Out-of-Plane Dynamic Stability of Unreinforced Masonry Walls Connected to Flexible Diaphragms

by

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Abstract

The vulnerability of unreinforced masonry (URM) buildings to out-of-plane damage and collapse has been clearly demonstrated in past earthquakes. Given sufficient anchorage to the diaphragms (a minimum-level retrofit), a URM wall subjected to out-of-plane inertial forces will likely develop a horizontal crack at an intermediate height. This crack will cause the wall to behave as two semi-rigid bodies, which rock in the out-of-plane direction. Past studies have demonstrated that the out-of-plane stability of a URM wall connected to the diaphragms can be related to the height to thickness ratio \((h/t)\) and the spectral acceleration at 1 s. However, treatment of the effects of diaphragm flexibility and ground motion variability on out-of-plane wall stability in studies to date has been limited.

This dissertation presents an experimental and analytical study examining the out-of-plane stability under seismic loading of URM walls connected to flexible diaphragms. In the experimental phase, five full-scale unreinforced solid clay brick wall specimens spanning one storey were subjected to earthquake ground motions using a shake table. The top and bottom of the walls were connected to the shake table through coil springs, simulating the flexibility of the diaphragms. The apparatus allowed the wall supports to undergo large absolute displacements, as well as out-of-phase top and bottom displacements, consistent with the expected performance of URM buildings with timber diaphragms. Variables examined experimentally included diaphragm stiffness and wall height.

An analytical rigid body model was validated against the experimental results, and it was demonstrated that the model was able to reproduce the observed rocking behaviour with reasonable accuracy. The validated model was used to undertake a parametric study investigating the effects of numerous parameters on out-of-plane wall stability. Ground motion variability was accounted for by using a large suite of motions. Based on the results of the modelling, an updated out-of-plane assessment procedure was proposed. The procedure, which could be incorporated into ASCE 41, provides reference curves of \(h/t\) vs. spectral acceleration at 1 s, along with correction factors for axial load, wall thickness, ground-level walls, and exposure.
Preface

This dissertation is based on original work by the author, Osmar Penner. The concept and design of the test apparatus is my own. I carried out a large portion of the construction of the test apparatus and the set-up of the tests, and supervised all testing. The set-up of the model in this study, including all code and data processing, is my work.
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List of Symbols

Note: the subscript $t,b$ indicates two variants of that symbol, referring to top and bottom locations. This is indicated in the description as t/b.

- $a$: Acceleration
- $a_{crack}$: Acceleration at crack
- $a_{t,b}$: Acceleration at t/b of wall
- $B$: Diaphragm depth
- $C_a$: Axial load correction factor
- $C_a'$: Base parameter for axial load correction factor
- $C_e$: Exposure correction factor
- $C_g$: Ground level correction factor
- $C_t$: Thickness correction factor
- $c_{t,b}$: t/b damping constant
- $c_v$: Coefficient of variation
- $d_b$: Displacement of top of bottom wall segment relative to base of wall
- $d_{diff}$: Differential displacement between top and bottom carriages
- $d_m$: Moment arm
- $d_{rel,t,b}$: Relative displacement of t/b carriage
- $d_{rel}$: Relative displacement
- $d_{rock}$: Rocking displacement
- $d_{rock,norm}$: Rocking displacement normalized to wall thickness
- $d_{rock,threshold}$: Threshold at which wall is considered to have started rocking
- $E$: Modulus of elasticity
- $e$: Eccentricity of axial load OR base of natural logarithm
- $F_d$: Total force applied to diaphragm
- $F_{It,b}$: Inertial force of t/b wall segment
### List of Symbols

- $F_{t,b}$: Horizontal reaction force at $t/b$ of wall
- $F_v$: Vertical reaction force at base of wall
- $F_w$: Total force on wall
- $f$: Frequency
- $f'_b$: Brick compressive strength
- $f'_fb$: Masonry flexural bond strength
- $f'_j$: Mortar compressive strength
- $f'_m$: Masonry compressive strength
- $f_{r_{\text{max}}}$: Peak tensile stress in wall
- $G_d$: Diaphragm shear stiffness
- $G_{d_{\text{eff}}}'$: Effective diaphragm shear stiffness
- $g$: Acceleration due to gravity
- $h$: Wall height
- $h_{cr}$: Height of crack relative to height of wall
- $h_{t,b}$: Height of $t/b$ wall segment
- $k_{t,b}$: $t/b$ spring constant
- $L$: Diaphragm span
- $\tilde{L}$: Generalized excitation factor
- $M_{d_{t,b}}$: Mass of $t/b$ diaphragm
- $M_W$: Moment magnitude
- $M_w$: Mass of wall
- $M_{w_{t,b}}$: Mass of $t/b$ wall segment
- $\tilde{m}$: Generalized mass
- $m(x)$: Mass per unit length
- $m_{\text{trib}}$: Total tributary mass
- $P$: Axial load applied to wall (overburden) OR probability
- $P_{\text{col}}$: Probability of collapse
- $p$: Axial load per unit length
- $R^2$: Coefficient of determination
- $R_{\text{ins}}$: Static instability ratio
- $R_{M_{t,b}}$: t/b ratio of diaphragm mass to wall mass
- $r_{jb}$: Joyner-Boore distance, to surface projection
### List of Symbols

- $S_a$: Spectral acceleration
- $S'_a(1)$: Corrected level of allowable spectral acceleration for given $h/t$
- $S_{ab}(1)$: Base level of allowable spectral acceleration for given $h/t$
- $S_{acol}$: Spectral acceleration at level causing collapse
- $S_d$: Spectral displacement
- $S_{major}$: Major ground motion scale increment
- $S_{minor}$: Minor ground motion scale increment
- $S_{start}$: Ground motion scale at start of simulation
- $S_{X1}$: Spectral acceleration at 1 s, in ASCE 41
- $s_h$: Horizontal depth of spalling
- $s_v$: Joint thickness at spalling location
- $T$: Period
- $T_1$: Fundamental period of general structural system
- $T_{im}$: Period used in intensity measure
- $T_{rock}$: Effective period of rocking excursion
- $T_s$: Fundamental period of a diaphragm
- $T_{t,b}$: Fundamental period of $t/b$ diaphragm
- $t$: Wall thickness OR time
- $u$: Displacement of structure relative to supports
- $u_{eq}$: Displacement of equivalent SDOF system
- $u_g$: Ground displacement
- $u^t$: Absolute displacement of structure
- $V_{S_{30}}$: Average shear wave velocity in upper 30 m of soil
- $W$: Weight of wall
- $W_d$: Diaphragm tributary weight
- $W_{t,b}$: Weight of $t/b$ wall segment
- $x$: Horizontal co-ordinate along a span
- $y$: Vertical co-ordinate up a span
- $\bar{y}$: Height to centroid of wall force
- $y_{T_{max}}$: Height to point of maximum tensile stress
- $z$: Generalized displacement
**List of Symbols**

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<tr>
<th>Symbol</th>
<th>Description</th>
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<tr>
<td>( \tilde{\Gamma} )</td>
<td>Generalized SDOF scale factor</td>
</tr>
<tr>
<td>( \Delta_d )</td>
<td>Diaphragm deflection at mid-span</td>
</tr>
<tr>
<td>( \varepsilon )</td>
<td>Ground motion shape parameter</td>
</tr>
<tr>
<td>( \zeta )</td>
<td>Damping ratio</td>
</tr>
<tr>
<td>( \zeta_{t,b} )</td>
<td>t/b damping ratio</td>
</tr>
<tr>
<td>( \theta_{i_{L,R}} )</td>
<td>L/R rotation limit for top block under axial load case ( i )</td>
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<tr>
<td>( \theta_p )</td>
<td>Processed rotation angle of bottom wall segment</td>
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<tr>
<td>( \theta_{t,b} )</td>
<td>Rotation angle from vertical of t/b wall segment</td>
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<td>( \mu )</td>
<td>Mean</td>
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<tr>
<td>( \mu_{dynamic} )</td>
<td>Dynamic coefficient of friction</td>
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<tr>
<td>( \mu_{static} )</td>
<td>Static coefficient of friction</td>
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<td>( \rho )</td>
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<td>( \psi )</td>
<td>Shape function</td>
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Chapter 1

Introduction

A significant stock of unreinforced masonry (URM) buildings is present throughout the world, including seismically active regions in Canada [Brueneau and Lamontagne, 1994]. An extensive building survey of Victoria, British Columbia found URM buildings up to 3 stories to be the second most common building type in the city centre [Onur et al., 2005]. A survey in Vancouver, BC found that URM accounts for the majority of high-seismic-vulnerability buildings [Eng, 2013]. Both of these cities are located in areas of moderate to high seismic hazard.

URM buildings representative of those considered in this study were constructed in the late 1800s to early 1900s, and typically have a simple, usually rectangular footprint with minimal vertical irregularities. Exterior walls are constructed of clay brick and consist of multiple wythes (vertical sections of masonry one brick thick). Bricks could be laid in a number of possible bond patterns, but the wythes are tied together intermittently by headers (bricks laid perpendicular to the wall, so that one brick spans two wythes). Floor and roof diaphragms are typically timber joists (or trusses) sheathed with narrow (∼ 150 mm) boards nailed to the joists. These diaphragms are typically very flexible in-plane, and are often poorly connected to the perimeter walls. Characterization of buildings is presented in more detail in Section 2.1.

1.1 Earthquake performance of URM buildings

URM buildings have performed consistently poorly in past earthquakes. Typical damage includes chimney, parapet, and gable failures, in-plane wall failures due to sliding or diagonal shear, toe crushing, or rocking, and out-of-plane wall failures. Examples of typical failures in Christchurch, New Zealand, following the Darfield earthquake of 4 September 2010 are shown in Figures 1.1–1.5. Where URM buildings have completely collapsed (i.e. only a pile of rubble remains), it is typically not possible to determine a failure mode; however, many buildings have sustained significant damage without undergoing total collapse (e.g., Figure 1.5a).
1.1. Earthquake performance of URM buildings

Figure 1.1: Chimney failures (credit: D. Dizhur)

Figure 1.2: Parapet failures (credit: D. Dizhur)
1.1. Earthquake performance of URM buildings

Figure 1.3: Gable wall failures (credit: D. Dizhur)

Figure 1.4: In-plane failures (credit: D. Dizhur)
1.1. Earthquake performance of URM buildings

Figure 1.5: Out-of-plane wall failures (credit: D. Dizhur)
1.1. Earthquake performance of URM buildings

Bruneau and Lamontagne [1994] reported on damage from numerous earthquakes in eastern Canada, and found that out-of-plane wall failure was both the most common failure mode as well as the mode presenting the greatest life safety hazard. Dizhur et al. [2011] likewise found that out-of-plane wall collapse was the most commonly observed failure in URM buildings following the 22nd February 2011 earthquake in Christchurch, New Zealand. Extensive in-plane damage was also observed, particularly shear cracking in piers and spandrels, most often occurring in a stair step pattern along mortar joints. While such damage affects the lateral load-carrying capacity of a building, gravity loads can remain supported even under large lateral displacements where cracks follow mortar joints. Shear cracks through bricks will result in much sooner loss of gravity support [Russell et al., 2013]. In-plane damage by itself thus often does not lead to collapse without excessive deformations or accompanying out-of-plane failure.

Vintage URM buildings often have limited or no lateral connections between load-bearing walls and floor and roof diaphragms. This leads to walls essentially falling off the building by tipping over when subjected to ground shaking, in what is termed a ‘cantilever’ out-of-plane failure. If walls are adequately anchored to diaphragms at each level (e.g., Figure 1.6a), this failure mode is prevented. Inadequate anchorage leads back to cantilever wall failure, commonly leaving bare anchors protruding from the diaphragms (e.g., Figure 1.6b). Given effective anchorage, out-of-plane failures occur by bending of the wall between support points. Where limited vertical supports are present between floors, such as in long walls or in piers between windows, walls can fail in one-way vertical bending — spanning between adjacent dia-
1.2. Research motivation and objectives

Due to its recognized poor earthquake performance, URM is no longer permitted as a structural system in the construction of new buildings in regions of notable seismicity in Canada [National Research Council of Canada, 2010]. In contrast, mitigation of the life safety risk presented by existing URM buildings in seismically active areas remains to date on a primarily voluntary basis in most jurisdictions [Paxton et al., 2013]. A lack of owner incentives for retrofit combined with an often high heritage building value (making demolition unlikely) highlight the need to maximize the efficiency and cost-effectiveness of retrofit schemes. Sufficient knowledge to allow identification of the most vulnerable components and systems is required to be able to prioritize retrofits and achieve the greatest reduction in seismic risk for the limited available funding.

The prevalence of out-of-plane wall failures, in combination with the associated life safety risks, have resulted in the prevention of out-of-plane failure becoming a high priority in the retrofit of URM buildings. While out-of-plane wall response has been the subject of significant research efforts in the past three decades (see Section 2.2), the treatment of the effect of diaphragm flexibility on this response in the literature has been limited. In particular, there has been a lack of dynamic testing in which the effect of diaphragm flexibility has been addressed. Current assessment guidelines (see Section 2.2.3) do not adequately address the issue, and the current state of knowledge does not provide a basis for comment on whether the existing
guidelines are conservative or unconservative in this respect. The present study aims to make a significant contribution towards filling this gap in knowledge.

The objectives of the study, each constituting a significant contribution to the body of knowledge relating to the seismic performance of URM walls, are as follows:

- Obtain observations of the dynamic one-way out-of-plane bending response of URM walls connected to flexible supports over a variety of support stiffnesses, including variable top and bottom support stiffnesses.
- Validate a modelling approach that can accurately simulate the observed dynamic response over the range of tested conditions.
- Using the validated model, conduct simulations over a large range of parameters to:
  - characterize wall response as a function of relevant parameters, and
  - investigate the effect of ground motion variability on the wall response.
- Produce recommendations for a new out-of-plane wall assessment procedure.

1.3 Thesis outline

A multi-phase study including both experimental and analytical work was undertaken to achieve the objectives listed above. This dissertation, documenting the study, is arranged as follows:

**Background and Literature Review, Chapter 2**: Typical characteristics of URM buildings to be used in the study are established, including the idealization of walls and diaphragms, and the relevant ranges of parameters describing them. Past research regarding out-of-plane response of URM walls and the effect of diaphragm flexibility is reviewed. A summary of current assessment standards is provided.

**Experimental Program, Chapter 3**: Five URM wall specimens were constructed in the laboratory and dynamically tested on a shake table in a purpose-built apparatus. The apparatus included provisions to simulate
1.3. Thesis outline

The effect of flexible diaphragms at both the top and base of the wall, and allowed walls to be tested to collapse. Details of the specimen construction, the apparatus, the instrumentation, the shake table inputs, and the testing procedure are presented.

Shake Table Test Results, Chapter 4: The wall specimens were each tested at incrementally greater amplitudes of input motion until collapse was achieved. The displacement and acceleration responses of the wall, as well as force demands, are examined in detail and discussed.

Validation of Analytical Model, Chapter 5: A rigid body computer model representing a cracked one-way out-of-plane wall spanning one storey was set up using commercially-available software. The model was validated using the results of the dynamic testing presented in the previous chapter. Details of the model construction and inputs, and comparisons of measured to predicted responses are presented.

Parametric Study, Chapter 6: The validated model was used to carry out an extensive parametric study. Over 300 different wall configurations were modelled, collectively investigating the effects of eight parameters. Each configuration was run through incremental dynamic analysis with up to 100 ground motions, producing a total of over 220,000 runs. The study was conducted in two phases. In the first phase, the effects of each model parameter are first investigated relative to a reference configuration. The second phase consists of a full combination matrix of a subset of primary parameters: slenderness ratio, diaphragm period, axial load, and crack height. The results are interpreted in terms of fragility curves providing probability of collapse for a given intensity measure, and are compiled into a series of slenderness ratio curves for varying risk levels.

Recommendations for Assessment Guidelines, Chapter 7: A new procedure for the seismic assessment of out-of-plane URM walls is proposed. The procedure accounts for diaphragm flexibility, axial load, exposure level, wall thickness, and whether a wall is at ground level or above. Considerations involved in the interpretation of the study results are discussed, and the procedure is presented. The new procedure is applied to several example scenarios and assessment results are compared with the current standard.
Chapter 2

Background and Literature Review

The following sections provide background and review of previous research on the characterization of the most relevant components — in-plane walls and diaphragms — of typical URM buildings, followed by a review of previous research on the subject of out-of-plane URM wall response.

2.1 Building characterization

Derakhshan [2010] addressed the issue of the effects of diaphragm flexibility on out-of-plane wall response from a static testing approach. While the approach to that study was different than that for the present one, the subject matter is effectively the same. In his literature review, he noted that the following characteristics had been shown to be of importance:

- Overall shape and size of building, number of stories
- Wall thickness
- Wall slenderness
- Diaphragm in-plane stiffness

Russell [2010] conducted a comprehensive assessment of New Zealand’s URM stock, grouping buildings into typologies and compiling typical characteristics for each. A comparably detailed typological study of URM buildings in western Canada has not been conducted to date. A comparison can be drawn, however, between URM construction in western Canada and in New Zealand due to the heavy British colonial influence in both areas. The cities of Victoria, BC and Christchurch, NZ are considered briefly as examples. Both are the oldest cities in their respective regions, with Victoria being incorporated in 1856 and Christchurch in 1862. URM construction in both occurred primarily between the 1880s and the 1930s. The URM stock in the
central business districts consists mostly of two- and three-storey buildings, many in row arrangements.

A small-scale survey carried out by Paxton et al. [2013] and discussions with local engineering consultants point towards general consistency between URM construction practices in western Canada and in New Zealand. While Russell found top storeys commonly constructed with both 2-wythe and 3-wythe walls, local engineers noted that 2-wythe walls were scarce, and that 3-wythe walls were typical. In addition, the upper ranges of the typical storey heights listed by Russell were considered to be higher than is typically found locally. Russell’s work, with the addition of local input, formed the basis of the building characteristics considered in the current study.

Focusing on local conditions, the default wall considered in this study is three wythes thick, with an assumed thickness of 330 mm. A 2-wythe wall is also considered, with an assumed thickness of 220 mm. Slenderness ratios of between 8 and 22 are considered, which translate to maximum heights of 7.3 m for 3-wythe walls, and 4.8 m for 2-wythe walls.

The excitation applied to the out-of-plane walls is the ground motion, filtered through the building’s in-plane system and the diaphragms. The treatment of these components is discussed in the sections below.

### 2.1.1 In-plane system

The in-plane walls for low-rise URM buildings were assumed to be very stiff compared to flexible diaphragms. As such, they were modelled as rigid, transferring the ground motion unaltered to the endpoints of each diaphragm. This assumption is consistent with past work on out-of-plane response of URM walls [ABK Joint Venture, 1981b, Doherty, 2000, Meisl, 2006].

One instrumented URM building with flexible diaphragms has been subjected to strong motion shaking. A 2-storey former firehouse in Gilroy, California was subjected to peak ground accelerations up to 0.29 g during the Loma Prieta earthquake of 1989. Tena-Colunga andAbrams [1992] present an analysis of the data, which shows that motions recorded in-plane at the top of the central URM wall exhibited notable amplification at short periods (below 0.5 s), but minimal amplification at periods longer than this. It is shown later in the study that the response of cracked walls is most sensitive to long-period energy content (see Chapter 6). It is therefore rea-
2.1. Building characterization

reasonable to assume that short-period amplification would have minimal effect on out-of-plane wall response.

Non-linear modelling of two, three, and five-storey URM buildings carried out by Knox [2012] showed significant amplification of absolute accelerations up the height of in-plane walls for the two and three-storey buildings, and minimal amplification in the five-storey building. Spectral analysis of the output motions was not published. Menon and Magenes [2011a,b] conducted a parametric study of URM buildings with rigid diaphragms using a non-linear model, which also showed the possibility of acceleration amplification up the height of the building. However, it was cautioned that buildings with flexible diaphragms behave fundamentally differently and that further work was necessary to address the topic.

The topic of ground motion amplification up the height of URM buildings with flexible diaphragms is one of great interest, but it is beyond the scope of the present study. Available data suggests the amplification in low-rise buildings is likely concentrated at short periods, to which out-of-plane walls are least sensitive. While the assumption of rigid in-plane walls is in this context considered reasonable, efforts are made to present the results of the study in sufficient detail to allow re-interpretation in the light of possible future work regarding the amplification issue.

2.1.2 Diaphragms

Floor diaphragms in vintage URM buildings commonly consist of timber sheathing supported on timber framing. In smaller buildings, joists typically span directly between load-bearing URM walls, and are either supported on the ledge created by a change in the number of wythes between adjacent stories, or are embedded in cavities created in the walls for this purpose. In larger buildings, joists may be supported by heavier timber or steel beams, and by columns in large open plan areas. Sheathing arrangements vary, and include either straight sheathing (perpendicular to the joists) or diagonal sheathing (typically at 45° to the joists), applied in either one or two layers.

While the in-plane stiffness of such diaphragms varies depending on the configuration, in general the stiffness is very low compared to alternative diaphragm systems such as concrete slabs. The matter of quantifying the stiffness of vintage timber diaphragms has been the subject of much uncertainty over the past decades. ASCE 41 [ASCE, 2014] contains suggested stiffness parameters for various diaphragm configurations, but the source and accuracy of these values has been questionable, and they have not seen any recent updates.
Cohen [2001] constructed two half-scale, single-story, reinforced concrete block buildings and subjected them to shake table testing. The roof diaphragm of one specimen consisted of diagonal timber sheathing on timber joists, the other of corrugated metal decking on open-web steel joists. Rocking behaviour of the out-of-plane walls was not observed since the walls were reinforced, but notably it was confirmed that the overall structural response was dominated by the deformation of the diaphragm rather than of the in-plane walls. Non-linear analyses [ABK Joint Venture, 1981b, Paquette and Bruneau, 1999] have likewise shown that response of low-rise URM buildings is dominated by the response of their flexible diaphragms. It is therefore clear that the characterization of flexible diaphragm response is important in the study of out-of-plane wall response.

Testing of wood diaphragms began in the 1980s with a large experimental program by ABK Joint Venture [1981a]. Additional testing, primarily within the last 10 years, has significantly improved understanding of the effects of diaphragm condition, configuration, perforations, interaction with walls, and orthotropic behaviour [Brignola et al., 2012, Giongo et al., 2013, Peralta et al., 2004, Wilson, 2012]. Wilson et al. [2013] provides the most up-to-date compilation of research on the matter, including recommendations for stiffness assessment. That analysis put significant weight on the results of Giongo et al. [2013], who conducted the first full-scale in-situ testing of the as-built diaphragms in a vintage building. The recommendations of Wilson et al. [2013] were adopted in the present study.

2.1.2.1 Idealization

Wilson [2012] showed that the deformation response of wood sheathed diaphragms is most aptly represented by a shear beam model. The stiffness of a uniform shear beam is proportional to its depth; diaphragms can therefore be characterized by the shear stiffness, $G_d$, which is independent of the plan dimensions of the diaphragm. For a diaphragm with span $L$ and depth $B$ loaded parabolically with a total load of $F_d$, the mid-span displacement is defined by Wilson et al. [2013] as:

$$\Delta_d = \frac{3}{16} \cdot \frac{F_d L}{G_d B}$$  \hspace{1cm} (2.1)$$

Wilson et al. [2013] conclude that the shear stiffness varies with diaphragm condition (defined as good, fair, or poor), orientation (perpendicular or parallel to joists) and joist continuity (continuous or discontinuous joists). For the base case of single straight-sheathed diaphragms, loading
2.1. Building characterization

parallel to joists produces the stiffest response \(G_d = 225 - 350 \text{kN/m}\), followed by perpendicular to joists–continuous \(G_d = 170 - 265 \text{kN/m}\) and perpendicular to joists–discontinuous \(G_d = 135 - 210 \text{kN/m}\). The highest value in this series, 350 kN/m, matches the value for straight-sheathed diaphragms listed in ASCE 41. Wilson et al. [2013] recommend that for other diaphragm configurations, the relative multipliers derived from ASCE 41 are applied to the new base-case values. The multipliers range up to a maximum of 9.0 for chorded diaphragms with double layered sheathing, producing a maximum base stiffness of 3150 kN/m. The listed stiffness values must finally be modified to include the effects of diaphragm penetrations and the added stiffness of boundary walls, producing the effective stiffness, \(G'_{d,eff}\). The stiffness values listed above are representative of secant stiffnesses at 100 mm displacement.

The natural period of a diaphragm with a uniformly distributed tributary weight, \(W_d\), is given by Wilson et al. [2013] as:

\[
T_s = 0.7 \cdot \sqrt{\frac{W_d L}{G'_{d,eff} B}}
\]  

(2.2)

This relation is derived by assuming a quartic displacement shape function (due to the parabolic loading recommended by ASCE 41), and calculating the properties of the generalized single-degree-of-freedom system. Here, the shape function was:

\[
\psi(x) = \frac{32}{5L} \left( \frac{1}{2} - \frac{x^3}{L^2} + \frac{x^4}{2L^3} \right)
\]  

(2.3)

The generalized mass, \(\tilde{m}\) and generalized excitation factor, \(\tilde{L}\) are calculated as functions of the total tributary mass of the diaphragm, \(m_{trib}\) [Chopra, 2007]:

\[
\tilde{m} = \int_0^L m(x) |\psi(x)|^2 \, dx = \frac{3968}{7875} \cdot m_{trib}
\]  

(2.4)

\[
\tilde{L} = \int_0^L m(x) \psi(x) \, dx = \frac{16}{25} \cdot m_{trib}
\]  

(2.5)

The above formulations are for a generalized co-ordinate, \(z\), of translation at mid-span of the diaphragm. The response of the equivalent SDOF system, \(u_{eq}\), is not equal to the response at this co-ordinate, however. To obtain the response of the generalized co-ordinate, the SDOF response must be scaled by the factor:
2.1. Building characterization

\[ \tilde{\Gamma} = \frac{\tilde{L}}{\tilde{m}} = \frac{315}{248} = 1.27 \quad (2.6) \]

Thus, the response at the generalized co-ordinate is:

\[ z(t) = \tilde{\Gamma} \cdot \text{eq}(t) = 1.27 \cdot \text{eq}(t) \quad (2.7) \]

The response of the diaphragm at any point along its span, \( x \), is:

\[ u(x, t) = \psi(x) \cdot z(t) \quad (2.8) \]

The response along the length of the diaphragm and that of the equivalent SDOF system are shown in Figure 2.1. This plot illustrates how the equivalent SDOF system is representative of approximately 0.8 times the peak diaphragm response. In this study, the equivalent SDOF response was considered to be representative of the conditions around mid-span of the diaphragm. Using the mid-span response, \( z \), as the input to out-of-plane wall excitation would be conservative when also not accounting for two-way bending effects. Nevertheless, the reader is advised to keep this simplification in mind throughout the study.

![Figure 2.1: Response of diaphragm and equivalent SDOF system](image)

The displacement history of any point along the diaphragm relative to its endpoints can be determined using Equation (2.8). This relative displacement is the same as the relative displacement of the equivalent SDOF system when subjected to the ground motion at a different scale. The total displacement, however, is the sum of the relative displacement and the displacement at the endpoints of the diaphragm (here assumed to be equal to the ground displacement):
2.1. Building characterization

\[ u^t(x, t) = u_g(t) + u(x, t) = u_g(t) + \psi(x) \cdot z(t) \] (2.9)

The total displacement is thus the sum of the unscaled ground motion and the scaled SDOF response. This total response can not be reproduced by any SDOF system, except at the points where \( u(x) = u_{eq} \) (Figure 2.1). While this was illustrated here for total displacements, the same holds for total velocities and total accelerations. Considering the equivalent SDOF response to be representative of the conditions around mid-span therefore allows the simplification of directly modelling this response as an SDOF system. This greatly simplified the representation of the system both in the experimental and analytical phases of the project.

Two conditions were covered by the study: (1) the performance around mid-span, represented by the equivalent SDOF system, and (2) the performance near the ends of the diaphragm, represented by the rigid diaphragm model. While not directly addressed due to the limitations of the simplified models, the conditions at other points along the span can reasonably be assumed to be bracketed by these two conditions.

2.1.2.2 Stiffness range

The period of a diaphragm system — defined by its stiffness and tributary mass — governs its seismic response, as opposed to its stiffness value alone. In this study, the period will therefore be used as an indicator of diaphragm stiffness.

The representative range of diaphragm periods to be used in the study was determined by considering a number of typical building configurations. The upper floor diaphragm in a two-storey building was used as the base case. Walls were assumed to be 3-wythe with a thickness of 330 mm. A relatively heavy masonry density of 2100 kg/m\(^3\) was assumed, representative of walls with few voids. Wall heights used were 4 m in the storey below and 3.6 m in the storey above. A diaphragm self weight of 0.5 kPa was used (the total tributary weight is dominated by the wall weight, so results are not very sensitive to the assumed diaphragm self weight). Four different building plans were considered: 6 \( \times \) 10 m, 8 \( \times \) 14 m, 16 \( \times \) 24 m, and an elongated plan of 8 \( \times \) 24 m. Joists were assumed to span the short length of each building, meaning the short diaphragm span is loaded perpendicular to joists, and the long span is loaded parallel to joists. Effects of penetrations and boundary wall added stiffness were ignored for this order-of-magnitude study, since the factors counter one another and the effect of either is moderate. Three stiffness values were used: the ‘poor’ and ‘good’ base case values for single
2.2. Out-of-plane wall behaviour

straight sheathing, and those using the maximum multiplier of 9.0 on the ‘good’ case. The calculated periods are listed in Table 2.1.

Table 2.1: Range of diaphragm periods

<table>
<thead>
<tr>
<th>Plan (m)</th>
<th>Span</th>
<th>Period (s)</th>
<th>Single straight sheathing</th>
<th>Two-layer, chorded</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Poor</td>
<td>Good</td>
</tr>
<tr>
<td>6 × 10</td>
<td>short</td>
<td>0.86</td>
<td>0.61</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>long</td>
<td>1.41</td>
<td>1.13</td>
<td>0.38</td>
</tr>
<tr>
<td>8 × 14</td>
<td>short</td>
<td>0.99</td>
<td>0.70</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>long</td>
<td>1.72</td>
<td>1.38</td>
<td>0.46</td>
</tr>
<tr>
<td>16 × 24</td>
<td>short</td>
<td>1.57</td>
<td>1.12</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td>long</td>
<td>2.16</td>
<td>1.73</td>
<td>0.58</td>
</tr>
<tr>
<td>8 × 24</td>
<td>short</td>
<td>0.79</td>
<td>0.56</td>
<td>0.19</td>
</tr>
<tr>
<td></td>
<td>long</td>
<td>2.95</td>
<td>2.37</td>
<td>0.79</td>
</tr>
<tr>
<td>Minimum</td>
<td></td>
<td>0.79</td>
<td>0.56</td>
<td>0.19</td>
</tr>
<tr>
<td>Maximum</td>
<td></td>
<td>2.95</td>
<td>2.37</td>
<td>0.79</td>
</tr>
</tbody>
</table>

For buildings with single straight sheathed diaphragms — the most common in vintage URM buildings — periods range between approximately 0.5 and 3 s. For very stiff diaphragms, periods range between 0.2 and 0.8 s. Values outside these ranges are possible, due to lighter or heavier walls or diaphragms and more or less elongated floor plans, but this range is representative of a majority of URM buildings.

In this study, periods up to 2 s are considered; this range covers most typical buildings. A 2 s period is already pushing the boundary of what would likely be considered an acceptable diaphragm flexibility when assessing a building due to the large deformations involved. Diaphragms at periods beyond this range (e.g., a large elongated building with single straight sheathed diaphragm in poor condition) should likely be retrofitted regardless of the predicted out-of-plane wall performance. Consequently, it is not of great interest to examine wall performance at very large periods.

2.2 Out-of-plane wall behaviour

At its simplest, an out-of-plane wall can be idealized as a one-way vertically spanning strip with a horizontal crack at some height within the span. Such
a crack does not cause collapse at its initiation; instead, the cracked wall can form a stable out-of-plane rocking response about the crack location. This one-way idealization is representative of a real wall when two-way effects are minimal (e.g., in a long wall away from cross walls and corners) and the wall is adequately anchored to the diaphragms at both the top and bottom. Where two-way effects become significant, this idealization will produce more conservative results, but nevertheless forms a good starting point for understanding the response.

Past research has predominantly focused on the one-way model, which has remained overall poorly understood despite its apparent simplicity. The following sections provide a review of past experimental work and analytical representations of this idealized behaviour. Detailed review of the relevant literature was provided most recently by Derakhshan [2010] and by Meisl [2006]. The following review serves to provide the reader with a brief background on the subject, with a focus on dynamic testing.

2.2.1 Experimental testing

Early experimental work on the topic of out-of-plane URM wall response was restricted to quasi-static testing [Anderson, 1984, West et al., 1977, Yokel and Dikkers, 1971]. Walls were supported at the top and base with varying boundary conditions, and transverse load was applied slowly. Observations included consistent horizontal cracking of wall panels near or slightly above mid-height, and the stabilizing effect of axial load and its eccentricity.

Dynamic testing began with the large-scale programme carried out by ABK Joint Venture [1981b]. In this study, 22 wall specimens with different overburden loads and height to thickness \((h/t)\) ratios were tested under dynamic loading. The tests were carried out using displacement-controlled actuators at both the top and bottom of the walls (Figure 2.2). The issue of diaphragm flexibility was addressed by estimating the input motions at the top and bottom of walls using a computer model that consisted of a non-linear shear-deformable beam representing the diaphragm, and lumped masses on the beam representing the out-of-plane walls. The calculated diaphragm response was then applied to the actuators. This design excluded the possibility of observing the effects of interaction between out-of-plane wall rocking and diaphragm flexibility. Cracking was observed near mid-height and at the base of walls, and it was noted that dynamically stable rocking was possible at relative mid-height displacements significantly in excess of those at crack initiation.
2.2. Out-of-plane wall behaviour

Figure 2.2: Test setup, ABK
2.2. Out-of-plane wall behaviour

Doherty [2000] carried out shake table testing of half-scale (1.5 m tall) 50 and 110 mm thick walls. These tests applied equal top and bottom input motions to the wall via a stiff frame on a shake table (Figure 2.3). Axial load was applied to the top of the wall by a series of springs. Notably, the upward displacement at the top of the wall resulting from out-of-plane rocking increased the applied axial force (arching action), and so a constant axial load could not be assumed at larger displacements with the thicker walls.

In commentary on these tests, Griffith et al. [2004] suggested that spectral displacements seem to be a more direct and convenient parameter to define seismic demand and capacity than accelerations. Furthermore, it was noted that for the rigid-support conditions in these tests, the static force-displacement relationship developed for the wall was a reasonable bound of the dynamic hysteretic behaviour. Testing also confirmed that wall rocking frequency and damping are both displacement dependent.

Simsir [2004] performed shake-table testing of an idealized URM building

Figure 2.3: Test setup, Doherty

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Simsir [2004] performed shake-table testing of an idealized URM building
to study the influence of diaphragm flexibility on out-of-plane URM wall performance. The test was conducted at half-scale using hollow concrete masonry units. A high axial load was applied, representing a bottom storey wall in a multi-storey building. In-plane walls were reinforced and connected to the top of the unreinforced out-of-plane walls using a flexible steel beam, representing a diaphragm (see schematic representation in Figure 2.4). The walls were observed to crack just above the base at very low input levels, and subsequently underwent rocking about this crack. Due to the high axial load, rocking about a mid-height crack was not observed until additional mass was added to one wall; the wall collapsed in the same run. Consequently, limited information on the dynamic rocking response of walls was obtained from these tests.

![Figure 2.4: Test setup, Simsir](image)

Meisl [2006] performed full-scale shake table testing of solid clay brick URM walls subjected to out-of-plane excitation, with approximately equal input motions at the top and bottom of the walls (Figure 2.5). Two different ground motions — one recorded on firm ground and one on soft ground, both during the 1989 Loma Prieta earthquake — were used as input motions. Rigid-body rocking about cracks above mid-height was observed for each specimen. Cracking occurred above mid-height in all walls. It was found that the quality of the collar joint (whether mostly voids or fully slushed) made minimal difference in the rocking response of the walls.
2.2. Out-of-plane wall behaviour

The tests provided an important dataset of full-scale rigid-body rocking to which an analytical model could be calibrated, within the limitations of rigid-diaphragm conditions.

![Diagram of test setup]

Figure 2.5: Test setup, Meisl

Dazio [2008] conducted shake table testing of 2.4 m tall walls with thickness between 125 and 200 mm, varying the boundary conditions including fixity, axial load, and axial load eccentricity. It was observed that the simply supported case with no overburden was not always the most vulnerable condition; small axial loads applied at large eccentricity in some cases resulted in lower stability levels than in walls without overburden. Considerable rocking displacement capacity was observed in tests without overburden, but it was noted that collapse occurred more suddenly with less rocking in walls that were axially loaded.

Derakhshan [2010] conducted quasi-static air bag testing of full-scale URM wall panels, including both panels built in the lab and in-situ testing of wall segments in vintage buildings. The in-situ testing demonstrated that arching action due to vintage timber diaphragms is negligible, and that the force-displacement characterization obtained in the lab was representative.
2.2. Out-of-plane wall behaviour

of in-situ conditions.

In testing of 5 m square single-storey full-scale masonry houses [Clough et al., 1990, Gülkan et al., 1990], it was observed that “typical single-storey masonry houses are so rigid that they do not develop very complicated dynamic response mechanisms during earthquakes”. Under strong excitation, out-of-plane walls were observed to crack near mid-height and underwent stable rocking about this crack. Notably, neither greater damage nor response limits were observed when applying three-axis table motions, compared with one- or two-axis motions.

Paquette and Bruneau [2003] conducted pseudo-dynamic testing of a 4 × 6 m full-scale, single-storey URM building with a flexible timber diaphragm. The base of the building was fixed, while a single actuator applied an earthquake motion to the roof diaphragm at mid-span. Relative diaphragm deformation was limited to 20 mm, and the diaphragm remained elastic. The test focused on in-plane performance, and out-of-plane wall rocking was not observed.

Three 4 × 6 m two-storey full-scale stone masonry houses with flexible timber diaphragms were subjected to shake table testing by Magenes et al. [2010, 2012]. One specimen was unretrofitted, the second was retrofitted by adding wall-diaphragm connections, but without stiffening the diaphragms, and the third was retrofitted including significant diaphragm stiffening. Both retrofitted specimens withstood significantly higher levels of shaking than the unretrofitted specimen. Comparisons between the two retrofitted buildings suggested that the observed improvement of the seismic performance was related more to the wall-diaphragm connections, and less so to the stiffening of the diaphragms.

2.2.2 Analysis methods

Uncracked URM walls behave elastically, and can be modelled accurately by simple methods. Given an assumed acceleration distribution, theoretical crack locations can be predicted by solving for the location of peak tensile stress. Derakhshan [2010] conducted a detailed analysis of wall cracking behaviour for varying overburden and tensile strength (typically governed by flexural bond strength). He found that for a uniform applied load, the predicted crack height changes appreciably only for very low values of tensile strength (less than 0.1 MPa), and is most sensitive under low overburden.

Approaches to modelling cracked walls can be grouped into three broad categories. The methods are summarized briefly here; Derakhshan [2010] provides a more detailed review.
2.2. Out-of-plane wall behaviour

Complex finite element models: The most detailed models involve the discretization of the wall into masonry units and mortar joints. Approaches have included a block-interface model [Martini, 1998], rigid bricks with cohesionless friction joints [Felice and Giannini, 2001], lumped masses with fibre-element mortar joints [Simsir, 2004], and continuum modelling (non-discrete) [Hamed and Rabinovitch, 2008]. While some of these methods have proven to be able to simulate the out-of-plane response of URM walls, including crack formation and rocking, they are invariably computationally demanding, and consequently ill-suited for use in a large-scale parametric study.

Stick models: Doherty et al. [2002] proposed a simplified representation of a cracked URM wall with rigid top and bottom supports as an equivalent single-degree-of-freedom (SDOF) model. A modal mass was calculated based on the assumed triangular displaced shape, and a triangular inertial force distribution was assumed. A [total force]–[displacement at crack] relationship was derived for rigid and deformable wall conditions (Figure 2.6). The deformable model, idealized as a tri-linear curve, included the effects of (1) elastic deformation prior to rocking and (2) finite dimensions of pivot points due to mortar strength limits. Viscous damping was empirically approximated by observing the decay of the free rocking motion. With mass, damping, and stiffness defined, non-linear analysis can be carried out using the model.

![Figure 2.6: Quasi-static response of SDOF wall model](image)

Linearized displacement-based procedures have been proposed based on these curves, involving the definition of a secant stiffness and assumed damping [Doherty et al., 2002, Priestly et al., 2007]. Simsir [2004] extended the SDOF model to two degrees of freedom by adding a flexible top diaphragm,
2.2. Out-of-plane wall behaviour

and Derakhshan [2010] extended it further to a two-storey model with flexible diaphragms.

**Rigid body models:** Makris and Konstantinidis [2003] demonstrated that the response of a rocking system is fundamentally different from that of a regular SDOF oscillator, and recommended that “the response of one should not be used to draw conclusion on the response of the other”. While the restoring force in an SDOF oscillator is due to the elasticity of the structure \((k)\), that in a rocking block is due to gravity. An oscillator has a unique period, while a rocking block does not. In addition, even non-linear models like the tri-linear model in Figure 2.6 do not capture the change in pivot point location with rocking direction reversal in a cracked wall. Instead, the effects of this location change and its associated impact and energy dissipation are approximated by viscous damping.

In rigid body modelling, like that used by Makris and Konstantinidis, bodies are represented with finite geometry. Consequently, changes in pivot point locations in a cracked wall can be accurately captured, and energy dissipation due to impacts is explicitly accounted for by a coefficient of restitution. A rocking wall can therefore be modelled more directly using this method, without the need for empirically-calibrated viscous damping values.

Konstantinidis and Makris [2005] validated the ability of commercially-available rigid body software Working Model 2D [Design Simulation Technologies, Inc., 2010] to simulate the pure sliding and pure rocking responses of a block as part of an investigation into the seismic performance of multi-drum columns. Meisl [2006] further demonstrated that the software could adequately simulate the rocking response of one-way spanning URM walls with rigid diaphragm boundary conditions.

The stick-model approaches use workarounds to approximate the rocking response of a cracked wall, while rigid body modelling represents the rocking response explicitly. The drawbacks of the rigid body approach are a lack of a simple representation of initial elastic wall stiffness and finite mortar strength. However, these issues are relatively minor compared with the benefit of explicit representation of rocking behaviour, and workarounds are possible to adjust contact point geometry to simulate finite mortar strength. Given that rigid body models are computationally efficient, and that commercially-available software makes configuration and setup relatively simple, there is not a strong driver towards using a stick model approach. Consequently, the rigid body approach will be used in this study (see Chapter 5).
2.2.3 Current assessment standards

The most widely used current standard for assessment of out-of-plane wall stability is ASCE 41 [ASCE, 2014], which is based on the recommendations of ABK Joint Venture [1984]. In ASCE 41, two options are presented for wall out-of-plane evaluation: (1) a simplified procedure, contained within the main body of the standard, and (2) a more detailed procedure, contained within a standalone special procedure for URM. The Canadian “Guidelines for Seismic Evaluation of Existing Buildings” [National Research Council of Canada, 1993] also contains a special procedure for URM in which the out-of-plane provisions are effectively equivalent to those in the special procedure in ASCE 41. Differences are limited to notation and hazard definition; this procedure will therefore not be reviewed separately. Application of either of the special procedures is restricted to buildings with flexible diaphragms at all levels above the base of the structure, a minimum of two lines of walls in each principal direction (except for single-storey buildings with an open front on one side), and a maximum of six stories above the base of the structure.

2.2.3.1 Rationale

Bruneau [1994] provides an overview of the rationale based on which this special procedure was developed. From their experimental data, ABK Joint Venture produced non-linear regression curves of \( h/t \) as a function of the overburden ratio (axial load divided by wall weight) and the square root sum of squares (SRSS) of top and bottom peak input velocities, for fixed ‘probabilities of survival’. It is critical to note that the data set on which the regressions were performed was very limited: only two ground motions were used at each hazard level. Recent work by Baker [2011] suggests that considerably more ground motions are required to adequately address motion-to-motion variability.

Maximum \( h/t \) values to satisfy a 98% probability of survival were established. Overburden ratios were taken as 0 for walls without stories above, and 0.5 for all other walls — these are conservative values. Input velocities at wall supports were taken as ground velocities amplified by the flexible diaphragms.

Amplification was assessed based on the diaphragm demand-to-capacity ratio (DCR). Note that this is a strength parameter only, and diaphragm stiffness is not explicitly accounted for in the procedure. As defined by ABK, the DCR is hazard-independent: the demand is calculated based on dynamic
2.2. Out-of-plane wall behaviour

loading of the tributary diaphragm weight multiplied by 1.0 g. This demand was assumed based on modelled diaphragm amplifications in the highest seismic hazard zone considered, with an effective peak acceleration (EPA) of 0.4 g (refer to the UBC 1994 [International Conference of Building Officials, 1994] regarding EPA). A maximum allowable DCR was incorporated, which increases with decreasing diaphragm span, reaching a maximum of 5.0 for diaphragms with a span of less than \( \sim 10 \text{ m} \) (see Figure 2.7). This DCR corresponded to a peak diaphragm deformation of \( \sim 125 \text{ mm} \) in experimental testing of diaphragms at the highest hazard level.

![Figure 2.7: ASCE 41 diaphragm DCR zones](image)

Lower amplification factors were used for ‘softer’ diaphragms (higher DCR), resulting in more stringent \( h/t \) limits for diaphragms with higher strength. The rationale here was that greater non-linear response in the diaphragm would result in lower amplification. These lower amplification factors were also allowed for long-span low-DCR diaphragms if adequate wood-framed cross walls spanning between diaphragms were present to provide added damping. Amplifications were varied depending on the dia-
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phragm DCR only at the highest hazard level; at the lower levels, amplification was assumed constant regardless of the diaphragm capacity.

2.2.3.2 Limits

Limits on \( h/t \) were derived in this way for three levels of seismic hazard, with a peak ground velocity of 0.3 m/s used for the highest hazard level. Where allowable \( h/t \) limits produced from the data were deemed too high, they were arbitrarily reduced. The recommended \( h/t \) limits were further tightened in some cases, and reformulated based on spectral acceleration. These final limits were then incorporated in various guidelines, including FEMA 273 [FEMA, 1997], forming an out-of-plane assessment procedure that went effectively unchanged into the FEMA 356 prestandard [FEMA, 2000] and the ASCE 41 standard [ASCE, 2014]. The procedure, as presented in the ASCE 41 special procedure and in the main document, is briefly outlined below.

Special procedure: Diaphragms are categorized into one of three possible zones based on their DCR and span (see Figure 2.7): (1) long-span diaphragms, (2) short-span diaphragms with high DCR, and (3) short-span diaphragms with low DCR. Here, diaphragms are considered long-span beginning at a span of 55 m — the majority of typical commercial URM buildings will therefore be classified as short-span. The DCR at the transition between (2) and (3) is between 2.5 and 2.8, depending on the diaphragm span. In the adaptation of the original rationale to form this procedure, the calculation of the DCR has been tied to the hazard: the demand is defined as \( 2.1 S_{X1} W_d \), where \( S_{X1} \) is the design spectral acceleration at 1 s (2/3 of the MCE value specified in US codes), adjusted for site class, and \( W_d \) is the weight tributary to the diaphragm. Here, the factor of 2.1 appears to be back-calculated to produce results roughly consistent with ABK’s original intentions for amplifications, which were only specified at a single hazard level.

