiffness of the diaphragm system to be quantified for analytical models such as those generated for this research. Recommendations and tests by Naito and Cao [32] related to connector performance were relied upon for the analytical models developed.

1.2.3.1.3 Hollow-core tests

Tests on untopped hollow-core units have been conducted to quantify their horizontal diaphragm shear capacity [34], [35]. Davies et al. [35] observed that shear friction is a primary component for shear capacity but after slip, dowel action acts as a secondary mechanism. Innovative ways to increased shear capacity between hollow-core members have been proposed and tested. These include adding deformations to the joint [36] and providing a carbon fiber reinforced polymer connection [37]. Several tests on hollow-core units for seating length support during effects of seismic response were conducted in New Zealand [38], [39]
1.2.3.2 Reinforced concrete buildings

1.2.3.2.1 Shake table tests

Shake table tests of complete structural systems are rare and typically conducted at small scale due to limitations of the table dimensions. When large or full-scale [40] tests are conducted, diaphragm action is typically not considered to accommodate this limitation. Of the small scale tests available, only a 17% scale test on a single story structure by Panahshahi et al. reported nonlinear diaphragm behavior [41].

1.3 Objectives

As a part of the Diaphragm Seismic Design Methodology (DSDM) research project funded by the Precast/Prestressed Concrete Institute (PCI) and the National Science Foundation, this research was intended to support one of the project’s design deliverables: “An appropriate diaphragm design force pattern and design force levels that target elastic [Design Basis Earthquake] DBE response.” [44]. The primary objective of this research was to provide the design method by which the force pattern and force levels could be obtained. This is formulated in CHAPTER 2.

1.3.1 Analytical investigation

The analytical component of this research was conducted in support of this primary objective. It involved the earthquake simulations of multi-degree of freedom (MDOF) models of precast concrete buildings designed to the force levels for elastic diaphragm response. The SRH analysis method permitted [1] as a means for obtaining structural demands was implemented. The objective of the analytical study was to
validate the application of the proposed design method on buildings with flexible and rigid diaphragms. Simplified models of generic buildings were implemented in this study to facilitate variation of several important design parameters.

1.3.2 Experimental work

Experimental research, involving shake table testing of a precast concrete building, was conducted for validation of the analytical study. A three story structure was tested under static and dynamic conditions to observe system behavior. Objectives for the experimental research were to provide information on the (1) vertical distribution of lateral load along the structure, (2) force path within floor diaphragms, and (3) hysteretic characteristics of precast concrete diaphragms under realistic boundary conditions.

1.4 Methodology

Based on the success of the MRS procedure in shear wall buildings with rigid diaphragms, this research will focus on its application to assess the demands on buildings with diaphragm flexibility. The scope of this research relates to long span, multi-story precast concrete buildings with perimeter shear walls. Of configurations typically used, this layout of vertical LFRS has the largest influence on structural response. To coincide with current seismic performance objectives related to the wall response and to conform to the implied elastic diaphragm design, diaphragm design forces were conservatively estimated for elastic response.
1.5 Thesis layout

CHAPTER 1 discusses the accepted and proposed alternative analysis procedures for estimating diaphragm forces in buildings. The formulation of a previously developed simplified approach to the modal response spectrum procedure is reviewed. Modifications to this formulation are presented, which results in the proposed method for elastic diaphragm force estimation.

In CHAPTER 3, design ground motions are sourced and scaled to design hazard spectra for four sites in the United States. Site specific record sets consisting of ten records were utilized for input to the analytical investigation of CHAPTER 5. Selected records from three of these sites were also used in the experimental shake table testing discussed in CHAPTER 4.

CHAPTER 4 presents the design, scaling, construction, and results of a three-story precast concrete structure and tested with input ground motions. An overview of the post-processing procedures and instrumentation metadata necessary for results interpretation is provided. Brief outcomes of each input ground motion test are provided and relevant pre-test repair details discussed. Processed results include system, wall, and diaphragm demands.

Validation of the analytical models is included in CHAPTER 5. The model development and a comparison of results for three of the shake table tests are provided. The validation results for each comparison include system, wall, and diaphragm demands.

In CHAPTER 6, the prototype structures, modeling approach, model description and results of the analytical study are discussed. Design considerations for the
diaphragm and shear walls used as the vertical LFRS are included in the discussion of the prototype structures. Nonlinear modeling of the shear walls relied on a hysteresis rule for reinforced concrete members developed in CHAPTER 7.

The development of an empirical hysteresis rule for reinforced concrete members is presented in CHAPTER 7. Validation of the model is presented in terms of its ability and inability to capture experimental test results of reinforced concrete members.

In CHAPTER 8, a summary of the research is provided. Based on the results, the main conclusions drawn, and recommendations for remaining future work are provided.
CHAPTER 2 DIAPHRAGM DESIGN FORCE ESTIMATION

2.1 Equivalent lateral force procedure

Methods available for diaphragm force estimation include the equivalent lateral force (ELF), Modal Response Spectrum (MRS), and Seismic Response History (SRH) procedures. The ELF procedure distributes the design base shear to floors in a building according to a first mode response. Floor forces are distributed to diaphragm forces to account for non-concurrence of maximum loading [12]. This is captured in ASCE 7 [1], through equation 12.10-1:

\[
F_{px} = \sum_{i=x}^{n} F_i w_{px} / \sum_{i=x}^{n} w_i
\]

Eqn. 2.1

where

- \( F_{px} \) is the diaphragm design force
- \( F_i \) is the design force applied to Level \( i \)
- \( w_i \) is the weight tributary to Level \( i \)
- \( w_{px} \) is the weight tributary to the diaphragm at Level \( x \)

Upper and lower limits for the diaphragm force are specified. A lower limit of \( 0.2S_{DSI}w_{px} \) is correlated to one-half of the peak ground acceleration specified through the design response spectrum [1]. An upper limit relates directly to the peak ground acceleration: \( 0.4S_{DSI}w_{px} \) [1]. This upper limit is significant since any amplification of the ground acceleration due to structural response is neglected. Interestingly, the
diaphragm design force, when normalized by the tributary floor seismic weight, tends to the base shear coefficient, \( C_s = \frac{V}{W} \), as floors approach the ground level. This value, however, may be limited by the lower limit of \( 0.2 S w_{px} \), as illustrated in Figure 2-1 (c) at \( i=1 \). This figure considers the peak ground acceleration normalized by the acceleration of gravity to obtain parameter PGA. The base shear, \( V \), includes the effect of the response modification factor, so for buildings of different lateral force resisting systems at the same site the diaphragm accelerations tend to different values as the ground is approached from above. Similarly, buildings with the same LFRS but different heights will have floor accelerations that tend to their different base shear coefficients multiplied by the acceleration of gravity when the ground is approached from above. However, as the ground is approached from above, the acceleration in each of these buildings must tend to the peak ground acceleration. The inference in the distribution of forces by Eqn. 1.1 is that acceleration attenuation always occurs between the ground and first floor for a response modification factor that reduces the base shear coefficient below the PGA. Although the base shear normalized by the seismic weight will be less than the PGA for these response modification factors, the concept that only attenuation and not amplification occurs from nonlinear response is not conservative. Furthermore, the upper limit of diaphragm acceleration corresponding to peak ground acceleration enforces the concept that only attenuation occurs from nonlinear behavior.

Although a preferred method for analysis because of its simplicity, inadequacy of the ELF procedure eliminates it from consideration as a viable method for further investigation. Amplification factors intended to rectify this were not deemed feasible
due to the multiple layers of variables requiring calibration. A complex overhaul to a simple but inadequate procedure was not warranted.

2.2 Seismic response history procedure

The complexity and resources necessary to conduct a SRH analysis do not make it a suitable routine analysis procedure. Although justified for and necessary for many structures, this procedure is not practical for the design of typical precast concrete structures. The method involves mathematical representations of the structural distribution of mass and stiffness [1]. Ground motion selection, scaling, and result assessment requirements are specified for linear and nonlinear analyses.

2.3 Modal response spectrum analysis

Limiting the scope of analysis procedures to those accepted by current code provisions, the remaining procedure for further investigation is the MRS analysis. Theoretical aspects of this procedure are well-established [42]. The horizontal acceleration at floor $j$ due to mode $i$ can be found from the modal participation factor, $\Gamma_i$, modal amplitude, $\phi^j_i$, and spectral acceleration, $S_a(T_i, \zeta_i)$ at the modal period and damping associated with the $i^{th}$ mode:

$$a^i_j = \Gamma_i \phi^j_i S_a(T_i, \zeta_i)$$  \hspace{1cm} \text{Eqn. 2.2}

For code conformity, this procedure involves determination of the modes of vibration such that 90% of the combined modal mass, $M_i = (\phi_i)^T [m] (\phi_i)$, is captured in the analysis, where $m$ is the mass matrix [1]. It requires that response spectra values
obtained at each mode of response be divided by $R/I$, and displacement quantities amplified by $C_d/\bar{I}$, where $C_d$ is the deflection amplification factor. These modifications are intended to capture the nonlinear demands computed from elastic response spectra. Dividing each mode by $R$ means that diaphragm modes significant enough to be participating in the required modal mass have an explicit nonlinear design requirement attributed to the expected nonlinear performance of the vertical LFRS. This is not consistent with the implied elastic design criteria.

Permitted methods of combining peak spectral response parameters are the square root of the sum of the squares (SRSS) and the complete quadratic combination (CRC) [1]. The simplicity of the SRSS made this the preferred combination technique. A drawback of this technique is that it may produce un-conservative results if natural frequencies are not sufficiently separated [43]. A deficiency in estimating floor accelerations in buildings with diaphragm flexibility caused by closely spaced natural frequencies of the diaphragm and vertical subsystem would be apparent in the analytical study. This would necessitate a re-formulation of the modal combinations using the CRC method.

2.3.1 Simplified “First Mode Reduced” method

Based on findings by Rodríguez et al. [12] that simplification of the SRSS method is possible for rigid floor systems by (1) attributing ductility to the first mode of response and (2) assuming that all higher modes respond in the period range banded by the constant acceleration plateau of the design 5% damped response spectrum, their
simplified approach was adopted. The assumptions made in this method eliminate the rigor of the MRS procedure. Their proposed approach is less arduous than the ELF procedure and is appealing in a design setting because of its simplicity. Their simplified approach is herein referred to as the modal First Mode Reduced (FMR) method and is most clearly defined by equation 22 of the above reference:

\[
C_{pn} = \frac{\eta_1 R_1 C_h \left( T_1, 1 \right)}{\ln(n)} + 1.75 \ln(n) C_{ho}^2
\]

Eqn. 2.3

where

- \( C_{pn} \) is the seismic coefficient defined as the horizontal acceleration divided by the acceleration of gravity at level \( n \).
- \( n \) corresponds to the uppermost floor or roof.
- \( \eta_1 \) is the first mode contribution coefficient taken as 1 for a single story building or 1.5 for multi-story buildings.
- \( R_1 \) is the first mode reduction factor recommended as the ratio of displacement ductility to overstrength: \( \mu/\lambda \), but not less than 1.0.
- \( \lambda \) is the overall structural overstrength recommended as 2.0.
- \( C_h \left( T_1, 1 \right) \) is the 5% damped spectral acceleration at the building’s fundamental period, \( T_1 \).
- \( C_{ho} \) is the peak ground acceleration normalized by the acceleration of gravity.
Normalized by the peak ground acceleration, this acceleration becomes an acceleration magnification factor, $\Omega_n$. The recommended vertical distribution of accelerations was also simplified so that the uppermost floor diaphragm acceleration magnification factor, $\Omega_n$, is provided at all levels at or above 20% of the building’s height. Floors below this were prescribed a linear variation from 1.0 at the ground to $\Omega_n$ at $0.2h_n$.

2.3.2 Proposed formulation

This simplified framework of the modal FMR method was proposed as a basis for the estimation of elastic diaphragm forces in shear wall buildings with diaphragm flexibility. The following refinements were made in the formulation of the modified modal FMR method proposed:

- The vertical distribution of lateral forces is taken as:

$$\Omega_i = \begin{cases} \Omega_n & (0.15 < \frac{h_i}{h_n} \leq 1) \\ \frac{20}{3} \left( \frac{h_i}{h_n} \right) (\Omega_n - 1) + 1 & (0 < \frac{h_i}{h_n} < 0.15) \end{cases}$$

where $\Omega_i$ is the acceleration magnification factor at level $i$, $\Omega_n$ is the acceleration magnification factor at the roof, $h_i$ is the height of the level $i$, and $h_n$ is the height of the roof. This was decreased to account for amplification at lower floors due to diaphragm flexibility.

- The structure’s importance factor and redundancy factor were included in the estimation of the first mode spectral acceleration.
• The proposed value for $\eta^2 \omega^2$ which accounts for the contribution of higher modes ($\eta^2$) and ratio of $S_{DS}$ to the peak ground acceleration ($\omega^2$) was transformed from $1.75 \ln(n)$ to $1.4 \sqrt{n - 1}$ but need not be greater than 5.

• To facilitate the consistency between design parameters and the analysis method, the first mode reduction factor, $R_1$, was taken as the response modification factor, $R$, for the LFRS.

These changes produce the form of the modified-simplified modal FMR method as:

$$\Omega_n = \sqrt{\frac{\eta_1 \lambda R S_a(T_1, 0.05)}{R} + \eta_n (PGA)^2}$$

Eqn. 2.6

where

$\Omega_n$ is the floor acceleration magnification factor computed as the horizontal floor acceleration at level $n$ divided by the peak ground acceleration.

$n$ corresponds to the uppermost floor or roof.

$\eta_1$ is the first mode contribution coefficient taken as 1 for a single story building or 1.5 for multi-story buildings.

$R$ is the response modification factor.

$\lambda$ is the overall structural overstrength recommended as 1.75.

$S_a(T_1, 0.05)$ is the 5% damped spectral acceleration at the building’s fundamental period, $T_1$.

$S_{DS}$ is the 5% damped spectral response acceleration parameter at short periods.
PGA is peak ground acceleration in units consistent with the spectral acceleration parameters.

The extent to which this method estimates diaphragm design forces in relation to the ELF procedure is illustrated schematically in Figure 2-1 (c). For the even distribution of story heights and floor mass of subfigure (a), the two procedures result in dramatically dissimilar diaphragm design forces. Despite utilizing the same vertical LFRS based on ELF story shear forces, subfigure (b), the diaphragm force estimations are significantly different. Both procedures’ normalized design forces are shown schematically in subfigure (c) for a design scenario from the analytical study. In dark gray and indicated with “ELF” are the design forces computed from Eqn. 2.1 including the lower limit. For this particular case, the lower limit governs the first floor diaphragm force. If it had not governed, the design force would follow the dashed red line. The procedure adopted here has forces computed with Eqn. 2.6 that are indicated by the lighter gray shading with the letters “FMR.”

Figure 2-1: Equivalent lateral force procedure and diaphragm design forces
CHAPTER 3  HAZARD ESTIMATION

3.1  Introduction

The precast concrete industry operates throughout the world, but the objectives of the DSDM project pertained to the design of structures in the United States. Deliverables from the project needed applicability to all regions of seismicity in the United States. Four sites were selected by the DSDM Consortium to represent the variation of seismic hazard found throughout the country. These were Knoxville, TN, Charleston, SC, Seattle, WA, and Berkeley, CA. The seismic hazard was assessed through code provisions and prescribed for the analytical investigation of CHAPTER 6 via ten ground motions selected and scaled to represent possible design level events.

Charleston and Berkeley sites included likely local site effects. Soft soil conditions found in portions of Charleston were included in the seismic hazard by attributing to it a site class F for the formulation of the design response spectrum. This significantly influences the shape of the design spectrum with emphasis in the long period range. The result is large design forces for structures at this site compared to sites nearby without the soft soil condition. The Berkeley site’s proximity to the Hayward fault was considered. Although this near fault scenario is not considered in the formulation of the design spectrum, the effect was accounted for in the ground motion selection.
3.2 Design spectra

The seismic hazard was quantified in the design spectrum obtained through the International Building Code (IBC) [5]. Five percent damped elastic response spectra were obtained from the design parameters in Table 3-1 for each site. Design spectra for the design basis earthquake (DBE) are shown Figure 3-1 (a). Corresponding displacement response spectra are shown in Figure 3-1 (b).

Table 3-1: Site seismic design parameters

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Ss</th>
<th>F_a</th>
<th>S_MS</th>
<th>S_DS</th>
<th>S1</th>
<th>F_V</th>
<th>S_M1</th>
<th>S_DS1</th>
<th>SDC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Knoxville, TN (57915)</td>
<td>C</td>
<td>0.58</td>
<td>1.17</td>
<td>0.68</td>
<td>0.45</td>
<td>0.147</td>
<td>1.65</td>
<td>0.24</td>
<td>0.16</td>
</tr>
<tr>
<td>Charleston, SC (29403)</td>
<td>F</td>
<td>1.39</td>
<td>0.94</td>
<td>1.31</td>
<td>0.87</td>
<td>0.4</td>
<td>2.75</td>
<td>1.10</td>
<td>0.73</td>
</tr>
<tr>
<td>Seattle, WA (98101)</td>
<td>C</td>
<td>1.58</td>
<td>1.00</td>
<td>1.58</td>
<td>1.05</td>
<td>0.55</td>
<td>1.30</td>
<td>0.71</td>
<td>0.47</td>
</tr>
<tr>
<td>Berkeley, CA (94705)</td>
<td>C</td>
<td>2.08</td>
<td>1.00</td>
<td>2.08</td>
<td>1.39</td>
<td>0.92</td>
<td>1.30</td>
<td>1.21</td>
<td>0.81</td>
</tr>
</tbody>
</table>

Figure 3-1: Target response spectra (a) acceleration and (b) displacement
In terms of the design spectrum, the Knoxville site has a significantly lower seismic hazard than the other sites. Soft soil conditions result in the displacement demands at the Charleston site on par with those of the Berkeley site. In terms of PGA, however, the Charleston site’s code specified value only 63% that of the Berkeley site. The Berkeley site has the largest anticipated demands of the four sites. At short periods, the Seattle site has larger spectral acceleration demands than the Charleston site, but displacement demands are approximately 67% of the Charleston demands.

3.3 Hazard deaggregation

Deaggregations of hazards were obtained from the United States Geological Survey (USGS) website [45]. These are provided in Figures 3-3, 3-3, 3-4, and 3-5 for the Knoxville, Charleston, Seattle, and Berkeley sites, respectively. The deaggregation at the PGA for the hazards at return periods of 475 and 2,475 years, are provided in those figures as (a) and (b), respectively. These were intended to correlate with the hazard at the DBE and maximum considered earthquake (MCE). The deaggregation provides magnitude and distance relationships for the hazard scenarios. This insight helped as guidance for the ground motion selections, but the selected ground motions did not necessarily conform to the deaggregation. It was particularly difficult to match the magnitude and distance relationships for the eastern United States due to the source mechanisms and plate tectonics for which there are few recorded ground motions at the DBE level.
Figure 3-2: Knoxville site hazard deaggregation at (a) 475 and (b) 2,475 year return periods [45]

Figure 3-3: Charleston site hazard deaggregation at (a) 475 and (b) 2,475 year return periods [45]

Figure 3-4: Seattle site hazard deaggregation at (a) 475 and (b) 2,475 year return periods [45]
3.4 Ground motion selection

Sourcing of ground motion time histories came from two earthquake databases of strong motion recordings from historic events. The Pacific Earthquake Engineering Research Center’s (PEER) database [46] and the Consortium of Organizations for Strong Motion Observation Systems’ (COSMOS) database [47] were used to obtain processed acceleration time histories. Ten ground motions formed the record set for each site. The selections were based on the record’s fit to the design response spectrum and match to the hazard deaggregation. Care was taken in the selection process to exclude records requiring large scale factors. Ground motions selected for the Knoxville, Charleston, Seattle, and Berkeley sites are listed in Table 3-2 though Table 3-5, respectively. The earthquake, recording station, orientation and source database are noted.

Ground motions for the MCE event were not sourced separately, but obtained as the inverse of the code specified 2/3 ratio of the DBE to MCE. MCE records were obtained by multiplying the scaled DBE records by 1.5. This procedure did not account for
for different mechanisms of fault rupture between the two hazard levels. The hazard deaggregation was not considered. These two hazards pose different ground shaking scenarios, but this is not accounted for in the method implemented for obtaining MCE records. The primary need for ground motions was at the design level event so a more rigorous approach for obtaining MCE records was not justified.

3.4.1 Knoxville site

Table 3-2: Ground motion selections for the Knoxville site

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Date</th>
<th>Station</th>
<th>Magnitude</th>
<th>Distance [km]</th>
<th>Duration [sec]</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Duzce, Turkey</td>
<td>11/12/99</td>
<td>Lamont 531</td>
<td>7.1</td>
<td>11.4</td>
<td>31.3</td>
<td>PEER</td>
</tr>
<tr>
<td>Imperial Valley</td>
<td>10/15/79</td>
<td>Parachute Test Site</td>
<td>6.5</td>
<td>14.2</td>
<td>39.3</td>
<td>PEER</td>
</tr>
<tr>
<td>Kocaeli, Turkey</td>
<td>8/17/99</td>
<td>Arcelik</td>
<td>7.4</td>
<td>17</td>
<td>25.3</td>
<td>PEER</td>
</tr>
<tr>
<td>Kocaeli, Turkey</td>
<td>8/17/99</td>
<td>Atakoy</td>
<td>7.4</td>
<td>67.5</td>
<td>75.2</td>
<td>PEER</td>
</tr>
<tr>
<td>Landers</td>
<td>6/28/92</td>
<td>Yermo Fire Station</td>
<td>7.3</td>
<td>24.9</td>
<td>44.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Loma Prieta</td>
<td>10/18/89</td>
<td>Saratoga - Aloha Ave</td>
<td>6.9</td>
<td>13</td>
<td>40.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Nahanni, Canada</td>
<td>12/23/85</td>
<td>Site 1</td>
<td>6.8</td>
<td>6</td>
<td>20.6</td>
<td>PEER</td>
</tr>
<tr>
<td>Nahanni, Canada</td>
<td>12/23/85</td>
<td>Site 2</td>
<td>6.8</td>
<td>8</td>
<td>20.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Spitak, Armenia</td>
<td>12/7/88</td>
<td>Gukasian</td>
<td>6.8</td>
<td>30</td>
<td>19.9</td>
<td>PEER</td>
</tr>
<tr>
<td>Tabas, Iran</td>
<td>9/16/78</td>
<td>Dayhook</td>
<td>7.4</td>
<td>17</td>
<td>23.8</td>
<td>PEER</td>
</tr>
</tbody>
</table>
3.4.2 Charleston site

Soil conditions at the recording station were considered when selecting the record set for the Charleston site. Although exact soil conditions cannot be matched, the record set consisted of recordings from stations whose site soil classification was either C or D. Two stations’ soil classification was undetermined.

Table 3-3: Ground motion selections for the Charleston site

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Date</th>
<th>Station</th>
<th>Magnitude</th>
<th>Distance [km]</th>
<th>Duration [sec]</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>CHY015</td>
<td>7.6</td>
<td>43.51</td>
<td>96.8</td>
<td>PEER</td>
</tr>
<tr>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>CHY101</td>
<td>7.6</td>
<td>11.14</td>
<td>70.2</td>
<td>PEER</td>
</tr>
<tr>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>TCU052</td>
<td>7.6</td>
<td>0.24</td>
<td>71.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>TCU067</td>
<td>7.6</td>
<td>0.33</td>
<td>70.5</td>
<td>PEER</td>
</tr>
<tr>
<td>Hector Mine</td>
<td>10/16/99</td>
<td>Amboy</td>
<td>7.13</td>
<td>47.97</td>
<td>60.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Kocaeli, Turkey</td>
<td>8/17/99</td>
<td>Ambarli</td>
<td>7.4</td>
<td>78.9</td>
<td>95.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Kobe</td>
<td>1/16/95</td>
<td>Takatori</td>
<td>6.9</td>
<td>0.3</td>
<td>41.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Superstition Hills (B)</td>
<td>11/24/87</td>
<td>Westmorland Fire Station</td>
<td>6.7</td>
<td>13.3</td>
<td>40.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Taiwan SMART1 (45)</td>
<td>11/14/86</td>
<td>25 SMART1 C00</td>
<td>7.3</td>
<td>39</td>
<td>40.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Victoria, Mexico</td>
<td>6/9/80</td>
<td>Chihuahua</td>
<td>6.1</td>
<td>36.6</td>
<td>26.9</td>
<td>PEER</td>
</tr>
</tbody>
</table>

3.4.3 Seattle site

The record set for the Seattle site was derived primarily from recommendations from research partners at Lehigh University [48]. It also included ground motions from the Seattle record set developed for the SAC Steel Project [49] and records sourced based on the criteria discussed above.
Table 3-4: Ground motion selections for the Seattle site

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Date</th>
<th>Station</th>
<th>Magnitude</th>
<th>Distance [km]</th>
<th>Duration [sec]</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cape Mendocino</td>
<td>4/25/92</td>
<td>Cape Mendocino</td>
<td>7.1</td>
<td>9.5</td>
<td>30.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>TCU089</td>
<td>7.6</td>
<td>2.0</td>
<td>58.7</td>
<td>PEER</td>
</tr>
<tr>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>TCU129</td>
<td>7.6</td>
<td>1.2</td>
<td>59.5</td>
<td>PEER</td>
</tr>
<tr>
<td>Imperial Valley</td>
<td>10/15/79</td>
<td>Delta</td>
<td>6.4</td>
<td>44.0</td>
<td>99.9</td>
<td>PEER</td>
</tr>
<tr>
<td>Imperial Valley</td>
<td>10/15/79</td>
<td>El Centro Array #5</td>
<td>6.5</td>
<td>1.0</td>
<td>39.3</td>
<td>PEER</td>
</tr>
<tr>
<td>Kern County</td>
<td>7/21/52</td>
<td>Taft Lincoln School</td>
<td>7.5</td>
<td>41.0</td>
<td>54.2</td>
<td>PEER</td>
</tr>
<tr>
<td>Nisqually</td>
<td>2/28/01</td>
<td>Halverston Resid.</td>
<td>6.8</td>
<td>15.6</td>
<td>60.1</td>
<td>COSMOS</td>
</tr>
<tr>
<td>Northridge</td>
<td>1/17/94</td>
<td>Sylmar-Olive View Med FF</td>
<td>6.7</td>
<td>6.4</td>
<td>25.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Superstition Hills</td>
<td>11/24/87</td>
<td>Westmoreland Fire Station</td>
<td>6.7</td>
<td>13.0</td>
<td>40.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Western Washington</td>
<td>4/13/49</td>
<td>Olympia, WA - Washington Dept of Transportation Highway Test</td>
<td>7.1</td>
<td>74.7</td>
<td>60.1</td>
<td>COSMOS</td>
</tr>
</tbody>
</table>

3.4.4 Berkeley site

The record set for the Berkeley site relied on the work by Somerville [50] [51] for the PEER test bed of the UC Berkeley Life Sciences Building and near fault ground motions for the SAC Steel Project [49]. The motions include near-fault directivity effects recorded ground motions within 8 miles (13 km) of a fault rupture. Un-rotated ground motions from the PEER test bed were obtained from the corresponding source in Table 3-5.
Table 3-5: Ground motion selections for the Berkeley site

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Date</th>
<th>Station</th>
<th>Magnitude</th>
<th>Distance [km]</th>
<th>Duration [sec]</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>TCU065-W</td>
<td>7.6</td>
<td>1.0</td>
<td>70.1</td>
<td>PEER</td>
</tr>
<tr>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>TCU076-N</td>
<td>7.6</td>
<td>2.0</td>
<td>69.2</td>
<td>PEER</td>
</tr>
<tr>
<td>Erzincan, Turkey</td>
<td>3/13/92</td>
<td>Erzincan, Turkey</td>
<td>6.7</td>
<td>1.8</td>
<td>20.8</td>
<td>PEER</td>
</tr>
<tr>
<td>Imperial Valley</td>
<td>10/15/79</td>
<td>El Centro Array #8</td>
<td>6.5</td>
<td>3.8</td>
<td>37.6</td>
<td>PEER</td>
</tr>
<tr>
<td>Loma Prieta</td>
<td>10/17/89</td>
<td>Los Gatos Presentation Center</td>
<td>7.0</td>
<td>3.5</td>
<td>25.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Loma Prieta</td>
<td>10/17/89</td>
<td>Saratoga</td>
<td>7.0</td>
<td>8.3</td>
<td>40.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Northridge</td>
<td>1/17/94</td>
<td>Sylmar Hospital</td>
<td>6.7</td>
<td>6.4</td>
<td>25.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Northridge</td>
<td>1/17/94</td>
<td>Rinali Receiving Station</td>
<td>6.7</td>
<td>7.1</td>
<td>15.0</td>
<td>PEER</td>
</tr>
<tr>
<td>Superstition Hills (B)</td>
<td>11/24/87</td>
<td>Parachute Test Site</td>
<td>6.7</td>
<td>0.7</td>
<td>22.4</td>
<td>PEER</td>
</tr>
<tr>
<td>Tabas, Iran</td>
<td>9/16/78</td>
<td>Tabas</td>
<td>7.4</td>
<td>3.0</td>
<td>32.8</td>
<td>PEER</td>
</tr>
</tbody>
</table>

3.5 Ground motion scaling procedure

Selected ground motions were scaled to match the design level response spectrum. A single amplification scale factor was found for each record. Frequency content was unmodified. This leaves intact the peaks and troughs inherent in an elastic response spectrum. A scale factor was obtained by minimization of the square error between the design spectrum and the 5% damped elastic acceleration response spectrum.

Minimization accounted for periods from 0.1 to 4-sec at steps of 0.1-sec. It was performed on the summation of square error in this period range. Brent’s method, a
hybrid bracketing technique for scalar optimization programmed in Matlab by Bewley [52], was implemented to locate the minimum of the function:

$$J(SF_i) = \sum_{T=0.1}^{4.0} \frac{(S_{a,D}(T) - SF_i \cdot S_{a,i}(T))^2}{(S_{a,D}(T))^2}$$

Eqn. 3.1

where $S_{a,D}$ is the design response spectrum, $SF_i$ is the scale factor for ground motion $i$, $S_{a,i}$ is the 5% damped acceleration response spectrum of ground motion $i$, $T$ is the period, and the summation is evaluated at discrete steps of 0.1-sec.

Resulting scale factors and peak ground accelerations for the four sites are provided in Tables 3-6 through 3-9. A maximum scale factor of 4.78 was obtained for all records. The scaled records were given name identifiers. These distinguished the record sets with record names beginning with the first two letters of the site’s name. The remaining portion of the name identifier includes a two-digit number for distinction.

3.6 Scaled ground motions

Scaled acceleration time histories are shown in Figures 3-6, 3-8, 3-10, and 3-12 for the Knoxville, Charleston, Seattle, and Berkeley sites, respectively. The scaled response spectra and average of the ten spectra are shown with the design spectrum in terms of acceleration and displacement in Figures 3-7, 3-9, 3-11, and 3-13 for the Knoxville, Charleston, Seattle, and Berkeley sites, respectively. Individual records have significant deviation from the design spectrum due to the peaks and troughs, but in terms of the average response the scaling procedure resulted in a satisfactory agreement
with the design spectrum in the period range of interest. Deviation of the average spectrum from the design displacement spectrum is present in most sites above 4-sec. A deficiency in the long period displacement spectra impacts the nonlinear displacement demand, which is why the scaling method accounted for periods as long as 4-sec. Fundamental periods of structures analyzed with thee records were anticipated to be 1-sec or lower.

