Title
Experimental and analytical investigation of the seismic performance of low-rise masonry veneer buildings

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Experimental and Analytical Investigation of the Seismic Performance of Low-Rise Masonry Veneer Buildings

A dissertation submitted in partial satisfaction of the requirements for the degree Doctor of Philosophy

in

Structural Engineering

by

Hussein Okail

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Professor Vlado Lubarda
Professor Jose Restrepo
Professor Peter Shearer

2010
The Dissertation of Hussein Okail is approved, and it is acceptable in quality and form for publication on microfilm and electronically:


University of California, San Diego
2010
To my parents Osama and Kadria
Your love is my compass
TABLE OF CONTENTS

SIGNATURE PAGE........................................................................................................ iii
DEDICATION.................................................................................................................. iv
TABLE OF CONTENTS ................................................................................................. v
LIST OF TABLES ........................................................................................................... ix
LIST OF FIGURES ......................................................................................................... x
LIST OF SYMBOLS ..................................................................................................... xvi
VITA................................................................................................................................. xx
ABSTRACT OF THE DISSERTATION ................................................................... xxi

CHAPTER 1 ...................................................................................................................... 1
INTRODUCTION............................................................................................................. 1
  1.1 Background ........................................................................................................... 1
  1.2 Overview of Research Program ............................................................................. 3
  1.3 Scope and Objectives ......................................................................................... 4
  1.4 Dissertation Outline ............................................................................................ 5

CHAPTER 2 ...................................................................................................................... 9
LITERATURE REVIEW ................................................................................................ 9
  2.1 Introduction ......................................................................................................... 9
  2.2 Low-Rise Timber Houses ..................................................................................... 9
  2.3 Masonry Veneer ................................................................................................ 12
  2.4 Design of Masonry Veneer ............................................................................... 14
  2.5 Performance in Earthquakes and Typical Veneer Failures ................................... 18
  2.6 Previous Experimental Research Efforts ............................................................. 19
  2.7 Previous Analytical Research Efforts .................................................................. 24

CHAPTER 3 .................................................................................................................... 33
TESTING PROGRAM ON VENEER WALL SEGMENTS...................................... 33
  3.1 Introduction ............................................................................................................ 33
  3.2 Experimental Program ......................................................................................... 33
  3.2.1 Wood-Stud Frame ........................................................................................... 34
  3.2.2 Clay Masonry Veneer ...................................................................................... 35
  3.2.3 Veneer Ties and Joint Reinforcement ............................................................. 35
  3.3 Instrumentation Schemes .................................................................................... 36
  3.4 Test Setup ............................................................................................................. 37
  3.5 Testing Protocols ................................................................................................ 39
  3.6 Material Properties ............................................................................................. 41
  3.7 Acknowledgement of Publications ..................................................................... 49
CHAPTER 4 .................................................................................................................... 50
OUT-OF-PLANE RESPONSE OF WALL SEGMENTS .................................................. 50
  4.1 Introduction........................................................................................................... 50
  4.2 Experimental Observations.................................................................................. 51
  4.2.1 Specimen Wood 5........................................................................................... 51
  4.2.2 Specimen Wood 6........................................................................................... 52
  4.2.3 Specimen Wood 7........................................................................................... 52
  4.2.4 Specimen Wood 7X......................................................................................... 53
  4.2.5 Specimen Wood 8........................................................................................... 53
  4.2.6 Specimen Wood 9........................................................................................... 54
  4.2.7 Specimen Wood 10......................................................................................... 54
  4.2.8 Summary of the Experimental Observations................................................. 55
  4.3 Evaluation of the Experimental Results.............................................................. 57
  4.3.1 Dynamic Properties of the Tested Walls.......................................................... 57
  4.3.2 Displacement and Acceleration Responses...................................................... 58
  4.4 Analysis of Masonry Veneer Subjected to Out-of-Plane Excitation...................... 59
  4.4.1 Beam Element Discretization and Boundary Conditions.................................. 59
  4.4.2 Materials and Cross-Sectional Properties...................................................... 61
  4.4.3 Hysteretic Models for Veneer Anchors............................................................. 61
  4.4.4 Time History Analyses..................................................................................... 63
  4.4.5 Comparison of Analytical and Experimental Results....................................... 64
  4.4.5.1 Specimen Wood 5...................................................................................... 64
  4.4.5.2 Specimen Wood 7...................................................................................... 65
  4.4.5.3 Specimen Wood 9...................................................................................... 66
  4.4.5.4 Specimen Wood 10.................................................................................... 67
  4.4.6 Capacity of Veneer Ties.................................................................................... 67
  4.4.7 Simulation of Failure Mode............................................................................. 69
  4.5 Parametric Study on the Out-of-Plane Response.................................................. 70
  4.5.1 Influence of Corrugated Anchor Capacity....................................................... 71
  4.5.2 Influence of Corrugated Anchor Initial Stiffness.............................................. 73
  4.5.3 Influence of Rigid Anchor Initial Stiffness...................................................... 73
  4.5.4 Influence of Corrugated Anchor Spacing....................................................... 74
  4.5.5 Influence of Corrugated Anchor Initial Slack................................................. 74
  4.5.6 Conclusions of Parametric Study..................................................................... 75
  4.6 Summary and Conclusions.................................................................................. 76
  4.7 Acknowledgement of Publications...................................................................... 100

CHAPTER 5 .................................................................................................................. 101
IN-PLANE RESPONSE OF WALL SEGMENTS .......................................................... 101
  5.1 Introduction......................................................................................................... 101
  5.2 Experimental Observations................................................................................ 101
  5.2.1 Specimen Wood 1.......................................................................................... 101
  5.2.2 Specimen Wood 2.......................................................................................... 102
  5.2.3 Specimen Wood 3.......................................................................................... 102
  5.2.4 Specimen Wood 4.......................................................................................... 103
CHAPTER 6

SEISMIC PERFORMANCE OF FULL-SCALE ONE-STORY MASONRY VENEER WOOD-STUD HOUSE

6.1 Introduction ........................................................................................................... 131
6.2 The Test Structure .............................................................................................. 131
6.2.1 General Description, Design and Construction of the Prototype Building 131
6.2.2 Wood-Stud Frames ....................................................................................... 133
6.2.3 Wood-Joist Roof Diaphragm ........................................................................ 134
6.2.4 Clay Masonry Veneer .................................................................................. 136
6.2.5 Veneer Ties and Joint Reinforcement .......................................................... 137
6.3 Instrumentation Schemes .................................................................................... 137
6.4 Test Setup ........................................................................................................... 138
6.5 Testing Protocols ............................................................................................... 139
6.6 Experimental Observations ............................................................................... 140
6.6.1 Behavior under Low Level Ground Motions ............................................. 140
6.6.2 Behavior under the Design Basis and Maximum Considered Earthquakes 140
6.6.3 Behavior under Severe Ground Shaking ...................................................... 141
6.7 Analysis of the Experimental Results ................................................................ 143
6.8 Design Implications based on Test Results .................................................... 147
6.9 Summary and Conclusions .............................................................................. 150
6.10 Acknowledgement of Publications ................................................................ 167

CHAPTER 7

PARAMETRIC STUDIES ON SYSTEM PERFORMANCE AND DESIGN RECOMMENDATIONS

7.1 Summary .............................................................................................................. 168
7.2 Influence of In-Plane Veneer on the Seismic Performance of Wood Shear Walls ......................................................................................................................... 168
7.2.1 Outline of Analytical Study ......................................................................... 168
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.2.2</td>
<td>Seismic Mass Representation</td>
<td>169</td>
</tr>
<tr>
<td>7.2.3</td>
<td>Time History Analyses</td>
<td>170</td>
</tr>
<tr>
<td>7.2.4</td>
<td>Discussion of Analytical Results</td>
<td>171</td>
</tr>
<tr>
<td>7.2.5</td>
<td>Evaluation of the Relative Velocity Index</td>
<td>172</td>
</tr>
<tr>
<td>7.2.6</td>
<td>Conclusions of the Study</td>
<td>172</td>
</tr>
<tr>
<td>7.3</td>
<td>Out-of-Plane Seismic Performance of Two-Story Masonry Veneer</td>
<td>173</td>
</tr>
<tr>
<td>7.3.1</td>
<td>Outline of Analytical Study</td>
<td>173</td>
</tr>
<tr>
<td>7.3.2</td>
<td>Finite Element Discretization</td>
<td>174</td>
</tr>
<tr>
<td>7.3.3</td>
<td>Modeling of In-Plane Wood-Stud Shear wall</td>
<td>174</td>
</tr>
<tr>
<td>7.3.4</td>
<td>Incremental Dynamic Analysis</td>
<td>175</td>
</tr>
<tr>
<td>7.3.5</td>
<td>Discussion of Analytical Results for Discontinuous Veneer</td>
<td>176</td>
</tr>
<tr>
<td>7.3.6</td>
<td>Discussion of Analytical Results for Continuous Veneer</td>
<td>178</td>
</tr>
<tr>
<td>7.3.7</td>
<td>Conclusions of the Study</td>
<td>178</td>
</tr>
<tr>
<td>7.4</td>
<td>Simplified Three-Dimensional Modeling of Prototype Building</td>
<td>179</td>
</tr>
<tr>
<td>7.4.1</td>
<td>Model Description and Finite Element Discretization</td>
<td>179</td>
</tr>
<tr>
<td>7.4.2</td>
<td>Comparison of Analytical and Experimental Results</td>
<td>180</td>
</tr>
<tr>
<td>7.5</td>
<td>Design Recommendations</td>
<td>180</td>
</tr>
<tr>
<td>8</td>
<td>SUMMARY AND CONCLUSIONS</td>
<td>205</td>
</tr>
<tr>
<td>8.1</td>
<td>Summary</td>
<td>205</td>
</tr>
<tr>
<td>8.2</td>
<td>Conclusions</td>
<td>210</td>
</tr>
<tr>
<td>8.2.1</td>
<td>Testing</td>
<td>210</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design Recommendations</td>
<td>213</td>
</tr>
<tr>
<td>8.3</td>
<td>Recommendations for Future Research</td>
<td>214</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Analytical Modeling</td>
<td>212</td>
</tr>
</tbody>
</table>
LIST OF TABLES

Table 2.1. Prescriptive Requirements for Corrugated Ties for SDC C and Below ........ 28

Table 3.1. Summary of Wood-Stud Wall Specimens ....................................................... 43
Table 3.2. Test Sequence for the Wood-Stud Specimens ............................................... 43
Table 3.3. Parameters for Design Response Spectra ...................................................... 44

Table 4.1. Summary of Experimental Observations ......................................................... 78
Table 4.2. Evolution of the Fundamental Mode Frequency (Hz) ..................................... 78
Table 4.3. Hysteretic Rule Parameters ............................................................................ 78
Table 4.4. Summary of Maximum Anchor Forces ......................................................... 79
Table 4.5. Properties of Analyzed Models (Parametric Study) ........................................ 79
Table 4.6. Summary of Parametric Study – Corrugated Ties .......................................... 80
Table 4.7. Summary of Parametric Study – Rigid Ties .................................................... 81
Table 4.8. Summary of Parametric Study – Corrugated Ties, Closer Spacing ................. 81
Table 4.9. Summary of Parametric Study – Slack Models ............................................. 82

Table 5.1. Summary of Experimental Observations ......................................................... 118
Table 5.2. Change of Fundamental Mode Frequency (Hz) .............................................. 118

Table 6.1. Test Sequence for the One-Story House ......................................................... 151
Table 6.2. Evolution of the Fundamental Mode Frequency (Hz) .................................... 151
LIST OF FIGURES

Figure 2.1. Components of a Typical Timber Building .................................................... 29
Figure 2.2. Components of a Typical Masonry Veneer Wall ........................................... 29
Figure 2.3. Veneer Anchors ............................................................................................ 30
Figure 2.4. Out-of-Plane Failure .................................................................................... 30
Figure 2.5. In-Plane Failure ........................................................................................... 30
Figure 2.6. Anchor Pullout from Backing ........................................................................ 31
Figure 2.7. Anchor Pullout from Joints .......................................................................... 31
Figure 2.8. Ties not Embedded in Mortar Joints .............................................................. 31
Figure 2.9. Poor Bond .................................................................................................... 31
Figure 2.10. Severe Corrosion ...................................................................................... 31
Figure 2.11. Distress due to Foundation Settlement ....................................................... 31
Figure 2.12. Tie Misalignment ...................................................................................... 32

Figure 3.1. Typical Details of the Tested Walls (1 m = 39.4 in.) .................................... 45
Figure 3.2. Veneer Ties ................................................................................................. 45
Figure 3.3. Locations of Displacement Transducers and Accelerometers on Veneer and
Back ing ........................................................................................................................ 46
Figure 3.4. Test Setup for In-Plane and Out-of-Plane Walls ........................................... 47
Figure 3.5. Ground Motion Records .............................................................................. 47
Figure 3.6. Pseudo-Acceleration Response Spectra (5% Damping) ................................ 48
Figure 3.7. Sieve Analysis of the used Sand .................................................................. 48

Figure 4.1. Partial Nail Extraction .................................................................................. 83
Figure 4.2. Nail Pullout ................................................................................................. 83
Figure 4.3. Partial Nail Extraction ................................................................................ 83
Figure 4.4. Bed-Joint Cracking ..................................................................................... 83
Figure 4.5. Wood 7 after Tarzana 125% ....................................................................... 83
Figure 4.6. Failure Pattern of Wood 7X ....................................................................... 83
Figure 4.7. Extraction of the Top row of Ties ................................................................. 84
Figure 4.8. Failure Pattern of Wood 8 .......................................................................... 84
Figure 4.9. Uplift of Double Top Plate ......................................................................... 84
Figure 4.10. Failure Pattern of Wood 9 ......................................................................... 84
Figure 4.11. Tie Pullout .............................................................................................. 84
Figure 4.12. Failure Pattern of Wood 10 ...................................................................... 84
Figure 4.13. Tie Pullout from Mortar Joint .................................................................... 85
Figure 4.14. Anchor Pull-Through ............................................................................... 85
Figure 4.15. Deformed Tie Hole .................................................................................. 85
Figure 4.16. Crack Patterns of the O-O-P Walls .............................................................. 86
Figure 4.17. Wood 5 (D2mt) – T70 .............................................................................. 86
Figure 4.18. Wood 6 (D2mt) – T70 .............................................................................. 86
Figure 4.19. Wood 7 (D2et) – T70 ................................................................................ 87
Figure 4.20. Wood 8 (D4mt) – T70 .............................................................................. 87
Figure 5.15. Sliding Spring Model ................................................................. 125
Figure 5.16. Rocking Spring Model ............................................................... 125
Figure 5.17. Load Deflection Response for the Top of the Stud-Wall from Quasi-Static Test and Analytical Model (1 m = 39.4 in.) ................................................................. 126
Figure 5.18. Response History – Sylmar 150 % (for sensor location see Figure 3.3) .... 127
Figure 5.19. Tie Deformation Distribution - Wood 1 ....................................... 128
Figure 5.20. Tie Deformation Distribution - Wood 2 ....................................... 128
Figure 5.21. Mechanism of Tie Deformation Distribution ................................ 129
Figure 5.22. Tie Deformation Distribution - Wood 2 – Quasi-Static .................... 129