Limits on \( h/t \) are presented at three discrete hazard levels and four wall location types, as shown in Table 2.2. Diaphragm classification affects the allowable \( h/t \) only at the highest hazard level, where distinction is made between values in columns A and B. The more lenient \( h/t \) limits in column A may be used with long-span diaphragms that meet minimum cross-wall requirements or with short-span, high-DCR diaphragms regardless of cross-walls. Other systems, including short-span, low-DCR diaphragms, must use the more stringent limits in column B. For evaluation of earthquake-damaged
walls, FEMA 306 [FEMA, 1998] stipulates damage-dependent $\lambda_{h/t}$ factors by which to multiply allowable $h/t$ values; the factors range between 0.6 for heavy damage to 1.0 for insignificant damage.

Table 2.2: ASCE 41 special procedure $h/t$ limits

<table>
<thead>
<tr>
<th>Wall Location</th>
<th>$0.13 \leq S_{X1} &lt; 0.25$</th>
<th>$0.25 \leq S_{X1} &lt; 0.40$</th>
<th>$S_{X1} \geq 0.40$</th>
</tr>
</thead>
<tbody>
<tr>
<td>One-storey</td>
<td>20</td>
<td>16</td>
<td>16(^{[c]}) 13</td>
</tr>
<tr>
<td>Top storey</td>
<td>14</td>
<td>14</td>
<td>14(^{[c]}) 9</td>
</tr>
<tr>
<td>First storey</td>
<td>20</td>
<td>18</td>
<td>16 15</td>
</tr>
<tr>
<td>All other walls</td>
<td>20</td>
<td>16</td>
<td>16 13</td>
</tr>
</tbody>
</table>

\(^{[a]}\) For long-span diaphragms (zone 1 in Figure 2.7) meeting minimum cross-wall requirements, or for short-span, high-DCR diaphragms (zone 2) regardless of crosswalls

\(^{[b]}\) For systems not meeting requirements in \(^{[a]}\)

\(^{[c]}\) Minimum in-plane shear requirements apply to use these values

**Simplified procedure:** As an alternative to the special procedure, a simplified procedure not requiring diaphragm assessment is presented as part of the main body of ASCE 41. Here, the assessment procedure is divided into three performance levels: immediate occupancy (IO), life safety (LS), and collapse prevention (CP). At the IO level, wall flexural cracking must be prevented. This is evaluated based on the masonry tensile strength and the seismic demands. Both the LS and CP performance levels allow cracking, but require that the walls remain dynamically stable. Stability must either be evaluated by an analytical time-step integration model, or else the walls must meet the specified $h/t$ limits (Figure 2.8). The $h/t$ limits in the simplified procedure are the same as those in the special procedure using the more stringent column B requirements for all buildings, with slightly different breakpoints in $S_{X1}$. These simplified limits will be used as a base case for comparison of results in this dissertation. Note that the requirements for ‘all other walls’ are the same as for one-storey walls.

**2.2.3.3 Discussion**

Where applicable, the following discussion refers to both the special and simplified procedures. First, while there are maximum permissible $h/t$ ratios, there is no upper limit on the hazard level. This is a remnant of seismic hazard being classified in discrete zones at the time of the work of ABK
2.2. Out-of-plane wall behaviour

Joint Venture, and is not reflective of the current state of hazard analysis. The fact that the same outcome is obtained from assessment of a given wall at $S_{X_1} = 0.40\, g$ as at $S_{X_1} = 0.60\, g$ or higher may be cause for concern. Characterization of wall stability with respect to seismic hazard is a major objective of this dissertation, and this topic is investigated in Chapter 6. In addition, in the simplified procedure, no guidance is provided on modelling wall stability, which effectively prevents that option from being exercised in practice.

Furthermore, the procedure accounts for important parameters only very coarsely, implicitly, or not at all. For example, axial load can vary significantly among top-storey walls, depending on the size of parapets and whether roof joists or trusses are bearing on the wall or running parallel to it, but the procedure does not distinguish between these cases. Diaphragm flexibility could vary among diaphragms with the same DCR, but this effect is disregarded. In addition, the procedure assumes that wall stability is independent of scale effects, accounting only for the $h/t$ ratio, but not for variation in wall thickness.

Finally, the data set on which the ABK Joint Venture analysis was based was very limited. Ground motion variability was not accounted for, and the effects of parameters other than those varied in the experimental work were not considered. Recommended limits on $h/t$ were arbitrarily reduced at var-

Figure 2.8: ASCE 41 $h/t$ limits
ious stages prior to incorporation in assessment guidelines, with inadequate documentation.

Aside from hazard definition, no revisions have been made to the out-of-plane portion of the procedure since its inception, despite significant research progress having been made since ABK Joint Venture [1984]. The limitations of the original research and the following adaptations of its recommendations create great uncertainty about the risk levels produced by the current standard. New research and modern computational power offer the ability to re-examine the out-of-plane response of URM walls far more thoroughly than before, with the potential to produce a new assessment procedure that better defines and controls the associated seismic risks.
Chapter 3

Experimental Program

3.1 Introduction

Full-scale shake table tests were carried out on URM wall specimens using a testing apparatus which allows for the simulation of flexible diaphragm boundary conditions. Five wall specimens were tested. This chapter describes the test apparatus, wall specimens, data collection and testing protocols. More details are provided in Appendix C.

3.2 Wall specimens

Five wall specimens were constructed by professional masons in the Earthquake Engineering Research Facility (EERF) at the University of British Columbia (UBC). Specimens were intended to represent a portion of a top-storey wall in an early 1900s load-bearing URM building in British Columbia (see Section 2.1). Additionally, they were to be generally consistent with those tested by Meisl [2006], which were representative of an early 1900s school building in British Columbia.

Wall dimensions are listed in Table 3.1 and typical dimensions illustrated in Figure 3.1. Detailed measurements of each specimen are provided in Appendix A. Four 3-wythe walls and one 2-wythe wall were constructed (Figures 3.2–3.6). American bond was used in all walls with a single header course at every sixth course. Specimens are named by diaphragm condition ([F]lexible, [S]tiff, or [R]igid at top and bottom) and the number of wythes in the wall specimen. The 3-wythe walls are slightly shorter and thinner than the walls tested by Meisl, which were 4.2 m tall, 0.33 m thick, and 1.5 m long.

Mortar was mixed on site by the masons in an electric mixer. To represent the deterioration of the mortar in existing buildings, a Type O mortar mix (1:2:9 cement:lime:sand by volume) was selected due to its low compressive strength. Brick units were solid clay and measured 64 mm × 89 mm × 191 mm. Brick units were placed dry to further minimize the bond strength.
3.2. Wall specimens

Table 3.1: Wall geometry

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Thickness (mm)</th>
<th>Length (mm)</th>
<th>Height (mm)</th>
<th>$h/t$</th>
<th>Mass (kg)</th>
<th>Density (kg/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF-3</td>
<td>291</td>
<td>1509</td>
<td>3947</td>
<td>13.6</td>
<td>3627</td>
<td>2095</td>
</tr>
<tr>
<td>FR-3</td>
<td>291</td>
<td>1500</td>
<td>3984</td>
<td>13.7</td>
<td>3614</td>
<td>2081</td>
</tr>
<tr>
<td>FF-2</td>
<td>191</td>
<td>1504</td>
<td>2790</td>
<td>14.6</td>
<td>1739</td>
<td>2176</td>
</tr>
<tr>
<td>SS-3</td>
<td>300</td>
<td>1518</td>
<td>3985</td>
<td>13.3</td>
<td>3833</td>
<td>2113</td>
</tr>
<tr>
<td>RR-3</td>
<td>296</td>
<td>1513</td>
<td>3973</td>
<td>13.4</td>
<td>3768</td>
<td>2118</td>
</tr>
</tbody>
</table>

Figure 3.1: Wall geometry
3.2. Wall specimens

While the workmanship in older buildings may be variable, this factor can be difficult to quantify. To achieve reasonable consistency among specimens in this regard, it was therefore decided to employ good construction practices - bricks were precisely placed and collar joints were slushed in all specimens. Meisl [2006] investigated the effect of varying the quality of the collar joints and found that it had no significant effect on the out-of-plane response of walls.

The walls were constructed and cured under dry conditions in the laboratory. The total construction time for any single wall varied between 5 and 7 days. The walls were built in two phases: walls FF-3, FR-3 and FF-2 were built simultaneously and then tested. Walls SS-3 and RR-3 were built after testing of the first three walls was complete. The age of the walls at testing varied between approximately 2 and 10 months (Table 3.2).

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Date completed</th>
<th>Date tested</th>
<th>Age at testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF-3</td>
<td>2011-06-21</td>
<td>2012-02-27</td>
<td>251</td>
</tr>
<tr>
<td>FR-3</td>
<td>2012-04-04</td>
<td>2012-04-04</td>
<td>288</td>
</tr>
<tr>
<td>FF-2</td>
<td>2012-04-26</td>
<td>2012-04-26</td>
<td>310</td>
</tr>
<tr>
<td>RR-3</td>
<td>2012-04-24</td>
<td>2012-06-13</td>
<td>50</td>
</tr>
<tr>
<td>SS-3</td>
<td>2012-04-24</td>
<td>2012-07-11</td>
<td>78</td>
</tr>
</tbody>
</table>

Each wall was built on the web surface of a steel wide-flange section. To move the wall from the place of construction onto the test apparatus, a steel lifting beam was placed on top of the wall and a threaded rod was installed between the lifting beam and the base beam at each corner. A piece of plywood was placed between the top of the wall and the lifting beam to absorb surface irregularities. The threaded rods were tightened, placing compressive stress in the wall. The wall could then be lifted using the lab’s bridge crane. The wall was first weighed using a load cell placed between the lifting beam and the crane hook (Figure 3.7). It was then moved into position on the testing apparatus (Figure 3.8). No cracking or other damage occurred during movement of the walls.
3.2. Wall specimens

Figure 3.2: Two-wythe header course

Figure 3.3: Construction of three-wythe header course
3.2 Wall specimens

Figure 3.4: Three-wythe header course after slushing of joints

Figure 3.5: Pointing the joints
3.2. Wall specimens

Figure 3.6: Completed walls in the EERF (shake table in foreground)
3.2. Wall specimens

Figure 3.7: Weighing a wall
Figure 3.8: Lifting a wall into the test frame
3.2. Wall specimens

3.2.1 Material tests

Material samples were made during construction of the walls. Since the purpose of the materials testing was to estimate the as-tested properties of the walls, the samples were cured in the laboratory under the same conditions to which the walls were subject. Masonry prims were cured dry (exposed), while mortar cubes were cured in a sealed bag for the first 7 days, then exposed to cure dry. Materials testing was carried out for each of the two phases: after shake table testing of walls FF-3, FR-3 and FF-2, and after shake table testing of walls SS-3 and RR-3. Testing procedures specified by the indicated standards were followed with the exception of the curing conditions and curing times noted above. Test results are summarized in Tables 3.3–3.5.

Table 3.3: Mortar properties

<table>
<thead>
<tr>
<th>Walls</th>
<th>Compressive strength $f'_{j}$ (MPa)</th>
<th>$c_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF-3</td>
<td>4.0</td>
<td>0.23</td>
</tr>
<tr>
<td>FR-3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FF-2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS-3</td>
<td>4.2</td>
<td>0.35</td>
</tr>
<tr>
<td>RR-3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$[a]$ CAN/CSA A179-04 (R2009) [CSA, 2009]

Table 3.4: Brick properties

<table>
<thead>
<tr>
<th>Bricks</th>
<th>Compressive strength $f'_{b}$ (MPa)</th>
<th>Absorption $c_v$</th>
<th>24-hr soak $c_v$ (%)</th>
<th>5-hr boil $c_v$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type A</td>
<td>119</td>
<td>0.06</td>
<td>5.1</td>
<td>7.0</td>
</tr>
<tr>
<td>Type B</td>
<td>157</td>
<td>0.07</td>
<td>4.8</td>
<td>6.6</td>
</tr>
</tbody>
</table>

$[a]$ CAN/CSA A82-06 (R2011) [CSA, 2011]

Two sub-varieties of bricks with slightly differing appearance and properties were used in the walls; type A bricks were more porous and had

39
3.3. Experimental set-up

Table 3.5: Masonry properties

<table>
<thead>
<tr>
<th>Walls</th>
<th>Compressive strength [a] (f_m') (MPa)</th>
<th>CV</th>
<th>Elastic modulus (E) (MPa)</th>
<th>CV</th>
<th>Flexural strength [b] (f_{fb}') (MPa)</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF-3</td>
<td>33</td>
<td>0.20</td>
<td>6.4×10^3</td>
<td>0.40</td>
<td>0.38</td>
<td>0.42</td>
</tr>
<tr>
<td>FR-3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FF-2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS-3</td>
<td>46</td>
<td>0.13</td>
<td>10.0×10^3</td>
<td>0.21</td>
<td>0.55</td>
<td>0.31</td>
</tr>
<tr>
<td>RR-3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

[a] ASTM C1314 - 11a [ASTM, 2011]
[b] ASTM C1072 - 10 [ASTM, 2010]

deep colouring than type B bricks, which had a higher strength. The two brick types were mixed in the construction of the walls. Both bricks were of extraordinarily high strength - the stronger type B bricks had a mean compressive strength of 157 MPa, based on the gross area of half-bricks with the load applied against the bedding faces. The bricks were for refractory use, and were selected due to a lack of availability of solid regular face bricks. The high brick strength also produced a correspondingly high masonry compression strength despite relatively weak mortar being used.

In out-of-plane behaviour of URM walls, the flexural bond strength (tested values in Table 3.5) is the primary parameter influencing cracking load, and brick strength is expected to have minimal impact on out-of-plane response. The greatest expected difference, should regular-strength bricks have been used, would be greater spalling of the bricks at the crack.

Batching and mixing of mortar on-site by the masons produced notable batch-to-batch variability in properties. Typically three to four batches were mixed in a day; six mortar cubes were made from each of three batches daily, and eight masonry prisms were made from one batch daily. Note that the low target strength of the Type O mix exaggerates the magnitude of \(c_v\) for the mortar compression strength. Further details can be found in Appendix B.

3.3 Experimental set-up

The walls were tested on the single degree of freedom shake table in the EERF in a purpose-built test frame. The following sections detail the configuration of the experimental set-up.
3.3. Experimental set-up

3.3.1 Shake table

The shake table (Figure 3.6) consists of a steel frame and steel top sheet approximately 3 m by 4 m in plan. The top sheet was removed from the table for the duration of the testing to reduce force demands on the actuator. The table is supported on four double v-groove casters running on steel angle tracks and is not restrained against uplift. The test frame was designed to minimize overturning demands and the table was monitored for uplift in the initial tests; no uplift was detected.

The table is controlled by a single hydraulic actuator housed in an opening in the centre of the shake table frame. The actuator has a maximum displacement of ±457 mm and a static capacity of 298 kN. The hydraulic system is powered by a 5.7 L/s pump supplying 20.7 MPa, with a 454 L accumulator. The table is displacement controlled; hydraulic pressure is controlled by an MTS 458 servo-controller. The displacement waveform is input to a PC, where it is converted to a voltage signal by the software DasyLab. This command signal is sent to the servo-controller, which then sends a signal to a MOOG hydraulic proportional servo valve, thereby regulating the hydraulic pressure. The table position is fed back to the servo-controller by an analog MTS Temposonic displacement transducer.

3.3.2 Test frame

The test frame imposed a simplified representation of flexible diaphragm boundary conditions on the wall. The simplifications that were made were a function of construction and testing simplicity, in addition to ensuring that the test results could be modelled accurately in the analytical portion of the study. A description of pertinent features is provided in this section, and more details can be found in the sketches in Appendix C.

An overview photo of the test frame is shown in Figure 3.9. A 3-d rendering is provided in Figure 3.10, illustrating the various components by varying colours. A stiff steel braced frame, representing the in-plane walls, was constructed on the shake table. The table motion was transferred to the top of this frame with minimal amplification (see Section 3.7.1). The inclusion of in-plane wall flexibility was outside the scope of the experimental work. It was instead assumed that the flexibility of the in-plane walls could be considered negligible compared to that of the flexible timber diaphragms. Top and bottom diaphragms were represented by rolling steel carriages connected to the frame by coil springs. The carriages were able to roll parallel to the direction of motion of the shake table. Spring stiffnesses were selected
3.3. Experimental set-up

to achieve natural periods of vibration in the test setup representative of first-mode in-plane behaviour of typical diaphragm-wall assemblies (refer to Section 2.1.2.2). The top and bottom of the wall were connected to the respective carriages (Figures 3.11–3.12).

Each carriage could be ‘locked out’ to simulate a rigid diaphragm condition. This was achieved by connecting the carriage to the braced frame with \( \varnothing 19 \) mm threaded steel rods. Four rods were used for each carriage.
3.3. Experimental set-up

Figure 3.10: Model depiction of test apparatus
3.3. Experimental set-up

Figure 3.11: Bottom of test frame

(one on each corner) and all rods were pre-tensioned to prevent any ‘slop’ in this connection.

Overturning of the top carriage was prevented by a hold-down assembly consisting of a steel channel spanning across the top carriage, with a pair of angles welded to each end. The lower ends of the angles were bolted to the top frame. A pair of polyurethane-coated steel casters were bolted underneath the channel in line with the longitudinal beams of the top carriage. Each beam was also supported underneath by 2 v-groove casters mounted on the top frame near the ends of the carriage, thus creating a 3-point constraint and preventing overturning. The bottom carriage did not require an overturning constraint, since the weight of the wall specimen in the middle of the carriage provided sufficient overturning resistance.

Coil springs were custom-fabricated by a local supplier\(^2\). A total of four spring assemblies were used (two for each carriage). Each assembly (Figure 3.13) consisted of a base housing, a shaft, two springs, end caps, and a carriage connection plate. The base housing was made of a square steel hollow structural section (HSS) roughly 450 mm long, with 25 mm plates

\(^2\)Dendoff Springs Ltd., Surrey, British Columbia
3.3. Experimental set-up

Figure 3.12: Top of test frame
3.3. Experimental set-up

welded into each end of the HSS. The plates were bored to accommodate nylon bushings. The shaft, a 73 mm round HSS, slid through these bushings. The coil springs were designed to fit over the shaft; one spring was placed on each side of the base housing. The springs were then compressed to roughly one-half of their maximum displacement and end caps were threaded onto the shaft. Finally, a carriage connection plate (not shown in Figure 3.13) was threaded on one end of the shaft. This plate had two slotted holes which mated with holes in each carriage, and allowed for adjustment of the carriage ‘neutral’ position prior to connecting the spring assemblies. The base housings of the spring assemblies were bolted to the braced frame. Pre-loading the springs to half-capacity resulted in a system with no ‘slop’ when transitioning from positive to negative displacement. The shafts and springs were greased, resulting in very little resistance to motion.

Figure 3.13: Spring assembly

Force-displacement response of each spring strut assembly was measured by bolting the assembly to a Tinius-Olsen screw-drive compression testing machine and running the assembly through nearly the full design range of motion. All assemblies showed a force-displacement response that was nearly perfectly linear over the entire measured range. Key parameters are shown in Table 3.6. Mean and coefficient of variation are shown for each pair of assemblies (north and south sides).

The base of the wall was supported by the bottom carriage, and the wall base beam was bolted to the carriage. The connection between the wall and the base beam is shown in Figure 3.14. A strip of ultra-high molecular weight polyethylene (UHMW) was fastened to each side of the wall at the base using masonry screws. A steel bar with a stiff rubber spacer, also lined with a UHMW strip, was snug-tightened against the wall on each side using bolts in the flanges of the base beam. A shorter steel bar was then placed across these wall restraint bars at each end of the wall and welded to both the longer bars and the flanges of the base beam, thus fixing the wall restraint bars in place relative to the wall base beam. The contact surfaces between the two pieces of UHMW were coated with grease to reduce friction. This
### 3.3. Experimental set-up

Table 3.6: Spring test data (per strut assembly)

<table>
<thead>
<tr>
<th>System</th>
<th>Location</th>
<th>Travel (mm)</th>
<th>Spring rate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Design maximum</td>
<td>Tested to</td>
</tr>
<tr>
<td>Flexible</td>
<td>Top</td>
<td>460</td>
<td>430</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stiff</td>
<td>Top</td>
<td>190</td>
<td>180</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

connection effectively restrained the lateral displacement of the base of the wall relative to the carriage, but allowed the base of the wall to rotate and lift up with minimal resistance.

A steel channel assembly was placed on the top of the wall (Figure 3.15). The channel’s long axis was oriented parallel to the long edge of the wall, and three plates were welded underneath perpendicular to the channel. Slotted holes were cut in each end of the plates, and heavy angles were bolted to the plates from below. The inside faces of the angles were lined with rubber to absorb surface irregularities. The angles were clamped tight against the wall and then the bolts were tightened, thereby securing the channel assembly to the wall. A $64\text{mm}$ steel pin was inserted into a bored rectangular block bolted to each end of this channel, with the pin protruding beyond the edge of the wall by approximately $200\text{mm}$.

A steel plate with a milled vertical slot was bolted underneath the top carriage on each side, as shown in Figure 3.15. The pins at the top of the wall travelled within the vertical slots on these carriage plates, allowing the top of the wall free rotation and vertical displacement while restraining the lateral displacement of the top of the wall relative to the carriage. The slot was machined approximately $0.03\text{mm}$ wider than the diameter of the pins to allow for imperfections in assembly and to prevent binding of the system. The bearing interface was thoroughly greased before each test. To minimize the risk of damage to the interface, the pins were machined from re-purposed high-strength machine shafts and the slotted plates were milled of ASTM A514 steel. No damage or alignment error was observed during the testing.

No provisions were made in the test frame for the application of overburden load to the wall beyond that imposed by the steel channel assembly bolted to the top of the wall (approximately $179\text{kg}$). The simulations instead concentrated on the worst-case stability conditions found in upper-storey
3.3. Experimental set-up

(a) prior to restraint installation

(b) with restraint in place; unsecured

(c) with restraint in place and secured

Figure 3.14: Detail of bottom connection
3.3. Experimental set-up

Figure 3.15: Detail of top connection
walls with minimal tributary gravity loads.

Testing was carried out until collapse of the walls was achieved. In an effort to minimize damage to the test frame components and the instrumentation, nylon webbing was installed on either side of the wall to catch the falling wall segments after collapse (see Figure 3.9). The channel assembly that was clamped to the top of the wall was tethered to the top carriage, also using nylon webbing. Timber cover assemblies were built for the bottom carriage, the lower test frame, and the bottom spring assemblies to block falling debris. All of these measures were installed so as not to interfere with the response of any of the components during the testing.

3.4 Instrumentation and data collection

Data were recorded from 33 instrumentation channels measuring the response of the shake table, test frame, carriages, and the wall specimen. Instrumentation consisted of displacement transducers and accelerometers. A summary of the instrumentation setup is provided in this section; more details can be found in Appendix D.

3.4.1 Wall instrumentation

One displacement transducer and one accelerometer were mounted at each header course on the walls. IC Sensors (ICS) model 3028 piezoresistive silicon accelerometers with an output range of $\pm 10$ g were used. The accelerometers were housed in steel boxes, which were screwed to plywood mounts glued to the wall. String potentiometers were used to measure displacements, and consisted of Celesco models PT101 and 632036 units. The instruments were mounted on a purpose-built timber instrumentation frame mounted on the east wall of the building, and the strings were attached to hooks mounted on the walls. These units were thus measuring total displacement of the wall (including the shake table displacement).

In addition to the header-course instrumentation, a tri-directional accelerometer, consisting of three ICS model 3026 units ($\pm 5$ g), was mounted on the steel channel assembly at the top of the wall. Two linear voltage potentiometers (Novotechnik model TR100) were installed at the base of the wall on the south side to measure uplift of the edges of the wall relative to the bottom carriage. Two of the same units measured vertical displacement of the pins at the top of the wall relative to the top carriage.
3.4. Instrumentation and data collection

The potentiometers at the base of the wall were removed for high-intensity tests to avoid damage during collapse of the wall. All other instrumentation remained in place and active for all runs.

3.4.2 Shake table and test frame instrumentation

Shake table displacement was measured by an MTS Temposonics 6 analog transducer (model LPRCVU03601). Shake table acceleration was measured by an ICS model 3026 accelerometer (±10 g).

Top and bottom carriage accelerations were measured by ICS model 3028 accelerometers (±10 g). Carriage displacements were measured relative to the test frame (not including shake table displacements) using Celesco model SP1-50 string potentiometers.

The response of the top of the test frame was recorded to compare with the shake table response. An ICS 3028 accelerometer (±10 g) and a Celesco SP1-50 string potentiometer (mounted on the instrumentation frame) were used.

3.4.3 Data collection and processing

A computer-based data acquisition system using a National Instrument 16-bit PCI 6052E multi-function board and an SCXI signal conditioning chassis with a SCXI 1520 module was used. This module has a programmable filter, which was set as a (1 kHz) low pass, 4th order Butterworth filter, and was used for all instrumentation. The software DasyLab was used to acquire, control and store the data. Data were sampled at 200 Hz (Δt = 0.005 s).

High-frequency noise was removed from all data using zero-phase digital filtering with the built-in Matlab \texttt{filtfilt} function, using a 4th-order Butterworth filter. This function carries out two filtering passes — forwards and backwards — which in this case produces effectively an 8th-order filter. Only low-pass filtering was applied, with the cutoff frequency set at 25 Hz.

3.4.4 Video recording

Regular video (1280 × 720 resolution at 30 fps) was recorded from four angles during dynamic testing. Wide angle shots from floor level and from an elevated position, plus close-up shots of the top and base of the wall were made.

In addition, high-speed video was recorded using a Phantom v4.2 camera. The combination of resolution, recording time and frame rate of which the camera is capable are limited by its 2 GB internal memory; 100 fps and
the maximum resolution of $512 \times 512$ were selected, providing a recording window of roughly 80 s. The high-speed video was shot normal to the south end of the wall, lined up with the centre of the wall, from an elevation of roughly 2 m.

### 3.5 Ground motions

Two ground motions were used as input to the shake table, with one motion selected for significant long-period spectral response and the other for a dominant short-period spectral response. The long-period motion selected (CHHC1) was recorded during the 22 February 2011 earthquake in Christchurch, New Zealand at the Christchurch Hospital. The short-period motion selected (NGA0763) was recorded during the 18 October 1989 Loma Prieta earthquake at the Gavilan College in Gilroy, California. Acceleration response spectra as recorded on the shake table are shown in Figure 3.16 along with design spectra for Seattle, WA, USA and Victoria, BC, Canada. Displacement and acceleration time histories of the two motions as recorded on the shake table are shown in Figures 3.17 and 3.18, respectively. Scale factors are shown relative to the original motion as recorded during the earthquake, and reference the amplitude of the displacement time history.

It can be observed that the displacement control of the shake table results in significant response amplification at the natural frequency of the hydraulic system, producing a large response peak at a period of about 0.10 to 0.15 seconds. The effect of this amplification may be notable for runs in which the carriages were ‘locked out’; however, for runs in which the carriages were driven through the springs, this amplification was filtered out due to the much longer natural period of the spring-carriage-wall system.
3.5. Ground motions

Figure 3.16: Response spectra of recorded table motions (5% damped elastic)

Figure 3.17: Table displacement time history
3.6 Shake table tests

The mortar used in the construction of the test walls (Type O) is of significantly lower strength than that used in modern structural masonry. However, in particular the flexural bond strength of walls found in early 1900s buildings may be weaker still than that of the test walls. It was therefore decided not to rely on the cracking resistance of the test walls in assessing their dynamic stability on the shake table, but rather to assume that the walls would experience cracking at very low levels of excitation.

Initial runs for each uncracked wall were made with the desired diaphragm conditions and the CHHC1 motion, to verify correct operation of the apparatus. To ensure that walls would remain stable after crack initiation, allowing further tests to be carried out, cracking was initiated by running the NGA0763 motion with both top and bottom carriages locked out (rigid diaphragm conditions). After cracking was achieved, the carriage connections were adjusted for the desired diaphragm conditions, and subsequent runs were made using the CHHC1 motion at increasing amplitude of input motion until collapse was observed. Shake table tests are summarized in Tables 3.7–3.11.
3.6. Shake table tests

Table 3.7: Test protocol - wall FF-3

<table>
<thead>
<tr>
<th>Run</th>
<th>Motion</th>
<th>Scale</th>
<th>Diaphragm state</th>
<th>Recorded table response</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>10%</td>
<td></td>
<td>Top</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>30%</td>
<td>Flexible</td>
<td>0.05</td>
</tr>
<tr>
<td>3</td>
<td>CHHC1</td>
<td>50%</td>
<td>Flexible</td>
<td>0.19</td>
</tr>
<tr>
<td>4</td>
<td>CHHC1</td>
<td>70%</td>
<td>Flexible</td>
<td>0.28</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>80%</td>
<td>Flexible</td>
<td>0.42</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>100%</td>
<td>Flexible</td>
<td>0.52</td>
</tr>
<tr>
<td>7</td>
<td>NGA0763</td>
<td>50%</td>
<td>Rigid</td>
<td>0.64</td>
</tr>
<tr>
<td>8</td>
<td>NGA0763</td>
<td>60%</td>
<td>Rigid</td>
<td>0.23</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>30%</td>
<td>Flexible</td>
<td>0.18</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>50%</td>
<td>Flexible</td>
<td>0.28</td>
</tr>
<tr>
<td>11</td>
<td>CHHC1</td>
<td>70%</td>
<td>Flexible</td>
<td>0.44</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>80%</td>
<td>Flexible</td>
<td>0.49</td>
</tr>
<tr>
<td>13</td>
<td></td>
<td>100%</td>
<td>Flexible</td>
<td>0.64</td>
</tr>
</tbody>
</table>

Table 3.8: Test protocol - wall FR-3

<table>
<thead>
<tr>
<th>Run</th>
<th>Motion</th>
<th>Scale</th>
<th>Diaphragm state</th>
<th>Recorded table response</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>CHHC1</td>
<td>50%</td>
<td>Flexible</td>
<td>Top</td>
</tr>
<tr>
<td>2</td>
<td>CHHC1</td>
<td>70%</td>
<td>Flexible</td>
<td>0.43</td>
</tr>
<tr>
<td>3</td>
<td>NGA0763</td>
<td>60%</td>
<td>Rigid</td>
<td>0.28</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>50%</td>
<td>Rigid</td>
<td>0.29</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>70%</td>
<td>Rigid</td>
<td>0.43</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>80%</td>
<td>Rigid</td>
<td>0.41</td>
</tr>
<tr>
<td>7</td>
<td>CHHC1</td>
<td>90%</td>
<td>Flexible</td>
<td>0.45</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>100%</td>
<td>Rigid</td>
<td>0.53</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>110%</td>
<td>Rigid</td>
<td>0.49</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>120%</td>
<td>Rigid</td>
<td>0.70</td>
</tr>
</tbody>
</table>
### 3.6. *Shake table tests*

Table 3.9: Test protocol - wall *FF-2*

<table>
<thead>
<tr>
<th>Run</th>
<th>Motion</th>
<th>Scale</th>
<th>Diaphragm state</th>
<th>Recorded table response</th>
</tr>
</thead>
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<tr>
<td></td>
<td></td>
<td></td>
<td>Top</td>
<td>Bottom</td>
</tr>
<tr>
<td>1</td>
<td>CHHC1</td>
<td>50%</td>
<td>flexible</td>
<td>flexible</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>80%</td>
<td>flexible</td>
<td>flexible</td>
</tr>
<tr>
<td>3</td>
<td>NGA0763</td>
<td>60%</td>
<td>rigid</td>
<td>rigid</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>70%</td>
<td>rigid</td>
<td>rigid</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>50%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>70%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>80%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>CHHC1</td>
<td>90%</td>
<td>flexible</td>
<td>flexible</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>100%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>110%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
<td>120%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.10: Test protocol - wall *SS-3*

<table>
<thead>
<tr>
<th>Run</th>
<th>Motion</th>
<th>Scale</th>
<th>Diaphragm state</th>
<th>Recorded table response</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Top</td>
<td>Bottom</td>
</tr>
<tr>
<td>1</td>
<td>CHHC1</td>
<td>30%</td>
<td>stiff</td>
<td>stiff</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>40%</td>
<td>stiff</td>
<td>stiff</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>50%</td>
<td>stiff</td>
<td>stiff</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>60%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>NGA0763</td>
<td>50%</td>
<td>rigid</td>
<td>rigid</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>60%</td>
<td>rigid</td>
<td>rigid</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>30%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>50%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>60%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>CHHC1</td>
<td>65%</td>
<td>stiff</td>
<td>stiff</td>
</tr>
<tr>
<td>11</td>
<td></td>
<td>70%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>75%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td></td>
<td>80%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.7. Performance of test apparatus

Table 3.11: Test protocol - wall RR-3

<table>
<thead>
<tr>
<th>Run</th>
<th>Motion</th>
<th>Scale</th>
<th>Diaphragm state</th>
<th>Recorded table response</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Top</td>
<td>Bottom</td>
<td>PGA (g)</td>
</tr>
<tr>
<td>1</td>
<td>NGA0763</td>
<td>50%</td>
<td>rigid</td>
<td>0.25</td>
</tr>
<tr>
<td>2</td>
<td>60%</td>
<td>rigid</td>
<td>rigid</td>
<td>0.28</td>
</tr>
<tr>
<td>3</td>
<td>70%</td>
<td>rigid</td>
<td>rigid</td>
<td>0.47</td>
</tr>
<tr>
<td>4</td>
<td>CHHC1</td>
<td>50%</td>
<td>rigid</td>
<td>0.43</td>
</tr>
<tr>
<td>5</td>
<td>60%</td>
<td>rigid</td>
<td>rigid</td>
<td>0.53</td>
</tr>
<tr>
<td>6</td>
<td>55%</td>
<td>rigid</td>
<td>rigid</td>
<td>0.44</td>
</tr>
<tr>
<td>7</td>
<td>65%</td>
<td>rigid</td>
<td>rigid</td>
<td>0.77</td>
</tr>
</tbody>
</table>

3.7. Performance of test apparatus

Two aspects of the performance of the test frame are briefly examined in this section: the lateral stiffness of the frame in the in-plane direction, and the response of the spring-carriage systems.

3.7.1 In-plane response of test frame

Accelerations were measured on the shake table as well as at the top of the test frame during all testing. Run six from wall FF-3 was used as a case study to examine the frame response. This run was with an uncracked wall running the CHHC1 motion at 100% scale. A power spectral density plot of the table acceleration is shown in Figure 3.19a. This plot shows that the energy content of the table response is concentrated at frequencies lower than roughly 12 Hz.

The transmissibility of the test frame can be defined as the ratio of the absolute values of the Fourier transform of the acceleration at the top of the test frame divided by that of the table acceleration. This transmissibility function is plotted as a function of the frequency in Figure 3.19b. Here it can be observed that in the range of significant energy content (f < 12 Hz), the transmissibility is very close to 1. This indicates that the majority of the input motion energy content is transferred to the top of the frame with minimal amplification, meaning also that the natural frequency of the test frame under those test conditions is greater than 12 Hz. A rudimentary interpretation of the transmissibility plot suggests that the fundamental
3.7. Performance of test apparatus

Figure 3.19: Power spectral density of table acceleration
3.7. Performance of test apparatus

The frequency of the test frame is likely around 21 Hz. While the transmissibility at this frequency is high (> 10), the energy content of the input is very low.

A two-second portion of the acceleration time histories at the top of the test frame and at the shake table is shown in Figure 3.20. This plot confirms that there is negligible phase lag between the two locations, and that amplification is minor. Consequently, the response of the test frame is expected to have minimal effect on the response of the wall, and it is reasonable to simplify the system as having equal top and bottom inputs.

![Figure 3.20: Time history of acceleration at table and top of test frame](image)

3.7.2 Response of springs

The static response of the springs as measured prior to installation on the test frame was nearly perfectly linear. The response of the assembled wall-carriage-spring system in the test frame was also close to linear when excited at large displacement amplitudes, but exhibited some ‘stiction’ at low levels of excitation. The response characteristics of the system are examined using the first six tests of wall FF-3. In these tests the wall was uncracked, which resulted in in-phase top and bottom carriage displacements; the entire system is thus analogous to an SDOF oscillator. Maximum carriage displacements from these runs are shown in Table 3.12 and Figure 3.21.

The additional resistance in the system evident at low motion amplitudes could be attributed to several factors, including misalignment between each pair of spring shafts connected to a carriage, misalignment between the spring shafts and the carriage rails, and/or rolling resistance of the casters.
3.7. Performance of test apparatus

on the carriage. This non-linearity in the spring-carriage systems should be of minimal importance, particularly for collapse and near-collapse tests, which occur at moderate to high amplitudes.

Table 3.12: Peak carriage displacements, wall FF-3, uncracked

<table>
<thead>
<tr>
<th>Test</th>
<th>Motion scale</th>
<th>Top</th>
<th>Bottom</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10%</td>
<td>0.7</td>
<td>0.5</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>30%</td>
<td>18.2</td>
<td>17.0</td>
<td>18</td>
</tr>
<tr>
<td>3</td>
<td>50%</td>
<td>69.9</td>
<td>70.6</td>
<td>70</td>
</tr>
<tr>
<td>4</td>
<td>70%</td>
<td>130.5</td>
<td>129.3</td>
<td>130</td>
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<td>5</td>
<td>80%</td>
<td>170.9</td>
<td>172.0</td>
<td>171</td>
</tr>
<tr>
<td>6</td>
<td>100%</td>
<td>246.5</td>
<td>249.8</td>
<td>248</td>
</tr>
</tbody>
</table>

Figure 3.21: Peak carriage displacements, wall FF-3, uncracked. Reference line drawn through 100% scale point
Chapter 4

Shake Table Test Results

4.1 Introduction

This chapter presents the results of dynamic shake table testing. General visual observations from wall tests are presented, followed by a detailed analysis and discussion of numerical test data. Pertinent response history results for each run are provided in Appendix F.

4.2 Visual observations

Each of the five wall specimens developed a single horizontal crack near mid-height during the tests in which the carriages were locked out (Figure 4.1; refer to Tables 3.7–3.11 for test sequence). In every case, the crack occurred at the brick-mortar interface, but the crack location varied between the specimens. In wall RR-3, the crack was located at a height of 0.74 times the wall height, while in the other four specimens the crack height varied between 0.47 and 0.55 times the wall height. Cracks occurred both at header courses and at common courses. In Walls FF-3, FR-3, and SS-3, the crack was located in a single horizontal plane across the entire wall section. In Walls FF-2 and RR-3, the crack stepped down by one course. Even after sustained rocking in later runs, all cracks consistently closed up without horizontal offset and with minimal spalling of mortar or brick.

Prior to undergoing significant rigid-body rocking, cracks were visually nearly imperceptible (for example see Figure 4.2). Walls FF-2 and RR-3 underwent the greatest amount of rigid body rocking, and correspondingly also sustained the most spalling damage at the crack. The maximum amount of spalling that was observed prior to collapse is shown in Figure 4.3. The other wall specimens exhibited less damage than shown here. Additional photos showing the cracking patterns in each wall are included in Appendix E.

The extent of spalling at the base of the wall could not be observed due to the configuration of the base restraint. Every wall underwent rocking about the [wall]-[base beam] interface; no cracking was observed within the
4.2. Visual observations

Figure 4.1: Typical crack configuration

Figure 4.2: Typical detail of fresh crack

Figure 4.3: Maximum observed spalling
4.2. Visual observations

wall near the base. A video frame of wall FF-3 in the process of collapse (Figure 4.4) illustrates how the lower section of the wall remained intact and the wall was able to rock unrestrained as intended.

At sufficiently high input motion intensities, each wall specimen eventually underwent rigid-body rocking. Since only a single crack was formed in each wall, the rocking was clearly defined as occurring between two bodies: the lower wall block, from the base beam to the horizontal crack, and the upper wall block, from the crack to the top pin connection. This is illustrated in a video frame of wall FR-3 just before collapse (Figure 4.5).
4.2. Visual observations

Figure 4.4: Lower wall section at collapse

Figure 4.5: Wall rocking before collapse
4.3 Numerical results

This section presents recorded and derived data obtained from the shake-table testing. Response of the cracked walls are presented first, including rocking behaviour, wall stability limits, carriage response, and force demands. Cracking response of the walls is presented subsequently in Section 4.3.6.

4.3.1 Cracked response summary

The crack height, system period, and motion scales at collapse and in the run prior to collapse for each specimen are listed in Table 4.1. The system period is defined independently for the top and bottom of each wall, and should be interpreted as an indicator of the stiffness of each support rather than a true period of response, since in a multi-degree-of-freedom (MDOF) system like this one a period can only be defined for a modal response, and the modes in this system link both the top and bottom responses. In addition the height of the crack affects the distribution of tributary mass to the top and bottom diaphragms, and once the wall is cracked it no longer exhibits a periodic response.

For consistency and simplicity, each of these periods is calculated using exactly half the total wall mass plus the mass of the respective carriage system; it is thus independent of the crack height. The wall height used to normalize the crack height is taken as the distance from the base of the wall to the centerline of the top pin.

Table 4.1: Results summary

<table>
<thead>
<tr>
<th>Wall</th>
<th>Normalized crack height[a]</th>
<th>Period (sec)</th>
<th>Motion scale</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Top</td>
<td>Bottom</td>
</tr>
<tr>
<td>FF-3</td>
<td>0.47</td>
<td>1.59</td>
<td>1.59</td>
</tr>
<tr>
<td>FR-3</td>
<td>0.55</td>
<td>1.59</td>
<td>0</td>
</tr>
<tr>
<td>FF-2</td>
<td>0.49</td>
<td>1.24</td>
<td>1.24</td>
</tr>
<tr>
<td>SS-3</td>
<td>0.51</td>
<td>0.83</td>
<td>0.83</td>
</tr>
<tr>
<td>RR-3</td>
<td>0.74</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

[a] Normalized with respect to wall height
4.3. Numerical results

4.3.2 Displacement response

Prior to cracking, the wall behaves as a single rigid body. The out-of-plane elastic stiffness of the wall at this point is high, and the limitations of the string pot instrumentation setup prevented reliable measurement of the elastic bending displacements (the instruments could not resolve beyond a 2–3 mm precision). Consequently, no further analysis regarding the elastic behaviour of the wall was included here.

Once cracked, the wall behaves as two rigid bodies. In the test specimens, the combination of surface friction and potential interlock effect due to a non-planar crack surface effectively prevented any notable sliding at the crack interface. For analysis purposes, the two wall segments were thus assumed to be linked at the crack location. The relative displacement of the wall was assumed to be equal to that of the carriage at the base and the top of the wall, which is consistent with the observed performance of the wall-carriage connections during testing.

Displacements of the carriages were measured relative to the table, while displacements of the wall headers were measured relative to the ground (absolute). The nomenclature is illustrated in Figure 4.6. To obtain relative displacements of the wall, \( d_{rel} \), the table displacement was subtracted from the measured wall displacements. Relative displacements are zeroed at the initial at-rest positions of the carriages. Top and bottom carriage displacements, \( d_{rel_t} \) and \( d_{rel_b} \), remain as measured — relative to the shake table. The differential carriage displacement, \( d_{diff} \), is the difference between \( d_{rel_t} \) and \( d_{rel_b} \). Rocking displacement, \( d_{rock} \), is defined as the difference between the measured horizontal displacement of the wall at the crack height and the straight-line interpolation between the top and bottom of the wall at the same height. The normalized rocking displacement, \( d_{rock, norm} \), is simply the rocking displacement divided by the wall thickness.

A typical displacement profile of a cracked wall with flexible diaphragm conditions is shown in Figure 4.7a. The displacement at the crack was estimated as the mean of the straight-line extrapolated values from the top and bottom wall segments. The lines used in this process were defined as the least-squares fit subject to being forced through the carriage displacement value, since the carriage displacements were subject to less measurement noise than the wall displacements. This method minimizes the effect of errors for any single string pot measurement. Slopes of the segments were calculated from these best-fit lines. The corresponding rocking displacement profile is shown in Figure 4.7b.
4.3. Numerical results

Figure 4.6: Displacement nomenclature

Figure 4.7: Typical displacement profiles of cracked wall
4.3. Numerical results

4.3.2.1 Static stability limits

It is convenient to describe a stability limit to aid in understanding how ‘near to collapse’ a wall specimen comes in a shake table run. The stability of a single rigid rocking block depends on the time variation of the force balance and the body’s position. For a given ground motion, stability is a binary outcome of the whole run that can only be determined at the end of the run: either the block collapsed, or it did not. It is thus not possible to determine the actual stability at a given timestep, since whether or not the block will reach a collapsed state depends on the details of the applied motion following that timestep.

For example, a rocking block may have reached a significant rotation at some particular instant during a ground motion. Proceeding forwards from this time, a ground motion pulse might follow that would push the block towards the upright position, stabilizing it. Alternatively, a pulse in the opposite direction might follow, which would result in the block fully collapsing. At the original time instant considered, therefore, it is not possible to directly quantify the stability of the block. This is different from, say, a ductile column being subjected to a ground motion, where at any instant one can objectively calculate that the column is at \( x \% \) of yield strength — regardless of the time variation of the ground motion after that instant.

It is possible, however, to define a static stability limit for a body that can be used as an indicator of approximately how ‘near to collapse’ a body comes. Without dynamic effects (horizontal reactions at contact points and inertial forces), the stability can be described in terms of whether the force balance pushes the body towards a stable (in the case of a wall — vertical) or an unstable (tipped over) position. At any time step in a dynamic scenario, a body’s force and position conditions can be compared to this static limit. While this criterion will not accurately predict the stability outcome of a dynamic run, it can provide a good idea of whether the body was on the verge of collapsing or was relatively stable.

Two rocking mode shapes are possible for a wall with a single crack and at least one flexible diaphragm (Figure 4.8). The concepts here have been adapted from Derakhshan et al. [2014]. Assuming that the wall-diaphragm anchorage remains intact and that slip at the crack interface is negligible, the top wall segment is unconditionally stable and collapse of the wall can be determined by assessing the stability of the bottom segment. The upper portion of the figure shows the whole wall section, including dynamic effects, while the lower portion shows just the bottom wall segment simplified to the static conditions. Under static conditions, no inertial load is acting on the
4.3. Numerical results

wall, and it is assumed that the horizontal force transferred between the two wall segments at the crack interface and the horizontal force imparted by the base restraint are both negligible. At the peaks of large rocking excursions (near collapse, or at collapse initiation), accelerations at the crack are relatively small and the static simplification may not be too far from reality (e.g., Figure 4.35).

In Case 1, $\theta_t/\theta_b < 1$, and the points of contact at the crack and at the base of the wall are on opposite sides of the bottom wall segment. The weight of the top wall segment ($W_t$) and the overburden load ($P$) both act as restoring forces against overturning of the bottom segment while $\theta_b$ remains below the instability threshold.
4.3. Numerical results

In Case 2, $\frac{\theta_t}{\theta_b} > 1$, and the points of contact at the crack and at the base of the wall are on the same side of the bottom wall segment. Here, $W_t$ and $P$ both act as destabilizing forces for any value of $\theta_b$, and consequently the instability threshold rotation is significantly smaller for Case 2 than for Case 1.