3.6.1 Knoxville site

Table 3-6: Ground motion parameters for the Knoxville site

<table>
<thead>
<tr>
<th>Record name</th>
<th>Earthquake</th>
<th>Date</th>
<th>Station</th>
<th>Component</th>
<th>Scale Factor</th>
<th>PGA [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>KN01</td>
<td>Duzce, Turkey</td>
<td>11/12/99</td>
<td>Lamont 531 E</td>
<td>1.139</td>
<td>0.13</td>
<td></td>
</tr>
<tr>
<td>KN02</td>
<td>Imperial Valley</td>
<td>10/15/79</td>
<td>Parachute Test Site 315</td>
<td>1.488</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td>KN03</td>
<td>Kocaeli, Turkey</td>
<td>8/17/99</td>
<td>Arcelik 000</td>
<td>1.295</td>
<td>0.28</td>
<td></td>
</tr>
<tr>
<td>KN04</td>
<td>Kocaeli, Turkey</td>
<td>8/17/99</td>
<td>Atakoy 090</td>
<td>1.113</td>
<td>0.18</td>
<td></td>
</tr>
<tr>
<td>KN05</td>
<td>Landers</td>
<td>6/28/92</td>
<td>Yermo Fire Station 360</td>
<td>0.845</td>
<td>0.13</td>
<td></td>
</tr>
<tr>
<td>KN06</td>
<td>Loma Prieta</td>
<td>10/18/89</td>
<td>Saratoga - Aloha Ave 090</td>
<td>0.627</td>
<td>0.20</td>
<td></td>
</tr>
<tr>
<td>KN07</td>
<td>Nahanni, Canada</td>
<td>12/23/85</td>
<td>Site 1 280</td>
<td>0.325</td>
<td>0.36</td>
<td></td>
</tr>
<tr>
<td>KN08</td>
<td>Nahanni, Canada</td>
<td>12/23/85</td>
<td>Site 2 240</td>
<td>0.701</td>
<td>0.34</td>
<td></td>
</tr>
<tr>
<td>KN09</td>
<td>Spitak, Armenia</td>
<td>12/7/88</td>
<td>Guksian 000</td>
<td>0.670</td>
<td>0.13</td>
<td></td>
</tr>
<tr>
<td>KN10</td>
<td>Tabas, Iran</td>
<td>9/16/78</td>
<td>Dayhook TR</td>
<td>0.549</td>
<td>0.22</td>
<td></td>
</tr>
</tbody>
</table>
Figure 3-6: Scaled acceleration time histories - Knoxville site
3.6.2 Charleston Site

Table 3-7: Ground motion parameters for the Charleston site

<table>
<thead>
<tr>
<th>Record name</th>
<th>Earthquake</th>
<th>Date</th>
<th>Station</th>
<th>Component</th>
<th>Scale Factor</th>
<th>PGA [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH01</td>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>CHY015</td>
<td>North</td>
<td>2.059</td>
<td>0.32</td>
</tr>
<tr>
<td>CH02</td>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>CHY101</td>
<td>North</td>
<td>0.849</td>
<td>0.37</td>
</tr>
<tr>
<td>CH03</td>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>TCU052</td>
<td>North</td>
<td>0.628</td>
<td>0.26</td>
</tr>
<tr>
<td>CH04</td>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>TCU067</td>
<td>North</td>
<td>1.240</td>
<td>0.40</td>
</tr>
<tr>
<td>CH05</td>
<td>Hector Mine</td>
<td>10/16/99</td>
<td>Amboy</td>
<td>360</td>
<td>3.003</td>
<td>0.45</td>
</tr>
<tr>
<td>CH06</td>
<td>Kocaeli, Turkey</td>
<td>8/17/99</td>
<td>Ambarli</td>
<td>000</td>
<td>2.263</td>
<td>0.56</td>
</tr>
<tr>
<td>CH07</td>
<td>Kobe</td>
<td>1/16/95</td>
<td>Takatori</td>
<td>090</td>
<td>0.542</td>
<td>0.33</td>
</tr>
<tr>
<td>CH08</td>
<td>Superstition Hills(B)</td>
<td>11/24/87</td>
<td>Westmorland Fire Station</td>
<td>180</td>
<td>1.829</td>
<td>0.39</td>
</tr>
<tr>
<td>CH09</td>
<td>Taiwan SMART1(45)</td>
<td>11/14/86</td>
<td>25 SMART1 C00</td>
<td>EW</td>
<td>2.349</td>
<td>0.29</td>
</tr>
<tr>
<td>CH10</td>
<td>Victoria, Mexico</td>
<td>6/9/80</td>
<td>Chihuahua</td>
<td>102</td>
<td>3.013</td>
<td>0.45</td>
</tr>
</tbody>
</table>

Figure 3-7: Knoxville site response spectra at 5% damping
Figure 3-8: Scaled acceleration time histories - Charleston site
Figure 3-9: Charleston site response spectra at 5% damping

3.6.3 Seattle Site

Table 3-8: Ground motion parameters for the Seattle site

<table>
<thead>
<tr>
<th>Record name</th>
<th>Earthquake</th>
<th>Date</th>
<th>Station</th>
<th>Comp.</th>
<th>Scale Factor</th>
<th>PGA [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>SE01</td>
<td>Cape Mendocino</td>
<td>4/25/92</td>
<td>Cape Mendocino</td>
<td>000</td>
<td>0.538</td>
<td>0.81</td>
</tr>
<tr>
<td>SE02</td>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>TCU089</td>
<td>N</td>
<td>1.934</td>
<td>0.48</td>
</tr>
<tr>
<td>SE03</td>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>TCU129</td>
<td>W</td>
<td>0.732</td>
<td>0.74</td>
</tr>
<tr>
<td>SE04</td>
<td>Imperial Valley</td>
<td>10/15/79</td>
<td>Delta</td>
<td></td>
<td>1.314</td>
<td>0.46</td>
</tr>
<tr>
<td>SE05</td>
<td>Imperial Valley</td>
<td>10/15/79</td>
<td>El Centro Array #5</td>
<td>140</td>
<td>1.140</td>
<td>0.59</td>
</tr>
<tr>
<td>SE06</td>
<td>Kern County</td>
<td>7/21/52</td>
<td>Taft Lincoln School</td>
<td>111</td>
<td>2.534</td>
<td>0.40</td>
</tr>
<tr>
<td>SE07</td>
<td>Nisqually</td>
<td>2/28/01</td>
<td>Halverston Resid.</td>
<td>270</td>
<td>4.783</td>
<td>0.52</td>
</tr>
<tr>
<td>SE08</td>
<td>Northridge</td>
<td>1/17/94</td>
<td>Sylmar - Olive View Med FF</td>
<td>090</td>
<td>0.750</td>
<td>0.45</td>
</tr>
<tr>
<td>SE09</td>
<td>Superstition Hills</td>
<td>11/24/87</td>
<td>Westmoreland Fire Station</td>
<td>090</td>
<td>2.358</td>
<td>0.41</td>
</tr>
<tr>
<td>SE10</td>
<td>Western Washington</td>
<td>4/13/49</td>
<td>Olympia, WA - Washington DTHT</td>
<td>86</td>
<td>2.107</td>
<td>0.59</td>
</tr>
</tbody>
</table>
Figure 3-10: Scaled acceleration time histories - Seattle site
3.6.4 Berkeley Site

Table 3-9: Ground motion parameters for the Berkeley site

<table>
<thead>
<tr>
<th>Record name</th>
<th>Earthquake</th>
<th>Date</th>
<th>Station</th>
<th>Comp.</th>
<th>Scale Factor</th>
<th>PGA [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>BE01</td>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>TCU065-W</td>
<td>North</td>
<td>0.876</td>
<td>0.53</td>
</tr>
<tr>
<td>BE02</td>
<td>Chi Chi</td>
<td>9/20/99</td>
<td>TCU076-N</td>
<td>North</td>
<td>1.668</td>
<td>0.69</td>
</tr>
<tr>
<td>BE03</td>
<td>Erzincan, Turkey</td>
<td>3/13/92</td>
<td>Erzincan, Turkey</td>
<td>West</td>
<td>1.380</td>
<td>0.68</td>
</tr>
<tr>
<td>BE04</td>
<td>Imperial Valley</td>
<td>10/15/79</td>
<td>El Centro Array #8</td>
<td>140</td>
<td>1.728</td>
<td>1.04</td>
</tr>
<tr>
<td>BE05</td>
<td>Loma Prieta</td>
<td>10/17/89</td>
<td>LGPC</td>
<td>000</td>
<td>0.723</td>
<td>0.41</td>
</tr>
<tr>
<td>BE06</td>
<td>Loma Prieta</td>
<td>10/17/89</td>
<td>Saratoga</td>
<td>090</td>
<td>2.140</td>
<td>0.69</td>
</tr>
<tr>
<td>BE07</td>
<td>Northridge</td>
<td>1/17/94</td>
<td>Sylmar Hospital</td>
<td>360</td>
<td>0.817</td>
<td>0.69</td>
</tr>
<tr>
<td>BE08</td>
<td>Northridge</td>
<td>1/17/94</td>
<td>Rinali Receiving Station</td>
<td>318</td>
<td>1.057</td>
<td>0.50</td>
</tr>
<tr>
<td>BE09</td>
<td>Superstition Hills (B)</td>
<td>11/24/87</td>
<td>Parachute Test Site</td>
<td>315</td>
<td>1.764</td>
<td>0.67</td>
</tr>
<tr>
<td>BE10</td>
<td>Tabas, Iran</td>
<td>9/16/78</td>
<td>Tabas</td>
<td>Long.</td>
<td>0.930</td>
<td>0.78</td>
</tr>
</tbody>
</table>
Figure 3-12: Scaled acceleration time histories - Berkeley site
Figure 3-13: Berkeley site response spectra at 5% damping
CHAPTER 4  EXPERIMENTAL TESTING OF A PRECAST
CONCRETE STRUCTURE

4.1 Introduction

This chapter presents the design, construction, and results of a three-story precast concrete structure built at 54% scale and tested under input ground motions on the outdoor shake table at the University of California, San Diego. The research objective of the shake-table test was to provide a means of validating the nonlinear finite-element and structural analysis computer models developed at the three participating universities: University of Arizona; the University of California, San Diego; and Lehigh University. The ability of a computer model to capture the behavior of the structure when subjected to the shake-table test provided the basis of validation for all other computer simulations within the project. Therefore, a comparison structure with realistic dynamic interaction between precast concrete components was important. This requirement necessitated a large-scale test whereby the connections between components could be reliably reproduced. This ensured that the behavior of a full-scale building would be accurately reproduced in the half-scale test structure. The test structure’s floor systems were designed using the DSDM project’s developing design methodology. The simplified test structure facilitated this objective because of the easily identifiable regions of high flexure and shear.
Connection details between floor elements were selected based on the performance characteristics of individual full-scale connection tests conducted at Lehigh University. With strength and deformation capacities determined by Lehigh University, the shake-table test was a first step in determining whether the seismic demands exceed the capacities in a system test. Providing reasonable connection deformation demands was another objective for the shake-table program. Other connection behaviors, such as group effects, coupled shear and tensile demands, field details, and cyclic and dynamic loading conditions, were identified as areas of interest that could be most reliably observed with the shake-table test.

4.2 Design

The design procedure involved an assumed prototype structure with typical precast units and standard connection details whose dimensions depended on limitations set by the scale factor and test setup. Based on test site restrictions, the longitudinal diaphragm dimension of the test structure was set to 54-ft. With a diaphragm aspect ratio of 3.5, the prototype structure was developed with a scale factor 0.5. Standard precast geometries were used in the design of this prototype structure, which resulted in average floor weights excluding the wall weight of 149-psf, 160-psf, and 150-psf for the first, second, and third floors, respectively. However, the production of the test structure utilized full-scale precast beds. For the double tee floors, a precast bed 4-ft wide double tee with 14-in. stem was used. For the test units, a 10-in. stem was created by placing an appropriate blockout. The hollow-core units were created with from full-scale 4-in. deep units. The test elements were more squat and heavier than the
appropriately scaled elements from the prototype structure. As a result, the prototype structure would have had floor weights 10 to 18% larger. To achieve a prototype structure whose average floor weights were closer to the weights with standard precast units, the scale factor was adjusted to compensate for the heavier test elements. A length scale factor of $S_l=1/1.855$ produced floor weights without considering walls of 146-psf, 155-psf, and 147-psf in the first, second, and third floors of a revised prototype structure. This reduced the additional weight in the prototype structure required because of the squat floor units to between 3 and 11% above the prototype structure’s weight. This scale factor was used in the scaling of ground motions resulting in a 54% scale model structure. This value corrects the scale factor reported as 0.5 by Schoettler et al. [53]. The plan dimensions of this prototype structure’s diaphragms were 100.2 x 29.7-ft, and its story heights would be 12.1-ft.

Three of the four sites discussed in CHAPTER 3 were selected for shake table testing, but the test structure’s diaphragms could only be designed to one strength level. For practical purposes, the strength was set based on the Berkeley site because this represented the largest demands of the four sites. These demands were established from nonlinear dynamic time history analyses conducted by the DSDM members at the University of Arizona using detailed finite element analyses.

The wall ultimate capacity was fixed by the wall geometry and amount of post-tensioning steel provided. However, the strength at onset of nonlinear behavior could be modified on a per site basis. This was accomplished with the initial post-tensioning force applied to the wall and the grouting of energy dissipating bars. This provided a means to tailor the system response through adjustments in the walls.
A response modification factor of 6 was applicable based on the LFRS. However, design forces with R=6 at the Knoxville and Seattle sites were considerably lower than the strength provided in the structure. This could be considered a design scenario with extreme overstrength. Alternatively, a fictitious R value not correlated with the LFRS could be assigned for comparison between design and demands. Although this design procedure breaks from acceptable code provisions, it provides a better correlation of design strength and capacity. An R value of 4.0 was assigned to the Knoxville site resulting in design strength in Table 4-1. With the fictitious R value, the design forces are on the order of those for the Berkeley site. For the design at the Seattle site, an R value of 4.5 was prescribed, giving the design strengths slightly larger than those at the Berkeley site. Despite nonconformity with code requirements, these fictitious R values provide the basis for comparison of design strength and demand in the results section 9 of this chapter.

<table>
<thead>
<tr>
<th>Site</th>
<th>R</th>
<th>M_u [kip-ft]</th>
<th>V_u [kip]</th>
<th>Floor 1</th>
<th></th>
<th>Floor 2</th>
<th></th>
<th>Floor 3</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Knoxville</td>
<td>4.0*</td>
<td>832</td>
<td>55</td>
<td>19</td>
<td>131</td>
<td>24</td>
<td>167</td>
<td>27</td>
<td>187</td>
</tr>
<tr>
<td>Seattle</td>
<td>4.5*</td>
<td>1,753</td>
<td>116</td>
<td>39</td>
<td>270</td>
<td>49</td>
<td>344</td>
<td>57</td>
<td>400</td>
</tr>
<tr>
<td>Berkeley</td>
<td>6.0</td>
<td>1,731</td>
<td>115</td>
<td>46</td>
<td>320</td>
<td>49</td>
<td>340</td>
<td>56</td>
<td>395</td>
</tr>
</tbody>
</table>

* Indicates an assumed value not correlated to the LFRS.
** Indicates code prescribed design forces, not the design values resulting from nonlinear analyses.

The displacement based design procedure for the hybrid rocking wall was reported by Belleri [54]. It followed procedures presented by Restrepo and Rahman [55]. System design strengths based on ASCE 7-05 requirements were met in this procedure. Overturning moment capacity accounted for contributions from the walls.
only as the column bases were modeled as pinned. Design shear forces were checked and adjusted according to nonlinear dynamic time history analyses.

The primary performance based objectives were to achieve a maximum base rotation at two performance levels and ensure gap closure. Maximum base rotation at the Berkeley MCE event was established as 2.74-rad. A maximum base rotation of 1.80-rad was targeted for the Berkeley DBE. With mild steel reinforcement used to increase energy dissipation \[55\], gap closure can only be assured if the initial post-tensioning force can overcome the mild steel reinforcement’s ultimate strength. This ensures no residual deformation at the base of the wall. These intertwined design considerations include the amount of mild steel reinforcement, the debonded length of this reinforcement for strain distribution, and quantity of post-tensioning steel for the wall selected wall configuration.

For the Knoxville test, the energy dissipating mild reinforcing bars at the base of the wall were omitted. A partial grouting of one bar in the South wall was performed, but the grout was flushed with water before setting. The levels of initial post-tensioning in the walls are provided in Table 4-2. The initial axial force in the Knoxville DBE test was lower than that for the other tests to account for the reduced demands at this site.

<table>
<thead>
<tr>
<th>Test</th>
<th>Axial force on the wall [kip]</th>
<th>Axial stress on the wall [ksi]</th>
<th>Tendon stress $f_{si}/f_{pu}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Knoxville DBE – trial 1</td>
<td>50</td>
<td>0.13</td>
<td>24%</td>
</tr>
<tr>
<td>Seattle DBE – trial 4</td>
<td>76</td>
<td>0.20</td>
<td>37%</td>
</tr>
<tr>
<td>Berkeley DBE – trial 1</td>
<td>74</td>
<td>0.19</td>
<td>36%</td>
</tr>
</tbody>
</table>
4.3 Scaling

To facilitate testing, a scaling procedure that did not include mass substitution was implemented. The additional cost of this substitute mass and its placement on the floors was prohibitive. Attaching this mass to the floors presented a problem of specimen inspection and would have involved additional labor and time to remove and re-install this mass for inspection between tests. To achieve similitude, the input acceleration was scaled to account for the mass discrepancy in the model structure [56].

Without mass substitution, similitude is achieved by scaling the model space in length, force, time, displacement, velocity, and acceleration. A consistent prototype and model mass density and material modulus are relied upon to achieve the scale factors for length, \( S_L \), force, \( S_F \), time, \( S_t \), velocity, \( S_v \), and acceleration, \( S_a \). The length scale factor is used to determine the remaining scale factors. It is defined as the ratio of the model unit length to the prototype unit length:

\[
S_L = \frac{l_m}{l_p} \tag{Eqn. 4.1}
\]

For a consistent stress in both the model and prototype structures, the force scale factor is derived as:

\[
S_F = \frac{F_m}{F_p} = \frac{\sigma_m A_m}{\sigma_p A_p} = 1.0 \left( \frac{l_m^2}{l_p^2} \right) = S_L^2 \tag{Eqn. 4.2}
\]
To obtain the time scale factor, the ratio of the model structure’s period to the prototype structure’s period can be used to find:

$$S_t = \frac{t_m}{T_p} = \frac{2\pi \sqrt{\frac{m_m}{k_m}}}{2\pi \sqrt{\frac{m_p}{k_p}}} = \sqrt{\frac{m_m}{m_p}} \frac{k_m}{k_p}$$

Eqn 4.3

This results in a compressed the time scale in model space as compared to prototype space.

The scale factor for displacements is obtained from the base unit of length and is equal to be the length scale factor, $S_l$.

The velocity scale factor also obtained from base units is unity:

$$S_v = \frac{v_m}{v_p} = \frac{t_m}{t_p} = S_l S_t^{-1} = 1.0$$

Eqn. 4.4

By first principles the horizontal acceleration scale factor is:

$$S_a = \frac{a_m}{a_p} = \frac{m_m}{m_p} \left(\frac{F_m}{F_p}\right) \left(\frac{F_m}{m_m}\right) = S_t \left(\frac{\rho_m l_p^3}{\rho_p l_p^3}\right) = S_l^2 \frac{1}{S_t} = S_t^{-1}$$

Eqn. 4.5

Model accelerations are therefore amplified or larger than the prototype accelerations to account for the lack of mass substitution. The gravitational field is unamplified resulting in a distorted model in the vertical direction. This is important when P-Δ effects are
significant. However, supplementary vertical load can be supplied in the form of post-tensioning to achieve the scaled moment-axial load interaction.

To compensate for the mass discrepancy in the self weight of the wall and the distorted gravitational field in the vertical direction, the initial post-tensioning force was increased above that required in the wall design. This compensation is the difference between the scaled axial force produced by the prototype structure’s wall and the axial force of the model wall: \[ \Delta N = N_p S_F - N_m = \left( l_{w,p} b_{w,p} H_p \rho_p a_p \right) S_F - \left( l_{w,m} b_{w,m} H_m \rho_m a_m \right) \]

However, the vertical acceleration fields in model and prototype space were the same, as were the mass unit densities. Therefore,

\[ \Delta N = \left( l_{w,m} b_{w,m} H_m w \right) S_t^{-3} S_F - \left( l_{w,m} b_{w,m} H_m w \right) = \left( l_{w,m} b_{w,m} H_m w \right) \left( S_t^{-3} - 1 \right) = 15.7 \text{-kip}, \]

where \( l_{w,p}, b_{w,p}, \) and \( H_p \) are the wall’s prototype dimensions, \( l_{w,m}, b_{w,m}, \) and \( H_m \) are the model wall’s dimensions used in the test, \( \rho_m \) is the mass unit density of concrete, \( a_p \) is the vertical acceleration field in prototype space, \( a_m \) is the vertical acceleration field in model space, and \( w \) is the unit weight of concrete in either space. However, the prototype structure’s tendon properties must be adjusted accordingly because the application of this load was not constant but varied with gap opening. The modification to the prototype structure’s tendon properties is shown in exaggeration in Figure 4-1.

The prototype structure’s ultimate stress should be reached at the model strain corresponding to ultimate stress. This takes into consideration the strain consumed by the model initial post-tensioning stress, \( f_{ui,m} \), being larger than the initial post-tensioning stress in the prototype structure, \( f_{ui,p} \), for axial similitude requirements in the wall.
4.4 Test setup

4.4.1 Shake table

Utilizing the George B. Brown Jr. Network for Earthquake Engineering Simulation’s (NEES) shake table at UCSD’s Englekirk Structural Engineering Center, the three story structure was erected in February 2008. The world’s largest outdoor shake table with a platen of 25-ft (7.6-m) wide by 40-ft (12.2-m) long permitted the testing of the precast concrete structure, which had the largest footprint area and mass of any structure tested in a shake table in the United States. The uni-directional shake table applied motions in the transverse direction of the structure, thus exciting the floor diaphragms in their flexible direction, see Figure 4-2. Two servo controlled dynamic actuators with a combined capacity of 1,530-kip (6.8-MN) provided sufficient force, displacement and velocity to shake the 836-kip (3.72-MN) structure. The test structure significantly exceeded the platen’s footprint. For this reason, the building was
constructed over a sturdy foundation structure, which extended 15.5-ft (4.72-m) off each side of the table and was tied down to the shake table platen.

4.4.2 Foundation

The foundation structure transferred the table motion to the base of the entire building. Erection drawings for the foundation structure are shown in Figures 4-3 through 4-7. The ends of the foundation were outfitted with massive outriggers to provide counterbalance weight resisting the overturning of the structure. The foundation components totaled 536-kip (2.38-MN) and included precast outrigger beams, support beams upon which a 7-in. (178-mm) thick cast-in-place topping was placed, and spacer blocks that sat directly on the shake table. Post-tensioning bars provided much of the interconnection between precast components and locked the foundation to the table by clamping the foundation structure to the table with 7.2
million pounds (32-MN) of initial post-tensioning force. The 7-in. (178-mm) cast-in-place topping created a very stiff and strong diaphragm that was designed to remain uncracked under the large in-plane inertia forces, thus ensuring consistent boundary conditions for the building throughout testing.

Because of the layout of the vertical components to the LFRS in the structure, much of the overturning moment had to be resisted by the outrigger beams running parallel off to the sides of the table. Rather than transfer this moment back to the table through warping of the foundation, the outrigger beams were made long and massive to counteract the overturning from the walls. However, the outrigger beams still had to be isolated from the ground surrounding the shake table. To achieve this, hydrostatic slider bearings, with a friction coefficient below 1% were employed. The bearings slid on mirrored-finish stainless steel plates, see Figure 4-8. The low friction minimized the effect of perturbing the input energy by introducing undesirable rectangular lateral force-displacement hysteresis. Decompression of the slider bearing was undesirable because the pressure of the oil film between the slider and stainless steel sliding surface would be lost and the detrimental effects of impact after uplift. Therefore, a minimum operating pressure was identified and ensured before testing via low profile 400-kip (1.78-MN) capacity hydraulic jacks that were sandwiched between the slider and the underside of the beam. The pressure of each jack and, for redundancy, each bearing was monitored separately.
Figure 4-3: Shake table test setup – Slider bearing layout

Figure 4-4: Shake table test setup – Plan view of the foundation level and column layout
Figure 4-5: Shake table test setup – Elevation view of the foundation level

Figure 4-6: Shake table test setup – Foundation level connection details, 1 of 2
Figure 4-7: Shake table test setup – Foundation level connection details, 2 of 2

Figure 4-8: Shake table test setup – slider bearing (a) bearing bottom and (b) assembly
4.4.3 Test structure

The three story building, see Figure 4-9, consisted of precast concrete elements with the completed structure weighing 302-kip (1.33-MN). Erection drawings shown in Figures 4-10 through 4-20 can also be found in the project’s data repository on the NEES central website [57]. A single 56-ft (17.07-m) by 16-ft (4.88-m) bay created a rectangular floor plan with an aspect ratio of 3.5. The simplified building with an open configuration resembled a parking garage, yet contained three unique floor systems. Floor-to-floor heights were 6.5-ft (1.98-m) with walls and columns extending 23-ft (7.01-m) above the foundation level. A precast wall was located at each end of the floor to provide lateral force resistance. The gravity load system was composed of two different column and beam types flanking the longitudinal direction of the building. Spandrel beams and columns represented the exterior of a building on the West side while ledger beams and corbel columns on the East side were intended to capture connections within the interior of typical precast buildings. Beams on the transverse edge of the structure were not included. Production drawings of individual precast elements and plate assembly drawings can be found at the DSDM project’s website on the NEES data repository [58].
Figure 4-9: Shake table test setup – test structure schematic

Figure 4-10: Shake table test setup – Floor one reinforcement layout, plan view
Figure 4-11: Shake table test setup – Floor one finished floor, plan view

Figure 4-12: Shake table test setup – Floor two reinforcement layout, plan view
Figure 4-13: Shake table test setup – Floor two finished floor, plan view

Figure 4-14: Shake table test setup – Floor three layout, plan view
Figure 4-15: Shake table test setup – West elevation and South elevation views

Figure 4-16: Shake table test setup – East elevation and North elevation views
Figure 4-17: Shake table test setup – Connection details, 1 of 4

Figure 4-18: Shake table test setup – Connection details, 2 of 4
Figure 4-19: Shake table test setup – Connection details, 3 of 4

Figure 4-20: Shake table test setup – Connection details, 4 of 4
As the primary interest for the research, the floor systems were the area where details were most carefully selected to gather the widest array of useful information. To accomplish this, three different floor systems were incorporated. The first floor level was a composite double tee diaphragm, the second floor level was a non-composite hollow-core diaphragm, and the third floor level incorporated pretopped double tee diaphragm. The following sections detail the main features of the three diaphragms and of the gravity and lateral load systems.

4.4.3.1 Floor one - composite double tee diaphragm

The first floor incorporated a composite double tee diaphragm. The double tees were 4-ft (1.22-m) wide by 16-ft (4.88-m) long, see Figure 4-21(a). These double tees were cast in a full scale 4-ft wide bed. Block outs added to the bed created the half scale stem of 10-in. (254-mm). A 1-in. (25.4-mm) thick flange replicated a full scale 2-in. (50.8-mm) flange. Each tee was prestressed with one 0.5-in. (12.7-mm) diameter strand in each stem. Shear reinforcement was provided at the stem ends.

This diaphragm was composite, meaning that the floor units and the cast-in-place topping were relied upon for the transfer of in-plane inertia forces. A rough broom finish was called for in production to maximize the composite action. This produced a finish with approximately 1/8-in. (3.2-mm) ridges. Four or five #2 hairpin flange-to-flange connectors were cast into the flange representing #4 hairpin connections for flange-to-flange composite shear action.

Shear reinforcement within the topping consisted of an innovative ductile mesh ladder developed by Cao and Naito [59]. The 10-in. (254-mm) on center by 12-in.
welded wire reinforcement had a 0.25-in. (6.4-mm) diameter. This was the smallest diameter wire available without cold working – an essential characteristic of the wire that provides the needed ductility. A strict requirement on the strain capacity ensured that the mesh performed in a ductile manner that was met with an 8.6% strain at peak stress. The ductile mesh was used only across the joints, see Figure 4-17 detail 01. Across the width of the double tee units, a 4x4 – W1.4xW1.4 conventional mesh made with conventional wire was used. The two meshes overlapped near the joint. The 3-in. (76.2-mm) overlap of the two types of meshes as required by ACI-3183 ensured each could be fully engaged via a strut-and-tie mechanism. The strength of the conventional mesh across the double tee units was designed to carry at yield the ultimate capacity of the ductile mesh at the joints. The 12-in. (305-mm) wire spacing was based on the required strength after accounting for the contribution due to the #2 hairpin flange-to-flange connectors and chord capacity in shear.

The 1.5-in. (38.1-m) topping included an additional 0.75-in. (19.1-mm) wash over the four #3 chord bars. Debonding material 8-in. (203.2-mm) long on the chord bars at the center three joints allowed strain penetration at these joints.

4.4.3.2 Floor two - non-composite hollow-core diaphragm

The second floor level incorporated a non-composite hollow-core diaphragm. The 4-in. (102-mm) deep hollow-core units were produced from a full scale bed representing an 8-in. (204-mm) deep prototype floor. The floor units were rip cut with a keyway from a 40-in. (1.02-m) wide standard bed. This created 20-in. (0.50-m) wide
by 16-ft (4.88-m) long units, see Figure 4-21(b). Widths of the end units and center units were modified to accommodate the bay widths.

Figure 4-21: Shake table test setup – floor elements (a) double tee units and (b) hollow-core units

The in-plane diaphragm flexural strength was provided by two #3 chord bars embedded in a 1.5-in. (38.1-mm) topping with a 0.75-in. (19-mm) wash over the chord reinforcement. Debonding material 8-in. (204-mm) long was placed on the chord bars at the column lines. A smaller number of chord bars were included in this floor due to its location in the structural system. Nonlinear dynamic time history analyses conducted by the research team indicated that lower demands were expected at this floor level. To achieve a similar demand-to-strength ratio as the other two floors, a reduced flexural capacity was provided.

With an in-plane shear strength based on a non-composite system and the same diameter ductile mesh as in the first floor, a smaller spacing between wires was used in comparison with the first floor. The welded wire reinforcement was 10-in. (254-mm) on center by 6-in. (152-mm) on center. The 6-in. (152-mm) spacing provided the required steel area for the hollow-core diaphragm and was reduced in comparison with
the 12-in. (305-mm) spacing used in the first floor because of the composite action and flange-to-flange connectors in that diaphragm. A capacity design approach ensured that the conventional mesh across the width of the hollow-core units had sufficient strength at yield to transfer the ultimate strength of the ductile mesh at the joints which it overlapped by 3-in. (76.2-mm). This overlap ensured the force transfer between the two meshes could be developed via a strut-and-tie mechanism and met the provisions of ACI-3183. The conventional mesh, 4x4 – W2.9xW2.9, was cut to the length and width of each hollow-core unit so only the ductile mesh crossed the joints. The grouted shear keys provided continuity between hollow-core units, but were not relied upon for shear strength in the non-composite design.

4.4.3.3 Floor three - pretopped double tee diaphragm

A pretopped double tee diaphragm was located on the third floor of the test structure. The precast double tees were cast from the same full scale bed as the first floor units with block outs in the stem creating the half scale units. A 2-in. (51-mm) flange was modeled after a 4-in. (102-mm) thick prototype flange thickness. The 4-ft (1.22-m) wide units were 16-ft (4.88-m) long with a wash at each end of the unit to provide sufficient coverage over the #3 chord bars.

The analyses of the test structure indicated the flexural demands at this level were the largest of the three floors. Six #3 chord bars at each end of the double tee provided sufficient strength. The reinforcing bars were discontinuous at the joints and grouped into two dry chord connectors. In each group of bars, three chord bars were welded at each end to 6.375-in. (162-mm) long by 1-in. (25-mm) high by 3/16-in. (5-
mm) thick steel end plates, see Figure 4-19 detail 25. End plates were exposed at the flange edge, which allowed field welding to provide the continuity at joints between units to transfer the chord force.