Figure 6.1. One-Story Test Building with Wood-Stud Walls and Masonry Veneer (1 inch = 25.4 mm) ................................................................................................. 152
Figure 6.2. Design Details of North and South Walls ...................................... 153
Figure 6.3. Design Details of East Wall (west wall had similar design but had different vertical tie spacing and had discontinuous veneer corners; dimensions in mm, 1 m = 39.4 in) ...................................................................................................................... 154
Figure 6.4. Erection of Separate Frames ......................................................... 155
Figure 6.5. Hold-Downs .................................................................................. 155
Figure 6.6. Orientation of Roof Joists ............................................................... 155
Figure 6.7. Joist to Rim Connection .................................................................. 155
Figure 6.8. Rim to Top Plate Connection ......................................................... 155
Figure 6.9. Blocking and Bracing ................................................................. 155
Figure 6.10. Locations of Displacement Transducers and Accelerometers in the Structure (Numbers without designation represent sensor locations in both veneer and wood walls) ............................................................................................................. 156
Figure 6.11. Shaking-Table Test Setup ............................................................ 157
Figure 6.12. Original Sylmar Ground Motion Record ....................................... 158
Figure 6.13. Pseudo-Acceleration Response Spectra (5% Damping) .................. 158
Figure 6.14. Collapse of Veneer on West Wall ................................................ 159
Figure 6.15. Typical Nail Extraction from the West Wall Wood-Studs ............... 159
Figure 6.16. Collapse of Veneer on East Wall (1 x Sylmar 200 %) ..................... 159
Figure 6.17. Anchor Pullout (Mortar Joints) and Pull-Through ............................ 159
Figure 6.18. Diagonal Cracking at the South-East Top Corner .......................... 159
Figure 6.19. Pier Diagonal Cracking (South Wall) ............................................ 159
Figure 6.20. Joint Reinforcement from the East Wall ....................................... 160
Figure 6.21. Collapse of Veneer on the South Wall ......................................... 160
Figure 6.22. Typical Veneer Anchor from the Collapsed South Wall .................. 160
Figure 6.23. South Wall Veneer Anchor Subjected to Severe Shear Deformations ................................................................. 160
Figure 6.24. Sliding of the In-Plane Veneer .................................................... 160
Figure 6.25. Nail Extraction by In-Plane Sliding (North Wall) .......................... 160
Figure 6.26. North Wall on the verge of Collapse ......................................... 161
Figure 6.27. Incipient Collapse of the Lintel (North Wall) ............................... 161
Figure 6.28. Extraction of End Nails from Blockings (East Wall) ..................... 161
Figure 6.29. Extraction of Diagonal Brace (Roof - East Side) ........................... 161
Figure 6.30. Average Peak Roof Acceleration and Dynamic Amplification versus PGA ................................................................. 162
Figure 6.31. Peak In-Plane Veneer Sliding versus PGA (1 m = 39.4 in.) ................. 162
Figure 6.32. Top In-Plane Veneer Displacements Subtracted by Base Sliding versus PGA (1 m = 39.4 in.) ................................................... 163
Figure 6.33. Relative Top In-Plane Displacements between Veneer and Wood Wall versus PGA (1 m = 39.4 in.) ................................................................. 163
Figure 6.34. Peak Out-of-Plane Accelerations versus PGA for West Wall ............... 164
Figure 6.35. Peak Out-of-Plane Accelerations versus PGA for East Wall ............... 164
Figure 6.36. Peak In-Plane Wood Wall Displacements versus PGA (1 m = 39.4 in.) 165
Figure 6.37. Peak Out-of-Plane Displacements versus PGA for East Wood Wall (1 m = 39.4 in.) ................................................................. 165
Figure 6.38. Peak Axial Tie Deformations versus PGA for East Wall (1 m = 39.4 in.) 166
Figure 6.39. Relative Velocity Index Time Histories (displacement transducer locations identified in Figure 6.36; 1 m = 39.4 in.)............................... 166

Figure 7.1. Prototype Building Models (1 m = 39.4 in.) ........................................ 183
Figure 7.2. Variation of Fundamental Frequency .................................................. 184
Figure 7.3. Peak Top Wood Displacement (Sylmar – DBE) (1 m = 39.4 in.) .......... 184
Figure 7.4. Peak Top Wood Displacement (Sylmar – MCE) (1 m = 39.4 in.) ......... 185
Figure 7.5. Peak Top Wood Displacement (Tarzana – DBE) (1 m = 39.4 in.) ........... 185
Figure 7.6. Peak Top Wood Displacement (Tarzana – MCE) (1 m = 39.4 in.) ......... 186
Figure 7.7. Peak Top Wood Displacement (El Centro – DBE) (1 m = 39.4 in.) .......... 186
Figure 7.8. Peak Top Wood Displacement (El Centro – MCE) (1 m = 39.4 in.) ......... 187
Figure 7.9. Relative Velocity Index – Aspect Ratio 3.75 (El Centro – DBE) .......... 187
Figure 7.10. Relative Velocity Index – Aspect Ratio 1.25 (El Centro – DBE) .......... 188
Figure 7.11. Finite Element Discretization – Two-Story Model ............................. 188
Figure 7.12. Monotonic Response of In-Plane Shear Walls ................................. 189
Figure 7.13. Cyclic Response of In-Plane Shear Walls ........................................... 189
Figure 7.14. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 406 mm, Average Strength, Discontinuous Veneer for Two-Story Model) .............. 190
Figure 7.15. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 406 mm, Upper Bound Strength, Discontinuous Veneer for Two-Story Model) .............. 190
Figure 7.16. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 406 mm, Lower Bound Strength, Discontinuous Veneer for Two-Story Model) .............. 191
Figure 7.17. Base Shear Time History (One-Story In-Plane Shear Wall) – El Centro 200% ........................................................................................................ 191
Figure 7.18. Base Shear Time History (Two-Story In-Plane Shear Wall) – El Centro 200% ........................................................................................................ 191
Figure 7.19. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 203 mm, Average Strength, Discontinuous Veneer for Two-Story Model) .............. 192
Figure 7.20. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 203 mm, Upper Bound Strength, Discontinuous Veneer for Two-Story Model) .............. 192
Figure A.7. Roof Joists Nailed to Rim and Connected by Two Clip Angles .................... 224
Figure A.8. 2 x 4 Blockings and Metal Braces ............................................................ 225
Figure A.9. Stapling of EPDM Flashing at the Frame Base .......................................... 225
Figure A.10. Installation of Frames OSB Sheathing .................................................... 226
Figure A.11. Installation of Roof OSB Sheathing ....................................................... 226
Figure A.12. OSB Sheathed Frames Ready for Veneer Construction ......................... 227
Figure A.13. Sheathed Frames, Blockings, Braces and Ceiling Board ....................... 227
Figure A.14. Interior of Structure after Installation of Gypsum Wall Board ............... 228
Figure A.15. Construction of Rigid Tie Wall with Joint Reinforcement ....................... 228
Figure A.16. Construction of Corrugated Tie Wall with Joint Reinforcement ............ 229
Figure A.17. Construction of Window Sill Blocks with Flashing ............................... 229
Figure A.18. Completed Structure ........................................................................... 230
LIST OF SYMBOLS

NEES  Network for Earthquake Engineering Simulation
NSF   National Science Foundation
DBE   Design Basis Earthquake
MCE   Maximum Considered Earthquake
SDC   Seismic Design Category
I-P   In-Plane
O-O-P Out-of-Plane
OSB   Oriented Strand Board
$E_m$ Modulus of Elasticity of Masonry
$G_m$ Shear Modulus of Masonry
$f_{m}^{'}$ Compressive Strength of Masonry Prism
R.V.I. Relative Velocity Index
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ABSTRACT OF THE DISSERTATION

Experimental and Analytical Investigation of the Seismic Performance
of Low-Rise Masonry Veneer Buildings

by

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Doctor of Philosophy in Structural Engineering

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Professor P. Benson Shing, Chair

This dissertation presents an experimental and analytical evaluation of the seismic performance of clay masonry veneer in wood-stud buildings. The experimental program involved the shaking-table testing of eleven wall assemblies as well as a full-scale one-story building. Walls were subjected to separate in- and out-of-plane seismic excitation. The specimens had different anchor types, anchor spacing, aspect ratio, presence and absence of joint reinforcement and window openings. All the specimens were designed
and constructed in accordance with the prescriptive requirements of the MSJC for masonry veneer for Seismic Design Categories D and E.

The shaking-table tests showed that veneer complying with the current MSJC provisions can sustain ground motions far in excess of representative Design Basis and Maximum Considered Earthquakes for Seismic Design Categories D and E. The out-of-plane response was governed by the anchor axial strength. The in-plane response was characterized by veneer sliding for the squat panels and a combination of rocking and sliding for the slender ones. Experiments showed the possible reduction of the extraction capacities of the nails due to high moisture content in wood studs. Test data showed that veneers oriented parallel to the direction of shaking could restrain the wood structure. However, slender rocking veneer panels would induce additional seismic force to the wood structure under severe excitation.

In the analytical phase of the research, numerical models were developed and calibrated by the experimental results. The models were capable of capturing the displacement and acceleration responses of the tested specimens as well as the failure mechanisms. The models were used to conduct parametric studies to examine the influence of different design variables on veneer response, including the effect of in-plane veneer on the seismic performance of wood-stud shear walls, and the behavior of two-story veneers under out-of-plane seismic excitation.
The dissertation finally proposes a set of recommendations to improve current design provisions. Results highlighted the need for a minimum anchor strength requirement to assure a satisfactory level of performance under the Design Basis and Maximum Considered Earthquakes. The mass of the squat veneers oriented parallel to the direction of motion should not be treated as merely added mass.
CHAPTER 1

INTRODUCTION

1.1 Background

Low-rise residential and commercial buildings account for a significant portion of the building inventory in the United States (Drysdale et al., 2007). Timber, light-gage steel framing and reinforced concrete masonry are widely used in such form of construction. An external cladding of clay masonry is usually applied to the façade giving the structure an attractive architectural appearance. The two systems, the veneer and the load-bearing backing, are connected together by metal anchors, which are also referred to as veneer ties. The anchors span over an air gap (typically 25 or 50 mm (1 or 2 in.)) that acts as a drainage cavity. It allows the passage of moisture out of the structure through weep holes that are normally located at the bottom course of the brick wall. Waterproof flashing at the base of the veneer and at the openings collects moisture and directs it to the exterior of the wall system (Reneckis et al., 2004). Masonry veneer wall systems provide numerous advantages for the building designers such as:

- High durability and long lasting wall system.
- Minimal long-term maintenance cost.
- Great design flexibility allowing the use of different colors, bond patterns, brick sizes, and surface textures.
- Cavity wall system is inherently weatherproof which is highly suitable for moist environments.
- Excellent insulation and heat capacity.
- Fire proof.
- Acts as sound barrier especially in noisy environments.

As a non-structural component, veneer is designed to support only its own weight and to transfer out-of-plane loads such as wind or seismic forces to the structural backing system through veneer anchors. The anchor forces can be significant due to the dynamic interaction between the veneer and the backing. A veneer wall system can be subjected to both in-plane and out-of-plane shaking. Veneer ties can be subjected to high demands of axial and shear forces depending on the direction of the earthquake excitation. The design of a veneer system is based on prescriptive code provisions considering the tributary area per veneer anchor. However, little information is available on the dynamic performance of such system, including the capacities of various anchor types and the force demand on the anchors. Most of veneer failures are almost exclusively related to anchor failures due to various reasons (Reneckis et al., 2004, McGinley and Hamoush, 2008, Choi and LaFave, 2004). Low fastener withdrawal strength, insufficient embedment, corrosion of tie material and poor workmanship are among the common observed failure causes. A great deal of research effort have been directed towards studying veneer systems with steel-stud backing however little information is available on systems with wood-stud frames or concrete masonry backing. Furthermore, most of the research conducted
considered veneers built with older construction practices that generally varied from current code prescriptive provisions (Reneckis et al., 2004).

1.2 Overview of Research Program

Under the auspices of the George E. Brown, Jr. Network for Earthquake Engineering Simulation Program (NEES) of the US National Science Foundation (NSF), a collaborative research project has been carried out to study the seismic performance of masonry veneer walls and veneer anchors in wood-stud and reinforced masonry buildings. The research project is a collaborative effort of the University of Texas at Austin, The University of California at San Diego, Washington State University, University of Louisville and North Carolina A&T University.

In this project, wall assemblies that were representative of typical wall configurations found in one-story buildings were tested quasi-statically and on a shaking table under separate in-plane and out-of-plane loads. In addition, two single-story building specimens with plan dimensions of 6.33 m x 6.33 m (20.7 ft x 20.7 ft) were tested on a shaking table. One building specimen had wood-stud walls and the other had reinforced concrete masonry walls as the structural system. The shaking-table tests were conducted on the NEES Large High-Performance Outdoor Shaking Table (LHPOST) at the University of California at San Diego. Results of the tests have been used to develop analytical models and to evaluate current code requirements for these systems.
1.3 Scope and Objectives

The work presented in this dissertation, which is a part of the research conducted in the aforementioned project, is one-of-a-kind systematic experimental and analytical evaluation of the behavior of clay masonry veneer systems. This dissertation focuses on the part of the research project related to the behavior of clay masonry veneer backed by wood-stud walls. The dissertation includes a comprehensive experimental program that evaluated the behavior of typical wall assemblies representative of portions of low-rise buildings. The walls were tested dynamically on a shaking table using scaled historic ground motion records representative of the Design Basis (DBE) and the Maximum Considered Earthquakes (MCE). The experimental study also involved the shaking-table testing of a full-scale one-story masonry veneer house with timber backing. The two phases of the experimental program have provided the necessary data for the development and calibration of analytical models for the evaluation of the dynamic behavior of masonry veneer wall systems. The combined experimental and analytical studies presented in the following chapters were conducted to study and address the following objectives:

- Understanding the seismic performance of masonry veneer on wood-stud backing.
- Assessment and appraisal of current prescriptive code provisions for masonry veneer.
- Development of analytical tools for the analysis of masonry veneer.
- Improvement of current design guidelines for masonry veneer.
1.4 Dissertation Outline

The dissertation is composed of eight chapters covering different aspects of the research program. The following presents a brief description and overview of the individual chapters.

Chapter 2 provides a detailed overview of the design and construction of low-rise timber and masonry veneer houses. It discusses the different building blocks of the integrated system as well as the veneer anchors used to bridge the walls together. A review of the prescriptive provisions in various building design codes and standards is presented along side a review of the seismic performance of masonry veneer in previous earthquakes, pointing out typical veneer failures as observed in damaged structures. The chapter discusses in detail the previous research efforts in the seismic performance of timber and masonry veneer structures. The review encompasses both the experimental and analytical research conducted on wood buildings with clay masonry veneer. A discussion of the various analytical modeling techniques is presented.

Chapter 3 presents a description of the first phase of the experimental program to investigate the seismic performance of full-scale wall assemblies representative of typical portions of low-rise masonry veneer/timber buildings. The shaking-table tests encompassed a wide range of design variables, including the type of veneer anchors, anchor spacing, wall aspect ratio, presence or absence of mortar joint reinforcement and presence or absence of window openings. The walls were tested in either their in-plane or out-of-plane direction. The design variables are representative of designs corresponding
to Seismic Design Categories (SDC) D and E of the Masonry Standards Joint Committee (MSJC) for masonry veneer. The chapter presents the instrumentation schemes used for data collection, the test setups, the testing protocols and a summary of the properties of the used materials.

Chapter 4 presents the results of the first phase of the experimental program dealing with the out-of-plane response of the wall assemblies. It starts with a detailed account of the experimental observations and the damage evolution throughout the shaking sequence. The chapter then presents a numerical study conducted to model the behavior of the tested walls. A two-dimensional finite element model using beam and truss elements is presented. The model is capable of capturing the displacement and acceleration response of the tested walls as well as the failure mechanisms. Furthermore, the chapter presents an extended parametric study on some of the influential parameters that affect the global response of a wall system under out-of-plane seismic excitation.

Chapter 5 presents in a similar fashion the results of the wall assembly tests pertaining to the in-plane excitation. Experimental observations are presented, followed by a detailed analytical study on the wall response. A more general and comprehensive model is presented that has the capability of modeling walls under in- and out-of-plane loads. The model outlined in this chapter presents an innovative modeling technique to capture the nonlinearities of the wood-stud frame that was calibrated with monotonic and cyclic loading tests of wood-stud frames with different aspect ratios. The model uses a
combination of nonlinear beam and truss elements to model the behavior of masonry. The model was shown to provide very close match with the experimental results.