For both cases, the static instability threshold rotation can be calculated by taking the sum of the moments acting on the lower wall segment about the point of contact at the base. It should be noted that in this model, the eccentricity of the overburden load does not affect the static instability rotation of the bottom segment — but it does still affect the dynamics of the wall behaviour, which determine when and how the conditions near instability are reached. The instability threshold was calculated in terms of both rotations and displacements for each wall specimen (Table 4.2). Small angle approximations were used in these calculations; for angles up to $15^\circ$, the maximum resulting error is roughly 2%.

The instability rotations for generic wall configurations with no overburden load are shown in Figure 4.9. The instability rotations decrease with increasing slenderness ratio. The instability conditions are also plotted as normalized displacements (displacement of bottom block at crack divided by wall thickness), for varying levels of overburden (Figure 4.10). Plotting the normalized displacement results in the same lines for all slenderness ratios. The overburden load in this plot is indicated as normalized to the total wall weight, $W$. Note that while the overburden load does not affect the Case 1 static stability threshold. The addition of overburden loads significantly reduce the stability thresholds for Case 2.

<table>
<thead>
<tr>
<th>Case</th>
<th>$\theta_{b_1}$</th>
<th>$d_{b_1}$ (mm)</th>
<th>$\theta_{b_2}$</th>
<th>$d_{b_2}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF-3</td>
<td>9.1°</td>
<td>291.0</td>
<td>2.6°</td>
<td>83.0</td>
</tr>
<tr>
<td>FR-3</td>
<td>7.6°</td>
<td>291.0</td>
<td>2.7°</td>
<td>103.0</td>
</tr>
<tr>
<td>FF-2</td>
<td>8.0°</td>
<td>191.0</td>
<td>2.3°</td>
<td>54.0</td>
</tr>
<tr>
<td>SS-3</td>
<td>8.5°</td>
<td>300.0</td>
<td>2.7°</td>
<td>96.0</td>
</tr>
<tr>
<td>RR-3</td>
<td>5.8°</td>
<td>296.0</td>
<td>3.1°</td>
<td>160.0</td>
</tr>
</tbody>
</table>
4.3. Numerical results

Figure 4.9: Instability rotations, without overburden load

Figure 4.10: Normalized instability displacements, varying overburden load
4.3. Numerical results

4.3.2.2 Time history response

Displacement time history results of run 10 of wall FF-2 are shown in Figure 4.11. This run illustrates sustained rocking behaviour. Three time instants of interest are indicated by the dotted vertical lines. The displacement profiles of the wall at these instants are shown in Figure 4.12. Figures 4.11a–4.11c simply show the previously-described displacement parameters. Figure 4.11d shows the processed bottom rotation, $\theta_p$, which is defined as:

$$\theta_p = \begin{cases} 0, & \text{if } |\theta_t - \theta_b| < \theta_{\text{threshold}} \quad \text{no rocking} \\ -1, & \text{if } \frac{\theta_t}{\theta_b} < 1 \quad \text{Case 1 rocking} \\ 1, & \text{if } \frac{\theta_t}{\theta_b} > 1 \quad \text{Case 2 rocking} \end{cases}$$ (4.1)

When rocking is negligible ($\theta_{\text{threshold}} = 0.1^\circ$ was used to accommodate errors in the calculated top and bottom rotations), the processed rotation is zero. Otherwise, the sign indicates the type of rocking (Case 1 or 2) as opposed to the direction of rotation. Sudden changes in sign therefore result when the top block is rotating relative to the bottom block and the point of contact at the crack switches sides (an impact occurs at this time). The static instability rotations (see Table 4.2) for both rocking cases are superimposed on this plot, providing a subjective idea of how near the wall is.

The instability ratio, $R_{\text{ins}}$, provides a normalized representation of the processed rotation, where the instability limit is reached at either $-1$ (Case 1 rocking) or $1$ (Case 2 rocking). It is shown in Figure 4.11e, and is defined as:

$$R_{\text{ins}} = \begin{cases} 0, & \text{if } |\theta_t - \theta_b| < \theta_{\text{threshold}} \quad \text{no rocking} \\ \frac{\theta_t}{\theta_b}, & \text{if } \frac{\theta_t}{\theta_b} < 1 \quad \text{Case 1 rocking} \\ \frac{\theta_t}{\theta_b}, & \text{if } \frac{\theta_t}{\theta_b} > 1 \quad \text{Case 2 rocking} \end{cases}$$ (4.2)

It can be observed that the rocking displacement becomes zero when the top and bottom rotations are equal, at which point the relative displacement at the crack falls between the top and bottom relative displacements. The largest rocking displacement occurs concurrently with the largest difference in rotations (e.g., at time C, Figure 4.12c). However, the processed bottom rotation, and thus the instability ratio, do not necessarily peak at these same instances. The instability ratio is directly related to the bottom block rotation; it can reach large values even when rocking is minimal (e.g., at time
4.3. Numerical results

B, Figure 4.12b). The top block needs to rotate only slightly relative to the bottom block to place all its weight on one edge of the bottom block, and thus produce one of the rocking cases of Figure 4.8. This slight difference in rotation can arise while both blocks were already at a significant rotation (along with differential diaphragm displacement). Conversely, large rocking displacements can occur while the instability ratio is small, if the bottom block is near vertical while the top block experiences a large rotation (e.g., at time A, Figure 4.12a).

The normalized rocking displacement is plotted for two runs for each wall in Figure 4.13: the highest stable run, and the run causing collapse. Significant differences are apparent among the different specimens. Wall RR-3 exhibited almost no rocking, even in the collapse run. Wall FF-3 showed limited rocking in the highest stable run, but this could be partially due to the large difference in scale between the last two runs for this wall (80% → 100%). Wall RR-3 underwent a long series of gradually decaying rocking excursions in the highest stable run, something not seen in any of the specimens with flexible diaphragms.

The instability ratio is plotted in Figure 4.14 for the same two runs for each wall (highest stable and collapse). As expected, wall RR-3 (Figure 4.14e) exhibits only Case 1 rocking (negative $R_{ins}$ values). Case 2 rocking requires some degree of diaphragm flexibility, which was not present in this configuration. The next-stiffest configuration, wall SS-3 (Figure 4.14d), showed a few instances of Case 2 rocking, but of short duration and small amplitude. The walls with the more flexible diaphragms showed more frequent instances of Case 2 rocking with longer durations and larger amplitudes. Of these, wall FR-3 (Figure 4.14b) showed the largest amplitudes in the Case 2 direction, but these peaks were typically abruptly terminated, indicating that the amplitude of rocking was very minor and the crack kept closing up, all while the rotations of both blocks were large due to the large differential diaphragm displacements.

In each of the five wall configurations, collapse occurred as Case 1 rocking (negative $R_{ins}$), despite significant Case 2 rocking sometimes occurring in the previous run or earlier in the collapse run. Unless the diaphragm stiffness is extremely soft — much softer than in any of the tests — collapse will always occur as Case 1 rocking. Instances of Case 2 rocking can contribute momentum to the bottom block that will push it towards collapse, but the top block will always flip back into the Case 1 direction before an actual collapse will occur (e.g., the collapse sequence of wall FR-3, to a minor degree). To accommodate a Case 2 collapse, the differential diaphragm displacement would need to be exceedingly large. Such a condition may be
4.3. Numerical results

Figure 4.11: Displacement time histories, wall FF-2, run 10
4.3. Numerical results

Figure 4.12: Displacement profiles, wall FF-2, run 10

possible for some building configurations (e.g., a long, narrow, unretrofitted diaphragm), but for most practical purposes collapse can be assumed to occur as Case 1 rocking.
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Figure 4.13: Rocking time histories, highest stable and collapse runs
4.3. Numerical results

Figure 4.14: Instability ratio time histories, highest stable and collapse runs
4.3. Numerical results

4.3.2.3 Peak response

For each specimen, peak values of the displacement parameters observed in the most intense stable run are listed in Table 4.3. The peak rocking displacements occur at the crack height. Peak values for the collapse run are listed in Table 4.4. None of the peak carriage displacement values occur as a result of the wall collapsing — they occur before collapse in every case.

Table 4.3: Peak displacement response in highest stable run

<table>
<thead>
<tr>
<th>Wall</th>
<th>$d_{rel_t}$ (mm)</th>
<th>$d_{rel_b}$ (mm)</th>
<th>$d_{diff}$ (mm)</th>
<th>$d_{rock}^{[a]}$ (mm)</th>
<th>$\theta_t^{[a]}$ (°)</th>
<th>$\theta_b^{[a]}$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF-3</td>
<td>157</td>
<td>180</td>
<td>73</td>
<td>46</td>
<td>0.16</td>
<td>2.3</td>
</tr>
<tr>
<td>FR-3</td>
<td>206</td>
<td>0</td>
<td>206</td>
<td>14</td>
<td>0.05</td>
<td>2.8</td>
</tr>
<tr>
<td>FF-2</td>
<td>102</td>
<td>142</td>
<td>74</td>
<td>89</td>
<td>0.47</td>
<td>4.7</td>
</tr>
<tr>
<td>SS-3</td>
<td>61</td>
<td>91</td>
<td>52</td>
<td>182</td>
<td>0.61</td>
<td>5.3</td>
</tr>
<tr>
<td>RR-3</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>132</td>
<td>0.45</td>
<td>6.7</td>
</tr>
</tbody>
</table>

$^{[a]}$ Values calculated based on linear fit of measured displacements on each wall segment

Table 4.4: Peak displacement response in collapse run

<table>
<thead>
<tr>
<th>Wall</th>
<th>$d_{rel_t}$ (mm)</th>
<th>$d_{rel_b}$ (mm)</th>
<th>$d_{diff}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF-3</td>
<td>182</td>
<td>273</td>
<td>144</td>
</tr>
<tr>
<td>FR-3</td>
<td>227</td>
<td>0</td>
<td>227</td>
</tr>
<tr>
<td>FF-2</td>
<td>130</td>
<td>169</td>
<td>110</td>
</tr>
<tr>
<td>SS-3</td>
<td>65</td>
<td>84</td>
<td>48</td>
</tr>
<tr>
<td>RR-3</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

In Figures 4.15–4.20, peak displacement parameters are plotted against the intensity of the ground motion in each run. Peak values of some of these parameters can not be defined when the wall collapses, and arbitrary response values have been selected to indicate collapse in these cases (as noted on the plots).

Significant rocking without collapse was observed in four of the five specimens (Figure 4.15). Wall FR-3, with rigid bottom diaphragm condition, underwent limited rocking in all runs prior to the collapse run despite large displacements of the top carriage. Of the remaining walls, FF-3 displayed
the least rocking in the run prior to collapse, but presumably this is in large part because the change in intensity was larger between the last two runs for this wall than for the others (20% vs. 5-10%).

![Figure 4.15: Peak rocking displacements](image)

Instability ratios reached in each run are shown in Figure 4.16. As discussed previously (page 73), collapse is reached in the Case 1 direction for each specimen. Significant ratios in the Case 2 direction were reached by walls FR-3 and FF-2, and small ratios by walls FF-3 and SS-3. Wall SS-3 showed decreasing Case 2 ratios in the highest two runs, with the largest Case 2 ratio reached in the third-highest run. All of these ratios were relatively small, however, and differences such as these can easily arise due to the chaotic nature of the wall’s rocking response. Recall that for a given instability ratio, $\theta_b$ in Case 1 will be significantly larger than in Case 2. Correspondingly, it can be noted that frequently Case 2 ratios in a particular run are larger than those for Case 1. Case 2 ratios are largest for wall FR-3 by a significant margin, which is due to the nature of the flexible/rigid diaphragm configuration forcing large differential diaphragm displacements, leading to large rotations of the bottom wall segment. Any small amount of rocking at this point can produce a large instability ratio of either case.

Peak displacements of top and bottom carriages are shown in Figures 4.17 and 4.18, and differential displacements in Figure 4.19. Peak top and bot-
4.3. Numerical results

![Graph](image)

**Figure 4.16:** Peak instability ratio

Bottom carriage displacements follow roughly linear trends with respect to the motion intensity, but the extrapolated fit lines would intersect the intensity scale at a value greater than zero. This can be largely attributed to the non-linear resistance discussed in Section 3.7.2, though some additional effects are likely due to the interaction of the rocking wall with the diaphragm. Peak differential carriage displacements are significantly larger than the difference between peak top and bottom displacements, indicating that the motion of the carriages consistently ends up out of phase, and thus the peak differential displacement does not necessarily coincide with the peaks of either the top or the bottom.

Peak angles of rotation of the top and bottom wall segments are shown in Figure 4.20. At a given instant for a particular wall, the distribution between top and bottom angles depends on the top and bottom diaphragm displacements and the rocking displacement, and additionally on the relative crack height. A crack higher on the wall will produce larger rotations in the top segment and smaller rotations in the bottom segment (e.g., wall RR-3, with a crack at 0.74h). Walls FF-3 and FF-2 also showed larger rotations in the top segment than in the bottom one, despite the cracks in these walls being near mid-height. A contributing factor may have been some degree of rotational restraint imparted by the bottom connection, although the connection was designed to minimize any such restraint. Wall SS-3
4.3. Numerical results

Figure 4.17: Peak top carriage displacements

Figure 4.18: Peak bottom carriage displacements
4.3. Numerical results

Figure 4.19: Peak differential carriage displacements

Figure 4.20: Peak wall segment angles
produced fairly similar peak rotations in top and bottom segments, and wall FR-3 produced bottom rotations that were slightly, but consistently, larger than top rotations. Both of these walls were also cracked near mid-height. The variability in these trends among the different walls suggests that the bottom connection rotational restraint was likely not an important effect, but rather that the characteristics of the motion, the exact crack height, and the diaphragm response were the primary factors of influence.

The rocking displacement at the crack is compared to the top carriage displacement in Figure 4.21. The spectral displacement at 1s is indicated on the y-axis on the right side. The rocking displacements are shown by the heavy lines, while the top carriage displacements are shown by the lighter weight lines of the corresponding colours. This plot illustrates that while the carriage (i.e. diaphragm) displacements roughly follow the spectral displacement linearly, the rocking response is fundamentally different.

4.3.2.4 Rocking period

Makris and Konstantinidis [2003] showed that the rocking response of a simple rectangular block subjected to ground shaking can not be characterized by a single degree of freedom system with a fixed period. Griffith et al. [2004]
4.3. Numerical results

further determined that the rocking frequency of a cracked wall is displacement dependent. Neither Griffith et al. nor Makris and Konstantinidis had considered the effect of flexible supports in their work.

The ‘period’ of the rocking response of the walls in the current tests was evaluated by considering each rocking excursion separately. One rocking excursion was defined as the response between adjacent points at which $d_{rock} = 0$. The period of an excursion was defined as two times the duration of the excursion, with one excursion approximating a half-cycle sine pulse. The amplitude of the excursion was recorded as the peak rocking displacement. Amplitudes of less than 5 mm were eliminated due to the limited precision of the measurements from which the rocking displacement was derived. The highest stable run from each wall was considered, with the exception of wall $RR-3$, due the lack of rocking observed. The observed periods are plotted against the corresponding normalized rocking displacements in Figure 4.22.

![Figure 4.22: Rocking period](image)

The rocking response of all walls follows generally the same trend: the minimum observed period increases as the rocking amplitude increases. For a given rocking amplitude (at moderate values), wall $RR-3$ exhibits the overall shortest rocking periods, while the walls with flexible diaphragms exhibit generally longer periods. Wall $RR-3$ also produces the most consis-
4.3. Numerical results

tently sine-like rocking oscillations; the shape of the rocking excursions of walls with flexible diaphragms are more irregular, and one or more direction reversal cycles may be contained within a single rocking excursion. This corresponds with the increased scatter present in Figure 4.22 for these walls relative to wall RR-3.

In general, one can conclude that at smaller rocking amplitudes, the more flexible diaphragms have the capacity to allow longer rocking periods, but also create greater variability in the rocking period. As the rocking excursions become larger, the effect of diaphragm flexibility becomes less important, implying that the period at these points becomes more a function of the wall characteristics rather than of the support conditions. The approximate shortest observed periods are listed in Table 4.5.

Table 4.5: Shortest observed rocking periods

<table>
<thead>
<tr>
<th>$d_{rock\text{norm}}$</th>
<th>$T_{rock}$ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>0.5</td>
</tr>
<tr>
<td>0.2</td>
<td>0.8</td>
</tr>
<tr>
<td>0.3</td>
<td>1.0</td>
</tr>
</tbody>
</table>

4.3.3 Acceleration response

An uncracked wall, idealized as rigid, undergoing dynamic excitation will exhibit a linear acceleration profile. If the excitations at the top and bottom of the wall happen to be in-phase and equal, the wall will exhibit a uniform acceleration profile. However, such idealized conditions can not be produced in a real structure (nor in a real test apparatus), and the difference in top and bottom excitations will produce a linearly-varying profile. Under these conditions, the acceleration profile is uniform only for brief instances as the slope of the linear profile changes sign.

Since a real wall has a finite elastic stiffness, the acceleration will deviate from this idealized linear profile and will instead assume a curved profile approximating a quadratic shape. Uncracked walls in this study that were run with flexible diaphragms — producing gentle direction reversals and thus minimal impact effects at the top pin connection — exhibited only small curvatures of the acceleration profile. With rigid diaphragm conditions, the more sudden direction reversals produced more pronounced profile curvatures (see Section 4.3.6).
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Once the wall is cracked, the acceleration profile becomes triangular, with local peaks at the crack height and at the top and bottom of the wall. The free boundary condition created by the crack prevents the formation of significant bending moments in the wall. Each segment of the cracked acceleration profile is thus very close to linear.

Typical acceleration profiles of cracked and uncracked walls with flexible diaphragm conditions are shown in Figures 4.23a and 4.23b. The acceleration at the crack was estimated as the mean of the straight-line extrapolated values from the top and bottom wall segments. The lines used in this process were defined as the least-squares fit through the wall sensors only. The carriage accelerations are shown on the profile plots, but were not considered in the line fitting since accelerations were not transferred continuously through the wall-carriage connections. In the top connection, the tolerance between the pin and the slot allowed for a small amount of 'slop', while in the bottom connection, the rubber spacers provided some flexibility. Consequently, the accelerations extrapolated from the wall sensors to the top or bottom of the wall are often significantly different than the accelerations measured on the carriages. It is also common for the acceleration profile to exhibit a discontinuity at the crack height. Figure 4.23c shows an example profile with discontinuities at all three locations, while within each of the wall segments, the measured accelerations are nearly perfectly linear.
4.3 Numerical results

4.3.3.1 Time history response

Acceleration time history results of run 10 of wall SS-3 are shown in Figure 4.24, and the shaded time is shown in greater detail in Figure 4.25. This run illustrates a large rocking excursion and the associated impact when the crack closes up. Three time instants of interest are indicated by the vertical dashed lines, with the acceleration profiles at these instants shown in Figure 4.26. Figures 4.24a and 4.24b show the relative displacements and the rocking displacement, respectively, while Figure 4.24c shows the accelerations of the top and bottom carriages and the calculated mean acceleration at the crack.

In general, the acceleration time history during rocking is characterized by periods of relatively smoothly varying accelerations during rocking excursions followed by periods of rapid variation following the impacts caused by the crack closing up. Peak accelerations at the crack and at the carriages occur at impact times.

At the peak of the large rocking excursion (time A, Figure 4.25), relative velocities of the carriages and of the crack are small, and the crack acceleration is in the opposite direction of the carriage accelerations — they are accelerating towards closing up the crack (Figure 4.26a). The accelerations slowly converge as the crack closes up, until roughly 0.015 s before the impact point (time B, and Figure 4.26b), at which point the acceleration profile is fairly uniform. The crack is now moving in the negative direction with considerable velocity, while the carriages are moving slightly in the positive direction. As the crack now closes, the wall at crack height quickly picks up positive acceleration (slowing its negative travel) while the carriages pick up negative acceleration. Peak acceleration is reached roughly 0.015 s after the crack has closed (time C, and Figure 4.26c). The rocking response of the wall has therefore dragged the carriages ‘along for the ride’, demonstrating that significant two-way interaction can occur between walls and diaphragms under the right conditions.

In this case, the carriages were lighter than the wall, facilitating this two-way interaction. If the carriages had been much heavier than the wall, the carriage motions would have approached that predicted for SDOF oscillators, and the wall would have been ‘along for the ride’ instead. It is likely that in this case, there would have been a larger rocking excursion following the main excursion, as the cracked wall would snap through between the carriages while they would carry on moving in their original (opposite) direction. One might intuitively expect that this characteristic would reduce the overall stability of the system.
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Figure 4.24: Acceleration and displacement time histories, wall SS-3, run 12
4.3. Numerical results

Figure 4.25: Close-up of acceleration and displacement time histories, wall SS-3, run 12
4.3. Numerical results

Figure 4.26: Acceleration profiles, wall SS-3, run 12

4.3.3.2 Peak response

For each specimen, peak values of the accelerations observed in the most intense stable run are listed in Table 4.6. Peak values for the collapse run are listed in Table 4.7; these values do not include the impacts caused by collapse of the wall.

Table 4.6: Peak acceleration response in highest stable run (g)

<table>
<thead>
<tr>
<th></th>
<th>Bottom Carriage</th>
<th>Bottom Wall$^a$</th>
<th>Crack Mean$^a$</th>
<th>Crack Top$^a$</th>
<th>Top Carriage</th>
<th>Top Wall$^a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF-3</td>
<td>0.57</td>
<td>0.56</td>
<td>0.55</td>
<td>0.56</td>
<td>0.58</td>
<td>0.34</td>
</tr>
<tr>
<td>FR-3</td>
<td>0.52</td>
<td>0.55</td>
<td>0.42</td>
<td>0.42</td>
<td>0.42</td>
<td>0.56</td>
</tr>
<tr>
<td>FF-2</td>
<td>0.57</td>
<td>0.56</td>
<td>0.93</td>
<td>0.96</td>
<td>0.99</td>
<td>0.38</td>
</tr>
<tr>
<td>SS-3</td>
<td>1.21</td>
<td>1.33</td>
<td>1.35</td>
<td>1.35</td>
<td>1.35</td>
<td>1.10</td>
</tr>
<tr>
<td>RR-3</td>
<td>0.54</td>
<td>0.57</td>
<td>0.48</td>
<td>0.46</td>
<td>0.46</td>
<td>0.73</td>
</tr>
</tbody>
</table>

$^a$ Values calculated based on linear fit of measured accelerations on each wall segment

Peak accelerations measured on the carriages for each run are shown in Figure 4.27. Peak accelerations calculated for the top and base of the wall based on a linear fit to the measured accelerations on the wall are shown
4.3. Numerical results

Table 4.7: Peak acceleration response in collapse run (g)

<table>
<thead>
<tr>
<th></th>
<th>Bottom Carriage</th>
<th>Wall[a]</th>
<th>Crack Bottom[a]</th>
<th>Mean[a]</th>
<th>Top[a]</th>
<th>Top Carriage</th>
<th>Wall[a]</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF-3</td>
<td>1.33</td>
<td>1.04</td>
<td>1.71</td>
<td>1.79</td>
<td>1.87</td>
<td>0.79</td>
<td>0.57</td>
</tr>
<tr>
<td>FR-3</td>
<td>0.65</td>
<td>0.67</td>
<td>0.58</td>
<td>0.55</td>
<td>0.52</td>
<td>0.60</td>
<td>0.74</td>
</tr>
<tr>
<td>FF-2</td>
<td>0.52</td>
<td>0.53</td>
<td>0.95</td>
<td>0.95</td>
<td>0.94</td>
<td>0.49</td>
<td>0.49</td>
</tr>
<tr>
<td>SS-3</td>
<td>1.01</td>
<td>0.99</td>
<td>1.92</td>
<td>1.80</td>
<td>1.67</td>
<td>0.93</td>
<td>0.90</td>
</tr>
<tr>
<td>RR-3</td>
<td>0.66</td>
<td>0.70</td>
<td>0.54</td>
<td>0.51</td>
<td>0.56</td>
<td>0.86</td>
<td>0.82</td>
</tr>
</tbody>
</table>

[a] Values calculated based on linear fit of measured accelerations on each wall segment in Figure 4.28. The ratios between peak top and bottom accelerations for both the wall and carriage values are shown in Figure 4.29. The peak mean accelerations calculated at the crack height are shown in Figure 4.30.

Figure 4.27: Peak carriage accelerations

In Figures 4.27 and 4.28, it is evident that some specimens produced monotonic increases of peak accelerations with increasing motion intensity. However, in several specimens, at least one of the top or bottom accelerations produced a reversal of the direction near the collapse run. Recalling that peak accelerations are produced upon impact after rocking excursions, these
4.3. Numerical results

Figure 4.28: Peak wall accelerations at top and bottom (extrapolated)

trend reversals would be expected for some scenarios. For wall SS-3, for example, the peak acceleration in run 12 (the highest stable run) is produced by the impact shown in Figure 4.25. In run 13 (the collapse run), this same rocking excursion becomes large enough to continue into collapse, without producing that particular impact, thus resulting in a lower peak acceleration for the whole run. In other specimens, collapse can result from new rocking excursions (e.g., a larger follow-through excursion), and the accelerations produced by particular large impacts can remain present in the collapse run.

Wall FF-2 produced the lowest peak accelerations despite undergoing significant rocking. This is likely in large part due to the significantly lower wall mass (less than 50% of the mass of the 3-wythe walls), meaning the response of the system was more heavily dominated by the mass of the carriages. Furthermore, the combination of the reduced mass and thickness would have resulted in lower impact energy from a given magnitude of normalized rocking.

Wall FR-3 produced larger peak accelerations at low motion intensities than FF-3, but the peak accelerations of FR-3 increased only moderately with increasing motion intensity, as would be expected given the very minimal rocking response of this wall in all runs. In contrast, wall FF-3 produced large increases in the runs where rocking became significant. The larger ini-
4.3. Numerical results

Initial peak accelerations of FR-3 could be due to the change in effective period of the rigid-base system vs. the fully flexible system, in combination with a possibly higher response at the base due to the rigid connection.

![Figure 4.29: Ratio of peak top to bottom accelerations](image)

The ratios of the peak top to peak bottom accelerations varied significantly among the various specimens and runs. The ratios calculated from the carriage response were generally very to those calculated from the wall response. Wall RR-3 was the only specimen to exhibit ratios in a consistent direction — larger top accelerations; all other walls produced ratios that fell on either side of 1.0 depending on the run or the measurement location.

Peak crack accelerations increased mainly monotonically for walls FF-3, FF-2, and SS-3, and remained fairly constant for walls FR-3 and RR-3 (Figure 4.30). The lack of large increases for FR-3 are again likely due to the lack of large rocking impacts. In contrast, wall RR-3 was subject to large rocking impacts without significant increases in crack accelerations. The fixed diaphragm conditions appear to ‘damp out’ the acceleration peaks by preventing the ‘flicking’ effect at snap-through that is allowed by the flexible diaphragms. With a fixed diaphragm, a wall segment pivots about the diaphragm connection, while with a flexible diaphragm, the effective pivot is located within the wall segment, allowing snap-through to occur more violently by accelerating both ends of the wall segment in opposite directions.
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4.3.4 Force demands

Force demands imposed on the wall-to-diaphragm connections are of interest to engineers involved in building assessment and retrofit design. In this section, the forces imposed by the wall on the carriages during the shake table testing are examined. Two approaches were used to calculate the total force demands on the connections.

In the first approach, the demand at a single connection (top or bottom) was calculated by subtracting the inertial force of the carriage (obtained from the accelerometer on the carriage) from the force in the springs (obtained from the measured spring stiffness and spring displacement).

The second approach allows the calculation of the total connection demand (sum of top and bottom). The total force on the wall was calculated by multiplying the acceleration measured on the wall at each header course by the wall mass tributary to the elevation of that course. Data from either eight (for the 4 m high walls) or six (for the 2.8 m high wall) accelerometers were used in this calculation. The two methods should produce equivalent results; comparing the results serves as a check on these calculations and on the instrumentation.

Results are plotted for run 12 of wall SS-3 as an example in Figure 4.31. Time histories of each component calculated in the first method are shown.
for the top and bottom in Figures 4.31c and 4.31d, and the total demand is compared for the two methods in Figure 4.31e. The two methods produce nearly identical results, with the forces calculated from the carriages exhibiting a slight time lag behind those calculated from the wall. The time lag is smallest for large amplitude excursions; for small oscillations both the lag and difference in peak magnitudes increase, with the forces calculated from the carriages having a smaller amplitude than those measured on the wall.

It is important to note that the acceleration profile of a cracked wall is the sum of several components, as illustrated in Figure 4.32 (adapted from Meisl [2006]). Since the total force is the integration of this profile along with the mass density over the height, it is a high-level quantity, and consequently many details about the response can be hidden within it. Two drastically different acceleration profiles can produce the same total force. For example, a uniform profile equal to zero everywhere on the wall and a profile with large accelerations at the crack and at each carriage, where the crack acceleration has the opposite sign as that at the carriages, can both produce a total force that is equal to zero even though the wall response is very different at those two times. Plots in which only the total force is shown should therefore be interpreted cautiously.

The forces can be calculated from the carriages only for flexible diaphragm configurations. When a carriage is locked into the rigid mode, the stiffness is too high and the displacements are too low to reliably measure the spring force. To compare consistent demands among all five wall specimens, only the forces calculated from the wall inertia should be used. Peak demands from both sources are listed for the highest stable run and for the collapse run in Table 4.8. Here, total forces are normalized to the total wall weight, while individual connection forces are normalized to 50\% of the wall weight — representative of the way in which tributary wall weights would be assigned in the assessment of a real building.

The maximum recorded normalized force for a cracked wall recorded from any source, in any run, is 0.61 g. This includes forces caused by impact when cracks close up, and includes the collapse run (up to a rocking displacement of just over one wall thickness). Considering the wide range of boundary conditions encompassed by these results, it is reasonable to conclude that connection force demands in cracked, one-way rocking walls with no overburden are unlikely to exceed 0.6 g. However, these conditions form close to a lower bound on wall stability, and changing boundary conditions to improve stability could certainly result in higher demands if walls are excited to correspondingly higher intensities.
4.3. Numerical results

Figure 4.31: Force time histories, wall SS-3, run 12
4.3. Numerical results

Figure 4.32: Components of acceleration profile

Table 4.8: Peak normalized force demands (g)

<table>
<thead>
<tr>
<th>Wall</th>
<th>Run</th>
<th>Wall Total</th>
<th>Wall Top</th>
<th>Wall Bot.</th>
<th>Collapse run Total</th>
<th>Collapse run Top</th>
<th>Collapse run Bot.</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF-3</td>
<td>12</td>
<td>0.26</td>
<td>0.23</td>
<td>0.38</td>
<td>0.58</td>
<td>0.47</td>
<td>0.41</td>
</tr>
<tr>
<td>FR-3</td>
<td>9</td>
<td>0.31</td>
<td>—</td>
<td>0.31</td>
<td>0.33</td>
<td>—</td>
<td>0.33</td>
</tr>
<tr>
<td>FF-2</td>
<td>10</td>
<td>0.35</td>
<td>0.25</td>
<td>0.49</td>
<td>0.39</td>
<td>0.38</td>
<td>0.35</td>
</tr>
<tr>
<td>SS-3</td>
<td>12</td>
<td>0.44</td>
<td>0.34</td>
<td>0.52</td>
<td>0.58</td>
<td>0.50</td>
<td>0.39</td>
</tr>
<tr>
<td>RR-3</td>
<td>5</td>
<td>0.30</td>
<td>—</td>
<td>—</td>
<td>0.33</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

Maximum: 0.44 0.42 0.34 0.52 0.58 0.50 0.41 0.61

Total wall forces are normalized to total wall weight.
Top and bottom connection forces are each normalized to one-half of the wall weight.
4.3. Numerical results

Peak force demands were compared with those predicted using the spectral acceleration of the shake table motion at the diaphragm period. Damping values obtained from calibration of the analytical model were used (refer to Section 5.3). Periods, damping ratios, and $S_a$ values at 100% scale are shown in Table 4.9. Using these parameters, the force demands listed in Table 4.8 are compared to the corresponding scaled $S_a$ values in Table 4.10. The results indicate that the spectral acceleration is in general an reasonably good approximate predictor of demands. However, forces significantly higher than those predicted by this method were observed, particularly for the bottom connection demand, for which several specimens developed forces up to 1.6 times higher than those predicted from $S_a$. Total force demands up to 1.7 times higher than the $S_a$ predictions were also recorded. These high demands appear to be caused by the impacts induced when the crack closes up after large rocking excursions — refer to the illustration of force-displacement hysteresis in Figure 4.37.

Table 4.9: Period, damping, and full-scale $S_a$ values

<table>
<thead>
<tr>
<th>Wall</th>
<th>$T_s$ (s)</th>
<th>$\zeta$</th>
<th>$S_a(T_s)_{100%}$ (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF-3</td>
<td>1.59</td>
<td>0.08</td>
<td>0.35</td>
</tr>
<tr>
<td>FR-3</td>
<td>1.59</td>
<td>0.08</td>
<td>0.35</td>
</tr>
<tr>
<td>FF-2</td>
<td>1.24</td>
<td>0.12</td>
<td>0.31</td>
</tr>
<tr>
<td>SS-3</td>
<td>0.83</td>
<td>0.08</td>
<td>0.49</td>
</tr>
<tr>
<td>RR-3</td>
<td>0</td>
<td>0.08</td>
<td>0.50[a]</td>
</tr>
</tbody>
</table>

[a] PGA

Peak total force demands (from wall inertia) for each run are plotted in Figure 4.33. In general, trends for each wall are fairly linear, with some ‘softening’ possible towards the collapse run (i.e. the point for the collapse run falls at lower force than the linear extrapolation from the other runs would suggest). Wall FF-3 showed the greatest deviation from linearity in the collapse run, while in other walls the effect was less notable. The specimens with the flexible springs (FF-3, FR-3, and FF-2) showed a generally lower rate of change of force with respect to motion intensity than the stiff and rigid specimens, with FR-3 showing the lowest rate by a significant margin.
4.3. Numerical results

Table 4.10: Ratio of peak normalized force demands to $S_a(T_s)$

<table>
<thead>
<tr>
<th>Wall</th>
<th>Run</th>
<th>Total</th>
<th>Total</th>
<th>Top</th>
<th>Bot.</th>
<th>Run</th>
<th>Total</th>
<th>Top</th>
<th>Bot.</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF-3</td>
<td>12</td>
<td>0.94</td>
<td>1.03</td>
<td>0.84</td>
<td>1.39</td>
<td>13</td>
<td>1.69</td>
<td>1.36</td>
<td>1.17</td>
</tr>
<tr>
<td>FR-3</td>
<td>9</td>
<td>0.81</td>
<td>—</td>
<td>0.82</td>
<td>—</td>
<td>10</td>
<td>0.78</td>
<td>—</td>
<td>0.80</td>
</tr>
<tr>
<td>FF-2</td>
<td>10</td>
<td>1.04</td>
<td>1.08</td>
<td>0.75</td>
<td>1.46</td>
<td>11</td>
<td>1.05</td>
<td>1.04</td>
<td>0.94</td>
</tr>
<tr>
<td>SS-3</td>
<td>12</td>
<td>1.20</td>
<td>1.15</td>
<td>0.93</td>
<td>1.42</td>
<td>13</td>
<td>1.49</td>
<td>1.28</td>
<td>1.00</td>
</tr>
<tr>
<td>RR-3</td>
<td>5</td>
<td>1.00</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>7</td>
<td>1.01</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

Maximum: 1.20 1.15 0.93 1.46 1.69 1.36 1.17 1.56

Total wall forces are normalized to total wall weight
Top and bottom connection forces are each normalized to one-half of the wall weight

Figure 4.33: Peak force demands on cracked wall
4.3. Numerical results

4.3.5 Hysteretic response

The hysteretic response of the rocking displacement at the crack height is examined in this section with respect to two parameters: the acceleration at the crack, and the total force acting on the wall. As mentioned previously, it should be recalled that the rocking displacement is calculated from the string pot measurements. The configuration of the string pots created some inaccuracies in the measurements, and consequently the measured rocking displacement is unreliable below roughly 8 mm, i.e. $d_{\text{rock}} = 0.025$ for 3-wythe walls and $d_{\text{rock}} = 0.04$ for the 2-wythe wall. As a result, the uncracked stiffness of the walls could not be measured from the shake table tests. The rocking response is, however, adequately captured.

4.3.5.1 Acceleration-displacement response

Figure 4.34 shows the crack acceleration vs. rocking displacement hysteretic response for the highest stable run for each wall. Note that to better illustrate the response within each run, the x- and y-scales of the plots vary. In general, each hysteresis plot displays two zones: with and without rocking. Prior to a significant rocking cycle being initiated, the response is basically elastic, with a stiff slope. During a rocking excursion, the crack acceleration remains relatively constant while the rocking displacement increases. Oscillations of the acceleration occur during rocking, typically on the leading portion of the excursion following an impact from the previous excursion. In Figure 4.34a, a large double oscillation occurs on the main rocking excursion (towards negative rocking displacement) that results in a full reversal of the crack acceleration while the rocking displacement changes minimally. These oscillations transferred into the carriages, which also experienced a sign change of accelerations.

Figure 4.35 shows the same plots for the collapse run for each wall. The x-scale (rocking displacement) is consistent in all these plots at ±1, which is an approximation of the static instability condition (the exact static instability limit is determined by the angle of the bottom wall segment, as discussed in Section 4.3.2.1, but by the time the rocking displacement reaches ±1, the bottom block is usually near this rotation limit). In each specimen, the crack acceleration decays approximately towards zero as the rocking displacement increases towards one. This decay occurs smoothly in some cases (e.g., Figure 4.37c) and with large oscillations in others (e.g., Figure 4.37d). In Figure 4.37e, the acceleration decays through zero before the rocking displacement reaches 1.
4.3. Numerical results

Figure 4.34: Acceleration hysteresis in highest stable runs
4.3. Numerical results

Figure 4.35: Acceleration hysteresis in collapse runs
4.3. Numerical results

In each of the plots in Figures 4.34 and 4.35, the dashed lines indicate accelerations of ±0.35 g. While these lines serve as a reference for the vertical scale, in general, these accelerations also bracket the ‘mean’ accelerations (moving time-averaged, discounting the oscillatory peaks caused by impacts) occurring during rocking excursions. Meisl [2006] termed this the ‘effective rocking acceleration’, and found that it was fairly consistent among all of his tests. He found the mean effective rocking acceleration to be 0.57 g, which is considerably higher than the 0.35 g noted for the current tests. It should be noted that estimation of this parameter is subjective. Meisl’s test setup was reasonably equivalent to that used in wall RR-3 in the current testing, as were his typical crack heights. However, wall RR-3 — for which 0.35 g would likely be a high estimate — showed perhaps the largest difference in effective rocking acceleration from Meisl’s values. A significant difference between the tests was the ground motion used, suggesting that perhaps the details of the ground motion time history may affect the effective rocking acceleration at the crack, despite Meisl noting similar accelerations for both ground motions that he ran.

4.3.5.2 Force-displacement response

Figure 4.36 shows the total wall force vs. rocking displacement hysteretic response for the highest stable run for each wall. Note that in this case, the y-scale is constant among all runs, while the x-scale varies. Compared to Figure 4.34, oscillations and extreme values in the force response plots are generally less intense than in the crack acceleration plots. For example, the large double oscillation of wall FF-3 visible in the acceleration plot of Figure 4.34a is much less significant in terms of the total force on the wall as shown in Figure 4.36a. On the other hand, for wall RR-3, the acceleration and force plots look reasonably similar in terms of oscillations and extreme values (Figures 4.34e and 4.36e).

This is consistent with the concept presented earlier that when the carriages are more easily influenced by the wall response (more flexible and/or relatively lighter carriages), the cracked wall snaps through the neutral position harder, creating larger accelerations at the crack and the base. These large acceleration spikes can occur without much change in the total force on the wall, since they occur in opposing directions. With rigid diaphragms, the snap through is more subdued, and the accelerations are more reflective of the total force on the wall.

Figure 4.37 shows the same plots for the collapse run for each wall. In the acceleration plots, the collapse runs were marked by at least one significant
4.3. Numerical results

Figure 4.36: Force hysteresis in highest stable runs
4.3. Numerical results

Figure 4.37: Force hysteresis in collapse runs
4.3. Numerical results

large pulse, typically much larger than in the highest stable run. In the force plots, the differences between the collapse and highest stable runs for any given wall are more subtle, and peak forces are not much larger — note that the y-scales in Figure 4.37 are consistent with those in Figure 4.36.

In each of the plots in Figures 4.34 and 4.35, the dashed lines indicate a total wall inertia of ±0.35g — equivalent to the entire wall accelerating at the value of the dashed lines in the acceleration plots. The dashed force lines bracket the mean rocking force similarly to the way in which the effective rocking acceleration was bracketed. Meisl [2006] noted that multiplying the wall mass by the effective rocking acceleration at the crack produced a reasonable estimate of the maximum total force on the cracked wall. This appears to be roughly valid for the results of the current testing, with the caveat that the maximum forces can exceed this estimate due to the greater propensity for larger force oscillations with flexible diaphragm configurations.

4.3.6 Cracking

Cracking was initiated in each wall by subjecting it to the NGA0763 motion with both carriages locked into the rigid mode. Confirming visually whether or not a run had initiated cracking was difficult in some specimens, and since data viewing capabilities during testing were limited, an additional run of the same motion at slightly increased intensity was then carried out to ensure that a crack had in fact formed.

The precise time at which the crack formed was estimated during post-processing of the data. The cracking runs typically produced very small rocking displacements, and the limited precision of the string potentiometers did not provide sufficient resolution of the rocking displacement to pinpoint the time of crack formation. Instead, the shape of the wall’s acceleration profile at each time step was used to evaluate whether the wall was cracked or uncracked. An uncracked wall generally exhibits a roughly quadratic acceleration profile as opposed to the bilinear profile of a cracked wall. At each time step, a quadratic curve was fit to the acceleration profile using linear regression. The coefficient of determination, $R^2$, was calculated for the regression at each time step. This coefficient was used as an indicator of ‘how cracked’ the acceleration profile was at any given time.

The time histories of selected accelerations and of total force on the wall are shown in Figures 4.38a and 4.38b. Since the wall is not cracked for a portion of this run, the recorded acceleration at header 4 (just below the crack) is shown rather than the mean of the linearly extrapolated ac-
4.3. Numerical results

Figure 4.38: Acceleration and force time histories, wall FF-3, run 7
4.3. Numerical results

celerations. Figure 4.38c shows the time variation of $R^2$ (filtered to remove
high-frequency variations). Two times of interest are indicated by the dotted
vertical lines.

Prior to time A, $R^2$ remains very close to 1, with the exception of isolated
spikes where the coefficient drops — typically as the acceleration profile
straightens and the curvature switches direction. After time B, $R^2$ remains
consistently below 1, with the exception of reaching values near 1 briefly
as the profile nears the curvature switch and the bilinearity becomes less
pronounced. The differences in the $R^2$ time history may appear subtle at
first glance, but the transition is distinct and was consistently identifiable
for each wall specimen. The acceleration profile at time A is shown in
Figure 4.39a along with the fitted quadratic curve, and the profile at time
B is shown in Figure 4.39b along with the fitted bilinear curve.

Figure 4.39: Acceleration profiles of wall before and after cracking

In addition to the $R^2$ transition at cracking, the acceleration at header 4
transitions from being generally in-phase with the carriage accelerations to
significantly out-of-phase. This is illustrated in Figure 4.38a, and was also
consistently identified in each specimen. The $R^2$ method, however, allows
the precise time of transition to be identified more readily, and consequently
was used as the primary indicator — verified by checking individual accel-
eration profile shapes.

The peak forces occurring in the cracking runs (normalized to wall
4.3. Numerical results

weight) for each specimen are summarized in Table 4.11. Time A in Figure 4.38 is representative of the data under ‘crack initiation’. In each specimen, the force peak at crack initiation was smaller than the maximum force of the entire run. In addition, the maximum force of the entire run occurred before the crack initiation time in every specimen. This suggests that cracks are formed progressively — some damage is caused during the large initial force peaks, but not enough to propagate the crack through the entire wall thickness. Subsequent force peaks, though smaller, cause further damage until eventually the crack has propagated through the wall.

Table 4.11: Peak normalized forces in cracking run

<table>
<thead>
<tr>
<th>Wall</th>
<th>Run</th>
<th>Scale</th>
<th>Crack initiation</th>
<th>Entire run</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Time(^a) (s)</td>
<td>Force(^b) (g)</td>
</tr>
<tr>
<td>FF-3</td>
<td>7</td>
<td>50%</td>
<td>4.32</td>
<td>0.39</td>
</tr>
<tr>
<td>FR-3</td>
<td>3</td>
<td>60%</td>
<td>5.18</td>
<td>0.36</td>
</tr>
<tr>
<td>FF-2</td>
<td>4</td>
<td>70%</td>
<td>4.12</td>
<td>0.58</td>
</tr>
<tr>
<td>SS-3</td>
<td>6</td>
<td>60%</td>
<td>5.03</td>
<td>0.30</td>
</tr>
<tr>
<td>RR-3</td>
<td>2</td>
<td>60%</td>
<td>4.55</td>
<td>0.49</td>
</tr>
</tbody>
</table>

\(^a\) The latest time at which the wall could definitively be identified as not cracked.

\(^b\) In each specimen, a cracked profile was confirmed no more than 0.1 s after this time.

The wall forces listed in Table 4.11 are equal to the sum of the top and bottom connection demands, which are representative of the demands on wall-to-diaphragm anchors. The distribution of these total forces between the top and bottom locations could not be measured directly with carriages locked out. However, individual connection demands can be approximated by assuming the wall to be simply supported (i.e. no moment support at the connections), and then calculating the reactions from equilibrium based on the distribution of the forces. Here, the force distribution at each time step was assumed to be proportional to the quadratic fit of the acceleration profile, denoted here as \( f(y) \), calculated earlier. The height of the centroid of the force, \( \bar{y} \), was calculated as:

\[
\bar{y} = \frac{\int_0^h y \cdot f(y) \, dy}{\int_0^h f(y) \, dy} \quad (4.3)
\]
where $y$ is the height from the base of the wall. From equilibrium, the individual connection forces can then be approximated as:

$$F_t = \frac{\bar{y}}{h} \cdot F_w \quad (4.4)$$

$$F_b = \left(1 - \frac{\bar{y}}{h}\right) \cdot F_w \quad (4.5)$$

The relative height of the force centroid is plotted against the normalized total wall force, $F_{\text{norm}}$, in Figure 4.40. Note that numerical inaccuracies with the quadratic fitting at very low force levels sometimes resulted in a centroid calculated as out of the wall height range; these values were then capped to either top or bottom of the wall. At large force levels, the force centroid was consistently located slightly above mid-height of the wall (on average roughly at 55% of the wall height). The total force demand can be interpreted to be allocated to the top and bottom connections as between 50%/50% and 60% top/40% bottom. It is critical to note that these allocations are based on the observed values from this particular test apparatus only, and they do not necessarily reflect conditions that may be encountered in real buildings.