In this diaphragm the shear strength was provided by proprietary connectors specially produced at half scale for the test program. The connectors were cast in to the double tee units at the flange edges and field welded at the joints between units. Great care was taken to ensure the model connectors were precise scaled replicas. Individual connectors were tested at Lehigh University to quantify the scaled performance [60].

The differential camber between floor elements was not eliminated at time of erection. No vertical shear stress was present in shear connectors due to the construction process.

4.4.3.4 Gravity system

Five columns on each side of the structure created four bays of precast concrete beams. Spandrel beams were 3-ft (0.91-m) high and 5-in. (127-mm) thick with a 6-in. (152-mm) ledge for the floor elements. A similar size ledge was present in the ledger beams which were 6-in. thick and 19.5-in. (495-mm) tall.

The pocket columns measured 15.5-in. by 12-in. (394 x 305-mm) and the corbel column dimensions were 12-in. by 12-in. (305 x 305-mm). Well confined columns with #3 transverse hoops at 2.5-in. (63.5-mm) ensured the seismic integrity of the gravity system and eliminated a possible shear failure. A prestress force of 15-kip (66.7-kN) coupled with eight #5 bars for the longitudinal reinforcement in the columns. The 0.5-in. (12.7-mm), 270-ksi (1,862-MPa) prestressing strand was stressed to 0.36f_{pu} which
applied an average stress of 104-psi (0.72-MPa) on the 12-in. (305-mm) square concrete column. Column anchor bolts on the ten columns were specified as 3/8-in. (9.5-mm) diameter A36 threaded rod in an attempt to minimize the shear capacity of the gravity load system and maximize the floor flexibility.

4.4.3.5 Wall system

Two 8-ft (2.44-m) long by 23-ft (7.0-m) tall and 8-in. (203-mm) thick rocking walls composed the vertical elements of the LFRS. Rocking walls differ from typical reinforced concrete walls in that they concentrate the flexural cracks at one location at the base of the wall instead of distributing cracks over a plastic hinge length near the bottom [55], [61], [62]. For increased energy dissipation [55], two #7 reinforcing bars in each wall were provided across the joint at the base of the wall for the larger amplitude tests. The vertical energy dissipation bars crossed the horizontal joint between the wall and foundation where uplift occurred. During rocking, energy dissipation bars elongate as the joint opens and dissipate energy in the plastic deformation cycles of the hysteretic response. For this purpose, five headed reinforcing bars were grouted into each of the outrigger beams before the walls were erected. Grout ducts in the walls extended 6-ft (1.82-m) from the wall base for development purposes. Two of the five bars were grouted in the wall ducts to provide energy dissipation for tests in the moderate and high seismic range. The remaining three bars were replacements for the two grouted bars, which could fatigue after multiple cycles of loading. This proved to be a cost effective way to replace damaged bars with the damaged bars cut by core drilling to
ensure they no longer participated in the response. Eliminating the damaged bars and grouting new bars gave a known and more easily predictable response.

Post-tensioning was accommodated through two vertical ducts that contained five 0.5-in. (12.7-mm) diameter, grade 270 (1,862-MPa) tendons each. The ten strands passed through the wall and ducts in the outrigger beam and were anchored by wedge anchor plates beneath the beams. On top of the wall, anchorage plates were mounted on 100-ton (890-kN) hollow core plunger jacks that were positioned above the wall ducts. The hollow jacks allowed the strands to pass through and be seated in the anchor plate. The jacks were used to simultaneously seat the ten strands in one wall and apply the initial post-tensioning force of 106-kip (472-kN) for the Knoxville DBE tests and 144-kip (642-kN) for the Seattle DBE tests, Berkeley DBE and MCE tests through June 20th, 2008. The lower post-tensioning force was for the Knoxville site compensated for the lower design forces.

To transfer shear forces from the floors into the walls, vertical slotted shear connectors were used, see Figure 4-22. This detail precluded the walls from carrying gravity load. The slotted connectors were used to accommodate vertical uplift in the wall without introducing out of plane forces in the floor. The vertical uplift is a result of the wall’s flexural response. However, this uplift is not unique to the selected wall type. When a cantilever wall displaces beyond its elastic limit, a concentration of rotation occurs at its base. In a reinforced concrete wall, the rotation will be smeared along the plastic hinge length while in a rocking wall the rotation will concentrate at the joint. Because of the migration of the neutral axis depth towards the extreme fiber in compression, the centerline of both walls will lengthen approximately the same amount.
Full scale slotted shear connectors were selected to minimize the use of costly scaled connections. Capacity design was implemented to ensure failure was concentrated in the diaphragm. These connectors had an anchor strap that screwed into the vertical slot after placing the floor elements. These straps were then welded to embedded plates, which were cast into the third deck or embedded in the concrete topping on the first two decks.

4.4.3.6 Secondary connections

Remaining connection details between elements were selected based on their strength and flexibility characteristics. Connection details are included in the erection drawings in Figures 4-17 through 4-20. Spandrel-to-column connections were 0.5-in. (12.7-m) diameter threaded rod that threaded into an insert in the back of the spandrel, passed through oversized horizontal PVC sleeves in the columns and were snug tightened with a wrench. Two of these connections were used at each end of the spandrel. The ledger beams included a vertical sleeve at each end. Through each
sleeve passed a 0.5-in. (12.7-mm) diameter threaded rod which was screwed into an 
embed plate in the top of the column corbel. The sleeve was sand filled for the bottom 
6-in. (152-mm) and then grout filled to the top of the beam. A nut and oversized 
washer clamped a slotted angle down to the top of the beam via the protruding rod. The 
opposite leg of the angle was field welded to an embed plate on each side of the 
column. The slotted leg of the angle was intended to let the beam slide relative to the 
column, and friction was minimized by sandwiching the angle with Teflon pads and 
stainless steel slider plates.

Connecting the beams to the floors were two types of connectors. The hollow-
core floor incorporated 0.25-in. (6.4-mm) diameter threaded rod which screwed into 
inserts in the inside face of the spandrels and ledger beams. The rods were spaced at 
12-in. (305-mm) and cast into the topping. Number two hairpin connectors were used 
on the first and third floors at each end of the double tee units. These were welded to 
embed plates in the spandrels and ledger beams at the center of each double tee. The 
pretopped deck required the hairpins to be cast in while the topped floor permitted the 
reinforcing bars to be welded then embedded in the topping. The #2 threaded rod and 
hairpin connectors were selected because of their high deformability, which was 
intended to permit a spread floor opening caused by flexural deformation. A rigid 
connection could have caused concentrated joint opening at the column lines where 
breaks in the beams permit movement.
4.5 Material properties

All reinforcing steel was specified as grade 60 (414-MPa). A-706 weldable reinforcement was used where required or for regions like the chord steel or wall energy dissipation bars for its desirable stress-strain characteristics. Table 4-3 summarizes the actual stress-strain characteristics of reinforcement used in critical components. The #2 deformed reinforcing bars met the grade 60 minimum strength and chemical composition met the weldability requirements. This was used for the bent hairpin connectors that were welded during erection.

Column anchor bolts were specified as A36, but tensile testing showed the yield strength was 51.6-ksi (356-MPa) and the ultimate strength was 58.4-ksi (403-MPa) at 2.5% strain. The limited ultimate tensile strain of these bolts was a concern for the columns’ flexural response. A debonded length of 8-in (203-mm) provided 0.2-in (5.1-mm) of column uplift at a strain of 2.5% in the anchor bolts without further strain penetration. This corresponded to a column base rotation of 1.9%.

Table 4-3: Steel properties

<table>
<thead>
<tr>
<th>Location</th>
<th>Size</th>
<th>Specified yield strength</th>
<th>Measured yield strength</th>
<th>Measured ultimate strength</th>
<th>Measured peak strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>3rd Floor Chord</td>
<td>#3</td>
<td>60-ksi</td>
<td>72.5-ksi (^a)</td>
<td>108.1-ksi (^a)</td>
<td>12.1% (^b)</td>
</tr>
<tr>
<td>1st and 2nd Floor Chord</td>
<td>#3</td>
<td>60-ksi</td>
<td>70.4-ksi (^a,c)</td>
<td>108.0-ksi (^a)</td>
<td>9.4% (^b)</td>
</tr>
<tr>
<td>1st and 2nd Floor Shear Mesh</td>
<td>#2</td>
<td>60-ksi</td>
<td>65.2-ksi (^d)</td>
<td>76.7-ksi (^d)</td>
<td>8.6% (^e)</td>
</tr>
<tr>
<td>Wall Energy Dissipation Bars</td>
<td>#7</td>
<td>54-ksi</td>
<td>71.1-ksi (^a)</td>
<td>97.6-ksi (^a)</td>
<td>12.3% (^b)</td>
</tr>
<tr>
<td>Column Anchor Bolts</td>
<td>#3</td>
<td>36-ksi</td>
<td>51.6 (^a,c)</td>
<td>58.4-ksi (^a)</td>
<td>2.5% (^b)</td>
</tr>
</tbody>
</table>

\(^a\) Average of two samples  
\(^b\) Lowest value obtained from two samples
Concrete strengths were specified as 6,000-psi (41.4-MPa) and 4,000-psi (27.6-MPa) for the precast elements and concrete toppings, respectively. In addition to the twenty eight day strength, concrete cylinders were retained for tests corresponding to the initiation and conclusion of shake table testing. These strengths are found in Table 4-4. The maximum aggregate size was specified as 3/8-in (9.5-mm) diameter. Precast elements had 6-in (152-mm) diameter by 12-in (305-mm) long cylinders taken while 4-in (102-mm) diameter by 8-in (204-mm) long cylinders were taken of the toppings.

<table>
<thead>
<tr>
<th>Location</th>
<th>Specified strength</th>
<th>28 day strength</th>
<th>Start of testing</th>
<th>End of testing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Age (days)</td>
<td>Strength</td>
</tr>
<tr>
<td>Double Tee Units</td>
<td>6-ksi</td>
<td>7.3-ksi</td>
<td>124</td>
<td>7.0-ksi</td>
</tr>
<tr>
<td>Hollow-core Units</td>
<td>4-ksi</td>
<td>5.1-ksi</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Wall Units</td>
<td>6-ksi</td>
<td>7.76-ksi</td>
<td>98</td>
<td>7.4-ksi</td>
</tr>
<tr>
<td>1st and 2nd Floor Topping</td>
<td>4-ksi</td>
<td>4.4-ksi</td>
<td>47</td>
<td>5.1-ksi</td>
</tr>
</tbody>
</table>

\[ ^{\text{f}} \text{Average of three specimen}^{\text{g}} \text{Average of two specimen}\]

Grout strengths were obtained from 1-in (25.4-mm) diameter by 3-in (76.2-mm) long test cylinders. Grout joints below the columns and walls were specified as 8-ksi (55.2-MPa). However, polypropylene fibers at approximately 0.02% by weight were
added to the wall base grout to increase the toughness required by the expected impact loading during the rocking of the walls. The energy dissipation rebars in the wall base were grouted in two stages with 6-ksi (41-MPa) specified strength. The first grout stage grouted the bars into the outrigger beam before the wall was erected. The second stage was grouting the wall ducts to activate the rebar. Table 3 indicates the twenty eight day strength and strength at the start and end of testing.

Hollow-core key joints were grouted with a 3-to-1 ratio of sand to cement. The specified minimum strength was 3-ksi (21-MPa). The grout filled ledger beam-to-column connection was also specified as 3-ksi. Grout cylinders were not taken of these two grouts. However, a mock grout was mixed for the hollow-core keyway using the same proportions and tested for an estimated strength of the actual grout. This strength is indicated in Table 4-5.

<table>
<thead>
<tr>
<th>Location</th>
<th>Specified strength</th>
<th>28 day strength</th>
<th>Start of testing</th>
<th>End of testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column-to-foundation joint</td>
<td>6-ksi</td>
<td>6.8-ksi</td>
<td>81</td>
<td>6.4-ksi</td>
</tr>
<tr>
<td>Wall-to-foundation joint</td>
<td>6-ksi</td>
<td>7.0-ksi</td>
<td>82</td>
<td>7.1-ksi</td>
</tr>
<tr>
<td>Grout duct in the outrigger beam for wall energy dissipation bar</td>
<td>6-ksi</td>
<td>7.1-ksi</td>
<td>87</td>
<td>9.0-ksi</td>
</tr>
<tr>
<td>Grout duct in the wall for wall energy dissipation bar</td>
<td>6-ksi</td>
<td>5.0-ksi</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Hollow-core keyway grout</td>
<td>3-ksi</td>
<td>2.1-ksi</td>
<td>75</td>
<td>3.1-ksi</td>
</tr>
</tbody>
</table>

Table 4-5: Grout strengths

<table>
<thead>
<tr>
<th>Location</th>
<th>Specified strength</th>
<th>28 day strength</th>
<th>Start of testing</th>
<th>End of testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column-to-foundation joint</td>
<td>6-ksi</td>
<td>6.8-ksi</td>
<td>81</td>
<td>6.4-ksi</td>
</tr>
<tr>
<td>Wall-to-foundation joint</td>
<td>6-ksi</td>
<td>7.0-ksi</td>
<td>82</td>
<td>7.1-ksi</td>
</tr>
<tr>
<td>Grout duct in the outrigger beam for wall energy dissipation bar</td>
<td>6-ksi</td>
<td>7.1-ksi</td>
<td>87</td>
<td>9.0-ksi</td>
</tr>
<tr>
<td>Grout duct in the wall for wall energy dissipation bar</td>
<td>6-ksi</td>
<td>5.0-ksi</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Hollow-core keyway grout</td>
<td>3-ksi</td>
<td>2.1-ksi</td>
<td>75</td>
<td>3.1-ksi</td>
</tr>
</tbody>
</table>

* Average of three specimen
Table 4-5 continued

\(^{g}\) Average of two specimen  
\(^{h}\) Tested at 35 days  
\(^{i}\) Tested at 13 days  
\(^{j}\) Samples taken were not of the actual grout placed

4.6 Testing protocol

Three of the four sites discussed in CHAPTER 3 were selected for shake table testing. Due to similarities in the design response spectra for the Charleston and Berkeley sites, the Charleston site was omitted from the testing protocol. One ground motion at the DBE level was selected for the Knoxville, Seattle, and Berkeley sites. An MCE level test was conducted for the Berkeley site.

A test protocol of increasingly more demanding ground motions was used. Increasing demands as testing progressed ensured that a large number of tests could be completed, allowing sufficient data sets to be gathered for computer model validation. Different ground intensities were applied by ordering the ground motions for the sites according to increasing seismic hazard.

The test sequence called for a design basis earthquake (DBE) for the Knoxville site, followed by a DBE for Seattle, a DBE for Berkeley, and a maximum considered earthquake (MCE) for the Berkeley site. The representative ground motions selected for those events came from the 1979 Imperial Valley and 1989 Loma Prieta earthquakes. Table 4-7 identifies the historic ground motions used, reference record used from the record sets developed in the pervious chapter, and the recorded peak ground acceleration. For input to the shake table, the ground motions from the record
set were scaled according to the scaling procedure of section 4.3. The peak ground
acceleration provided in Table 4-7 and the acceleration time histories in Figure 4-24 are
average recordings of sensors located on the foundation next to the walls. Response
spectra of these records are also shown in model space in Figure 4-23.

The Berkeley MCE record was obtained by amplifying the DBE record by 1.5. The linear elastic response spectra for the Berkeley DBE and MCE matched well
overall with their target spectra, but contain a significant trough at the test structure’s
fundamental period. However, the nonlinearity of the structure - as predicted in
nonlinear dynamic time history analyses used to validate the design - is not captured in
the response spectrum. Scaling of the Berkeley site’s ground motions to better match
the target spectra at the building’s fundamental period would have overestimated these
earthquake scenarios.

Characterization of the structure’s dynamic properties was conducted throughout
the three month test period. These evaluations included white noise ground motions,
ambient vibration recordings, free vibration tests, and two shaker tests conducted using
equipment from the NEES at University of California, Los Angeles (UCLA) facility
including an eccentric mass shaker mounted on the third floor. Free vibration tests
initiated by impacting column lines at the third floor and the shaker tests provided
alternate methods of evaluating initial, pre-cracked conditions as did the first set of
white noise tests. The white noise tests consisted of essentially random vibrations
covering a particular band of frequencies with consistent energy content input to the
base of the structure with the shake table. These tests provided repeated structural
characterization, which was useful for assessing the damage incurred in earthquake simulations, by providing before and after evaluations of the structure’s response.

An extended scope of testing was conducted due to the viability of the structure after repairs to damage sustained from the intended loading protocol. The complete test sequence is provided in the summary of test results of section 4.9.1.1.

Table 4-6: Shake table test ground motion parameters

<table>
<thead>
<tr>
<th>Site</th>
<th>Target level</th>
<th>Historic earthquake</th>
<th>Station</th>
<th>Record name</th>
<th>Measured test PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Knoxville</td>
<td>DBE</td>
<td>1979 Imperial Valley</td>
<td>Parachute Test Site</td>
<td>KN02</td>
<td>0.35</td>
</tr>
<tr>
<td>Seattle</td>
<td>DBE</td>
<td>1979 Imperial Valley</td>
<td>El Centro Array #5</td>
<td>SE05</td>
<td>0.89</td>
</tr>
<tr>
<td>Berkeley</td>
<td>DBE</td>
<td>1989 Loma Prieta</td>
<td>Los Gatos Presentation Center</td>
<td>BE05</td>
<td>0.75</td>
</tr>
<tr>
<td>Berkeley</td>
<td>MCE</td>
<td>1989 Loma Prieta</td>
<td>Los Gatos Presentation Center</td>
<td>BE05</td>
<td>1.20</td>
</tr>
</tbody>
</table>
Figure 4-23: Shake table test measured acceleration time histories
4.7 Instrumentation and data acquisition

Six hundred and fifty one sensors were installed on the building to capture its response. Five types of sensors were mounted on the structure to monitor accelerations, displacements or deformations, strains, and pressures. These 651 sensors are separate from the control data that added another 64 channels of comparison data.

Five separate data acquisition (DAQ) systems were used to record the data including 85 channels in a mobile DAQ system from NEES at UCLA. Sampling rates for the five systems varied, but the results were post processed and resampled to a common 240 samples per second. Ten sensors came from the California Strong Motion Implementation Program (CSMIP) and recorded their response on their own automatically triggering DAQ system. A 128 channel strain gauge system from
UCSD’s Powell’s Structural Engineering Laboratory was utilized to monitor critical regions of the structure expected to undergo plastic deformation. Four channels of GPS data were recorded on a separate dedicated DAQ system, and the remaining sensors were routed to the shake table’s DAQ system operated by NEES at UCSD.

A majority of accelerometers were mounted in the direction of shaking, but some were also oriented transverse to the direction of excitation or vertically. Global displacements were measured from four GPS antennas mounted on the structure with two additional antennas acting as reference receivers. Relative deformations were captured by string potentiometers, linear voltage displacement transducers - see Figure 4-25 (a), and linear potentiometers. Critical reinforcing bars were strain gauged to monitor their deformation history through tests. These included the chord reinforcement, ductile mesh, and wall energy dissipation bars. Concrete strain gauges were deployed on the toes of one wall to capture compressive strains as the walls rocked. Strain gauge DAQ settings corresponded to saturation at 0.05 strain.

Pressure transducers were installed on the four slider bearings and four 200-ton (181-metric ton) jacks under the outrigger beams. These provided the dynamic response of the pressure change in the bearings during testing and allowed the computation of overturning moment. An additional four pressure transducers were installed on the jacks on the walls, see Figure 4-25 (b). These measured the force in the post-tensioning tendons during the tests.

Fourteen cameras recorded shaking during the earthquake simulations. Eleven cameras were mounted on the structure to help with visualization and interpretation of data. They captured crack opening and movement in the structure. Three additional
cameras recorded the overall structural response. Extensive photo documentation provided as-built construction details - see Figure 4-25 (c), instrumentation orientation, crack propagation, and damage.

Three dimensional sensor coordinates were essential for results analysis. Prior to testing, the sensors’ locations were measured and mapped in painstaking detail. These were transformed into tabulated data and the graphical representation for each test [66]. The mapped locations for tests on June 20, 2008 are provided in Appendix A.

Figure 4-25: Shake table test instrumentation examples [67]
4.8 Data post processing procedures

4.8.1 Introduction

Recorded test data underwent the following post-processing procedure to synchronize, convert, achieve a consistent sampling rate, and create a consistent file format. Basis for the file format, sampling rate, and synchronization was the data recorded by the site (NEES@UCSD). The sampling frequency of NEES@UCSD DAQ was 240Hz. Original, processed, and derived data was uploaded to the DSDM Project’s website at the NEEScentral data repository [63].

4.8.2 NEES@UCLA data

NEES@UCLA acceleration data was:

- Upsampled from 200Hz to 240Hz.
- Scaled to units of g.
- Synchronized with NEES@UCSD data by visual comparison of UCSD channel ‘0A-5’ and UCLA channel ‘0A-10’.
- Saved to the same binary format as NEES@UCSD.

4.8.3 Strain gauge data

Strain gauge data was:

- Scaled from microstrain to strain.
- Low pass filtered at 33Hz with an FIR filter of order 5000.
- Synchronized to NEES@UCSD data using the trigger channel (channel one).
• Upsampled from 237Hz to 240Hz. Note that saturated data is affected by the upsampling and ripples are present in the processed data just before and after saturation.
• Saved to the same binary format as NEES@UCSD.

4.8.4 Global positioning system data

50Hz GPS data acquisition system: Navcom network
• Not synchronized or post-processed

20Hz GPS data acquisition system: Leica network
• Not synchronized or post-processed

4.8.5 California Strong Motion Implementation Program data

CSMIP data acquisition system
• Not synchronized

4.8.6 Derived velocity and displacement data

Acceleration data was integrated into velocity and further integrated into displacement using a cumulative trapezoidal numerical integration approximation. The procedure is not capable of capturing residual displacements due to the high pass filtering. The following procedure was implemented to acquire the velocity and displacement data:
1. The acceleration data was filtered with a high pass FIR filter. A cutoff frequency of 0.25Hz was used with a Hamming-window filter of order 5000.

2. The numerical integration of the filtered acceleration data was performed.

3. The velocity is obtained by applying the same high pass FIR filter to the integrated data.

4. The numerical integration of the velocity data was performed.

5. The displacement is obtained by applying the same high pass FIR filter to the integrated data.

4.8.7 Filtered acceleration data

A lowpass FIR filter was applied to all acceleration data after obtaining velocity and displacement data. A Hamming-window filter of order 5000 was used with a cutoff frequency of 33Hz to obtain the processed acceleration data. Processed acceleration data was saved in the binary format provided by NEES@UCSD.

4.9 Results

4.9.1 Test observations

4.9.1.1 Summary

Before the test sequence commenced, a naturally occurring earthquake with an epicentral distance of 31-miles (49-km) away from the shake table site was recorded by
CSMIP sensors. The minor earthquake with a moment magnitude of 3.78 [64] resulted in a 0.015-g PGA at the structure’s base, yet was sufficient to trigger the CSMIP DAQ.

Fundamental periods and mode shapes were identified based on the results of ambient vibration recordings and the UCLA shaker tests [65]. A 0.22-sec period was obtained for the fundamental mode in the test structure’s transverse direction.

Pretest shrinkage cracking was observed in the two field topped floor systems. The first floor level, the topped double tee system, contained three hairline shrinkage cracks extending the length of the double tee at flange-to-flange joints near the center of the diaphragm. The second floor level, the topped hollow-core system, also contained hairline temperature or shrinkage cracks at the edges of the floor. Hairline crack width is defined here as approximately 0.004-in (0.1-mm). No other cracks were evident in the test structure prior to testing.

A significant number of tests were conducted. The complete test sequence is provided Table 4-7. This table includes the date of testing, test conducted, and a brief description if necessary. It also identifies the DAQ systems active for each test. A brief test outcome with observed damage is provided for each of the 16 earthquake tests shown in bold in Table 4-7.
Table 4-7: Experimental test sequence

<table>
<thead>
<tr>
<th>Date</th>
<th>Test</th>
<th>Description</th>
<th>NEES@UCSD</th>
<th>NEES@UCLA</th>
<th>50Hz GPS</th>
<th>20Hz GPS</th>
<th>Strain Gauges</th>
<th>CSMIP</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/2/08</td>
<td>Ambient vibration</td>
<td>CSMIP</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5/3/08</td>
<td>Ambient vibration</td>
<td>UCSD</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Free vibration 3B - trial 1</td>
<td>Impact at 3rd floor col. line 3B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Free vibration 3B - trial 2</td>
<td>Impact at 3rd floor col. line 3B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Free vibration 3B - trial 3</td>
<td>Impact at 3rd floor col. line 3B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Free vibration 3B - trial 4</td>
<td>Impact at 3rd floor col. line 3B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Free vibration 3B - trial 5</td>
<td>Impact at 3rd floor col. line 3B</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Free vibration 1B - trial 1</td>
<td>Impact at 3rd floor col. line 1B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Free vibration 1B - trial 2</td>
<td>Impact at 3rd floor col. line 1B</td>
<td></td>
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<td></td>
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<tr>
<td></td>
<td>Free vibration 1B - trial 3</td>
<td>Impact at 3rd floor col. line 1B</td>
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<td></td>
<td>Free vibration 1B - trial 4</td>
<td>Impact at 3rd floor col. line 1B</td>
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- No data available from this DAQ
- Test data available from this DAQ
- Test data is unreliable or missing for this DAQ

4.9.1.2 Knoxville DBE - trial 1

The Knoxville DBE – trail 1 test was the first shake table test conducted. The start date was May 6, 2008. The selected wall post-tensioning force allowed minor rocking of the wall during this test. No energy dissipation was used (i.e., the bars were not grouted) at this level of shaking. However, the South wall contained a partially grouted duct west of the centerline, which was flushed with water before setting of the grout. Despite flushing the grout duct until water ran clear, sufficient grout remained to activate this bar during the Knoxville DBE tests.
The maximum wall base rotation was 3.2-milliradian. The maximum chord strain on the third level was 0.0015-in./in. or 63% of the yield strain.

4.9.1.3 Knoxville DBE - trial 2

The Knoxville DBE – trial 2 test was conducted immediately following the trial 1 test on May 6, 2008. Rocking of the walls was observed, but no damage was apparent. The maximum chord strain on the third level was 66% of the yield strain.

Figure 4-26: Cracking (0.1-mm) in the second floor at column line A-3 after the Knoxville DBE – trial 2 test
4.9.1.4 Knoxville DBE - trial 3

The Knoxville DBE – trial 3 test was conducted on May 7, 2008. Despite three repeated tests, no damage was observed.

After testing, close inspection of the diaphragms showed minor cracking in the first two floors. The existing hairline cracks did not widen in either of the two levels. However, six additional cracks were present in the topped double tee floor. Nine of the thirteen joints on this level were cracked the full double tee length corresponding to the flange-to-flange joint. On the topped hollow-core floor, two hairline cracks at the center joint extended three to four feet from each end of the hollow-core units, see Figure 4-26. Temperature effects elongated the cracks on the two floors in the days following testing. The pretopped double tee system showed minor cracking in the flange at the wall-to-floor connection. These crack widths were 0.002-in (0.05-mm) and only visible on the bottom of the flange. Hairline cracks in the grout beds of the walls and columns were observed post-test. Crack widths are those measured in the test structure and are not translated to the prototype structure. No substantial damage was observed in the structure during the post test inspection. The maximum chord strain on the third level was 65% of the yield strain.

4.9.1.5 Seattle DBE - trial 1

The Seattle DBE – trial 1 test was conducted on May 21, 2008.

The first three Seattle DBE tests exposed problems with the chords of the pretopped double tee diaphragm at the third floor. Construction tolerances did not scale with the production or erection of the half scale test structure. Despite efforts to
maintain high quality during production and erection of the double tees, actual
collection tolerances were too large for the scaled building. Misalignment of chord
end plates in the horizontal and vertical directions produced offsets at full scale of ±2-in
(50.8-mm) and ±0.5-in (12.7-mm), respectively. Compounding the misalignment was
poor weld quality. In each of the first three Seattle DBE tests, weld fracture in the
pretopped chord connection resulted from the offsets and insufficient weld penetration
or thickness.

Weld fracture occurred at the critical in-plane flexure joint at the center of the
diaphragm during the Seattle DBE – trial 1 test, see Figure 4-27. Both chord connectors
failed at each end of the double tees. Tensile demand from diaphragm flexure was then
transferred to the shear connectors resulting in excessive demands on these connectors.
Damage was isolated to the center joint in the pretopped floor, but the center joint of the
topped hollow-core diaphragm at the floor below experienced large opening demands
when the pretopped double tee floor above lost diaphragm flexure strength. A 1.6-mm
crack was observed at the midspan joint on the hollow-core floor, see Figure 4-28. The
structure was stable and repairable.
Figure 4-27: Pretopped chord connection failure on the third floor after the Seattle DBE – trial 1 test

Figure 4-28: Cracking (1.6-mm) in the second floor at column line A-3 after the Seattle DBE – trial 1 test
4.9.1.6 Seattle DBE - trial 2

On June 12, 2008, the Seattle DBE – trial 2 test was conducted. Because of the localized damage in the Seattle DBE – trial 1, a repair was implemented which mimicked the dry chord connection and replaced the shear capacity of the center joint. Fractures of two spandrel-to-floor hairpin connections were welded to repair these connections in the double tee units adjacent to the center joint. An alternate shear connection replaced the failed shear connectors, and it is inferred that the spandrel-to-column connector, 0.5-in (12.7-mm) diameter threaded rod, yielded during testing at the center column as the anchor plate on the column was loose and not in contact with the column post-test.

A discontinuous pour strip was placed at either end of the double tees above the embedded chord connections. These curbs were placed on the center eight double tees with drill-and-bonded dowels providing continuity between the element and curb. The original chord connectors were cut at the center-most seven joints at either end of the double tees. This ensured that only the three #4 dual headed rebars placed in the discontinuous curbs would provide strength across the joints. The new chord reinforcement was cast with headed ends exposed at the joints. This allowed 5/8-in (15.9-mm) diameter by 12-in (304.8-mm) long weld slugs to be placed at the flange-to-flange joints between the heads of the chord bars over adjacent double tees. A field weld between the headed bars and weld slug mimicked the intended original conditions of a dry pretopped system for this test.

Inconsistent field welding left the critical flexural joint weaker than designed and once again vulnerable. During the Seattle DBE – trial 1 test, the field weld between
the weld slug and the headed chord reinforcement fractured. As the damage was once again isolated to the center joint, an immediate repair was made enlarging the field weld of the chord connection allowing a third Seattle DBE test.

4.9.1.7 Seattle DBE - trial 3

Immediately following the trial 2 test, the Seattle DBE – trial 3 test was conducted on June 12, 2008. A repair to the pretopped double tee floor involved enlarging the field weld of the chord connection. However, visual inspection of the failed chord connection after the Seattle DBE – trial 2 test did not identify an embedded chord fracture. The rebar fracture in the weld affected region was undetectable because of the surrounding concrete. As a result of this pre-existing weld fracture, a reduced capacity and asymmetric loading on the weld slug at this center joint were observed in the Seattle DBE – trial 3 test. The repeated failures of the critical flexure joint caused damage accumulation in the hollow-core diaphragm one floor below, see Figure 4-29.

Figure 4-29: Cracking (3.5-mm) in the second floor at column line A-3 after the Seattle DBE – trial 3 test
4.9.1.8 Seattle DBE - trial 4

The Seattle DBE – trial 4 test was the first of three tests conducted on June 20, 2008. The repair prior to the Seattle DBE – trial 4 test included new chord pour strips, a new shear connector, and the re-welding of the spandrel-to-floor hairpin connectors on the pretopped double tee diaphragm, see Figure 4-30 and 4-31. This repair involved the removal of the previous pour strip, new drill-and-bonded dowels, and six continuous #3 rebars which were lap spliced in two locations. The repair curb was placed at each end of the double tees and extended over the center-most nine joints.