Chapter 6 presents the second phase of the experimental program conducted on a full-scale one-story masonry veneer/timber house. The tested structure was configured in a way to allow the study of various design parameters similar to those studied in the first phase of the experimental program. The tested structure allowed for a more realistic seismic mass representation and an ability of capturing any three dimensional dynamic interaction of the intersecting walls. One of the building façades had a discontinuous corner joint to simulate the veneer found on the exterior of buildings with common walls. The chapter presents the instrumentation scheme, the test setup and the testing protocol used for the building test. It also provides a detailed description of the experimental observations and damage patterns at various levels of ground shaking. The chapter further presents an analysis of the experimental data in terms of acceleration and displacement responses and finally points out the observed defects in the current code provisions based on the combined knowledge gained up to this point from both phases of the experimental program.

Chapter 7 presents an extension of the numerical study conducted in Chapters 4 and 5. The developed general model was used to conduct a parametric study on the in-plane behavior of masonry veneer panels with different aspect ratios to examine the influence of the veneer on the in-plane seismic resistance of a wood-shear wall. Another study was conducted on the out-of-plane seismic response of two-story veneer structures
in order to assess the adequacy of the current design requirements. Furthermore, the model was used to conduct a simplified three-dimensional analysis on the prototype building specimen. Finally, the chapter presents an appraisal of the current MSJC code provisions and provides some recommendations to attain more efficient design and better seismic performance based on the results of the conducted research.

Chapter 8 presents a summary of the work conducted in the course of this research and the conclusions and contributions provided by the dissertation. Furthermore, the chapter presents some recommendations for future work to further extend the current work.
CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter provides a summary of the common practice of design and construction of low-rise timber houses with external claddings of masonry veneer. The first part of the chapter describes the components of the system, with emphasis on the different kinds of veneer anchors and presents a discussion of the mechanism of the gravity and lateral load transfer through the composite wall system. The chapter also presents a review of the performance of masonry veneer buildings in previous earthquakes, pointing out the different common failure mechanisms. A review of the prescriptive design provisions in various building codes and standards is presented thereafter. Finally, the chapter provides a summary of the previous research effort conducted on masonry veneer. The review encompasses both the experimental and analytical aspects of the research.

2.2 Low-Rise Timber Houses

Low-rise framed structures are a common kind of construction in many parts of the United States. Wood-stud frames are widely used for their competitive price and short erecting time. Figure 2.1 presents a typical wood-stud framed building showing the
different components of the assembled structure. Dimensional lumber is used for the intersecting frames and the roof. The wood-stud skeleton is externally and internally sheathed with oriented strand boards (OSB) and gypsum wallboards, respectively. The IRC (ICC, 2006) provides design and construction requirements for all structural components of wood buildings. On the other hand, the MSJC (MSJC, 2008) simply recommends that such designs comply with the ACI 318 Building Code Requirements for Structural Concrete (ACI 318-08, 2008) for the concrete foundation, and with the National Design Specification (NDS) for Wood Construction (NDS, 2001) for the wood framed structures; the MSJC code (MSJC, 2008) would be used only for the case of designing a masonry foundation.

The elements of the timber building are designed to support all kinds of gravity loads and are proportioned to provide sufficient resistance against lateral loads. The assembled frames resist lateral loads, such as wind and earthquake induced forces, through in-plane racking of the sheathing and transfer of the shear forces to the framing elements through the nailed connections. These fastener deformations are capable of dissipating a certain amount of energy. The behavior of the nailed sheathing-to-wall connection is the one that governs the response of the whole shear wall assembly. The load-deformation response of timber shear walls under reversed cyclic loading is characterized by pinched hysteresis with both stiffness degradation and strength deterioration (Dolan and Madsen, 1992, Shenton et al., 1998, Folz and Filiatrault, 2001).
Through the years, the testing of wood-stud structures has gone a long way. Atherton, 1983, tested wood-framed diaphragms with particleboard sheathing. The experiments investigated the effect of the sheathing thickness, nail size and spacing, blocking, and sheathing pattern on the strength of the diaphragms. He concluded that the nail spacing had the largest effect on the wall strength. Dolan and Madsen, 1992, tested full-scale shear walls investigating the effects of sheathing type, i.e., plywood versus wafer board, on the wall performance. They found that the type of sheathing did not have any significant role in the working stress range of the shear walls as well as the ultimate capacity. They found that the wafer board reduced the deflections at high loads more than the plywood sheathing. The conclusion that the type of sheathing has no significant effect on the wall shear strength was also confirmed by a study conducted by Serrette et al., 1997, that evaluated the static racking behavior of metal framed shear walls sheathed with OSB and plywood.

Patton-Mallory and Wolfe, 1985 tested 11 full size and 200 small walls of light-frame wood walls sheathed with plywood and gypsum. They deduced a linear proportionality between the racking strength and the wall length. They recommended the omission of the length of the wall containing openings for windows or doors from racking resistance calculations. Falk and Itani, 1987, also arrived at the same conclusion through free-vibration tests, to determine the natural frequencies and damping ratios, and forced-vibration tests, to develop displacement response spectra for the walls. They found that the reduction in stiffness of the walls is also proportional to the length of the wall occupied by the opening. Gypsum wallboard was found to contribute to shear wall
performance and its resistance is additive to the resistance provided by the plywood sheathing which is in good agreement with the later findings by Filiatrault et al., 2002, from the shaking-table tests they conducted on a two-story timber house.

Schmid et al., 1994, tested plywood sheathed shear walls and found that the dead load applied to walls helps to increase the capacity at lower displacement levels. They also found that uplift at the wall ends was the key factor to lateral wall displacement and suggested the use of proper anchoring to the exterior studs. Karacabeyli and Ceccotti, 1996, examined the effect of fastener type on wall performance and the effect of gypsum wallboard on shear wall capacity. Once again, they found that the gypsum wallboard increased the strength and stiffness at the price of lower ductility when compared with walls with no gypsum wallboard. Kamiya et al., 1996, investigated the effect of fastening the studs with bolts instead of nails using shaking-table tests. They observed little difference in the response of the wall whose studs were fastened with nails versus the response of the wall whose studs were fastened with bolts. However, when the acceleration was increased, the maximum displacement response of the wall with nailed studs increased significantly more than that of the bolted wall.

2.3 Masonry Veneer

External cladding of masonry veneer is sometimes applied to the exterior of timber houses. Figure 2.2 shows a typical construction detail of the system. The masonry wythe is spaced from the structural load-bearing backing by an air gap that is usually 25-
mm to 50-mm (1-in. to 2-in.) wide. The air gap acts as a drainage cavity to direct water and moisture out of the wall. The masonry wall is constructed over a flashing material at the base to prevent water permeation. The collected moisture is driven out of the wall system through weep holes, mortarless head joints, spaced at regular intervals. The masonry wall is connected to the backing through veneer anchors. These anchors provide lateral support for the wall as well as help transferring any face loads such as wind or earthquake induced forces to the structural backing. Brick veneer provides added value for the timber structure due to the external look of a brick building but at a lower construction cost. Masonry veneers are usually constructed from materials like concrete blocks, manufactured clay, artificial stone or natural stone product (Drysdale et al., 2007).

Veneer anchors may be considered the most important component of the wall assembly. These anchors, also referred to as veneer ties, bridge the two walls, the veneer and the structural backing, and provide the required support for the unreinforced, free standing masonry wythe. The behavior of wall assembly is highly affected and dependant on the mechanical properties of these ties. Figure 2.3 shows a wide variety of commonly used ties in masonry veneer and cavity wall construction. As can be seen from the figure, these ties differ in their axial and shear stiffness and strength depending on their geometrical and cross-sectional properties. Different fastening methods to the backing can be also used, including a broad class of anchors referred to as adjustable ties. This class of ties can easily accommodate any misalignment of mortar joints so that the mason will not be forced to bend the tie itself. In some cases, joint reinforcement is attached to
the tie in predefined attachment points and then embedded in the mortar joints of the veneer wythe. It should be noted that the requirements in some codes for the inclusion of wire joint reinforcement is debatable. Bennett and Bryja, 2003, have questioned the effectiveness of the joint reinforcement based on satisfactory performance of unreinforced brick veneer construction during some earthquakes and severe wind storms, as well as experimental test results.

2.4 Design of Masonry Veneer

The design of brick veneer systems is such that the contribution of the masonry wall to the lateral load resistance of the structural backing is neglected. In other words, it is considered as added seismic mass. The veneer wall is only designed to support its own weight and to transfer any face load such as wind or earthquake induced forces to the structural backing. Therefore, the wood-stud backing should be designed to resist all of the exterior lateral loading, as well as any gravity loads from the structural flooring or roofing (Drysdale et al., 2007). For adequate performance of masonry veneer systems, possible differential movement between the masonry and backup walls, as well as water penetration of the exterior masonry wall should be taken into account (Drysdale et al., 2007). Design of masonry veneer system is mainly based on prescriptive requirements for strength and serviceability design. These requirements are given by the Masonry Standards Joint Committee (MSJC) Building Code (MSJC, 2008), the International Residential Code (IRC) for One- and Two-Family Dwellings (ICC, 2006), and the Brick Industry Association (BIA) Technical Note 28 (BIA, 2002).
The concept of exclusion of load sharing between the wood shear wall and the masonry veneer is reflected in many design codes and standards. For example, the MSJC Code (MSJC, 2008) does not recognize any interaction between the two components pertaining to the in-plane load resistance, and assumes that the wood shear wall will carry all the racking loads. On the other hand, the BIA Tech Notes (BIA, 2002) acknowledge that brick veneers do carry a considerable share of the lateral load depending on the stiffness and strength properties of the veneer ties.

During earthquakes, the mass of the masonry induces higher inertia forces that will be transferred to the structural backing when the two are tied together with an anchoring system. The higher initial stiffness of the masonry veneer when compared to that of the wood-stud backing can attract a portion of the racking load depending on the stiffness and strength properties of the ties. In other words, brick veneer may be thought of as having a similar contribution to the wood frame as the gypsum wallboard had. Therefore, the exclusion of the brick veneer from the stiffness and capacity calculations of the wood shear wall would result in an underestimation of the lateral stiffness and an overestimation in the displacements, which results in highly conservative designs.

Various design and construction requirements of masonry veneer are provided by the MSJC (MSJC, 2008) code and BIA Technical Note 28 (BIA, 2002). The out-of-plane stability of a brick masonry veneer wall is controlled by the wall material strengths, height and thickness, and also by the layout and properties of the metallic connections.
that anchor it to the structural backing. The brick units should be at least 2-5/8 in. (66.7 mm) thick.

For seismic design category (SDC) C or below and for typical wind exposure conditions, with wind speeds up to 177 km/h (110 mph), brick masonry with Type N mortar is usually used, and is considered adequate for carrying the self-weight, transferring loads to the tie connections, and limiting flexural cracking of the veneer wythe. For seismic design categories D and above (and/or in areas of high wind) or when a higher masonry flexural strength is needed Type S or M mortars are recommended. Additionally, the MSJC (MSJC, 2008) limits the out-of-plane service load deflections of the backing wall to ensure the stability of the brick veneer and to control the cracking of the masonry.

Proper detailing is essential for wall system protection against water damage such as corrosion of the veneer anchors and to account for material dimensional changes. Wind driven rainwater and moisture can penetrate the veneer into the air gap, so flashing and weep holes are normally constructed along the edges of the wall and around openings to guide water out of the wall cavity. Another problem is the mortar droppings that bridge the air gap, especially at the veneer anchors, and provide a passage for water movement across the gap. This weakens the effectiveness of the drainage mechanism and accelerates the anchor corrosion. Finally, expansion joints should be installed in masonry veneer, to account for differential movement of the masonry walls, as well as for any dimensional changes in the masonry itself (MSJC, 2008, ICC, 2006 and BIA, 2002).
The MSJC (MSJC, 2008), the IRC (ICC, 2006), and BIA Technical Notes (BIA, 2002) provide prescriptive installation requirements for corrugated sheet metal ties used to connect masonry veneer to a wood-stud backing. These requirements are presented in Table 2.1. These requirements were specified such that the veneer anchors would satisfy a set of performance requirements such as: a) sufficient strength and stiffness for adequate load transfer to the backing wall, b) adequate transverse flexibility to accommodate differential vertical movements between the masonry veneer and the structural backing, and c) resistance to corrosion and moisture transfer across the air cavity.

The design codes and standards recognize different values for each seismic design category and wind speed. For seismic design category C or below and for typical wind exposure conditions, the maximum veneer wall area to be supported by a single anchor is 0.25 m² (2.67 ft²), generally resulting in a tie spacing of 406 mm (16 in.) horizontally (governed by stud spacing) and 610 mm (24 in.) vertically. MSJC requires that ties be installed within 305 mm (12 in.) of the wall edges and openings, with maximum 914 mm (36 in.) tie spacing around the perimeter of such an opening, while the BIA Technical Note 44B (BIA, 2002) recommends that ties be installed within 203 mm (8 in.) of wall edges. The height of a brick veneer wall is limited to be 30 ft (9.14 m) above its support, with an additional 8 ft (2.44 m) permitted at gable ends of a home structure.

For seismic design category D (or higher) as per MSJC (2008) and the IRC (ICC, 2006), as well as where wind pressures exceed 1.44 kN/m² (30 psf) per the IRC, the
tributary wall area per tie must be reduced to 0.19 m$^2$ (2 ft$^2$). Furthermore, the MSJC requires lowering the wall area per tie to 0.17 m$^2$ (1.87 ft$^2$) where wind speeds are above 177 km/h (110 mph). In addition, the MSJC code requires the use of horizontal joint reinforcement (with ties mechanically attached to the reinforcement) in all brick masonry for buildings with seismic design categories E and higher, as well as supporting the brick veneer independently at each level of the building.

2.5 Performance in Earthquakes and Typical Veneer Failures

Since masonry veneer is treated as a nonstructural component of the building system, it does not affect the structural integrity of the load bearing structural system. On the other hand, the potential of causing causalities and deaths resulting from falling bricks is not a function of its structural value prior to failure (Bruneau, 1994). During the 1994 Northridge earthquake in California, many timber residential constructions with masonry veneer suffered considerable damage. Generally, poor veneer anchorage and flexibility mismatch between the veneer and the structural backing resulted in severe damage to the nonstructural masonry. Many slender anchored veneers improperly connected to their backup structure have failed in an out-of-plane direction (Figure 2.4).

Failure of masonry veneer is in most cases triggered and caused by the failure of the veneer anchors. Common failure modes include fastener pull-out from the backing (Figure 2.6) or from the mortar joints (Figure 2.7), failure of masons to embed ties into the mortar (Figure 2.8), poor bonding between ties and mortar or poor mortar quality
Masonry veneer might also severely crack due to differential foundation settlements as shown in Figure 2.11. Ties are often installed on the backing wall before the brick laying begins. Due to construction tolerances, ties are often improperly placed above or below the mortar joints. Therefore, the masons will bend the anchors up or down in order to embed them into the mortar joints (Figure 2.12). Misalignment reduces the embedment depth as well as the effectiveness of the ties.

### 2.6 Previous Experimental Research Efforts

A number of researchers have investigated the behavior of masonry veneer for residential and commercial constructions. Though most of the research conducted was directed towards veneers constructed over steel-stud backing or veneers built with older construction practices, some of the latest research was directed towards veneers over wood-stud backings. This section presents an overview of the latest experimental research effort conducted on similar constructions.