![Figure 4.40: Location of centroid of wall force prior to cracking](image)

The total wall forces in the cracking run are compared with those measured in the two highest cracked runs for each wall in Table 4.12. Maximum
4.3. Numerical results

forces for each wall are shown in bold. Maximum forces attained in the highest stable rocking runs were lower in each case than those attained during the cracking runs. During collapse runs, force levels in walls FF-3 and SS-3 exceeded those attained during the respective cracking runs. In all cases, maximum wall forces recorded were between 0.5 and 0.6 g.

Table 4.12: Peak normalized forces in cracking run and cracked runs (g)

<table>
<thead>
<tr>
<th>Wall</th>
<th>Initiation</th>
<th>Entire run</th>
<th>Highest stable</th>
<th>Collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF-3</td>
<td>0.39</td>
<td>0.40</td>
<td>0.26</td>
<td>0.58</td>
</tr>
<tr>
<td>FR-3</td>
<td>0.36</td>
<td></td>
<td>0.31</td>
<td>0.33</td>
</tr>
<tr>
<td>FF-2</td>
<td>0.58</td>
<td>0.58</td>
<td>0.35</td>
<td>0.39</td>
</tr>
<tr>
<td>SS-3</td>
<td>0.30</td>
<td>0.49</td>
<td>0.44</td>
<td>0.58</td>
</tr>
<tr>
<td>RR-3</td>
<td>0.49</td>
<td>0.50</td>
<td>0.30</td>
<td>0.33</td>
</tr>
</tbody>
</table>

4.3.6.1 Cracking predictions

Flexural cracks like those observed in the test specimens are initiated when the tensile stress in the material exceeds the tensile strength. The flexural tensile strength of the masonry was tested using the bond wrench method (Table 3.5). The nature of this test method and the low tensile strength of the masonry produced large variability in the results, but the test nonetheless provides a rough idea of the expected tensile strength.

The wall in the test setup can be simplified as a simply-supported, one-way spanning beam. When undergoing excitation on the shake table, the wall’s inertia produces a distributed horizontal load on the wall, creating a bending moment, and thus compressive and tensile stress. In addition, the self-weight of the wall and the top beam assembly create axial compressive stress, which increases from the top of the wall towards the base. The total stress at any location in the wall is the sum of the axial and bending stresses.

To calculate the bending stresses in an uncracked wall at a given time step, a horizontal load distribution was assumed that had the same quadratic shape as the curve that was fit through the acceleration profile (as in Figure 4.39a), but was scaled to produce the total load previously calculated from the tributary weight method. Top and bottom horizontal reactions were calculated from equilibrium, and the shear and moment equations were obtained by integrating the load equation. Finally, a distribution for the
maximum net tensile stress was obtained by calculating the flexural tensile stress using half the wall thickness as the distance from the neutral axis to the tensile fiber and then adding the axial compression. The time histories of the peak tensile stress in the wall and the location of this peak stress are shown in Figure 4.41.

![Graphs showing stress time histories, peak tensile stress in wall, and location of peak tensile stress.](image)

Figure 4.41: Stress time histories, wall FF-3, run 7

The location of the peak stress varies significantly, and is plotted as a function of the stress in Figure 4.42, for all measurement points occurring prior to cracking (time A). The location of the peak tensile stress consistently decreases as the stress increases. At the largest stresses (those that would initiate cracking), the peak stress occurs at approximately 55% of the wall height. This trend, including the location at high stresses, was consistent among all specimens. This indicates that the variability in crack height among specimens (Table 4.1) can likely be attributed to the details of each wall’s construction — the strength of each joint in the wall will not be the
4.3. Numerical results

same due to construction variability and errors — rather than to a significant
difference in demands imposed upon the wall.

Figure 4.42: Location of peak tensile stress, wall FF-3, run 7

Calculated tensile stresses in the cracking run for each specimen are
compared with the flexural tensile strength measured by bond wrench testing
in Table 4.13. Shake table results are within one standard deviation of
the mean strength predicted by the static tests. While the bond wrench
results for walls SS-3 and RR-3 indicated a slightly higher strength than
for the other three walls, the calculated stresses during dynamic testing were
fairly consistent among all of the specimens. These results are reasonable,
considering the variability inherent in the bond wrench testing method as
well as in the wall construction and the dynamic load application.

Table 4.13: Peak stresses in cracking run

<table>
<thead>
<tr>
<th></th>
<th>Bond wrench</th>
<th>Shake table</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f'_{fb}$ (MPa)</td>
<td>$c_v$</td>
</tr>
<tr>
<td>FF-3</td>
<td>0.38</td>
<td>0.42</td>
</tr>
<tr>
<td>FR-3</td>
<td>0.38</td>
<td>0.42</td>
</tr>
<tr>
<td>FF-2</td>
<td>0.36</td>
<td>0.36</td>
</tr>
<tr>
<td>SS-3</td>
<td>0.55</td>
<td>0.31</td>
</tr>
<tr>
<td>RR-3</td>
<td>0.55</td>
<td>0.31</td>
</tr>
</tbody>
</table>
Chapter 5

Validation of Analytical Model

5.1 Introduction

The shake table tests conducted as part of the experimental program were able to examine a limited number and range of variables, and by themselves can not provide a comprehensive understanding of the out-of-plane response of URM walls with flexible diaphragms. Since shake table testing is prohibitively time- and resource-intensive, it is necessary to develop an analytical model which can reliably approximate the results of the physical testing.

As discussed in Section 2.2.2, two main simplistic approaches have been considered for modelling the rocking behaviour of a cracked URM wall:

1. A non-linear elastic system consisting of stick models for the wall segments connected by a rotational spring at the crack, producing a trilinear force-displacement response and using variable Rayleigh damping [Doherty, 2000], and

2. A rigid body model consisting of wall segments with a defined thickness, with rocking explicitly simulated using body geometry and impact mechanics [Meisl, 2006].

Makris and Konstantinidis [2003] demonstrated that the response of a rocking system is fundamentally different from that of a regular SDOF oscillator, and recommended that “the response of one should not be used to draw conclusion on the response of the other”. Since the out-of-plane response of URM walls has been extensively shown in laboratory tests to be dominated by rocking, the rigid body approach was selected for the analytical phase of the project.

In this chapter, an analytical rigid body model is validated to the shake table test results of Chapter 4. Modelling methodology and details are presented, followed by validation results.
5.2 Software

The commercially available software Working Model 2D [Design Simulation Technologies, Inc., 2010] was used to simulate the response of the wall-diaphragm system. Konstantinidis and Makris [2005] validated the ability of this software to simulate the pure sliding and pure rocking responses of a block as part of an investigation into the seismic performance of multi-drum columns. Meisl [2006] further demonstrated that the software could adequately simulate the rocking response of one-way spanning URM walls with rigid diaphragm boundary conditions.

Working Model 2D (WM) carries out time history analysis on a system of rigid bodies and constraints. A problem is time-discretized such that the program can compute motions and forces, while ensuring that the constraints are satisfied. The motion of bodies is governed by differential equations, which are solved in WM using the explicit Kutta-Merson (5th-order Runge-Kutta) integration method. The critical aspect of rigid body analysis is the treatment of contact between bodies. Sliding along a contact interface is treated with Coulomb friction. In addition, bodies are monitored continuously to detect collisions, and impact forces are calculated using an impulse-based model and the coefficient of restitution [Design Simulation Technologies, Inc., 2010].

5.3 Model construction

Models were created using scripts written in Working Model Basic [Knowledge Revolution, 1995], which is an offshoot of Visual Basic developed for Working Model 2D. The use of scripting rather than creating the models in the graphical interface provided much greater capabilities for parametrization, which were later used extensively, as well as allowing for greater precision and detail in the definition of body geometry and constraint placement.

The general configuration of the model is shown in Figure 5.1. The wall is modeled as two rigid bodies stacked one on top of the other, resting on a frictionless base, which represents the shake table. A rigid frame extends up from this base, representing the test frame. Spring-damper units connect this rigid frame to the top and bottom carriages. Each carriage is constrained to travel only horizontally using a square slot restraint.

Each wall segment was modeled with uniformly distributed mass, such that combined weight of the segments corresponds to the measured weight of the wall. WM calculates the corresponding mass moment of inertia. Spalling
5.3. Model construction

Figure 5.1: Model of test setup in Working Model 2D

Figure 5.2: Detail of crack in Working Model 2D (size of chamfer exaggerated for illustration purposes)
5.3. Model construction

at the rocking interfaces (at the base and at the crack) was represented by assigning a $45^\circ$ chamfer to the appropriate corners of the wall body. At the crack, the chamfer was set at 2 mm in all walls, consistent with the minimal amount of spalling that was visible during testing. The amount of spalling at the base could not be reliably assessed during testing due to the visual obstruction created by the bottom connection. Greater spalling of the mortar at the base than at the crack was expected due to the higher gravity loads and the fact that the base mortar was bearing on the steel beam. It was decided that a value of 10 mm at the base was reasonable, and it was found that this value produced good calibration results.

A triangular body, representing the top beam assembly, is connected to the top of the top wall segment by a rigid link. A pin node is placed at the peak of this triangle, connected to a vertical slot constraint in the top carriage. This modelled connection thus accurately simulates the response of the test setup by allowing rotation and vertical travel in the slot, while mating the horizontal displacement of the top of the wall to that of the top carriage.

The crack interface between the top and bottom wall segments proved challenging to model. In initial attempts, the blocks were modelled as simply resting on top of one another, with reasonable expected friction properties ($\mu_{\text{static}} = 0.75$, $\mu_{\text{dynamic}} = 0.75$). However, this resulted in the blocks sometimes experiencing large sliding instabilities during rocking, when no sliding was observed in the tests. This issue was resolved by constraining the blocks to one another horizontally (Figure 5.2).

Thin struts were added to the bottom wall segment, such that the top of each strut was at the same elevation as the crack. The additional mass contributed by the struts is negligible due to their small size. A workaround for a rigid link was connected from the end of each strut to the point of contact on the top wall segment when undergoing rocking (i.e. the inside of the chamfer). This link consists of a separator and a rope connected to the same nodes. The separator only acts as a constraint when its endpoints attempt to move closer together than its initial length, while the rope only acts as constraint when its endpoints attempt to move farther apart than its initial length. The initial length for the separator was set at the starting distance between the two nodes minus a tolerance dimension, and that of the rope was set at the starting distance plus the same tolerance dimension. The tolerance dimension was determined by trial and error. It was found that using a value of 0.2 mm, when the program’s integrator, overlap and assembly errors were also set to this same value, produced consistent stability. While this link configuration creates some slack in the connection...
5.3. Model construction

(compared to a true rigid link), the effects of this slack were found to be minimal with the small selected value of the tolerance dimension.

When the wall is undergoing rocking, there is only one contact point between the two wall segments at any time. To allow rotation of the top segment relative to the bottom segment, the link at the side of contact must be active, while the link at the other side must be inactive. This was achieved by setting the ‘active when’ property of the links to refer to the direction of relative rotation of the two wall segments. A tolerance of $\pm 10^{-7}$ rad was allowed during which links on both sides would be simultaneously active.

The base of the bottom wall segment rests on the frictionless body representing the shake table. The bottom carriage is connected horizontally to the bottom wall segment using the same links as at the crack interface. These links are connected to the wall segment at a height of 40 mm above the base (representing the middle of the rubber spacer in the test setup), and connected to the bottom carriage such that the links are horizontal in the at-rest position. The ‘active when’ property of these links referred to the absolute rotation of the bottom segment, and required no tolerance allowance. This link configuration simulates the test conditions accurately, and produces a different response than a slotted pin joint would.

Elasticity is implemented in WM using the coefficient of restitution, which is the ratio of the relative velocities of collided objects immediately before and after a collision. This is a property of a collision, but a coefficient is assigned to each body in WM. The coefficient of restitution in a collision is defined by WM as the lower value of the constants given to the two bodies involved in the collision. Meisl [2006] found that a small amount of elasticity was appropriate to simulate the rocking response of URM walls, and used values between 0.02 and 0.023. A value of 0.02 was assigned to the wall segments in this study. Due to the more constrained nature of the base connection as opposed to the totally free crack interface, a value of 0 was used for the shake table body. Overall, the model response was found to be minimally sensitive to the exact values of the coefficient of restitution, provided that they were in this general range.

The carriages were connected to the rigid frame with spring-damper units. The spring constants were set equal to the sum of those measured for each spring assembly. The damping values were empirically calibrated to achieve a good fit to the measured carriage response for the larger oscillations. The ‘stiction’ present in the system during low amplitude oscillations could not be captured by the viscous damping model, and this aspect was not further pursued in the modelling due to its low importance. The damping ratio for each carriage was defined in terms of the total spring constant.
for that carriage and the mass of the carriage plus half of the wall mass. A damping ratio of 12% was used for wall FF-2, and 8% was used for all other walls. Actual damping constants were then calculated by the WM script. Since the viscous damping in the model accounts for multiple sources of damping in the test setup (e.g., wheel friction, carriage rail misalignment, friction on the springs, spring assembly misalignment), and some of these sources of real damping can be dependent on the test specimen size and weight, it was decided that was little basis for applying the same damping constant for both the large and small walls. Consequently, the damping ratio was tuned independently for each of the two wall sizes.

The input motion in the model is applied to the shake table body by a displacement-controlled actuator connected to a fixed anchor block. The as-recorded table motions, low-pass filtered as described in Section 3.4.3, were fed into the actuator. The flexibility of the test frame was disregarded, and the rigid frame in the model applied the same input motion to both the top and bottom springs.

5.4 Modelled response

Each run on the shake table in which the CHHC1 ground motion was applied to a cracked wall was simulated using the model. Since wall stability is fundamentally a displacement-governed problem, the displacement response is of primary interest in the validation of the model. In particular, the rocking response of the wall is the critical output, for which a prerequisite is accurate modelling of the response of the diaphragms.

The peak normalized rocking displacements shown in Figure 4.15 are replicated in Figure 5.3, with the modelled results overlaid in blue. Note that for some walls — FR-3, SS-3, and RR-3 — the model did not produce a collapse outcome for the motion that caused collapse on the shake table. In these cases, further simulations were carried out by scaling up the final test motion until collapse was achieved in the model. The motion scale was incremented by scale factors of 1% at a time. In each case, collapse was produced in the model at scale factors no more than 3% greater than the scale factor used on the shake table.

The model approximates the general trend of rocking displacements vs. motion intensity reasonably well. The collapse scale is very well simulated; the largest discrepancy is the premature simulated collapse of wall FF-2 at a scale of 110% vs the observed collapse at a scale of 120%. The minimal rocking of wall FR-3 prior to the collapse run is well represented. There are
some discrepancies in the peak amounts of rocking observed in non-collapse runs, but significantly higher accuracy in the modelling of this parameter can not be expected given the nature of the rocking response.

A more detailed look at the model’s performance can be obtained by examining the time history output. Selected pairs of modelled and tested points are highlighted in Figure 5.3, labeled A and B. Points A illustrate low magnitude rocking from wall FF-2, and points B illustrate high magnitude rocking from wall SS-3. Time history comparisons are shown for points A in Figure 5.4 and points B in Figure 5.5.

Figure 5.4 illustrates the ability of the rocking model to accurately reproduce the non-periodic rocking response. The time variation of the rocking motion is matched very well by the model, though the magnitudes of some of the rocking excursion peaks are smaller in the model than recorded in the test. In addition, the response of both the top and bottom carriages are matched exceedingly well for the entire duration of the run.

The larger rocking displacements shown in Figure 5.5 are likewise reproduced well until the peak of the largest excursion is reached. Here, the model (which is running at 80% scale vs. the test at 75% scale) overshoots the rocking displacement slightly on this cycle. Since this happens to be on a very large rocking excursion (at a significant instability factor), this
5.4. Modelled response

Figure 5.4: Modelled vs. tested displacement time histories, wall FF-2, run 9 (modelled) vs. run 9 (tested)
Figure 5.5: Modelled vs. tested displacement time histories, wall SS-3, run 13 (modelled) vs. run 12 (tested)
5.4. Modelled response

overshoot significantly elongates the duration (‘period’) of that rocking cycle in addition to increasing the peak rocking displacement. Consequently, when the wall returns to the closed position (no rocking displacement) in the model, the diaphragms are at significantly different positions than at the earlier point in the test. The wall thus reacts quite differently at this point, following through with a large excursion in the opposing direction rather than damping out the rocking rapidly as in the test. Due to the strong interaction of this heavy wall with the carriages, the simulated response of the carriages is also thrown off significantly after this point, and the modelled response actually ends up nearly out-of-phase with the measured response.

This example illustrates the particular difficulty associated with replicating a measured response with a model for a non-periodic response like rocking: slight variances in a single excursion can throw off the remainder of the simulated time history by very large amounts. It is therefore important to note that consistent and exact matching of an entire run’s time history is an overly ambitious goal that serves little practical purpose. The inputs for the current model were calibrated roughly and consistently among wall specimens. Certainly further tweaking of the inputs specifically for each wall might lead to some gains in the accuracy of a particular simulation, but there is no need for this. In its current state, the model clearly reproduces with reasonable accuracy and consistency the following:

- the general time variation of the rocking response,
- peak rocking magnitudes, and
- motion scales causing wall collapse.

The model is thus adequate as a predictor of wall performance for other geometric configurations and ground motions, where the goal is to assess trends in wall stability as these variables change. Validation results for each run are provided in Appendix G.
Chapter 6

Parametric Study

6.1 Introduction

The experimental phase of the project was restricted to examining a very limited number of combinations of ground motions, wall geometries, and boundary conditions. The out-of-plane response of walls must be explored under a significantly larger range of these variables in order to allow generalized conclusions to be drawn. To fulfil this need, a parametric study was conducted using the analytical model described in Chapter 5. The effects of relevant variables were examined by running time history analyses with a large suite of ground motions for each configuration of interest. The threshold of out-of-plane wall collapse is evaluated for each run, and results are compiled and interpreted. This chapter describes the ground motions used, the model configuration, the modelling procedure, and the results.

6.2 Ground motions

The response of a non-linear system can be sensitive to the details of the time variation of the ground motion. These details of a motion's response history can not be precisely described by simple intensity measures, nor can the response of a non-linear system be predicted based only on such intensity measures. It is therefore necessary to run a model of a system through the entire history of a ground motion to determine the system’s response.

The inability to predict the system response based on intensity measures makes the selection of individual ground motions a gamble — it is not possible to determine before running an analysis whether a particular ground motion will result in below or above average system response for a given intensity measure. To obtain an accurate picture of the system response, it is necessary to account for motion-to-motion variability. In practice, for a particular project this variability might be estimated using a relatively small number of motions (e.g., seven). In this exploratory study, however, it is desired to take a detailed look at motion-to-motion variability and to
interpret the findings. A larger set of motions is needed to achieve these goals, and the selection of motions must be carefully considered. This section describes the rationale for the selection of ground motions, followed by a summary of pertinent characteristics of the selected set of motions.

6.2.1 Background

Ground motion selection methodology continues to be a contentious issue among researchers and practicing engineers. As discussed above, it is difficult to characterize the time-variable response details of motions using scalar intensity measures. The response spectrum achieves it adequately for linear single-degree-of-freedom (SDOF) systems — for these systems, it suffices to read the spectral value at the system’s natural period of vibration to determine precisely its peak response. For systems with non-linearity and/or multiple degrees of freedom (MDOF), a simple solution does not exist.

MDOF systems have (typically) different periods of vibration for each mode of response, with the fundamental period being the longest one. The values at multiple periods on the response spectrum are therefore relevant in estimating the response of such a system. Consider as an example two motions which have the same spectral value at a MDOF system’s fundamental period. At shorter periods, the spectrum of motion A is larger than that of motion B. The modal periods of the system’s higher modes will fall within this shorter period range. One can therefore reasonably predict that motion A should result in a larger system demands than motion B. The characteristic that differs between these two motions can be termed the spectral shape.

In the present study, the base system is a cracked wall connected at the top and bottom to flexible diaphragms. In its most basic representation, this system consists of three degrees of freedom: the lateral displacement of the top of the wall, the crack location, and the bottom of the wall. Given that the top and bottom diaphragm stiffnesses may not necessarily be the same, and that the rocking response does not even have a characteristic period, it is clear that MDOF effects may be significant. It was therefore decided to consider the effect of spectral shape in the analysis.

To account for this effect, Baker and Cornell [2005] proposed a vector-valued ground motion intensity measure consisting of spectral acceleration ($S_a$) and a shape parameter, $\varepsilon$. Both $S_a$ and $\varepsilon$ are period-dependent. Since the concept of $\varepsilon$ is significantly newer than that of response spectra, it is briefly explained here.
6.2. Ground motions

For a given ground motion record, $\varepsilon$ is calculated by comparing the elastic response spectrum of that record with the predicted response spectrum for the same magnitude, distance, site conditions, etc. Several ground motion attenuation relations are available in the literature; that proposed by Boore et al. [1997] was used in this study. The other relations are generally consistent with this one, and for the purposes of this study the differences can be expected to be of little consequence. This relation requires as input the period of interest ($T$), the type of fault (normal or reverse), and $M_W$, $r_jb$, and $V_{S30}$. It returns the predicted mean and standard deviation of the spectral acceleration at that period. Given both the recorded and predicted spectra, $\varepsilon$ is then calculated as the number of standard deviations that the recorded spectral acceleration falls from the mean predicted spectral acceleration (where the spectral accelerations are lognormally distributed):

$$
\varepsilon = \frac{\ln S_a(T) - \hat{\mu}_{\ln S_a(T)}}{\hat{\sigma}_{\ln S_a(T)}}
$$

(6.1)

The process is illustrated for a sample motion in Figure 6.1. Figure 6.1a shows the mean and $\pm 1\sigma$ of the prediction from the ground motion attenuation relation, in addition to the spectrum from the recorded ground motion. Figure 6.1b shows the calculated $\varepsilon$ values at each period. Periods where the recorded spectrum exceeds the mean prediction have positive $\varepsilon$ values, and vice versa. Ground motions will typically not have consistent $\varepsilon$ values over the entire period range, and thus a high $\varepsilon$ value at a particular period will tend to indicate a peaked shape of the response spectrum around that period, whereas a low $\varepsilon$ indicates a valley.
6.2. Ground motions

\[ \hat{\mu}_a S_a + \hat{\sigma}_a S_a \]

\[ \hat{\mu}_a S_a - \hat{\sigma}_a S_a \]

Recorded motion

\( \varepsilon = 1.32 \)

\( \varepsilon = -0.93 \)

Figure 6.1: Sample illustration of \( \varepsilon \) calculation
6.2. Ground motions

6.2.2 Ground motion selection

For site specific studies, such as those typically carried out in practice for a particular project at a single location, recent advances have greatly improved the basis for ground motion selection. Probabilistic seismic hazard analysis (PSHA) allows the deaggregation of the hazard, which provides insight into the main sources of contribution and their characteristics. At each period, a target $\varepsilon$ value can be derived by considering the $\varepsilon$ values of the dominant contributions to the hazard. A conditional mean spectrum (CMS) can be developed for a period of interest [Baker, 2011]. The CMS can then be used as a reference shape for matching ground motion spectra. For a simple structure whose response is dominated by the fundamental period, a single CMS at that period might be used; for a more complex structure multiple CMS’s may be used to examine several characteristic periods.

In a non-site specific study, for example where one might aim to draw conclusions about the performance of a class of structures, of which specimens could be located anywhere, the CMS method cannot be directly applied. This is due primarily to two factors: target $\varepsilon$ values depend on the hazard deaggregation, which cannot be generalized to different sites, and a CMS must be constructed around a single period of interest, which cannot be defined over a class of structures with variable configurations. Haselton et al. [2011] suggested a procedure whereby the effects of the spectral shape could be incorporated in the application of the results of a generalized study. The procedure is summarized as follows (adapted from Haselton et al.):

1. Select a general far-field ground-motion set without regard to $\varepsilon$ values of the motions
2. Calculate the collapse capacity by conducting a nonlinear incremental dynamic analysis (refer to Vamvatsikos and Cornell [2002])
3. Select a period of interest, and perform a linear regression analysis between the collapse capacity of each record and the $\varepsilon(T_1)$ of each record
4. Adjust the collapse capacity distribution, by using the regression relationship, to be consistent with the target $\varepsilon(T_1)$ for the site and hazard level of interest.
5. Repeat steps 3 and 4 for different periods as necessary

In step (1), the far-field motion set could be selected based on any number of criteria (e.g., magnitude, distance, peak response parameters, etc.).
6.2. Ground motions

In the absence of any well-defined justification for choosing selection criteria for a generalized study, the use of a standardized ground motion set carries some appeal. It offers the benefit of consistency with other research in which it is used, including the possibility of retroactively applying findings from future research using this motion set to conclusions from the current project.

The ground motion set prescribed by FEMA P695 [FEMA, 2009] for use in quantifying seismic performance factors was selected for the current study. It includes subsets of far-field and near-fault motions, with the near-fault motions further split into pulse and non-pulse motions. The set includes both horizontal components from 50 records, for a total of 100 motions. Of these, 22 records (44 motions) are classified as far-field, 14 (28) as near-fault pulse, and 14 (28) as near-fault non-pulse. Portions of the study were run using the full set of motions, though the analysis focuses primarily on the far-field set.

6.2.3 Ground motion characterization

The motions in the far-field set had moment magnitudes ($M_W$) between 6.5 and 7.62. Shear-wave velocities in the upper 30 m of soil ($V_{S30}$) varied between 192 and 724 m/s, and Joyner-Boore distances ($r_{jb}$) varied between 7 and 26 km (see Figure 6.2). This distance measure represents the horizontal distance from the station to a point on the earth’s surface that lies directly above the rupture [Boore et al., 1997].

The effects of spectral shape were examined for the far-field set as part of this study. In order to do so, the $\varepsilon$ values for each ground motion were calculated. The original (unscaled) response spectra of the far-field motions are shown in Figure 6.3a, with mean and $\pm 1\sigma$ curves overlaid in blue. Figure 6.3b shows the $\varepsilon$ calculated for each period for each motion. Note that on average, the motions have a slight positive-$\varepsilon$ bias, with $\mu_\varepsilon \approx 0.5$ for short and mid-range periods. The dip between periods of 0 and 0.1 s is due to the interpolation between the listed coefficients in Boore et al., and can be ignored — values in this range were not used in the study.

Scaling of ground motions is critical for procedures such as the incremental dynamic analysis (IDA) prescribed in FEMA P695, in which the whole set of motions is uniformly incremented at each step in the analysis. A variation of this procedure, in which each ground motion is incremented separately, was used in the current study. This renders the reference scaling of the ground motions irrelevant, except for the convenience aspect of obtaining similar scale factors at various performance points among differ-
6.2. Ground motions

Figure 6.2: Magnitude, distance, and shear wave velocity of far-field ground motions
6.2. Ground motions

Figure 6.3: Response spectra for far-field ground motions - original

Figure 6.4: Response spectra for far-field ground motions - normalized
ent motions. To achieve this convenience, the motions were normalized by the same method discussed in Appendix A of FEMA P695 — namely, by scaling the geometric mean of the peak ground velocities (PGV) of the two components of each record to a constant value, such that the average of the mean PGV did not change for the set of motions. The normalized spectra are shown in Figure 6.4. All motion scales referenced in this chapter are relative to these normalized motions. The full set of motions is tabulated in Appendix I.

6.3 Model configuration

The functional configuration of the model is illustrated in Figure 6.5. The Working Model Basic code from a sample script creating the model for a single configuration is listed in Appendix H. The details of the model are generally the same as those described in Chapter 5, with a few changes:

- The vertical offset of the wall-diaphragm connections relative to the top and bottom of the wall was changed to zero. The offsets used in Chapter 5 were intended only to replicate the conditions of the test rig. The height between supports is now equal to the exact height of the wall.

- The definition of the chamfers at the crack and at the base of the wall was changed to accommodate separate vertical and horizontal dimensions. The vertical dimension was set to represent one mortar bed thickness, while the horizontal dimension is a parameter within the study. The change to a fixed vertical dimension is expected to have minimal impact on the results.

- Provision for an applied overburden load was added to the model. Two methods of application were investigated, indicated by details (a) and (b) in Figure 6.5. In detail (a), the load is applied to a block resting on top of the wall. The block is frictionless and constrained from rotation. Its horizontal displacement is matched to that of the top diaphragm, but it is free to move vertically. The eccentricities $e_1$ and $e_2$ define the extents of the block. As the top wall segment rotates through vertical, the point of contact at which the overburden load is applied changes from one side of the block to the other. This detail is representative of the load being applied by either a wall in the storey above, a parapet, or joists. In detail (b), the overburden is applied as
a force at a fixed eccentricity. The location of the force remains the same regardless of the rotation of the top wall segment. This detail could be representative of a post-tensioning retrofit.

The input parameters used in a given configuration determine the values of the derived parameters. Input and derived parameters are defined in the upper and lower sections, respectively, of Table 6.1. When an input parameter is changed between two configurations, the remaining input parameters retain constant values, and the derived parameters are recalculated. For example, if changing the slenderness ratio, the thickness remains the same but the height is recalculated. The new height results in a new wall volume, which for the same density results in greater wall mass. The stiffness then changes to retain the same period. With the changed stiffness and mass, the damping constant changes to retain the same damping ratio.

It is worth revisiting the definition of the ‘diaphragm period’ ($T_b$ or $T_t$, or more generally the system period, $T_s$). A cracked wall connected to flexible diaphragms combines three responses: the vibration of two diaphragm systems and the rocking response of the wall. The diaphragm period, as defined here, is a reference indicator of diaphragm stiffness, and is an approximation of the initial period of vibration of a diaphragm connected to uncracked walls. As observed in the testing, the rocking motion of a cracked wall does not have a distinct period (Section 4.3.2.4). Furthermore, the additional degree of freedom created by the crack changes the effective mass of the diaphragm system, thereby changing its effective period relative to that calculated by simple tributary mass. While the diaphragm period is a convenient and intuitive way to characterize such a system, it is important to keep note of its limitations.
6.3. Model configuration

Figure 6.5: Model configuration
### 6.3. Model configuration

Table 6.1: Model parameters

<table>
<thead>
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<th>Parameter</th>
<th>Reference value</th>
<th>Description/Formulation</th>
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<tbody>
<tr>
<td>$t$</td>
<td>330 mm</td>
<td>wall thickness</td>
</tr>
<tr>
<td>$h/t$</td>
<td>11</td>
<td>slenderness ratio</td>
</tr>
<tr>
<td>$L$</td>
<td>1.0 m</td>
<td>wall length</td>
</tr>
<tr>
<td>$\rho$</td>
<td>2100 kg/m$^3$</td>
<td>wall density</td>
</tr>
<tr>
<td>$h_{cr}$</td>
<td>0.6</td>
<td>relative crack height</td>
</tr>
<tr>
<td>$s_v$</td>
<td>12 mm</td>
<td>joint thickness</td>
</tr>
<tr>
<td>$s_h$</td>
<td>10 mm</td>
<td>spall depth</td>
</tr>
<tr>
<td>$\zeta_t, \zeta_b$</td>
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<td>damping ratio, top &amp; bottom</td>
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<tr>
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<td>period, top &amp; bottom</td>
</tr>
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<td>$R_{M_t}, R_{M_b}$</td>
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<td>mass ratio, top &amp; bottom</td>
</tr>
<tr>
<td>$p$</td>
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<td>axial load per unit length</td>
</tr>
</tbody>
</table>

**Derived**

<p>| | | |</p>
<table>
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<th></th>
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<tbody>
<tr>
<td>$h$</td>
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<td>$t \cdot (h/t)$</td>
</tr>
<tr>
<td>$h_t$</td>
<td>1.452 m</td>
<td>$h \cdot (1 - h_{cr})$</td>
</tr>
<tr>
<td>$h_b$</td>
<td>2.178 m</td>
<td>$h \cdot h_{cr}$</td>
</tr>
<tr>
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<td>2516 kg</td>
<td>$\rho \cdot h \cdot t \cdot L$</td>
</tr>
<tr>
<td>$M_{w_t}$</td>
<td>1006 kg</td>
<td>$M_w \cdot (1 - h_{cr})$</td>
</tr>
<tr>
<td>$M_{w_b}$</td>
<td>1510 kg</td>
<td>$M_w \cdot h_{cr}$</td>
</tr>
<tr>
<td>$M_{d_t}, M_{d_b}$</td>
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<td>$R_{M_t,b} \cdot \frac{M_w}{2}$</td>
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<td>$k_{t,b}$</td>
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<td>$(\frac{2\pi}{T_{t,b}})^2 \cdot (\frac{M_w}{2} + M_{d_t,b})$</td>
</tr>
<tr>
<td>$c_{t,b}$</td>
<td>3160 Ns/m</td>
<td>$\zeta_{t,b} \cdot 2 \sqrt{k_{t,b} \cdot (\frac{M_w}{2} + M_{d_t,b})}$</td>
</tr>
<tr>
<td>$P$</td>
<td>0 kN</td>
<td>$p \cdot L$</td>
</tr>
</tbody>
</table>
6.4 Study methodology

The parametric study was grouped into two phases: (1) investigating the effect of each single parameter relative to a reference configuration, and (2) running the full matrix of parametric combinations of a subset of primary parameters. Phase 1 was split into two sub-phases: phase 1a investigated the effect of diaphragm flexibility, including ground motion characteristics, while phase 1b investigated the effects of other parameters. The intent of phase 1 is to provide a rudimentary look at effects of individual parameters: the subsection for each parameter is a figurative cross-section of the effect of that parameter on the reference configuration. The results are intended to be applied only conceptually, and care should be taken not to lose context of any numerical values shown in plots. Phase 2 is intended to provide a full dataset to be used as a basis for developing recommendations for assessment. Here, conclusions drawn in phase 1 are incorporated, and numerical values are intended for direct application. The analysis procedure for a single configuration was the same in both phases; only the selection of parameters used to create the configurations varied.

The normalized ground motions described in Section 6.2.3 were used in an IDA. It is difficult to pin down a fundamental period for a given configuration of the model, since there could be variable diaphragm periods at top and bottom, in addition to the non-periodic response of the rocking wall. A traditional IDA in which the whole set of motions is uniformly incremented after being scaled at the fundamental period of the structure would thus be difficult to both define and interpret. To circumvent these issues, each ground motion was incremented separately, and the results aggregated in post-processing. This renders the reference scale of each motion irrelevant.

Each motion was incremented (↑) and decremented (↓) as follows. Here, $S$ refers to motion scale — relative to the normalized motions (Figure 6.4) — and the parameter values are listed in Table 6.2.

1. Run at $S_{\text{start}}$
2. $\uparrow$ by $S_{\text{major}}$ until a run reaches $d_{\text{rock,threshold}}$
3. $\downarrow$ by $S_{\text{major}}$ once
4. $\uparrow$ by $S_{\text{minor}}$ until a run reaches collapse
5. $\downarrow$ by $\frac{1}{2} \cdot S_{\text{minor}}$ once

The range of parameter values considered in phase 1 is listed in Table 6.3, divided into phases 1a (top section) and 1b (bottom section). Parameters
6.4. Study methodology

Table 6.2: Incrementation parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{\text{start}}$</td>
<td>0.50</td>
</tr>
<tr>
<td>$S_{\text{major}}$</td>
<td>0.40</td>
</tr>
<tr>
<td>$S_{\text{minor}}$</td>
<td>0.10</td>
</tr>
<tr>
<td>$d_{\text{rock threshold}}$</td>
<td>0.10·$t$</td>
</tr>
</tbody>
</table>

were varied one at a time, i.e. each configuration is the same as the reference configuration with the exception of one parameter only. The selection of reference values and ranges for the various parameters are discussed in the applicable subsections in Sections 6.5.2 and 6.5.3. Where parameters are specified for both top and bottom locations, they were changed such that the top and bottom values were the same, except as specifically noted. While the primary reference period was $T_s = 1.0 \text{s}$, all the runs listed in Table 6.3 were repeated for a secondary reference period of $T_s = 0.5 \text{s}$. The total number of configurations tested in this phase was 73. The full set of 100 ground motions was run for each of these configurations.

Table 6.3: Phase 1 runs

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Reference value</th>
<th>Modelled values</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_l, T_b$</td>
<td>1.0 s</td>
<td>0.0 0.2 0.5 0.75 1.0 1.25 1.5 2.0</td>
</tr>
<tr>
<td>$T_l$ only[a]</td>
<td>1.0 s</td>
<td>0.0 0.2 0.5 0.75 1.0 1.25 1.5 2.0</td>
</tr>
<tr>
<td>$t$[b]</td>
<td>330 mm</td>
<td>110 220 330</td>
</tr>
<tr>
<td>$h/t$</td>
<td>11</td>
<td>4 8 11 14 18 22</td>
</tr>
<tr>
<td>$h_{cr}$</td>
<td>0.6</td>
<td>0.4 0.5 0.6 0.7 0.8</td>
</tr>
<tr>
<td>$s_h$</td>
<td>10 mm</td>
<td>5 10 15</td>
</tr>
<tr>
<td>$\zeta_l, \zeta_b$</td>
<td>0.05</td>
<td>0.03 0.05 0.07 0.10</td>
</tr>
<tr>
<td>$R_{Mt}, R_{Mb}$</td>
<td>3.0</td>
<td>0.3 1.0 3.0 10.0</td>
</tr>
<tr>
<td>$p$</td>
<td>0 kN/m</td>
<td>0 2.5 10 25 50</td>
</tr>
</tbody>
</table>

[a] For constant values of $T_b = 1.0 \text{s}$ and $T_s = 0.5 \text{s}$

[b] For all combinations of $h/t = 11$ and 22, and $T_s = 0, 0.5$, and 1.0 s

The range of parameter values considered in phase 2 is listed in Table 6.4. Each combination of parameters was run, resulting in a total of 210 unique configurations (though several of these configurations duplicate
6.5 Modelling results

This section presents the most pertinent results of the parametric modelling. Visualizing the data from roughly 200,000 runs of a multi-degree-of-freedom model will inevitably involve a significant number of plots. To aid the reader, the plots in this section have been designed to maintain consistency in style, size, and limits as much as possible, and reference curves that remain the same between various plots are shown in black. Unless indicated otherwise, all plots refer to the far-field motion subset only. Concepts are introduced for the reference configuration, followed by the presentation of results for phase 1 configurations, and finally results for phase 2.

6.5.1 Reference configuration

For any single configuration (in this case, the reference configuration), each ground motion was incrementally scaled until collapse of the wall was observed (see Section 6.4). The primary interest in the analysis of the results lies in quantifying the intensity of the motions that result in collapse. Once the analysis is complete, each motion’s scale at the lowest level causing collapse is known, and it remains to describe the intensity of these motions. Many possible intensity measures could be used to achieve this — e.g.,

A computer with an Intel i7 4770K CPU and 16 GB RAM was able to run the model about 20% faster than real-time.
6.5. Modelling results

peak ground acceleration/velocity/displacement, spectral acceleration/velocity/displacement at a period of choice, Arias intensity, etc. As a starting point, the acceleration response spectrum of each ground motion at the scale causing collapse was compiled. The distribution of spectral values at each period increment was evaluated, and the mean\(^4\), 10th percentile, and 90th percentile of these spectral values at collapse are shown in Figure 6.6. These spectral curves will subsequently be referred to collectively as *collapse spectra*.

\[ S_a(T) \] (g)

\[ T \] (s)

Figure 6.6: Collapse spectra, reference configuration

To allow comparison of results across various configurations, the same intensity measure should be used consistently throughout the analysis. Initially, \( S_a(1.0\text{s}) \) is selected as the intensity measure, with the choice of intensity measure discussed subsequently. The ordinate at \( T = 1.0\text{s} \) on the acceleration spectrum at collapse of each ground motion is noted. A set of \( S_a \) values is thus obtained — one for each ground motion. The distribution of these values can then be plotted as an empirical CDF, and a lognormal CDF can be fitted to the data, as shown in Figure 6.7. These curves represent the vertical distribution of points at \( T = 1.0\text{s} \) in Figure 6.6. The

\(^4\)Spectral values at collapse are assumed to be lognormally distributed. When referring to spectral values, the term *mean* shall be taken to signify the geometric mean, which is equivalent to the median in the case of a lognormal distribution.
6.5. Modelling results

intensity measure at a probability of collapse of interest (in this case, illustrated as 10%) can then be obtained from the fitted CDF. Here, we consider the term probability of collapse, $P_{col}$, to be synonymous with the proportion of ground motions causing collapse (which is what is actually plotted). It is therefore a conditional probability, conditioned on the parameters involved in the particular plot. In this case, it is conditioned on the spectral shape distribution of the far-field motions, among other factors (e.g., crack height, slenderness ratio, etc.).

![Figure 6.7: Reference fragility curve](image)

The choice of $S_a(1.0\text{s})$ as intensity measure is reasonable for a system with diaphragm periods of $T_a = 1.0\text{s}$. For systems with different diaphragm periods, or systems in which the top and bottom periods differ, the choice of measure is not clear. One approach to quantifying the appropriateness of an intensity measure is to consider the variability produced in the fragility curve using that intensity measure. Tightly distributed results — exhibiting less variance (steeper fragility curves) — are preferable to results exhibiting larger variance, in the sense that it is undesirable to use an intensity measure that introduces additional variance simply because the measure is unrelated to the response quantity of interest. A low variance suggests that the selected measure is in fact closely related to the response quantity. The coefficient of variation ($c_v$) is a convenient indicator of this variance, since it is normalized.
6.5. Modelling results

to the mean of the distribution. For a lognormal distribution, it can be calculated as:

\[
c_v = \sqrt{e^{\sigma^2} - 1}
\]  

(6.2)

Here, \( \sigma \) is the standard deviation in log space. For a given configuration, one can calculate \( c_v \) for any possible choice of intensity measure. Considering spectral acceleration as the measure of choice, \( c_v \) can be calculated for every possible period selection. These results are plotted for the reference configuration \( (T_s = 1.0\text{ s}) \) in Figure 6.8a, where the x-axis indicates the period, \( T_{im} \), for the intensity measure, \( S_a(T_{im}) \). It is clear that the lowest variance is produced when a period of \( T_{im} = 1.0\text{ s} \) is selected (i.e. the intensity measure becomes \( S_a(1.0\text{ s}) \)). The variance dips sharply from both shorter and longer periods to reach its minimum where \( T_{im} = T_s \).

The same analysis was conducted for configurations with different system periods (such that \( T_t = T_b \) within each configuration, denoted simply as \( T_s \)). The \( c_v \) curves for each configuration are overlaid onto the curve from the reference configuration in Figure 6.8b. Several noteworthy features can be observed. First, all configurations with system periods of 0.75 s or longer feature similar sharp minimums of \( c_v \) at \( T_{im} = T_s \), and the minimum \( c_v \) values are similar for each of these configurations. For configurations with shorter periods, however, there is no well-defined minimum of \( c_v \) at any \( T_{im} \). Second, configurations with system periods of 1.25 s or longer exhibit considerably larger variance for small \( T_{im} \) than those systems with periods of 1.0 s or shorter.

Examining the trends in this plot suggests that the choice of \( T_{im} = 1.0\text{ s} \) produces close to the lowest overall variance among systems with periods varying between 0 and 2 s. Long period systems are subject to the highest variance with this selection. For the systems with periods of 0, 0.2, and 0.5 s, this selection actually yields close to the lowest possible variance - better than at \( T_{im} = T_s \).

Spectral acceleration is a universal design parameter readily available to practicing engineers, which makes it a logical choice for selection as an intensity measure. In addition, \( S_a(1.0\text{ s}) \) specifically is typically a key benchmark within a design spectrum. It is also the parameter used by the existing assessment procedure for out-of-plane stability of URM walls in ASCE 41 [ASCE, 2014]. Combined with the results of Figure 6.8b discussed previously, these factors form a strong case for the selection of \( S_a(1.0\text{ s}) \) as the intensity measure of choice, and it will be used for most aspects in the remainder of this chapter.
6.5. Modelling results

Figure 6.8: Coefficient of variation, for varying intensity measure choice
6.5. Modelling results

6.5.2 Phase 1a: Effect of diaphragm stiffness

Configurations with periods between 0 (rigid) and 2 s were considered. Fragility curves are plotted in Figure 6.9. At first glance, it is difficult to spot any trends in this plot. It is also notable that there is a wide range of variance among these fragility curves; this is expected, considering that all the curves are all plotted for the same intensity measure. The variance with choice of intensity measure was illustrated earlier in Figure 6.8.

![Figure 6.9: Fragility curves, varying period](image)

Figure 6.9: Fragility curves, varying period

Figure 6.10 shows constant $P_{\text{col}}$ points for 10% and 50% levels, which clarifies the fragility plot. At a given period, the difference between the two curves indicates the variance of the fragility curve for that period. Variance is smallest at $T_s = T_{im} = 1.0$ s, is larger but reasonably consistent for shorter periods, and increases more significantly towards longer periods — consistent with Figure 6.8.

Due to the discrepancies in variance among the various periods, the 50% curve is perhaps most representative of the ‘real’ trend in this figure. Walls in systems where $T_s \leq 0.2$ s are significantly more resilient than those with longer periods. Resilience increases once more at periods beyond 1.25 s. Stability is lowest at $T_s = 0.75$ s, but not much lower than at other points between 0.5 and 1.25 s.
6.5. Modelling results

To confirm whether this trend is only a product of the choice of intensity measure, the same plot was repeated for $P_{col} = 10\%$ and $50\%$ using spectral acceleration at various $T_{im}$ as alternate intensity measures (Figure 6.11). Each curve on these plots is for a different intensity measure. The curves are shifted vertically corresponding roughly to the shape of the mean ground motion spectrum: mid-range periods produce the highest $S_a$ values. For $P_{col} = 50\%$ (Figure 6.11a), the trend is consistent among all intensity measures. For $P_{col} = 10\%$ (Figure 6.11b), local upward spikes in $S_a$ are visible for the longer periods where $T_s = T_{im}$, since the variance dips at these points. Aside from these differences, however, the trend is generally consistent among all the curves: short periods perform best, mid-range periods perform worst, and some improvement occurs towards the longest periods — regardless of the intensity measure used.

As a complementary look at the issue, Figure 6.12 shows the collapse spectra for each of the configurations overlaid on the results from the reference configuration ($T_s = 1.0\,$s, shaded in grey). For clarity, only two or three additional curves are shown on each plot. These plots illustrate the relative performance of walls with varying system periods perhaps least ambiguously. The fact that the shape of the mean curves varies little among all the configurations emphasizes that a single intensity measure is perfectly adequate to describe the relative performance of the different configurations.
Figure 6.11: Target points, varying period: effect of intensity measure
6.5. Modelling results

Figure 6.12: Collapse spectra, varying period
6.5. Modelling results

if the mean curves are used as the basis, and that it could almost equally well be any intensity measure — consistent with the findings in Figure 6.11a.

The variability in the variance among the configurations complicates matters when one is concerned with points on the tails of the distributions. At the $T = 1.0$ s ordinate in Figure 6.12b, for example, the 10% points for $T_s = 0.5$ and $0.75$ s configurations fall significantly below the 10% point for the $T_s = 1.0$ s configuration. This is despite the fact that there is little difference between the mean curves.