A curb on the second floor hollow-core diaphragm at column lines 3A was implemented to repair the buckled chord reinforcement on the ledger beam end of the hollow-core units, see Figures 4-32 and 4-33. The topped hollow-core diaphragm had undergone larger than anticipated opening demands at the center joint in the three previous tests. As a result of the third floor failures, the opening demands were larger than expected for a high seismic MCE event. The accumulation of damage was obvious and the chord reinforcement in the topping of the second floor buckled during the Seattle DBE – trial 3 test. The repair curb with hooked rebars lap splicing the critical joint was placed over the chord rebars after exposing and cutting the buckled portion. Further evidence of the accumulation of damage is the strain demands on the ductile mesh at the center joint of the hollow-core floor.
Figure 4-30: Shake table test repair for the third floor diaphragm before the Seattle DBE – trial 4 test

Figure 4-31: Shake table test repair formwork on the third floor at column line A before the Seattle DBE – trial 4 test
Figure 4-32: Shake table test repair for the second floor at column line A before the Seattle DBE – trial 4 test

(a) Reinforcement

(b) Curb after repair

Figure 4-33: Shake table test repair on the second floor at column line A before the Seattle DBE – trial 4 test
Maximum wall base rotation measured during the fourth trial of the Seattle DBE was 8.7-millirad where two #7 energy dissipation bars were grouted in each wall. A torsional response was evident in the structure during post test analysis. The maximum chord strain on the third level was 0.00375-in./in. or 1.6 times the apparent yield strain indicating that localized yielding occurred in this test.

4.9.1.9 Berkeley DBE – trial 1

The Berkeley DBE – trial 1 test was conducted on June 20, 2008 following the Seattle DBE – trial 4 test. The same energy dissipation bars from the previous test were used for this test. Chord yielding in the third floor level was on the same order of magnitude as the Seattle DBE – trial 4 test. The maximum chord strain on the third level was 0.00428-in./in. or 1.8 times the apparent yield strain. Maximum wall base rotation measured at the neutral axis was 19.2-milliradian.

Post-test evaluation showed that post-tensioning tendon in the South wall at column line 5 fractured during this test. Initial investigation concluded that it was an energy dissipating rebar fracture [53], but data analysis of the jack monitoring the wall’s group tendon force shows a drop in the force corresponding to the loss of the average initial post-tensioning force for one tendon. The test damage were unaffected by this change, which went unnoticed until further data processing could be completed. However, it had significant repercussions for the subsequent test.
4.9.1.10 Berkeley MCE – trial 1

The Berkeley MCE – trial 1 test was conducted on June 20, 2008 following the Berkeley DBE – trial 1 test.

The test schedule permitted no time between tests to discover that one wall had a reduced capacity. The fractured post-tensioning tendon in the South wall was not detected prior to testing. This change in initial post-tensioning force and capacity affected the results of the Berkeley MCE – trial 1 test.

Increased demands and a reduced wall flexural capacity combined to overload the walls. Force demands in the wall post-tensioning tendons increased beyond the reduced capacity. Tendon failures at the anchor wedges further reduced the wall’s capacity causing large displacement demands. Tendon failure was likely a result of the method employed to seat and stress the tendons, but initiated at the anchor wedges, see Figure 4-34. Failure occurred at an average strand stress of 0.45-$f_{pu}$. The ten strands in each wall were simultaneously seated with hollow core jacks rather than individually seating each strand. The tendon failure was likely influenced by uneven force distribution amongst the strands. Standard industry practice of individually seating each wedge may have prevented the overstressing of tendons which contributed to the tendon fracture.
Figure 4-34: Wall tendon strand that fractured at the wedge anchors during the Berkeley MCE - trial 1 test

Large displacement demands because of the reduced strength and stiffness resulted in fracture of the column anchor bolts (Figure 4-35) and impact of the test structure with restraint towers placed on either side of the building as a precautionary measure for site safety. The towers were set back from the structure to allow a 4.5% drift. There was no floor damage as a consequence of the Berkeley MCE. However,
the slotted floor-to-wall shear connector travel capacity was exceeded due to the large wall uplift. When the travel was exceeded, the floor units were picked up by their connection to the wall. This was evident by dislodged bearing pads on each floor, see Figure 4-36. Floor-to-beam connections fractured on each floor as a result of the unique loading when pounding initiated with the towers. Concrete damage at the toe of the wall was limited to cosmetic spalling of the concrete cover, see Figures 4-37 and 4-38.

Cracking of the pretopped diaphragm was isolated to the center of the pour strip where residual cracks were on the order of 0.004-in (0.1-mm). On the topped hollow-core diaphragm, a crack of 0.039-in (1.0-mm) width was observed in the curb replacing the buckled chord reinforcement, see Figure 4-39. The topping at the center joint in this floor had a 0.177-in (4.5-mm) residual crack, which had grown 0.0394-in (1-mm) from the Seattle DBE – trial 3 test. Additional hairline cracking was observed in the topping at the interior column lines. The topped double tee deck developed cracks at most flange-to-flange joints, but crack widths remained small on the order of 0.0039-in (0.1-mm).

Despite the wall failure, the robust structure remained standing. Repairs to the wall were completed and the testing program was extended for an additional seven tests.
Figure 4-35: Column anchor bolt failures at column lines (a) A-5 and (b) B-5 after the Berkeley MCE - trial 1 test

Figure 4-36: Double-tee uplift after the Berkeley MCE - trial 1 test
Figure 4-37: South wall base after the Berkeley MCE - trial 1 test

Figure 4-38: North wall base after the Berkeley MCE - trial 1 test
4.9.1.11 60% Berkeley DBE – trial 1

On July 14, 2008, the 60% Berkeley DBE – trial 1 test was conducted. Repairs to the structure prior to testing included:

- Wall post-tensioning tendons were replaced with (5) 0.6" diameter strands per duct.
- The initial post-tensioning force was increased.
- A repair curb was added to the second floor. Additional WWR contributed to the diaphragm’s strength, but unbonded post-tensioning strands added to the curb were not anchored or stressed.
- Angles were added for beam-to-floor connections on all floors (epoxy filled oversize holes) that were snug tight.
• The wall energy dissipating bars were core drilled (all had buckled and/or fractured due to the Berkeley MCE – trial 1 uplift demands).

• The remaining energy dissipating bars were grouted (2 each wall furthest from the centerline - bars numbered 1 and 5 from the East).

The test outcome was failure of the South wall’s wall-to-floor connectors at the third level. Damage included concrete cracking around embed plates in end double tees.

4.9.1.12 60% Berkeley DBE – trial 2

On July 16, 2008, the 60% Berkeley DBE – trial 2 test was conducted. Prior to testing, the repairs to the structure included:

• Repaired the wall-to-floor connection on the third level, see Figure 4-40. An angle with vertical slots was added to make this connection. The angle was through bolted to the wall and welded to embedded plates exposed on the top of the double tees.

• This repair was done to the North and South walls while leaving the intact wall-to-floor connectors in the North wall in place.
No failure was observed in this test but damage accumulation continued as cracks widened and extended.

4.9.1.13 Berkeley DBE – trial 2

On July 16, 2008, the Berkeley DBE – trial 2 test was conducted. No additional repairs were implemented before this test. No failure was observed in this test but damage accumulation continued as cracks widened and extended. This was evident particularly in the third floor near the wall-to-floor connections.

4.9.1.14 Berkeley DBE – trial 3

On July 17, 2008, the Berkeley DBE – trial 3 test was conducted. The repairs to the test structure included:

- A steel channel section was added to the third level double tees next to the walls to strengthen the shear capacity. These channels were welded
to the angle connecting the wall to the floor and were through bolted to
the double tee flange past the double tee stem for anchorage.

No evident damage was presented itself within the test structure. However, the
slider bearings under the North outrigger beam both lost pressure. The repercussions of
this on the loading have not been assessed.

4.9.1.15 60% Berkeley DBE – trial 3

On July 22, 2008, the 60% Berkeley DBE – trial 3 test was conducted. Prior to
testing the following repairs were implemented:

- The green O-rings inside the slider bearings were replaced in both of the
  North slider bearings

To investigate the effect of a decreased moment to shear capacity ratio in the
diaphragm, three of the six chord bars were cut at the column lines 2 and 4 on the third
level. The bars were exposed and torch cut to accomplish this task, see Figure 4-41.

![Figure 4-41: Experimental results – floor three capacity reduction prior to the 60% Berkeley DBE - trial 3 test (a) locations and (b) photo documentation](image)
During testing, the second floor wall-to-floor connection failed in the South wall.

4.9.1.16 Berkeley DBE – trial 4

On July 23, 2008, the Berkeley DBE – trial 4 test was conducted. The repairs implemented prior to testing included:

- An angle with slotted vertical holes was bolted between the South wall and floor on the second floor.

Observed damage after this tests included further delamination around the embed plates of the third floor’s wall-to-floor connection. The dislodging of the North-West slider bearing’s jack, see Figure 4-42, may have influenced the test loading, but this has not been assessed.
4.9.1.17 Berkeley MCE – trial 2

On July 28, 2008, the Berkeley MCE – trial 2 test was conducted. The repairs implemented prior to testing included:

- The bearing pad and sliding surface under North-West slider bearing was replaced.
- The slider bearing’s green O-ring was replaced at this location.
- For additional wall strength and energy dissipation, angles were added to the wall bases (both faces of both walls), see Figure 4-43. The bottom leg of the angles were plug welded to embed plates in the outrigger beam on either side of the wall. The vertical leg of the angle was welded to an extension plate that was bolted to the wall. Two high strength bolts were anchored through the wall connecting the extension plates. Wall uplift engaged the extension plates and angles causing via these bolts.

Figure 4-43: Experimental results – wall repair prior to Berkeley MCE – trial 2 test (a) location and (b) photo documentation
A complete shear failure in third floor’s South wall-to-floor connection resulted in partial unseating of the third and second floors on the South-West side, see Figure 4-44.

Figure 4-44: Experimental results - Berkeley MCE - trial 2 test damage (a) location and (b) photo documentation

4.9.2 Processed results

4.9.2.1 System demands
Figure 4-45: Shake table test results – wall moment envelope (a) North wall and (b) South wall

Figure 4-46: Shake table test results – normalized wall moment demand (a) North wall and (b) South wall
Figure 4-47: Shake table test results – wall shear envelope (a) North wall and (b) South wall

Figure 4-48: Shake table test results – normalized shear demand (a) North wall and (b) South wall
Figure 4-49: Shake table test results – resultant lateral force location

Figure 4-50: Shake table test results – wall demand in the Knoxville DBE - trial 1
Figure 4-51: Shake table test results – wall demand in the Seattle DBE - trial 4

Figure 4-52: Shake table test results – wall demand in the Berkeley DBE - trial 1
Figure 4-53: Shake table test results – North wall moment rotation response

Figure 4-54: Shake table test results – South wall moment rotation response
4.9.2.2 Wall demands

Figure 4-55: Shake table test results – North wall post-tensioning response (a) West jack and (b) East jack

Figure 4-56: Shake table test results – South wall post-tensioning response (a) West jack and (b) East jack
Figure 4-57: Shake table test results – wall neutral axis depth variation in the Knoxville DBE - trial 1 for M>0.25Mₜₛ

Figure 4-58: Shake table test results – wall neutral axis depth variation in the Seattle DBE - trial 4 for M>0.25Mₜₛ
Figure 4-59: Shake table test results – wall neutral axis depth variation in the Berkeley DBE - trial 1 for M>0.25Mₜ
4.9.2.3 Diaphragm demands

Results are presented separately at each floor level. The three tests are plotted together for comparison purposes and results are presented in two formats; (1) average moment-curvature response at specific regions along the diaphragm span and (2) demands along the floor span. For figures presenting the diaphragm demand along the floor span, sub-figures (a) and (b) show a snapshot the demand in terms of moment and curvature, respectively, at peak moment demand, and sub-figure (c) shows the normalized moment of inertia obtained from the linear fit of the moment-curvature relationships of the preceding figure. These normalized moments of inertia are shown as discontinuous lines along the floor span representing the region over which the curvature was measured. Similarly, in sub-figure (b), the lines are discontinuous representing the gauge length of the measured deformation.

4.9.2.3.1 First floor
Figure 4-60: Shake table test results – moment curvature response of the first floor diaphragm
Figure 4-61: Shake table test results – first floor diaphragm results (a) moment demand, (b) curvature demand, and (c) rigidity.
4.9.2.3.2 Second floor

Figure 4-62: Shake table test results – moment curvature response of the second floor diaphragm
Figure 4-63: Shake table test results – second floor diaphragm results (a) moment demand, (b) curvature demand, and (c) rigidity
4.9.2.3.3 Third floor

In Figure 4-66, the effective moment of inertia, leff, is computed about the centroid of the flange because, in this test, the system was a pretopped and the gap between double tee units provided no concrete area to transform the centroid. In the subsequent tests shown, the continuous pour strip provided the concrete area to transform the section.
Figure 4-64: Shake table test results – moment curvature response of the third floor diaphragm
Figure 4-65: Shake table test results – third floor diaphragm results (a) moment demand, (b) curvature demand, and (c) rigidity.
4.9.2.3.4 Acceleration magnification factor

As a preliminary validation of the proposed modal FMR method discussed in CHAPTER 2, a comparison of the experimental results and modal FMR method is provided, see Figure 4-66. The overstrength factors, l, were based on experimental results of 1.4, 1.4, and 1.6 for the Knoxville, Seattle, and Berkeley sites, respectively. The response modification factors were 4.0, 4.5, and 6.0 for the Knoxville, Seattle, and Berkeley sites, respectively. The acceleration magnification factor for moment is calculated as $\Omega_M = \frac{8M_{dia}}{L \cdot w_x \cdot PGA}$. Observations from Figure 4-66 are that the method is unsuccessful at predicting the Knoxville DBE – trial 1 demand. Results from the Seattle – trial 4 and Berkeley DBE – trial 1 are sufficiently bounded by the FMR method.

Figure 4-66: Shake table test results – acceleration magnification factor for moment
4.9.2.4 Mode shapes

4.9.2.4.1 Knoxville DBE - trial 1

Figure 4-67: Shake table test results – diaphragm midspan Fourier amplitude from the Knoxville DBE - trial 1 test at (a) the 3rd floor, (b) the 2nd floor, and (c) the 1st floor

Figure 4-68: Shake table test results - diaphragm midspan response from the Knoxville DBE - trial 1 test
Figure 4-69: Shake table test results – mode shape obtained from Knoxville DBE - trial 1 test at 3.08 Hz

Figure 4-70: Shake table test results – mode shape obtained from Knoxville DBE - trial 1 test at 3.84 Hz

Figure 4-71: Shake table test results – mode shape obtained from Knoxville DBE - trial 1 test at 7.00 Hz
Figure 4-72: Shake table test results – mode shape obtained from Knoxville DBE - trial 1 test at 11.2 Hz

4.9.2.4.2 Seattle DBE - trial 4
Figure 4-73: Shake table test results – diaphragm midspan Fourier amplitude from the Seattle DBE - trial 4 test at (a) the 3rd floor, (b) the 2nd floor, and (c) the 1st floor

Figure 4-74: Shake table test results - diaphragm midspan response from the Seattle DBE - trial 1 test
Figure 4-75: Shake table test results – mode shape obtained from Seattle DBE - trial 1 test at 2.74 Hz

Figure 4-76: Shake table test results – mode shape obtained from Seattle DBE - trial 1 test at 5.76 Hz

Figure 4-77: Shake table test results – mode shape obtained from Seattle DBE - trial 1 test at 6.45 Hz
Figure 4-78: Shake table test results – mode shape obtained from Seattle DBE - trial 1 test at 7.86 Hz

Figure 4-79: Shake table test results – mode shape obtained from Seattle DBE - trial 1 test at 10.3 Hz

4.9.2.4.3 Berkeley DBE - trial 1
Figure 4-80: Shake table test results – diaphragm midspan Fourier amplitude from the Berkeley DBE - trial 1 test at (a) the 3rd floor, (b) the 2nd floor, and (c) the 1st floor

Figure 4-81: Shake table test results - diaphragm midspan response from the Berkeley DBE - trial 1 test
Figure 4-82: Shake table test results – mode shape obtained from Berkeley DBE - trial 1 test at 3.02 Hz

Figure 4-83: Shake table test results – mode shape obtained from Berkeley DBE - trial 1 test at 5.32 Hz

Figure 4-84: Shake table test results – mode shape obtained from Berkeley DBE - trial 1 test at 5.96 Hz
4.10 Summary

A large precast prestressed structure was built and tested under significant earthquake demands. The heavily instrumented test structure survived the demands with exceptional resilience. Some repairs were necessary to accommodate the challenges of constructing a half scale structure with full scale tolerances. The primary objective was achieved in that the results have provided comparison data with which further research goals could be obtained. The design methodology was demonstrated in
well performing design basis earthquakes for prototype sites in Knoxville, Seattle, and Berkeley. The toughness of this structure was proven by the extensive testing and an overall lack of damage to the precast elements over the three months of seismic testing. All structural damage was incurred in the connector or in the concrete at a joint interface between precast members.

During the Knoxville DBE tests, elastic chord reinforcement demonstrated elastic diaphragm response. Maximum strains below the elastic limit and minor diaphragm cracking highlight the level of diaphragm performance that can be achieved. The pretopped chord connection on the third floor showed no sign of damage at this level of testing despite considerable connection plate misalignment. With tighter construction tolerances, this connection may have also performed as intended under larger seismic demands. This could be demonstrated with individual connector tests at full scale with realistic offsets.

Flexural yielding was the observed as the primary floor response mode. No shear degradation was observed in the diaphragms. Through a capacity design approach, shear damage was precluded in these tests. No cracking was observed in the shear connectors on the pretopped third floor at the critical shear joint before the Berkeley MCE – trial 1 test. The new ductile mesh joint reinforcement in the toppings of the first two floors performed successfully without damage in the shear regions of the diaphragm and had an affect in the high flexural regions of the floor. The ductile mesh contributed to the in-plane flexural strength of the diaphragms, and at the center joint of the hollow-core floor sustained significant yielding in repeated Seattle DBE tests beyond the anticipated deformation capacity of a conventional mesh.
Despite unanticipated damage as a result of offsets due to construction tolerances, inadequate weld quality, chord buckling, or wall strand failure due to lower than intended initial wall strength, the diaphragm failures occurred in regions of high flexure as predicated in the capacity based design. Construction tolerance issues, which led to unanticipated failures, do not necessarily reflect the robustness of the structure as failures were repairable allowing testing to continue. The Berkeley maximum considered earthquake failure occurred outside of the primary region of interest and repair work permitted seismic testing beyond the intended test sequence.

The jointed nature of precast construction was made apparent as in phase response between elements was not reliable throughout testing. Pounding between elements resulted from out of phase movement. Concentrated damage was observed at the jointed connections with little damage spreading into or occurring elsewhere in the precast units.

The performance of a rocking wall and building were demonstrated under seismic loading. The wall’s self centering capabilities and superior performance benefitted the testing program by concentrating damage in the floors, the primary focus of the test program, rather than in the vertical components of the LFRS.

Diaphragm stiffness recommendations are based on Knoxville DBE results. Elastic diaphragm behavior was achieved with a peak strain in chord reinforcement measured as 66% of yield. For elastic behavior, results from the remaining tests are not appropriate because of the secant stiffness resulting from diaphragm nonlinearity.
CHAPTER 5  ANALYTICAL MODEL VALIDATION

5.1  Introduction

Validation is a critical component analytical modeling. Accurate estimation with an analytical model is dependent on the model’s ability to produce realistic response. To enhance the reliability of the analytical models, experimental results from the shake table test were utilized in a validation procedure to assess the accuracy of lower bound diaphragm stiffness estimate.

For each of the successful DBE tests, model validation was conducted. The Knoxville DBE – trial 1, Seattle DBE – trial 4, and the Berkeley DBE – trial 1 tests were relied upon. A numerical model of the test structure was generated based on the formulation for the analytical investigation of CHAPTER 6. Validation was conducted in model space using the test structure’s configuration and geometry.

5.2  Model development

The model incorporated the test structure’s estimated component weights, which provided an average floor weight that was lumped into nodes along the longitudinal axis of the diaphragm. The model was tailored to match pre-test conditions for each of the validation tests. This included initial post-tensioning force in the walls, energy dissipating bar presence, and reduced diaphragm flexural stiffness caused by damage accumulation.
The symmetry model discussed in CHAPTER 6 was implemented for this investigation. Torsional diaphragm response observed in the test results would not be captured. Similarly, inconsistency in the test structure’s North and South walls’ response was precluded with the symmetry model. A distinct difference between the test structure and the prototype structures of the analytical study is the wall type. In the analytical study, traditional reinforced concrete walls are used, but the experimental test structure incorporated rocking walls. To capture this difference, a rocking wall model was developed.

5.2.1 Rocking wall model development

A simple wall model was formulated to capture the behavior of a rocking wall with supplementary energy dissipation. The two-dimensional model incorporated the linear contact springs, beam-column elements, linear springs, and nonlinear springs. Features captured in this model included uplift of the wall, onset of yielding and strain hardening in each mild reinforcing bar used for energy dissipation, post-tensioning tendon demands, and a wall rotation and uplift response corresponding with experimental results.

A standard beam-column element was implemented for the behavior of the wall above the foundation. This accounted for the gross-section properties of the wall and tension stiffening. One element was implemented between each floor of the structure along the wall’s centroid. The core of the model revolved around the zero-length, linear contact springs capturing the interface between the wall base and the foundation. These were distributed along the length of the wall with concentration of springs within the
anticipated neutral axis depth. Contact spring stiffness was derived based on the rotation-uplift relationship observed in the experimental shake table test. The disturbed region does not conform to Euler-Bernoulli beam theory, which necessitated the calibration based on experimental results. Contact springs provided no resistance to the wall uplift and acted in uncoupled vertical and horizontal directions to capture contact and shear force transfer. The vertical component of the springs was used to estimate the neutral axis location. These springs were rigidly linked to a node at the centroid of the wall’s base. Rotational compliance due to the rigid link necessitated the zero-length spring and calibrated vertical stiffness.

Supplementary energy dissipation was provided by mild reinforcement. Nonlinear springs with the Dodd-Restrepo hysteresis [71] captured this behavior. Zero-length spring locations corresponded to the location of the rebars along the wall length. These were also rigidly connected to the node at the centroid wall’s base.

Post-tensioning tendons were modeled with elastic springs with the stiffness of the tendon group. Springs were used to capture each of the tendon groups in the walls. The vertical springs coincided with the tendon duct locations along the wall length, and had end nodes coordinates corresponding to the tendon anchor points. Nodes at the top of the wall were rigidly connected to the wall’s roof node. The springs at the anchor point below the wall were fixed. Initial axial load was applied through these springs and compensated for the axial deformation of the wall and contact springs to achieve the average tendon force in the test structure prior to testing.
5.2.2 Column model formulation

The gravity columns were pinned at the base and accounted for 50% of the gross
section properties of the columns used in the test.

5.2.3 Diaphragm model formulation

Floor elements and boundary conditions were consistent with the model
formulation discussed in CHAPTER 6.

Beam-column (frame) elements connecting floor nodes represented the effective
diaphragm behavior. Beams, spandrels, and their secondary connections to the floors
and columns were not modeled. This simplified structural configuration is consistent
with the model formulation of the analytical study in CHAPTER 6.

Elastic elements were used to model diaphragm behavior. Diaphragm flexural
stiffness was based on the observed moment-curvature response in the diaphragm in
each test. For the Seattle and Berkeley DBE tests, the secant stiffness of the non-linear
diaphragm was used, while diaphragms in the Knoxville DBE responded in the linear
range. Diaphragm shear stiffness was modeled based on the Eqn. 6.5.

5.2.4 Additional boundary conditions

A special boundary condition between the wall and floors was developed for this
model validation, but was not included in the models developed in CHAPTER 6.
Connectors between the test structure’s wall and floors allowed vertical uplift of the
wall. This was captured in the model by the addition of another node to accommodate
this boundary condition. The additional node was used as the end of the diaphragm and
slaved to the lateral displacement of the wall node. The vertical component was
decoupled from the wall’s uplift, but gravity columns provided the necessary vertical
support to the diaphragm end nodes.

5.3 Comparison of results

5.3.1 Knoxville DBE - trial 1

5.3.1.1 System demands
Figure 5-1: Model validation with the Knoxville DBE – wall overturning demand

Figure 5-2: Model validation with the Knoxville DBE – wall shear demand
Figure 5-3: Model validation with the Knoxville DBE – system overturning moment time history

Figure 5-4: Model validation with the Knoxville DBE – system shear time history
5.3.1.2 Wall demands

Figure 5-5: Model validation with the Knoxville DBE – wall neutral axis depth variation

Figure 5-6: Model validation with the Knoxville DBE – wall base rotation response
Figure 5-7: Model validation with the Knoxville DBE – wall post-tensioning response
(a) West jacks and (b) East jacks

5.3.1.3 Diaphragm demands

5.3.1.3.1.1 First floor
Figure 5-8: Model validation with the Knoxville DBE – first floor diaphragm moment curvature response
Figure 5-9: Model validation with the Knoxville DBE – first floor diaphragm demands (a) moment, (b) curvature, and (c) normalized rigidity
5.3.1.3.1.2 Second floor

Figure 5-10: Model validation with the Knoxville DBE – second floor diaphragm moment curvature response
Figure 5-11: Model validation with the Knoxville DBE – second floor diaphragm demands (a) moment, (b) curvature, and (c) normalized rigidity.
5.3.1.3.1.3 Third floor

Figure 5-12: Model validation with the Knoxville DBE – third floor diaphragm moment curvature response
Figure 5-13: Model validation with the Knoxville DBE – third floor diaphragm demands (a) moment, (b) curvature, and (c) normalized rigidity.
Figure 5-14: Model validation with the Knoxville DBE – roof diaphragm moment time history

Figure 5-15: Model validation with the Knoxville DBE – roof diaphragm shear time history
Figure 5-16: Model validation with the Knoxville DBE – roof drift time history

5.3.2 Seattle DBE - trial 4

5.3.2.1 System demands
Figure 5-17: Model validation with the Seattle DBE – wall overturning demand

Figure 5-18: Model validation with the Seattle DBE – wall shear demand
Figure 5-19: Model validation with the Seattle DBE – system overturning moment time history

Figure 5-20: Model validation with the Seattle DBE – system shear time history
5.3.2.2 Wall demands

Figure 5-21: Model validation with the Seattle DBE – wall neutral axis depth variation

Figure 5-22: Model validation with the Seattle DBE – wall base rotation response
Figure 5-23: Model validation with the Seattle DBE – wall post-tensioning response (a) West jacks and (b) East jacks

5.3.2.3 Diaphragm demands

5.3.2.3.1.1 First floor
Figure 5-24: Model validation with the Seattle DBE – first floor diaphragm moment curvature response
Figure 5-25: Model validation with the Seattle DBE – first floor diaphragm demands (a) moment, (b) curvature, and (c) normalized rigidity.
5.3.2.3.1.2 Second floor

Figure 5-26: Model validation with the Seattle DBE – second floor diaphragm moment curvature response
Figure 5-27: Model validation with the Seattle DBE – second floor diaphragm demands (a) moment, (b) curvature, and (c) normalized rigidity.
5.3.2.3.1.3 Third floor

Figure 5-28: Model validation with the Seattle DBE – third floor diaphragm moment curvature response
Figure 5-29: Model validation with the Seattle DBE – third floor diaphragm demands (a) moment, (b) curvature, and (c) normalized rigidity
Figure 5-30: Model validation with the Seattle DBE – roof diaphragm moment time history

Figure 5-31: Model validation with the Seattle DBE – roof diaphragm shear time history
5.3.3 Berkeley DBE - trial 1

5.3.3.1 System demands

Figure 5-32: Model validation with the Seattle DBE – roof drift time history
Figure 5-33: Model validation with the Berkeley DBE – wall overturning demand

Figure 5-34: Model validation with the Berkeley DBE – wall shear demand
Figure 5-35: Model validation with the Berkeley DBE – system overturning moment time history

Figure 5-36: Model validation with the Berkeley DBE – system shear time history
5.3.3.2 Wall demands

Figure 5-37: Model validation with the Berkeley DBE – wall neutral axis depth variation
Figure 5-38: Model validation with the Berkeley DBE – wall base rotation response

Figure 5-39: Model validation with the Berkeley DBE – wall post-tensioning response
(a) West jacks and (b) East jacks
5.3.3.3 Diaphragm demands

5.3.3.3.1.1 First floor

Figure 5-40: Model validation with the Berkeley DBE – first floor diaphragm moment curvature response
Figure 5-41: Model validation with the Berkeley DBE – first floor diaphragm demands (a) moment, (b) curvature, and (c) normalized rigidity
5.3.3.3.1.2 Second floor

Figure 5-42: Model validation with the Berkeley DBE – second floor diaphragm moment curvature response
Figure 5-43: Model validation with the Berkeley DBE – second floor diaphragm demands (a) moment, (b) curvature, and (c) normalized rigidity.
5.3.3.3.1.3 Third floor

Figure 5-44: Model validation with the Berkeley DBE – third floor diaphragm moment curvature response
Figure 5-45: Model validation with the Berkeley DBE – third floor diaphragm demands (a) moment, (b) curvature, and (c) normalized rigidity
Figure 5-46: Model validation with the Berkeley DBE – roof diaphragm moment time history

Figure 5-47: Model validation with the Berkeley DBE – roof diaphragm shear time history
Figure 5-48: Model validation with the Berkeley DBE – roof drift time history
CHAPTER 6  ANALYTICAL INVESTIGATION OF LONG SPAN PRECAST CONCRETE STRUCTURES WITH PERIMETER SHEAR WALLS

6.1 Introduction

The framework for evaluating seismic demands in structures with diaphragm flexibility was a large scope analytical study. Long span, precast concrete structures with perimeter shear walls were the focus of this investigation. The objective of the study was to evaluate system demands in wall structures with elastic diaphragms. For elastic diaphragm behavior, floors were designed to expected forces. The parametric study included analyses of simplified structures located at the four sites discussed in CHAPTER 3. Two prototype structure configurations were investigated. Based on these configurations, parameters such as the number of stories and floor aspect ratio were considered. Four values of in-plane diaphragm stiffness and a rigid diaphragm scenario were analyzed. The analytical study involved evaluating demands through nonlinear dynamic time history analyses. Seismic demands were assessed with ground motion excitation. The main parameters investigated are summarized in Table 6-1.

A design procedure resulted in wall and diaphragm reinforcement details for each prototype structure. Idealized component response was evaluated from these section level details. A numerical model of the prototype structure based on the idealized behavior was generated and analyzed in response to ten ground motion time histories.
Table 6-1: Analytical investigation variables

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Variable</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site</td>
<td>Knoxville, Charleston, Seattle, and Berkeley</td>
<td></td>
</tr>
<tr>
<td>Prototype configuration</td>
<td>A</td>
<td>200-ft floor span, 14-ft story heights</td>
</tr>
<tr>
<td>Number of stories, n</td>
<td>B</td>
<td>300-ft floor span, 10.5-ft story heights</td>
</tr>
<tr>
<td>Aspect ratio, AR</td>
<td>2.0, 2.5, 3.0, 3.5</td>
<td></td>
</tr>
<tr>
<td>Diaphragm flexibility</td>
<td>Effective stiffness</td>
<td>Connector type 1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Connector type 2</td>
</tr>
<tr>
<td></td>
<td>Lower bound stiffness</td>
<td>Connector type 1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Connector type 2</td>
</tr>
<tr>
<td></td>
<td>Rigid diaphragm</td>
<td></td>
</tr>
</tbody>
</table>

6.2 Prototype structures

Prototype structure configurations were defined by floor spans of 200 and 300-ft. Prototype configuration A had a floor span of 200-ft and story heights of 14-ft, see Figure 6-1 (a). Prototype configuration B represented a likely upper limit of floor span length at 300-ft and had story heights of 10.5-ft, see Figure 6-1 (b). Story heights were consistent at each level in the structure. Configuration A was intended to represent an office building while the configuration B had similarities to a parking structure. However, neither prototype configuration accounted for the complex layout of actual buildings such as openings or vertical irregularities. Furthermore, an average seismic floor weight of 125-psf was used for the design and modeling of each structure.
Structural walls were selected as the vertical components of the LFRS because of large floor demands attributed to this system. A wall layout consistent with maximizing diaphragm flexibility was implemented. Walls were located at either end of the longitudinal floor span to evaluate the demands in the transverse direction. Vertical elements of the LFRS in the longitudinal direction (orthogonal to the direction of loading) were ignored. Confining effects on the diaphragm caused by longitudinal walls or frames were not considered, but the contribution of gravity columns was included. Gravity columns were assumed to be 3-ft x 2-ft with the shorter dimension parallel with the wall’s longitudinal axis. Columns were spaced at 30-ft along the longitudinal span and approximately 45-ft along the transverse span.