Johnson and McGinley, 2003, tested small assemblies in double shear applied uniformly to the specimen height. They used standard 22-gage corrugated sheet metal ties. The tests showed that the ties are capable of transferring shear forces from the veneer to the backing wall. The shear transfer was also significantly enhanced with the addition of truss type horizontal joint reinforcement into the mortar joints of the masonry veneer.
Choi and LaFave, 2004, investigated the performance of corrugated sheet metal ties using sub-assembly tests under axial (out-of-plane) and lateral (in-plane) monotonic and cyclic loading. The axial loading tests revealed that the dominant failure modes were nail pullout from the wood in nailed anchors under tension, tie pullout from the mortar joint in screwed connections under tension and buckling of the tie in both nailed and screwed connections under compressive loading. The load displacement relation of the subassembly under cyclic axial loading was characterized by stable nonsymmetrical pinched hysteresis that shows both stiffness degradation and strength deterioration. Under lateral loading the dominant failure mode was nail pullout from the wood as in the case of monotonic loading and tie fracture under cyclic lateral loading. They proposed idealized load-displacement relations for monotonic lateral loading and envelope curves for cyclic lateral loading.

Reneckis and LaFave, 2009, tested connector subassemblies monotonically and cyclically in continuation of the work by Choi and LaFave, 2004, to evaluate the strength and stiffness properties of the corrugated sheet metal ties. Investigated parameters included tie thickness (22-ga. (0.79 mm), 28-ga. (0.33 mm), and 16-ga. (1.57 mm)), tie attachment method to the wood stud namely, Galvanized 8d nails (64 mm long), Galvanized roofing nails of two different lengths (64 mm and 38 mm) and #8 bugle head galvanized deck screws. Most of the subassemblies were constructed with the corrugated sheet metal tie bent 90-degrees right over the head of the nail or screw fastener, while some specimens were tested with a greater eccentricity at the bend of 12.7 mm (1/2 in.) which is the maximum eccentricity permitted by the MSJC (MSJC, 2008).
The monotonic loading tests showed that for subassemblies with 22-gage corrugated sheet metal ties fastened with 8d nails and meeting the other minimum prescriptive requirements, the average tie connection tensile strength was 681 N (153 lbs), while assemblies with thinner (28 ga.) ties, the average tie connection tensile strength was 703 N (158 lbs), indicating that using a thinner tie does not necessarily result in a lower strength of a veneer-wood connection. However, when 8d nails were replaced with similar length roofing nails in subassemblies with 22 ga. ties, the average tie connection tensile strength dropped to 441 N (99 lbs) and 338 N (76 lbs) when 38 mm roofing nails were used with 28 ga. ties. This indicated that using short roofing nails (instead of 8d nails) resulted in a 50% or more reduction in tie connection tensile strength. The predominant failure mode observed was nail pullout from the wood-studs, confirming that the tie thickness would definitely have no effect on the average tie connection tensile strength.

During the cyclic testing multiple failure modes were observed such as: nail pullout, tie fracture, yield around the tie hole and pull-through of the nail and tie pullout from the mortar joint. Specimens with 64 mm (2.50 in.) long wood screws with 16 ga. and 22 ga. ties had an average tensile strength of 1820 N (409 lbs), which is significantly higher than specimens with nails. The predominant failure mode for specimens with screws was tie pullout from the mortar joint, with a few occurrences of either yield around the tie hole leading to pull-through of the screw head or tie fracture.
Zisi, 2009, tested narrow and wide 22-gage corrugated tie designs, complying with the minimum thickness permitted by the MSJC Code (MSJC, 2008). They conducted monotonic and cyclic loading tests on connector subassemblies. The tests investigated the effect of hole size, two fastener types 8d (63.5 mm long) common nails and #6 (41 mm long) screws, fastener quantity (specimens assembled with two nails instead of one), bent eccentricities (the minimum eccentricity, 4 mm for the narrow ties and 5 mm for the wide ties, maximum permitted bent eccentricity of 12.7 mm as well as excessive eccentricity of 25.4 mm). Parameters also included the effect of tie location in bed joint (subassemblies with ties placed at the center of the mortar joint versus ties placed flush on the underlying brick) and tie positions with respect to the head joint.

It was concluded that the nonlinear hysteretic behavior initiated at small displacements. Similar to Choi and LaFave, 2004, the loops were characterized by pinching and asymmetry as well as strength reduction and loss in absorbed energy capacity under repeated cycling at a given displacement level. Subassemblies constructed with wide ties had larger initial stiffness and higher energy absorption characteristics. Specimens fastened with screws had almost the same initial stiffness and cyclic envelope as specimens with nails. On the other hand the absorbed energy was higher at larger displacement levels. They observed the same failure modes for nailed and screwed connections as previous researchers. Subassemblies with two fasteners had higher initial stiffness and absorbed energy. Partial tie tear near the mortar joint along with moderate nail pullout was the governing failure mode. Smaller bent eccentricities resulted in higher initial stiffness, more energy dissipation and higher cyclic envelopes. The effect of tie
location in bed joints was quite significant with respect to initial stiffness but was also
masked by the effect of the mortar droppings. The effect of tie location with respect to
head joint was concluded to be insignificant as well as the effect of the tie hole size.

On the side of wall assembly testing, Reneckis and LaFave, 2004, tested the
performance of solid brick veneer on wood-stud backing under static and dynamic out-of-
plane loading. The assembly consisted of a full-scale brick veneer panel attached to a
wood-stud frame with 28-gage corrugated sheet metal ties (not permitted by MSJC)
utilizing two different installation methods, namely different nail eccentricity at the tie
bent, as well as with mechanical anchors installed as retrofit method. The static tests
showed that brick veneer increased the out-of-plane stiffness of the wall system,
compared to the stiffness of the bare wood-stud frame. Under dynamic shaking, the
veneer rotated as a rigid body about its base transferring the inertia forces through the ties
into the backing. As a result, the ties in the upper rows controlled the system performance
because they were subjected to the highest elongations. Mortar droppings in the air gap
were found to increase the initial stiffness of some anchors and also reduce the
compressive demands on the ties. Post-installed mechanical anchors enhanced the
performance of the wall system due to their higher capacity of inertia load transfer which
secured the veneer closely to the backing.

In continuation of their work, Reneckis and LaFave, 2009, dynamically tested a
full-scale, one-and-a-half story masonry veneer on a wood-stud backing. The wall was
designed and constructed to represent a typical gable-end wall, with a window opening.
The brick veneer was attached to the wood frame wall using 22-gage corrugated sheet metal ties fastened by 8d nails. The ties were bent at a small bend eccentricity of approximately 6 mm (1/4 in). They concluded that residential brick veneer construction built in general conformance with current code requirements performed satisfactorily under ground motions with peak ground accelerations (PGA) of 0.39g without damage, and up to a PGA of 0.75g without enough anchor and masonry damage to cause out-of-plane collapse. They deduced a close relation between the tensile properties of the tie connections and the overall wall performance. Tie connections anchored near stiffer regions of the wood-stud backing experienced the highest loads and were first to show signs of damage, after which the veneer started to crack. Horizontal cracks and hinges formed in the brick veneer along the base of the gable. Presence of a window opening resulted in greater backing flexibility, causing the wood framing to follow the masonry wall along the edges of the opening, reducing the load demands on the ties at those locations.

2.7 Previous Analytical Research Efforts

Analytical modeling of wood shear walls has evolved from simple racking equations for displacement and ultimate capacity to complex nonlinear dynamic analyses. Currently there are two modeling approaches. The first is a simple reduced degrees of freedom model which takes advantage of the fact that the behavior of the sheathing-to-stud connection is what governs the global nonlinear response of the shear wall. In this approach, the sheathing is assumed to be elastic in transferring in-plane shear forces,
while the framing members are usually assumed to be rigid in bending. The nonlinearities are only included in the sheathing-to-framing connector elements. Examples of these models are those developed by Easley et al., 1982, McCutcheon, 1985 and Gupta and Kuo, 1987, which were mainly for static analysis. The latest models by Filiatrault, 1990, Dinehart and Shenton, 2000, Folz and Filiatrault, 2001, were suited for all kinds of analyses. All of these models provided satisfactory agreement with the load-displacement response obtained from experiments, but were not able to capture the detailed interaction and load sharing between the components of the shear wall under lateral loading.

The second, rather complicated approach is the finite element models like those developed by Itani and Cheung, 1984, Gutkowski and Castillo, 1988, Falk and Itani, 1989, Dolan and Foschi, 1991, White and Dolan, 1995, Tarabia and Itani, 1997. In these models, the framing members are approximated with linear elastic beam elements usually pinned at their ends. The sheathing is typically represented by linear elastic, orthotropic plane stress elements, while some researchers have also tried the plate bending elements. The sheathing-to-framing connectors are modeled with a pair of independent orthogonal springs that follow nonlinear load-deformation relations. These relations, also used in the first approach, evolved from relatively simple exponential, polynomial or logarithmic functions suitable for quasi-static analysis to complete hysteretic relations that enable cyclic and dynamic analysis. Gap elements were used at the interface between sheathing panels to account for inter-panel bearing. Flexible supports with some degree of fixity were used as an alternative to the pinned connections. These complicated models are able to capture more fully the inter-component response within the wall. Their major
drawback is that they are computationally demanding. One interesting observation is that the global load-displacement responses from both modeling approaches showed the same level of correlation with the experimental results.

One of the latest efforts to model masonry veneer in wood-stud walls is the work by Zisi, 2009. The modeling approach is suitable for modeling veneer subjected to in-plane loading. Linear elastic beam elements were used to model the framing members of the shear wall. Both the sheathings and the brick veneer were modeled with four node quadrilateral elements. Homogenous isotropic linear elastic material behavior and plane stress idealization were assumed in the formulation. Zero-length elements in two-dimensions were used to simulate sheathing-to-stud fasteners and veneer anchors. Their stiffness in each of the two orthogonal directions was determined from material models with nonlinear inelastic behavior approximated from experimental results. These elements performed essentially like a pair of orthogonal independent nonlinear inelastic springs, a modeling approach that had been used to approximate individual fasteners (Itani and Cheung, 1984, Gutkowski and Castillo, 1988, Dolan and Foschi, 1991, White and Dolan, 1995). The boundary conditions of the veneer at the base were modeled with zero-length elements that connected two overlapping nodes. An unlimited, infinitely stiff in compression, no-tension law was assigned to the vertical support to simulate the panel rocking. In order to simulate the veneer sliding, a zero-length element with very stiff elastic perfectly plastic behavior in compression and no tension was used at the wall corner, with a yielding force determined by multiplying the weight of the wall by a coefficient of friction of 0.36 taken as the lowest reported in literature.
Reneckis and LaFave, 2004, developed a three-dimensional finite element model to model the out-of-plane behavior of masonry veneer. In this model, the wood frame, brick veneer and support conditions were modeled as linear elastic while the corrugated sheet metal ties were modeled as nonlinear inelastic based on the models developed from their experimental results. The wood-stud frame was modeled as linear elastic composite frame in which nail slip was neglected. The wood-stud and exterior OSB sheathing composite wall panel was represented by joined three-dimensional beam and shell elements. In the experiments conducted in their research, brick veneer walls generally exhibited more rigid body rotation (rocking about their base) than bending when subjected to out-of-plane static and moderate dynamic loading. Therefore, the brick veneer was assumed to be linearly elastic and was modeled using shell elements assigned the same section dimensions as in the test structure. The brick veneer and wood frame were connected together with truss elements representing the veneer anchors. The experimental load-displacement behaviors of the tie connections, evaluated both during tie subassembly tests and brick veneer wall panel testing, were modeled with a truss element using a hysteretic law.
Table 2.1. Prescriptive Requirements for Corrugated Ties for SDC C and Below
Adapted from Reneckis and LaFave (2009)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Tie thickness (gage) [min.]</td>
<td>22</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>Tie width (in.) [min.]</td>
<td>0.875</td>
<td>0.875</td>
<td>0.875</td>
</tr>
<tr>
<td>Typical wall area per tie (ft²) [max.]</td>
<td>2.67</td>
<td>2.67</td>
<td>2.67</td>
</tr>
<tr>
<td>Horizontal spacing (in.) [max.]</td>
<td>32</td>
<td>24</td>
<td>24</td>
</tr>
<tr>
<td>Vertical spacing (in.) [max.]</td>
<td>25</td>
<td>24</td>
<td>24</td>
</tr>
<tr>
<td>Wall area per tie in seismic areas (ft²)</td>
<td>2.00</td>
<td>2.00</td>
<td>n/a</td>
</tr>
<tr>
<td>Wall area per tie in severe wind zones</td>
<td>1.87⁹</td>
<td>2.00⁹</td>
<td>n/a</td>
</tr>
<tr>
<td>Fastener to wood backup [min.]</td>
<td>8d nail¹.getComponent(3) n/a</td>
<td>n/a</td>
<td></td>
</tr>
<tr>
<td>Bend distance from fastener (in.) [max.]</td>
<td>0.5</td>
<td>n/a</td>
<td>0.5</td>
</tr>
<tr>
<td>Embedment length into mortar (in.) [min.]</td>
<td>1.5</td>
<td>n/a</td>
<td>1.5⁶</td>
</tr>
<tr>
<td>Mortar cover on outside face (in.) [min.]</td>
<td>0.625</td>
<td>n/a</td>
<td>0.625</td>
</tr>
<tr>
<td>Air gap (in.) [min. and max.]</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

⁹ With seismic design category D and above. MSJC (2008) also requires installation of horizontal joint reinforcement for seismic design categories E and F.
⁶ For construction in areas where basic wind speed is between 110 mph and 130 mph (177 and 209 km/h); also, maximum horizontal spacing of ties is reduced to 18 in.
³ Wind regions with more than 30 psf (1.45 kPa) design pressure.
¹ “… or fastener having equivalent/greater pullout strength; should also be corrosion resistant.
² “… or half the thickness of the brick veneer.
³ 1 in. = 25.4 mm; 1 ft² = 0.0929 m²
Figure 2.1. Components of a Typical Timber Building

Figure 2.2. Components of a Typical Masonry Veneer Wall
Figure 2.3. Veneer Anchors

Figure 2.4. Out-of-Plane Failure  
Adapted from Reneckis and LaFave (2009)

Figure 2.5. In-Plane Failure  
Adapted from Reneckis and LaFave (2009)
Figure 2.6. Anchor Pullout from Backing

Figure 2.7. Anchor Pullout from Joints

Figure 2.8. Ties not Embedded in Mortar Joints
Adapted from FEMA (2006)

Figure 2.9. Poor Bond
Adapted from FEMA (2006)

Figure 2.10. Severe Corrosion
Adapted from FEMA (2006)

Figure 2.11. Distress due to Foundation Settlement
Figure 2.12. Tie Misalignment
Adapted from FEMA (2006)
CHAPTER 3

TESTING PROGRAM ON VENEER WALL SEGMENTS

3.1 Introduction

This chapter presents the experimental program conducted on wall assemblies representative of segments of a one-story masonry veneer wood-frame house. The chapter provides a detailed description of the tested specimens, design variables, used materials, construction process, development of testing protocols, test setup, instrumentation schemes and an experimental evaluation of the material properties used in specimen construction.

3.2 Experimental Program

A total of 11 wall specimens (masonry veneer and wood-stud frame assemblies) were tested in this phase of the experimental program. The specimens were tested on a shaking table in either their in-plane (I-P) or out-of-plane (O-O-P) direction. Table 3.1 presents a description of the wall specimens and Figures 3.1a and 3.1b show typical design details. The wall specimens had a range of design variables including different aspect ratios, presence or absence of a window opening, types of veneer anchors, presence or absence of mortar joint reinforcement and vertical tie spacing. The walls were designed for SDC D and E of the ASCE 7-05 (ASCE, 2005) using the
provisions of the Masonry Standards Joint Committee (MSJC, 2008), the International Residential Code for One and Two Family Dwellings (ICC, 2006), and the serviceability requirements of the Brick Industry Association (BIA, 2002) Technical Notes. The walls were constructed by professional masons in a manner conforming to common practice.