This illustrates a drawback of using a single intensity measure in the analysis of results. The variability in the variance exists because the $S_a$ values that are plotted are from the spectra of real ground motions, which are subject to significant peaks and valleys throughout the period range. When $T_s \neq T_{im}$, the mean collapse spectrum implicitly relates the mean shape of the spectra at $T_s$ to that at $T_{im}$. This translates well when comparing the results with a uniform hazard spectrum, which is an aggregation of hazards evaluated independently at each period. At low $P_{col}$, however, the collapse spectrum is implicitly picking out motions that have relative peaks at $T_{im}$ and relative valleys at $T_s$. This is technically correct, but care must be taken if these results are interpreted in the context of a comparison to a uniform hazard spectrum. If taken at face value, low $P_{col}$ values at $T_s \neq T_{im}$ may produce conservative assessments of wall stability.

When forming assessment criteria based on such results, some judgement should be exercised in the interpretation of the low $P_{col}$ values. An approximation of ‘design’ $P_{col} = 10\%$ values based on Figure 6.10 might be to take the $P_{col} = 50\%$ curve and shift it down until it hits the $P_{col} = 10\%$ value at $T = 1.0$ s. This would preserve the mean relative responses between systems with different periods while accounting for the lower collapse probability based on the best matched response at $T_s = T_{im}$. This issue will be explored further in the following sections as the effects of ground motion characteristics are investigated.

6.5.2.1 Near-fault effects

The findings of the analysis are conditioned on the characteristics, including the spectral shapes, of the ground motions used in the study. It is of some interest to examine the effects of near-fault motions. In addition to the far-field records, the FEMA P695 ground motion set includes 56 ground motions from 28 near-fault records: 14 of these are characterized as pulse motions, while 14 are characterized as non-pulse. For each record, the rotated components (fault-normal and fault-parallel) were used. It is important to note
that each of these subsets has a considerably smaller sample size (14) than the far-field set of 44 motions.

![Figure 6.13: Target points, varying period: with near-fault motions](image)

The plot of Figure 6.10 is repeated in Figure 6.13 with additional curves showing the results for the near-fault motions. In general, the trends identified earlier hold true for all ground motion types: short periods perform best, mid-range periods perform worst, and some improvement occurs towards the longest periods. For the most part, the far-field motions result in the least stable systems, with the exception that pulse motions are the least stable at periods of 1.5 s or longer. Non-pulse motions produce curves that are very similar to the far-field motions. Pulse motions, on the other hand, exhibit a nearly flat response for periods of 0.75 s or longer, with less of the increase in stability with long periods seen in the other motions. This is consistent with pulse motions generally containing relatively more energy at long periods compared with non-pulse (including far-field) motions.

The differences in response are examined further through the plotting of the collapse spectra in Figure 6.14. These plots show the collapse spectra for the pulse motions overlaid on the far-field spectra (shaded in grey), for the reference configuration, with $T_s = 1.0$ s (Figure 6.14a), and the configuration with $T_s = 0.2$ s (Figure 6.14b). Figure 6.14a shows that the pulse motions have noticeably flatter collapse spectra on average than the far-field motions.
6.5. Modelling results

Figure 6.14: Collapse spectra, pulse motions
6.5. Modelling results

This is reflective not only of the ‘collapse’ spectra, but rather the shape of the mean curve here is also simply an indication of the shape of the mean response spectra of the motions. It is thus evident that the pulse motions do, in fact, have significantly more energy content at long periods relative to short periods than the far-field motions. The near-fault spectra are very close to the far-field spectra around the ordinate of $T = 1.0$ s. At shorter periods, the near-fault spectra are thus significantly lower than the far-field ones as a result of the flatter shape.

Figure 6.14b shows that for the short-period configuration, the near-fault spectra are again lower than the far-field spectra for short periods, and are very similar in the mid-range periods. The near-fault spectra actually creep higher than the far-field ones in the longer periods. It so happens that at the ordinate of $T = 1.0$ s, the near-fault spectra are already a little higher than the far-field ones, thus creating the results at $T = 0.2$ s of Figure 6.13.

Perhaps the most noteworthy observation from this examination is that it seems to indicate fairly definitively that the wall response for systems with stiff diaphragms depends to a greater extent on the spectral values at longer periods than it does on the spectral values at the period of the diaphragm. For ground motions with significantly differing spectral shapes, the configuration with $T_s = 0.2$ s reached collapse at similar values of $S_a(1.0)$, but at very different values of $S_a(0.2)$ (Figure 6.14b). This behaviour is consistent with the long periods observed in testing for large-amplitude rocking excursions (Section 4.3.2.4), for both rigid and flexible diaphragms. For stiff systems, it thus appears that the original diaphragm period is of limited relevance once rocking occurs. This supports the use of a single intensity measure, at least for systems with relatively stiff diaphragms, and moreover confirms that $S_a(1.0)$ is in fact a reasonable choice.

6.5.2.2 Spectral shape effects

The previous section established that near-fault pulse motions exhibit different characteristics than far-field motions, and that this has a notable effect on out-of-plane wall stability under certain conditions. In particular, the flatter shape of the response spectra of the pulse motions produces predictable differences in spectral values at collapse once it is noted that the wall response is heavily dependent on the spectral acceleration at longer periods. In this section, the effect of the spectral shape of the far-field motions themselves is examined. Here, the spectral shape is quantified by the $\varepsilon$ parameter, which is dependent on the period ($\varepsilon$ values were plotted for far-field motions in Figure 6.3b).
6.5. Modelling results

For a given configuration, a regression analysis can be done for $S_{a,\text{col}}$ vs. $\varepsilon$. Typically, in such a procedure, $S_{a,\text{col}}$ is taken at the fundamental period of the structure [Haselton et al., 2011]; here it is therefore taken at $T_s$, which is dependent on the configuration. In keeping with the assumption of lognormally distributed $S_a$ values, a linear regression is done between $\ln S_a$ and $\varepsilon$. This results in a curved regression line in non-log space. Regression plots for selected periods are shown in Figure 6.15.

A particular regression can be summarized by the slope of the fit line and the coefficient of determination, $R^2$. These values are illustrated for each period in Figure 6.16. $R^2$ represents the proportion of the variability in the regressand ($\ln S_a$) that is explained by the regressor ($\varepsilon$). Since the regression is linear in the log space, the slope varies in the real space; a reference ‘slope’ is specified as the change in $S_a$ between $\varepsilon = 0$ and $\varepsilon = 1$. This slope is illustrated in both absolute (Figure 6.16b) and relative (Figure 6.16c) terms.

The regression has minimal significance at periods of $T = 0.75\,s$ or greater, with $R^2$ exceeding 0.1 only at $T = 0.2$ and 0.5 s. These results are consistent with a system that is primarily sensitive to long-period content, as explained below.

In general, a large $\varepsilon$ at one period indicates that the spectrum will tend to be peaked around that period, and will be relatively lower at other periods (e.g., the significant peak around $T = 1.2\,s$ in Figure 6.1) [Baker and Cornell, 2005]. Consider a hypothetical system that is sensitive to the spectral acceleration at 1 s, but not to that at other periods. All ground motions would cause collapse at roughly the same $S_a(1\,s)$ values, regardless of their spectral values around 0.2 s. At the collapse scale, the motions that have large $S_a(0.2\,s)$ values will then also have large relative $S_a(0.2\,s)/S_a(1\,s)$ values. This means that these motions will also tend to have large $\varepsilon$ values around 0.2 s. Consequently, there would be a direct correlation between $\varepsilon(0.2\,s)$ and $S_{a,\text{col}}(0.2\,s)$.

At the same time, the set of collapse motions would include a full range of $\varepsilon$ at 1 s, which are indicative of whether the spectrum of a particular motion is relatively high or low at 1 s. Since the motions all caused collapse at roughly the same $S_a(1\,s)$ values, there would be little correlation between $\varepsilon(1\,s)$ and $S_{a,\text{col}}(1\,s)$. Both of these predictions for this hypothetical system are observed in Figures 6.15 and 6.16 for short period systems. Notably, the correlation is roughly equally weak for all long period systems. This indicates that there is not one particular period to which all the systems are sensitive, but that for long periods, the system is sensitive at its own period. These observations are furthermore consistent with what was observed in the
6.5. Modelling results

Figure 6.15: $\ln(S_{a,\text{col}})$ vs. $\varepsilon$ regressions
6.5. Modelling results

Figure 6.16: $R^2$ and slope of $\ln (S_{a_{\text{col}}})$ vs. $\varepsilon$ regression
6.5. Modelling results

coefficient of variation plot in Figure 6.8. There, it was noted that for short-period configurations, the spectral acceleration around $T = 1.0\text{s}$ was as good or better of an indicator of collapse than the system period, while for long period systems the spectral acceleration at $T_s$ was the best indicator.

It can be concluded that the effect of spectral shape is negligible when appropriate intensity measures are used, given that the rocking walls are sensitive to long-period energy content. If one were to use $T_{im} = T_s$ at short periods, the results should be adjusted for spectral shape — but this would simply be a convoluted method of returning to the spectral content at the longer periods. It is more appropriate to use the long period intensity measures in the first place.

6.5.2.3 Variable top and bottom period

The previous sections have all dealt with configurations in which the top and bottom diaphragm periods are the same. In reality, diaphragms are likely to vary in stiffness, particularly in the top storey where one diaphragm is a roof while the other is a floor. The top and bottom inputs into the out-of-plane wall can thus be out of phase, which has the potential to change the response significantly. In this section, configurations with different periods at the top and bottom of the wall are examined. The top period is varied while the bottom period remains constant. Bottom periods of $T_b = 0.5$ and $1.0\text{s}$ were considered.

Fragility curves are shown in Figure 6.17 for $T_b = 1.0\text{s}$ and in Figure 6.18 for $T_b = 0.5\text{s}$. When compared with Figure 6.9, in which both periods vary simultaneously, it is immediately apparent that changing only the top period has a lesser effect on the results than changing both periods. Constant $P_{col}$ points for these three plots are shown in Figure 6.19, which illustrates the differences well.

The variance of the fragility curves remains more consistent when only changing the top period than when changing both periods. Notably, the particularly small variance observed at $T_s = 1.0\text{s}$ (resulting from the choice of $S_a(1.0\text{s})$ as the intensity measure) persists even as $T_t$ is decreased to $0.2\text{s}$. This suggests that between the top and bottom periods, the bottom period is the more dominant in terms of the effect on out-of-plane wall stability.

It was noted in Section 4.3.2.1 that wall collapse is dictated only by the stability of the bottom wall segment. In a cracked wall, the motion of the top diaphragm is only indirectly transmitted to the critical bottom segment, while the bottom diaphragm is directly connected to it. In this regard, the
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Figure 6.17: Fragility curves, varying top period only, $T_b = 1.0\,s$

Figure 6.18: Fragility curves, varying top period only, $T_b = 0.5\,s$
6.5. Modelling results

greater observed influence of the bottom diaphragm stiffness is consistent with expectations.

Selected fragility curves are isolated to illustrate these effects in greater detail. Figure 6.20 shows all four combinations of periods of 0.5 and 1.0 s distributed between the top and bottom of the wall. The four curves all show different variances, but intersect at roughly the same location near $P_{\text{col}} = 45\%$. The configurations with equal top and bottom periods bracket the responses, with the varying top and bottom periods falling in between. Beginning with a reference of an equal period curve, changing the bottom period pushes the new curve further towards the second equal period curve than does changing the top period. In this particular case, the curve for $T_t = 0.5\,s/T_b = 1.0\,s$ has changed very little relative to that of $T_t = 1.0\,s/T_b = 1.0\,s$, particularly for low $P_{\text{col}}$.

Figure 6.21 shows periods of 1.0 and 1.25 s. Here, changing the top period from 1.0 to 1.25 s pushes the fragility curve close to that for $T_t = 1.25\,s/T_b = 1.25\,s$, particularly at low $P_{\text{col}}$. While the earlier observations suggested that the bottom period appears to be the generally more dominant of the two, it is important to note that the top period can still have a large effect on wall stability.

In general, it can be concluded that the out-of-phase action produced
6.5. Modelling results

Figure 6.20: Fragility curves, $T_s = 0.5$ and $1.0$ s combinations

Figure 6.21: Fragility curves, $T_s = 1.0$ and $1.25$ s combinations
6.5. Modelling results

by diaphragms of different stiffness at the top and base of a wall will result in out-of-plane wall stability in between that of configurations when both diaphragms are equal to one of these periods. It is critical to clarify that the term \textit{stability}, in this sense, refers to the aggregated collapse levels of a sufficiently large suite of ground motions — i.e. the entire fragility curve. The conclusion does not necessarily hold for any single ground motion, where the stability at various periods is dependent on the distribution of peaks and valleys in the response spectrum. On fragility curves of systems with different periods, one can not assume that each ground motion falls at the same location on each curve — in fact, it is unlikely. Comparisons of these analysis results with the test results must therefore be made cautiously. Because the tests used only a single ground motion, the fact that wall \textit{FR-3} performed better than both \textit{FF-3} and \textit{RR-3} can still be considered consistent with the analysis results.

Typically there is significant uncertainty in the assessment of diaphragm periods, and it would not be prudent to rely on a calculated difference in period to produce an increase in assessed wall stability. For a given wall, it would be reasonable to use the least stable of the two calculated periods at the top and bottom when conducting an assessment. Accordingly, configurations with different top and bottom periods are not considered further in this study.

6.5.2.4 Allowable spectrum

All analysis discussed up to this point has involved the comparison of the various configurations at some single intensity measure. In particular for the simple model in question, and for configurations which have top and bottom periods equal, an alternative approach could be to tie the choice of intensity measure to the system configuration — i.e. set $T_{im} = T_s$ for each configuration. Plotting constant $P_{col}$ curves in such a way produces what could be termed an ‘allowable spectrum’. The $P_{col} = 10, 50, \text{ and } 90\%$ curves are plotted in Figure 6.22. Here, the allowable spectrum for the $P_{col}$ of choice is analogous to a capacity.

This plot illustrates the period dependency of the rocking wall system. At short periods (e.g., $T_s = 0.2$s), the variance is very large. As discussed in Sections 6.5.2.1 and 6.5.2.2, this is symptomatic of a system that is sensitive to long-period content. At long periods, the variance is consistently small — for such periods, this ‘allowable spectrum’ approach holds appeal.

If desired, the $\varepsilon$ regression procedure (see Section 6.5.2.2) could be used to create $\varepsilon$-specific allowable spectra. To do so, the $S_{a_{col}}$ value for each
6.5. Modelling results

![Graph showing allowable spectrum]

Figure 6.22: Allowable spectrum

ground motion for each configuration would be adjusted according to the regression curve and the actual and target $\varepsilon$ values. This creates a new empirical distribution for $S_{a_{col}}$ for each configuration; from this point, the process for creating the allowable spectrum is the same. Such a curve would have reduced variance at $T_s = 0.2\,s$, and potentially a significant shift in values at this period depending on the target $\varepsilon$. These effects would also be notable, though reduced, for $T_s = 0.5\,s$. At other periods, differences would be minimal.

The value of using the allowable spectrum approach should be carefully evaluated. At short periods, it was already concluded that using $T_{im} = T_s$ is a poor choice since the rocking response of the wall is not sensitive to content at short periods. For periods longer than 1 s, additional considerations relating to effects of large diaphragm displacements will likely become significant (e.g., damage at wall corners due to horizontal bending deformations). While these effects are beyond the scope of this study, it would be prudent to err on the conservative side when deciding allowable limits for long-period systems. Given that long-period systems have shown the same or better resiliency as systems with $T_s = 1\,s$, it is perhaps not necessary to quantify this additional resiliency by using the allowable spectrum, but rather to conservatively assign the same limits for long period systems as for those at $T_s = 1\,s$. With this intention in mind, the allowable spectrum
method is not pursued in the remainder of this study.

6.5.3 Phase 1b: Effect of other parameters

This section covers the single-parameter variations from the reference wall, as listed in Table 6.3. Results are presented and discussed for each parameter in turn. Recall that each of the parameter variations was run for the default reference configuration with $T_s = 1.0\,\text{s}$ as well as for the secondary reference configuration with $T_s = 0.5\,\text{s}$. Results in this section are plotted by default for configurations with $T_s = 1.0\,\text{s}$, unless otherwise indicated. Results at $T_s = 0.5\,\text{s}$ were examined to ensure that trends are consistent, but for brevity are not included here.

6.5.3.1 Crack height

The parameters examined in preceding and subsequent sections are tied to physical properties of the structure. They are subject primarily to epistemic uncertainty: ‘correct’ values of each parameter exist, but there is uncertainty in determining them due to measurement difficulties, poor representation in the model, etc. The crack location, on the other hand, is subject to aleatoric uncertainty: it does not have a ‘correct’ value at the assessment stage, since walls are likely uncracked. While the theoretical location of the crack can be calculated as the location of maximum stress (with some assumptions about acceleration distributions), the crack may well form at a different location due to spatial variability in materials and workmanship in the wall. This was demonstrated in the experimental phase of this project (see Section 4.3.6).

The crack location is thus aptly considered as a random variable. One could reasonably assume that the expectation of this variable should fall at the predicted location of maximum stress, with variance on either side of this value due to the construction details of the wall. As a basis to determine this variance, results from three sets of experimental tests were used: ABK Joint Venture [1981b], Meisl et al. [2007], and the tests from the current project. Sharif et al. [2007] compiled the crack locations from the first two sets and calculated a mean relative crack height of 0.63 with a standard deviation of 0.07. The current testing produced a mean crack height of 0.55 with a standard deviation of 0.11. The location of maximum observed stress in the current tests was at a relative height of roughly 0.55.

Considering all of these results, it is reasonable to assume normally-distributed crack heights with a mean of 0.60 and a standard deviation of 0.10. Crack heights of 0.4, 0.5, 0.6, 0.7, and 0.8 were considered, correspond-
6.5. Modelling results

To represent the assumed normal distribution with these discrete values, the distribution in Table 6.5 was assigned.

### Table 6.5: Probability of crack height occurring

<table>
<thead>
<tr>
<th>$x$</th>
<th>$P(h_{cr} = x)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4</td>
<td>0.07</td>
</tr>
<tr>
<td>0.5</td>
<td>0.24</td>
</tr>
<tr>
<td>0.6</td>
<td>0.38</td>
</tr>
<tr>
<td>0.7</td>
<td>0.24</td>
</tr>
<tr>
<td>0.8</td>
<td>0.07</td>
</tr>
<tr>
<td>Total</td>
<td>1.00</td>
</tr>
</tbody>
</table>

The fragility curve for each crack height is obtained from the analysis (Figure 6.23). At each $S_a$ value, we can then calculate the total probability of collapse as:

$$P_{col} = \sum_i [P_{col}(h_{cr} = x_i)] \cdot [P(h_{cr} = x_i)] \quad (6.3)$$

Plotting this result for each $S_a$ value yields the thick grey line in Figure 6.23, which is the total fragility curve. Note that the total curve has greater variance than any of the curves for single crack locations, because it has incorporated the additional uncertainty associated with the crack height distribution, whereas the individual curves are conditional on a particular crack height.

The constant $P_{col}$ values are plotted in Figure 6.24. A cubic function fits the data well within the domain considered. For this particular case, the relation is approximately:

$$S_a = -1.37 (h_{cr})^3 + 3.67 (h_{cr})^2 - 3.39 (h_{cr}) + 1.31 \quad (6.4)$$
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Figure 6.23: Fragility curves, varying crack height

Figure 6.24: Target points, varying crack height
6.5. Modelling results

6.5.3.2 Slenderness ratio

Walls of constant thickness and varying height were evaluated, where slenderness ratios varied between 4 and 22. Fragility curves are plotted in Figure 6.25. More slender (taller) walls consistently exhibited lower resilience than less slender (shorter) walls. Variance is fairly consistent among the distributions, with the exception of the curve at $h/t = 4$, which shows notably more variance than the others. Plotting the constant $P_{col}$ values (Figure 6.26) produces a smooth trend that is well defined by a power relation (the line shown). For this particular case, the relation is approximately:

$$S_a = 2.77 \cdot \left(\frac{h}{t}\right)^{-0.9}$$

(6.5)

Figure 6.25: Fragility curves, varying slenderness
6.5. Modelling results

6.5.3.3 Thickness

Walls with the same slenderness ratio but different thicknesses (and therefore heights) were evaluated — effectively, one wall was a scaled version of the other, with the exception of the spalling geometry, which remained fixed at its absolute dimensions. The thicknesses were representative of typical one-, two-, and three-wythe walls: 110, 220 and 330 mm, respectively. The thinner walls were evaluated at each combination of $h/t = 11$ and $22$, and $T_s = 0$, 0.5, and 1.0 s. The fragility curves for $T_s = 0.5$ and 1.0 s are plotted in Figures 6.27 and 6.28, both at $h/t = 11$. Relative collapse intensities of two-wythe vs. three-wythe and one-wythe vs. three-wythe walls are shown in Figure 6.29 for the various system periods and $h/t$ ratios. It is clear that thinner walls exhibited consistently lower stability than the thicker walls. The difference was most pronounced at $T_s = 0.5$ s, less so at $T_s = 0$, and the least so at $T_s = 1.0$ s. The effect showed minimal variation with $h/t$.

Makris and Konstantinidis [2003] conducted 2-dimensional analyses on rocking of simple rectangular blocks subjected to ground motions. They noted that for a given slenderness ratio, blocks exhibited roughly monotonically decreasing peak rocking rotations as size increased — i.e. larger blocks were more stable than smaller blocks of the same proportions. The results
6.5. Modelling results

Figure 6.27: Fragility curves, varying thickness, $T_s = 1.0$ s

Figure 6.28: Fragility curves, varying thickness, $T_s = 0.5$ s
6.5. Modelling results

The horizontal size of the chamfer representing mortar spalling ($s_h$, see Figure 6.5) at the crack location was varied between 5 and 15 mm. The fragility curves are shown in Figure 6.30. Greater spalling produced less resilient walls, as would be expected, but the magnitude of the influence was negligible within the considered range.

Figure 6.29: Relative collapse intensity, one- and two-wythe walls vs. three-wythe walls

of this section are consistent with those findings, and the effect is of considerable importance, particularly with short- to moderate-period systems.

6.5.3.4 Spalling

The horizontal size of the chamfer representing mortar spalling ($s_h$, see Figure 6.5) at the crack location was varied between 5 and 15 mm. The fragility curves are shown in Figure 6.30. Greater spalling produced less resilient walls, as would be expected, but the magnitude of the influence was negligible within the considered range.
6.5. Modelling results

6.5.3.5 Damping

Damping values of 3, 5, 7, and 10% were considered; fragility curves are plotted in Figure 6.31. Variance remains fairly consistent among all curves, with walls becoming notably more resilient at higher damping values. Constant $P_{col}$ values are plotted in Figure 6.32.

ASCE 41 allows the use of 10% damping for buildings with wood diaphragms and cross walls that interconnect the diaphragm levels at a maximum spacing of 12 m transverse to the direction of motion, however the rationale behind this value is not known. Wilson [2012] conducted full-scale dynamic testing on wood diaphragms representative of those in typical URM buildings, in which he found that “the results provide no evidence against the 5% inherent damping that is typically assumed for dynamically responding structures, and this value is therefore recommended for timber floor diaphragms”. Consequently, 5% damping was adopted as the reference value for the remainder of the study.
6.5. Modelling results

Figure 6.31: Fragility curves, varying damping

Figure 6.32: Target points, varying damping
6.5. Modelling results

6.5.3.6 Diaphragm mass ratio

The diaphragm mass ratio in the model is defined as the mass of one diaphragm divided by half the total wall mass. While this is unambiguous, it is less clear what this ratio should represent in a prototype building. The ratio is intended to capture the relative weight of translation-only elements vs. rocking elements. Two methods for calculating the mass ratio were used, illustrated in Figure 6.33 for the diaphragm at the base of an upper-storey wall. The elements contributing to the diaphragm mass are hatched in red, while those contributing to the wall mass are hatched in blue.

![Diagram showing mass ratio definitions](Figure 6.33: Mass ratio definitions)

In method 1, the tributary wall mass — half of the storey height — both above and below the diaphragm is counted as wall mass. However, the wall in the storey below will be more stable than the one above due to the larger overburden, and it is therefore less likely to undergo rocking. If it is not undergoing rocking, then it is more aptly modelled as a lumped mass attached to the diaphragm. In this case, method 2 may be a better approximation, where the tributary wall mass below the diaphragm is counted towards the diaphragm mass.

The range of possible ratios was examined for a sample prototype building 10 × 20 m in plan. Light (2-wythe, 3 m tall) and heavy (3-wythe, 4.5 m tall) walls were considered. Diaphragm loads were considered as either bare
6.5. Modelling results

floor (0.5 kPa), floor with partitions (1.0 kPa), or floor with partitions and 50% live load for office occupancy (2.2 kPa). Each of these combinations was considered in both the long and short span directions of the diaphragm, and for calculation methods 1 and 2.

For method 1, ratios fell between 0.2 and 3.2, while for method 2 they fell between 1.3 and 8.6. A reference ratio of $R_M = 3$ was used, and ratios of 0.5, 1, 3, and 10 were modelled. Fragility curves are shown in Figure 6.34. The plot demonstrates that in general, lighter diaphragms result in more resilient walls. Intuitively, this makes sense — a rocking wall will be able to ‘push back’ against a lighter diaphragm more readily than on a heavier one while undergoing rocking. The differences are notable, but not large, particularly at low probabilities of collapse. At $P_{col} = 10\%$, $S_a$ values for $R_M = 10$ and for $R_M = 1$ are within 7% of that at $R_M = 3$, while for $R_M = 0.5$, $S_a$ is 18% higher.

![Fragility curves](image)

Figure 6.34: Fragility curves, varying diaphragm mass ratio, $T = 1.0 \text{s}$

6.5.3.7 Axial load

The details of how the axial load is applied result in very significant differences in its effect. The two types of load application considered were introduced in details (a) and (b) of Figure 6.5. Five specific load applications were considered in this section (illustrated specifically in Figure 6.35):
6.5. Modelling results

- An applied force at a fixed location: (1) at the wall centerline, and (2) at 0.4·t away from the centerline.

- A load applied to a block that remains horizontal, where the block starts at one edge of the wall below and extends across the wall thickness by (3) one wythe, (4) two wythes, and (5) three wythes.

Figure 6.35: Axial load application

Overburden loads consist of live loads plus dead loads from roofs, floors, walls, and parapets. Since overburden increases stability, it is conservative to err on the low side when estimating loads. Live loads were therefore not considered in this analysis. At the lower bound, the overburden approaches zero — e.g., a top-storey (or single-storey) wall running parallel to roof joists, with a low or non-existent parapet. Heavily-loaded upper storey walls may see overburden in the range of 15 kN/m with a large parapet and a roof load. Walls in the 2nd-from-top storey may see overburden in the range of 20 to 50 kN/m, when including a parapet, roof and floor loads, and a top storey wall.

Fragility curves for Case 2 are shown in Figure 6.36, and for Case 3 in Figure 6.37. Constant $P_{col}$ values are plotted in Figure 6.38 for all 5 cases at all axial load levels, including the best-fit straight lines (constrained through the point at 0 load). The greatest stability gains for a given load are produced by Case 5 (3-wythe block), followed by Case 4 (2-wythe block). Case 3 (1-wythe block) produced nearly the same effect as Case 1 (concentric fixed force).
6.5. Modelling results

Figure 6.36: Fragility curves, varying axial load, Case 2

Figure 6.37: Fragility curves, varying axial load, Case 3
6.5. Modelling results

The results illustrate that the effects of axial load are the result of a combination of the effects in each of the two possible rocking directions. In every case, the axial load is stabilizing until the rotation of the top wall segment becomes such that the contact point between the top and bottom wall segments is directly below the point of action of the axial load on the top block. These rotation limits are illustrated in Figure 6.39 for Cases 1 and 2. In Case 1, the limit is the same in both directions (\(\theta_{1L} = \theta_{1R}\)), while in Case 2, the rotation limit is very small in one direction (\(\theta_{2L} < \theta_{1L}\)), but larger than the Case 1 limit in the other direction (\(\theta_{2R} > \theta_{1R}\)). For rigid diaphragm conditions, Doherty et al. [2002] related the static force-displacement curves of Case 1 and Case 5 systems to those of equivalent cantilever walls with suitably adjusted geometry. In this study, the axial load application is modelled explicitly for all cases.

While the top wall segment has not exceeded the rotation limit, the stabilization occurs as a result of the moment applied by the axial load about the point of contact between the wall segments. The moment arm, \(d_m\), is the horizontal distance between (1) the point of application of the axial force and (2) the point of contact between the wall segments (Figure 6.40). The moment arm is greatest at the onset of a rocking excursion, as the crack just begins to open. Beyond this point, any additional rotation of the top
### 6.5. Modelling results

![Figure 6.39: Stabilizing limits of axial load application](image)

In Case 2, the eccentric force applies a large restoring moment in one direction, but only a minimal one in the opposite direction — even at onset of cracking. In the latter direction the wall is thus able to rock easily, and once it rocks even slightly, the axial force is now destabilizing the upper block (i.e. accelerating the rotation of the upper segment). The relative net effect of stabilization vs. destabilization appears to be dependent on the magnitude of the axial load, resulting in progressively smaller stability gains.
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per unit axial force as the axial force increases (Figure 6.38).

Case 3 appears to show a milder version of this behaviour: in Figure 6.38, the slope from 0 to 50 kN/m is 16% lower than the slope from 0 to 10 kN/m. Cases 1, 4, and 5 — all of which are more symmetrical than Cases 2 and 3 — produce notably more linear results. Though the sample size is small, it would seem that in general, more symmetrical load application leads to more linear stability gains, while more asymmetrical load application leads to lower stability gains for higher loads than for lower loads.

6.5.4 Phase 2: Parametric combinations

This section provides the results of the Phase 2 runs, which consisted of a full combination matrix of selected parameters — crack height, slenderness ratio, period, and axial load (Table 6.4). The results were processed into a series of constant $P_{col}$ points, arranged in parallel plots as a function of $h/t$ and diaphragm period. Both sets of plots are included in this section since each illustrates different trends. On $h/t$ plots, the best-fit power function is drawn for each data series. Each plot type is presented as a series of four plots, one for each $P_{col}$ value of 5%, 10%, 20%, and 50%. For reference, ASCE 41 allowable slenderness limits (see Section 2.2.3) for top storey and one storey walls are included in the $h/t$ plots. Recall that these limits do not distinguish between diaphragm stiffnesses nor axial load levels.

As discussed in Sections 6.5.2.1 and 6.5.2.2, the effects of variations in spectral shape, including due to near-fault motions, is minimal when evaluating stability as a function of $S_a(1.0s)$. Consequently, results are presented for far-field motions only in this section.

Crack height was considered as a random variable, and the different crack heights were combined into a total probability curve using an assumed distribution (see Section 6.5.3.1). The distribution of Table 6.5 was modified to include only the three crack heights evaluated in Phase 2 (0.5, 0.6, 0.7) as opposed to the five used in Phase 1 (0.4, 0.5, 0.6, 0.7, 0.8). This was achieved by assigning the probabilities of occurrence for the 0.4 and 0.8 crack heights to the 0.5 and 0.7 crack heights, respectively (Table 6.6). A comparison of the total probability curves for these two distributions showed negligible differences for low to moderate $P_{col}$, with the modified curve returning somewhat higher $P_{col}$ values for $P_{col} > 0.7$. Since there is little interest in the behaviour at high $P_{col}$ values, the modified curve was considered an acceptable approximation of the original.
6.5. Modelling results

Table 6.6: Probability of crack height occurring

<table>
<thead>
<tr>
<th>( x )</th>
<th>( P(h_{cr} = x) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( 0.4 )</td>
<td>0.07 —</td>
</tr>
<tr>
<td>( 0.5 )</td>
<td>0.24 0.31</td>
</tr>
<tr>
<td>( 0.6 )</td>
<td>0.38 0.38</td>
</tr>
<tr>
<td>( 0.7 )</td>
<td>0.24 0.31</td>
</tr>
<tr>
<td>( 0.8 )</td>
<td>0.07 —</td>
</tr>
<tr>
<td>Total</td>
<td>1.00 1.00</td>
</tr>
</tbody>
</table>

**Base case:** Results are shown for the base case of no overburden, with \( T_b = T_t \). Slenderness ratio and period plots are provided in Figures 6.41 and 6.42, respectively. Both of these plots illustrate that the magnitude of the effect of the diaphragm period is heavily dependent on the slenderness ratio, with the effect being more important for less slender walls. In particular for the lower \( P_{col} \) values, it is evident that when \( h/t \) reaches 22, there is minimal difference in allowable \( S_a(1.0) \) between all the diaphragm periods. At low \( P_{col} \), it is also clear that the \( T_s = 0 \) and \( 0.2s \) configurations consistently result in the highest stability. At higher \( P_{col} \), the \( T_s = 2s \) configuration reaches the same level of stability as the stiff systems, with the \( T_s = 0.5 \) and \( 1.0s \) configurations remaining at notably lower stability levels.

**Overburden:** Results are shown for the runs otherwise equivalent to the base case runs, excepting the application of an overburden of \( P = 10 \text{kN/m} \). The overburden was applied entirely as joist pocket loading (Case 3 in Figure 6.35). Slenderness ratio and period plots are provided in Figures 6.43 and 6.44, respectively. To aid in comparing the results to the base case, plots of the ratio of \( S_a \) with overburden to \( S_a \) for the base case at constant \( P_{col} \) are shown, also as functions of \( h/t \) and \( T_s \), in Figures 6.45 and 6.46. This ratio will generally be denoted as the ‘stability gain ratio’ (SGR), not limited to overburden effects only. Linear regressions are plotted for the \( h/t \) curves. These plots demonstrate that, like the effect of period, the stabilizing effect of overburden depends heavily on \( h/t \), with greater relative stability increases occurring for smaller \( h/t \).

ABK Joint Venture [1981b] considered overburden as a ratio of overburden to wall self-weight \((P/W)\). Here, the wall weight increased linearly with \( h/t \), as the height increased and the thickness remained constant, while the
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--- top storey  ---- one storey \{ ASCE41 limits

\[ T_s = 0 \text{s} \quad T_s = 0.2 \text{s} \quad T_s = 0.5 \text{s} \quad T_s = 1 \text{s} \quad T_s = 2 \text{s} \]

![Graphs showing Sa(1.0) for different values of h/t and Pcol].

Figure 6.41: \( h/t \) curves, top storey, \( P = 0 \)
Figure 6.42: $T$ curves, top storey, $P = 0$
applied overburden was constant. Using a constant overburden and the actual wall weights provides a better illustration of the effect of increasing wall height alone in a building. In addition, it is clear that the effects can not be explained simply using $P/W$ rather than $P$. Consider Figure 6.45b as an example. Here, the wall self-weight doubles from $h/t = 11$ to 22. For this same range, the stability gain ratio goes from roughly 1.7 to 1.2 at $T_s = 1$ s, while for $T_s = 0.5$ s, the ratio goes from 1.2 to 1.0.

This also highlights a very defined trend best illustrated in Figure 6.46, namely that there is a very sharp dip in the stability gain around $T_s = 0.5$ s. The dip is most severe at low $P_{col}$. The points at $T_s = 0.2$ s are already on the descending arm of this dip with considerably lower stability gain than the points at $T_s = 0$, while the $T_s = 1$ s points are effectively out of the dip already. Presumably, the inclusion of additional period increments around 0.5 s would have rounded out the sharpness of the dip seen in this plot, but it would nevertheless still form a very notable feature. The combination of the low stability gains at $T_s = 0.5$ s and the decreasing effect of overburden with $h/t$ actually result in effectively no stability gain at $h/t = 22$ at this period for low $P_{col}$.

It is difficult to intuitively explain all of these trends. In general, it would be expected that the effect of constant overburden would decrease with increasing wall height. The stabilizing moment provided by the overburden does not change, as neither the available moment arm — related to the constant wall thickness — nor the load are changing. In contrast, the destabilizing loads are changing: there is more wall mass loaded across a taller span, both of which result in lower stability. The variation with period is less easily explained; it is possible that a reduction in the ‘effective rocking period’ of the wall due to the additional axial load results in an increased rocking response around a period of 0.5 s. With a diaphragm period near this value, the resulting amplification could counteract a portion of the stability gain from the axial load. Barring more detailed investigation into the matter, which is outside the scope of this study, the results produced here should serve as a caution to allow only conservative stability gains due to axial load.

**Bottom vs. top storey:** Runs otherwise equivalent to each of the previous cases, excepting that the bottom diaphragm period was set to rigid ($T = 0$ s), were also considered. This is representative of a wall in a one-storey building, where the base of the wall is on a foundation and the top is connected to a flexible diaphragm. These cases were examined for the longer periods only (0.5, 1, and 2 s), since minimal difference was expected from fixing the base
6.5. Modelling results

Figure 6.43: $h/t$ curves, top storey, $P = 10\text{kN/m}$
6.5. Modelling results

Figure 6.44: $T$ curves, top storey, $P = 10\text{kN/m}$
6.5. Modelling results

Figure 6.45: SGR due to $P = 10\,\text{kN/m}$, as $h/t$
6.5. Modelling results

Figure 6.46: SGR due to $P = 10 \text{kN/m}$, as $T$
6.5. Modelling results

in a short-period configuration. For brevity, the results are shown only as relative stability increases. Figures 6.47 and 6.48 show the results comparing the configurations with no overburden, while Figures 6.49 and 6.50 compare the results with overburden.

It can be noted that the slenderness ratio has minimal influence overall on the effect of fixing the bottom diaphragm. The period has notable influence, however, with the greatest stability gains occurring at $T_s = 0.5s$. For the most part, fixing the bottom diaphragm results in mild stability gains. The exception is that at $T_s = 2s$ for larger $P_{col}$, it actually results in slight stability decreases. This is consistent with earlier observations of systems with mixed top and bottom periods exhibiting stability in between that at either period (see Section 6.5.2.3), and the fact that the stability at $T_s = 2s$ is actually higher than at $T_s = 0s$ in some cases (see Figures 6.42d and 6.44d).
Figure 6.47: SGR due to rigid bottom diaphragm, at $P = 0$, as $h/t$
6.5. Modelling results

Figure 6.48: SGR due to rigid bottom diaphragm, at $P = 0$, as $T$
6.5. Modelling results

Figure 6.49: SGR due to rigid bottom diaphragm, at $P = 10\,\text{kN/m}$, as $h/t$
6.5. Modelling results

Figure 6.50: SGR due to rigid bottom diaphragm, at $P = 10\,\text{kN/m}$, as $T$
6.6 Summary

This section provides a summary of key observations from the various phases of the parametric modelling. These observations will form the basis for the recommended assessment procedure in Chapter 7. As mentioned earlier, the concept of stability in these observations refers to the aggregation of results for many ground motions. Results for any single ground motion do not necessarily obey these trends; for this reason, experimental results were not compared with fragility curves. The primary purpose of the testing was validation of the model, and this was successfully achieved (refer to Chapter 5).

Diaphragm period and ground motions

- Rigid and very stiff diaphragms \((T_s < 0.2\, s)\) resulted in the most stable walls, while diaphragms with moderate periods \((0.5\, s < T_s < 1.25\, s)\) resulted in the least stable walls. Stability increased at periods longer than 1.25 s.

- Wall stability is more dependent on ground motion content at long periods than on that at short periods. For \(T_s < 0.5\, s\), \(S_a(1\, s)\) was an equally good or better predictor of collapse than \(S_a(T_s)\). For \(T_s > 0.75\, s\), \(S_a(T_s)\) was the best predictor, but using \(S_a(1\, s)\) produced reasonable results.

- Using \(S_a(1\, s)\) as the intensity measure for long-period systems adds variance to the results, since this method implicitly relates \(S_a(T_s)\) to \(S_a(1\, s)\). This additional variance becomes apparent at low (or high) \(P_{col}\) levels. For long period systems, trends with respect to varying \(T_s\) are best described by the median results \((P_{col} = 50\% )\).

- Near-fault non-pulse motions produced results very similar to those of far-field motions. Near-fault pulse motions showed significantly less stability improvements at long periods than far-field motions.

- The spectral shape factor, \(\varepsilon\), of far-field motions is insignificant if characterizing stability by \(S_a(1\, s)\) or at longer periods. The spectral shape would only become significant if describing short-period systems by \(S_a(T_s)\). In this case, correcting outcomes for \(\varepsilon\) would serve as a workaround to relate \(S_a(T_s)\) to \(S_a(1\, s)\).

- Systems in which the diaphragm period differs between the top and bottom of the wall generally exhibit stability in between that of con-
6.6. Summary

Figurations when both diaphragms are equal to either one of these periods.

- The magnitude of the effect of diaphragm period is heavily dependent on the slenderness ratio, with more slender walls being less sensitive to differences in diaphragm periods.

- Ground floor walls in systems with $T_s > 0.5\, s$, in which the bottom ‘diaphragm’ is rigid, exhibited on the order of 10% higher stability than walls in which the bottom diaphragm has the same stiffness as the top diaphragm. This stability gain was lowest for very long period systems ($T_s = 2\, s$), where at high $P_{col}$ there was actually a slight stability decrease.

Other factors

- Stability increases with decreasing crack height. The magnitude of the effect on stability is largest at lower crack height — i.e. the difference in stability between relative heights of 0.4 and 0.5 is greater than that between heights of 0.7 and 0.8.

- Stability increases with decreasing slenderness ratio, $h/t$. The relation of $S_{a_{col}}$ to $h/t$ is described well by a power function.

- Stability increases with increasing wall thickness, for constant $h/t$. Two-wythe walls had $S_{a_{col}}$ on the order of 10–30% lower than that of three-wythe walls of the same $h/t$, with larger differences at $T_s = 0.5\, s$ than at $T_s = 1.0\, s$.

- The effect of the amount of spalling at the crack was minimal for values between 5 and 15 mm.

- Stability increases with increasing damping ratios.

- Stability increases as the relative mass of diaphragms to walls decreases.

- Stability increases with increasing axial load. The magnitude of the effect is heavily dependent on the manner in which the load is applied. More symmetrical load applications result in close to linear stability gains with increasing axial load, while more eccentric load applications result in progressively lower gains as the axial load increases.
6.6. Summary

- The stabilizing effects of axial load are highly dependent on the slenderness ratio of the wall and on the period. More slender walls gain less stability for a given axial load than do less slender walls. In addition, systems with $T_s = 0.5$ s exhibited significantly lower stability gains than systems at other periods.
Chapter 7

Recommendations for Assessment Guidelines

Moving from the results of the parametric analysis to an assessment procedure involves simplifications and the subjective evaluation of numerous factors. In this section, principal factors requiring consideration are discussed, and suggestions for an updated assessment procedure are offered based on the results of the parametric study, the experimental study, and the evaluation of these factors.

7.1 Considerations

Key factors that should be considered in the implementation of the study results into assessment guidelines are briefly discussed. Most of these topics would be of interest for future research.

1. Variance in $S_a(1\,s)$ at collapse

Among systems with varying periods, it was shown that comparing $S_a(1\,s)$ values at low $P_{col}$ can obscure the mean trend, particularly for long period systems, because the variance in results changes with system period. For shorter period systems, there is an inherent large variance in the collapse spectra at all periods; consequently, the large variance in $S_a(1\,s)$ is more representative of the actual trends. This is a continuously varying phenomenon, however, and there is no well-defined cutoff period at which the effect starts or stops being relevant.

For long periods ($T_s > 1\,s$), it would be appropriate to use the $P_{col} = 50\%$ values as the indicator of the relative stability between periods. The lower $P_{col}$ curves could be approximated by shifting the 50% curve through the appropriate $P_{col}$ points at $T_s = 1\,s$, which has the small variance representative of the long period systems.

2. Diaphragm displacements
7.1. Considerations

Results in previous sections have been presented in terms of spectral accelerations, in large part because this is the most common design criterion used in practice. When dealing with long-period diaphragms, it is important to maintain an awareness of what sort of deformations a given spectral acceleration creates. Spectral displacements are plotted as a function of spectral acceleration in Figure 7.1 for three periods: 0.5, 1, and 2s. The thickness of a typical 3-wythe URM wall is also indicated for reference. The displacements shown are those corresponding to a SDOF system; peak diaphragm displacement at mid-span would be approximately 27% larger (see Equation (2.7) and Figure 2.1).

![Figure 7.1: Spectral displacement vs spectral acceleration](image)

Diaphragm deformations at moderate $S_a$ values are less than roughly half a wall thickness for systems with diaphragm periods up to roughly 1 s, assuming that the linear response of a system with a secant stiffness is representative of the actual response of the diaphragm. (e.g., at $S_a(1s) = 0.6$ g, $S_d(1s) \approx 150$ mm). At periods much beyond this, deformations increase rapidly, since $S_d$ is proportional to $T^2$. Large deformations, particularly if occurring in buildings with short diaphragm spans, can cause issues not directly related to out-of-plane wall failure, such as damage at corners and cross walls and wall cracking due to horizontal bending.

The results presented in the parametric study indicate that the large displacements associated with long-period systems do not create a higher risk of collapse within the limitations of the one-way rocking model. In light of the potential issues associated with large diaphragm
7.1. Considerations

deformations other than out-of-plane wall failure, it would be prudent
to make assessment guidelines increasingly conservative as diaphragm
periods increase. This consideration must be balanced against the
findings of Point (1), which suggested that stability gains at long pe-
riods are actually more significant than is implied by results at low
$P_{col}$.

3. Response along the diaphragm

As discussed in Section 2.1.2 and mentioned in Point (2), the dia-
phragm response varies significantly along its length. Disregarding
two-way effects, one could conservatively imagine an out-of-plane wall
along the span of a diaphragm as a series of tall and narrow one-
way spanning strips. This concept is illustrated in Figure 7.2, which
shows a single wall with window perforations connected to flexible
diaphragms at top and bottom. The wall strips in the illustration are
cracked and undergoing rocking.

![Figure 7.2: Wall response along the length of a flexible diaphragm](image)

The strips near the ends of the diaphragm would be subject to the
equivalent of a rigid diaphragm input, while the strips in the middle
would be subject to a relative diaphragm response greater than that of
the equivalent SDOF model used in this study, which considered only
the strips at particular points near mid-span (see Figure 2.1). Not
only would the inputs to the wall strips at the other points along the
span be different, but the resulting differences in wall rocking response
would also affect the response of the rest of the diaphragm. Neither of these effects was accounted for within the scope of the present study.

Given that the parametric study showed that stiff and rigid diaphragm systems resulted in the highest out-of-plane stability, it is unlikely that points of intermediate response between the equivalent SDOF system and the rigid system would produce lower stabilities. However, it is possible that the mid-span points would result in lower stability than the SDOF system modelled due to the larger diaphragm diaphragm deformations at this location. Whether additional conservativeness beyond the modelled results is necessary to account for this effect may be a matter of opinion, but the effects of two-way bending (not modelled here) would in many cases provide some reserve resistance.

4. Two-way bending and wall geometry

Griffith et al. [2007] conducted cyclic air bag testing of two-way supported URM wall panels. They showed that the addition of vertical supports at the ends of wall panels resulted in significant additional reserve displacement capacity beyond that available through one-way bending. In this same study, however, it was noted that a large proportion of vertical cracking was due to line failure (cracking through bricks) rather than stepped failure (cracking at the brick–mortar interface). While “stepped cracks can possess significant reserve post-cracking moment capacity due to the torsional resistance from friction acting on the bed-joints”, line cracks can not. In the most extreme cases, full-height line cracks can effectively completely negate two-way bending effects.