Figure 6-1: Analytical investigation – schematic plan view of (a) prototype configuration A and (b) prototype configuration B
Floor diaphragms consisted of 10-ft wide double tee units. Although internal beams are necessary for gravity load continuity, the simplified prototype structures did not account for multiple bays or sub-diaphragms of precast floor units. The entire floor plan was considered as the diaphragm. This would mean that double tee units spanned the entire diaphragm depth, which was appropriate for only a limited number of prototype structures investigated. Topped and pretopped double tee systems were modeled. A 2-in. flange with 2-in. topping was the basis for the topped composite system. The pretopped system consisted of a 4-in. flange. Variation of practice throughout the United States and between individual designers was not feasible. The expertise of the DSDM Task Group was therefore relied upon for recommendations of representative details.

6.2.1 Design considerations

Design of the prototype structures followed ASCE 7-05 requirements [1]. A deviation from the provisions was implemented in the diaphragm design. This involved computing the diaphragm design force with the procedure outlined in CHAPTER 2. Design strength of the vertical LFRS was based on ASCE 7 requirements.

A response modification factor of 4.0 was used for the design of structures in the Knoxville and Charleston sites. For sites in Seattle and Berkeley, the response modification factor was taken as 6.0. Although the change in R value suggests a change in LFRS selection and detailing, this was not considered in the design procedure. The variation of response modification factor was intended to represent values likely used in the various regions of seismicity. The importance factor, I, and redundancy factors were taken as 1.0.
A structure’s fundamental period was estimated with the lowest value permissible by code provisions. Design forces were based on the approximate period \( T_a \) found by

\[
T_a = C_t h_n^x
\]

Eqn. 6.1

where \( h_n \) is the height above the base to the highest level of the structure, and \( C_t \) and \( x \) are constants depending on structural system.

6.2.1.1 Wall design

Walls were considered cast in place or following emulative design requirements for precast walls. Seismic loads were apportioned to the structure using the equivalent lateral force procedure. Overturning moment and shear were attributed equally among end walls neglecting the gravity columns’ contribution. At each end of the longitudinal floor span, the number of shear walls and their lengths depended on the overturning moment demand. When more than one wall was required at an end, the walls had consistent lengths. Considerations for the wall configuration included a wall aspect ratio greater than 2.5, a plastic hinge length contained between the ground and first floor, and a clear spacing between walls was kept as large as possible. A wall aspect ratio of 2.5 was selected to ensure dominant flexural behavior. This stipulation was usually but not always met. An aspect ratio below 2.5 was used in 8 out of 96 designs. The plastic hinge length was contained within the first floor for modeling considerations. Clear spacing between the walls attempted to minimize frame action between walls caused by out of plane rotations in the floor. Frame action was not
modeled, so the potential for this behavior was reduced with sufficient distance between walls. In 7 out of 96 designs, the clear spacing was less than 15-ft.

Based on the required strength, the number of walls and wall length were determined. Shear and moment demands were proportioned equally among the walls, and a single design performed for the identical walls of a particular prototype structure. The design was unique for each of the 96 structural configurations analyzed. Required longitudinal reinforcement was determined by assuming the tensile steel was located at a depth of $0.9l_w$ from the compression face. Nominal moment capacity was computed at a concrete compressive strain of 0.004. Specified concrete strength of 5-ksi and a steel strength of 60-ksi were utilized for this determination. Wall width was adjusted on an individual design basis to refine the capacity. Reinforcement ratios less than 0.5% were evenly distributed along the wall length. Lumped reinforcement was located at either end of the wall when the reinforcement ratio exceeded 0.5%. However, minimum reinforcement was provided between these confined regions. A continuous wall width was used along the wall length and up the height of the wall. Reinforcement details did not account for longitudinal bar termination up the height of the wall.

The required steel area computed at nominal capacity was distributed into standard bar sizes. Maximum spacing between bars was 18-in. This spacing was used to determine bar sizes for evenly distributed reinforcement or between confined regions. Bars within the confined regions were spaced at 8-in. on center along the wall length and three layers were provided within the wall. Based on this spacing, the bar size that met or exceeded the required steel area was determined. Longitudinal bars no larger than No. 11 were considered, so the confined region was lengthened to accommodate a
smaller bar or a redesign of the wall configuration (number, length, or width) was implemented. Confinement was provided at a spacing of six longitudinal bar diameters by stirrups whose diameter was based on recommendations by Paulay and Priestley [68]. Wall shear failures were not modeled in the analyses so sufficient shear reinforcement must be provided to prevent this mode of failure. The design process, however, did not address shear reinforcement requirements.

With reinforcement layout completed, a section analysis was conducted on the wall. The moment curvature program was developed in Matlab for rectangular sections. Expected material properties for steel were utilized in the program, which accounted for confined and unconfined concrete. The specified strength, $f'_c$, in the model was taken as 5-ksi and the elastic modulus was estimated as 3,828-ksi. The confined concrete material model was based on the formulation by Mander [69]. Steel reinforcement was modeled with Mander’s model for steel [70]. Expected yield stress of 67-ksi and an 82-ksi ultimate stress were used as the expected steel material properties. The elastic modulus for steel was taken as 29,000-ksi. The onset of strain hardening was modeled at 0.018 and the strain at ultimate stress was 0.13. Ninety percent of the dead load from the wall self weight and tributary floor area was included in the analyses. The tributary floor area accounted for the wall length plus 15-ft at each end and 15-ft of the floor span. Design parameters for the walls are provided in Appendix.

An idealized moment-curvature response was obtained from the detailed analysis for input to the MDOF model. A tri-linear curve captured the uncracked, cracked, and yielded response of the wall. Parameters defining the tri-linear curve are found in Appendix.
6.2.1.2 Diaphragm design

For an elastic design, the diaphragm forces needed adequate estimation. This was accomplished with the modal superposition method for horizontal floor accelerations discussed in section 2.3.2. These accelerations correspond to forces significantly larger than current code design values. The same design force was used for each floor level.

The diaphragms were considered as simply supported by the perimeter walls. A uniform distribution of diaphragm force along the floor span was used to calculate required moment and shear strength from the assumed boundary conditions. Strength reduction factors of 0.9 for bending and 0.75 for shear were used to obtain the required nominal capacity. This capacity was met by accounting for the web connectors’ tensile capacity and the chord reinforcement.

Nominal moment capacity was computed based on specified material strengths of 60 Ksi. Web connectors and topping mesh were assumed to be distributed at 12-ft on center within the midspan diaphragm region. Assumed connector tensile capacity is provided in Table 6-2. Flange connector number 1 was intended to represent a pretopped flange connection. Connector number 2 was intended to represent a topped flange-to-flange connector for composite diaphragm action. The connector and topping mesh were considered in the flexural strength of the topped system to calculate the required chord reinforcement. However, the distinction between connectors for this study relies on the variation of shear stiffness because their tensile contribution to the diaphragm flexural stiffness was not modeled.
Table 6-2: Diaphragm connector properties

<table>
<thead>
<tr>
<th>Diaphragm</th>
<th>Connector</th>
<th>Tension</th>
<th>Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Stiffness [kip/in]</td>
<td>Nominal strength [kip]</td>
</tr>
<tr>
<td>Pretopped</td>
<td>Conn. 1</td>
<td>546</td>
<td>18.6</td>
</tr>
<tr>
<td>Topped</td>
<td>Conn. 2</td>
<td>1273</td>
<td>25.5</td>
</tr>
<tr>
<td>wwr @ 6-in.</td>
<td></td>
<td>-</td>
<td>1.5</td>
</tr>
</tbody>
</table>

The required chord reinforcement formulated the effective in-plane diaphragm flexural stiffness. Two effective stiffness values were used in the modeling approach. One was formulated as a lower bound estimation of diaphragm stiffness to account for cracked section properties. The formulation accounted only for the contribution of the chord reinforcement to the cracked section’s moment of inertia. With a section centroid in the compression zone near approximated at the extreme compression fiber and chord reinforcement acting at the opposite diaphragm edge, the distance between the centroid and chord steel is approximated as the diaphragm depth, \(d_{dia}\). This is an oversimplification of the actual reinforcement layout and centroid but provides an efficient initial approximation. With the steel area of the chord reinforcement, \(A_{s,chord}\), used to compute the moment of inertia by the parallel axis theorem and neglecting the small contribution of the rebar’s moment of inertia about its own axis:

\[
I_{lb} = A_s d_{dia}^2
\]

Eqn. 6.2

For a comparison with the rigidity of the gross concrete section, this equation must be transformed by the modular ratio. The lower bound estimation, \(I_{lb}\), can then be normalized by the concrete’s gross section moment of inertia to obtain

\[
\frac{I_{lb}}{I_g} = 12 \frac{A_{s,chord}}{A_g} \frac{E_s}{E_c}
\]

Eqn. 6.3
where $A_g$ is the gross concrete area of the flange. This estimation highlights the direct correlation between diaphragm stiffness and strength. Although a rough approximation of actual diaphragm stiffness, it provides a lower bound upon which the analyses can be assessed.

Comparison with experimental testing results showed the “lower bound” estimation to be too low when considering elastic diaphragm behavior and the tension stiffening contribution of concrete, which was significant with concentrated deformation at pre-cracked joints. Therefore, a second stiffness, referred to as “effective” diaphragm stiffness was included in the analytical study. This stiffness accounted for the lower bound estimation at the joint acting over a 20 bar diameter gauge length. Gross concrete section properties accounted for the flexural stiffness over the remaining double tee width. The effective stiffness was computed as:

$$I_{eff} = \frac{(b - 20d_b)I_g + 20d_bI_{lb}}{b}$$

Eqn. 6.4

where $b$ is the double tee width, $d_b$ is the chord bar diameter, $I_g$ is the gross section moment of inertia for the flange, and $I_{lb}$ is the lower bound stiffness estimate.

The number of shear connectors was determined based on the nominal strength required, but accounted for the shear capacity of the chord reinforcement which was continuous along the floor span. Termination of chord reinforcement is a likely design scenario, but was not accounted for in this study. The chord shear stiffness was based on a 165-kip/in/bar estimate for a No. 6 bar.

Effective shear stiffness was obtained by accounting for a different stiffness in the flange and at the joint. These were assumed to act as springs in series. The joint
shear stiffness comprised contributions from the connectors and chord acting over a 6.5-
in. gauge length. Flange shear stiffness accounted for the gross sectional area of the
flange over the remaining double tee width. The effective shear stiffness was calculated
as:

$$K_v = \frac{1}{\left( n_{\text{conn}} K_{v,\text{conn}} + 2 n_{\text{chord}} K_{v,\text{chord}} \right) + \frac{b - g_v}{G_c \frac{5}{6} A_g}}$$

Eqn. 6.5

where $n_{\text{conn}}$ is the number of connectors required for shear capacity, $K_{v,\text{conn}}$ is the shear
stiffness of an individual connector, $n_{\text{chord}}$ is the number of chord bars required at either
end of the diaphragm, $K_{v,\text{chord}}$ is the chord shear stiffness of an individual chord bar, $b$ is
the double tee width, $g_v$ is the 6.5-in. gauge length over which the joint shear is
considered effective, $G_c$ is the concrete shear modulus, and $A_g$ is the gross sectional area
of the flange.

6.3 Modeling approach

Nonlinear dynamic time history analyses were conducted on numerical models
of the prototype structures. Necessitated by the large number of analyses, the efficiency
of a structural analysis program was selected over a more detailed finite element
analysis approach. The structural analysis program Ruaumoko3D [71] was utilized to
conduct these analyses. A three dimensional framework was necessitated by the nature
of the study. To capture in-plane diaphragm demands and apply gravity loads, a three
dimensional model was required.
The modeling technique implemented has the distinct advantage of computational efficiency. For the number of analyses required in the scope of this study, a more rigorous modeling approach would have significantly increased computational demands. The ability to undertake a larger scope outweighed the benefits of a more detailed model. Effective behavior modeled did not capture local effects. However, the global response estimations provided from element forces coincided with the investigation objective.

6.4 Model description

The simplified model did not capture internal force paths around openings or discontinuities. Discontinuities between multiple bays of sub-diaphragms were neglected. This assumption was facilitated by the design procedure that considered chord reinforcement at either end of the diaphragm.

Modeling involved simplification of the structure to a basic configuration of walls, columns and floor elements. Each of these was modeled with one-component Giberson beam elements [71]. A visual representation of this idealization is shown in Figure 6-1 for a three story prototype structure. Nonlinear behavior was restricted to the wall elements, which were modeled with a tri-linear backbone curve accounting for cracking and yielding. Diaphragm flexibility included flexural and shear deformations.
Figure 6-2: Analytical investigation – model idealization for n=3

An important boundary condition was enforced in the model formulation. Symmetry of the building about the diaphragm midspan was exploited. Only one half of the simplified model was analyzed, see Figure 6-2. By enforcing the appropriate boundary conditions at the diaphragm midspan, the computational and storage requirements were drastically reduced. Nodes at the diaphragm midspan were restrained from rotating about the vertical axis. Lateral displacement was allowed in the diaphragm transverse direction (the direction of ground motion excitation), but restricted in its longitudinal direction. These boundary conditions eliminated torsional response, which was observed in the experimental shake table test.
Wall-to-floor connections were modeled as rigid. Wall torsional rigidity was neglected and the possible confining effect of end walls was ignored. If end walls produce axial load variations in the floors, this affect would not have been captured by the modeling approach as there was no interaction of axial load and moment capacity in the beam-column elements. Modeling of the end walls as a single element assumed that forces transferred among them equally. Walls were fully fixed at their base, which excluded foundation flexibility.

Columns and floors were rigidly connected, but the absence of diaphragm torsional stiffness imposed no rotational restraint to the column or wall elements.
Columns were pinned at their base in an attempt to minimize their contribution to the lateral resistance.

Floor nodes were free to displace in the direction of excitation and the vertical direction. This permitted the application of gravity load at nodes, which introduced P-Δ effects based on the dead load estimation.

The three-dimensional analyses included only one component of excitation. This was in the direction of the transverse diaphragm axis. Lumped mass based on the 125-psf average floor weight was provided only in the direction of excitation. A constant damping model [72] was implemented with 3% damping in all modes. The damping value was justified by results of the shake table test validation in CHAPTER 5.

A material modulus equal to the assumed concrete modulus (3,828-ksi) was used for each element in the model. Columns were assumed elastic with section properties based on 50% of the gross section. Diaphragm shear area was found from the effective shear stiffness of section 6.2.1.2. For a given wall design, both the lower bound estimate and effective stiffness were implemented in diaphragm models. Each of these was evaluated with for the pretopped and topped systems referred to as connector type 1 and connector type 2 in the results. For the same wall design, a rigid diaphragm analysis was also performed. This was implemented by slaving nodes along the floor to the displacement of the appropriate wall node.

Calibration of the parameters defining cyclic behavior of the wall hysteresis was obtained through the hysteresis model validation of section 7.3 for a wall subjected to cyclic lateral loads. Calibration of parameters $\alpha$, $\beta$, and Pinch to match the experimental results was relied upon for the hysteresis input of the walls analyzed in
this study. These values are likely functions of the axial load and reinforcement ratio. This would necessitate a calibration for the walls of each prototype structure. In the absence of experimental tests matching each wall design, the parameters calibrated for the single test were deemed sufficient. Small variation in these parameters is not expected to significantly impact system demands.

6.5 Results

Results of the analytical study are provided in terms of the mean response of the ten ground motions in a record set. The peak demand of each analysis was obtained for the results in Figures 6-4 through 6-67. Demands are quantified for system and diaphragm demands. All demands are presented in as related to the structure’s height. The vertical axis of each figure is the building height normalized by roof height.

System demands are presented in terms of overturning moment and shear. They include the columns’ contribution to the resisted forces. Overturning moment demands were normalized by the idealized yield strength of the walls. The discrepancy between the design basis, or code required strength, and demand is a result of the strength reduction factor, expected material strengths, excess of reinforcement, and overstrength. Increased demands are also a result of the higher mode effects evident in the shape of the moment diagram.

System shear demands are normalized by the seismic weight of the structure. Discrepancies between the required shear capacity (design basis) and the shear demand exist for the same reasons as for the overturning moment. To ensure valid results, the
wall’s shear capacity must exceed the resulting demands to preclude shear failure, which was not considered in the model formulation.

Curvature demands in the walls are reported in terms of curvature ductility. Interstory drift ratios were computed at the midspan and end of the diaphragm. These are also included in the category for system demands as they pertain to the wall and gravity columns. The code allowable drift ratio of 0.02 is included in these results as the design basis.

Diaphragm results include moment, shear, and deformation demands in terms of the code definition for a flexible diaphragm. Force demands are presented in normalized terms as an acceleration magnification factor. The acceleration magnification factor for moment, $\Omega_M$, was calculated as:

$$\Omega_M = \frac{8M_{dia}}{L \cdot W \cdot PGA}$$  \hspace{1cm} \text{Eqn. 6.6}

where $M_{dia}$ is the peak diaphragm moment demand, $L$ is the floor span length, $W$, is the floor weight of the diaphragm, and $PGA$ is the peak ground acceleration normalized by the acceleration of gravity. This assumes a simply supported boundary condition for the floor, which is consistent with the model formulation. The acceleration magnification for shear, $\Omega_V$, was calculated from the peak diaphragm shear demand, $V_{dia}$ as:

$$\Omega_V = \frac{2V_{dia}}{L \cdot W \cdot PGA}$$  \hspace{1cm} \text{Eqn. 6.7}

These terms represent the uniformly distributed horizontal acceleration necessary to produce the diaphragm demand resulting from time history analysis. This acceleration is normalized by the peak ground acceleration, and mean results of the ten
ground motions presented. These magnification factors do not account for the acceleration profile at peak demand or for higher mode contributions within the diaphragm. Included in these results are the design forces from the equivalent lateral force procedure and the expected acceleration magnification factor based on the modal first mode reduced method formulation of section 2.3.2. To present a consistent comparison, the expected magnification factors were computed with the peak ground acceleration and spectral acceleration ordinate at the approximate building period of each ground motion. The mean value of the record set is shown with the results. The mean acceleration magnification factor based on the modal first mode reduced method was computed as:

$$\Omega_{FMR} = \frac{1}{10} \sum_{i=1}^{10} \frac{\left( \lambda \eta \rho IS_{a,1} \left( T_a, 0.05 \right) \right)^2 + 1.4 \sqrt{n - 1} PGA_i^2}{PGA_i}$$

Eqn. 6.8

where \( i \) is the ground motion considered, \( \lambda \), is the overstrength factor of 1.75, \( \lambda \), is the overstrength factor of 1.5, \( \rho \), is the redundancy factor taken from the design as 1.0, \( I \), is the importance factor taken from design as 1.0, \( S_{a,1} \), is the ground motion’s spectral acceleration at the approximate period and 5% damping, \( R \) is the response modification factor, \( n \) is the number of stories, and \( PGA \), is the peak ground acceleration of ground motion \( i \). As proposed in section 2.3.2, the roof acceleration magnification factor is used for floors at or above 15% of the roof height and below this height a linear variation to the peak ground acceleration at the ground.

Current code provisions were included in these results. The diaphragm floor forces were calculated from the equivalent lateral force procedure including upper and
lower limits. They were not used in the design process, but are included to illustrate the extent of underestimation. Normalized by the weight of the floor, the floor forces approach the base shear coefficient at the base of the structure. For comparison purposes, these accelerations are normalized by the average peak ground acceleration.

Diaphragm deformation demands are presented as they pertain to the code definition of a flexible diaphragm [1]. A flexible diaphragm condition is met when the ratio of the maximum diaphragm deflection (MDD) to the average drift of vertical elements (ADVE) is greater than two. This condition is shown with the analytical results.

In Figures 6-4 through 6-67, the results of the variation in diaphragm flexibility are presented. To succinctly show the remaining parameters, the variation of aspect ratio, AR, and the number of stories investigated are shown in figures (a) through (l). The variation of aspect ratio is shown in the rows of figures (a) through (l) and the number of stories varies across the columns. The scale and axis limits are consistent in each sub-figure (a) though (l). Each sub-figure includes the diaphragm stiffness parameters for shear and flexure as modeled for every floor. To the right of each figure, the moment of inertia is normalized by the gross section moment of inertia and the shear area is normalized by the gross flange area. These provide the variation of diaphragm stiffness investigated in the study.

6.5.1 Summary

In general, observations from the analytical study are:
• System overturning moment demands show the effects of higher modes in the demand at upper floors.

• System shear demands also exhibit higher mode demands as evident in the distribution of shear demand up the structure.

• Interstory drift demands at the wall were all within acceptable limits. Interstory drift ratio demands at the diaphragm midspan were all below 3%.

• Curvature ductility demands were less than ten for all sites. Average response in the Knoxville site showed little post-yield deformation demand.

• Diaphragm demands were bound by the modal first mode reduced method in all cases for the Knoxville, Seattle, and Berkeley sites.

• For the Charleston site, 52 of 120 cases had $\Omega_M$ demands greater than the estimate by the first mode reduced method. However, the maximum exceedance was only 17%. Each of these cases were at the roof of the structure and in buildings with $n=5$ or 8.

• Acceleration magnification factor for shear was lower than the factor for moment. This is likely due to the presence of gravity columns that contributed to the lateral force resistance but provided no torsional resistance to the in-plane diaphragm moment capacity.

• Results pertaining to the definition of a flexible diaphragm show significant scatter, but a general trend is that the lower floors exceed the limiting factor of 2.0 while upper floors are well below this value.
The floor span of 300-ft, configuration B, accounted for all but one instance of interstory drift ratios exceeding 2%. The one instance of a drift exceeding 2% in the Berkeley site for prototype configuration A, L=200-ft, was only marginally larger than this value. Out of sixty analyses for each site with prototype configuration B, none exceeded the drift limit at the Knoxville site, thirteen exceeded the drift limit at the Charleston site, two exceeded the limit at the Seattle site, and thirty exceeded the limit for the Berkeley site.

6.5.2 Knoxville DBE
6.5.2.1 System demands
6.5.2.1.1 Forces
Figure 6-4: Analytical results – Knoxville site, prototype configuration A – system moment demand
Figure 6-5: Analytical results – Knoxville site, prototype configuration B – system moment demand
Figure 6-6: Analytical results – Knoxville site, prototype configuration A – system shear demand

Knoxville site
R = 4
Floor span = 200-ft
Figure 6-7: Analytical results – Knoxville site, prototype configuration B – system shear demand

Knoxville site
R = 4
Floor span = 300-ft
6.5.2.1.2 Deformations

Figure 6-8: Analytical results – Knoxville site, prototype configuration A – interstory drift ratio at the wall
Figure 6-9: Analytical results – Knoxville site, prototype configuration B – interstory drift ratio at the wall.
Figure 6-10: Analytical results – Knoxville site, prototype configuration A – interstory drift ratio at the diaphragm midspan
Figure 6-11: Analytical results – Knoxville site, prototype configuration B – interstory drift ratio at the diaphragm midspan.
Figure 6-12: Analytical results – Knoxville site, prototype configuration A – wall curvature ductility demand
Figure 6-13: Analytical results – Knoxville site, prototype configuration B – wall curvature ductility demand

Knoxville site
R = 4
Floor span = 300-ft
6.5.2.2 Diaphragm demands

6.5.2.2.1 Forces

Figure 6-14: Analytical results – Knoxville site, prototype configuration A – floor acceleration magnification factor based on diaphragm moment.
Figure 6-15: Analytical results – Knoxville site, prototype configuration B – floor acceleration magnification factor based on diaphragm moment
Figure 6-16: Analytical results – Knoxville site, prototype configuration A – floor acceleration magnification factor based on diaphragm shear
Figure 6-17: Analytical results – Knoxville site, prototype configuration B – floor acceleration magnification factor based on diaphragm shear.
6.5.2.2.2 Deformations

Figure 6-18: Analytical results – Knoxville site, prototype configuration A – code diaphragm flexibility factor.
Figure 6-19: Analytical results – Knoxville site, prototype configuration B – code diaphragm flexibility factor
6.5.3 Charleston DBE

6.5.3.1 System demands

6.5.3.1.1 Forces

![Analytical results – Charleston site, prototype configuration A – system moment demand](image)

**Charleston site**

**R = 4**

**Floor span = 200-ft**

Figure 6-20: Analytical results – Charleston site, prototype configuration A – system moment demand
Figure 6-21: Analytical results – Charleston site, prototype configuration B – system moment demand
Figure 6-22: Analytical results – Charleston site, prototype configuration A – system shear demand
Figure 6-23: Analytical results – Charleston site, prototype configuration B – system shear demand
6.5.3.1.2 Deformations

Figure 6-24: Analytical results – Charleston site, prototype configuration A – interstory drift ratio at the wall
Figure 6-25: Analytical results – Charleston site, prototype configuration B – interstory drift ratio at the wall
Figure 6-26: Analytical results – Charleston site, prototype configuration A – interstory drift ratio at the diaphragm midspan

Charleston site
R = 4
Floor span = 200-ft
Figure 6-27: Analytical results – Charleston site, prototype configuration B – interstory drift ratio at the diaphragm midspan.
Figure 6-28: Analytical results – Charleston site, prototype configuration A – wall curvature ductility demand
Figure 6-29: Analytical results – Charleston site, prototype configuration B – wall curvature ductility demand
6.5.3.2 Diaphragm demands

6.5.3.2.1 Forces

Figure 6-30: Analytical results – Charleston site, prototype configuration A – floor acceleration magnification factor based on diaphragm moment
Figure 6-31: Analytical results – Charleston site, prototype configuration B – floor acceleration magnification factor based on diaphragm moment.
Figure 6-32: Analytical results – Charleston site, prototype configuration A – floor acceleration magnification factor based on diaphragm shear
Figure 6-33: Analytical results – Charleston site, prototype configuration B – floor acceleration magnification factor based on diaphragm shear

Charleston site
R = 4
Floor span = 300-ft
6.5.3.2.2 Deformations

Figure 6-34: Analytical results – Charleston site, prototype configuration A – code diaphragm flexibility factor
Figure 6-35: Analytical results – Charleston site, prototype configuration B – code diaphragm flexibility factor
6.5.4 Seattle DBE

6.5.4.1 System demands

6.5.4.1.1 Forces

Figure 6-36: Analytical results – Seattle site, prototype configuration A – system moment demand
Figure 6-37: Analytical results – Seattle site, prototype configuration B – system moment demand
Figure 6-38: Analytical results – Seattle site, prototype configuration A – system shear demand
Figure 6-39: Analytical results – Seattle site, prototype configuration B – system shear demand
6.5.4.1.2 Deformations

Figure 6-40: Analytical results – Seattle site, prototype configuration A – interstory drift ratio at the wall

Seattle site
R = 6
Floor span = 200-ft
Figure 6-41: Analytical results – Seattle site, prototype configuration B – interstory drift ratio at the wall
Figure 6-42: Analytical results – Seattle site, prototype configuration A – interstory drift ratio at the diaphragm midspan.
Figure 6-43: Analytical results – Seattle site, prototype configuration B – interstory drift ratio at the diaphragm midspan.
Figure 6-44: Analytical results – Seattle site, prototype configuration A – wall curvature ductility demand
Figure 6-45: Analytical results – Seattle site, prototype configuration B – wall curvature ductility demand
6.5.4.2 Diaphragm demands

6.5.4.2.1 Forces

Figure 6-46: Analytical results – Seattle site, prototype configuration A – floor acceleration magnification factor based on diaphragm moment

Seattle site
R = 6
Floor span = 200-ft
Figure 6-47: Analytical results – Seattle site, prototype configuration B – floor acceleration magnification factor based on diaphragm moment.
Figure 6-48: Analytical results – Seattle site, prototype configuration A – floor acceleration magnification factor based on diaphragm shear
Figure 6-49: Analytical results – Seattle site, prototype configuration B – floor acceleration magnification factor based on diaphragm shear
6.5.4.2.2 Deformations

Figure 6-50: Analytical results – Seattle site, prototype configuration A – code diaphragm flexibility factor
Figure 6-51: Analytical results – Seattle site, prototype configuration B – code diaphragm flexibility factor
6.5.5 Berkeley DBE

6.5.5.1 System demands

6.5.5.1.1 Forces

Figure 6-52: Analytical results – Berkeley site, prototype configuration A – system moment demand
Figure 6-53: Analytical results – Berkeley site, prototype configuration B – system moment demand
Figure 6-54: Analytical results – Berkeley site, prototype configuration A – system shear demand
Figure 6-55: Analytical results – Berkeley site, prototype configuration B – system shear demand

Berkeley site
R = 6
Floor span = 300-ft
6.5.5.1.2 Deformations

Figure 6-56: Analytical results – Berkeley site, prototype configuration A – interstory drift ratio at the wall
Figure 6-57: Analytical results – Berkeley site, prototype configuration B – interstory drift ratio at the wall.
Figure 6-58: Analytical results – Berkeley site, prototype configuration A – interstory drift ratio at the diaphragm midspan
Figure 6-59: Analytical results – Berkeley site, prototype configuration B – interstory drift ratio at the diaphragm midspan
Figure 6-60: Analytical results – Berkeley site, prototype configuration A – wall curvature ductility demand
Figure 6-61: Analytical results – Berkeley site, prototype configuration B – wall curvature ductility demand
6.5.5.2 Diaphragm demands

6.5.5.2.1 Forces

Figure 6-62: Analytical results – Berkeley site, prototype configuration A – floor acceleration magnification factor based on diaphragm moment
Figure 6-63: Analytical results – Berkeley site, prototype configuration B – floor acceleration magnification factor based on diaphragm moment
Figure 6-64: Analytical results – Berkeley site, prototype configuration A – floor acceleration magnification factor based on diaphragm shear
Figure 6-65: Analytical results – Berkeley site, prototype configuration B – floor acceleration magnification factor based on diaphragm shear
6.5.5.2.2 Deformations

Figure 6-66: Analytical results – Berkeley site, prototype configuration A – code diaphragm flexibility factor

Berkeley site
R = 6
Floor span = 200-ft
Figure 6-67: Analytical results – Berkeley site, prototype configuration B – code diaphragm flexibility factor
6.5.6 Additional validation parameters investigated

6.5.6.1 Introduction

Additional analyses were conducted to verify selected variables. These included the variation of assumed floor weight at and variation of response modification factor. The floor weight variation was performed at the Berkeley site and results are presented in Figure 6-68 through Figure 6-75. A response modification factor of 6.0 was investigated at the Charleston site, which replaced the previously assumed value of 4.0. These results are presented in Figures 6-82 through 6-89.