### 3.2.1 Wood-Stud Frame

The wood-stud backing frame was constructed with 38-mm x 89-mm nominal (2 x 4-in.) No.2 Douglas fir studs according to the IRC specifications (ICC, 2006). The vertical studs in each wall were spaced at 406 mm (16 in.) on center. A double top plate and a sole plate were nailed to the vertical studs using two 16d end nails. On the exterior (veneer side) of the wood frame, an 11-mm (7/16 in.) thick oriented strand board (OSB) was nailed to the edge studs using 6d nails spaced at 152 mm (6 in.) on center, and to the interior studs at 304 mm (12 in.) on center. A 12-mm (1/2 in.) gypsum wall board was attached to the interior of the wood-stud frame using #8 drywall screws spaced at 406 mm (16 in.) on center along the studs. The sole plate was connected to the reinforced concrete foundation using 12-mm (1/2 in.) threaded rods pre-embedded in the concrete. In addition, the exterior double vertical studs were connected to the foundation using Simpson HDU4-SDS2.5 hold-downs by Simpson Strong Ties. The hold-downs were connected to the vertical studs with screws and to the foundation using 19-mm (3/4 in.) threaded rods embedded with epoxy adhesive in the concrete slab according to the manufacturer’s specifications.
3.2.2 Clay Masonry Veneer

The clay masonry veneer wythe was separated from the wood-stud backing by a 25-mm (1 in.) specified air gap and constructed over a concrete slab with a 30-mil (0.76 mm) flashing made of Ethylene Propylene Diene Monomer (EPDM) in between. The flashing was stapled to the backing near the base. Weep holes were introduced at the bottom veneer course by removing the mortar from the head joints at intervals less than 0.90 m (3 ft). The masonry veneer wythe was constructed in running bond using nominal 101-mm x 68-mm x 203-mm (4 x 2-5/8 x 8-in.) standard modular clay masonry units conforming to ASTM Standard C 216 (ASTM, 2006). Type N masonry cement mortar was used conforming to the single-bag proportion specification of ASTM C 270 (ASTM, 2006) with a volume ratio of masonry cement to sand of 1:3. The bed joints in the walls were 10-mm (0.4 in.) thick. The weight of the veneer as measured from prism specimens was 1650 N per square meter (35 psf) of wall area.

3.2.3 Veneer Ties and Joint Reinforcement

Two types of metal anchors were used to attach the clay masonry veneer to the wood-stud backing. One was 22-gage (0.80-mm) corrugated sheet metal ties, as shown in Figure 3.2a, connected to the wood studs using bright 8d common nails. In some specimens, the ties were mechanically attached to 9-gage (4-mm) wire joint reinforcement embedded in bed joints through a built-in hook (see Table 3.1). The corrugated ties were installed with minimal eccentricity from the tie bent, which was way below the maximum eccentricity of 12 mm (1/2 in.) allowed by the code. The other type
of veneer anchors, termed a “rigid tie,” is shown in Figure 3.2b. These ties were 16-gage (1.60-mm) steel brackets bent at a 90-degree angle. Each rigid tie was connected to the wood-stud backing with a #8 screw at the 90-degree bent. Joint reinforcement (9-gage or 4-mm wire) was used with the rigid ties. As with the corrugated ties, the joint reinforcement was attached to the ties through a built-in hook. Table 3.1 summarizes the configuration of each tested wall. Wood Specimen 7X had the same configuration as Wood 7 except that the top row of ties was missing due to a construction error. The walls were cured for at least 28 days before testing.

3.3 Instrumentation Schemes

Displacement transducers (string potentiometers) and accelerometers were used to monitor the veneer and backing displacements with respect to the table and the absolute accelerations at the positions of the veneer anchors, respectively. The displacement transducers were mounted on wooden reference frames located close to the tested walls. The wood reference frames had very small mass and were stiff enough that their deformations were negligible as compared to those of the walls. Prior to the wall tests, the reference frames were tested to verify that their deformations were negligible.

Figure 3.3 shows the instrumentation scheme for the tested walls, where each dot represents the location of a displacement transducer and an accelerometer on both the veneer and the backing. The instruments were oriented parallel to the direction of the table motion, i.e., perpendicular to the O-O-P walls and parallel to the I-P walls. The
possible sliding of a brick wall along its base was monitored with a linear potentiometer. Three additional accelerometers were mounted on the top of the veneer in the I-P walls (two on the wall corners to monitor the wall rocking and one in the middle to monitor any possible out-of-plane motion). Prior to the anticipated collapse run of the shaking table, most of the instruments on the veneer side of the system were removed to avoid possible damage from falling veneer.

3.4 Test Setup

The wall specimens were tested on the Large High-Performance Outdoor Shaking Table (LHPOST) at the University of California at San Diego. The table has plan dimensions of 7.60 m x 12.2 m (25 ft x 40 ft) and is capable of carrying a maximum payload of 20 MN (4,500 kips). The two hydraulic actuators controlling the table motion have a displacement stroke of ± 0.75 m (29.5 in.) and are capable of driving the table to a maximum velocity of 1.80 m/sec (70 in. /sec).

The wall specimens were placed on the shaking table over a layer of gypsum plaster to help leveling and ensure full contact between the base slab and the table surface. The concrete base was tied to the table by post-tensioned steel rods. In most cases, two walls were tested at a time. One wall was oriented parallel and the other perpendicular to the direction of the table motion to impose in-plane and out-of-plane shaking, respectively. Figure 3.4 shows the test setup for the walls. Steel frames were used to support the top of the wood-stud backing wall.
The out-of-plane restraining frame consisted of a steel C-channel which was connected to the double top plate of the wood-stud wall with two and four 12-mm (1/2 in.) threaded rods spaced at 609 mm (24 in.) on center for the 1.21-m (4-ft) and 2.43-m (8-ft) wide walls, respectively. The channel was welded to two inclined trusses, one on each side of the wall, which were bolted to the table.

This frame might have a stiffer and more secure restraint to the top plates of the wood-stud walls than an actual roof diaphragm. However, the actual restraint condition of a roof diaphragm can vary significantly depending on the roof span, the orientation of the roof joists with respect to the out-of-plane walls, and the rim joist connection details. A numerical study using the analytical model presented in Chapter 4 has shown that a more flexible restraint at the top can reduce the tie forces by allowing the veneer and the backing to rotate together about the wall base. The stiff support would not allow the bowing of the top plate, which might lead to non-uniform tie forces near the top. Furthermore, the test setup would not simulate the two-way bending of a veneer wall, which can occur in a building where the veneer is continuous at the corners. However, the purpose of these wall tests was to examine the behavior of the veneer anchors and the interaction between the veneer and wood backing, and to provide data for the development and calibration of analytical models that can be used to examine additional parameters not considered in the tests. For this purpose, the simple boundary conditions used here would be desirable.
For the in-plane walls, The frame consisted of a steel C-channel which was connected to the double top plate of the wood-stud wall with two and four 12-mm (1/2 in.) threaded rods spaced at 609 mm (2 ft) on center for the 1.21-m (2-ft) and 2.43-m (4-ft) wide walls, respectively. The threaded rods were installed through longitudinal slots in the steel channel to allow the motion of the wood-stud frame in its in-plane direction. The frame is to restrain the wood shear wall from accidental out-of-plane motion. The channel was welded to four inclined angles, two on each side of the wall, which were bolted to the table.

### 3.5 Testing Protocols

Shaking-table tests were conducted using two ground motion records from the 1994 Northridge (California) Earthquake: the Sylmar (S) – 6-story County Hospital Parking Lot record (360-degree component) and the Tarzana (T) – Cedar Hill Nursery A record (90-degree component). The records were obtained from the Center for Engineering Strong Motion Data (www.strongmotioncenter.org). Figures 3.5 (a) and 3.5 (b) show the acceleration time histories for both records.

Each wall was first subjected to a sequence of Sylmar ground motion histories with the acceleration scaled to different levels and then to scaled Tarzana ground motions as shown in Table 3.2. The scaling used representative design basis response spectra that correspond to SDC D and E as a reference, as shown in Figure 3.6. The figure also shows an upper-bound spectrum for Seismic Design Category (SDC) E and a lower-bound
spectrum for SDC D determined from the seismic maps of California. The representative spectra are more or less the mean of the upper and lower-bound spectral curves and are representative of the seismic intensity in many areas in California. The parameters shown in Table 3.3 were used to derive the response spectra for the representative design basis earthquake. The spectral values for SDC D were picked based on a location in the East Coast, while those for SDC E were for a location in California using Figures 22-1 and 22-2 in ASCE 7-05 (ASCE, 2005). It was decided that the values of $S_s$ for SDC D and SDC E be the same, while those of $S_1$ would be different. The value of $S_s$ selected is on the high side for either region but not the maximum expected for the west coast to have a broader representation. Site Class D (Soil Type) was used for the determination of $F_a$ and $F_v$ from Tables 11.4-1 and 11.4-2 in ASCE 7-05 (ASCE, 2005).

It can be seen from the figure 3.6 that based on the initial fundamental periods of the out-of-plane specimens (0.10 sec – 0.15 sec), obtained from the white noise tests conducted at the beginning of the shaking sequence, 80% level of the original Sylmar record and 36% of the original Tarzana correspond to a design basis earthquake (DBE), while 120% of Sylmar and 54% of Tarzana records correspond to a maximum considered earthquake (MCE). The first level of Tarzana applied was 70% of the original level. It was chosen to have the spectral ordinate at the fundamental period close to the highest level of the Sylmar record (150%) applied. While the Sylmar record has strong acceleration pulses, the Tarzana record is much more demanding in the frequency range of interest and has a much longer duration of strong shaking as shown in Figure 3.5b. Both records have the spectral ordinates peak at a period of about 0.4 sec, as shown in
Figure 3.6, which could be demanding for a wall specimen experiencing significant progressive softening due to veneer cracking. White noise excitation was used in between the earthquake runs to assess the dynamic properties of the wall system and to track the progression of damage. The white noise had a root-mean-squared acceleration of 0.03g and swept a frequency range of 1 – 33 Hz.

3.6 Material Properties

Material tests were conducted on the used sand, mortar, bricks and on masonry prisms, constructed at the same time as the wall specimens. This section summarizes the results of the sieve analysis of the used sand, compressive strength of type N masonry cement mortar, compressive strength of clay masonry units and compressive strength of clay masonry prisms.

The sand was tested at the National Concrete Masonry Association Research and Development Laboratory in accordance with ASTM C136 (ASTM, 2006). Results of sieve analysis are shown by a solid line in Figure 3.7, while the upper and lower grading limits of ASTM C144 (ASTM, 2006) are shown by the dashed lines. The figure suggests that the used sand did not satisfy the grading requirements of ASTM C144 (ASTM, 2006). The calculated fineness modulus of the sand was 2.07.

Type N masonry cement 50-mm (2-in.) mortar cylinders were tested in compression according to ASTM C780 (ASTM, 2006). The tests conducted on cylinders
representative of the mortar used in the construction of the masonry prism specimens showed an average compressive strength of 6.55 MPa (951 psi). Though this strength is considered a bit low, the actual strength of the mortar in the walls is expected to be highly variable due to the difference of water content between different batches to enhance the mortar workability which is only controlled by the individual mason.

On the other hand, five representative clay masonry units were tested in compression according to ASTM C67 (ASTM, 2006). Their average compressive strength turned out to be 104 MPa (15200 psi). The compressive strength of clay masonry prisms was determined according to ASTM C1314 (ASTM, 2006) by three two-course masonry prisms. The tests showed an average of 29.86 MPa (4332 psi) compressive strength.
Table 3.1. Summary of Wood-Stud Wall Specimens

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Veneer Dimensions (m/ft)</th>
<th>Window Anchor Type</th>
<th>Joint Rein.</th>
<th>Horizontal Spacing (mm/in.)</th>
<th>Vertical Spacing (mm/in.)</th>
<th>SDC</th>
<th>Shaking Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood 1</td>
<td>1.21x2.58 (4x8.4)</td>
<td>Corrugated</td>
<td>406 (16)</td>
<td>406 (16)</td>
<td>D</td>
<td>I-P</td>
<td></td>
</tr>
<tr>
<td>Wood 2</td>
<td>2.43x2.58 (8x8.4)</td>
<td>x</td>
<td>Corrugated</td>
<td>406 (16)</td>
<td>406 (16)</td>
<td>D</td>
<td>I-P</td>
</tr>
<tr>
<td>Wood 3</td>
<td>1.21x2.58 (4x8.4)</td>
<td>Corrugated</td>
<td>x</td>
<td>406 (16)</td>
<td>406 (16)</td>
<td>D</td>
<td>I-P</td>
</tr>
<tr>
<td>Wood 4</td>
<td>2.43x2.58 (8x8.4)</td>
<td>x</td>
<td>Corrugated</td>
<td>406 (16)</td>
<td>406 (16)</td>
<td>D</td>
<td>I-P</td>
</tr>
<tr>
<td>Wood 5</td>
<td>1.21x2.58 (4x8.4)</td>
<td>Corrugated</td>
<td>x</td>
<td>406 (16)</td>
<td>406 (16)</td>
<td>D</td>
<td>O-O-P</td>
</tr>
<tr>
<td>Wood 6</td>
<td>1.21x2.58 (4x8.4)</td>
<td>Corrugated</td>
<td>x</td>
<td>406 (16)</td>
<td>406 (16)</td>
<td>D</td>
<td>O-O-P</td>
</tr>
<tr>
<td>Wood 7</td>
<td>2.43x2.58 (8x8.4)</td>
<td>Corrugated</td>
<td>x</td>
<td>406 (16)</td>
<td>406 (16)</td>
<td>D</td>
<td>O-O-P</td>
</tr>
<tr>
<td>Wood 7X*</td>
<td>2.43x2.58 (8x8.4)</td>
<td>Corrugated</td>
<td>x</td>
<td>406 (16)</td>
<td>406 (16)</td>
<td>D</td>
<td>O-O-P</td>
</tr>
<tr>
<td>Wood 8</td>
<td>1.21x2.58 (4x8.4)</td>
<td>Corrugated</td>
<td>406 (16)</td>
<td>203 (8)</td>
<td>D</td>
<td>O-O-P</td>
<td></td>
</tr>
<tr>
<td>Wood 9</td>
<td>1.21x2.58 (4x8.4)</td>
<td>Rigid</td>
<td>x</td>
<td>406 (16)</td>
<td>609 (24)</td>
<td>E</td>
<td>O-O-P</td>
</tr>
<tr>
<td>Wood 10</td>
<td>2.43x2.58 (8x8.4)</td>
<td>Rigid</td>
<td>x</td>
<td>406 (16)</td>
<td>609 (24)</td>
<td>E</td>
<td>O-O-P</td>
</tr>
</tbody>
</table>

* Same as specimen Wood 7 but without the top row of ties due to construction error
** Upgraded east-coast construction