Full-scale dynamic testing of two-way supported wall panels has not been carried out to date, and the existing work does not clearly conclude how much additional dynamic stability is achieved by two-way supports vs. one-way supports. Two-way effects appear unlikely to be detrimental to dynamic stability, however the demonstrated potential for line cracking necessitates caution in incorporating any benefits from two-way bending into assessment guidelines. Due to these issues, two-way effects were ignored for the time being in the derivation of guidelines.

In addition to two-way effects, other geometry issues can complicate wall assessment. Most notably, gable end walls have continuously varying heights along their length, and the relationship between their dynamic stability and that of a constant-height wall is unclear. Use of
7.1. Considerations

the peak gable height is simple, but likely over-conservative. The use of some intermediate height is likely reasonable, but the selection of this height has not received adequate treatment in the literature. Both of these topics are of interest for future research.

5. Amplification up the building

As discussed in Section 2.1.1, there is very limited research available on the amplification of ground motions up the height of URM buildings. ABK Joint Venture [1981b] ignored the issue entirely by rationalizing that between foundation rocking and non-linearity of URM in-plane response it was unlikely that accelerations would be greater at the top of a building than at the base. However, one available instrumentation record in a real building revealed notable amplification [Tena-Colunga and Abrams, 1992], particularly at short periods, and recent modelling [Knox, 2012], while not exhaustive, certainly suggested that some amplification may be likely.

The present study demonstrated that out-of-plane wall stability — for both rigid and flexible diaphragm systems — is more dependent on long-period input motion content than short-period content. Short period amplification of input motions would therefore not be expected to cause significant changes in wall stability, and the use of the results of the current study, which used unamplified ground motions as input to diaphragms, should be reasonable. The subject should definitely be addressed in greater detail by future work; new efforts on this front are currently getting under way [Paxton, 2014].

6. Arching action

Derakhshan [2010] found during in-situ airbag testing of URM walls in vintage buildings that the effect of arching action provided by timber roof diaphragms was negligible. Other construction details, like concrete ring beams, can provide notable arching action effects that could substantially increase the wall stability. While investigations like the aforementioned have shown significant strength increases due to arching, the effect has not been demonstrated in dynamic testing.

An approximation could be rationalized based on the effects of axial load demonstrated in the present study by calculating the vertical stiffness of the support. It would be expected that the effect would be less than that due to an axial load corresponding to the maximum force predicted due to the arching, since arching resistance is only
7.1. Considerations

Mobilized as the wall rocking displacement increases. Alternatively, the effect could be incorporated into future modelling as a spring-loaded support block placed on top of the wall. These considerations are beyond the scope of the current study, however. Excluding the effect is conservative in all cases, and for flexible timber diaphragms it is in fact accurate.

7. Masonry strength

Meisl et al. [2007] showed that the quality of collar joints in multi-wythe walls had little effect on their out-of-plane response during dynamic testing. Type O mortar was used in both those tests and in the current tests. Vintage URM construction may exhibit considerably weaker mortar than that used during testing (e.g., Lumantarna [2012]), in terms of both flexural bond strength and mortar compressive strength.

While wall segments in the current tests sustained negligible damage outside of the characteristic horizontal cracks, it is not clear how well vintage masonry with very low strength mortar would hold together under sustained rocking behaviour. It would be of interest to carry out dynamic testing with mortars of varying strengths to observe the difference in degradation during out-of-plane rocking. The current modelling did, however, include the effects of moderate amounts of spalling at the crack location. While it is out of the scope of this study, it may be reasonable to include limits on acceptable mortar strength in out-of-plane assessment guidelines, below which reduction factors would be applied to allowable $S_a$ values.

Additionally, masonry strength can be a consideration for anchorage design. A recent anchor-testing program conducted in New Zealand showed anchor capacity to be dependent on masonry strength, but also that properly-installed anchors can still function adequately in low-strength walls [Dizhur, 2012]. Provisions for adequate anchoring in low-strength walls should be carefully considered along with those for out-of-plane stability.

8. Damping and non-linearity

Damping in flexible timber diaphragms in URM buildings has received limited study to date. At this point, the available evidence suggests that 5% should be reasonable. However, the parametric study showed that damping does in fact have a moderately significant effect on out-
of-plane stability. Further testing regarding this issue, particularly in-situ, would improve confidence in this assumption. Based on available data, it seems unlikely that 5% would be unconservative, and so the study results using this value should be appropriate for use.

Diaphragm non-linearity was not accounted for in this study. The stiffness recommendations on which the period ranges were based were approximating secant stiffnesses at 100 mm deformation. This is a significant deformation, and tested response characteristics [Wilson, 2012] would suggest that stiffness degradation beyond this point is likely to be minor. In addition, the reduced input accelerations that would result from non-linearity would be unlikely to reduce wall stability. Nevertheless, it would be of interest to incorporate non-linearity into further parametric modelling, and particularly to examine the effects of the potential for increased diaphragm displacements.

9. In-plane damage

Perhaps the most significant simplification in the current study (and in most previous work on the topic) is that it considers out-of-plane wall response independently of in-plane response. In reality, all walls in a building are both ‘out-of-plane walls’ and ‘in-plane walls’ simultaneously when subjected to an actual earthquake. It could be argued that this issue is not of significant importance since in-plane damage tends to be concentrated in lower floors where in-plane demands accumulate, while out-of-plane failures tend to occur in the top floor, where axial load is lowest.

The effect of in-plane damage on out-of-plane dynamic stability remains one of great interest, and it has not been addressed specifically. Whole buildings have been tested, but in such tests it is difficult to draw conclusions on this issue in particular. Clough et al. [1990], Gülkan et al. [1990] noted in shake table testing of one-storey masonry houses that minimal differences in response were observed when testing with three-axis input versus one-axis input, but these observations can hardly be considered conclusive. Dynamic testing similar to that done in this study, but with wall specimens subjected to diagonal cracking prior to the test would be helpful.

10. Vertical accelerations

An issue that has received limited treatment across structural engineering is the effect of vertical accelerations. Historically, it has been
neglected due to two factors: (1) vertical motions are typically dominated by high-frequency content, and as such they input less energy into a structure as well as attenuate more rapidly with distance from the source, and (2) structures are typically designed with a high safety factor against gravity (vertical) loads, in addition to which is unlikely that the gravity system would be stressed by full snow and occupancy loads when an earthquake occurs. While the bases for these factors are accurate, the conclusion that vertical accelerations are insignificant is overly simplistic for a number of reasons in typical modern construction (see Papazoglou and Elnashai [1996]).

In the particular case of out-of-plane stability of URM walls, the issue is worthy of investigation, but is out of the scope of this study. The fact that out-of-plane response depends heavily on the axial force on the wall suggests that vertical effects could be important. On the other hand, the high-frequency nature of the vertical excitation means that several full cycles of vertical motion may occur during a single rocking excursion, which may in the end result in minimal overall difference. At this point, the issue is unresolved, and merits further attention.

11. Acceptable risk level

It is common for retrofit legislation to allow compliance with a lower hazard level for existing buildings than for new construction, to avoid retrofits becoming prohibitively expensive and thus not happening at all — the argument is that some retrofit is better than no retrofit. For example, the “bolts plus” provision in San Francisco’s ordinance 225-92 of 1992 mandated retrofits of URM buildings, but allowed buildings meeting certain occupancy and configuration restrictions to satisfy retrofit requirements simply by installing wall-to-diaphragm connections and satisfying wall h/t limits [Paxton et al., 2013]. Both ASCE 41 and New Zealand’s equivalent document, NZSEE [2006], allow the selection of performance targets from a range of options. While high performance is encouraged, in New Zealand the minimum national requirement is only that existing buildings undergoing retrofit satisfy earthquake demands of at least 33% of the design level for new buildings. Clearly, buildings retrofitted to a lower hazard level will still pose a higher risk to the public than new buildings, but this is considered acceptable given the alternative (no retrofit).

It is proposed that variation in risk levels could similarly be considered among different specimens of the same building, and within a building
itself, based on an assessment of the exposure. In the context of out-of-plane wall failure, the greatest life safety risk is due to wall debris falling onto an occupied street or sidewalk, rather than precipitating total building collapse. Ingham and Griffith [2011] noted during a post-earthquake survey of URM damage in Christchurch that walls and gables are significantly more likely to fall outwards from a building than inwards due to restraint from the diaphragms, resulting in a larger life safety risk for passers-by than for building occupants.

Consider as an example the top storey of a two-storey commercial URM building, with the façade located on a busy pedestrian street and roof joists supported on the front and back walls. Out-of-plane failure of the front wall in the daytime has the potential to cause significant loss of life, both for pedestrians on the ground in front of the building, and for occupants of the top storey, since the roof is supported on this wall. In comparison, out-of-plane failure of a side wall has much lower potential to cause loss of life, since the area below (say, an alley) is unlikely to be occupied, and this wall is not integral to the support of the roof.

While the conservative (and simple) solution would be to apply the same assessment standards to both of these walls, in the context of a limited retrofit budget, a greater overall risk reduction would be achieved by applying a higher standard to the high-risk front wall and a lower standard to the low-risk side wall. To this end, it is proposed to add an ‘exposure factor’ to the out-of-plane wall assessment procedure (see Section 7.2).

### 7.2 Recommended assessment procedure

In this section, an out-of-plane wall assessment procedure is proposed as an update to the current procedure in ASCE 41. As discussed in Section 7.1, many considerations are involved in the transformation of study results to assessment procedure. The subjective nature of most of these considerations means that significant judgement is involved in defining an assessment procedure, and there will likely be differences of opinion on these issues among readers. The proposed procedure in this section should be viewed as a reasonable starting point for further discussion among members of the relevant standards committees and the practising community in general.

The recommended procedure is summarized as follows, with subsequent sections providing more details regarding each step.
7.2. Recommended assessment procedure

- Classify diaphragms as stiff or flexible
- Obtain the corresponding base curve of $S_{ab}(1)$ vs. $h/t$
- Obtain a correction factor for axial load, $C_a$
- Obtain a correction factor for wall thickness, $C_t$
- Obtain a correction factor for exposure level, $C_e$
- Obtain a correction factor for ground level walls, $C_g$
- Compute the final relationship of $S'_a(1) = C_a \cdot C_t \cdot C_e \cdot C_g \cdot S_{ab}(1)$

7.2.1 Base curves and classification of diaphragms

The parametric study showed that very stiff and rigid diaphragms resulted in the most stable walls, while mid-range periods resulted in considerably lower stability. Very long periods showed some improvement over mid-range periods. It is recommended that diaphragm flexibility classification be limited to two categories: stiff and flexible. Allowing more detailed classification of flexible diaphragms is not recommended for two reasons: (1) there is significant uncertainty in assessing diaphragm periods, and (2) allowing benefits for longer periods is not prudent due to the other issues that may arise with the larger deformations to which long-period systems are subject (refer to Point (2) in Section 7.1). The recommended classifications are listed in Table 7.1.

Table 7.1: Diaphragm classification

<table>
<thead>
<tr>
<th>Classification</th>
<th>Period (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>stiff</td>
<td>&lt; 0.2</td>
</tr>
<tr>
<td>flexible</td>
<td>&gt; 0.2</td>
</tr>
</tbody>
</table>

The base curves for each case are defined for no axial load, at $P_{col} = 10\%$, in 3-wythe walls in upper stories. For each classification, the base curves are selected to approximate the results for the most conservative periods: ($T = 0.2\; s$ for stiff systems, and $T = 0.5\; s$ for flexible systems). The recommended relationships are listed in Equation (7.1), and the curves are plotted against the relevant model data (from Figure 6.41b) and against the current ASCE 41 limits in Figures 7.3a and 7.3b, respectively. The base curve for stiff diaphragms is reasonably consistent with current ASCE 41 limits for top
7.2. Recommended assessment procedure

storey walls, at low $h/t$, and with the limits for one storey walls, at higher $h/t$. The base curve for flexible diaphragms is notably more conservative than current limits, in accordance with the findings of this study.

$$S_{a_b}(1) = \begin{cases} 4 \cdot \left( \frac{h}{t} \right)^{-1} & \text{stiff diaphragms} \\ 3 \cdot \left( \frac{h}{t} \right)^{-3} & \text{flexible diaphragms} \end{cases} \quad (7.1)$$

7.2.2 Correction factors

The base curves must be adjusted to account for the effects of axial load, wall thickness, exposure level, and whether walls are at ground level or in an upper storey. Correction factors for each of these effects are presented in this section.

**Axial load:** Modelling results indicated that stability gains due to axial load decreased with increasing $h/t$ (see Figure 6.45). Results varied significantly for different periods, and again the most conservative points were at $T = 0.2$ s for stiff systems, and $T = 0.5$ s for flexible systems. These results are all based on joist pocket loading (onto the outer wythe of a 3-wythe wall). Figure 6.38 suggested that for such a load case, the relative stability gains decrease with increasing load. Furthermore, it is prudent not to rely too heavily on axial load gains due to possible countering effects from vertical accelerations.

The following formulation is recommended for the axial load correction factor, $C_a$:

$$C_a = \begin{cases} 1 + C'_a \cdot \left( \frac{P}{10} \right) & \frac{h}{t} < 8 \\ 1 + C'_a \cdot \left( \frac{P}{10} \right) \left( 1 - \frac{1}{12} \left( \frac{h}{t} - 8 \right) \right) & 8 \leq \frac{h}{t} \leq 20 \\ 1 & \frac{h}{t} > 20 \end{cases} \quad (7.2)$$

Here, $P$ is in kN/m and $C'_a$ is defined in Table 7.2. While the modelling results suggest that these relationships should remain valid even at large $P$, it is recommended to implement a cap at $P = 20$ kN/m to avoid excessive stability gains. $C_a$ values are plotted for $P = 10$ and $20$ kN/m in Figure 7.4. Notably, very slender walls are not allocated stability benefits from axial load.
7.2. Recommended assessment procedure

Figure 7.3: Base curves, for 3-wythe wall, no axial load, high exposure, upper storey
7.2. Recommended assessment procedure

Table 7.2: Axial load base factor

<table>
<thead>
<tr>
<th>Classification</th>
<th>$C_a'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>stiff</td>
<td>0.5</td>
</tr>
<tr>
<td>flexible</td>
<td>0.2</td>
</tr>
</tbody>
</table>

The suggested curves are compared to the modelled stability gains at $P_{col} = 10\%$ (Figure 6.45b) in Figure 7.5. The stability gains at other $P_{col}$ values were similar (refer to Figure 6.45). Figure 7.5 illustrates that the recommended axial load correction factors are generally conservative, which is prudent when not considering the effects of vertical accelerations.

![Figure 7.4: Axial load correction factors](image)

**Wall thickness:** Past research and modelling results indicate that for a given aspect ratio, rocking stability is greater for larger bodies than for smaller ones. In section Section 6.5.3.3 it was shown that the relative collapse intensity of thinner walls shows some variability with period, but is affected minimally by $h/t$. Differences between $T_s = 0$ and $0.5\, s$ are moderate, and it could be expected that results for $T_s = 0.2\, s$, though not modelled here, would fall somewhere in between those values. For simplicity, it is recommended that the thickness correction factor be defined as period-
7.2. Recommended assessment procedure

Figure 7.5: Axial load correction factors, compared to model results at $P_{\text{col}} = 10\%$

independent for all diaphragms. The results of Figure 6.29 are repeated with the recommended correction factor, calculated using Equation (7.3), overlaid in Figure 7.6.

$$C_t = 0.2 + \frac{5}{2} \cdot t \leq 1.0, \ t \ \text{in m} \quad (7.3)$$

**Exposure:** As discussed in Section 7.1, it would be of interest to account for varying levels of risk corresponding to higher or lower exposure conditions. An exposure factor, $C_e$, is proposed that would account for variations from the base curve case of $P_{\text{col}} = 10\%$ according to the assessed exposure caused by a particular wall. The assessment should take into account the wall’s role in the support of the structure’s gravity system, and also the likelihood of occupants being located in the impact zone in the case of wall failure. In regards to the latter, it has been shown to be far more common for debris from out-of-plane wall failures to fall outward than inward [Ingham and Griffith, 2011]; as such, assessment emphasis should be placed on the exposure outside the building. The base case of $P_{\text{col}} = 10\%$ is deemed to be a reasonable risk level for default high-risk conditions. It is recom-
7.2. Recommended assessment procedure

Figure 7.6: Thickness correction factors, compared to model results at $P_{col} = 10\%$

It is recommended to define $C_e$ as approximating the differences between $P_{col}$ values from parametric studies, as listed in Table 7.3. These values were derived by comparing results within the subfigures of Figures 6.41 and 6.43.

Table 7.3: Exposure factor

<table>
<thead>
<tr>
<th>Exposure</th>
<th>$P_{col}$</th>
<th>$C_e$ (Stiff)</th>
<th>$C_e$ (Flexible)</th>
</tr>
</thead>
<tbody>
<tr>
<td>very high</td>
<td>5%</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>high</td>
<td>10%</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>low</td>
<td>20%</td>
<td>1.15</td>
<td>1.1</td>
</tr>
<tr>
<td>very low</td>
<td>50%</td>
<td>1.5</td>
<td>1.25</td>
</tr>
</tbody>
</table>

The definition of what constitutes each level of exposure should be carefully considered. In particular, the ‘very low’ case defined here should be used only under stringent conditions. Note that the study used as a basis for these results considered only the uncertainty in the wall response. A complete risk study should include the uncertainty in both the hazard and
7.2. Recommended assessment procedure

the exposure before any decisions are made regarding adopting specific factors. This is a highly subjective matter that merits further discussion, but the numbers are included here for completeness.

**Ground level:** In flexible diaphragm systems, modelling results showed mild stability gains for walls in which the base was connected to a rigid diaphragm (e.g., for a one-storey building) vs. those connected to flexible diaphragms at top and bottom (upper storey walls) — see Figures 6.47–6.50. While these results showed a slight decrease in stability at very long periods (2s), the simplifications made in the definition of the base curves left reserve capacity at these periods, making it reasonable to apply constant stability gains at all periods. These gains can be accounted for by applying a ground level correction factor, $C_g$, defined as follows:

$$C_g = \begin{cases} 
1.0 & \text{stiff diaphragms} \\
1.1 & \text{flexible diaphragms} 
\end{cases} \quad (7.4)$$

7.2.3 Safe seismic hazard level

The ASCE 41 special procedure specifies that out-of-plane stability need not be evaluated for sites at which $S_{X1} \leq 0.133$. This lower bound is re-evaluated using the new procedure. Conservative conditions are chosen to represent worst-case walls: $t = 200\,\text{mm}$ represents two-wythe walls, and walls are assumed to be in the top storey with no axial load and very high exposure. This produces $C_t = 0.7$, $C_e = 0.9$, and $C_a = C_g = 1.0$. Flexible diaphragm conditions are used. At $h/t = 26$, this produces an allowable $S''_a(1)$ of 0.08 g.

It can be concluded that at sites with a hazard level of $S_a(1s) \leq 0.08\,\text{g}$, out-of-plane stability need not be evaluated for URM walls that meet all of the following criteria:

- have a thickness of at least 200 mm,
- are adequately anchored to the diaphragms at all levels, and
- are within reasonable $h/t$ bounds bearing in mind other stability issues.

7.2.4 Anchorage demands

Design forces for wall-to-diaphragm anchors are currently specified in the ASCE 41 special procedure as the maximum of (a) $2.1 \cdot S_{X1} \cdot W$, where $W$
7.2. Recommended assessment procedure

is the weight of the wall, or (b) 2.9 kN/m. The amplification factor of 2.1 is derived from ABK Joint Venture’s original conclusions regarding the amplification of the ground motion caused by the flexible diaphragms, which resulted in the recommendation of designing anchorage for a demand of 1.0 times the wall weight in a seismic hazard zone with an effective peak acceleration of 0.4 g. This recommendation was reformulated in terms of $S_{X1}$, and the amplification factor adjusted to suit. The investigation of anchorage demands was outside the scope of the analytical study, but observations can be made regarding the anchorage demands observed in the experimental phase of the project.

In shake table testing of cracked walls, it was observed that peak connection (i.e. anchorage) demands in runs causing collapse (but prior to the time of collapse) reached levels up to 1.6 times greater than those predicted from the spectral acceleration of the applied motion at the period of the diaphragm system. Total wall forces (which would be taken up by the combination of top and bottom anchorage) reached levels up to 1.7 times greater than predicted. In the highest stable cracked runs, connection demands were up to 1.5 times predicted levels, and total wall forces were up to 1.2 times predicted levels. These large force peaks occurred due to impact when the wall crack closed up after a large rocking excursion. Individual connection demands were consistently largest at the bottom — the peak top connection demand observed in all testing was only 1.2 times greater than predicted, compared with 1.6 for the bottom connection.

When considering the shape of a typical uniform hazard spectrum, the specified force of $2.1 \cdot S_{X1} \cdot W$ would be sufficiently large to meet all the demands incurred in the testing, which included diaphragm periods of approximately 0, 0.8, 1.2, and 1.6 s. Here, the amplification factor is covering the demands created by (1) uneven distribution of forces between top and bottom connections, and (2) force peaks caused by impacts during rocking. Neither of these matches the original rationale for the amplification factor, but by chance the existing factor happens to reasonable for the ground motion considered.

In walls that undergo limited or no rocking, the force amplification would be lower due a reduction in the above-mentioned issues. The peak force demands would in this case be more aptly described by the unamplified spectral acceleration at the period of the diaphragm. Without impact effects, peak anchorage demands will be limited by the strength of the diaphragm. In particular where periods are in the high-amplification range (e.g., under 0.5 s), using full elastic spectral acceleration may be over-conservative for a timber diaphragm due to non-linear effects.
Anchorage should be capacity-designed — that is, designed so that other components are force-limiting, and that the anchorage will not fail under any circumstances. The two components to which anchorage is connected are timber diaphragms, whose highly ductile nature makes them good candidates as force limiters, and URM walls, which have some ‘ductility’ capacity due to rocking. The experimental tests in this study have demonstrated that anchorage force demands can be amplified if URM rocking is mobilized.

The existing demands specified by the ASCE 41 special procedure are reasonable for URM walls connected to long-period diaphragms. It is recommended that further work be conducted regarding anchorage demands, particularly for diaphragms with periods in the high-amplification spectral region. Retrofitted diaphragms in small to moderate buildings could be representative of such periods. Design demands might be formulated as an amplified spectral acceleration at the diaphragm period, with possible reduction allowed to an amplified upper-bound diaphragm capacity (e.g., refer to Wilson et al. [2013] for diaphragm properties). In addition, a value that is possibly close to the current requirement could serve as a lower bound to prevent demand reductions for long-period diaphragms. The temptation to reduce anchorage demands should be avoided — they are arguably the most critical component in a URM retrofit, and relatively inexpensive.

### 7.3 Assessment examples

In this section, assessment curves are produced for several example buildings and compared with applicable results from ASCE 41. Wall height is considered as a variable. Masonry density is assumed to be 1800 kg/m$^3$. Wall thickness is assumed to be 110 mm per wythe.

**Example 1:** A small one-storey building, 6 × 8 m in plan, with roof joists spanning the short dimension is considered. Walls are three wythes thick, and a two-wythe parapet, 1 m high, is on the front wall, which is 6 m long. No significant parapet is present along the sides and back of the building. The vintage straight-sheathed wooden diaphragm is classified as flexible in both directions. The side walls are classified as high risk because they are supporting the roof structure. The front and rear walls are also classified as high risk due to the exposure of sidewalk and parking areas in front and behind the building. The self-weight of the roof diaphragm is assumed to be 0.5 kN/m$^2$.

The axial load on the walls is calculated as follows:
7.3. Assessment examples

- Front wall: \( P = 0.22 \text{ m} \cdot 1.0 \text{ m} \cdot 17.7 \text{kN/m}^2 = 3.9 \text{kN/m} \)
- Side walls: \( P = 3 \text{ m} \cdot 0.5 \text{kN/m}^2 = 1.5 \text{kN/m} \)
- Rear wall: \( P = 0 \)

The axial load correction factor, \( C_a \), is calculated from Equation (7.2), for flexible diaphragm conditions. The factor is plotted as a function of \( h/t \) for each wall in Figure 7.7.

![Diagram showing axial load correction factors for front, side, and rear walls.](image)

**Figure 7.7:** Axial load correction factors, example 1

From Equation (7.3), the thickness factor is determined to be \( C_t = 1.0 \) for all walls since they are three wythes thick.

From Table 7.3, the exposure factor is determined to be \( C_e = 1.0 \) for all walls based on the risk assessment.

From Equation (7.4), the ground level factor is determined to be \( C_g = 1.1 \) for all walls since they are at ground level.

The final assessment curve is computed for each wall as \( S'_a(1) = C_a \cdot C_t \cdot C_e \cdot C_g \cdot S_a(1) \). The curves are shown in Figure 7.8, along with the limits for walls in one-storey buildings from ASCE 41.

In this example, the differences in axial load among the various walls are moderate, and thus there is minimal effect on the respective assessment curves. Of note is the significant discrepancy between the recommended procedure and the current standard, depending on \( h/t \). In the vicinity of \( h/t = 16 \) (equivalent to a wall height of 5.3 m for a thickness of 0.33 mm), the two methods produce very similar results, and for more slender walls the differences are relatively minor. For less slender walls, the new procedure is significantly more conservative.
Example 2: The front walls of a large two-storey commercial building are to be assessed. The building has three-wythe walls in both levels, and a 1.5 m tall parapet on the front. The building is 16 × 24 m in plan, with vintage single-sheathed wooden diaphragms classified as flexible. The front walls are bearing a 2.5 m tributary width of diaphragm weight. The walls have been assessed as high risk.

For the purpose of calculating axial load, the top storey wall will be assumed to have a height of 3.5 m. The axial load on the walls is calculated as follows:

- Top storey:
  \[ P = 0.22 \text{ m} \cdot 1.5 \text{ m} \cdot 17.7 \text{kN/m}^3 + 2.5 \text{ m} \cdot 0.5 \text{kN/m}^2 = 7.1 \text{kN/m}\]

- Bottom storey:
  \[ P = 7.1 \text{kN/m} + 0.33 \text{ m} \cdot 3.5 \text{ m} \cdot 17.7 \text{kN/m}^3 + 2.5 \text{ m} \cdot 0.5 \text{kN/m}^2 = 28.8 \text{kN/m}\]

The axial load correction factor, \( C_a \), is calculated from Equation (7.2), for flexible diaphragm conditions. The factor is plotted as a function of \( h/t \) for each wall in Figure 7.9. Note that while \( P = 28.8 \text{kN/m} \) for the bottom storey wall, a maximum contribution of \( P = 20 \text{kN/m} \) is permitted in the calculation of \( C_a \).
7.3. Assessment examples

From Equation (7.3), the thickness factor is determined to be \( C_t = 1.0 \) for both walls since they are three wythes thick.

From Table 7.3, the exposure factor is determined to be \( C_e = 1.0 \) for both walls based on the risk assessment.

From Equation (7.4), the ground level factor is determined to be \( C_g = 1.0 \) for the top storey wall, and \( C_g = 1.1 \) for the first storey wall.

The final assessment curve is computed for each wall as 
\[
S_a(1) = C_a \cdot C_t \cdot C_e \cdot C_g \cdot S_{a_b}(1)
\]

The curves are shown in Figure 7.10, along with the limits for walls in multi-storey buildings from ASCE 41.

Figure 7.9: Axial load correction factors, example 2

Figure 7.10: Assessment curves, example 2
7.3. Assessment examples

In this case, the assessment curves show significant differences between the two walls. While the bottom storey wall is assessed to withstand substantially higher intensity ground motions than the top storey wall, both differ significantly from the current limits. In each case, the new curves are similar to the most conservative points on the ASCE 41 limits: around \( h/t = 9 \) for the top storey, and around \( h/t = 18 \) for the first storey. At other \( h/t \) values, the new curves are more conservative than the current limits.

**Example 3:** The building from example 2 is retrofitted by pouring a 100 mm thick concrete slab on top of the existing wooden diaphragms, creating effectively rigid diaphragm conditions. The total self-weight of the new diaphragms is assumed to be 2.5 kN/m².

The axial load on the walls, including the additional diaphragm loads, is calculated as follows:

- **Top storey:**
  
  \[
  P = 0.22 \text{ m} \cdot 1.5 \text{ m} \cdot 17.7 \text{ kN/m}^3 + 2.5 \text{ m} \cdot 2.5 \text{ kN/m}^2 = 12.1 \text{ kN/m}
  \]

- **Bottom storey:**
  
  \[
  P = 12.1 \text{ kN/m} + 0.33 \text{ m} \cdot 3.5 \text{ m} \cdot 17.7 \text{ kN/m}^3 \\
  + 2.5 \text{ m} \cdot 2.5 \text{ kN/m}^2 = 38.8 \text{ kN/m}
  \]

The axial load correction factor, \( C_a \), is calculated from Equation (7.2), for stiff diaphragm conditions. The factor is plotted as a function of \( h/t \) for each wall in Figure 7.9. Note that while \( P = 38.8 \text{ kN/m} \) for the bottom storey wall, a maximum contribution of \( P = 20 \text{ kN/m} \) is permitted in the calculation of \( C_a \).

The other factors, \( C_t, C_e, \) and \( C_g \), remain unchanged from Example 2.

The final assessment curve is computed for each wall as \( S'_a(1) = C_a \cdot C_t \cdot C_e \cdot C_g \cdot S_{a_k}(1) \). The curves are shown in Figure 7.12, along with the limits for walls in multi-storey buildings from ASCE 41.

Comparing Figure 7.12 to Figure 7.10, it is apparent that the retrofitted building is allowed significantly higher demands on out-of-plane walls. In the top storey, the new curve compares well with the least conservative point in the current standard, while in the bottom storey, the new curve is representative of the middle range of the old standard.
7.3. Assessment examples

Figure 7.11: Axial load correction factors, example 3

Figure 7.12: Assessment curves, example 3
7.4 Summary and additional considerations

The new assessment method recommended in this study is a significant change from the procedure in the current ASCE 41 standard. For flexible diaphragm systems, the new method tends to be more conservative than the current standard. In some cases, the differences are minor, while in other cases they are significant. For stiff diaphragm systems, the new method tends to be consistent with or slightly less conservative than the current standard.

The new method has been specified based on the conclusions drawn from a large-scale parametric model. Key aspects of the model, including the wall response under varying diaphragm flexibility, were calibrated to the results of shake table testing. While there remains significant room for refinement and additional research, the method presented here offers significant improvements over the current method in terms of risk consistency among varying wall configurations, and by providing a thorough rationale for its details based in analytical and experimental work.

Several additional considerations are noteworthy in the process of implementing potential changes to the current assessment standard. The current ASCE 41 limits cut off allowable $h/t$ values at maximum values. Results from the current study do not objectively support an $h/t$ cut-off based purely on the stability of the one-way rocking model. While wall heights and $h/t$ values are already effectively limited by construction practices, general prudence and engineering judgement may favour applying hard limits in the assessment procedure.

In addition, the current limits do not impose a maximum $S_a$ limit. For the sake of prudence, it may be of interest to cap the base curves at some $S_a$ limit. Again, the results of this study do not objectively support the definition of such a limit, but cutting the curves off horizontally at the $S_a$ values corresponding to $h/t = 8$ would be a reasonable starting point.

Finally, the definition of base curves and correction factors in these recommendations err towards conservativeness, most prominently so in flexible diaphragm portions. The notable variance in modelling results with varying system period (refer to Section 6.5.4) resulted in systems with periods of 0.5 s typically governing the definition of parameters. Particularly in regards to the axial load effect, these values were significantly more conservative than at other periods. The subjectivity inherent in the choice of parameter values may well allow for more relaxed selections in certain cases.
Chapter 8

Summary and Conclusions

This study has investigated the out-of-plane seismic performance of URM walls, with a specific focus on the effect of diaphragm flexibility. The study consisted of experimental and analytical phases, ultimately leading to recommendations for an improved out-of-plane seismic assessment procedure.

A review of past research showed that the timber floor and roof diaphragms typical of vintage URM buildings have very low stiffness, with fundamental periods of vibration of typical diaphragm-wall systems ranging between 0.5 and 3\(\text{s}\). Ground motions applied to buildings are filtered through relatively stiff in-plane walls into the ends of diaphragms, then filtered through the diaphragms into the attached out-of-plane walls. Flexible diaphragms therefore have the potential to significantly affect out-of-plane wall performance relative to walls connected to rigid diaphragms. This matter has received very limited treatment in past research, and no dynamic tests specifically addressing this effect have been conducted.

8.1 Experimental phase

A dynamic testing programme was developed and conducted at the University of British Columbia’s Earthquake Engineering Research Facility. Five full-scale unreinforced solid clay brick wall specimens spanning one storey were subjected to earthquake ground motions on a shake table. The top and bottom of the walls were connected to the shake table through coil springs, simulating the flexibility of the diaphragms. The apparatus allowed the wall supports to undergo large absolute displacements, as well as out-of-phase top and bottom displacements, consistent with the expected performance of URM buildings with timber diaphragms. Variables examined included diaphragm stiffness and wall height.

Walls were cracked by applying a ground motion under rigid diaphragm conditions, then tested to failure with flexible diaphragm conditions by incrementally increasing the intensity of the applied ground motion. All walls sustained horizontal cracks at an intermediate height, with four out of five walls cracking between 0.47 and 0.55 times the wall height, and one wall...
8.2 Analytical phase

The primary purpose of the tests was to provide a dataset for validation of an analytical model. A rigid body rocking model, first proposed by Meisl [2006] for use in modelling out-of-plane URM wall response, was created in the software Working Model 2D and successfully validated against the test data. The model was able to reproduce the time-variation of the chaotic, non-linear rocking behaviour of the cracked walls, and also reproduce the motion intensity at collapse for each wall with reasonable accuracy. The ability of the rigid body model to explicitly represent the true rocking behaviour of
the wall is a significant benefit over approaches in previous research (e.g., Doherty et al. [2002]).

Conclusions regarding the relative stability of walls with different boundary conditions cannot be drawn from the tests using a single ground motion. The highly non-linear rocking response of the wall is very sensitive to the details of the time variation of the applied excitation, and these details cannot adequately be quantified by simple intensity measures. To address this issue, and to investigate a wider range of parameters not feasible to investigate on the shake table, the analytical model was used to conduct a parametric study on out-of-plane performance. The study used the suite of 100 ground motions prescribed by FEMA P695 to examine motion-to-motion variability, and investigated the effect of numerous variables, most notably including the diaphragm period. Motion-to-motion variability consistently followed a lognormal distribution closely, and was significant under some conditions, highlighting the importance of a sufficiently large suite of motions in such a study.

It was determined that the spectral acceleration at a period of 1s was the best predictor of collapse for walls connected to most diaphragms with periods of less than 1s. For periods longer than this, the spectral acceleration at the diaphragm period was the best predictor. In the interest of simplicity and consistency with the current ASCE 41 standard, $S_a(1)$ was used as the intensity measure for all results.

In general, short-period systems ($T \leq 0.2\,s$) were the most resilient to collapse. Periods between 0.5 and 1s resulted in the lowest resilience levels, while some increase in resilience occurred at periods longer than this. Where top and bottom periods were different, results typically ended up in between those bracketed by the cases where both periods were equal to either the top or the bottom period.

The addition of overburden load had a stabilizing effect on wall response, with the magnitude of the effect heavily dependent on the manner in which the load was applied. Simulating a joist-pocket load produced roughly the same stability increase as by applying the same load fixed in the centre of the wall, despite the eccentricity of the joist pocket loading. For a given load level, the relative stability increases varied with period and $h/t$, with stability increases lowest at a period of 0.5s, and consistently decreasing with increasing $h/t$.

The relative crack height had a significant effect on stability, with higher cracks resulting in less stable walls. As the crack height approaches the top of the wall, its behaviour will approach that of a cantilever failure mode. Since the height of cracking can not be controlled, however, it was treated...
as a random variable in the parametric study and explicitly accounted for in the resulting probabilities of collapse.

The results of the parametric study were compiled and interpreted to form recommendations for an improved assessment procedure. Like the current ASCE 41 procedure, the new procedure is based on $h/t$ vs. $S_a(1)$ curves. Two base curves are provided: one for stiff diaphragms and one for flexible ones. Due to the high uncertainty involved in assessing the stiffness of an existing diaphragm, it is not recommended to allow assessment benefit based on the precise period of the diaphragm; all flexible diaphragms are thus grouped together and a reasonably conservative curve is used for the group. Correction factors are provided to account for axial load, wall thickness, and walls at ground level. Notably, the concept of accepting variable risk is proposed, by including an additional correction factor to tolerate higher risks of collapse depending on the assessed exposure of a particular wall.

The new procedure was compared with the existing simplified ASCE 41 procedure for several example scenarios for the common case of three-wythe walls. It was found that the new procedure was generally more conservative than the existing one for flexible diaphragms, most prominently so when comparing results for the first storey of multi-storey buildings. For stiff diaphragms, the new procedure was generally on par with, or more lenient than the existing procedure, particularly for top-storey walls subjected to significant axial loads. For two-wythe walls, the thickness correction factor would make results more conservative than those for three-wythe walls.

8.3 Contributions

The study documented in this dissertation has made significant contributions to the body of knowledge relating to the seismic performance of URM walls by completing the objectives of Section 1.2. The dynamic one-way out-of-plane bending response of URM walls connected to flexible supports was observed for the first time over a variety of support stiffnesses, including variable top and bottom support stiffnesses. An analytical model was validated to the experimental data, and it was demonstrated that the model could accurately simulate the dynamic response of the walls over the range of tested conditions. A large-scale parametric study, consisting of over 300 wall configurations and over 200,000 individual simulations, was conducted using the model, and insights into the effects of eight parameters, as well as ground motion variability, were obtained. The results of the modelling were
8.4 Future research

The need for further research in specific areas has been discussed at various points in this thesis, notably in Section 7.1. The most pertinent needs are briefly listed below.

- The effect of two-way bending should be investigated by dynamic testing. Data is needed specifically comparing two-way bending to one-way stability for otherwise equivalent walls. This could be added to the assessment procedure as an additional correction factor, as warranted.

- The effect of arching action should be investigated. This could likely be achieved analytically by extending the current model’s axial load capabilities to model various top support vertical stiffnesses.

- The effect of varying wall thickness should be investigated more thoroughly than was possible in this study. Ideally, the entire second phase of the parametric study would be re-run at several thickness levels.

- The effect of different wall geometries should be investigated. In particular, the response of gable walls should be compared with regular one-way spanning walls, and additional correction factors or other guidelines could be added to the assessment procedure to account for geometrical differences.

- The effect of varying response along the span of a flexible diaphragm should be investigated. The current SDOF idealization represents only one point on the diaphragm. Ideally, a 3-dimensional model would include one-way spanning strips at short intervals along the diaphragm span, all excited simultaneously.

- Practical limits on diaphragm flexibility should be developed. The stability of one-way spanning walls does not by itself form a basis for suggesting any limits, but other considerations like damage to corners and non-structural components may warrant displacement limits.

- The effect of diaphragm non-linearity should be investigated. This can be done analytically, possibly by extending the current model,
8.4. Future research

but implementation of complicated non-linearity in Working Model 2D may prove challenging.

• Amplification up the height of URM buildings needs to be adequately characterized, and incorporated into out-of-plane recommendations.

• A lower bound on masonry strength should be established, below which out-of-plane rocking can not be relied upon as a stable mechanism. With very low-strength walls, it is possible that wall-diaphragm anchorage may be unreliable, and additionally that walls may fall apart during rocking. Further dynamic testing may be warranted to develop confidence on this matter.

• The effect of in-plane damage on out-of-plane response should be investigated. Dynamic tests that are suitably controlled to provide reference to existing one-way tests could be conducted with pre-damaged walls.

• The effect of vertical accelerations should be investigated. This could likely be achieved analytically by extending the current model, but it would be of great interest to demonstrate the effects in full-scale dynamic testing.
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Appendix A

Wall Dimensions

Notes: This section contains detailed measurements for each wall specimen.
## Appendix A. Wall Dimensions

### Table A.1: Wall dimensions

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Appendix B

Materials Testing

Notes: This section contains detailed testing data for mortar compression, masonry prism compression, masonry bond wrench, brick compression, and brick absorption testing carried out as a portion of the experimental study.
B.1 Mortar compression

A total of 102 mortar cubes were tested in compression: 66 specimens created during construction of the first three walls (FF-3, FR-3, FF-2) and 36 specimens created during construction of the last two walls (SS-3, RR-3). Testing was carried out in accordance with CAN/CSA A179-04 (R2009) [CSA, 2009] on the compression testing machine in the UBC materials lab. A typical compression failure from the testing is shown in Figure B.1.

Cubes measured 50 mm on each edge. Due to the relatively small number of batches and large variability within individual batches, mean and $c_v$ were calculated simply from the list of all successful tests, without regard for batches. Test results are shown in Tables B.1 and B.2. Due to the large number of batches and small variability within individual batches, mean and $c_v$ were calculated from the list of batch means, thereby preventing bias due to varying numbers of samples per batch.

Table B.1: Mortar cube compression results, walls FF-3, FR-3, and FF-2

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<th>Specimen</th>
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<th>$f_{\text{max}}$ (MPa)</th>
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## B.1. Mortar compression

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Mean of batch means: 3.97 22.5%
Min. of batch means: 2.56
Max. of batch means: 5.75

#### Table B.2: Mortar cube compression results, walls SS-3 and RR-3

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<tr>
<th>Date cast</th>
<th>Batch</th>
<th>Date tested</th>
<th>Age</th>
<th>Specimen</th>
<th>(F_{\text{max}}) (kN)</th>
<th>(f_{\text{max}}) (MPa)</th>
<th>mean of (f_{\text{max}})</th>
<th>(c_v) of (f_{\text{max}})</th>
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<td>3</td>
<td>7.3</td>
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### B.1. Mortar compression

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<th>Specimen</th>
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<th>$f_{\text{max}}$ (MPa)</th>
<th>mean of $f_{\text{max}}$</th>
<th>$c_v$ of $f_{\text{max}}$</th>
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Mean of batch means: 4.16 35.7%
Min. of batch means: 2.31
Max. of batch means: 7.52
B.2 Masonry compression

A total of 10 masonry prisms were tested in compression: six specimens (1–6) created during construction of the first three walls (FF-3, FR-3, FF-2) and four specimens (A–D) created during construction of the last two walls (SS-3, RR-3). Testing was carried out in accordance with ASTM C1314 - 11a [ASTM, 2011] on the Baldwin testing machine in the UBC structures lab. Prior to testing, all prisms were capped top and bottom with hydrostone, cast against a precision-ground steel plate to ensure total flatness of the bearing surface. Specimens 1–6 were tested on 25 May 2012, and specimens A–D were tested on 16 July 2012. A typical compression failure from the testing is shown in Figure B.2. Displacements were measured by two linear potentiometers, one on each side of the loading plate; the recorded displacement was taken as the average of the two readings, thus compensating for tilt of the loading plate. Load and displacement readings were low-pass filtered to reduce noise.

Figure B.2: Typical masonry prism compression failure

Prism dimensions are listed in Table B.3. In accordance with ASTM C1314, the maximum gross stress, \( f_{\text{max}} \) is calculated by dividing the peak load, \( F_{\text{max}} \), by the area, \( A \). The gross stress is then multiplied by a correction factor for slenderness, \( C_{h/t} \), which is specified in ASTM C1314, to obtain the masonry compressive strength, \( f'_{m} \). The elastic modulus, \( E \), is calculated based on the change in gross stress, \( df \), and the change in strain, \( d\epsilon \), between the points on the force-displacement curve at 5% and 33% of peak load. Results are summarized in Tables B.4 and B.5. Force-displacement curves are shown in Figure B.3.
### B.2. Masonry compression

Table B.3: Dimensions of masonry prisms (mm, mm²)

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<th>2</th>
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<th>4</th>
<th>5</th>
<th>6</th>
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<th>B</th>
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<td>190</td>
<td>189</td>
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<td>90</td>
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<td>89</td>
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<td>89</td>
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<td>299</td>
<td>295</td>
<td>296</td>
<td>299</td>
<td>291</td>
<td>292</td>
<td>291</td>
<td>292</td>
<td>287</td>
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<td>295</td>
<td>296</td>
<td>299</td>
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<td>292</td>
<td>291</td>
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<tr>
<td>H4</td>
<td>299</td>
<td>295</td>
<td>296</td>
<td>299</td>
<td>291</td>
<td>292</td>
<td>291</td>
<td>292</td>
<td>287</td>
<td>288</td>
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</table>

L_{mean} = 190.8 \quad 188.8 \quad 190.8 \quad 190.5 \quad 189.8 \quad 189.3 \quad 191.0 \quad 189.8 \quad 190.8

W_{mean} = 88.5 \quad 89.3 \quad 87.5 \quad 88.5 \quad 88.3 \quad 89.3 \quad 89.0 \quad 87.5 \quad 89.3

H_{mean} = 299.0 \quad 295.0 \quad 296.0 \quad 299.0 \quad 291.0 \quad 292.0 \quad 291.0 \quad 287.0 \quad 288.0

h/t = 3.38 \quad 3.37 \quad 3.32 \quad 3.35 \quad 3.29 \quad 3.31 \quad 3.26 \quad 3.22 \quad 3.29 \quad 3.24

A = 16881 \quad 16516 \quad 17024 \quad 17002 \quad 16793 \quad 16701 \quad 17047 \quad 16888 \quad 16691 \quad 17024

Table B.4: Prism compression test results, walls FF-3, FR-3, and FF-2

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<th>Specimen</th>
<th>df (MPa)</th>
<th>dc (MPa)</th>
<th>E (MPa)</th>
<th>P_{max} (kN)</th>
<th>f_{max} (MPa)</th>
<th>C_{h/t}</th>
<th>f_m (MPa)</th>
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<td>638</td>
<td>38.2</td>
<td>1.095</td>
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mean = 8.504 \quad 0.00159 \quad 6357 \quad 513 \quad 30.5 \quad 1.097 \quad 33.4

c_p = 0.20 \quad 0.60 \quad 0.40 \quad 0.20 \quad 0.20 \quad 0.00 \quad 0.20

Table B.5: Prism compression test results, walls SS-3 and RR-3

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<th>Specimen</th>
<th>df (MPa)</th>
<th>dc (MPa)</th>
<th>E (MPa)</th>
<th>P_{max} (kN)</th>
<th>f_{max} (MPa)</th>
<th>C_{h/t}</th>
<th>f_m (MPa)</th>
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mean = 11.660 \quad 0.00122 \quad 10008 \quad 703 \quad 41.6 \quad 1.090 \quad 45.4

c_v = 0.12 \quad 0.32 \quad 0.21 \quad 0.12 \quad 0.13 \quad 0.00 \quad 0.13
B.2. Masonry compression

![Graph of masonry compression tests](image1.png)

(a) Walls FF-3, FR-3, and FF-2

![Graph of masonry compression tests](image2.png)

(b) Walls SS-3 and RR-3

Figure B.3: Masonry prism compression tests
B.3 Masonry flexural tension

A total of 9 batches of masonry prisms, each consisting of three prisms four bricks high, were subjected to bond wrench testing. Each batch was sampled from a different day of construction of the walls; five batches (1, 3, 4, 5, 6) were created during construction of the first three walls (FF-3, FR-3, FF-2) and four batches (A–D) were created during construction of the last two walls (SS-3, RR-3). Batch 2 from the first set of walls was damaged and was not suitable for testing. Bond wrench testing was carried out in accordance with ASTM C1072 - 10 [ASTM, 2010], with an apparatus constructed as per the drawings in that standard. The force was applied to the apparatus by the Tinius Olsen testing machine in the UBC structures lab, and was measured digitally using a load cell. The setup is shown in Figure B.4.