6.5.6.2 Floor weight variation

6.5.6.2.1 System demands

6.5.6.2.1.1 Forces
Figure 6-68: Analytical results – Berkeley site, prototype configuration A with 140-psf floor weight – system moment demand
Figure 6-69: Analytical results – Berkeley site, prototype configuration B with 110-psf floor weight – system moment demand

Berkeley site

R = 6

Floor span = 300-ft
Figure 6-70: Analytical results – Berkeley site, prototype configuration A with 140-psf floor weight – system shear demand
Figure 6-71: Analytical results – Berkeley site, prototype configuration B with 110-psf floor weight – system shear demand
6.5.6.2.1.2 Deformations

Figure 6-72: Analytical results – Berkeley site, prototype configuration A with 140-psf floor weight – interstory drift ratio at the wall

Berkeley site
R = 6
Floor span = 200-ft
Figure 6-73: Analytical results – Berkeley site, prototype configuration B with 110-psf floor weight – interstory drift ratio at the wall.
Figure 6-74: Analytical results – Berkeley site, prototype configuration A with 140-psf floor weight – interstory drift ratio at the diaphragm midspan.
Figure 6-75: Analytical results – Berkeley site, prototype configuration B with 110-psf floor weight – interstory drift ratio at the diaphragm midspan
Figure 6-76: Analytical results – Berkeley site, prototype configuration A with 140-psf floor weight – wall curvature ductility demand
Figure 6-77: Analytical results – Berkeley site, prototype configuration B with 110-psf floor weight – wall curvature ductility demand
6.5.6.2.2 Diaphragm demands
6.5.6.2.2.1 Forces

Figure 6-78: Analytical results – Berkeley site, prototype configuration A with 140-psf floor weight – floor acceleration magnification factor based on diaphragm moment

Berkeley site
R = 6
Floor span = 200-ft
Figure 6-79: Analytical results – Berkeley site, prototype configuration B with 110-psf floor weight – floor acceleration magnification factor based on diaphragm moment
Figure 6-80: Analytical results – Berkeley site, prototype configuration A with 140-psf floor weight – floor acceleration magnification factor based on diaphragm shear
Figure 6-81: Analytical results – Berkeley site, prototype configuration B with 110-psf floor weight – floor acceleration magnification factor based on diaphragm shear.
6.5.6.3 Strength reduction factor

6.5.6.3.1 System demands

6.5.6.3.1.1 Forces

![Figure 6-82: Analytical results – Charleston site, prototype configuration A with R=6 – system moment demand](image)
Figure 6-83: Analytical results – Charleston site, prototype configuration B with R=6 – system moment demand
Figure 6-84: Analytical results – Charleston site, prototype configuration A with $R=6$ – system shear demand
Figure 6-85: Analytical results – Charleston site, prototype configuration B with R=6 – system shear demand
6.5.6.3.1.2 Deformations

Figure 6-86: Analytical results – Charleston site, prototype configuration A with R=6 – interstory drift ratio at the wall
Figure 6-87: Analytical results – Charleston site, prototype configuration B with R=6 – interstory drift ratio at the wall
Figure 6-88: Analytical results – Charleston site, prototype configuration A with R=6 – interstory drift ratio at the diaphragm midspan
Figure 6-89: Analytical results – Charleston site, prototype configuration B with R=6 – interstory drift ratio at the diaphragm midspan
Figure 6-90: Analytical results – Charleston site, prototype configuration A with R=6 – wall curvature ductility demand
Figure 6-91: Analytical results – Charleston site, prototype configuration B with R=6 – wall curvature ductility demand
6.5.6.3.2 Diaphragm demands

![Diagram showing diaphragm demands for different AR values](image)

**Charleston site**
- **R** = 6
- **Floor span = 200-ft**

**Figure 6-92**: Analytical results – Charleston site, prototype configuration A with R=6 – floor acceleration magnification factor based on diaphragm moment
Figure 6-93: Analytical results – Charleston site, prototype configuration B with R=6 – floor acceleration magnification factor based on diaphragm moment.
Figure 6-94: Analytical results – Charleston site, prototype configuration A with R=6 – floor acceleration magnification factor based on diaphragm shear
Figure 6-95: Analytical results – Charleston site, prototype configuration B with R=6 – floor acceleration magnification factor based on diaphragm shear

Charleston site
R = 6
Floor span = 300-ft
CHAPTER 7  EMPIRICALLY BASED REINFORCED CONCRETE MODEL

7.1 Introduction

Accurate predictions of nonlinear structural demands depend on the hysteretic modeling of the structure’s components. To this end, a general reinforced concrete model was developed. The empirically based model captures distinct features of flexural behavior. It was formulated as a piecewise linear model for use as a hysteresis in the structural analysis program Ruaumoko [71], [73]. The model was adopted into the Ruaumoko program and given the name “Schoettler-Restrepo Reinforced Concrete Column Hysteresis.” It incorporated key macro behavior such as cracking, yielding, pinching, and stiffness degradation based on peak deformation. A penta-linear backbone curve defines the force deformation envelope in either direction, see Figure 7-1.

Numerous reinforced concrete hysteretic models have been developed over the last 40 years. Of these, many modified or simplified the model proposed by Takeda [74]. Increasingly more complex models have been developed to account for pinching, stiffness degradation, strength degradation, or all three [75]. Advanced modeling capabilities available in Ruaumoko led to this program’s use in the analytical studies of CHAPTER 5 and CHAPTER 6. The need for a new hysteretic model was based on the preference for a tri-linear model with pinching and the detailed small cycle behavior. Considerable interest was given to the small cycle behavior resulting in a tri-level
nested loop to ensure the desired small cycle path is followed. Thirteen input parameters were necessary to facilitate user preferences and provide considerable versatility. Model validation was conducted to qualitatively confirm the intended behavior with experimental results.

![Figure 7-1: Hysteresis force-deformation response – Backbone curve](image)

7.2 Model formulation

This model is of the form \( F = f(\Delta) \) where \( F \) is the force at a given deformation, \( \Delta \). Force and deformation are used in general terms to refer to the vertical and horizontal axes of the hysteresis. The model formulation was independent of the units of the input parameters. The force-deformation relationship could be defined as force-
displacement, moment-rotation, or moment-curvature depending on the inputs supplied by the user.

Developed in the Matlab programming language, the code was transformed into the Fortran programming language for adoption into Ruaumoko. The history dependent model relies on critical points in the previous path to assign future behavior. Sufficient points of the prior path must be retained to adequately assign the future path. Twenty force-deformation points in each direction are required to define the cyclic behavior. This requirement makes the hysteresis more memory intensive to implement than other hysteresis models, but current computing capabilities are sufficient for this demand. The benefit to the user is a versatile hysteresis rule with application beyond its intended purpose.

Coding of the model utilized a piecewise linear model formulation. The model is comprised of independent linear segments connected at their ends. Increasing model complexity requires more segment definitions. The path is defined by the relationship between connected segments. Relationships developed for this hysteresis are defined in Table 7-1. Segments were numbered for reference purposes and identification in Table 7-1 and Figures 7-2 through 7-9.

During a call from the structural analysis program to the hysteresis subroutine with an element’s deformation, the element’s force and tangential stiffness must be computed. After checking to confirm that the deformation is within the allowed boundaries of the segment, the force is computed from the segment’s tangential stiffness and the force-deformation coordinates at the end of the segment. If the element’s deformation is outside the segment’s boundaries, then the segment whose
boundaries bound the deformation must be determined. This coincides with updated tangential stiffness and end coordinates. To determine which segment becomes current, it is determined whether the upper deformation boundary or the lower limit was exceeded. The lower limit could be either a lower boundary of the segment or the deformation at the previous time step as is the case when unloading from the post-yield backbone curve.

Segmentation simplified the code by allowing positive and negative deformations to utilize the same segment definitions. An integer variable keeps track of whether the segment is positive or negative and a switch of the integer is triggered when stepping between certain segments. Instances when this integer switch is triggered are identified with an asterisk in Table 7-1. For clarification, the blue segments in Figures 7-1 through 7-8 are positive while the negative segments are shown in red. Coordinates retained to define the path are shown in shaded gray for points corresponding to positive deformation and shown in light red for negative deformation.

7.2.1 Monotonic behavior

A penta-linear backbone curve envelopes the possible force-deformation relationship. It prescribes the monotonic loading (1) before cracking, (2) to yield, (3) post-yield, (4) during post-peak strength degradation, and (5) at a residual force capacity, see Figure 7-1. Input parameters \( K_1, F_{y,1}, F_{y,2}, r_1, K_{neg}, R_{neg}, F_{CR,+}, F_{CR,-}, \rho_1, \rho_2, \mu_{ult,1}, \mu_{ult,2}, \kappa, \) and \( F_{resid} \) define the backbone curve. Input parameters \( \alpha, \beta, \) and Pinch define the cyclic response. Initial stiffness, \( K_1, \) is the positive uncracked stiffness. Uncracked negative stiffness is defined as \( K_2 = K_{neg}K_1, \) where \( K_{neg} \) is the input
parameter defining the negative to positive stiffness ratio. The cracking force is determined from the input parameter $F_{CR,+}$ or $F_{CR,-}$, and the corresponding yield force $F_{y,1}$ or $F_{y,2}$, as $F_{cr,i} = F_{CR,i} F_{y,i}$ or $F_{cr,2} = F_{CR,2} F_{y,2}$. Secant stiffness to yield is determined from the initial stiffness and the input parameter $\rho_1$ or $\rho_2$: $K_{sec,1} = \rho_1 K_1$ or $K_{sec,2} = \rho_2 K_2$.

The yield force, $F_{y,1}$ or $F_{y,2}$, and secant stiffness to yield are used to define the yield deformation. Beyond the yield deformation, the post-yield tangential stiffness is determined from the stiffness factor, $r_i$, and initial stiffness for a positive deformation or $R_{neg}$ and $K_2$ for a negative deformation. The ultimate deformation is found from the input ductility factor, $\mu_{ult,1}$ or $\mu_{ult,2}$, and the yield deformation $\Delta_{ult,1} = \mu_{ult,1} \Delta_{y,1}$ or $\Delta_{ult,2} = \mu_{ult,2} \Delta_{y,2}$. Beyond this ultimate deformation, strength degradation occurs until a residual force is obtained. The slope of the strength degradation is determined by the stiffness factor $\kappa$. The residual force is found by the residual force factor, $F_{resid}$, and the cracking force: $F_{r,1} = F_{resid} F_{cr,1}$ or $F_{r,2} = F_{resid} F_{cr,2}$.

### 7.2.2 Cyclic behavior

The input parameter $\alpha$ dictates the unloading slope according to the Emori [76] unloading rule. The unloading stiffness is a factor of the initial stiffness, yield deformation, plastic deformation, and unloading parameter $\alpha$: $K_u = \rho_i K_i \left( \frac{\Delta_{y,i}}{\Delta_{m,i}} \right)^\alpha$,

where $i$ denotes the positive or negative side. This parameter has an upper bound to ensure that the unloaded deformation at zero force crosses the horizontal axis at or before crossing the vertical axis:
Reloading stiffness factor, $\beta$, defines the deformation at which the hysteresis rejoins the backbone curve. This input parameter allows overshooting of the peak deformation upon reloading which is computed as $\Delta = \beta \Delta_m$. An input value of one causes the hysteresis to target the peak deformation. Pinching of the hysteresis is captured by the input parameter $Pinch$. A value less than 1.0 will cause a load reversal to target a pinching force at $F = F_m, Pinch$. A value of 1.0 eliminates pinching behavior. The targeted deformation when parameter $Pinch$ is less than 1.0 is one half of the plastic deformation on the unloading side, see Figure 7-7.

Cyclic behavior can be categorized into 6 stages. These depend on the maximum deformation attained in either direction. The most basic of these is uncracked-uncracked behavior, see Figure 7-2. When loading and unloading, the hysteresis follows the elastic portion of the backbone curve to cracking, segment 100.

A cracked-elastic condition loads along the backbone curve of segment 200 toward the yield point, see Figure 7-3. Unloading and reloading follow segment 300. This unloads through the origin and reloads on the same path targeting the peak force-deformation.
Figure 7-2: Hysteresis force-deformation response – Elastic-elastic

Figure 7-3: Hysteresis force-deformation response – Cracked-elastic
A cracked-cracked condition loads and unloads along segment 300 through the origin until the peak deformation is surpassed, see Figure 7-4. After exceeding the previous peak deformation, the backbone curve of segment 200 is rejoined.

![Figure 7-4: Hysteresis force-deformation response – Cracked-cracked](image)

The yielded-elastic stage is detailed in Figure 7-5. The post-yield backbone curve is defined with segment 400. After yielding, unloading from the backbone curve always unloads through segment 500, which unloads and reloads with stiffness $K_u$. Reloading in this stage follows segment 600 to the point on the backbone curve midway between the cracking and the yield forces. Small cycle behavior in this stage is provided in the detail insert of Figure 7-5.

Behavior of the cracked-yielded stage is shown in Figure 7-6. After the cracked side unloads through the origin with segment 300, segment 700 targets the peak deformation. Small cycle behavior is prescribed with segments 700 through 713.
Figure 7-5: Hysteresis force-deformation response – Yielded-elastic

Figure 7-6: Hysteresis force-deformation response – Yielded-cracked
The yielded-yielded stage shown in Figure 7-7 includes the optional pinching behavior. Pinching behavior is further outlined in Figure 7-8 and detailed with segments 900 through 913. Before pinching, the small cycle response is detailed in segments 800 through 813 shown in Figure 7-7. When a load reversal occurs from segments 800 or 900, the pinching point will be targeted. However, if the pinching point is outside the straight line path to the maximum deformation, then it is not targeted as shown with the red pinching point number 9 in Figure 7-7.

Figure 7-7: Hysteresis force-deformation response – Yielded-yielded
Figure 7-8: Hysteresis force-deformation response – Yielded-yielded with pinching

The pinching deformation is defined as $\frac{1}{2}$ of the plastic deformation on the side being reloaded from while the pinching force is computed as the pinching factor, $Pinch$, multiplied by the peak force on the side being reloaded towards. Pinching behavior is complicated by three factors: large pinching factors, large plastic deformations on the side being unloaded from, and strength degradation on the side being reloaded towards.

When the pinching factor is large, the pinching coordinate may fall outside the region bounded by a straight line from the unloaded deformation to the peak force-deformation point on the opposite side, see regions A of Figure 7-9. If the pinching coordinate falls into this category, pinching behavior is nullified and a straight line to the peak force-deformation point on the opposite side is followed. Regions in this figure marked with a B indicate that the pinching coordinates are satisfactory.
When the plastic deformation on the side being unloaded from is large, there is a possibility that the pinching coordinate will lie outside the peak deformation on the side being reloaded towards. In this case, the targeted force and deformation point is not the peak deformation but is found by extrapolating to the backbone curve the point where it intersects with the pinching behavior. Instances of this in Figure 7-9 are indicated by shaded regions marked with a C.

Deformation regions indicated as 2 and 3 in Figure 7-9 required special attention when verifying the pinching coordinates within the hysteresis. These are regions where pinching behavior interferes with the strength degradation, region 2, or residual force, region 3. Additional checks were coded into the pinching behavior to verify and modify or exclude its response when the pinching ordinate is outside of acceptable regions.

Figure 7-9: Hysteresis force-deformation response – possible pinching point locations
Considerable attention was given to the small cycle behavior. Paths were prescribed for three inscribed loops of unloading, reloading, and load reversals. These loops are referred to as primary, secondary, and tertiary. A primary unloading branch is defined as unloading from the cracked backbone curve, segments 200, 400, 1000, or 2000, through the horizontal axis, see Figures 7-5, 7-6, and 7-7, respectively. If unloading from segment 200, the primary unloading branch will be segment 300. Otherwise unloading to the horizontal axis follows segment 500. Primary reloading is the path followed from the unloaded state toward the peak deformation on the opposite side. This involves segments 600, 700, or 800, see Figures 7-5, 7-6, and 7-7, respectively. A load reversal occurs when the deformation reverses direction from a reloading branch. This triggers a reversal point to be stored in memory. The load reversal encompasses two segments, one that unloads to the horizontal axis following the unloading stiffness of the opposite side and another that targets the peak deformation. Primary load reversals are identified as segments 601 and 602 in Figure 7-5, 701 and 702 in Figure 7-6, and 801 and 802 in Figure 7-7. The primary reversal points are numbered 1, 5, and 10 in these figures.

A secondary unloading branch is created if the deformation changes direction while on the second reloading branch. This creates a secondary unloading point identified as number 2, 6, and 11 in Figure 7-5 through Figure 7-7, respectively. Secondary unloading stiffness follows the primary unloading stiffness, \( K_u \). A secondary reloading branch targets the corresponding primary reversal point. If the deformation exceeds the reversal point, then the loop is closed, the memory points are cleared, and the primary reversal branch becomes current. If, however, the deformation changes
direction, then a secondary reversal point, number 3, 7, or 12, is created. A secondary reloading branch is contained within the primary reloading branch. Similar to the primary reloading branch, it has two segments, but the second segment targets the secondary unloading point not the peak deformation. If the deformation exceeds the secondary unloading point, then the second segment of the primary reloading branch is rejoined. Otherwise, a deformation reversal creates the tertiary unloading point, number 4, 8, or 13, and a tertiary unloading branch is followed.

The tertiary unloading branch also follows unloading stiffness, $K_u$. It is followed by a tertiary reloading branch that targets the secondary reversal point. If the secondary reversal point is surpassed, then the secondary reloading branch is rejoined. A deformation reversal from the tertiary reloading branch does not create a reversal point, but a tertiary load reversal branch is followed. The tertiary load reversal branch has two segments and targets the tertiary unloading point. No additional unloading points are created by a deformation reversal from the tertiary reloading branch. This is the finest level of detail formulated in the hysteresis. Cycling will continue inside the tertiary level until the time history is completed or the loop is broken by a deformation exceeding the secondary reversal point or tertiary unloading point.

Table 7-1: Hysteresis segment definitions

<table>
<thead>
<tr>
<th>No.</th>
<th>Segment description</th>
<th>Next state</th>
<th>Unloading state</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>Elastic Backbone</td>
<td>200</td>
<td>100&quot;</td>
</tr>
<tr>
<td>200</td>
<td>Cracked Backbone</td>
<td>400</td>
<td>300</td>
</tr>
<tr>
<td>300</td>
<td>Origin unloading</td>
<td>200</td>
<td>100&quot;, 300&quot;, or 700&quot;</td>
</tr>
<tr>
<td>400</td>
<td>Yielded Backbone</td>
<td>1000</td>
<td>500</td>
</tr>
<tr>
<td>500</td>
<td>Emori unloading</td>
<td>400</td>
<td>600 or 800</td>
</tr>
</tbody>
</table>
### Uncracked – Yielded States (Figure 7-5)

<table>
<thead>
<tr>
<th>Step</th>
<th>Action Description</th>
<th>Force</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>600</td>
<td>Reverse to midpoint between cracking force and yield force</td>
<td>200°</td>
<td>601 &amp; update point 1</td>
</tr>
<tr>
<td>601</td>
<td>Primary Cycle: Unloading (opposite minimum stiffness)</td>
<td>600</td>
<td>602</td>
</tr>
<tr>
<td>602</td>
<td>Primary Cycle: Peak oriented loading</td>
<td>400</td>
<td>603 &amp; update point 2</td>
</tr>
<tr>
<td>603</td>
<td>Primary Cycle: Unloading (current minimum stiffness)</td>
<td>602</td>
<td>604</td>
</tr>
<tr>
<td>604</td>
<td>Primary Cycle: Loading to close the primary cycle (point 1)</td>
<td>600</td>
<td>605 &amp; update point 3</td>
</tr>
<tr>
<td>605</td>
<td>Secondary Cycle: Unloading (opposite minimum stiffness)</td>
<td>604</td>
<td>606</td>
</tr>
<tr>
<td>606</td>
<td>Secondary Cycle: Loading to point 2</td>
<td>602</td>
<td>607 &amp; update point 4</td>
</tr>
<tr>
<td>607</td>
<td>Secondary Cycle: Unloading (current minimum stiffness)</td>
<td>606</td>
<td>608</td>
</tr>
<tr>
<td>608</td>
<td>Tertiary Cycle: Loading to point 3</td>
<td>604</td>
<td>610</td>
</tr>
<tr>
<td>609</td>
<td>Tertiary Cycle: Loading to point 3</td>
<td>604</td>
<td>610</td>
</tr>
<tr>
<td>610</td>
<td>Tertiary Cycle: Unloading (opposite minimum stiffness)</td>
<td>609</td>
<td>611</td>
</tr>
<tr>
<td>611</td>
<td>Tertiary Cycle: Loading to close I.C. (point 4)</td>
<td>606</td>
<td>613</td>
</tr>
<tr>
<td>612</td>
<td>Tertiary Cycle: Loading to close I.C. (point 4)</td>
<td>606</td>
<td>613</td>
</tr>
<tr>
<td>613</td>
<td>Tertiary Cycle: Unloading (current minimum stiffness)</td>
<td>612</td>
<td>608</td>
</tr>
</tbody>
</table>

### Cracked – Yielded States (Figure 7-6)

<table>
<thead>
<tr>
<th>Step</th>
<th>Action Description</th>
<th>Force</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>700</td>
<td>Primary Cycle: Peak-Center oriented</td>
<td>400</td>
<td>701 &amp; update point 5</td>
</tr>
<tr>
<td>701</td>
<td>Primary Cycle: Unloading from peak-center (current minimum stiffness)</td>
<td>700</td>
<td>702</td>
</tr>
<tr>
<td>702</td>
<td>Primary Cycle: Reverse to opposite peak</td>
<td>200°</td>
<td>703 &amp; update point 6</td>
</tr>
<tr>
<td>703</td>
<td>Primary Cycle: Unloading (opposite minimum stiffness)</td>
<td>702</td>
<td>704</td>
</tr>
<tr>
<td>704</td>
<td>Secondary Cycle: Loading to point 5</td>
<td>700</td>
<td>705 &amp; update point 7</td>
</tr>
<tr>
<td>705</td>
<td>Secondary Cycle: Unloading (current minimum stiffness)</td>
<td>704</td>
<td>706</td>
</tr>
<tr>
<td>706</td>
<td>Secondary Cycle: Loading to close S.C. (point 6)</td>
<td>702</td>
<td>707 &amp; update point 8</td>
</tr>
</tbody>
</table>
Table 7-1 continued

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
<th>Step 1</th>
<th>Step 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>707</td>
<td>Tertiary Cycle: Unloading (opposite minimum stiffness)</td>
<td>706</td>
<td>708</td>
</tr>
<tr>
<td>708</td>
<td>Tertiary Cycle: Loading to point 7</td>
<td>704</td>
<td>710</td>
</tr>
<tr>
<td>709</td>
<td>Tertiary Cycle: Loading to point 7</td>
<td>704</td>
<td>710</td>
</tr>
<tr>
<td>710</td>
<td>Tertiary Cycle: Unloading (current minimum stiffness)</td>
<td>709</td>
<td>711</td>
</tr>
<tr>
<td>711</td>
<td>Tertiary Cycle: Loading to close S.C. (point 8)</td>
<td>706</td>
<td>713</td>
</tr>
<tr>
<td>712</td>
<td>Tertiary Cycle: Loading to close S.C. (point 8)</td>
<td>706</td>
<td>713</td>
</tr>
<tr>
<td>713</td>
<td>Tertiary Cycle: Unloading (opposite minimum stiffness)</td>
<td>712</td>
<td>708</td>
</tr>
</tbody>
</table>

**Yielded – Yielded States (Figure 7-7)**

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
<th>Step 1</th>
<th>Step 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>800</td>
<td>Reload to opposite</td>
<td>900</td>
<td>801 &amp; update point 10</td>
</tr>
<tr>
<td>801</td>
<td>Primary Cycle: Unloading (opposite minimum stiffness)</td>
<td>800</td>
<td>802</td>
</tr>
<tr>
<td>802</td>
<td>Primary Cycle: Loading to current peak</td>
<td>400</td>
<td>803 &amp; update point 11</td>
</tr>
<tr>
<td>803</td>
<td>Primary Cycle: Unloading (current minimum stiffness)</td>
<td>802</td>
<td>804</td>
</tr>
<tr>
<td>804</td>
<td>Primary Cycle: Loading to close S.C. (point 10)</td>
<td>800</td>
<td>805 &amp; update point 12</td>
</tr>
<tr>
<td>805</td>
<td>Secondary Cycle: Unloading (opposite minimum stiffness)</td>
<td>804</td>
<td>806</td>
</tr>
<tr>
<td>806</td>
<td>Secondary Cycle: Loading to point 11</td>
<td>802</td>
<td>807 &amp; update point 13</td>
</tr>
<tr>
<td>807</td>
<td>Secondary Cycle: Unloading (current minimum stiffness)</td>
<td>806</td>
<td>808</td>
</tr>
<tr>
<td>808</td>
<td>Tertiary Cycle: Loading to close E.C. (point 12)</td>
<td>804</td>
<td>810</td>
</tr>
<tr>
<td>809</td>
<td>Tertiary Cycle: Loading to close E.C. (point 12)</td>
<td>804</td>
<td>810</td>
</tr>
<tr>
<td>810</td>
<td>Tertiary Cycle: Unloading (opposite minimum stiffness)</td>
<td>809</td>
<td>811</td>
</tr>
<tr>
<td>811</td>
<td>Tertiary Cycle: Loading to close I.C. (point 13)</td>
<td>806</td>
<td>813</td>
</tr>
<tr>
<td>812</td>
<td>Tertiary Cycle: Loading to close I.C. (point 13)</td>
<td>806</td>
<td>813</td>
</tr>
<tr>
<td>813</td>
<td>Tertiary Cycle: Unloading (current minimum stiffness)</td>
<td>812</td>
<td>808</td>
</tr>
</tbody>
</table>

**Yielded – Yielded States (Figure 7-8): Pinching included, Pinch<1.0**

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
<th>Step 1</th>
<th>Step 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>900</td>
<td>Pinching point to peak</td>
<td>400</td>
<td>901 &amp; update point 14</td>
</tr>
<tr>
<td>901</td>
<td>Primary Cycle: Unloading (opposite minimum stiffness)</td>
<td>900</td>
<td>902</td>
</tr>
</tbody>
</table>
Table 7-1 continued

<table>
<thead>
<tr>
<th>902</th>
<th>Primary Cycle: Loading to opposite pinching point (point 9)</th>
<th>900*</th>
<th>903 &amp; update point 15</th>
</tr>
</thead>
<tbody>
<tr>
<td>903</td>
<td>Primary Cycle: Unloading (current minimum stiffness)</td>
<td>902</td>
<td>904</td>
</tr>
<tr>
<td>904</td>
<td>Primary Cycle: Loading to close S.C. (point 14)</td>
<td>900</td>
<td>905 &amp; update point 16</td>
</tr>
<tr>
<td>905</td>
<td>Secondary Cycle: Unloading (opposite minimum stiffness)</td>
<td>904</td>
<td>906</td>
</tr>
<tr>
<td>906</td>
<td>Secondary Cycle: Loading to point 15</td>
<td>902</td>
<td>907 &amp; update point 17</td>
</tr>
<tr>
<td>907</td>
<td>Secondary Cycle: Unloading (current minimum stiffness)</td>
<td>906</td>
<td>908</td>
</tr>
<tr>
<td>908</td>
<td>Tertiary Cycle: Loading to close E.C. (point 16)</td>
<td>904</td>
<td>910</td>
</tr>
<tr>
<td>909</td>
<td>Tertiary Cycle: Loading to close E.C. (point 16)</td>
<td>904</td>
<td>910</td>
</tr>
<tr>
<td>910</td>
<td>Tertiary Cycle: Unloading (opposite minimum stiffness)</td>
<td>909</td>
<td>911</td>
</tr>
<tr>
<td>911</td>
<td>Tertiary Cycle: Loading to close T.C. (point 17)</td>
<td>906</td>
<td>913</td>
</tr>
<tr>
<td>912</td>
<td>Tertiary Cycle: Loading to close T.C. (point 17)</td>
<td>906</td>
<td>913</td>
</tr>
<tr>
<td>913</td>
<td>Tertiary Cycle: Unloading (current minimum stiffness)</td>
<td>912</td>
<td>908</td>
</tr>
</tbody>
</table>

**Strength Degradation States (Figure 7-9)**

<table>
<thead>
<tr>
<th>Strength Degradation States</th>
<th>Degrading Backbone</th>
<th>Residual Force Backbone</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000</td>
<td></td>
<td>2000</td>
</tr>
<tr>
<td>2000</td>
<td></td>
<td>500</td>
</tr>
</tbody>
</table>

* Switch between positive and negative states.

7.3 Validation with experimental tests

To ensure the model captures the behavior of reinforced concrete under complex loading histories, the model was validated with experimental tests. The validation was conducted with three data sets. Experimental results from Kawashima [77] provided insight on reinforced concrete bridge pier behavior under unique loading scenarios. Two experimental data sets of bridge pier tests were utilized for the comparison. A cyclic reinforced concrete wall test provided additional validation data.
This validation was performed to confirm the adequacy of the idealized hysteretic behavior. It was not intended as validation of the discussed input parameters’ validity for all applications. Specific applications of this model require calibration to the expected behavior.

Six identical square columns tested under unique loading protocols to investigate the effect of loading on hysteresis comprised one data set. Specimen details, test setup, and loading protocols can be found in the test report [78]. These tests had ID numbers TP-001 through TP-006 provided in the test report. From visual inspection of the six force displacement test results, an idealized backbone curve and parameters defining cyclic behavior were obtained. To model this behavior, the values in Table 7-2 were used to determine input values to the hysteresis model. Symmetric behavior was assumed, so the amplification factors for negative response were unity. Displacement history to the hysteresis was the measured experimental displacement.

Comparison of the experimental results and analytical estimation show sufficient modeling capabilities, see Figure 7-10. Test ID numbers TP-001 through TP-006 correspond with Figure 7-10 (a) through (g), respectively.

Table 7-2: Modeling parameters for validation with Kawashima [78] square column tests

<table>
<thead>
<tr>
<th>(\Delta_{cr}) [mm]</th>
<th>(F_{cr}) [kN]</th>
<th>(\Delta_y) [mm]</th>
<th>(F_y) [kN]</th>
<th>(r)</th>
<th>(\rho)</th>
<th>(\mu_{ult})</th>
<th>(\kappa)</th>
<th>(F_{resid})</th>
<th>(\alpha)</th>
<th>(\beta)</th>
<th>Pinch</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>60</td>
<td>14</td>
<td>125</td>
<td>1x10^{-4}</td>
<td>0.744</td>
<td>100</td>
<td>1x10^{-3}</td>
<td>0.5</td>
<td>0.2</td>
<td>0.19</td>
<td>0.80</td>
</tr>
</tbody>
</table>

The second data set was taken from eight tests on circular cantilevered columns. This experimental investigation focused on later confinement effects. Specimen details, test setup, and loading protocols can be found in the test report [79]. ID numbers TP-
054 through TP-061 referred to these tests. From visual inspection of the eight force displacement test results, an idealized backbone curve and parameters defining cyclic behavior were obtained. To model this behavior, values in Table 7-3 were used to determine inputs to the hysteresis model. Symmetric behavior was assumed, so the amplification factors for negative response were unity. The displacement history for the hysteresis was the measured experimental displacement. Comparison results are shown in Figure 7-11 with test ID numbers TP-054 through TP-061 corresponding to Figure 7-11 (a) through (g), respectively.

Table 7-3: Modeling parameters for validation with Kawashima [79] circular column tests

<table>
<thead>
<tr>
<th>$\Delta_{cr}$ [mm]</th>
<th>$F_{cr}$ [kN]</th>
<th>$\Delta_y$ [mm]</th>
<th>$F_y$ [kN]</th>
<th>$r$</th>
<th>$\rho$</th>
<th>$\mu_{ult}$</th>
<th>$\kappa$</th>
<th>$F_{resid}$</th>
<th>$\alpha$</th>
<th>$\beta$</th>
<th>Pinch</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>60</td>
<td>10</td>
<td>155</td>
<td>$1 \times 10^{-3}$</td>
<td>0.517</td>
<td>200</td>
<td>$1 \times 10^{-3}$</td>
<td>0.5</td>
<td>0.1</td>
<td>0.5</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Figure 7-10: Hysteresis validation – comparison with Kawashima [77] square column tests
Figure 7-11: Hysteresis validation – comparison with Kawashima [79] circular column tests
A precast concrete wall test was included in the validation. Test Unit 1 reported by Holden et al. [80] was relied upon to assess cyclic behavior. Details of the test unit, test setup, and material properties are available in the reference. The displacement controlled loading protocol included cycles at ±0.25% and ±0.5% drift followed by increasing drift amplitudes in steps of 0.5% to 3%. At each drift amplitude, two cycles were completed followed by a third cycle at the previous drift. The displacement history for the validation followed this loading sequence, but did not utilize the test’s measured displacements and was terminated at 2.5% drift. The longitudinal bar fracture and bar buckling reported after cycles to 2.5% drift are not features the hysteretic model was intended to capture, so no attempt was made to model behavior above this drift.