Table 3.2. Test Sequence for the Wood-Stud Specimens

<table>
<thead>
<tr>
<th>Level</th>
<th>Wood 1</th>
<th>Wood 2</th>
<th>Wood 3</th>
<th>Wood 4</th>
<th>Wood 5</th>
<th>Wood 6</th>
<th>Wood 7</th>
<th>Wood 7X*</th>
<th>Wood 8</th>
<th>Wood 9</th>
<th>Wood 10</th>
</tr>
</thead>
<tbody>
<tr>
<td>S20%</td>
<td>0.19g</td>
<td>0.19g</td>
<td>0.18g</td>
<td>0.18g</td>
<td>0.19g</td>
<td>0.19g</td>
<td>0.21g</td>
<td>0.19g</td>
<td>0.19g</td>
<td>0.19g</td>
<td>0.20g</td>
</tr>
<tr>
<td>S40%</td>
<td>0.41g</td>
<td>0.47g</td>
<td>0.39g</td>
<td>0.43g</td>
<td>0.39g</td>
<td>0.47g</td>
<td>0.39g</td>
<td>0.43g</td>
<td>0.39g</td>
<td>0.38g</td>
<td>0.41g</td>
</tr>
<tr>
<td>S80%</td>
<td>0.90g</td>
<td>0.92g</td>
<td>0.83g</td>
<td>0.82g</td>
<td>0.81g</td>
<td>0.92g</td>
<td>0.83g</td>
<td>0.82g</td>
<td>0.82g</td>
<td>0.88g</td>
<td>0.90g</td>
</tr>
<tr>
<td>S100%</td>
<td>1.10g</td>
<td>1.09g</td>
<td>1.06g</td>
<td>1.03g</td>
<td>1.11g</td>
<td>1.09g</td>
<td>1.06g</td>
<td>1.03g</td>
<td>1.05g</td>
<td>1.06g</td>
<td>1.10g</td>
</tr>
<tr>
<td>S125%</td>
<td>1.38g</td>
<td>1.41g</td>
<td>1.35g</td>
<td>1.35g</td>
<td>1.38g</td>
<td>1.41g</td>
<td>1.35g</td>
<td>1.35g</td>
<td>1.36g</td>
<td>1.32g</td>
<td>1.38g</td>
</tr>
<tr>
<td>S150%</td>
<td>1.63g</td>
<td>1.63g</td>
<td>1.67g</td>
<td>1.68g</td>
<td>1.63g</td>
<td>1.63g</td>
<td>1.67g</td>
<td>1.65g</td>
<td>1.64g</td>
<td>1.64g</td>
<td>1.63g</td>
</tr>
<tr>
<td>T70%</td>
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<td>1.36g</td>
<td>1.35g</td>
<td>1.35g</td>
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<td>1.36g</td>
<td>1.35g</td>
<td>1.36g</td>
<td>1.40g</td>
<td></td>
<td></td>
</tr>
<tr>
<td>T100%</td>
<td>2.03g</td>
<td>2.03g</td>
<td>2.01g</td>
<td>2.05g</td>
<td>2.03g</td>
<td>2.01g</td>
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<td>2.03g</td>
<td>2.03g</td>
<td>2.03g</td>
</tr>
<tr>
<td>T125%</td>
<td>2.53g</td>
<td>2.50g</td>
<td>2.55g</td>
<td>2.59g</td>
<td>2.50g</td>
<td>2.55g</td>
<td>2.55g</td>
<td>2.56g</td>
<td>2.56g</td>
<td>2.56g</td>
<td>2.53g</td>
</tr>
<tr>
<td>T150%</td>
<td>3.07g</td>
<td>3.08g</td>
<td>3.10g</td>
<td>3.08g</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T150%</td>
<td>3.10g</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

Values in the table represent the actual Peak Ground Accelerations (PGA) as recorded by the table control system.
Table 3.3. Parameters for Design Response Spectra

<table>
<thead>
<tr>
<th>Parameter</th>
<th>SDC D</th>
<th>SDC E</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_s$</td>
<td>1.50</td>
<td>1.50</td>
</tr>
<tr>
<td>$S_1$</td>
<td>0.40</td>
<td>1.00</td>
</tr>
<tr>
<td>$F_a$</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>$F_v$</td>
<td>1.60</td>
<td>1.50</td>
</tr>
<tr>
<td>$S_{MS}$</td>
<td>1.50</td>
<td>1.50</td>
</tr>
<tr>
<td>$S_{M1}$</td>
<td>0.64</td>
<td>1.50</td>
</tr>
<tr>
<td>$S_{DS}$</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>$S_{D1}$</td>
<td>0.43</td>
<td>1.00</td>
</tr>
<tr>
<td>$T_o$</td>
<td>0.085</td>
<td>0.20</td>
</tr>
<tr>
<td>$T_1$</td>
<td>0.43</td>
<td>1.00</td>
</tr>
</tbody>
</table>
Figure 3.1. Typical Details of the Tested Walls (1 m = 39.4 in.)

(a) Solid Wall (Tie Spacing Varies)

(b) Wall with Window Opening (Tie Spacing Varies)

(a) Corrugated Ties

(b) Rigid Ties

Figure 3.2. Veneer Ties
A (Accelerometer), D (Displacement Transducer), L (Potentiometer)
Level (See Figure 3.3)
m (Middle), e (East), w (West), me (Middle-East), mw (Middle-West)
v (Veneer), w (Wood), c (CMU), t (Tie)

(a) Sensor Nomenclature Scheme

(b) Sensor Layout

Figure 3.3. Locations of Displacement Transducers and Accelerometers on Veneer and Backing
Figure 3.4. Test Setup for In-Plane and Out-of-Plane Walls

(a) Sylmar Record

(b) Tarzana Record

Figure 3.5. Ground Motion Records
Figure 3.6. Pseudo-Acceleration Response Spectra (5% Damping)

Figure 3.7. Sieve Analysis of the used Sand
3.7 Acknowledgement of Publications

Parts of this chapter were published in the following journal papers or conference proceedings in which the author of this dissertation was either the primary or a co-author.


CHAPTER 4

OUT-OF-PLANE RESPONSE OF WALL SEGMENTS

4.1 Introduction

This chapter presents the results of the experiments conducted on the out-of-plane wall assemblies outlined in the previous chapter. It starts with a detailed account of the experimental observations and damage progression, followed by an evaluation of the displacement and acceleration responses of the walls. The second part of the chapter presents the numerical modeling framework used to simulate the out-of-plane seismic behavior of masonry veneer. The experimental results of the shaking-table testing program were used for the purpose of model calibration and verification.

The models developed in this chapter are intended to combine the simplicity and the ability to capture the wall behavior and failure mode. The developed models provide the designer with an easy but a rather comprehensive tool for the analysis of masonry veneer in wood-stud walls. In other words, provide an analysis scheme that can be used for design and replace the current prescriptive requirements readily available in building codes. With this scheme, it would be easy to input different properties for the veneer anchors and the used materials for the masonry and the wood-studs and obtain a more reliable assessment of the wall performance. The same modeling approach can be easily expanded to model veneers on different kinds of structural backing like light-gage steel
studs and reinforced concrete masonry backing. The software package Open System for Earthquake Engineering Simulation (OpenSees) developed at the Pacific Earthquake Engineering Center was used for the dynamic simulations conducted on the developed finite element models.

4.2 Experimental Observations

This section presents the qualitative observations of damage evolution in the wall specimens. Pictures of damaged specimens are provided with each specimen, and schematics of cracking patterns in the veneer walls prior to collapse are shown in Figure 4.16. A summary of the experimental observations is presented in Table 4.1. It should be noted that some cracks were not immediately noticeable when they first occurred and a combination of visual inspection and processing of test videos was used to deduce the presented crack patterns.

4.2.1 Specimen Wood 5

No damage was observed in Wood 5 before Tarzana 70%. Right after the test with Tarzana 70%, partial nail extraction from the wood studs was observed for the ties close to the mid-height of the wall (Figure 4.1). After Tarzana 100%, a horizontal bed-joint crack was observed above the 17th brick course from the wall base, two courses above the partially extracted tie, but the veneer remained standing. The wall failed under Tarzana 125% after the formation of an additional bed-joint crack just beneath the fifth brick course from the top of the wall (16 courses above the first crack), as shown in
Figure 4.16, and the rotation of the portion of the wall below the first crack about the wall base. This led to the formation of a collapse mechanism. Examination of the veneer debris showed that failure had occurred by a combination of nail pullout (Figure 4.2) and detachment of veneer anchors from the mortar joint.

4.2.2 Specimen Wood 6

Similar to Wood 5, right after Tarzana 70%, partial nail extraction from wood studs was observed for those ties close to the mid height of Wood 6 (Figure 4.3). During Tarzana 100%, a horizontal bed-joint crack propagated immediately above the 19th brick course from the wall base, four courses above the partially extracted tie, but the veneer remained standing. The wall failed under the Tarzana 150% shaking after the formation of two additional bed-joint cracks, one four courses above the wall base and the other six courses above the first crack (Figure 4.4). This led to the rotation of the lower portion of the veneer about the bed-joint crack four courses from the wall base with most of the ties pulled out from the wood-studs. The crack pattern of the specimen prior to collapse is presented in Figure 4.16.

4.2.3 Specimen Wood 7

After the Tarzana 70% loading, partial nail extraction from wood-studs was observed at the ties at the window piers (above the eighth brick course from the window sill) of Wood 7. A bed-joint crack also formed in the west pier at the level of the window sill. During the Tarzana 100% loading, the bed joint in the west pier at the location of the
extracted tie sheared away. Under the Tarzana 125% loading, the two window piers collapsed after the formation of additional bed-joint cracks in the piers close to the lintel beam and window sill. The top rows of ties above the window were still supporting the veneer both laterally and vertically, as shown in Figure 4.5. Detachment of ties from the mortar joints was also observed in the collapsed veneer. The crack pattern of the specimen prior to collapse is presented in Figure 4.16.

4.2.4 Specimen Wood 7X

This wall failed under Sylmar 125% without any prior signs of damage. The wall failed by the formation of a horizontal bed-joint crack at the level of the lintel beam, about which the top portion of the veneer rotated (Figure 4.6), and nails were pulled out from the backing at the first row of ties above the window (Figure 4.7). This premature failure of the wall can be attributed to the absence of the top row of ties (due to a construction error). This lack of ties resulted in a larger unsupported height of brick above the window. The crack pattern of the specimen is presented in Figure 4.16.

4.2.5 Specimen Wood 8

The failure of this wall occurred under the Tarzana 125% shaking without any prior signs of damage. It was characterized by the formation of a horizontal bed-joint crack below the fifth course from the top of the wall (three brick courses below the top row of ties), as shown in Figure 4.8. This cracking was accompanied by a pullout of the top row of ties from the wood studs. This row of veneer anchors had a slightly larger
tributary area those in the rest of the wall because of masonry coursing limitations. In this specimen, relative motion was observed in the wood-stud wall between the double top plate and the vertical studs (Figure 4.9). This is believed to have been caused by the lack of a firm connection between the two components. The top plate had been lifted slightly from the vertical studs when it was fastened to the steel channel at the top of the steel restraining frame.

4.2.6 Specimen Wood 9

Wall 9 failed under Tarzana 100% shaking with a collapse mechanism formed by bed-joint cracks right above the 14th, 23rd, 28th and 32nd brick courses from the wall base (Figure 4.10). The crack above the 28th brick course was formed in the previous level of excitation (Tarzana 70%). Tie failure was due to a combination of tie pullout from mortar joints (Figure 4.11) and the detachment of ties from the screws by the pull-through of the screw heads in the drilled fastener holes in the tie, which were deformed and enlarged by the screw heads. The crack pattern of the specimen is presented in Figure 4.16.

4.2.7 Specimen Wood 10

After the Tarzana 70% shaking, partial screw extraction from the wood-studs was observed for some ties at the window piers of Wood 10. During the Tarzana 125% loading, a total collapse of the wall occurred without warning, starting with the failure of the window piers followed by the collapse of the portion of the veneer above the window (Figure 4.12). The failure was characterized by bed-joint cracking and the pullout of ties
from the mortar joints (Figure 4.13). Moreover, many screws had pulled through the ties holes due to deformation of the tie holes as in Specimen Wood 9 (Figures 4.14, 4.15). The crack pattern of the specimen prior to collapse is presented in Figure 4.16.

4.2.8 Summary of the Experimental Observations

In general, all the out-of-plane (O-O-P) walls performed well under the DBE and MCE levels without any visible signs of damage with the only exception of specimen Wood 7X. Specimens Wood 5, Wood 6, Wood 9, Wood 7 and Wood 10 show a distribution of cracks that is consistent with a wall that is experiencing one-way bending, in which the veneer acts as a vertically spanning simple beam. This is due to its attachment to a wood-stud frame that is supported at the top with the steel restraint frame and to the footing at the base. Most of the cracks formed in the vicinity of the wall mid height. Wood 7X showed a crack pattern that is consistent with the observed construction defect in which the masons omitted the top row of veneer ties. Only specimen Wood 8 showed a slightly strange and rather unexpected crack pattern. This was caused by the observed movement of the wood-studs due to the top-plate uplift leading to an amplification of the experienced acceleration at the veneer top confirmed by the numerical results.

For the walls with the corrugated ties, damage was characterized by progressive nail extraction from the wood-studs and cracking of a bed joint at or in the vicinity of the partially extracted ties. With increasing nail extraction under higher ground accelerations,
the lateral deflection of the veneer at the cracked bed joint increased and an additional bed-joint crack formed usually above the first crack and the lower portion of the wall rotated about the base, leading to the formation of a collapse mechanism. Complete extraction of the nails from the studs was observed in most locations along the wall height. For the walls with rigid ties, the specimen failure was sudden and was accompanied by the detachment of ties from mortar joints that fractured. Pullout of the screws from the studs (in only a few cases) and the pull-through of the screw heads through deformed tie holes were also observed at some locations. The walls with the rigid ties exhibited a sudden failure as compared to those with the corrugated ties. This may be attributed to the higher strength of the rigid ties (mainly due to a stronger connection to the backing through the screws), which led to the pullout of the ties from mortar joints.

Even though Wood 6 (which had joint reinforcement), collapsed at a slightly higher peak ground acceleration than Wood 5, it is difficult to attribute this slightly higher capacity to the joint reinforcement. Both walls had similar cracking patterns and failure mechanisms. In both walls, most tie failures were due to nail extraction, and only a few were detached from the mortar joints. The latter failure mode may be viewed as a consequence of the fracture of the mortar joints at the position of the veneer anchors during the wall collapse.
4.3 Evaluation of the Experimental Results

4.3.1 Dynamic Properties of the Tested Walls

Acceleration time histories from the veneer and the backing under white-noise excitation were analyzed for all walls to determine their fundamental frequencies after they had been progressively damaged. Four levels were chosen for each wall, namely, the initial undamaged state, after Sylmar 80% (DBE), after Sylmar 125% (MCE) and the last conducted white noise test. Table 4.2 presents the fundamental frequency as well as the progression of the fundamental frequency and its reduction after each one of the aforementioned tests. The table also provides the percentage reduction from the initial fundamental frequency after the DBE and the MCE and before collapse.

All the O-O-P walls, other than specimen Wood 7X which failed at the MCE level, all the tested walls had a reduction in fundamental frequency of less than 15% from the initial value for loadings up to MCE. This reduction in stiffness could be attributed to the cracking of the bed joints of the veneer, straightening of the corrugated ties, and reduction in tie stiffness due to dislodging of mortar dropping, and almost imperceptible amounts of nail extraction. Examining the frequencies at the final measurement stage and extrapolating to the point of eminent failure, a reduction of about 35 to 45% in the fundamental frequency could be expected at the verge of the wall collapse.
4.3.2 Displacement and Acceleration Responses

Figures 4.17 through 4.22 show the time histories of the relative displacement between veneer and backing wall for all the O-O-P walls (Wood 5 to Wood 10) at the tie locations that correspond to the largest relative displacements. They represent the tie deformation time histories at the respective locations and were obtained at the ground motion level of Tarzana 70%. The choice of this level is motivated by the fact that in most of the tested walls, the measurement devices on the veneer side were removed after this ground motion level to avoid damage when the veneer collapsed. This will provide a base of comparison of the observed response under the same ground motion level.

The figures show that the ties experienced larger tensile deformations (positive) than compressive deformations (negative). For the corrugated ties in specimens Wood 5 through Wood 8, this difference may be attributed to the mortar droppings between the veneer and the OSB restricting movements in the air space and protecting the ties from compressive buckling. The figures also show that the rigid ties in Wood 9 and Wood 10 had as low as half the tensile deformation in the corrugated tie wall in specimen Wood 5 and negligible compressive deformation, due to the high strength and stiffness of these ties.

Figure 4.23 presents the plots of the peak backing acceleration versus the peak ground acceleration from all seven O-O-P wall tests. The peak backing acceleration shown is the maximum absolute acceleration measured along the height of the wall during a test run. With the exception of Specimen Wood 7X, the relation is quite linear
up to the DBE level, and close to linear up to the MCE level. The plots indicate that the walls experienced a dynamic amplification of less than two for the Sylmar ground motion and about two for the Tarzana ground motion. Furthermore, based on the data collected up to the point of partial nail extraction, the measured veneer acceleration were very close to the backing acceleration in all cases.