![Bond wrench test apparatus](image)

(a) End view  (b) Side view

Figure B.4: Bond wrench test apparatus

Masonry prisms were extremely fragile due to the low tensile bond strength, and numerous prisms were broken accidentally during handling. A number of courses broke at very low loads; it is likely that these were subjected to damage in preparation for the test, and these results were therefore neglected. Nearly all specimens broke cleanly at the brick-mortar interface (Figure B.5), while a few specimens had portions of the failure surface within the mortar bed.

ASTM C1072 specifies the formula to calculate the flexural tensile strength, here denoted $f'_{fb}$, as follows:

$$f'_{fb} = \frac{6 (P_L + P_l L_l)}{bd^2} - \frac{P + P_l}{bd}$$

where the parameters are defined as follows (where applicable, values specific to the apparatus and bricks used are indicated):
B.3. Masonry flexural tension

![Figure B.5: Typical bond wrench failure](image)

Figure B.5: Typical bond wrench failure

- $f'_b = -$  
  
  gross area flexural tensile strength

- $P = -$  
  
  maximum applied load

- $P_l = 159 \text{ N}$  
  
  weight of loading arm including one brick

- $L = 369.5 \text{ mm}$  
  
  distance from center of prism to loading point

- $L_t = -13.5 \text{ mm}$  
  
  distance from center of prism to centroid of loading arm

- $b = 191 \text{ mm}$  
  
  cross-sectional width of the mortar-bedded area

- $d = 89 \text{ mm}$  
  
  cross-sectional depth of the mortar-bedded area

A few of the mortar beds had notable void spaces in the tension zone. Where it was deemed significant, the estimated void ratio in the tension zone was estimated by eye, and the strength was multiplied by $(1 - R_{\text{void}})^{-1}$ to adjust for this effect. Due to the relatively small number of batches and large variability within individual batches, mean and $c_v$ were calculated simply from the list of all successful tests, without regard for batches.
### B.3. Masonry flexural tension

#### Table B.6: Bond wrench test results, walls FF-3, FR-3, and FF-2

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<th>$c_v$ of $f_{fb}'$</th>
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Mean of all samples: 0.378 0.42
# B.3. Masonry flexural tension

## Table B.7: Bond wrench test results, walls SS-3 and RR-3

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<th>$f''_{tb}$ (MPa)</th>
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<th>$c_v$ of $f'_{tb}$</th>
<th>$c_v$ of $f''_{tb}$</th>
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Mean of all samples: 0.552 0.31
### B.4 Brick compression

A total of 10 half-brick specimens were tested under compression, according to CAN/CSA A82-06 (R2011) [CSA, 2011]. Bricks were saw-cut to form the half-brick specimens. Two types of bricks were mixed through construction of the wall specimens: type A being more rough-faced, and type B being more smooth-faced. Each specimen was capped with hydrostone on top and bottom prior to testing to ensure flat bearing surfaces. Specimens were tested in the compression machine in the concrete lab at BCIT. As specified in CAN/CSA A82-06, bricks were tested normal to their bedding face (i.e. in the direction of lowest profile). A typical compression failure is shown in Figure B.6. Dimensions and test results are shown in Table B.8.

![Typical brick compression failure](image)

#### Figure B.6: Typical brick compression failure

#### Table B.8: Brick compression test results, type A

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Mean: 119  
\(c_v: 0.06\)
Table B.9: Brick compression test results, type B

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<th>L (mm)</th>
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<th>H (mm)</th>
<th>A (mm²)</th>
<th>P (kN)</th>
<th>( f'_b ) (MPa)</th>
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Mean: 157  
\( c_v \): 0.07

B.5  Brick absorption

A total of 10 half-brick specimens, five each of types A and B (refer to Section B.4), were tested for absorption, according to CAN/CSA A82-06 (R2011) [CSA, 2011]. Both 24-hour immersion and 5-hour boiling absorption were measured. Results are shown in Tables B.10 and B.11.

Table B.10: Absorption test results, type A

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<th>ID</th>
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<th>Immersed in water</th>
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<th>5-hour boil</th>
<th>Absorption (%)</th>
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<td>1207</td>
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Mean: 5.10  
\( c_v \): 0.166

\( c_v \): 0.120
## B.5. Brick absorption

Table B.11: Absorption test results, type B

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<th>ID</th>
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<th>5-hour boil</th>
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Mean: 4.78 6.61

c_v: 0.025 0.021
Appendix C

Apparatus Sketches

Notes: This section contains sketches detailing the functional configuration of the test apparatus. Overall dimensions and primary member sizes are indicated. All bolts used in primary connections are ASTM A325, and are $\varnothing 1$ in unless otherwise indicated. All bolts, except those clamping the wall base connection, were tightened by turn-of-nut to satisfy slip-critical requirements. In general, bolting capacity far exceeded design forces and no slip was noted during any point of the testing. Welded connections — those not shown as bolted — are typically 6 mm fillets all around.
Appendix C. Apparatus Sketches

ELEVATION - SIDE

PLAN

C250x23 (TYP)

HSS102x102x6.4 (TYP)

4 - $\frac{3}{4}$" threaded rod through table (TYP)

C

A

B

290 (VARIES)

3947 (VARIES)

4273 (VARIES)

2606

150

1149

248
Appendix C. Apparatus Sketches

DETAIL A
SCALE 1 : 20

DETAIL B
SCALE 1 : 20

SECTION A-A

L102x102x6.4 (TYP)
C250x23 (TYP)

W460x52
C200x17 (TYP)

1850
1800

SECTION B-B

L102x102x6.4 (TYP)
W150x30 (TYP)
C250x23 (TYP)

1725
2125

SECTION A-A

L102x102x6.4 (TYP)

W460x52
C200x17 (TYP)

C250x23 (TYP)
Appendix C. Apparatus Sketches

DETAIL - TOP CONNECTION

DETAIL - BOTTOM CARRIAGE (WALL HIDDEN)
Appendix D

Instrumentation

Notes: This section contains a sketch of the instrumentation layout and a full instrumentation listing.
### Appendix D. Instrumentation

#### Table D.1: Instrumentation listing

<table>
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<th>#</th>
<th>Ch.</th>
<th>Designation</th>
<th>ID</th>
<th>Mount</th>
<th>Reference</th>
<th>Brand</th>
<th>Model</th>
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<td>8-1</td>
<td>LP E TABLE</td>
<td>LP5</td>
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<td>Floor</td>
<td>Duncan</td>
<td>606 R6k</td>
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<td>Floor</td>
<td>Duncan</td>
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Appendix E

Photos of Shake Table Testing

Notes: This section contains photos and video frame captures taken during shake table testing. The photos illustrate the crack configurations in walls, and damage (where applicable). Observed damage was sparse, and limited to minor spalling at crack locations in some walls only. Video frame captures are provided for each wall at one point in the highest stable run (typically illustrating the point of greatest rocking or carriage displacement) and in the collapse run, immediately prior to collapse. Finally, high speed video frame captures are provided for each wall in the highest stable run, illustrating the maximum displaced shape in profile.
Appendix E. Photos of Shake Table Testing

(a) East face

(b) West face

Figure E.1: Wall FF-3, cracking pattern
Appendix E. Photos of Shake Table Testing

(a) East face

(b) West face

Figure E.2: Wall FR-3, cracking pattern
Appendix E. Photos of Shake Table Testing

Figure E.3: Wall FF-2, cracking pattern

(a) East face

(b) West face
Appendix E. Photos of Shake Table Testing

Figure E.4: Wall FF-2, detail of crack step

Figure E.5: Wall FF-2, detail of spalling
Appendix E. Photos of Shake Table Testing

Figure E.6: Wall FF-2, apparatus in lowered position
Appendix E. Photos of Shake Table Testing

Figure E.7: Wall SS-3, cracking pattern
Appendix E. Photos of Shake Table Testing

(a) East face

(b) West face

Figure E.8: Wall RR-3, cracking pattern
Figure E.9: Wall RR-3, detail of spalling
Appendix E. Photos of Shake Table Testing

Figure E.10: Wall FF-3, video frame captures
Appendix E. Photos of Shake Table Testing

(a) Run 9: stable

(b) Run 10: at collapse

Figure E.11: Wall FR-3, video frame captures
Appendix E. Photos of Shake Table Testing

(a) Run 10: stable

(b) Run 11: at collapse

Figure E.12: Wall FF-2, video frame captures
Appendix E. Photos of Shake Table Testing

(a) Run 12: stable

(b) Run 13: at collapse

Figure E.13: Wall SS-3, video frame captures
Appendix E. Photos of Shake Table Testing

(a) Run 5: stable

(b) Run 7: at collapse

Figure E.14: Wall RR-3, video frame captures
Appendix E. Photos of Shake Table Testing

(a) Wall $FF-3$, run 12
(b) Wall $FR-3$, run 9
(c) Wall $FF-2$, run 10
(d) Wall $SS-3$, run 12
(e) Wall $RR-3$, run 5

Figure E.15: High speed video frame captures, stable runs
Appendix F

Shake Table Test Results

Notes: This section contains a summary page of plotted data for each shake table run. There are four time series plots:

• Relative displacement (relative to the shake table) of the top and bottom carriages and the crack location
• Rocking displacement of the crack location
• Acceleration of the top and bottom carriages and the crack location
• The total force on the wall, calculated from the measured wall accelerations

Profiles up the height of the wall are shown for the first three time series at one selected time step. The time step chosen was typically at the peak force, for uncracked walls, or at the peak of the largest rocking excursion, where rocking was notable. In runs in which initiated cracking, the profiles are shown at the last time step at which it was confirmed that the wall was uncracked (i.e. immediately before cracking). In the bottom right is a plot of [total force]–[rocking displacement at the crack] hysteresis.

It is important to note that the scales of the plots vary among the different runs so as to best illustrate the results in each case. In addition, the displacement measurements in runs with significant table or carriage motion are subject to at least ± several mm error. The rocking displacements in particular should therefore be interpreted carefully where amplitudes are small. The rocking profile is a good indicator of the relative importance of the displacement measurement error for a given run.
Appendix F. Shake Table Test Results

Wall: FF-3  Run: 1  Motion: CHHC1 @ 10%  PGA: 0.05 g  PGD: 18 mm  Time = 10.405 s

Top carriage  Bottom carriage  Crack

[Graphs showing time series data for various parameters such as displacement, acceleration, and force.]
Appendix F. Shake Table Test Results

Wall: FF-3  Run: 2  Motion: CHHC1 @ 30%  PGA: 0.19 g  PGD: 57 mm  Time = 10.955 s

- Top carriage  - Bottom carriage  - Crack
Appendix F. Shake Table Test Results

Wall: FF-3  Run: 3  Motion: CHHC1 @ 50%  PGA: 0.27 g  PGD: 96 mm  Time = 11.090 s
Appendix F. Shake Table Test Results

Wall: FF-3  Run: 4  Motion: CHHC1 @ 70%  PGA: 0.41 g  PGD: 135 mm  Time = 11.115 s

Top carriage  Bottom carriage  Crack

---

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Appendix F. Shake Table Test Results

Wall: FF-3  Run: 5  Motion: CHHC1 @ 80%  PGA: 0.51 g  PGD: 155 mm  Time = 11.140 s

Top carriage  Bottom carriage  Crack

$\frac{d_{rel}}{d_{rock}} (\text{mm})$

$\frac{a}{g}$

$F (\text{kN})$
Appendix F. Shake Table Test Results

Wall: FF-3  Run: 6  Motion: CHHC1 @ 100%  PGA: 0.64 g  PGD: 194 mm  Time = 11.175 s
Appendix F. Shake Table Test Results

Wall: FF-3   Run: 7   Motion: NGA0763 @ 50%   PGA: 0.24 g   PGD: 29 mm

\[ \text{Time} = 4.320 \text{ s} \]

\[ \text{FF-3 Run: 7 Motion: NGA0763 @ 50\% PGA: 0.24 g PGD: 29 mm} \]

\[ \text{Top carriage} \quad \text{Bottom carriage} \quad \text{Crack} \]

\[ d_{rel} (\text{mm}) \]

\[ d_{rock} (\text{mm}) \]

\[ a (g) \]

\[ F (\text{kN}) \]

\[ \text{Time (sec)} \]
Appendix F. Shake Table Test Results

Wall: FF-3  Run: 8  Motion: NGA0763 @ 60%  PGA: 0.31 g  PGD: 36 mm  Time = 4.170 s

---

Top carriage  Bottom carriage  Crack

---

Time (sec)  \( d_{rel} \) (mm)  Height (m)

---

Time (sec)  \( a \) (g)  Height (m)

---

Time (sec)  \( F \) (kN)  \( d_{rock} \) (mm)
Appendix F. Shake Table Test Results

Wall: FF-3  Run: 9  Motion: CHHC1 @ 30%  PGA: 0.18 g  PGD: 57 mm  Time = 10.955 s

Top carriage  Bottom carriage  Crack

Time (sec)  Height (m)  $d_{rel}$ (mm)  $d_{rock}$ (mm)  $a$ (g)  $F$ (kN)
Appendix F. Shake Table Test Results

Wall: FF-3  Run: 10  Motion: CHHC1 @ 50%  PGA: 0.28 g  PGD: 96 mm  Time = 11.110 s

Top carriage  Bottom carriage  Crack

---

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Appendix F. Shake Table Test Results

Wall: FF-3  Run: 11  Motion: CHHC1 @ 70%  PGA: 0.43 g  PGD: 135 mm  Time = 12.275 s

- $d_{rel}$ (mm)
- $d_{rock}$ (mm)
- $a$ (g)
- $F$ (kN)

Graphs showing time vs. displacement and acceleration for the shake table test.
Appendix F. Shake Table Test Results

Wall: FF-3  Run: 12  Motion: CHHC1 @ 80%  PGA: 0.50 g  PGD: 155 mm  Time = 12.145 s

Top carriage  Bottom carriage  Crack

Time (sec)

$d_{rel}$ (mm)

$-200$  $-100$  $0$  $100$  $200$

$6$  $8$  $10$  $12$  $14$  $16$  $18$  $20$

$d_{rock}$ (mm)

$-40$  $-20$  $0$  $20$  $40$

$6$  $8$  $10$  $12$  $14$  $16$  $18$  $20$

$a$ (g)

$-0.5$  $0.5$

$6$  $8$  $10$  $12$  $14$  $16$  $18$  $20$

$F$ (kN)

$-5$  $0$  $5$

$6$  $8$  $10$  $12$  $14$  $16$  $18$  $20$

$d_{rock}$ (mm)
Appendix F. Shake Table Test Results

Wall: FF-3  Run: 13  Motion: CHHC @ 100%  PGA: 0.63 g  PGD: 194 mm  Time = 11.555 s

Top carriage  Bottom carriage  Crack

- $d_{rel}$ (mm)
- $d_{rock}$ (mm)
- $a$ (g)
- $F$ (kN)

Time (sec)
Appendix F. Shake Table Test Results

Wall: FR-3  Run: 1  Motion: CHHC1 @ 50%  PGA: 0.43 g  PGD: 96 mm  Time = 10.910 s

- d_{rel} (mm)
- d_{rock} (mm)
- a (g)
- F (kN)

Top carriage  Bottom carriage  Crack
Appendix F. Shake Table Test Results

Wall: FR-3  Run: 2  Motion: CHHC1 @ 70%  PGA: 0.57 g  PGD: 135 mm  Time = 10.985 s

- $d_{rel}$ (mm) vs. Time (sec)
- $d_{rock}$ (mm) vs. Time (sec)
- $a$ (g) vs. Time (sec)
- $F$ (kN) vs. Time (sec)
Appendix F. Shake Table Test Results

Wall: FR-3  Run: 3  Motion: NGA0763 @ 60%  PGA: 0.27 g  PGD: 35 mm  Time = 5.180 s

- $d_{rel}$ (mm)
- $d_{rock}$ (mm)
- $a$ (g)
- $F$ (kN)

Top carriage  Bottom carriage  Crack
Appendix F. Shake Table Test Results

Wall: FR-3 Run: 4 Motion: CHHC1 @ 50% PGA: 0.30 g PGD: 96 mm Time = 10.870 s

Top carriage Bottom carriage Crack

-50 0 50 d_{rel} (mm)

-10 0 10 10 d_{rock} (mm)

-0.4 0 0.4 a (g)

-5 0 5 F (kN)
Appendix F. Shake Table Test Results

Wall: FR-3  Run: 5  Motion: CHHC1 @ 70%  PGA: 0.44 g  PGD: 135 mm  Time = 9.340 s

- $d_{rel}$ (mm)
- $d_{rock}$ (mm)
- $a$ (g)
- $F$ (kN)

Graphs showing displacement, acceleration, and force over time.
Appendix F. Shake Table Test Results

Wall: FR-3  Run: 6  Motion: CHHC1 @ 80%  PGA: 0.43 g  PGD: 155 mm  Time = 10.850 s

Top carriage  Bottom carriage  Crack

\begin{figure}
\centering
\includegraphics[width=\textwidth]{figure.png}
\end{figure}
Appendix F. Shake Table Test Results

Wall: FR-3  Run: 7  Motion: CHHC @ 90%  PGA: 0.44 g  PGD: 174 mm  Time = 9.380 s

- $d_{rel}$ (mm)
- $d_{rock}$ (mm)
- $a$ (g)
- $F$ (kN)

Time (sec)
Appendix F. Shake Table Test Results

Wall: FR-3  Run: 8  Motion: CHHC1 @ 100%  PGA: 0.51 g  PGD: 194 mm  Time = 10.620 s
Appendix F. Shake Table Test Results

Wall: FR-3  Run: 9  Motion: CHHC1 @ 110%  PGA: 0.50 g  PGD: 213 mm  Time = 13.875 s

Top carriage  Bottom carriage  Crack

-\(d_{\text{rel}}\) (mm)

-\(d_{\text{rock}}\) (mm)

\(a\) (g)

\(F\) (kN)

Time (sec)
Appendix F. Shake Table Test Results

Wall: FR-3  Run: 10  Motion: CHHC1 @ 120%  PGA: 0.61 g  PGD: 233 mm  Time = 14.030 s

Top carriage  Bottom carriage  Crack

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Appendix F. Shake Table Test Results

Wall: FF-2  Run: 1  Motion: CHHC1 @ 50%  PGA: 0.36 g  PGD: 96 mm  Time = 10.940 s

Top carriage  Bottom carriage  Crack
Appendix F. Shake Table Test Results

Wall: FF-2  Run: 2  Motion: CHHC1 @ 80%  PGA: 0.53 g  PGD: 155 mm  Time = 10.975 s

- Top carriage
- Bottom carriage
- Crack
Appendix F. Shake Table Test Results

Wall: FF-2  Run: 3  Motion: NGA0763 @ 60%  PGA: 0.27 g  PGD: 35 mm  Time = 3.375 s
Appendix F. Shake Table Test Results

Wall: FF-2  Run: 4  Motion: NGA0763 @ 70%  PGA: 0.36 g  PGD: 41 mm  Time = 4.120 s

Time (sec)  Height (m)  

Top carriage  Bottom carriage  Crack

$d_{rel}$ (mm)  $d_{rock}$ (mm)  $a$ (g)  $F$ (kN)

-10  -5  0  5  10

-10  -5  0  5  10

-0.5  0  0.5

-0.8  0  0.8

-10  -5  0  5  10

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Appendix F. Shake Table Test Results

Wall: FF-2  Run: 5  Motion: CHHC1 @ 50%  PGA: 0.31 g  PGD: 96 mm  Time = 10.930 s
Appendix F. Shake Table Test Results

Wall: FF-2  Run: 6  Motion: CHHC1 @ 70%  PGA: 0.45 g  PGD: 135 mm  Time = 11.035 s

- Top carriage
- Bottom carriage
- Crack

Time (sec)

- $d_{rel}$ (mm)
- $d_{rock}$ (mm)
- $\alpha$ (g)
- $F$ (kN)

Height (m)
Appendix F. Shake Table Test Results

Wall: FF-2  Run: 7  Motion: CHHC1 @ 80%  PGA: 0.53 g  PGD: 155 mm  Time = 11.585 s

- $d_{rel}$ (mm)
- $d_{rock}$ (mm)
- $a$ (g)
- $F$ (kN)

Graphs showing the results of the shake table test.
Appendix F. Shake Table Test Results

Wall: FF-2  Run: 8  Motion: CHHC1 @ 90%  PGA: 0.59 g  PGD: 174 mm  Time = 11.570 s

Top carriage | Bottom carriage | Crack

- $d_{rel}$ (mm)
- $d_{rock}$ (mm)
- $a$ (g)
- $F$ (kN)

![Graphs and plots of shake table test results](image-url)
Appendix F. Shake Table Test Results

Wall: FF-2  Run: 9  Motion: CHHC1 @ 100%  PGA: 0.66 g  PGD: 194 mm  Time = 11.110 s
Appendix F. Shake Table Test Results

Wall: FF-2  Run: 10  Motion: CHHC1 @ 110%  PGA: 0.69 g  PGD: 214 mm  Time = 14.640 s

Top carriage  Bottom carriage  Crack
Appendix F. Shake Table Test Results

Wall: FF-2  Run: 11  Motion: CHHC1 @ 120%  PGA: 0.74 g  PGD: 233 mm  Time = 9.245 s

Top carriage  Bottom carriage  Crack

Time (sec)  d_{rel} (mm)  d_{rock} (mm)  a (g)  F (kN)

Height (m)  Height (m)  Height (m)  a (g)  F (kN)

Time = 9.245 s
Appendix F. Shake Table Test Results

Wall: RR-3  Run: 1  Motion: NGA0763 @ 50%  PGA: 0.25 g  PGD: 29 mm  

Time = 4.305 s

Top carriage  Bottom carriage  Crack
Appendix F. Shake Table Test Results

Wall: RR-3  Run: 2  Motion: NGA0763 @ 60%  PGA: 0.27 g  PGD: 35 mm  Time = 4.550 s

- $d_{rel}$ (mm)
- $d_{rock}$ (mm)
- $a$ (g)
- $F$ (kN)
- Height (m)

Top carriage  Bottom carriage  Crack

$T = 4.550$ s
Appendix F. Shake Table Test Results

Wall: RR-3 Run: 3 Motion: NGA0763 @ 70% PGA: 0.47 g PGD: 42 mm

Time = 4.260 s

Top carriage Bottom carriage Crack

- $d_{\text{rel}}$ (mm)

- $d_{\text{rock}}$ (mm)

- $\alpha$ (g)

- $F$ (kN)

- Height (m)

- Time (sec)

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Appendix F. Shake Table Test Results

Wall: RR-3  Run: 4  Motion: CHHC1 @ 50%  PGA: 0.43 g  PGD: 96 mm

Time = 11.465 s

Top carriage  Bottom carriage  Crack
Appendix F. Shake Table Test Results

Wall: RR-3  Run: 6  Motion: CHHC1 @ 55%  PGA: 0.44 g  PGD: 106 mm  Time = 9.590 s

- Diagram showing changes in $d_{rel}$ (mm), $d_{rock}$ (mm), $a$ (g), and $F$ (kN) over time.

- Details of the test parameters and results.
Appendix F. Shake Table Test Results

Wall: RR-3  Run: 5  Motion: CHHC1 @ 60%  PGA: 0.52 g  PGD: 116 mm  Time = 10.995 s

Graphs showing displacement and acceleration over time along with force applied.

- Top carriage
- Bottom carriage
- Crack movement

Table: | Time (sec) | Top carriage | Bottom carriage | Crack |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10.995</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Appendix F. Shake Table Test Results

Wall: RR-3  Run: 7  Motion: CHHC1 @ 65%  PGA: 0.62 g  PGD: 125 mm  Time = 10.235 s

- Top carriage  - Bottom carriage  - Crack

Time (sec)  |  d_{rel} (mm)  |  d_{rock} (mm)  |  a (g)  |  F (kN)  
---|---|---|---|---
6 8 10 12 14 16 18 20  | -200 0 200 | -200 0 200 | -10 0 10 | -300 0 300

Top carriage  Bottom carriage  Crack
Appendix F. Shake Table Test Results

Wall: SS-3  Run: 1  Motion: CHHC1 @ 30%  PGA: 0.19 g  PGD: 57 mm  Time = 10.720 s

- Top carriage
- Bottom carriage
- Crack

Time (sec)
0 10 20 30 40
-10 0 10 20

$\text{d}_{\text{rel}}$ (mm)
6 8 10 12 14 16 18 20
-10 0 10

$\text{d}_{\text{rock}}$ (mm)
6 8 10 12 14 16 18 20
-10 0 10

$\alpha$ (g)
6 8 10 12 14 16 18 20
-0.1 0 0.1

$F$ (kN)
6 8 10 12 14 16 18 20
-4 -2 0 2 4

Height (m)
0 1 2 3 4
-10 0 10

$\text{d}_{\text{rel}}$ (mm)

$\text{d}_{\text{rock}}$ (mm)

$\alpha$ (g)

$F$ (kN)
Appendix F. Shake Table Test Results

Wall: SS-3  Run: 2  Motion: CHHC1 @ 40%  PGA: 0.22 g  PGD: 76 mm

Time = 10.770 s
Appendix F. Shake Table Test Results

Wall: SS-3  Run: 3  Motion: CHHC1 @ 50%  PGA: 0.27 g  PGD: 96 mm  Time = 12.790 s
Appendix F. Shake Table Test Results

Wall: SS-3  Run: 4  Motion: CHHC1 @ 60%  PGA: 0.33 g  PGD: 116 mm  Time = 12.825 s

Time (sec)

-50 0 50

F (kN)

-10 0 10

a (g)

-0.2 0 0.2

d_{rock} (mm)

-10 0 10

a (g)

-0.3 0 0.3

d_{rock} (mm)

-10 0 10

d_{rel} (mm)

-50 0 50

Height (m)

-10 0 10

Height (m)

-10 0 10

Height (m)

-10 0 10

d_{rock} (mm)

-10 0 10

d_{rel} (mm)

-50 0 50

Top carriage  Bottom carriage  Crack
Appendix F. Shake Table Test Results

Wall: SS-3  Run: 5  Motion: NGA0763 @ 50%  PGA: 0.23 g  PGD: 29 mm  Time = 4.445 s
Appendix F. Shake Table Test Results

Wall: SS-3  Run: 6  Motion: NGA0763 @ 60%  PGA: 0.27 g  PGD: 35 mm  Time = 5.030 s

Top carriage  Bottom carriage  Crack

-10 -5 0 5 10 d_{rel} (mm)

-10 -5 0 5 10 d_{rock} (mm)

-0.5 0 0.5 a (g)

-10 0 10 F (kN)

1 2 3 4 5 6 7
time (sec)

Height (m)

F (kN)
d_{rock} (mm)

1 2 3 4 5 6 7
-10 -5 0 5 10

1 2 3 4 5 6 7
-10 0 10

1 2 3 4 5 6 7
-0.3 0 0.3 a (g)

1 2 3 4 5 6 7
-10 0 10
Appendix F. Shake Table Test Results

Wall: SS-3  Run: 7  Motion: CHHC1 @ 30%  PGA: 0.18 g  PGD: 57 mm  Time = 10.740 s

Time (sec)

\[ d_{rel} \text{ (mm)} \]

Height (m)

\[ d_{rock} \text{ (mm)} \]

\[ a \text{ (g)} \]

\[ F \text{ (kN)} \]
Appendix F. Shake Table Test Results

Wall: SS-3  Run: 8  Motion: CHHC1 @ 50%  PGA: 0.26 g  PGD: 96 mm  Time = 10.905 s

- Top carriage  - Bottom carriage  - Crack

-\( d_{rel} \) (mm)

-\( d_{rock} \) (mm)

-\( a \) (g)

-\( F \) (kN)
Appendix F. Shake Table Test Results

Wall: SS-3  Run: 9  Motion: CHHC1 @ 60%  PGA: 0.33 g  PGD: 116 mm  Time = 12.920 s

- $d_{rel}$ (mm)
- $d_{rock}$ (mm)
- $a$ (g)
- $F$ (kN)
- Time (sec)

Top carriage  Bottom carriage  Crack
Appendix F. Shake Table Test Results

Wall: SS-3  Run: 10  Motion: CHHC1 @ 65%  PGA: 0.36 g  PGD: 125 mm  Time = 13.070 s

- $d_{\text{rel}}$ (mm)
- $d_{\text{rock}}$ (mm)
- $a$ (g)
- $F$ (kN)

Top carriage  Bottom carriage  Crack
Appendix F. Shake Table Test Results

Wall: SS-3  Run: 11  Motion: CHHC1 @ 70%  PGA: 0.41 g  PGD: 135 mm  Time = 11.035 s

Top carriage  Bottom carriage  Crack

\begin{align*}
\text{Time (sec)} & \quad \text{\text{d\textsubscript{rel}} (mm)} \\
\text{Height (m)} & \quad \text{\text{d\textsubscript{rock}} (mm)} \\
\text{\text{a} (g)} & \quad \text{Height (m)} \\
\text{F (kN)} & \quad \text{\text{d\textsubscript{rock}} (mm)}
\end{align*}
Appendix F. Shake Table Test Results

Wall: SS-3  Run: 12  Motion: CHHC1 @ 75%  PGA: 0.45 g  PGD: 145 mm  Time = 11.045 s

Time (sec)

Top carriage  Bottom carriage  Crack

\[ \begin{align*}
  d_{\text{rel}} (\text{mm}) & \quad \text{Height (m)} \\
  d_{\text{rock}} (\text{mm}) & \quad \text{Height (m)} \\
  a (g) & \quad \text{Height (m)} \\
  F (\text{kN}) & \quad \text{d}_{\text{rock}} (\text{mm})
\end{align*} \]
Appendix F. Shake Table Test Results

Wall: SS-3  Run: 13  Motion: CHHC1 @ 80%  PGA: 0.46 g  PGD: 155 mm  Time = 10.600 s
Appendix G

Model Validation Results

Notes: This section compares output from Working Model 2D with results recorded during shake table testing. Rocking displacement at the crack and relative displacements of the top and bottom carriages are plotted for each run.
Figure G.1: Modelled vs. tested displacements, wall FF-3, run 9
Figure G.2: Modelled vs. tested displacements, wall FF-3, run 10
Appendix G. Model Validation Results

Figure G.3: Modelled vs. tested displacements, wall $FF-\beta$, run 11
Figure G.4: Modelled vs. tested displacements, wall FF-β, run 12
Appendix G. Model Validation Results

Figure G.5: Modelled vs. tested displacements, wall FF-3, run 13
Figure G.6: Modelled vs. tested displacements, wall FR-3, run 4
Appendix G. Model Validation Results

Figure G.7: Modelled vs. tested displacements, wall FR-3, run 5
Appendix G. Model Validation Results

Figure G.8: Modelled vs. tested displacements, wall FR-3, run 6
Figure G.9: Modelled vs. tested displacements, wall FR-3, run 7
Figure G.10: Modelled vs. tested displacements, wall FR-3, run 8
Appendix G. Model Validation Results

Figure G.11: Modelled vs. tested displacements, wall FR-3, run 9
Appendix G. Model Validation Results

Figure G.12: Modelled vs. tested displacements, wall FR-3, run 10
Appendix G. Model Validation Results

Figure G.13: Modelled vs. tested displacements, wall FF-2, run 5
Appendix G. Model Validation Results

Figure G.14: Modelled vs. tested displacements, wall FF-2, run 6
Appendix G. Model Validation Results

Figure G.15: Modelled vs. tested displacements, wall FF-2, run 7
Appendix G. Model Validation Results

Figure G.16: Modelled vs. tested displacements, wall FF-2, run 8
Appendix G. Model Validation Results

Figure G.17: Modelled vs. tested displacements, wall FF-2, run 9
Figure G.18: Modelled vs. tested displacements, wall FF-2, run 10
Appendix G. Model Validation Results

Figure G.19: Modelled vs. tested displacements, wall FF-2, run 11
Appendix G. Model Validation Results

![Graphs showing model validation results for crack, top carriage, and bottom carriage displacements.](image)

Figure G.20: Modelled vs. tested displacements, wall SS-3, run 7

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Appendix G. Model Validation Results

Figure G.21: Modelled vs. tested displacements, wall SS-3, run 8
Figure G.22: Modelled vs. tested displacements, wall SS-3, run 9
Appendix G. Model Validation Results

Figure G.23: Modelled vs. tested displacements, wall SS-3, run 10
Figure G.24: Modelled vs. tested displacements, wall SS-3, run 11
Figure G.25: Modelled vs. tested displacements, wall SS-3, run 12
Figure G.26: Modelled vs. tested displacements, wall SS-3, run 13
Figure G.27: Modelled vs. tested displacements, wall *RR-3*, run 4
Appendix G. Model Validation Results

Figure G.28: Modelled vs. tested displacements, wall RR-3, run 5
Figure G.29: Modeled vs. tested displacements, wall RR-3, run 6
Appendix G. Model Validation Results

Figure G.30: Modelled vs. tested displacements, wall RR-3, run 7
Appendix H

Working Model 2D Code Listing

Notes: This section contains the complete code listing used to construct the reference case model from Chapter 6.
Sub Single_Motion(motion_filename as String, ground_motion_folder as String)

'********************************
'**** DECLARE SCRIPT VARIABLES ****
'********************************

'** Parameter inputs **

Dim wall_height as double
Dim wall_thickness as double
Dim wall_length as double
Dim wall_density as double
Dim wall_mass as double
Dim crack_height_rel as double
Dim spall_chamfer_crack as double
Dim spall_chamfer_base as double
Dim joint_thickness as double

Dim coef_friction_static as double
Dim coef_friction_kinetic as double
Dim coef_restitution as double
Dim coef_restitution_table as double

Dim damping_ratio_top as double
Dim damping_ratio_bot as double
Dim coef_damping_bot as double
Dim coef_damping_top as double
Dim rot_K_top as double
Dim rot_K_bot as double
Dim trans_K_top as double
Dim trans_K_bot as double
Dim diaphragm_mass_top as double
Dim carriage_mass_top as double
Dim beam_mass_top as double
Dim diaphragm_mass_bot as double

Dim rigid_diaphragm_top as integer
Dim rigid_diaphragm_bot as integer

Dim axial_load as double
Dim axial_load_eccentricity as double

Dim cracked as integer
Dim top_diaphragm_locked as integer
Dim bot_diaphragm_locked as integer

'** Model inputs **

Dim base_constraint_slack as double
Dim crack_constraint_slack as double
Dim frame_horiz_offset as double
Dim diaphragm_length as double
Dim bot.diaphragm.vert.offset as double
Dim top.diaphragm.vert.offset as double

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** File handling variables **

'Dim ground.motion_folder as string' removed when written as function instead of sub

'Dim motion_filename as string' removed when written as function instead of sub

** Internal variables **

Dim bot.block.height as double
Dim bot.block.mass as double
Dim bot.block.x.pos as double
Dim bot.block.y.pos as double

Dim top.block.height as double
Dim top.block.mass as double
Dim top.block.x.pos as double
Dim top.block.y.pos as double

Dim frame.height as double

Dim GM.table.inputID as integer
Dim GM.scale.input.inputID as integer
Dim top.block.inputID as integer
Dim bot.block.inputID as integer

Dim crack.height.rel_eff as double

Dim num.time.steps.to.run as long
Dim num.time.steps.to.run.checked as long

Dim GM.scale as double
Dim GM.scale.start as double
Dim GM.scale.minor.inc as double
Dim GM.scale.major.inc as double
Dim GM.scale.to.not.repeat as double
Dim GM.scale.to.not.repeat.collapsed as double

Dim collapse.check as integer
Dim final.increment as integer
Dim rocking.check.current as integer
Dim rocking.check.previous as integer
Dim rocking.threshold as double
Dim increment.number as integer
Appendix H. Working Model 2D Code Listing

113
114
115
116 '***********************************************************
117 '**** DECLARE OBJECT VARIABLES ****
118 '***********************************************************
119
120 '** Document **
121
122 Dim Doc as WMDocument
123 '** Bodies **
124
125 Dim bot_block as WMBody
126 Dim top_block as WMBody
127 Dim table as WMBody
128 Dim left_frame as WMBody
129 Dim right_frame as WMBody
130 Dim top_diaphragm as WMBody
131 Dim bot_diaphragm as WMBody
132 Dim actuator_anchor as WMBody
133 Dim top_pin as WMBody
134 Dim left_crack_block as WMBody
135 Dim right_crack_block as WMBody
136 Dim trigger1_base as WMBody
137 Dim trigger1_stop as WMBody
138 Dim trigger1_bullet as WMBody
139 '** Slots **
140
141 Dim table_slot as WMConstraint
142 Dim top_slot as WMConstraint
143 Dim bot_diaphragm_slot as WMConstraint
144 Dim top_diaphragm_slot as WMConstraint
145
146 '** Joints **
147
148 Dim left_frame_joint as WMConstraint
149 Dim right_frame_joint as WMConstraint
150 Dim top_pin_joint as WMConstraint
151 Dim left_crack_block_joint as WMConstraint
152 Dim right_crack_block_joint as WMConstraint
153 Dim uncracked_wall_joint as WMConstraint
154 Dim trigger1_joint as WMConstraint
155 '** Links **
156
157 Dim left_bot_rope as WMConstraint
158 Dim right_bot_rope as WMConstraint
159 Dim left_bot_separator as WMConstraint
160 Dim right_bot_separator as WMConstraint
161 Dim left_crack_rope as WMConstraint
162 Dim right_crack_rope as WMConstraint
163 Dim left_crack_separator as WMConstraint
164 Dim right_crack_separator as WMConstraint
Dim right_crack_separator as WMConstraint
Dim bot_spring as WMConstraint
Dim top_spring as WMConstraint
Dim left_bot_diaphragm_rope as WMConstraint
Dim right_bot_diaphragm_rope as WMConstraint
Dim left_top_diaphragm_rope as WMConstraint
Dim right_top_diaphragm_rope as WMConstraint

'** Actuator **
Dim actuator_constraint as WMConstraint

'** Inputs **
Dim GM_table as WMInput
Dim GM_scale_input as WMInput

'** Points **
Dim anchor_point as WMPoint
Dim wall_base_measuring_point as WMPoint
Dim wall_crack_bot_measuring_point as WMPoint
Dim wall_crack_top_measuring_point as WMPoint
Dim table_measuring_point as WMPoint
Dim trigger1_anchor_point as WMPoint

'** Meters **
Dim meter_left_rocking_constraints as WMOutput
Dim meter_right_rocking_constraints as WMOutput
Dim meter_rel_displacements as WMOutput
Dim meter_accelerations as WMOutput
Dim meter_table as WMOutput

'** Forces **
Dim trigger1_force as WMConstraint
Dim axial_load_force as WMConstraint

'***************
'**** INPUTS ****
'***************

%%% START OF EXCEL GENERATED FILE %%%

'** Configuration inputs **
config.ID = 100
version.ID = 014

'** Wall-specific parameters **
wall.height = 3.630 'm
wall_thickness = 0.330 'm
wall_length = 1.000 'm
wall_density = 2100 'kg/m3

crack_height_rel = 0.600 'unitless
joint_thickness = 0.012 'm
spall_chamfer_crack = 0.010 'm
spall_chamfer_base = 0.010 'm

damping_ratio_top = 0.050 'unitless
damping_ratio_bot = 0.050 'unitless
trans_K_top = 198623 'N/m
trans_K_bot = 198623 'N/m

cracked = 1 'binary (1 = cracked, 0 = uncracked)
top_diaphragm_locked = 0 'binary (1 = locked, 0 = flexible)
bot_diaphragm_locked = 0 'binary (1 = locked, 0 = flexible)

time_step = 0.005 'sec

'' Common parameters **
coef_friction_static = 0.75 'unitless
coef_friction_kinetic = 0.70 'unitless
coef_restitution = 0.02 'unitless
rot_K_top = 0 'N/rad
rot_K_bot = 0 'N/rad
carriage_mass_top = 3773 'kg
beam_mass_top = 0.1 'kg
diaphragm_mass_bot = 3773 'kg

axial_load = 0 'N
axial_load_eccentricity = 0 'm

'' Model inputs **
base_constraint_slack = 0.0002 'm

frame_horiz_offset = 4.00 'm
diaphragm_length = 0.80 'm
bot_diaphragm_vert_offset = 0.000 'm
top_diaphragm_vert_offset = -0.001 'm

overlap_error = 0.0002 'm

sig_figs = 8 'integer

'' File handling inputs **
file_prefix = "c100v014"
ground_motion_folder = "D:\Google Drive\Thesis\Analysis\Ground Motions\FEMA P695\Resampled Motions\"
output_folder = "D:\My Documents\Thesis\WM2D Runs\c100v014\"

'' Ground motion incrementing inputs **
GM_scale_start = 0.50

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Appendix H. Working Model 2D Code Listing

281 GM_scale_minor_inc = 0.10
282 GM_scale_major_inc = 0.40
283 rocking_threshold = 0.1
284
285 '%%% END OF EXCEL-GENERATED FILE %%%
286 '*****************************************************************************
287 '**** MODEL VARIABLE CALCULATIONS ****
288 '*****************************************************************************
289
290 bot_diaphragm_vert_offset = 0 'override the bottom vertical offset to 0 (new in V12)
291 diaphragm_mass_top = carriage_mass_top + beam_mass_top
292 bot_block.height = wall.height*crack_height_rel
top_block.height = wall.height - bot_block.height
293 bot_block.x_pos = 0
top_block.x_pos = 0
294 bot_block.y_pos = bot_block.height/2
top_block.y_pos = bot_block.height + top_block.height/2
295 wall_mass = (wall.height * wall.thickness * wall.length) * wall_density
296 bot_block.mass = wall_mass * crack_height_rel
top_block.mass = wall_mass * (1 - crack_height_rel)
297 frame.height = wall.height + top_diaphragm_vert_offset
298 crack_height_rel_eff = crack_height_rel * wall.height / frame.height
299 coef_damping_bot = damping_ratio_bot * (2*(trans.K_bot*(wall_mass/2+diaphragm_mass_bot))^0.5)
299 coef_damping_top = damping_ratio_top * (2*(trans.K_top*(wall_mass/2+diaphragm_mass_top))^0.5)
300
301 '*****************************************************************************
302 '**** MODEL CONSTRUCTION ****
303 '*****************************************************************************
304
305 Set Doc = WM.ActiveDocument
306 Set Doc = WM.Open("D:\Google Drive\Thesis\Analysis\WM2D\Startup File\startupC1.wm2d")
307 '**** Delete everything before starting ****
308 Doc.SelectAll True
309 Doc.Delete
310 Doc.SelectAll False
311
312 '**** Wall bodies ****
313 Set bot_block = Doc.NewBody("polygon")
314 bot_block.name = "bot_block"
315 Set top_block = Doc.NewBody("polygon")
top_block.name = "top_block"
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337  bot_block.PX.Value = bot_block.x_pos: bot_block.PY.Value = bot_block.y_pos
338  top_block.PX.Value = top_block.x_pos: top_block.PY.Value = top_block.y_pos
339
340  '**** Bottom block vertex definition ****
341  If spall_chamfer_crack = 0 And spall_chamfer_base = 0 Then
342    bot_block.AddVertex 1, -wall_thickness/2, -bot_block_height/2 'bottom left corner
343    bot_block.AddVertex 2, -wall_thickness/2, bot_block_height/2 - 0.20 'bottom of left spike
344    bot_block.AddVertex 3, -wall_thickness/2 - 0.20, bot_block_height/2 'outside of left spike
345    bot_block.AddVertex 4, -wall_thickness/2, bot_block_height/2 - 0.195 'top of left spike
346    bot_block.AddVertex 5, -wall_thickness/2, bot_block_height/2 - joint_thickness/2 'top left OUTSIDE corner
347    bot_block.AddVertex 6, -wall_thickness/2 + spall_chamfer_crack, bot_block_height/2 'top left INSIDE corner
348    bot_block.AddVertex 7, wall_thickness/2 - spall_chamfer_crack, bot_block_height/2 'top right INSIDE corner
349    bot_block.AddVertex 8, wall_thickness/2, bot_block_height/2 - joint_thickness/2 'top right OUTSIDE corner
350    bot_block.AddVertex 9, wall_thickness/2, bot_block_height/2 - 0.195 'top of right spike
351    bot_block.AddVertex 10, wall_thickness/2 + 0.20, bot_block_height/2 'outside of left spike
352    bot_block.AddVertex 11, wall_thickness/2, -bot_block_height/2 'bottom right corner
353    bot_block.DeleteVertex 13 'delete three default vertices
354    bot_block.DeleteVertex 12
355    bot_block.DeleteVertex 11
356  ElseIf spall_chamfer_crack <> 0 And spall_chamfer_base = 0 Then
357    bot_block.AddVertex 1, -wall_thickness/2, -bot_block_height/2 'bottom left corner
358    bot_block.AddVertex 2, -wall_thickness/2, bot_block_height/2 - 0.20 'bottom of left spike
359    bot_block.AddVertex 3, -wall_thickness/2 - 0.20, bot_block_height/2 'outside of left spike
360    bot_block.AddVertex 4, -wall_thickness/2, bot_block_height/2 - 0.195 'top of left spike
361    bot_block.AddVertex 5, -wall_thickness/2, bot_block_height/2 - joint_thickness/2 'top left OUTSIDE corner
362    bot_block.AddVertex 6, -wall_thickness/2 + spall_chamfer_crack, bot_block_height/2 'top left INSIDE corner
363    bot_block.AddVertex 7, wall_thickness/2 - spall_chamfer_crack, bot_block_height/2 'top right INSIDE corner
364    bot_block.AddVertex 8, wall_thickness/2, bot_block_height/2 - joint_thickness/2 'top right OUTSIDE corner
365    bot_block.AddVertex 9, wall_thickness/2, bot_block_height/2 - 0.195 'top of right spike
366    bot_block.AddVertex 10, wall_thickness/2 + 0.20, bot_block_height/2 'outside of left spike
367    bot_block.AddVertex 11, wall_thickness/2, -bot_block_height/2 'bottom right corner
368    bot_block.DeleteVertex 15 'delete three default vertices
369    bot_block.DeleteVertex 14
370    bot_block.DeleteVertex 13
371  ElseIf spall_chamfer_crack = 0 And spall_chamfer_base <> 0 Then
372    bot_block.AddVertex 1, -wall_thickness/2, -bot_block_height/2 'bottom left corner
373    bot_block.AddVertex 2, -wall_thickness/2, bot_block_height/2 - 0.20 'bottom of left spike
374    bot_block.AddVertex 3, -wall_thickness/2 - 0.20, bot_block_height/2 'outside of left spike
375    bot_block.AddVertex 4, -wall_thickness/2, bot_block_height/2 - 0.195 'top of left spike
376    bot_block.AddVertex 5, -wall_thickness/2, bot_block_height/2 - joint_thickness/2 'top left OUTSIDE corner
377    bot_block.AddVertex 6, -wall_thickness/2 + spall_chamfer_crack, bot_block_height/2 'top left INSIDE corner
378    bot_block.AddVertex 7, wall_thickness/2 - spall_chamfer_crack, bot_block_height/2 'top right INSIDE corner
379    bot_block.AddVertex 8, wall_thickness/2, bot_block_height/2 - joint_thickness/2 'top right OUTSIDE corner
380    bot_block.AddVertex 9, wall_thickness/2, bot_block_height/2 - 0.195 'top of right spike
381    bot_block.AddVertex 10, wall_thickness/2 + 0.20, bot_block_height/2 'outside of left spike
382    bot_block.AddVertex 11, wall_thickness/2, -bot_block_height/2 'bottom right corner
383    bot_block.DeleteVertex 15 'delete three default vertices
384    bot_block.DeleteVertex 14
385    bot_block.DeleteVertex 13
386
387  ElseIf spall_chamfer_crack = 0 And spall_chamfer_base <> 0 Then
bot_block.AddVertex 1, -wall_thickness/2 + spall_chamfer_base, -bot_block_height/2 'bottom left INSIDE corner
bot_block.AddVertex 2, -wall_thickness/2, -bot_block_height/2 + joint_thickness/2 'bottom left OUTSIDE corner

bot_block.AddVertex 3, -wall_thickness/2, bot_block_height/2 - 0.20 'bottom of left spike
bot_block.AddVertex 4, -wall_thickness/2 - 0.20, bot_block_height/2 'outside of left spike
bot_block.AddVertex 5, -wall_thickness/2, bot_block_height/2 - 0.195 'top of left spike

bot_block.AddVertex 6, -wall_thickness/2, bot_block_height/2 'top left corner
bot_block.AddVertex 7, wall_thickness/2, bot_block_height/2 'top right corner

bot_block.AddVertex 8, wall_thickness/2, bot_block.height/2 - 0.195 'top of right spike
bot_block.AddVertex 9, wall_thickness/2 + 0.20, bot_block.height/2 'outside of right spike
bot_block.AddVertex 10, wall_thickness/2, bot_block.height/2 - 0.20 'bottom of right spike

bot_block.AddVertex 11, wall_thickness/2, -bot_block.height/2 + joint_thickness/2 'bottom right OUTSIDE corner
bot_block.AddVertex 12, wall_thickness/2 - spall_chamfer_base, -bot_block.height/2 'bottom right INSIDE corner

bot_block.DeleteVertex 15 'delete three default vertices
bot_block.DeleteVertex 14
bot_block.DeleteVertex 13