Reinforcing details and measured material properties were used as input for sectional analysis with the moment-curvature program discussed in CHAPTER 6. The resulting moment-curvature relationship was integrated along the wall’s 3.75-m height to obtain the force-displacement curve shown in green in Figure 7-12. An idealized moment-curvature relationship was obtained as input to Ruaumoko3D [73] using the Schoettler-Restrepo hysteresis. The input parameters obtained directly from the idealized moment-curvature relationship are the cracking and yield coordinates, the secant stiffness factor, and post-yield stiffness factor, see Table 7-4. The remaining parameters in Table 7-4 were defined by visual inspection of the test results.

<table>
<thead>
<tr>
<th>$\phi_{cr}$ [1/m]</th>
<th>$M_{cr}$ [kN-m]</th>
<th>$\phi_y$ [1/m]</th>
<th>$M_y$ [kN-m]</th>
<th>$r$</th>
<th>$\rho$</th>
<th>$\mu_{ult}$</th>
<th>$\kappa$</th>
<th>$F_{resid}$</th>
<th>$\alpha$</th>
<th>$\beta$</th>
<th>Pinch</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5x10^{-4}</td>
<td>160</td>
<td>3.4x10^{-3}</td>
<td>536</td>
<td>6x10^{-4}</td>
<td>0.149</td>
<td>21</td>
<td>1x10^{-3}</td>
<td>1.0</td>
<td>0.55</td>
<td>0.1</td>
<td>0.63</td>
</tr>
</tbody>
</table>
Figure 7-12: Hysteresis validation – comparison with Holden, et al. [80] test unit 1

7.3.1 Results

This validation was performed to confirm the model’s cyclic behavior sufficiently represents the nonlinear response of specific reinforced concrete tests. To this end, the validation showed the model sufficiently captures the response of reinforced concrete members subjected to complex loading histories. The validation was not intended as a justification that the input parameters used are appropriate for all uses of the model. Rather the input parameters must be justified through calibration for any application of the model to ensure the appropriate behavior is achieved.

The general behavior of the experimental results was captured. Discrepancies exist primarily in the load reversal portion of cyclic response. In quadrants II and IV of Figures 7-10 through 7-12, the response is underestimated. This reduced energy
dissipation is conservative so no attempt was made to more precisely capture observed behavior.

Figure 7-10 (a) through (d) contain unsatisfactory results due to cyclic strength degradation. From these results, a clear limitation of the hysteresis is defined. The application of this model to conditions that present this behavior should not be avoided. Figure 7-11 (c) also shows significant error. However, the experimental result appears to have suffered a premature failure, which is not within the capacity of the model formulation.

7.4 Summary

An empirical piecewise linear model was developed to capture reinforced concrete flexural behavior. Versatility was of primary importance in the model formulation. This facilitates user specific applications but requires a number of input parameters to be defined.

The hysteresis model was adopted into Ruaumoko2D and Ruaumoko 3D. The required inputs follow the same format in each version, see Table 7-5.

Table 7-5: Ruaumoko input format

<table>
<thead>
<tr>
<th>Line 1</th>
<th>Kneg</th>
<th>Rneg</th>
<th>Fcr+</th>
<th>Fcr-</th>
<th>Rho+</th>
<th>Rho-</th>
<th>Dult+</th>
<th>Dult-</th>
<th>iop</th>
</tr>
</thead>
<tbody>
<tr>
<td>Line 2</td>
<td>Alpha</td>
<td>Beta</td>
<td>Pinch</td>
<td>Kappa</td>
<td>Fresid</td>
<td>Dfactor</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Lines 1 and 2 are repeated for each component of deformation in the element utilizing this hysteresis. The parameters iop and Dfactor or reserved for later modifications and should be set as zero. Bounds on the remaining input values are provided in Table 7-6.
Error checks are performed within the hysteresis to confirm that specified values are within the acceptable ranges.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Limiting value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kneg</td>
<td>Kneg &gt; 0.0</td>
</tr>
<tr>
<td>Rneg</td>
<td>Rneg &gt; 0.0</td>
</tr>
<tr>
<td>Fcr+/-</td>
<td>Fcr+/- &lt; 1.0</td>
</tr>
<tr>
<td>Rho+/-</td>
<td>Rho+/- &lt; 1.0</td>
</tr>
<tr>
<td>Dult+/-</td>
<td>Dult+/- &gt; 1.0</td>
</tr>
<tr>
<td>Alpha</td>
<td>$0.0 &lt; \text{Alpha} &lt; \ln\left(\frac{F_{y,i} + r_i K_{ij} \left(\mu_{\lambda,alt,i} - \frac{F_{y,i}}{\rho K_{ij}}\right)}{\rho_i K_{ij} \mu_{\lambda,alt,i}}\right)$ and $\ln\left(\frac{F_{y,i}}{\rho K_{ij} \mu_{\lambda,alt,i}}\right)$</td>
</tr>
<tr>
<td>Beta</td>
<td>$0.0 &lt; \text{Beta} &lt; 5.0$</td>
</tr>
<tr>
<td>Pinch</td>
<td>$0.0 &lt; \text{Pinch} &lt; 1.0$</td>
</tr>
<tr>
<td>Kappa</td>
<td>$0.0 &lt; \text{Kappa} &lt; 1.0$</td>
</tr>
<tr>
<td>Fresid</td>
<td>$0.0 &lt; \text{Fresid} &lt; 1/(Fcr+/-)$</td>
</tr>
</tbody>
</table>

As an example, the required hysteresis inputs for the validation with Holden’s test Unit 1 in Ruaumoko3D is provided in Table 7-7:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Limiting value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kneg</td>
<td>Kneg &gt; 0.0</td>
</tr>
<tr>
<td>Rneg</td>
<td>Rneg &gt; 0.0</td>
</tr>
<tr>
<td>Fcr+/-</td>
<td>Fcr+/- &lt; 1.0</td>
</tr>
<tr>
<td>Rho+/-</td>
<td>Rho+/- &lt; 1.0</td>
</tr>
<tr>
<td>Dult+/-</td>
<td>Dult+/- &gt; 1.0</td>
</tr>
<tr>
<td>Alpha</td>
<td>$0.0 &lt; \text{Alpha} &lt; \ln\left(\frac{F_{y,i} + r_i K_{ij} \left(\mu_{\lambda,alt,i} - \frac{F_{y,i}}{\rho K_{ij}}\right)}{\rho_i K_{ij} \mu_{\lambda,alt,i}}\right)$ and $\ln\left(\frac{F_{y,i}}{\rho K_{ij} \mu_{\lambda,alt,i}}\right)$</td>
</tr>
<tr>
<td>Beta</td>
<td>$0.0 &lt; \text{Beta} &lt; 5.0$</td>
</tr>
<tr>
<td>Pinch</td>
<td>$0.0 &lt; \text{Pinch} &lt; 1.0$</td>
</tr>
<tr>
<td>Kappa</td>
<td>$0.0 &lt; \text{Kappa} &lt; 1.0$</td>
</tr>
<tr>
<td>Fresid</td>
<td>$0.0 &lt; \text{Fresid} &lt; 1/(Fcr+/-)$</td>
</tr>
</tbody>
</table>
CHAPTER 8 SUMMARY, CONCLUSIONS, AND FUTURE RESEARCH NEEDS

8.1 Summary

The vulnerability of shear wall buildings to floor accelerations larger than anticipated by commonly used code provisions makes nonlinear diaphragm behavior likely. Comparison of the elastic diaphragm demands estimated analytically with the strength provided by the ELF procedure suggests that the nominal capacity will be exceeded. However, ductile performance is not guaranteed unless detailed accordingly.

Seismic detailing in buildings with precast concrete diaphragms is complicated by the jointed nature of this construction method that requires external connectors between elements. Performance of these connectors is a crucial part of system behavior and should be considered in seismic detailing. With quantified connector performance criteria and sufficient seismic detailing in the remaining diaphragm components, a nonlinear design is feasible provided that it has no detrimental effects on overall system performance.

For a nonlinear design, the extent of seismic detailing and connector toughness are related to selected diaphragm ductility. This in turn can be related to the reduced elastic force demand. With this nonlinear design procedure, elastic diaphragm demands are necessary. The dependency of either a nonlinear or elastic design on elastic design forces made the estimation of elastic seismic demands the priority of this research.
A concern for long span diaphragms is amplification caused by diaphragm flexibility. This is not to say that a floor must be codified as a flexible diaphragm to be susceptible to the amplification. Rather, the amplification occurs because of inherent diaphragm stiffness that is not assessed in the structural dynamics when the diaphragm is assumed rigid.

Therefore, an investigation on the seismic response of precast concrete buildings related to the diaphragm response was conducted. Elastic diaphragm response at the design basis earthquake was its focus. The result of this research is an analysis method for estimating diaphragm forces necessary for its elastic design. This was achieved through formulation of the design forces through the modal response spectrum procedure, comparison of this procedure with analytical estimations of diaphragm demands, and an experimental validation test providing justification of the analytical models.

To estimate the seismic demands generated in the floors of building structures, a large scope analytical study was conducted on simplified numerical models. The objective was to assess global structural demands. A simplified formulation of the modal, first mode reduced, response spectrum method was the basis for estimating elastic diaphragm forces because forces estimated with this method by others showed reliable results for rigid floor diaphragms. Based on the scope of the analytical study, the demands estimated with this method relate specifically to buildings with flexurally dominated shear wall systems and reinforced concrete floor diaphragms.

Two effective flexural stiffness values were incorporated into the scope of the analytical study. These were included to identify response differences when distributed
cracking is expected and when the tension stiffening effect observed in the experimental research program is included. The tension stiffening effect was experimentally observed by the discrete crack pattern formed at the joints between precast floor units. Cracking was confined to these regions in loading scenarios when diaphragm behavior was elastic. This behavior formed the basis of the effective stiffness estimation. A lower bound stiffness estimate was developed for a uniform crack pattern in the diaphragm, which did not materialize at elastic diaphragm behavior. Both of these were formulated in section 6.2.1.2.

Analytical model validation was conducted for three design basis earthquake scenarios. The Knoxville DBE test provided the elastic diaphragm behavior upon which the stiffness formulation was based. Seattle and Berkeley DBE tests provided valuable information on the nonlinear response of precast concrete diaphragms. The validation conducted with these tests provided a comparison of nonlinear diaphragm response and the elastic secant stiffness modeled. The elastic secant stiffness was based on a linear fit of the experimental results.

System behavior between the experimental test and prototype structures of the analytical study was significantly different. Hybrid rocking walls used in the shake table test had a nonlinear but self-centering response. The prototype structures modeled in the analytical study included traditional reinforced concrete walls with nonlinear deformation concentrated at a plastic hinge within the first story height. For accurate validation of the analytical models, the hybrid rocking wall response was modeled in the validation. This aspect was critical for the validation, and resulted in exceptional agreement between the nonlinear experimental result and the simplified numerical
model. The hybrid rocking wall model consisted of standard frame elements representing the initially uncracked wall properties and contact springs distributed along the wall length to capture the concentrated deformation at the interface between the wall and foundation. Mild steel reinforcement and post-tensioning tendons were each modeled with spring elements to capture their contribution to the hybrid behavior.

8.2 Conclusions

8.2.1 Elastic diaphragm design forces

Flexurally dominated shear wall buildings should not be designed for elastic behavior with the equivalent lateral force procedure’s diaphragm design forces. This procedure does not account for structural amplification of the ground acceleration.

For elastic design of diaphragms with or without their flexibility considered in the building’s response, a modal response spectrum analysis is recommended. This analysis should not include elastic strength reduction in higher modes because this could reduce forces associated with diaphragm modes of vibration that are to be designed elastically.

The simplified FMR method proposed for shear wall buildings showed sufficient conservatism in the estimation of demands as compared to the results of the analytical study. The scope of the study included rigid diaphragm analyses and analyses with diaphragm stiffness modeled. The proposed method was found to be appropriate for both of these diaphragm conditions.

A case of un-conservative estimation with the simplified FMR method resulted at the Charleston site. Soft soil conditions were the likely variable producing
inconsistent results with the other sites. However, sufficient capacity is likely present in the system at nominal capacity and post-yield ductility demands are very small relative to the ductility demand expected in from an ELF procedure.

8.2.2 Diaphragm demands in precast concrete floors evaluated through analytical investigation

8.2.2.1 Acceleration magnification factors

The acceleration magnification factors utilized for comparison do not take into account the horizontal floor force distribution. Acceleration amplification along the floor span due to diaphragm flexibility was not considered. Rather, a uniformly distributed lateral load along the floor span was considered. The uniform load that generates the peak diaphragm moment or shear from time history analyses was used. Complications due to this simplification were not apparent in the results.

8.2.2.2 Effective stiffness models for precast diaphragms

Diaphragm flexibility produced significant amplification over the rigid analysis at the roof and floors between 25% and 40% of the roof height for structures with 5 and 8 floors. The largest amplification was found at the Charleston site, while the Knoxville site had consistent attenuation for the lower bound diaphragm stiffness estimate. Demands for the 3 story structures showed both amplification and attenuation due to diaphragm flexibility, but typically there was attenuation at the 30% height where the structures with more floors had amplification. The amplification and attenuation discussed are in relation to the rigid diaphragm analysis.
8.2.2.3 Lateral drift demands

Interstory drift ratios all fell below 3%. Eighty-one percent of all analyses conducted had interstory drift ratios lower than 2%. This is in contrast with previous research findings. A possible source of discrepancy is the presence of gravity columns in the analytical models presented here. During model development, the exclusion of gravity columns caused floors to response out of phase. This phenomenon is capable of producing significant interstory drift ratios, which was motivation for developing models that included gravity columns.

8.2.2.4 Flexible diaphragm condition

The code definition for a flexible diaphragm was not successful at predicting the influence of diaphragm flexibility. It captured the lower floors’ enhanced demands, but missed amplification near the roof. The code definition is not an accurate indicator of when flexibility causes increased demands.

8.2.2.5 Hybrid rocking wall model viability

An analytical model was developed for the response of hybrid rocking walls. The model successfully captured nonlinear demands of three experimental tests. This provided an essential link in the validation process of the main analytical research. The model’s relative simplicity and ability to capture critical aspects of the wall response make it a viable option for future research in this field.
8.2.3 Experimental evidence on the response of precast concrete diaphragms

8.2.3.1 Evaluation of initial stiffness

Three of the sixteen experimental shake table tests provide information on elastic diaphragm behavior. Results from the Knoxville DBE – trials 1 through 3 are the basis for diaphragm stiffness recommendations. The yield strain in each diaphragm was less than 67% of yield. Tests at larger amplitudes showed evidence of localized yielding or significant damage upon which elastic diaphragm behavior cannot be based.

Elastic diaphragm behavior at this level of testing provided flexural stiffness values for the basis of the analytical investigation. These effective stiffness values were larger than those used for most previous research work. The significant tension stiffening contribution from concrete was the underlying justification for observed average stiffness. Localized regions of reduced stiffness were confined to the joints between precast members. Direct inference of generic diaphragm stiffness for applications outside the scope of testing is not possible because of the direct relationship of strength and stiffness.

Results from the Seattle and Berkeley DBE tests are not appropriate for elastic diaphragm modeling because of their nonlinear response. The secant stiffness values utilized in the model validation of CHAPTER 5 were obtained from the nonlinear moment-curvature response and therefore are not recommended for elastic design purposes. Furthermore, damage at the midspan of the second floor diaphragm precludes application of results from this region for tests after the Knoxville DBE trials.
8.2.3.2 Post-elastic stiffness

Significant but repairable damage was incurred during testing as a result of weld failure in the pretopped double tee floor. Damage accumulation caused by repeated failure elsewhere was a significant factor for poor performance at the midspan joint in the topped hollow-core floor. Only a limited number of tests provided reliable data from this joint. The repeated failures in the pretopped double tee floor’s midspan joint highlight the repercussions of weld and tolerance quality control. No post-yield data was available from this floor until a continuous pour strip was used.

Effective stiffness used in the analytical model validation was obtained from the secant stiffness of the experimental moment-curvature response. The secant stiffness was used for elastic analysis to predict nonlinear behavior. The scope of the analytical research was limited to the elastic diaphragm response, so no conclusions are drawn relating to the post-elastic stiffness.

8.2.3.3 Topped double tee systems

Temperature and shrinkage cracking in the topping was isolated to the joints between precast double tees, which were not tooled. Crack propagation and widening resulted as demands increased, but significant damage was not incurred. Flexural cracking was primarily isolated to those cracks that were initiated at the joints. Cracking was only evident in the topping. No flexural cracks were observed in the double tee units from below. Further analysis is needed for the demands in this floor at larger amplitudes after extended testing, post June 20th, 2008.
8.2.3.4 Topped hollow core systems

Temperature and shrinkage cracking in the topping was isolated to the joints between precast double tees, which were not tooled. Flexural cracking was primarily isolated to those cracks that were initiated at the joints. Cracking was only evident in the topping. No flexural cracks were observed in the hollow-core units from below.

Partial delamination of the embedded plate providing wall-to-floor connection was observed in one location at the South wall’s West end after the Berkeley MCE – trial 1 test. This may have been caused by vertical uplift of the floor by the wall during the overloaded condition rather than shear failure.

Bar buckling in the second floor diaphragm was a result of exceptional demands placed on this floor in the Seattle DBE tests (trials 1 through 3) after failure of the third floor midspan joint. The second floor diaphragm was largely responsible for maintaining structural integrity during repeated failures as evident in the accumulated damage. Chord buckling highlighted the consequences of not including anti-buckling restraint in topped diaphragm systems. However, this was not anticipated under expected demands and there were no evident consequences for the damage. A lap splice repair was implemented to permit continued testing, but incurred damage was still evident in the response of the diaphragm at this joint.

The sustained damage limited the capacity and dynamic characteristics of the system. Significant stiffness degradation occurred in continued testing. The large crack at the midspan joint resulted in asymmetrical response with an acceleration profile on either side of this joint which were, at times, out of phase. The width of the crack may have prevented shear friction in the concrete causing this disjointed behavior.
8.2.3.5 Pretopped double tee systems

Significant damage was sustained to the test structure in the first three trials of the Seattle DBE tests. Fracture of pretopped chord connections in the third floor was the source of failure. Insufficient weld penetration on the embedded side of the connector was viewed as being responsible for the fracture, but performance may have been affected by vertical and horizontal alignment of embed plates. This was exacerbated by construction tolerances related to the half scale structure.

A continuous pour strip modified the pretopped system for tests after the Seattle DBE – trial 3. This successfully mitigated the previous failures resulting from weld quality control and misalignment in the load path between chord connectors. Under larger loading conditions, cracking was more distributed in the pour strip than in the toppings of the other floors. Cracking was observed in the pour strip, but not in the flanges of the double tee units. Embedded in the flanges were the original chord bars, so although they were discontinuous there was considerable reinforcement at the ends of the pretopped double tee units.

8.2.3.6 Wall-to-floor connections

A floor-to-wall connection detail that permitted vertical movement between these elements was included in the test structure. This detail may not be practical in conventional buildings, but it highlights the need for compatibility between elements. Failure of these connectors after the June 20th, 2008 test was likely a consequence of the large wall rotation demands during the Berkeley MCE – trial 1 test. Uplift of the floor
units by the wall through these connectors was apparent and may have reduced the
connector strength.

8.2.4 Recommendations based on experimental evidence

8.2.4.1 Diaphragm stiffness

Based on the developed crack pattern at joint locations and the tension stiffening
effect of concrete, the lower bound estimate of diaphragm stiffness formulated in Eqn.
6.3 was unacceptably low for elastic diaphragm behavior. A revised estimation is
recommended in Eqn. 6.4 to account for the tension stiffening.

8.2.4.2 Mitigation techniques for damage incurred by the diaphragms

To prevent chord reinforcement from buckling in topped diaphragm systems,
anti-buckling is recommended. This is necessary if nonlinear deformation demands of
the diaphragm are expected.

To prevent weld failure of pretopped chord systems, it is recommended that the
weld affected region not be subjected to plastic strain demands. This is achievable by
relocating double tee joints away from column lines where concentrated joint opening
coincides with the discontinuity of beams. Alternatively, a capacity design to the
diaphragm can be achieved with a reduced section at the double tee’s centerline that
provides capacity protection to the joint. This is consistent with objectives of a reduced
beam flange connection developed for steel frame structures.
8.2.4.3 Wall-to-floor connections

Wall-to-floor connections are critical to the lateral load path. These must be designed along with other critical shear regions with a capacity based approach. Diaphragm flexural overstrength will likely be generated at hazards greater than the design level where nonlinear response may be appropriate and acceptable. Therefore, capacity protection of the load path enabling flexural overstrength is critical.

8.2.4.4 Hybrid walls

To prevent failure of un-bonded post-tensioning strands at their anchors in hybrid walls, it is recommended that ends of the strands be grouted in their ducts.

Rocking wall systems with mild steel reinforcement included for energy dissipation should include auxiliary bars in un-grouted ducts for post-earthquake repairs. The strain state in a bar post-event cannot be known without instrumentation. The un-grouted bars provide a means to ensure expected performance in a subsequent event. Core drilling of the original bars is recommended and practical to ensure gap closure, which provides the self-centering characteristic of this system.

8.3 Future research needs

8.3.1 Diaphragm design forces

Estimation of diaphragm demands using the proposed modal FMR method at sites with soft soil conditions needs further investigation. Un-conservative estimates at the Charleston site prompted this recommendation. Although nominal capacity of the diaphragm based on a strength reduction factor of 0.9 and expected material strengths
would have exceeded the average demand from ten ground motions, this result signifies
that further investigation on the application of the proposed design forces in areas with
soft soil conditions is needed. Relative values between the peak ground acceleration
and spectral acceleration at the building’s fundamental period may provide insight for
this further investigation.

8.3.2 Analytical investigations

Incorporation of columns in the analytical models had an important effect on
interstory drift ratios by reducing or eliminating out of phase response at consecutive
floors. This small and often neglected contribution had a profound reduction in
interstory drift demands as compared with other research. It is supposed, therefore, that
the effect of beams, spandrels, and their secondary connections to the diaphragm or
columns may also be important. The simplified models investigated did not account for
these components because there was no perceived influence in the experimental test
results. However, the secondary connectors utilized in the test were selected because
their flexibility was expected to have the least contribution to diaphragm action.
Alternate force paths may develop with connectors of different strength and stiffness
characteristics than those tested. This is recommended as future analytical needs
because of the ability to model varied connector influence.
8.3.3 Experimental research

8.3.3.1 Continued research

The significant amount of validation data obtained in the experimental research presented provides a path for future analysis. Scopes of the analytical and theoretical formulations were limited to the elastic behavior of precast diaphragms. The nonlinear performance of the floors is an area that needs further exploration. This can be investigated with the results already obtained yet not fully analyzed.

Accumulated damage in the second resulted in significant changes to the dynamic properties of the test structure. This is unfortunate because it makes validation models impractical and results difficult to interpret. However, diaphragm response was measured for tests no matter the outcome. This provides the opportunity to investigate diaphragm behavior even though loading conditions were not ideal.

8.3.3.2 Pretopped chord connections

The brittle chord failures observed in the shake table test deserve special attention by future research. Although the sources of failure are preventable with high quality assurance, recommended techniques may mitigate this potential problem. The practicability of these recommendations or proof tests of existing details with eccentricities in the load path under expected field tolerances need further experimental investigation.
8.3.3.3 Secondary connections

Alternate force paths may develop because of secondary connections to diaphragms. The affect of this should be assessed analytically, but diaphragm subsystem test are critical for validation of assumptions inherently made through the analytical modeling process.

8.3.3.4 Wall-to-floor connections

Highlighted by the repeated fatigue failure of wall-to-floor connections, a comprehensive investigation is warranted for the cyclic response of this critical linkage. Fatigue of the connectors is suggested as the cause of failure, but the consequence of vertical connector impact at wall failure in the Berkeley MCE – trial 1 test likely had an important role in the subsequent connector fatigue failures. Although the non-traditional connector provided boundary conditions at this connection that do not represent typical construction, the severe consequence of a shear failure, which terminated the test program with partial collapse due to unseating of the floor units from their end support, cannot be underestimated justifying further investigation.

8.3.3.5 Anti-buckling restraint for chord reinforcement

Seismic detailing in precast concrete diaphragms should address unrestrained buckling problem that exists in topped diaphragm systems. Whether an elastic design is sufficient to eliminate this phenomenon in the chord reinforcement or if nonlinear diaphragm design provisions are warranted, analytical and experimental research on this topic are necessary. The location of chord buckling coincides with the joint between
precast units because of concentrated deformation demands at these locations, especially at column lines. This configuration makes anti-buckling restraint challenging because anchorage of the restraining reinforcement must be placed a distance away from the flange edge and the detail would have to accommodate caulking of the joint or, more typically, wide tar paper rolled over the joint. These practical detail considerations should be accounted for in the development of potentially necessary restraint mechanisms.

8.3.3.6 Concluding remarks

For over 30 years, the call for experimental programs on precast concrete diaphragm behavior has been made [81]. The monumental effort of the DSDM Consortium to mobilize the support for and execution of the experimental shake table test was extraordinary and made it possible to address this need. The remarkable dedication of the DSDM Task Group to the improvement of their industry was evident in their participation in and support of the research conducted. Hopefully, the significance of the presented research will inspire continued endeavors to enhance the seismic performance of precast concrete structures.
APPENDIX A

Instrumentation layout for the experimental shake table tests conducted in June 20, 2008 is shown in Figures A-1 through A-24. Changes to the instrumentation location between test days necessitated test date specific instrumentation layout drawings. These are archived at the DSDM project's website on the NEEScentral data repository [66]. Included in the layout drawings are the sensor reference name, sensor type, sensor orientation, three-dimensional coordinates from the centroid of the South wall, corresponding DAQ system, and a gauge length if appropriate.

Figure A-1: Shake table test instrumentation layout (6/20/2008) – Foundation level, 1 of 2
Figure A-2: Shake table test instrumentation layout (6/20/2008) – Foundation level, 2 of 2

Figure A-3: Shake table test instrumentation layout (6/20/2008) – Foundation level wall LVDTs
Figure A-4: Shake table test instrumentation layout (6/20/2008) – Foundation level energy dissipating strain gauges

Figure A-5: Shake table test instrumentation layout (6/20/2008) – Foundation level wall concrete strain gauges
Figure A-6: Shake table test instrumentation layout (6/20/2008) – First floor accelerometers and chord LVDTs, plan view

Figure A-7: Shake table test instrumentation layout (6/20/2008) – Second floor accelerometers and chord LVDTs, plan view
Figure A-8: Shake table test instrumentation layout (6/20/2008) – Third floor accelerometers and chord LVDTs, plan view

Figure A-9: Shake table test instrumentation layout (6/20/2008) – First floor column accelerometers and joint shear string potentiometers, plan view
Figure A-10: Shake table test instrumentation layout (6/20/2008) – Second floor column accelerometers and joint shear string potentiometers

Figure A-11: Shake table test instrumentation layout (6/20/2008) – Third floor column accelerometers and joint shear string potentiometers, plan view
Figure A-12: Shake table test instrumentation layout (6/20/2008) – First floor beam-to-slab LVDTs, plan view

Figure A-13: Shake table test instrumentation layout (6/20/2008) – Second floor beam-to-slab LVDTs, plan view
Figure A-14: Shake table test instrumentation layout (6/20/2008) – Third floor beam-to-slab LVDTs, plan view

Figure A-15: Shake table test instrumentation layout (6/20/2008) – First floor slab strain gauges, plan view
Figure A-16: Shake table test instrumentation layout (6/20/2008) – Second floor slab strain gauges, plan view

Figure A-17: Shake table test instrumentation layout (6/20/2008) – Third floor slab strain gauges, plan view
Figure A-18: Shake table test instrumentation layout (6/20/2008) – Third floor payload project, plan view

Figure A-19: Shake table test instrumentation layout (6/20/2008) – Interior West elevation view
Figure A-20: Shake table test instrumentation layout (6/20/2008) – Interior East elevation view

Figure A-21: Shake table test instrumentation layout (6/20/2008) – Exterior West elevation view
Figure A-22: Shake table test instrumentation layout (6/20/2008) – Exterior East elevation view

Figure A-23: Shake table test instrumentation layout (6/20/2008) – Interior North and South elevation views
Figure A-24: Shake table test instrumentation layout (6/20/2008) – Exterior North and South elevation views
APPENDIX B

Design and modeling parameters are provided for the analytical study of CHAPTER 6. The tabulated wall properties for each floor aspect ratio, AR, are the number of walls at each end of the longitudinal floor span, \( N_{\text{walls}} \), the length of these walls, \( l_w \), the width of these walls, \( b_w \), the longitudinal reinforcement ratio, \( \rho_l \), the axial load ratio, \( \frac{N}{A_g f'_c} \), and the ratio of yield moment to design moment, \( \frac{M_y}{M_u} \). These values are provided for both prototype configurations.

The tabulated information for the wall hysteresis includes the ratio of initial modulus to gross section modulus, \( \frac{I_o}{I_g} \), the ratio of shear area to gross section area, \( \frac{A_{sh}}{A_g} \), the cracking to yield moment ratio, \( \frac{M_{cr}}{M_y} \), the ratio of secant stiffness to yield to the initial stiffness, \( \rho_{sec} \), the ratio of post-yield stiffness to initial stiffness, \( r \), and the plastic hinge length normalized by the wall length, \( \frac{l_p}{l_w} \).

Wall properties for the prototype structures in the Knoxville site are provided in Tables B-1 through B-6.

Wall properties for the prototype structures in the Charleston site are provided in Tables B-7 through B-12.

Wall properties for the prototype structures in the Seattle site are provided in Tables B-13 through B-18.
Wall properties for the prototype structures in the Berkeley site are provided in Tables B-19 through B-24.

Wall properties for the prototype structures in the Berkeley site with alternative floor weights are provided in Tables B-25 through B-30.

Wall properties for the prototype structures in the Charleston site with R=6 are provided in Tables B-31 through B-36.