4.4 Analysis of Masonry Veneer Subjected to Out-of-Plane Excitation

In this section the modeling scheme, related to the dynamic simulations of the out-of-plane behavior of the wall assemblies, is discussed in detail. It starts with a detailed description of the finite element discretization and the used material models and hysteretic rules. A comparison between the experimental and analytical results is then presented to validate the ability of the model to capture the wall observed performance. The results of a companion parametric study are discussed in detail to shed some light on some of the influential parameters that affect the model sensitivity as well as the dynamic behavior of masonry veneer.

4.4.1 Beam Element Discretization and Boundary Conditions

The model of a typical tested wall assembly is shown in Figure 4.24. The model is essentially a two-dimensional finite elements model composed of one-dimensional line elements, namely beam and truss elements. These elements had a displacement-based formulation that is readily available in OpenSees. Beam elements were used to model the wood-stud frame and the masonry panel, while the veneer anchors were modeled using
truss elements. Pinned supports were specified for the nodes at the base of the masonry veneer and the wood-stud frame. A spring support is defined at the top of the wood-stud frame to take into account the effect of the restraining frame flexibility. The node at the top of the wood frame is also restrained in the vertical direction to simulate the support provided by the steel restraining frame. The modeling scheme implicitly considers the following:

- Bending in the veneer and the wood-stud wall is one-way in that the displacements and accelerations of the veneer and the backing at any given elevation do not vary with the horizontal distance. This assumption was confirmed by the experimental data.
- All the wood studs in the backing can be lumped into a single beam. The cross-sectional area of the beam is taken as \((n \times 38 \, \text{mm} \times 89 \, \text{mm})\) as shown in Figure 5.1, where \(n\) is the number of wood-studs.
- All the veneer anchors at any given elevation are lumped together into one equivalent truss element with a cross-sectional area equal to the sum of that of the individual anchors (i.e., \(n \times A_{\text{Anchor}}\), where \(n\) is the number of anchors at any given row of ties).
- For the walls with window opening, the beam elements representing the veneer and the backing have a reduced width and number of studs, respectively, to account for the opening.
4.4.2 Materials and Cross-Sectional Properties

The masonry veneer was modeled by a concrete material model with linear tension softening (OpenSEES ID: Concrete 02). The mechanical properties of the material were chosen based on the experimental data from masonry prisms and mortar cylinders tested in compression. Figures 4.25 and 4.26 show the behavior for the concrete material model under monotonic compression and tension, respectively. The tensile strength of the concrete model was set to be 1% of the mortar compressive strength. This was intended to simulate the lower tensile strength of the mortar-brick interface. A uniaxial linear elastic material model was chosen for the wood with an elastic modulus of 10.3 GPa (1500 ksi). An 890 kN/m (5 kips/in.) linear elastic spring was assigned to the top of the wood-stud frame to simulate the restraint provided by the steel support frame. This value was chosen so that the displacements of the backing would match the experimentally recorded displacements. The axial load due to the self-weight of the elements was applied as concentrated vertical loads at the element nodes. Similarly, the tributary horizontal mass was lumped at the nodes of the beam elements along the height. A mass density of 1.83 gm/cm$^3$ (114 lb/ft$^3$) was used for the masonry veneer and 0.45 gm/cm$^3$ (28 lb/ft$^3$) for the wood-studs.

4.4.3 Hysteretic Models for Veneer Anchors

Nonlinear truss elements are used to model the corrugated and rigid veneer anchors. The hysteretic rules used for the two types of veneer anchors are shown in Figures 4.27 and 4.28. The hysteretic relationships are developed based on the perceived
response of a single anchor. The figures show that the tie behavior in compression is essentially linear elastic with stiffness $K_2$; this stiffness is higher than that in the linear ascending branch in tension ($K_1$) for the corrugated ties and is the same as $K_1$ for the rigid ties. The tie capacity corresponds to either the nail pullout capacity from the wood-studs for the case of corrugated anchors or the tie pullout capacity from the mortar joints for the rigid anchors, since these are the dominant failure modes for these kinds of anchors under axial loading. The higher stiffness and non-yielding behavior in compression is a result of the presence of mortar droppings around the corrugated ties. These droppings, not controlled in specimen construction, helped to transfer the compressive load between the veneer and the backing by bearing and restrain the anchors from buckling. For the rigid ties, the same stiffness is assumed for tension and compression due to the higher stiffness of the rigid steel brackets, for which the influence of the mortar droppings becomes insignificant. A descending branch with stiffness $K_3$ characterizes the post-yield domain of the anchor response in tension.

The unloading and reloading branches of the tie model in tension have the same stiffness $K_1$ as the loading branch with full pinching of the force component in the force-deformation relationship. The parameters for the hysteretic models for the corrugated and rigid anchors are shown in Table 4.3. These models represent a simplified idealization for the veneer anchors’ behavior, which as reported in the literature is generally characterized by an unsymmetrical pinched hysteresis that shows strength deterioration. As will be shown in the following subsections, these detailed features of the hysteretic behavior of the veneer anchors, such as the asymmetric loops, did not have much
influence on the global response of the wall assembly. Very close match with experimental results was easily achievable with the simple uncomplicated hysteresis, which represents an average overall behavior of the veneer anchors in the wall assembly.

4.4.4 Time History Analyses

Nonlinear time history analyses were conducted with the models of the tested wall assemblies described above. In these analyses, Rayleigh damping was used with the damping ratios specified to be 5% of the critical for the fundamental and the 12th natural vibration modes in order to minimize the damping effect for the high-frequency axial modes that can be excited by veneer cracking. The damping matrix was formulated as a linear combination of the mass and the tangent stiffness matrices. The constant-average-acceleration method was used to integrate the equations of motion for the system with an integration time step of 0.001 second. The models were analyzed with a sequence of ground motions recorded from the shake table to better represent the actual base excitation experienced by the tested specimens. Numerical results obtained for wall specimens Wood 5, 7, 9, and 10 are presented in the following sub-section. Wood 5 had a solid veneer with corrugated ties, Wood 7 had a window opening and corrugated ties, Wood 9 had a solid veneer with rigid ties, and Wood 10 had a window opening and rigid ties. The results include the veneer and wood-stud frame displacement and acceleration profiles at the peak response and the anchor force distribution along the wall height.
4.4.5 Comparison of Analytical and Experimental Results

4.4.5.1 Specimen Wood 5

Figures 4.29, 4.30 and 4.31 show the displacement profiles at the instant of the peak positive veneer displacement (with the veneer moving away from the wood backing) for the ground motion records of Sylmar 40%, Sylmar 150% and Tarzana 100%, respectively. Figure 4.29 shows that for the low-level shaking, the model exhibits a stiffer response than the test. The softer response of the actual walls could be attributed to the presence of shrinkage cracks in the mortar joints. It is also worth mentioning that the magnitudes of the displacements are very small and the differences between the numerical and experimental results are in the order of 1mm. Under higher ground motion levels, the model shows a very close match with the experimental results as shown in Figures 4.30 and 4.31. The model can predict the tie deformation (defined as the difference between the veneer and the wood-stud backing displacements at the positions of the veneer anchors) with a high degree of accuracy.

Figure 4.32 shows the acceleration profiles of the wall subjected to the aforementioned ground motions. This plot is generated at the moment when the acceleration of the wall reaches the peak value. The figure shows that the numerical model matches the experimental results well. Figure 4.33 shows the distribution of the force per anchor along the wall height. These plots are generated at the same time instants as the acceleration plots. Under a very low ground motion, when the wall is uncracked, the distribution of the forces largely depends on the backing flexibility and support
conditions. In this case, the tie force is higher at the top of the wall and decreases with the height. This behavior is observed under Sylmar 40\% and Sylmar 80\%. Under higher ground motion levels, starting with Sylmar 100\%, the veneer is cracked at an elevation close to the mid-height and the distribution of the tie forces is changed significantly. After cracking, the tie forces are higher in the vicinity of the cracks near the mid-height and lower towards the top and bottom of the wall (see Sylmar 150\% in Figure 18). With high ground motion levels, the ties in the vicinity of the cracks enter the post-peak regime of the response and the adjacent rows of ties start to share a larger portion of the load leading to a more even distribution of tie forces along the wall height (see Tarzana 100\% in Figure 4.33). The aforementioned mechanism is likely due to the ductility of the corrugated ties, even though this is limited, as shown in Figure 4.27.

4.4.5.2 Specimen Wood 7

Figures 4.34 and 4.35 show the displacement profiles at the instant of the peak positive veneer displacement for Sylmar 40\% and Tarzana 70\%, respectively. The results show good correlation with the tests. Figure 4.36 shows the acceleration profiles of the veneer for the Sylmar 40\% and Tarzana 70\% ground motions. Once again, this plot is generated at the moment when the acceleration of the veneer reaches the peak. The figure shows a good agreement between the numerical simulations and the shake table experiments. Figure 4.37 shows the distribution of the force per anchor along the wall height. The plot presents a behavior similar to that observed in Wood 5. Under a low-level ground motion, when the veneer is uncracked, the distribution of the forces is
characterized by higher forces at the top of the wall and decreases with height. Under higher ground motion levels, a redistribution of the anchor forces is observed (see Tarzana 70% in Figure 4.37), which can also be attributed to the ductility of the corrugated ties. Compared to Wood 5, veneer cracking is observed at a lower ground motion level (Sylmar 80% for Wood 7 and Sylmar 100% for Wood 5). This may be attributed to the reduced veneer cross section at the window opening.

**4.4.5.3 Specimen Wood 9**

Figures 4.38, 4.39 and 4.40 show the displacement profiles at the instant of the peak positive veneer displacement for Sylmar 40%, Sylmar 80% and Tarzana 70%, respectively. Figure 4.41 shows the acceleration profiles of the masonry wall under the same ground motions. The figure shows a good match between the numerical simulations and the shake table experiments. Figure 4.42 shows the distribution of the force per anchor along the wall height. The plot shows an initially similar behavior to that observed in the previous specimens. However, under higher ground motion levels, the behavior of the wall with rigid ties is quite different from that with corrugated ties. The localized high tie force near the crack keeps increasing with no signs of redistribution of the tie forces, which is observed in the previous walls. This force distribution is maintained up to the tie pullout capacity, after which the wall suddenly collapses. This observation from the model is in perfect agreement with the experimental observations. This behavior can be attributed to the stiff brittle behavior of the rigid ties as shown in Figure 4.28.
Another difference between the corrugated-tie and rigid-tie walls is the level of excitation at which cracking is observed in the model. Although the rigid ties has a significantly higher pullout capacity as compared to that of the corrugated ties, Wood 5 has veneer cracking under Sylmar 100%, while Wood 9 is cracked under Sylmar 80%. This discrepancy may be explained by the difference in the vertical spacing of the veneer anchors in the two cases. Since the corrugated ties have closer spacing than the rigid ties, the bending moments induced in the veneer in the rigid-tie wall will be higher than those in the corrugated tie-walls for the same lateral load; hence, it is expected that the veneer in Wood 9, which had rigid ties, will reach the cracking moment earlier than Wood 5.

4.4.5.4 Specimen Wood 10

Figures 4.43 and 4.44 show the displacement profiles at the instant of the peak positive veneer displacement for Sylmar 40% and Tarzana 70%, respectively. Figure 4.45 shows the acceleration profiles of the veneer under the same ground motions. The figure shows a close match between the numerical simulations and the shake table experiments. Figure 4.46 shows the distribution of the force per anchor along the wall height. The distribution replicates that of Wood 9. Cracking is observed in the model under Sylmar 80%.

4.4.6 Capacity of Veneer Ties

In the course of the model development and validation, the hysteretic tie model parameters, including the tie strengths, are tuned to arrive at the values shown in Table
4.4, which provide a good match between the experimental and analytical results. The values shown in this table reveal that there is a higher variation in the estimated tie strengths for the case of the corrugated ties as compared to the rigid ties. A 25% difference in tie strengths is observed for the corrugated ties used in Wood 5 and Wood 7, while only 4% difference is observed for the rigid ties. These differences qualitatively match the expectations for these two types of anchors. Nail pullout capacities are expected to have a large coefficient of variation.

The analyses have shown that for the wall specimens, tie pullout is always preceded by veneer cracking, and that right after a veneer cracks, the tie forces near a crack become higher than those at other locations and the distribution of tie forces resembles that of the veneer acceleration. Hence, it is reasonable to assume that once a veneer has lost its bending stiffness due to cracking, the tie forces are governed by the inertia force developed by the veneer. With this assumption, one can also estimate the tie capacities from the shaking-table test results by considering the weight of the veneer, the tributary wall area per tie, and the veneer acceleration at which significant nail extraction (for the corrugated ties) or tie failure (for the rigid ties) first occurred. For this calculation, the maximum accelerations of the wood backing under the respective records are considered because veneer acceleration data at tie failure is not available for Wood 9 and 10 (due to the removal of the accelerometers), and the backing acceleration is expected to be very close to the veneer acceleration before the occurrence of significant nail extraction or tie failure. The respective maximum backing accelerations can be
obtained from Figure 4.23 by identifying the peak ground accelerations at which these events occurred.

The tie capacities estimated this way are shown in the second row of Table 4.4. One can observe from the table that the tie capacities estimated from the inertia forces are a little lower than those determined with the analytical models. For comparison purposes, the withdrawal strengths of bright common 8d nails and #8 screws as given in the load and resistance factor design provisions of the National Design Specification for Wood Construction (NDS). One can see that the capacities of the ties deduced from the test results are higher than those specified in NDS and show a significant scatter. Rammer et al. have indicated that the withdrawal strengths of smooth shank nails have the coefficients of variation between 22% and 48%. Nail withdrawal strengths highly depend on the wood grade, nail length, nail shank type, and moisture content of the wood. The implication of the large variability in nail extraction strength on the seismic performance of masonry veneer needs attention. The capacities of the rigid anchors (Wood 9 and 10) observed from the tests are significantly higher than the screw withdrawal strength given by the code. It should be noted that screw withdrawal was not observed in the tests and the rigid anchors failed mainly by detachment from the mortar joints.

4.4.7 Simulation of Failure Mode

This section presents the failure mode produced by the analytical models. Since most of the instruments on the veneer side were disconnected before the collapse run,
only numerical results can be presented in this section and qualitatively compared with the experimental observations. Figures 4.47 through 4.50 present the displacement profiles of the veneer and the backing. The plots are for illustrative purposes only as the high displacement values shown could mean that the walls would have already collapsed. Here, collapse is defined as the pullout of at least one row of ties either from the backing or the masonry veneer. This definition is motivated by the fact that once a row of ties is completely extracted, its load will be immediately transferred to the adjacent rows causing them to be pulled out and initiating a progressive collapse.

The plots show that the model predicts that all the ties in specimen Wood 5 are pulled out under Tarzana 125% starting from the mid height and spreading towards the wall ends. This result is in perfect agreement with the experimental observations. The analytical model predicts that specimen Wood 7 has the sixth and seventh rows of ties, located in the window piers, pulled out while the rest of the ties remains connected to the veneer and the backing. Once again, this is in agreement with the experimental observations. Specimens Wood 9 and Wood 10 collapsed under Tarzana 100% and 125 %, respectively, by complete pullout of all the ties in the wall starting from the top of the wall. This is clearly demonstrated in Figures 4.49 and 4.50.

4.5 Parametric Study on the Out-of-Plane Response

This section presents an extension of the analytical study presented above. The study is intended to investigate the sensitivity of the model and the influence of the
change of some parameters on the wall's global seismic response. The parameters studied were the anchor capacity of the corrugated ties, anchor spacing of the corrugated ties, anchor initial stiffness for corrugated and rigid ties and the effect of the initial slack of the corrugated ties. The following sub-sections present the used variations of the parameter value and the conclusions based on the global wall behavior. Comparisons were made in terms of the deflected shape of the wall, state of veneer cracking, tie yielding and/or pullout and whether the wall did collapse or not. The models were analyzed dynamically under three historic ground motion records, namely the Sylmar, Tarzana and El Centro ground motions. These records were scaled appropriately to represent both the Design Basis (DBE) and Maximum considered earthquakes for Seismic Design Categories D and E of ASCE 7-05 (ASCE, 2005), in a manner conforming to that used in the wall assembly tests presented in Chapter 3. Scaling factors of 80% and 120% were applied to the Sylmar record, 36% and 54% to the Tarzana record and 150% and 225% to the El Centro to arrive at the DBE and MCE, respectively.