Else

bot_block.AddVertex 1, -wall_thickness/2 + spall_chamfer_base, -bot_block.height/2 'bottom left INSIDE corner
bot_block.AddVertex 2, -wall_thickness/2, -bot_block.height/2 + joint_thickness/2 'bottom left OUTSIDE corner

bot_block.AddVertex 3, -wall_thickness/2, bot_block.height/2 - 0.20 'bottom of left spike
bot_block.AddVertex 4, -wall_thickness/2 - 0.20, bot_block.height/2 'outside of left spike
bot_block.AddVertex 5, -wall_thickness/2, bot_block.height/2 - 0.195 'top of left spike

bot_block.AddVertex 6, -wall_thickness/2, bot_block.height/2 - joint_thickness/2 'top left corner
bot_block.AddVertex 7, -wall_thickness/2 + spall_chamfer.crack, bot_block.height/2 'top left INSIDE corner
bot_block.AddVertex 8, wall_thickness/2 - spall_chamfer.crack, bot_block.height/2 'top right INSIDE corner
bot_block.AddVertex 9, wall_thickness/2, bot_block.height/2 - joint_thickness/2 'top right OUTSIDE corner

bot_block.AddVertex 10, wall_thickness/2, bot_block.height/2 - 0.195 'top of right spike
bot_block.AddVertex 11, wall_thickness/2 + 0.20, bot_block.height/2 'outside of right spike
bot_block.AddVertex 12, wall_thickness/2, bot_block.height/2 - 0.20 'bottom of right spike

bot_block.AddVertex 13, wall_thickness/2, -bot_block.height/2 + joint_thickness/2 'bottom right OUTSIDE corner
bot_block.AddVertex 14, wall_thickness/2 - spall_chamfer_base, -bot_block.height/2 'bottom right INSIDE corner

bot_block.DeleteVertex 17 'delete three default vertices
bot_block.DeleteVertex 16
bot_block.DeleteVertex 15

End If

'**** Top block vertex definition ****
top_block.AddVertex 1, wall_thickness/2, top_block_height/2 'top right corner
If spall_chamfer_crack = 0 Then
top_block.AddVertex 2, wall_thickness/2, -top_block_height/2 'bottom right corner
  top_block.AddVertex 3, -wall_thickness/2, -top_block_height/2 'bottom left corner
  top_block.AddVertex 4, -wall_thickness/2, top_block_height/2 'top left corner
  top_block.DeleteVertex 7 'delete three default vertices
top_block.DeleteVertex 6
top_block.DeleteVertex 5
Else
  top_block.AddVertex 2, wall_thickness/2, -top_block_height/2 + joint_thickness/2 'bottom right OUTSIDE corner
top_block.AddVertex 3, wall_thickness/2 - spall_chamfer_crack, -top_block_height/2 'bottom right INSIDE corner
top_block.AddVertex 4, -wall_thickness/2 + spall_chamfer_crack, -top_block_height/2 'bottom left INSIDE corner
top_block.AddVertex 5, -wall_thickness/2, top_block_height/2 'bottom left OUTSIDE corner
top_block.AddVertex 6, -wall_thickness/2, top_block_height/2 'top left corner
  top_block.DeleteVertex 9 'delete three default vertices
top_block.DeleteVertex 8
top_block.DeleteVertex 7
End If

'**** Assign body ID values for wall blocks ****
top_block_inputID = top_block.ID
bot_block_inputID = bot_block.ID

'**** Fixed joint between wall blocks (if uncracked) ****
If cracked = 0 Then
  Set uncracked_wall_joint = Doc.NewConstraint("SquarePin")
  uncracked_wall_joint.name = "uncracked_wall_joint"
  uncracked_wall_joint.Point(1).Body = top_block
    uncracked_wall_joint.Point(1).PX.Value = -0.02 'noticed instability when this fixed joint was located in the center of the wall body, seems to be OK when moved off center
  uncracked_wall_joint.Point(1).PY.Value = -top_block_height/2
  Set uncracked_wall_joint.Point(2).Body = bot_block
  uncracked_wall_joint.Point(2).PX.Value = -0.02
  uncracked_wall_joint.Point(2).PY.Value = bot_block_height/2
top_block.PY.Value = top_block_y_pos 'put the top block back to to the correct height, for some reason it moves when defining the above constraint

End If

'**** Table ****

Set table = Doc.NewBody("rectangle")
table.name = "table"
table.PX.Value = 0
table.PY.Value = -0.10

table.width.value = 4.00
table.height.value = 0.20

table.staticfriction.value = 0
table.kineticfriction.value = 0

'**** Table constraint ****

Set table_slot = Doc.NewConstraint("KeyedHSlot")

Set table_slot.Point(2).Body = table 'point 2 is the point

Set table_slot.Point(2).PX.Value = -2
table_slot.Point(2).PY.Value = -.1

Set table_slot.Point(1).PY.Value = -.2 'point 1 is the slot - this MUST be defined after point 2 for some reason

table_slot.name = "table_slot"
table_slot.Point(2).name = "table_slot_pt"
table_slot.Point(1).name = "table_slot_bk_pt"

'**** Vertical frame segments ****

Set left_frame = Doc.NewBody("rectangle")
left_frame.name = "left_frame"
left_frame.PX.Value = -frame_horiz_offset
left_frame.PY.Value = frame_height/2
left_frame.width.value = 0.10
left_frame.height.value = frame.height

Set right_frame = Doc.NewBody("rectangle")
right_frame.name = "right_frame"
right_frame.PX.Value = frame_horiz_offset
right_frame.PY.Value = frame_height/2
right_frame.width.value = 0.10
right_frame.height.value = frame.height

'**** Vertical frame fixed joints ****

Set left_frame_joint = Doc.NewConstraint("SquarePin")

Set left_frame_joint.Point(2).Body = left_frame

Set left_frame_joint.Point(2).PX.Value = 0.00
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538  left_frame_joint.Point(2).PY.Value = -frame_height/2
539  Set left_frame_joint.Point(1).Body = table
540  left_frame_joint.Point(1).PX.Value = -frame_horiz_offset
541  left_frame_joint.Point(1).PY.Value = 0.10
542
543  left_frame_joint.name = "left_frame_joint"
544  left_frame_joint.Point(2).name = "left_frame_joint_pt2"
545  left_frame_joint.Point(1).name = "left_frame_joint_pt1"
546
547  Set right_frame_joint = Doc.NewConstraint("SquarePin")
548  Set right_frame_joint.Point(2).Body = right_frame
549  right_frame_joint.Point(2).PX.Value = 0.00
550  right_frame_joint.Point(2).PY.Value = -frame_height/2
551  Set right_frame_joint.Point(1).Body = table
552  right_frame_joint.Point(1).PX.Value = frame_horiz_offset
553  right_frame_joint.Point(1).PY.Value = 0.10
554
555  right_frame_joint.name = "right_frame_joint"
556  right_frame_joint.Point(2).name = "right_frame_joint_pt2"
557  right_frame_joint.Point(1).name = "right_frame_joint_pt1"
558
559  table.PX.Value = 0  
560  'put the table back to 0, for some reason it moves when defining the above
561  constraints
562
563  '**** Diaphragms ****
564
565  Set bot_diaphragm = Doc.NewBody("rectangle")
566  bot_diaphragm.name = "bot_diaphragm"
567  bot_diaphragm.PX.Value = 0
568  bot_diaphragm.PY.Value = bot_diaphragm_vert_offset
569  bot_diaphragm.width.value = diaphragm_length
570  bot_diaphragm.height.value = 0.050
571
572  Set top_diaphragm = Doc.NewBody("rectangle")
573  top_diaphragm.name = "top_diaphragm"
574  top_diaphragm.PX.Value = 0
575  top_diaphragm.PY.Value = wall_height + top_diaphragm_vert_offset
576  top_diaphragm.width.value = diaphragm_length
577  top_diaphragm.height.value = 0.050
578
579  '**** Top pin connection ****
580
581  Set top_pin = Doc.NewBody("polygon")
582  top_pin.name = "top_beam"
583  top_pin.PX.Value = 0: top_pin.PY.Value = wall_height
584
585  top_pin.AddVertex 1, wall_thickness/2, 0  'bottom right corner
586  top_pin.AddVertex 2, -wall_thickness/2, 0  'bottom left corner
587  top_pin.AddVertex 3, 0, top_diaphragm.vert_offset 'top corner
588
589  top_pin.DeleteVertex 6  'delete three default vertices
590  top_pin.DeleteVertex 5
591  top_pin.DeleteVertex 4
592
593  '**** Top pin fixed joint ****
Set top_pin_joint = Doc.NewConstraint("SquarePin")
Set top_pin_joint.Point(1).Body = top_block
Set top_pin_joint.Point(1).PX.Value = 0
Set top_pin_joint.Point(1).PY.Value = top_block.height/2
Set top_pin_joint.Point(2).Body = top_pin
Set top_pin_joint.Point(2).PX.Value = 0.00
Set top_pin_joint.Point(2).PY.Value = 0
Set top_pin_joint.name = "top_beam_fixed_joint"
Set top_pin_joint.Point(1).name = "top_beam_fixed_joint_pt1"
Set top_pin_joint.Point(2).name = "top_beam_fixed_joint_pt2"

Set top_block.PY.Value = top_block_y_pos 'put the top block back to to the correct height, for some reason it moves when defining the above constraint

'**** Slot for top pin ****
Set top_slot = Doc.NewConstraint("VSlot")
Set top_slot.Point(2).Body = top_pin 'point 2 is the point
Set top_slot.Point(2).PX.Value = 0
Set top_slot.Point(2).PY.Value = top_diaphragm_vert_offset
Set top_slot.Point(1).Body = top_diaphragm
Set top_slot.Point(1).PX.Value = 0
Set top_slot.Point(1).name = "top_pin_slot_bkpt" 'point 1 is the slot - this MUST be defined after point 2 for some reason
Set top_slot.name = "top_pin_slot"
Set top_slot.Point(2).name = "top_pin_slot_pt"
Set top_slot.Point(1).name = "top_pin_slot_bkpt"

'**** Lateral restraints at base of the wall (to diaphragm) ****
Set left_bot_rope = Doc.NewConstraint("Rope")
left_bot_rope.Point(2).PX.Value = -wall_thickness/2 + spall_chamfer_base
left_bot_rope.Point(2).PY.Value = -bot_block_height/2 + bot_diaphragm_vert_offset
Set left_bot_rope.Point(2).Body = bot_block 'set the body after defining the relative co-ordinates on that body, then it avoids moving the body
left_bot_rope.Point(1).PX.Value = -diaphragm_length/2
left_bot_rope.Point(1).PY.Value = 0
Set left_bot_rope.Point(1).Body = bot_diaphragm
left_bot_rope.length.value = diaphragm_length/2 - wall_thickness/2 + base_constraint_slack + spall_chamfer_base

Set right_bot_rope = Doc.NewConstraint("Rope")
right_bot_rope.Point(2).PX.Value = wall_thickness/2 - spall_chamfer_base
right_bot_rope.Point(2).PY.Value = -bot_block_height/2 + bot_diaphragm_vert_offset
Set right_bot_rope.Point(2).Body = bot_block
right_bot_rope.Point(1).PX.Value = diaphragm_length/2
right_bot_rope.Point(1).PY.Value = 0
Set right_bot_rope.Point(1).Body = bot_diaphragm
right_bot_rope.length.value = diaphragm_length/2 - wall_thickness/2 + base_constraint_slack + spall_chamfer_base
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644
645  Set left_bot_separator = Doc.NewConstraint("Separator")
646  left_bot_separator.length.value = diaphragm_length/2 - wall_thickness/2 - base_constraint_slack
647  left_bot_separator.Point(2).PX.Value = -wall_thickness/2 - spall_chamfer_base
648  left_bot_separator.Point(2).PY.Value = -bot_block_height/2 + bot_diaphragm_vert_offset
649  Set left_bot_separator.Point(2).Body = bot_block
650  left_bot_separator.Point(1).PX.Value = -diaphragm_length/2
651  left_bot_separator.Point(1).PY.Value = 0
652  Set left_bot_separator.Point(1).Body = bot_diaphragm
653  left_bot_separator.length.value = diaphragm_length/2 - wall_thickness/2 - base_constraint_slack +
654  spall_chamfer_base

655
656  Set right_bot_separator = Doc.NewConstraint("Separator")
657  right_bot_separator.length.value = diaphragm_length/2 - wall_thickness/2 - base_constraint_slack
658  right_bot_separator.Point(2).PX.Value = wall_thickness/2 - spall_chamfer_base
659  right_bot_separator.Point(2).PY.Value = -bot_block_height/2 + bot_diaphragm_vert_offset
660  Set right_bot_separator.Point(2).Body = bot_block
661  right_bot_separator.Point(1).PX.Value = diaphragm_length/2
662  right_bot_separator.Point(1).PY.Value = 0
663  Set right_bot_separator.Point(1).Body = bot_diaphragm
664  right_bot_separator.length.value = diaphragm_length/2 - wall_thickness/2 - base_constraint_slack +
665  spall_chamfer_base

666  left_bot_rope.name = "left_bot_rope"
667  left_bot_rope.Point(2).name = "left_bot_rope_pt2"
668  left_bot_rope.Point(1).name = "left_bot_rope_pt1"
669
670  right_bot_rope.name = "right_bot_rope"
671  right_bot_rope.Point(2).name = "right_bot_rope_pt2"
672  right_bot_rope.Point(1).name = "right_bot_rope_pt1"
673
674  left_bot_separator.name = "left_bot_separator"
675  left_bot_separator.Point(2).name = "left_bot_separator_pt2"
676  left_bot_separator.Point(1).name = "left_bot_separator_pt1"
677
678  right_bot_separator.name = "right_bot_separator"
679  right_bot_separator.Point(2).name = "right_bot_separator_pt2"
680  right_bot_separator.Point(1).name = "right_bot_separator_pt1"
681
682  '**** Lateral restraints at crack (between wall blocks) ****
683
684  Set left_crack_rope = Doc.NewConstraint("Rope")
685  left_crack_rope.length.value = 10   'make rope temporarily long so it doesn’t move other bodies
during definition
686  left_crack_rope.Point(2).PX.Value = -wall_thickness/2 + spall_chamfer_crack
687  left_crack_rope.Point(2).PY.Value = -top_block_height/2
688  Set left_crack_rope.Point(2).Body = top_block
689  left_crack_rope.Point(1).PX.Value = -wall_thickness/2 - 0.20
690  left_crack_rope.Point(1).PY.Value = bot_block.height/2
691  Set left_crack_rope.Point(1).Body = bot_block
692  top_block.PX.Value = top_block.x.pos: top_block.PY.Value = top_block.y.pos
693  left_crack_rope.length.value = 0.20 + spall_chamfer_crack + crack_constraint_slack
694
695  Set right_crack_rope = Doc.NewConstraint("Rope")
right_crack_rope.length.value = 10  # make rope temporarily long so it doesn't move other bodies during definition
right_crack_rope.Point(2).PX.Value = wall_thickness/2 - spall_chamfer_crack
right_crack_rope.Point(2).PY.Value = -top_block_height/2
Set right_crack_rope.Point(2).Body = top_block
right_crack_rope.Point(1).PX.Value = wall_thickness/2 + 0.20
right_crack_rope.Point(1).PY.Value = bot_block.height/2
Set right_crack_rope.Point(1).Body = bot_block
top_block.PX.Value = top.block.x_pos; top_block.PY.Value = top.block.y_pos
right_crack_rope.length.value = 0.20 + spall_chamfer_crack + crack_constraint_slack

Set left_crack_separator = Doc.NewConstraint("Separator")
left_crack_separator.length.value = 0.01  # make separator temporarily short so it doesn't move other bodies during definition
left_crack_separator.Point(2).PX.Value = -wall_thickness/2 + spall_chamfer_crack
left_crack_separator.Point(2).PY.Value = -top_block_height/2
Set left_crack_separator.Point(2).Body = top_block
left_crack_separator.Point(1).PX.Value = -wall_thickness/2 - 0.20
left_crack_separator.Point(1).PY.Value = bot_block.height/2
Set left_crack_separator.Point(1).Body = bot_block
top_block.PX.Value = top.block.x_pos; top_block.PY.Value = top.block.y_pos
left_crack_separator.length.value = 0.20 + spall_chamfer_crack - crack_constraint_slack

Set right_crack_separator = Doc.NewConstraint("Separator")
right_crack_separator.length.value = 0.01  # make separator temporarily short so it doesn't move other bodies during definition
right_crack_separator.Point(2).PX.Value = wall_thickness/2 - spall_chamfer_crack
right_crack_separator.Point(2).PY.Value = -top_block_height/2
Set right_crack_separator.Point(2).Body = top_block
right_crack_separator.Point(1).PX.Value = wall_thickness/2 + 0.20
right_crack_separator.Point(1).PY.Value = bot_block.height/2
Set right_crack_separator.Point(1).Body = bot_block
top_block.PX.Value = top.block.x_pos; top_block.PY.Value = top.block.y_pos
right_crack_separator.length.value = 0.20 + spall_chamfer_crack - crack_constraint_slack

left_crack_rope.name = "left_crack_rope"
left_crack_rope.Point(2).name = "left_crack_rope_pt2"
left_crack_rope.Point(1).name = "left_crack_rope_pt1"

right_crack_rope.name = "right_crack_rope"
right_crack_rope.Point(2).name = "right_crack_rope_pt2"
right_crack_rope.Point(1).name = "right_crack_rope_pt1"

left_crack_separator.name = "left_crack_separator"
left_crack_separator.Point(2).name = "left_crack_separator_pt2"
left_crack_separator.Point(1).name = "left_crack_separator_pt1"

right_crack_separator.name = "right_crack_separator"
right_crack_separator.Point(2).name = "right_crack_separator_pt2"
right_crack_separator.Point(1).name = "right_crack_separator_pt1"

'**** Set "Active When" conditions for constraints at crack and at base of wall ****
left_crack_rope.AlwaysActive = False
left_crack_separator.AlwaysActive = False
Appendix H. Working Model 2D Code Listing

```plaintext
750  right_crack_rope.AlwaysActive = False
751  right_crack_separator.AlwaysActive = False
752  left_bot_rope.AlwaysActive = False
753  left_bot_separator.AlwaysActive = False
754  right_bot_rope.AlwaysActive = False
755  right_bot_separator.AlwaysActive = False
756
757  left_crack_rope.ActiveWhen.Formula = "Body["+ str$(top_block_inputID) +"]\ p \ r - Body["+ str$(bot_block_inputID) +"]\ p \ r >= -0.0000001"
758  left_crack_separator.ActiveWhen.Formula = "Body["+ str$(top_block_inputID) +"]\ p \ r - Body["+ str$(bot_block_inputID) +"]\ p \ r <= 0.0000001"
759  right_crack_rope.ActiveWhen.Formula = "Body["+ str$(top_block_inputID) +"]\ p \ r - Body["+ str$(bot_block_inputID) +"]\ p \ r <= 0.0000001"
760  right_crack_separator.ActiveWhen.Formula = "Body["+ str$(top_block_inputID) +"]\ p \ r - Body["+ str$(bot_block_inputID) +"]\ p \ r <= 0.0000001"
761  left_bot_rope.ActiveWhen.Formula = "Body["+ str$(bot_block_inputID) +"]\ p \ r >= 0"
762  left_bot_separator.ActiveWhen.Formula = "Body["+ str$(bot_block_inputID) +"]\ p \ r >= 0"
763  right_bot_rope.ActiveWhen.Formula = "Body["+ str$(bot_block_inputID) +"]\ p \ r <= 0"
764  right_bot_separator.ActiveWhen.Formula = "Body["+ str$(bot_block_inputID) +"]\ p \ r <= 0"
765
766
767  '**** Bottom diaphragm spring or rigid link ****
768
If bot_diaphragm_locked = 0 Then
769
    Set bot_spring = Doc.NewConstraint("SpringDamper")
770    bot_spring.Point(1).PX.Value = 0
771    bot_spring.Point(1).PY.Value = -frame_height/2 + bot_diaphragm_vert_offset
772    Set bot_spring.Point(1).Body = left_frame
773    bot_spring.Point(2).PX.Value = -diaphragm_length/2
774    Set bot_spring.Point(2).Body = bot_diaphragm
775    bot_spring.Point(2).PY.Value = 0
776    bot_spring.length.value = frame_horiz_offset - diaphragm_length/2
777    bot_spring.name = "bot_spring"
778    bot_spring.Point(2).name = "bot_spring_pt2"
779    bot_spring.Point(1).name = "bot_spring_pt1"
780    bot_spring.K.value = trans_K_bot
781    bot_spring.damperK.value = coef_damping_bot
782
Else
783    Set bot_spring = Doc.NewConstraint("Rod")
784    bot_spring.Point(1).PX.Value = 0
785    bot_spring.Point(1).PY.Value = -frame_height/2 + bot_diaphragm_vert_offset
786    Set bot_spring.Point(1).Body = left_frame
787    bot_spring.Point(2).PX.Value = -diaphragm_length/2
788    Set bot_spring.Point(2).Body = bot_diaphragm
789    bot_spring.Point(2).PY.Value = 0
790    bot_spring.length.value = frame_horiz_offset - diaphragm_length/2
791    bot_spring.name = "bot_rod"
792    bot_spring.Point(2).name = "bot_rod_pt2"
793    bot_spring.Point(1).name = "bot_rod_pt1"
```

372
End If

bot_diaphragm.PX.Value = 0
top_diaphragm.PX.Value = 0

'**** Diaphragm slot constraints ****
Set bot_diaphragm_slot = Doc.NewConstraint("KeyedHSlot")
Set bot_diaphragm_slot.Point(2).Body = bot_diaphragm 'point 2 is the point
top_diaphragm_slot.Point(2).PX.Value = diaphragm_length/2
bot_diaphragm_slot.Point(1).PY.Value = bot_diaphragm_vert_offset 'point 1 is the slot - this MUST be defined after point 2 for some reason

Set top_diaphragm_slot = Doc.NewConstraint("KeyedHSlot")
Set top_diaphragm_slot.Point(2).Body = top_diaphragm 'point 2 is the point
top_diaphragm_slot.Point(2).PX.Value = diaphragm_length/2
top_diaphragm_slot.Point(1).PY.Value = frame_height 'point 1 is the slot - this MUST be defined after point 2 for some reason

bot_diaphragm_slot.name = "bot_diaphragm_slot"
bot_diaphragm_slot.Point(2).name = "bot_diaphragm_slot_pt"
bot_diaphragm_slot.Point(1).name = "bot_diaphragm_slot_bkpt"

top_diaphragm_slot.name = "top_diaphragm_slot"
top_diaphragm_slot.Point(2).name = "top_diaphragm_slot_pt"
top_diaphragm_slot.Point(1).name = "top_diaphragm_slot_bkpt"

'**** Top diaphragm spring or rigid link ****
If top_diaphragm_locked = 0 Then
Set top_spring = Doc.NewConstraint("SpringDamper")
top_spring.Point(1).PX.Value = 0
top_spring.Point(1).PY.Value = frame_height/2
Set top_spring.Point(1).Body = left_frame
top_spring.Point(2).PX.Value = -diaphragm_length/2
Set top_spring.Point(2).Body = top_diaphragm
top_spring.Point(2).PY.Value = 0
top_spring.length.value = frame_horiz_offset - diaphragm_length/2
top_spring.name = "top_spring"
top_spring.Point(2).name = "top_spring_pt2"
top_spring.Point(1).name = "top_spring_pt1"
top_spring.K.value = trans_K_top
top_spring.damperK.value = coef_damping_top
Else
Set top_spring = Doc.NewConstraint("Rod")

373
top_spring.length.value = frame_horiz_offset - diaphragm_length/2

top_spring.Point(1).PX.Value = 0

top_spring.Point(1).PY.Value = frame_height/2

top_spring.Point(2).PX.Value = -diaphragm_length/2

top_spring.Point(2).PY.Value = 0

Set top_spring.Point(1).Body = left_frame
Set top_spring.Point(2).Body = top_diaphragm

top_spring.name = "top_rod"
top_spring.Point(2).name = "top_rod_pt2"
top_spring.Point(1).name = "top_rod_pt1"

End If

bot_diaphragm.PX.Value = 0
top_diaphragm.PX.Value = 0

table.PX.value = 0
top_block.PX.value = 0
bot_block.PX.value = 0

'**** Axial load ****

Set axial_load_force = Doc.NewConstraint("Force")
axial_load_force.AlwaysActive = True
Set axial_load_force.Point(1).Body = top_block
axial_load_force.Point(1).px.value = axial_load_eccentricity
axial_load_force.Point(1).py.value = top_block_height/2
axial_load_force.Point(1).name = "axial_load_pt"

axial_load_force.FX.Value = 0
axial_load_force.FY.Value = -axial_load
axial_load_force.name = "axial_load"

'**** Actuator anchor block ****

Set actuator_anchor = Doc.NewBody("rectangle")
actuator_anchor.PX.Value = 10
actuator_anchor.PY.Value = -0.10
actuator_anchor.width.value = 1.00
actuator_anchor.height.value = 1.00

Set anchor_point = Doc.NewPoint("Anchor")
Set anchor_point.Body = actuator_anchor
actuator_anchor.name = "actuator_anchor"
anchor_point.name = "actuator_anchor_pt"

'**** Actuator ****

Set actuator_constraint = Doc.NewConstraint("Actuator")
actuator_constraint.Point(1).PX.Value = -0.50
Appendix H. Working Model 2D Code Listing

```plaintext
actuator_constraint.Point(1).PY.Value = 0
Set actuator_constraint.Point(1).Body = actuator_anchor
actuator_constraint.Point(2).PX.Value = 2
Set actuator_constraint.Point(2).Body = table
actuator_constraint.Point(2).PY.Value = 0
actuator_constraint.ActuatorType = "Length"
actuator_constraint.name = "actuator"
actuator_constraint.Point(2).name = "actuator_pt2"
actuator_constraint.Point(1).name = "actuator_pt1"

'**** Measuring points on bodies ****
Set wall_base_measuring_point = Doc/NewPoint("Point")
wall_base_measuring_point.name = "wall_base_measuring_point"
Set wall_crack_bot_measuring_point = Doc/NewPoint("Point")
wall_crack_bot_measuring_point.name = "wall_crack_bot_measuring_point"
Set wall_crack_top_measuring_point = Doc/NewPoint("Point")
wall_crack_top_measuring_point.name = "wall_crack_top_measuring_point"
Set table_measuring_point = Doc/NewPoint("Point")
table_measuring_point.name = "table_measuring_point"
Set wall_base_measuring_point.Body = bot_block
wall_base_measuring_point.py.value = -bot_block_height/2
Set wall_crack_bot_measuring_point.Body = bot_block
wall_crack_bot_measuring_point.py.value = bot_block_height/2
Set wall_crack_top_measuring_point.Body = top_block
wall_crack_top_measuring_point.py.value = -top_block_height/2
Set table_measuring_point.Body = table
'table_measuring_point.py.value = -top_block_height/2

'**** GM data table ****
Set GM_table = Doc/NewInput()
GM_table.Format = "Table"
GM_table.X = 20  'position of the text box from top left corner of screen
GM_table.Y = 20
GM_table.Name = "Ground Motion"
GM_table.inputID = GM_table.ID 'get the input ID number (for reference in formlas)
GM_table.TimeColumn = 1
GM_table.DataColumn = 2

'**** GM scale box ****
Set GM_scale_input = Doc/NewInput()
GM_scale_input.Format = "TextBox"
GM_scale_input.X = 40  'position of the text box from top left corner of screen
GM_scale_input.Y = 70
GM_scale_input.Name = "Ground Motion Scale"
GM_scale_input.Min = 0
GM_scale_input.Max = 100
GM_scale_input.inputID = GM_scale_input.ID 'get the input ID number (for reference in formlas)
```

375
Appendix H. Working Model 2D Code Listing

```
968 '**** Set actuator formula to reference the ground motion table and ground motion scale ****
969 actuator_constraint.field.formula = "input[" + str$(GM_table_inputID) + "]*input[" + str$(GM_scale_input_inputID) + "] + 7.50"
972 '**** Set collision properties ****
974 'Deselect everything
976 Doc.SelectAll False
979 'Select pairs of objects and turn off collisions
980 Doc.Select bot_diaphragm
981 Doc.Select bot_block
982 Doc.Collide False
983 Doc.SelectAll False
985 Doc.Select bot_diaphragm
986 Doc.Select table
987 Doc.Collide False
988 Doc.SelectAll False
990 Doc.Select top_diaphragm
991 Doc.Select top_block
992 Doc.Collide False
994 Doc.SelectAll False
995 Doc.Select top_diaphragm
996 Doc.Select top_pin
998 Doc.Collide False
999 Doc.SelectAll False
1000 Doc.Select actuator_anchor
1001 Doc.Select right_frame
1003 Doc.Collide False
1004 Doc.SelectAll False
1005 Doc.Select actuator_anchor
1007 Doc.Select left_frame
1008 Doc.Collide False
1009 Doc.SelectAll False
1010 Doc.Select actuator_anchor
1012 Doc.Select table
1013 Doc.Collide False
1014 Doc.SelectAll False
1015 '**** Assign object parameters ****
1018 bot_block.elasticity.value = coef_restitution
1019 bot_block.staticfriction.value = coef_friction_static
1020 bot_block.kineticfriction.value = coef_friction_kinetic
1021 bot.block.mass.value = bot.block.mass
1022```

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```apl
1023 top_block.elasticity.value = coef_restitution
1024 top_block.staticfriction.value = coef_friction_static
1025 top_block.kineticfriction.value = coef_friction_kinetic
1026 top_block.mass.value = top_block_mass
1027
1028 top_pin.mass.value = beam_mass_top
1029 top_diaphragm.mass.value = carriage_mass_top
1030 bot_diaphragm.mass.value = diaphragm_mass_bot
1031
1032 table.elasticity.value = coef_restitution_table
1033
1034 '**************************
1035 '**** MODEL PROPERTIES ****
1036 '**************************
1037
1038 Doc.SimulationMode = "accurate"
1039 Doc.AutoAnimationStep = False
1040 Doc.VariableIntegrationStep = False
1041 Doc.AutoOverlapError = True
1042 Doc.AutoIntegratorError = False
1043 Doc.WarnInaccurate = False
1044 Doc.WarnInconsistent = False
1045 Doc.WarnOverlap = False
1046 Doc.WarnRedundant = False
1047 Doc.AutoSignificantDigits = False
1048 Doc.AutoAssemblyError = True
1049
1050 Doc.IntegrationStep = time_step
1051 Doc.IntegratorError = overlap_error
1052 Doc.AnimationStep = time_step
1053
1054 '****************
1055 '**** METERS ****
1056 '****************
1057
1058 '**** Left side rocking constraints ****
1059
1060 'Set meter_left_rocking_constraints = Doc.NewOutput()
1061 'meter_left_rocking_constraints.X = 200
1062 'meter_left_rocking_constraints.Y = 20
1063 'meter_left_rocking_constraints.Name = "Left Side Constraints"
1064 'meter_left_rocking_constraints.Height = 200
1065 'meter_left_rocking_constraints.Width = 300
1066 'meter_left_rocking_constraints.Column(0).Label = "Time"
1067 'meter_left_rocking_constraints.Column(0).Cell.Formula = "time"
1068 'meter_left_rocking_constraints.Column(1).Label = "Crack rope length"
1069 'meter_left_rocking_constraints.Column(1).Cell.Formula = "Constraint["+ str$(left_crack_rope.ID) +"].length"
1070 'meter_left_rocking_constraints.Column(2).Label = "Crack separator length"
```
Appendix H. Working Model 2D Code Listing

1077 'm1 serious
1078 'm1 0
1079 'm1
1080 'm1
1081 'm1
1082 'm1
1083 'm1
1084 'm1
1085 'm1
1086 'm1
1087 'm1
1088 'm1
1089 'm1
1090 'm1
1091 'm1
1092 'm1
1093 'm1
1094 'm1
1095 'm1
1096 'm1
1097 'm1
1098 'm1
1099 'm1
1100 'm1
1101 'm1
1102 'm1
1103 'm1
1104 'm1
1105 'm1
1106 'm1
1107 'm1
1108 'm1
1109 'm1
1110 'm1
1111 'm1
1112 'm1
1113 'm1
1114 'm1
1115 'm1
1116 'm1
1117 'm1
1118 'm1
1119 'm1
1120 'm1
1121 'm1
1122 'm1
1123 Set meter_rel_displacements = Doc.NewOutput()
Appendix H. Working Model 2D Code Listing

```plaintext
1124 meter_rel_displacements.X = 600
1125 meter_rel_displacements.Y = 300
1126 meter_rel_displacements.Name = "Relative Displacements (to table)"
1127 meter_rel_displacements.Height = 200
1128 meter_rel_displacements.Width = 300
1129 meter_rel_displacements.Column(0).Label = "Time"
1130 meter_rel_displacements.Column(0).Cell.Formula = "time"
1131 meter_rel_displacements.Column(1).Label = "Top diaphragm"
1132 meter_rel_displacements.Column(1).Cell.Formula = "top_for"
1133 meter_rel_displacements.Column(2).Label = "Crack (mean)"
1134 meter_rel_displacements.Column(2).Cell.Formula = crack_for
1135 meter_rel_displacements.Column(3).Label = "Bottom diaphragm"
1136 meter_rel_displacements.Column(3).Cell.Formula = bot_for
1137 meter_rel_displacements.Column(4).Label = "Rocking displacement"
1138 meter_rel_displacements.Column(4).Cell.Formula = rock_for

1140 '**** Accelerations ****
1141 Set meter_accelerations = Doc.NewOutput()
1142 meter_accelerations.X = 200
1143 meter_accelerations.Y = 300
1144 meter_accelerations.Name = "Accelerations"
1145 meter_accelerations.Height = 200
1146 meter_accelerations.Width = 300
1147 meter_accelerations.Column(0).Label = "Time"
1148 meter_accelerations.Column(0).Cell.Formula = "time"
1149 meter_accelerations.Column(1).Label = "Top diaphragm"
1150 meter_accelerations.Column(1).Cell.Formula = "Point[" + str$(top_slot.Point(2).ID) + "]\cdot a.x"
1151 meter_accelerations.Column(2).Label = "Crack (mean)"
1152 meter_accelerations.Column(2).Cell.Formula = "(Point[" + str$(wall_crack_top_measuring_point.ID) + "]\cdot a.x + Point[" + str$(wall_crack_bot_measuring_point.ID) + "]\cdot a.x)/2"
1153 meter_accelerations.Column(3).Label = "Bottom diaphragm"
1154 meter_accelerations.Column(3).Cell.Formula = "Point[" + str$(wall_base_measuring_point.ID) + "]\cdot a.x"

1158 '**** Table ****
1159 Set meter_table = Doc.NewOutput()
1160 meter_table.X = 200
1161 meter_table.Y = 150
1162 meter_table.Name = "Table"
1163 meter_table.Height = 200
1164 meter_table.Width = 300
1165 meter_table.Column(0).Label = "Time"
1166 meter_table.Column(0).Cell.Formula = "time"
1167 meter_table.Column(1).Label = "Table Accel"
1168 meter_table.Column(1).Cell.Formula = "Point[" + str$(table_measuring_point.ID) + "]\cdot a.x"
1169 meter_table.Column(2).Label = "Table Displ"
1170 meter_table.Column(2).Cell.Formula = "Point[" + str$(table_measuring_point.ID) + "]\cdot p.x"

1174 '***************
1175 '**** TRIGGERS ****
1176 '***************
1177 '**** Trigger 1 (for rocking threshold) ****
```
Appendix H. Working Model 2D Code Listing
1179
1180
1181
1182
1183

Set trigger1_base = Doc.NewBody("polygon")
Set trigger1_bullet = Doc.NewBody("rectangle")
trigger1_base.name = "trigger1_base"
trigger1_bullet.name = "trigger1_bullet"

1184
1185
1186
1187

trigger1_base.PX.Value = 0
trigger1_base.PY.Value = -1.00

1188
1189
1190
1191
1192
1193
1194

trigger1_base.addVertex
trigger1_base.addVertex
trigger1_base.addVertex
trigger1_base.addVertex
trigger1_base.addVertex
trigger1_base.addVertex

1,
2,
3,
4,
5,
6,

-0.10,0
-0.10,0.1
0.3,0.1
0.3,0.3
0.4,0.3
0.4,0

1195
1196
1197
1198

trigger1_base.DeleteVertex 9
trigger1_base.DeleteVertex 8
trigger1_base.DeleteVertex 7

’delete three default vertices

1199
1200
1201
1202

Set trigger1_anchor_point = Doc.NewPoint("Anchor")
Set trigger1_anchor_point.Body = trigger1_base
trigger1_anchor_point.name = "trigger1_anchor_pt"

1203
1204

’Projectile

1205
1206
1207
1208
1209

trigger1_bullet.PX.Value = 0
trigger1_bullet.PY.Value = -0.80
trigger1_bullet.width.value = 0.20
trigger1_bullet.height.value = 0.20

1210
1211
1212

trigger1_base.elasticity.value = 0
trigger1_bullet.elasticity.value = 0

1213
1214
1215
1216

’trigger1_bullet.staticfriction.value = 0.3
’trigger1_bullet.kineticfriction.value = 0.3

1217
1218

’Set the trigger to activate when rocking displacement exceeds X% of wall thickness

1219
1220
1221
1222
1223
1224
1225
1226
1227

Set trigger1_force = Doc.NewConstraint("Force")
trigger1_force.AlwaysActive
= False
trigger1_force.ActiveWhen.Formula = "abs(Output["+ str$(meter_rel_displacements.ID) +"].y4) >= " &
wall_thickness * rocking_threshold
Set trigger1_force.Point(1).Body = trigger1_bullet
trigger1_force.FX.Value = 500
trigger1_force.FY.Value = 0
trigger1_force.name = "trigger1_force"
trigger1_force.Point(1).name = "trigger1_force_pt"

1228
1229
1230
1231
1232
1233

’*******************
’**** EXECUTION ****

380


Appendix H. Working Model 2D Code Listing

'********************

1234 '**** Read the ground motion ****
1235 'MsgBox "Ground motion file: " & motion_filename
1236 '**** Read the ground motion into the data table in the model ****
1237 GM_table.ReadTable ground_motion_folder & motion_filename
1238 '**** Read ground motion prefix, ID, and duration from filename ****
1239 GM_prefix = Left(motion_filename,7)
1240 'MsgBox GM_prefix
1241 GM_ID = Left(motion_filename,17)
1242 'MsgBox GM.ID
1243 GM_duration = CInt(Left(Right(motion_filename,9),3))
1244 'MsgBox GM.duration
1245 '**** Calculate number of time steps to run to match motion duration ****
1246 num_time_steps_to_run = CLng(GM_duration/time_step)
1247 'MsgBox num_time_steps_to_run
1248 '**** Clear and re-assign pause controls ****
1249 While Doc.PauseControlCount > 0
1250 Doc.DeletePauseControl 1
1251 Wend
1252 Doc.NewPauseControl
1253 Doc.NewPauseControl
1254 Doc.PauseControl(1).Formula = "time > " & str$(GM_duration+2)
1255 Doc.PauseControl(2).Formula = "Point[" + str$(top_slot.Point(2).ID) + "]\.p\$.y < " + str$(wall_height - 0.5)
1256 Doc.SetPauseControlType 1, "stop"
1257 Doc.SetPauseControlType 2, "stop"
1258 '############ START LOOP THROUGH GROUND MOTION SCALES ############'
1259 GM_scale = GM_scale_start
1260 GM_scale_input.Value = GM_scale
1261 'GM_scale_input.Value = 1.00
1262 rocking.check.previous = 0
1263 rocking.check.current = 0
1264 collapse.check = 0
1265 increment_number = 1
1266 final.increment = 0
1267 GM.scale.to.not.repeat = 0
1268 GM.scale.to.not.repeat.collapsed = 0
Appendix H. Working Model 2D Code Listing

1289 'Exit Sub
1290
1291 While collapse_check <= 1
1292
1293 '**** Assemble output file name ****
1294 save_name = file_prefix & " - " & GM_ID & " - " & Format$(GM_scale,"0.00")
1295 'MsgBox "Filename: " & save_name
1296
1297 '**** Export meter data for calculated number of frames ****
1298 Doc.ExportStartFrame = 0
1299
1300 'MsgBox "Number of time steps to run: " & num_time_steps_to_run
1301 If num_time_steps_to_run > 32767 Then
1302 num_time_steps_to_run = 32767
1303 End If
1304
1305 'MsgBox "Number of time steps to run: " & num_time_steps_to_run
1306 Doc.ExportStopFrame = num_time_steps_to_run
1307 'Doc.ExportStopFrame = 13000
1308 Doc.ExportMeterData output_folder & save_name & ".txt"
1309
1310 '**** Check if rocking occurred ****
1311 If trigger1_bullet.PX.Value > 0.0001 Then
1312 'msgbox "Rocking detected"
1313 rocking_check_current = 1
1314 End If
1315
1316 '**** Check if run ended due to collapse or time expiry ****
1317 'msgbox "final position: " & Format$(top_pin.PY.Value, "0.00") & " initial position: " & Format$(wall_height, "0.00")
1318 If top_pin.PY.Value < (wall_height - 0.3) Then
1319 'msgbox "Wall collapse"
1320 collapse_check = 1
1321 Else
1322 'msgbox "Time expiry"
1323 End If
1324
1325 '**** If this is the final run after previous collapse, then exit the while loop ****
1326 If final_increment = 1 Then
1327 Goto End_GM_Loop
1328 End If
1329
1330 '**** Reset for next run ****
1331 Doc.Reset
1332
1333 '**** Increment ground motion scale by appropriate amount ****
1334 If rocking_check.current = 0 Then
1335 GM.scale = GM.scale + GM.scale_major_inc '*** if no rocking has ever been detected , then do a big increment
1336 ElseIf rocking_check.current = 1 Then '*** if rocking has been detected now,
1337 If rocking_check.previous = 1 Then '*** if rocking has been detected before,
If collapse_check = 0 Then
  GM_scale = GM_scale + GM_scale_minor_inc '*** and the wall has not
collapsed yet, then keep going with small increments
  ’Msgbox "GM_scale: " & GM_scale & ": to skip: " &
  GM_scale_to_not_repeat & ": to skip (collapse): " &
  GM_scale_to_not_repeat_collapsed
  ’Msgbox GM_scale - GM_scale_to_not_repeat_collapsed
  If Abs(GM_scale - GM_scale_to_not_repeat)<0.00001 Then
    GM_scale = GM_scale + GM_scale_minor_inc '*** if we have already
    hit this GM scale before (with the major increments) then
    don’t repeat it (skip to next minor inc up)
  End If
  If Abs(GM_scale - GM_scale_to_not_repeat_collapsed)<0.00001 Then
    GM_scale = GM_scale - GM_scale_minor_inc/2 '*** and the wall had
    previously collapsed at this increment, then go back half
    of one small increment and make next one the last increment
    final_increment = 1
  End If
Else
  GM_scale = GM_scale - GM_scale_minor_inc/2 '*** and the wall has
  collapsed, then go back half of one small increment and make next
  one the last increment
  final_increment = 1
End If
Else '*** if this is the first time rocking has been detected,
  ‘*** make a note of current GM scale so we don’t repeat it with the small
  increments
  If collapse_check = 0 Then
    GM_scale_to_not_repeat = GM_scale '*** if it has NOT collapsed
  Else
    GM_scale_to_not_repeat_collapsed = GM_scale '*** if it has collapsed
  End If
  ‘*** go back in increment
  If increment_number = 1 Then
    GM_scale = GM_scale - GM_scale_major_inc '*** and if this is the first
    run, then go back one full big increment (whether wall has
    collapsed or not)
  Else
    GM_scale = GM_scale - GM_scale_major_inc + GM_scale_minor_inc '*** and
    if this is not the first run, then go back to one small
    increment above the previous large one
  End If
  collapse_check = 0 '*** reset collapse check to 0 (in case the wall had
collapsed - we want to reach first collapse with small increments)
End If
Appendix H. Working Model 2D Code Listing

1384        GM_scale_input.Value = GM_scale
1385
1386        '**** Set previous rocking check to current value ****
1387        rocking_check_previous = rocking_check_current
1388
1389        '**** Update increment number ****
1390        increment_number = increment_number + 1
1391
1392        '**** Update collapse_check_previous ****
1393        If collapse_check = 1 Then
1394            collapse_check_previous = 1
1395        End If
1396
1397        Wend
1398
1399        End_GM_Loop:
1400        '############ END LOOP THROUGH GROUND MOTION SCALES ############'
1401
1402
1403
1404
1405
1406
1407
1408
1409        'GM.scale_input.Value = 1.00
1410
1411
1412
1413
1414        End Sub
1415
1416
1417        '%%% END OF FILE %%%
Appendix I

Ground Motions

Notes: This section contains a listing of the ground motions used in the parametric study of Chapter 6. The accelerograms were downloaded from the PEER database (http://peer.berkeley.edu/peer_ground_motion_database). The motions and normalization factors used match those prescribed by FEMA P695 [FEMA, 2009].
### Appendix I. Ground Motions

Table I.1: Listing of ground motions

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## Appendix I. Ground Motions

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