Table B-1: Analytical investigation - wall properties for the Knoxville site with n=3

<table>
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<th>AR</th>
<th>$N_{\text{walls}}$</th>
<th>$l_w$ [ft]</th>
<th>$b_w$ [in]</th>
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<th>$\frac{N}{A_g f'c}$</th>
<th>$\frac{M_y}{M_u}$</th>
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</table>

Table B-2: Analytical investigation - wall modeling parameters for the Knoxville site with n=3

<table>
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<tr>
<th>AR</th>
<th>$\frac{I_p}{I_g}$</th>
<th>$\frac{A_{sh}}{A_g}$</th>
<th>$\frac{M_{cr}}{M_y}$</th>
<th>$r$</th>
<th>$\frac{r}{l_w}$</th>
<th>$\frac{I_p}{I_g}$</th>
<th>$\frac{A_{sh}}{A_g}$</th>
<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{\sec}$</th>
<th>$r$</th>
<th>$\frac{r}{l_w}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>1.30</td>
<td>0.34</td>
<td>0.39</td>
<td>0.15</td>
<td>0.030</td>
<td>0.50</td>
<td>1.35</td>
<td>0.28</td>
<td>0.28</td>
<td>0.21</td>
<td>0.10</td>
</tr>
<tr>
<td>2.5</td>
<td>1.29</td>
<td>0.34</td>
<td>0.49</td>
<td>0.14</td>
<td>0.022</td>
<td>0.50</td>
<td>1.26</td>
<td>0.32</td>
<td>0.39</td>
<td>0.15</td>
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<td>3.0</td>
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<td>0.32</td>
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<td>0.14</td>
<td>0.020</td>
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<td>0.32</td>
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<td>1.28</td>
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<td>0.54</td>
<td>0.14</td>
<td>0.020</td>
<td>0.50</td>
<td>1.34</td>
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Table B-3: Analytical investigation - wall properties for the Knoxville site with n=5

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<tr>
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<th>l_w [ft]</th>
<th>b_w [in]</th>
<th>\rho_l</th>
<th>N \cdot \frac{A_g f_c'}{M_y}</th>
<th>\frac{M_c}{M_y}</th>
<th>N_{walls}</th>
<th>l_w [ft]</th>
<th>b_w [in]</th>
<th>\rho_l</th>
<th>N \cdot \frac{A_g f_c'}{M_y}</th>
<th>\frac{M_c}{M_y}</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>2</td>
<td>22</td>
<td>10</td>
<td>0.33</td>
<td>0.046</td>
<td>1.33</td>
<td>3</td>
<td>20</td>
<td>10</td>
<td>0.88</td>
<td>0.044</td>
<td>1.42</td>
</tr>
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<td>2</td>
<td>20</td>
<td>10</td>
<td>0.34</td>
<td>0.048</td>
<td>1.41</td>
<td>2</td>
<td>20</td>
<td>10</td>
<td>1.01</td>
<td>0.044</td>
<td>1.30</td>
</tr>
<tr>
<td>3.0</td>
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<td>18</td>
<td>10</td>
<td>0.32</td>
<td>0.050</td>
<td>1.37</td>
<td>2</td>
<td>20</td>
<td>10</td>
<td>0.88</td>
<td>0.044</td>
<td>1.42</td>
</tr>
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<td>0.32</td>
<td>0.053</td>
<td>1.31</td>
<td>2</td>
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</tr>
</tbody>
</table>

Table B-4: Analytical investigation - wall modeling parameters for the Knoxville site with n=5

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<tr>
<th>AR</th>
<th>I_o</th>
<th>I_g</th>
<th>A_{sh}</th>
<th>\frac{\rho_{sec}}{I_o}</th>
<th>r</th>
<th>I_o</th>
<th>I_g</th>
<th>A_{sh}</th>
<th>\frac{\rho_{sec}}{I_o}</th>
<th>r</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>1.32</td>
<td>0.40</td>
<td>0.49</td>
<td>0.14</td>
<td>0.031</td>
<td>0.50</td>
<td>1.40</td>
<td>0.36</td>
<td>0.28</td>
<td>0.23</td>
</tr>
<tr>
<td>2.5</td>
<td>1.32</td>
<td>0.38</td>
<td>0.48</td>
<td>0.15</td>
<td>0.029</td>
<td>0.50</td>
<td>1.39</td>
<td>0.36</td>
<td>0.26</td>
<td>0.24</td>
</tr>
<tr>
<td>3.0</td>
<td>1.30</td>
<td>0.34</td>
<td>0.49</td>
<td>0.15</td>
<td>0.026</td>
<td>0.50</td>
<td>1.40</td>
<td>0.36</td>
<td>0.28</td>
<td>0.23</td>
</tr>
<tr>
<td>3.5</td>
<td>1.30</td>
<td>0.39</td>
<td>0.48</td>
<td>0.15</td>
<td>0.023</td>
<td>0.50</td>
<td>1.36</td>
<td>0.33</td>
<td>0.36</td>
<td>0.18</td>
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</table>

Table B-5: Analytical investigation - wall properties for the Knoxville site with n=8

<table>
<thead>
<tr>
<th>AR</th>
<th>N_{walls}</th>
<th>l_w [ft]</th>
<th>b_w [in]</th>
<th>\rho_l</th>
<th>N \cdot \frac{A_g f_c'}{M_y}</th>
<th>\frac{M_c}{M_y}</th>
<th>N_{walls}</th>
<th>l_w [ft]</th>
<th>b_w [in]</th>
<th>\rho_l</th>
<th>N \cdot \frac{A_g f_c'}{M_y}</th>
<th>\frac{M_c}{M_y}</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
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<td>25</td>
<td>12</td>
<td>1.01</td>
<td>0.063</td>
<td>1.24</td>
<td>3</td>
<td>20</td>
<td>12</td>
<td>1.26</td>
<td>0.062</td>
<td>1.33</td>
</tr>
<tr>
<td>2.5</td>
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<td>12</td>
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<td>0.063</td>
<td>1.25</td>
<td>3</td>
<td>20</td>
<td>12</td>
<td>0.90</td>
<td>0.062</td>
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<td>0.063</td>
<td>1.27</td>
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<td>20</td>
<td>12</td>
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<td>0.062</td>
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</tr>
<tr>
<td>3.5</td>
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<td>0.065</td>
<td>1.28</td>
<td>2</td>
<td>22</td>
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</table>

Table B-6: Analytical investigation - wall modeling parameters for the Knoxville site with n=8

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<th>I_g</th>
<th>A_{sh}</th>
<th>\frac{\rho_{sec}}{I_o}</th>
<th>r</th>
<th>I_o</th>
<th>I_g</th>
<th>A_{sh}</th>
<th>\frac{\rho_{sec}}{I_o}</th>
<th>r</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>1.42</td>
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<td>0.28</td>
<td>0.26</td>
<td>0.147</td>
<td>0.50</td>
<td>1.45</td>
<td>0.36</td>
<td>0.24</td>
<td>0.29</td>
</tr>
<tr>
<td>2.5</td>
<td>1.41</td>
<td>0.37</td>
<td>0.34</td>
<td>0.22</td>
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<td>0.50</td>
<td>1.43</td>
<td>0.35</td>
<td>0.30</td>
<td>0.24</td>
</tr>
<tr>
<td>3.0</td>
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<td>0.37</td>
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<td>1.39</td>
<td>0.38</td>
<td>0.37</td>
<td>0.20</td>
</tr>
<tr>
<td>3.5</td>
<td>1.38</td>
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<td>0.36</td>
<td>0.21</td>
<td>0.080</td>
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<td>1.41</td>
<td>0.37</td>
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</tbody>
</table>
Table B-7: Analytical investigation - wall properties for the Charleston site with n=3

<table>
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<tr>
<th>AR</th>
<th>$N_{walls}$</th>
<th>$l_w$ [ft]</th>
<th>$b_w$ [in]</th>
<th>$\rho_l$</th>
<th>$\frac{N}{A_g f'_c}$</th>
<th>$\frac{M_x}{M_y}$</th>
<th>$N_{walls}$</th>
<th>$l_w$ [ft]</th>
<th>$b_w$ [in]</th>
<th>$\rho_l$</th>
<th>$\frac{N}{A_g f'_c}$</th>
<th>$\frac{M_x}{M_y}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>3</td>
<td>16</td>
<td>10</td>
<td>1.01</td>
<td>0.032</td>
<td>1.25</td>
<td>5</td>
<td>12</td>
<td>12</td>
<td>1.94</td>
<td>0.030</td>
<td>1.42</td>
</tr>
<tr>
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<td>3</td>
<td>16</td>
<td>10</td>
<td>0.85</td>
<td>0.032</td>
<td>1.37</td>
<td>4</td>
<td>12</td>
<td>12</td>
<td>1.94</td>
<td>0.030</td>
<td>1.42</td>
</tr>
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<td>3.0</td>
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<td>16</td>
<td>10</td>
<td>1.01</td>
<td>0.032</td>
<td>1.28</td>
<td>4</td>
<td>12</td>
<td>12</td>
<td>1.54</td>
<td>0.030</td>
<td>1.40</td>
</tr>
<tr>
<td>3.5</td>
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<td>10</td>
<td>0.85</td>
<td>0.032</td>
<td>1.28</td>
<td>3</td>
<td>12</td>
<td>12</td>
<td>1.57</td>
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</table>

Table B-8: Analytical investigation - wall modeling parameters for the Charleston site with n=3

<table>
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<tr>
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<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
<th>$r$</th>
<th>$I_p$</th>
<th>$I_g$</th>
<th>$\frac{A_{sh}}{A_g}$</th>
<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
<th>$r$</th>
<th>$I_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>1.37</td>
<td>0.31</td>
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<td>0.24</td>
<td>0.13</td>
<td>0.50</td>
<td>1.46</td>
<td>0.31</td>
<td>0.15</td>
<td>0.35</td>
<td>0.20</td>
<td>0.50</td>
</tr>
<tr>
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<td>0.27</td>
<td>0.22</td>
<td>0.12</td>
<td>0.50</td>
<td>1.46</td>
<td>0.31</td>
<td>0.15</td>
<td>0.35</td>
<td>0.20</td>
<td>0.50</td>
</tr>
<tr>
<td>3.0</td>
<td>1.37</td>
<td>0.31</td>
<td>0.24</td>
<td>0.24</td>
<td>0.13</td>
<td>0.50</td>
<td>1.45</td>
<td>0.31</td>
<td>0.18</td>
<td>0.31</td>
<td>0.26</td>
<td>0.50</td>
</tr>
<tr>
<td>3.5</td>
<td>1.38</td>
<td>0.32</td>
<td>0.27</td>
<td>0.22</td>
<td>0.12</td>
<td>0.50</td>
<td>1.40</td>
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Table B-9: Analytical investigation - wall properties for the Charleston site with n=5

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<th>$l_w$ [ft]</th>
<th>$b_w$ [in]</th>
<th>$\rho_l$</th>
<th>$\frac{N}{A_g f'_c}$</th>
<th>$\frac{M_x}{M_y}$</th>
<th>$N_{walls}$</th>
<th>$l_w$ [ft]</th>
<th>$b_w$ [in]</th>
<th>$\rho_l$</th>
<th>$\frac{N}{A_g f'_c}$</th>
<th>$\frac{M_x}{M_y}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
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<td>0.036</td>
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<td>14</td>
<td>1.65</td>
<td>0.035</td>
<td>1.24</td>
</tr>
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<td>0.036</td>
<td>1.34</td>
<td>3</td>
<td>20</td>
<td>14</td>
<td>1.91</td>
<td>0.035</td>
<td>1.31</td>
</tr>
<tr>
<td>3.0</td>
<td>1</td>
<td>25</td>
<td>14</td>
<td>1.81</td>
<td>0.036</td>
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<td>3</td>
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<td>14</td>
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<td>0.035</td>
<td>1.39</td>
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<td>0.036</td>
<td>1.39</td>
<td>2</td>
<td>20</td>
<td>14</td>
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<td>0.035</td>
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</table>

Table B-10: Analytical investigation - wall modeling parameters for the Charleston site with n=5

<table>
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<th>$I_g$</th>
<th>$\frac{A_{sh}}{A_g}$</th>
<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
<th>$r$</th>
<th>$I_p$</th>
<th>$I_g$</th>
<th>$\frac{A_{sh}}{A_g}$</th>
<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
<th>$r$</th>
<th>$I_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>1.42</td>
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<td>0.22</td>
<td>0.27</td>
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<td>0.50</td>
<td>1.46</td>
<td>0.31</td>
<td>0.17</td>
<td>0.33</td>
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<td>0.50</td>
</tr>
<tr>
<td>2.5</td>
<td>1.39</td>
<td>0.31</td>
<td>0.25</td>
<td>0.24</td>
<td>0.182</td>
<td>0.50</td>
<td>1.46</td>
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<td>0.35</td>
<td>0.28</td>
<td>0.50</td>
</tr>
<tr>
<td>3.0</td>
<td>1.46</td>
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<td>0.50</td>
<td>1.46</td>
<td>0.34</td>
<td>0.17</td>
<td>0.33</td>
<td>0.29</td>
<td>0.50</td>
</tr>
<tr>
<td>3.5</td>
<td>1.45</td>
<td>0.33</td>
<td>0.18</td>
<td>0.32</td>
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<td>0.16</td>
<td>0.35</td>
<td>0.28</td>
<td>0.50</td>
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</table>
Table B-11: Analytical investigation - wall properties for the Charleston site with n=8

<table>
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<th>Nwalls</th>
<th>lw [ft]</th>
<th>bw [in]</th>
<th>$\rho_l$</th>
<th>$\frac{N}{A_g f'_c}$</th>
<th>$\frac{M_y}{M}_y$</th>
<th>$\frac{N}{A_g f'_c}$</th>
<th>$\frac{M_y}{M}_y$</th>
</tr>
</thead>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
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<td>20</td>
<td>1.65</td>
<td>0.046</td>
<td>1.26</td>
<td>4</td>
<td>1.76</td>
</tr>
<tr>
<td>2.5</td>
<td>2</td>
<td>27</td>
<td>18</td>
<td>1.39</td>
<td>0.048</td>
<td>1.23</td>
<td>3</td>
<td>1.84</td>
</tr>
<tr>
<td>3.0</td>
<td>2</td>
<td>26</td>
<td>18</td>
<td>1.20</td>
<td>0.049</td>
<td>1.26</td>
<td>3</td>
<td>1.81</td>
</tr>
<tr>
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Table B-12: Analytical investigation - wall modeling parameters for the Charleston site with n=8

<table>
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<th>$\frac{I_o}{I_g}$</th>
<th>$\frac{A_{sh}}{A_g}$</th>
<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
<th>$\frac{r}{I_o}$</th>
<th>$\frac{l_p}{lw}$</th>
<th>$\frac{I_o}{I_g}$</th>
<th>$\frac{A_{sh}}{A_g}$</th>
<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
<th>$\frac{r}{I_o}$</th>
<th>$\frac{l_p}{lw}$</th>
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Table B-13: Analytical investigation - wall properties for the Seattle site with n=3

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<th>bw [in]</th>
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<th>$\frac{M_y}{M}_y$</th>
<th>$\frac{N}{A_g f'_c}$</th>
<th>$\frac{M_y}{M}_y$</th>
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Table B-14: Analytical investigation - wall modeling parameters for the Seattle site with n=3

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<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
<th>$\frac{r}{I_o}$</th>
<th>$\frac{l_p}{lw}$</th>
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<th>$A_{sh}/A_g$</th>
<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
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<td>$b_w$ [in]</td>
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<td>$\frac{M_y}{M_y}$</td>
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<td>1.41</td>
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Table B-15: Analytical investigation - wall properties for the Seattle site with n=5

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Table B-16: Analytical investigation - wall modeling parameters for the Seattle site with n=5

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Table B-17: Analytical investigation - wall properties for the Seattle site with n=8

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Table B-18: Analytical investigation - wall modeling parameters for the Seattle site with n=8
Table B-19: Analytical investigation - wall properties for the Berkeley site with n=3

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<th>$b_w$ [in]</th>
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<th>$\frac{M_y}{M_u}$</th>
<th>$N_{walls}$</th>
<th>$l_w$ [ft]</th>
<th>$b_w$ [in]</th>
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<th>$\frac{N}{A_g f_c'}$</th>
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Table B-20: Analytical investigation - wall modeling parameters for the Berkeley site with n=3

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<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
<th>$%$</th>
<th>$\frac{l_p}{l_w}$</th>
<th>$\frac{l_p}{l_g}$</th>
<th>$\frac{A_{sh}}{A_g}$</th>
<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
<th>$%$</th>
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Table B-21: Analytical investigation - wall properties for the Berkeley site with n=5

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<th>$\frac{M_y}{M_u}$</th>
<th>$N_{walls}$</th>
<th>$l_w$ [ft]</th>
<th>$b_w$ [in]</th>
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<th>$\frac{N}{A_g f_c'}$</th>
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Table B-22: Analytical investigation - wall modeling parameters for the Berkeley site with n=5

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Table B-23: Analytical investigation - wall properties for the Berkeley site with n=8

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</tr>
<tr>
<td>3.5</td>
<td>1</td>
<td>32</td>
<td>18</td>
<td>0.88</td>
<td>0.046</td>
<td>1.20</td>
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<td></td>
</tr>
</tbody>
</table>

Table B-24: Analytical investigation - wall modeling parameters for the Berkeley site with n=8

<table>
<thead>
<tr>
<th>AR</th>
<th>I_o</th>
<th>A_sh</th>
<th>M_cr</th>
<th>ρ_sec [%]</th>
<th>I_p</th>
<th>A_g</th>
<th>M_y</th>
<th>ρ_sec [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>1.41</td>
<td>0.36</td>
<td>0.29</td>
<td>0.23</td>
<td>0.157</td>
<td>0.41</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>1.43</td>
<td>0.30</td>
<td>0.26</td>
<td>0.26</td>
<td>0.220</td>
<td>0.50</td>
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</tr>
<tr>
<td>3.0</td>
<td>1.41</td>
<td>0.35</td>
<td>0.28</td>
<td>0.24</td>
<td>0.181</td>
<td>0.50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.5</td>
<td>1.26</td>
<td>0.32</td>
<td>0.27</td>
<td>0.24</td>
<td>0.049</td>
<td>0.42</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table B-25: Analytical investigation - wall properties for the Berkeley site with alternate floor weights and n=3

<table>
<thead>
<tr>
<th>AR</th>
<th>Nw</th>
<th>lw [ft]</th>
<th>bw [in]</th>
<th>N</th>
<th>M_y</th>
<th>M_x</th>
<th>M_y</th>
<th>M_x</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>3</td>
<td>16</td>
<td>12</td>
<td>1.05</td>
<td>0.028</td>
<td>1.30</td>
<td>1.0</td>
<td>1.30</td>
</tr>
<tr>
<td>2.5</td>
<td>3</td>
<td>16</td>
<td>12</td>
<td>0.76</td>
<td>0.028</td>
<td>1.23</td>
<td>1.0</td>
<td>1.23</td>
</tr>
<tr>
<td>3.0</td>
<td>2</td>
<td>16</td>
<td>12</td>
<td>1.05</td>
<td>0.028</td>
<td>1.30</td>
<td>1.0</td>
<td>1.30</td>
</tr>
<tr>
<td>3.5</td>
<td>2</td>
<td>16</td>
<td>12</td>
<td>0.86</td>
<td>0.028</td>
<td>1.27</td>
<td>1.0</td>
<td>1.27</td>
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</tbody>
</table>

Table B-26: Analytical investigation - wall modeling parameters for the Berkeley site with alternate floor weights and n=3

<table>
<thead>
<tr>
<th>AR</th>
<th>l_o/l_g</th>
<th>A_sh/A_g</th>
<th>M_cr/M_y</th>
<th>r</th>
<th>l_o/l_w</th>
<th>A_sh/A_g</th>
<th>M_cr/M_y</th>
<th>r</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>1.44</td>
<td>0.35</td>
<td>0.23</td>
<td>0.26</td>
<td>0.198</td>
<td>0.50</td>
<td>1.37</td>
<td>0.31</td>
</tr>
<tr>
<td>2.5</td>
<td>1.30</td>
<td>0.31</td>
<td>0.28</td>
<td>0.21</td>
<td>0.107</td>
<td>0.50</td>
<td>1.29</td>
<td>0.34</td>
</tr>
<tr>
<td>3.0</td>
<td>1.44</td>
<td>0.31</td>
<td>0.23</td>
<td>0.26</td>
<td>0.198</td>
<td>0.50</td>
<td>1.40</td>
<td>0.34</td>
</tr>
<tr>
<td>3.5</td>
<td>1.37</td>
<td>0.31</td>
<td>0.27</td>
<td>0.22</td>
<td>0.153</td>
<td>0.50</td>
<td>1.41</td>
<td>0.33</td>
</tr>
</tbody>
</table>

Table B-27: Analytical investigation - wall properties for the Berkeley site with alternate floor weights and n=5

<table>
<thead>
<tr>
<th>AR</th>
<th>Nw</th>
<th>lw [ft]</th>
<th>bw [in]</th>
<th>N</th>
<th>M_y</th>
<th>M_x</th>
<th>M_y</th>
<th>M_x</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>2</td>
<td>26</td>
<td>12</td>
<td>1.25</td>
<td>0.039</td>
<td>1.21</td>
<td>1.18</td>
<td>1.21</td>
</tr>
<tr>
<td>2.5</td>
<td>2</td>
<td>26</td>
<td>12</td>
<td>1.03</td>
<td>0.039</td>
<td>1.30</td>
<td>1.18</td>
<td>1.30</td>
</tr>
<tr>
<td>3.0</td>
<td>1</td>
<td>26</td>
<td>18</td>
<td>1.20</td>
<td>0.031</td>
<td>1.27</td>
<td>1.54</td>
<td>1.27</td>
</tr>
<tr>
<td>3.5</td>
<td>1</td>
<td>26</td>
<td>18</td>
<td>0.90</td>
<td>0.031</td>
<td>1.18</td>
<td>1.48</td>
<td>1.18</td>
</tr>
</tbody>
</table>

Table B-28: Analytical investigation - wall modeling parameters for the Berkeley site with alternate floor weights and n=5

<table>
<thead>
<tr>
<th>AR</th>
<th>l_o/l_g</th>
<th>A_sh/A_g</th>
<th>M_cr/M_y</th>
<th>r</th>
<th>l_o/l_w</th>
<th>A_sh/A_g</th>
<th>M_cr/M_y</th>
<th>r</th>
</tr>
</thead>
<tbody>
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<td>1.45</td>
<td>0.33</td>
<td>0.22</td>
<td>0.28</td>
<td>0.237</td>
<td>0.50</td>
<td>1.46</td>
<td>0.34</td>
</tr>
<tr>
<td>2.5</td>
<td>1.41</td>
<td>0.34</td>
<td>0.25</td>
<td>0.25</td>
<td>0.188</td>
<td>0.50</td>
<td>1.46</td>
<td>0.34</td>
</tr>
<tr>
<td>3.0</td>
<td>1.39</td>
<td>0.31</td>
<td>0.21</td>
<td>0.28</td>
<td>0.216</td>
<td>0.50</td>
<td>1.46</td>
<td>0.36</td>
</tr>
<tr>
<td>3.5</td>
<td>1.39</td>
<td>0.32</td>
<td>0.26</td>
<td>0.23</td>
<td>0.175</td>
<td>0.50</td>
<td>1.39</td>
<td>0.30</td>
</tr>
</tbody>
</table>
### Table B-29: Analytical investigation - wall properties for the Berkeley site with alternate floor weights and \( n=8 \)

| AR  | \( N_{\text{walls}} \) | \( l_w \) [ft] | \( b_w \) [in] | \( N \) \( A_g/f_c \) | \( M_y \) \( M_g \) | \( N_{\text{walls}} \) | \( l_w \) [ft] | \( b_w \) [in] | \( N \) \( A_g/f_c \) | \( M_y \) \( M_g \) |
|-----|-----------------|----------------|----------------|----------------|----------------|-----------------|----------------|----------------|----------------|----------------|----------------|
| 2.0 | 2               | 26             | 20             | 1.30           | 0.046          | 1.20            | 4              | 24             | 20             | 1.22           | 0.042          | 1.18            |
| 2.5 | 2               | 26             | 20             | 1.02           | 0.046          | 1.25            | 3              | 24             | 22             | 1.24           | 0.039          | 1.22            |
| 3.0 | 2               | 26             | 14             | 1.09           | 0.056          | 1.15            | 3              | 20             | 20             | 1.76           | 0.044          | 1.22            |
| 3.5 | 1               | 26             | 20             | 1.56           | 0.046          | 1.20            | 2              | 24             | 20             | 1.53           | 0.042          | 1.22            |

### Table B-30: Analytical investigation - wall modeling parameters for the Berkeley site with alternate floor weights and \( n=8 \)

<table>
<thead>
<tr>
<th>AR</th>
<th>( I_o/I_g )</th>
<th>( A_{sh}/A_g )</th>
<th>( M_{cr}/M_g ) ( \rho_{sec} )</th>
<th>( r )</th>
<th>( I_p/I_w )</th>
<th>( I_o/I_g )</th>
<th>( A_{sh}/A_g )</th>
<th>( M_{cr}/M_g ) ( \rho_{sec} )</th>
<th>( r )</th>
<th>( I_p/I_w )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>1.40</td>
<td>0.33</td>
<td>0.22</td>
<td>0.29</td>
<td>0.206</td>
<td>0.50</td>
<td>1.40</td>
<td>0.31</td>
<td>0.22</td>
<td>0.28</td>
</tr>
<tr>
<td>2.5</td>
<td>1.41</td>
<td>0.32</td>
<td>0.26</td>
<td>0.25</td>
<td>0.195</td>
<td>0.50</td>
<td>1.37</td>
<td>0.31</td>
<td>0.21</td>
<td>0.28</td>
</tr>
<tr>
<td>3.0</td>
<td>1.42</td>
<td>0.32</td>
<td>0.26</td>
<td>0.26</td>
<td>0.180</td>
<td>0.50</td>
<td>1.38</td>
<td>0.31</td>
<td>0.17</td>
<td>0.33</td>
</tr>
<tr>
<td>3.5</td>
<td>1.39</td>
<td>0.33</td>
<td>0.19</td>
<td>0.32</td>
<td>0.187</td>
<td>0.50</td>
<td>1.39</td>
<td>0.33</td>
<td>0.19</td>
<td>0.31</td>
</tr>
</tbody>
</table>

### Table B-31: Analytical investigation - wall properties for the Charleston site with \( R=6 \) and \( n=3 \)

<table>
<thead>
<tr>
<th>AR</th>
<th>( N_{\text{walls}} )</th>
<th>( l_w ) [ft]</th>
<th>( b_w ) [in]</th>
<th>( N ) ( A_g/f_c )</th>
<th>( M_y ) ( M_g )</th>
<th>( N_{\text{walls}} )</th>
<th>( l_w ) [ft]</th>
<th>( b_w ) [in]</th>
<th>( N ) ( A_g/f_c )</th>
<th>( M_y ) ( M_g )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>3</td>
<td>16</td>
<td>10</td>
<td>0.58</td>
<td>0.032</td>
<td>1.24</td>
<td>4</td>
<td>13</td>
<td>12</td>
<td>1.13</td>
</tr>
<tr>
<td>2.5</td>
<td>2</td>
<td>16</td>
<td>10</td>
<td>0.85</td>
<td>0.032</td>
<td>1.37</td>
<td>4</td>
<td>13</td>
<td>12</td>
<td>0.87</td>
</tr>
<tr>
<td>3.0</td>
<td>2</td>
<td>16</td>
<td>10</td>
<td>0.58</td>
<td>0.032</td>
<td>1.24</td>
<td>3</td>
<td>13</td>
<td>12</td>
<td>0.87</td>
</tr>
<tr>
<td>3.5</td>
<td>2</td>
<td>16</td>
<td>10</td>
<td>0.42</td>
<td>0.032</td>
<td>1.17</td>
<td>3</td>
<td>13</td>
<td>12</td>
<td>0.87</td>
</tr>
</tbody>
</table>

### Table B-32: Analytical investigation - wall modeling parameters for the Charleston site with \( R=6 \) and \( n=3 \)

<table>
<thead>
<tr>
<th>AR</th>
<th>( I_o/I_g )</th>
<th>( A_{sh}/A_g )</th>
<th>( M_{cr}/M_g ) ( \rho_{sec} )</th>
<th>( r )</th>
<th>( I_p/I_w )</th>
<th>( I_o/I_g )</th>
<th>( A_{sh}/A_g )</th>
<th>( M_{cr}/M_g ) ( \rho_{sec} )</th>
<th>( r )</th>
<th>( I_p/I_w )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>1.34</td>
<td>0.36</td>
<td>0.35</td>
<td>0.17</td>
<td>0.073</td>
<td>0.50</td>
<td>1.32</td>
<td>0.31</td>
<td>0.21</td>
<td>0.26</td>
</tr>
<tr>
<td>2.5</td>
<td>1.38</td>
<td>0.32</td>
<td>0.27</td>
<td>0.22</td>
<td>0.125</td>
<td>0.50</td>
<td>1.29</td>
<td>0.35</td>
<td>0.26</td>
<td>0.22</td>
</tr>
<tr>
<td>3.0</td>
<td>1.34</td>
<td>0.36</td>
<td>0.35</td>
<td>0.17</td>
<td>0.073</td>
<td>0.50</td>
<td>1.29</td>
<td>0.35</td>
<td>0.26</td>
<td>0.22</td>
</tr>
<tr>
<td>3.5</td>
<td>1.28</td>
<td>0.39</td>
<td>0.42</td>
<td>0.14</td>
<td>0.037</td>
<td>0.50</td>
<td>1.29</td>
<td>0.35</td>
<td>0.26</td>
<td>0.22</td>
</tr>
</tbody>
</table>
Table B-33: Analytical investigation - wall properties for the Charleston site with $R=6$ and $n=5$

<table>
<thead>
<tr>
<th>AR</th>
<th>$N_{\text{walls}}$</th>
<th>$l_w$ [ft]</th>
<th>$b_w$ [in]</th>
<th>$\rho_l$</th>
<th>$\frac{N}{A_g f_y}$</th>
<th>$M_y$</th>
<th>$M_x$</th>
<th>$N_{\text{walls}}$</th>
<th>$l_w$ [ft]</th>
<th>$b_w$ [in]</th>
<th>$\rho_l$</th>
<th>$\frac{N}{A_g f_y}$</th>
<th>$M_y$</th>
<th>$M_x$</th>
</tr>
</thead>
<tbody>
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<td>2.0</td>
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<td>0.99</td>
<td>0.044</td>
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<td>2.5</td>
<td>2</td>
<td>25</td>
<td>10</td>
<td>0.71</td>
<td>0.044</td>
<td>1.19</td>
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<td></td>
</tr>
<tr>
<td>3.0</td>
<td>1</td>
<td>25</td>
<td>12</td>
<td>1.28</td>
<td>0.039</td>
<td>1.30</td>
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<td></td>
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<tr>
<td>3.5</td>
<td>1</td>
<td>25</td>
<td>12</td>
<td>1.06</td>
<td>0.039</td>
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</tbody>
</table>

Table B-34: Analytical investigation - wall modeling parameters for the Charleston site $R=6$ and with $n=5$

<table>
<thead>
<tr>
<th>AR</th>
<th>$I_o$</th>
<th>$I_g$</th>
<th>$A_{sh}$</th>
<th>$A_g$</th>
<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
<th>$r$</th>
<th>$\frac{I_p}{l_w}$</th>
<th>$I_o$</th>
<th>$I_g$</th>
<th>$A_{sh}$</th>
<th>$A_g$</th>
<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
<th>$r$</th>
<th>$\frac{I_p}{l_w}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>1.39</td>
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<td>0.26</td>
<td>0.24</td>
<td>0.111</td>
<td>0.50</td>
<td></td>
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</tr>
<tr>
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<td>0.50</td>
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<td></td>
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</tr>
</tbody>
</table>

Table B-35: Analytical investigation - wall properties for the Charleston site with $R=6$ and $n=8$

<table>
<thead>
<tr>
<th>AR</th>
<th>$N_{\text{walls}}$</th>
<th>$l_w$ [ft]</th>
<th>$b_w$ [in]</th>
<th>$\rho_l$</th>
<th>$\frac{N}{A_g f_y}$</th>
<th>$M_y$</th>
<th>$M_x$</th>
<th>$N_{\text{walls}}$</th>
<th>$l_w$ [ft]</th>
<th>$b_w$ [in]</th>
<th>$\rho_l$</th>
<th>$\frac{N}{A_g f_y}$</th>
<th>$M_y$</th>
<th>$M_x$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>2</td>
<td>26</td>
<td>18</td>
<td>1.20</td>
<td>0.049</td>
<td>1.26</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>2</td>
<td>26</td>
<td>18</td>
<td>0.90</td>
<td>0.049</td>
<td>1.27</td>
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<td></td>
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</tr>
<tr>
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<td>26</td>
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<td>1.67</td>
<td>0.049</td>
<td>1.21</td>
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Table B-36: Analytical investigation - wall modeling parameters for the Charleston site with $R=6$ and $n=8$

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<th>AR</th>
<th>$I_o$</th>
<th>$I_g$</th>
<th>$A_{sh}$</th>
<th>$A_g$</th>
<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
<th>$r$</th>
<th>$\frac{I_p}{l_w}$</th>
<th>$I_o$</th>
<th>$I_g$</th>
<th>$A_{sh}$</th>
<th>$A_g$</th>
<th>$\frac{M_{cr}}{M_y}$</th>
<th>$\rho_{sec}$</th>
<th>$r$</th>
<th>$\frac{I_p}{l_w}$</th>
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<td>0.24</td>
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REFERENCES


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Large Scale-Scale Structural Testing, SP 211-8, American Concrete Institute, Michigan, pp. 161-182.