4.5.1 Influence of Corrugated Anchor Capacity

In order to investigate the effect of the capacity of the corrugated anchor, mainly the withdrawal strength of the used fastener, three models as shown in Table 4.5 were analyzed. Models RC16-S1-II, RC16-S2-II and RC16-S3-II all had corrugated ties spaced at 406 mm (16 in.) vertically and used the same initial stiffness ($K_1$) for the tie hysteretic model as that deduced from the model calibration. The three models had peak tie strengths of 222 N, 667 N and 1780 N (50 lbs., 150 lbs. and 400 lbs.), respectively.
The choice of this variation reflects: 1) lower bound strength for nail withdrawal due to the effect of moisture conditions and any other factors that may reduce this strength, 2) average nail withdrawal strength commonly reported and expected for the 8d nails and 3) an upper bound value representing the withdrawal strength of an alternative faster such as screws and ring-shank nails. All these models were analyzed with the assumption of a rigid roller support at the top of the wood-stud frame.

Table 4.6 summarizes the results of the three models. The table shows that for the lower bound tie capacity, the veneer deforms under the so-called "Deflection Mode I", defined as the veneer leaning away from the backing as a rigid body rotating about the wall base. Tie yielding is expected at the DBE starting from the top row of ties which experience the most elongation due to this deflection mode. Eventually collapse is guaranteed under the MCE level due to the pullout of the extracted anchors. On the other hand, identical behavior is observed for veneer secured to the backing with nails having the average withdrawal capacity or fastened with screws or ring-shank nails having the upper-bound tie capacity. For these models the veneer deformed in "Deflection Mode II", which is the veneer experiencing bending about mid-height following the backing deflected shape. In most cases, veneer is expected to remain intact under the DBE and to crack, at about mid-height, under the MCE. The anchors did not experience any yielding or pullout under either ground motion levels.
4.5.2 Influence of Corrugated Anchor Initial Stiffness

Models RC16-S2-I1, RC16-S2-I2 and RC16-S2-I3, as shown in Table 4.5, all had corrugated ties spaced at 406 mm (16 in.) vertically and peak tie strengths of 667 N (150 lbs.). Three values of the initial stiffness were used, namely, $K_1$, 50% of $K_1$ and 150% of $K_1$. Once again, Table 4.6 summarizes the results of these models. The results shows that for higher stiffness (150% of $K_1$), no difference in behavior is observed while for the lower stiffness, severe tie yielding and pullout, veneer distress and even collapse is observed under the DBE.

4.5.3 Influence of Rigid Anchor Initial Stiffness

Models RR24-S2-I1, RR24-S2-I2 and RR24-S2-I3, as shown in Table 4.5, all had rigid ties spaced at 609 mm (24 in.) vertically and peak tie strengths of 1780 N (400 lbs.). Three values of the initial stiffness we used, namely, $K_1$, 75% of $K_1$ and 125% of $K_1$. Smaller values for variation were used here in comparison to the corrugated ties due to the higher confidence in the rigid ties and the more secure fastening method to the backing (screws). Table 4.7 summarizes the results of these models. The results show no difference in response and performance for the different tie initial stiffness. All the models deformed in mode II with no veneer distress or tie yielding under the DBE, and only veneer cracking at mid-height at the MCE.
4.5.4 Influence of Corrugated Anchor Spacing

Table 4.5 shows model RC8-S1-I1 that was analyzed and compared with model RC16-S1-I1. Both had corrugated ties spaced at 203mm and 406 mm (8 in. and 16 in.) vertically, respectively. The same initial stiffness \( (K_i) \) for the tie hysteretic model as that deduced from the model calibration was used with a peak tie capacity of 222 N (50 lbs.). Tables 4.6 and 4.8 summarize the results of these models. One can see the effect of reducing the vertical spacing to half have on the wall performance. First the closer spacing forced the veneer in most cases to deform in mode II rather than mode I in the case of the wider spacing. This may be attributed to a more secure connection between the veneer and the wood-stud backing. The main striking difference is that the wall did not collapse under the DBE or the MCE. Yielding of the higher rows of ties is observed but none of the ties pulled out.

4.5.5 Influence of Corrugated Anchor Initial Slack

One parameter that required special attention was the initial slack found in the corrugated sheet metal ties. Models RC16-S1-I1 and RC16-S2-I1 discussed previously were analyzed but with the addition of a 3.20 mm (1/8 in.) initial slack to the corrugated tie hysteretic model. This was added by shifting the ascending branch of the tensile domain of the hysteretic model by the slack value towards the right. Table 4.9 shows the unfavorable effect the slack had on the response. The wall with the lower bound extraction capacity and the one with the average strength collapsed under the DBE and
MCE, respectively. More importantly is the observation of the generation of big acceleration spikes on both the veneer and the backing.

### 4.5.6 Conclusions of Parametric Study

The results of the previous parametric study suggest that the pullout strength of the fastener is the most influential parameter on the dynamic response of the walls. Walls having sufficient anchor strength are thought to deform in Mode II due to sufficient capacity of inertia load transfer that securely ties the veneer and the backing together. These walls usually crack about the mid-height and allow for force redistribution due to the anchor ductility. Reducing the anchor spacing was found to have a similar effect as increasing the anchor strength, but more importantly it helps the veneer to follow the backing deflected shape and hence force the wall in Mode II deformation. Initial slack found in the corrugated ties magnified the accelerations experienced by the backing and the veneer, subjecting the ties to a higher inertial load. This is mainly attributed to the shock the veneer induces into the backing when the anchors pull taut. Initial stiffness of both kinds of veneer anchors as those observed in the test are sufficient for satisfactory performance. One can conclude that the superior performance of the rigid ties may be mainly attributed to the fastener capacity rather than the anchor design. Screws and ring-shank nails have not only higher withdrawal strengths but also the ability to sustain this strength under cyclic moisture conditions with minimal reduction of extraction capacity.
4.6 Summary and Conclusions

The experimental study presented in this chapter shows that the failure of clay masonry veneer over a wood-stud backing wall was triggered by nail extraction for the case of the corrugated ties, and by the pullout of the ties from the mortar joints or the pull-through of the screw heads from the screw holes in the tie brackets for the case of the rigid ties. The rigid ties were much stronger than the corrugated ties, but the failure of the veneer with the rigid ties occurred suddenly. The analyses indicate that nail extraction and tie failure in the tests were most likely preceded by veneer cracking.

Test results show that both the corrugated and rigid veneer ties conforming to the MSJC code (MSJC, 2008) sustained ground motions in excess of the Design Basis and Maximum Considered Earthquakes. However, one should be particularly cautious about corrugated ties that are secured to wood backing with nails. The nail extraction capacity has a high variability depending on the quality and moisture content of the wood and the surface condition of the nails. Furthermore, for taller walls, the dynamic amplification of the base acceleration could be more severe than what was observed here.

The test results do not provide evidence that joint reinforcement could increase the capacity of the corrugated ties or improve the performance of clay masonry veneer under out-of-plane seismic loading. Since the failure of the ties was governed by nail extraction, one can conclude that joint reinforcement did not provide any added value to the anchor system.
The seismic performance and collapse mechanism of masonry veneer in wood-stud walls have been analyzed using computational models developed and calibrated with the results of the shaking-table experiments conducted on wall assemblies. Hysteretic models for veneer anchors subjected to axial loading were proposed and verified with the shaking-table test results. A parametric study was conducted on the influential parameters on the response of masonry veneer. Anchor pullout strength seemed to be the most influential parameter and the numerical results showed the high variability of the pullout capacities of the nails used to attach the corrugated ties as compared to the screws. Slack found in the corrugated ties showed a devastating effect on the wall performance.
Table 4.1. Summary of Experimental Observations

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Shaking Direction</th>
<th>Failure Motion</th>
<th>PGA at Failure</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood 5</td>
<td>O-O-P</td>
<td>T 125%</td>
<td>2.59 g</td>
<td>Nail pullout, bed joint cracking</td>
</tr>
<tr>
<td>Wood 6</td>
<td>O-O-P</td>
<td>T 150%</td>
<td>3.08 g</td>
<td>Nail pullout, bed joint cracking</td>
</tr>
<tr>
<td>Wood 7</td>
<td>O-O-P</td>
<td>T 125%</td>
<td>2.55 g</td>
<td>Nail pullout, bed joint cracking in window piers</td>
</tr>
<tr>
<td>Wood 7X*</td>
<td>O-O-P</td>
<td>S 125%</td>
<td>1.35 g</td>
<td>Nail pullout, bed joint cracking at lintel beam</td>
</tr>
<tr>
<td>Wood 8</td>
<td>O-O-P</td>
<td>T 125%</td>
<td>2.56 g</td>
<td>Nail pullout, bed joint cracking near the top</td>
</tr>
<tr>
<td>Wood 9</td>
<td>O-O-P</td>
<td>T 100%</td>
<td>2.03 g</td>
<td>Tie pullout from screw head, bed joint cracking, tie pullout from mortar joints</td>
</tr>
<tr>
<td>Wood 10</td>
<td>O-O-P</td>
<td>T 125%</td>
<td>2.53 g</td>
<td>Tie pullout from screw head, bed joint cracking in window piers, tie pullout from mortar joints</td>
</tr>
</tbody>
</table>

Note: Under PGA, “S” denotes the Sylmar record and “T” denotes the Tarzana record with the acceleration scaling indicated. PGA values reflect actual table acceleration.

Table 4.2. Evolution of the Fundamental Mode Frequency (Hz)

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Initial</th>
<th>S80</th>
<th>S125</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood 5</td>
<td>7.91</td>
<td>7.71 (2.52%)</td>
<td>7.71 (2.52%)</td>
<td>S150 7.22 (8.72%)</td>
</tr>
<tr>
<td>Wood 6</td>
<td>7.91</td>
<td>6.84 (13.5%)</td>
<td>6.73 (14.9%)</td>
<td>T125 6.15 (22.3%)</td>
</tr>
<tr>
<td>Wood 7</td>
<td>9.47</td>
<td>9.18 (3.06%)</td>
<td>8.59 (9.29%)</td>
<td>S125 7.72 (18.48%)</td>
</tr>
<tr>
<td>Wood 7X</td>
<td>8.11</td>
<td>7.91 (2.47%)</td>
<td>7.32 (9.47%)</td>
<td>S100 5.18 (36.13%)</td>
</tr>
<tr>
<td>Wood 8</td>
<td>9.47</td>
<td>8.78 (7.29%)</td>
<td>8.59 (9.29%)</td>
<td>T70 7.81 (17.53%)</td>
</tr>
<tr>
<td>Wood 9</td>
<td>9.86</td>
<td>9.08 (7.91%)</td>
<td>8.59 (12.88%)</td>
<td>T70 7.42 (24.75%)</td>
</tr>
<tr>
<td>Wood 10</td>
<td>9.47</td>
<td>8.78 (7.29%)</td>
<td>8.11 (14.36%)</td>
<td>T70 7.23 (23.65%)</td>
</tr>
</tbody>
</table>

Table 4.3. Hysteretic Rule Parameters

<table>
<thead>
<tr>
<th>Anchor Type</th>
<th>Tie Capacity $P_{ult}$</th>
<th>$K_1$</th>
<th>$K_2$</th>
<th>$K_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corrugated</td>
<td>667 N – 889 N (150 lbs. – 200 lbs.)</td>
<td>263 N/mm</td>
<td>700 N/mm</td>
<td>29.18 N/mm - 40.37 N/mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(1501 lbs./in.)</td>
<td>(3997 lbs./in.)</td>
<td>(166.62 lbs./in.) - (230.52 lbs./in.)</td>
</tr>
<tr>
<td>Rigid</td>
<td>1779 N – 1853 N (400 lbs. – 416 lbs.)</td>
<td>633 N/mm</td>
<td>633 N/mm</td>
<td>1577 N/mm - 1824 N/mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(3614 lbs./in.)</td>
<td>(3614 lbs./in.)</td>
<td>(9005 lbs./in.) - (10415 lbs./in.)</td>
</tr>
</tbody>
</table>
### Table 4.4. Summary of Maximum Anchor Forces

<table>
<thead>
<tr>
<th>Method</th>
<th>Specimen</th>
<th>Wood 5</th>
<th>Wood 6</th>
<th>Wood 7</th>
<th>Wood 8</th>
<th>Wood 9</th>
<th>Wood 10</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(200 lbs.)</td>
<td>N.A.</td>
<td>(150 lbs.)</td>
<td>N.A.</td>
<td>(416 lbs.)</td>
<td>(400 lbs.)</td>
</tr>
<tr>
<td>Analytical Model</td>
<td></td>
<td>889 N</td>
<td>N.A.</td>
<td>667 N</td>
<td>N.A.</td>
<td>1853 N</td>
<td>1779 N</td>
</tr>
<tr>
<td>Inertia Force</td>
<td></td>
<td>870 N</td>
<td>762 N</td>
<td>580 N</td>
<td>792 N</td>
<td>1693 N</td>
<td>1552 N</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(216 lbs.)</td>
<td>(171 lbs.)</td>
<td>(130 lbs.)</td>
<td>(178 lbs.)</td>
<td>(380 lbs.)</td>
<td>(349 lbs.)</td>
</tr>
<tr>
<td>NDS Design</td>
<td></td>
<td>512 N</td>
<td>512 N</td>
<td>512 N</td>
<td>512 N</td>
<td>1326 N</td>
<td>1326 N</td>
</tr>
<tr>
<td>Nail/Screw Withdrawal Strength</td>
<td></td>
<td>(115 lbs.)</td>
<td>(115 lbs.)</td>
<td>(115 lbs.)</td>
<td>(298 lbs.)</td>
<td>(298 lbs.)</td>
<td></td>
</tr>
</tbody>
</table>

### Table 4.5. Properties of Analyzed Models (Parametric Study)

<table>
<thead>
<tr>
<th>Model ID</th>
<th>Tie Type</th>
<th>Vertical Spacing</th>
<th>Strength</th>
<th>Initial Stiffness $K_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC16-S1-I1</td>
<td>Corrugated</td>
<td>406 mm (16 in.)</td>
<td>222 N (50 lbs.)</td>
<td>$K_1 = 263$ N/mm</td>
</tr>
<tr>
<td>RC16-S2-I1</td>
<td>Corrugated</td>
<td>406 mm (16 in.)</td>
<td>667 N (150 lbs.)</td>
<td>$K_1 = 263$ N/mm</td>
</tr>
<tr>
<td>RC16-S3-I1</td>
<td>Corrugated</td>
<td>406 mm (16 in.)</td>
<td>1780 N (400 lbs.)</td>
<td>$K_1 = 263$ N/mm</td>
</tr>
<tr>
<td>RC8-S1-I1</td>
<td>Corrugated</td>
<td>203 mm (8 in.)</td>
<td>222 N (50 lbs.)</td>
<td>$K_1 = 263$ N/mm</td>
</tr>
<tr>
<td>RC16-S2-I2</td>
<td>Corrugated</td>
<td>406 mm (16 in.)</td>
<td>667 N (150 lbs.)</td>
<td>50% $K_1 = 132$ N/mm</td>
</tr>
<tr>
<td>RC16-S2-I3</td>
<td>Corrugated</td>
<td>406 mm (16 in.)</td>
<td>667 N (150 lbs.)</td>
<td>150% $K_1 = 395$ N/mm</td>
</tr>
<tr>
<td>RR24-S2-I1</td>
<td>Rigid</td>
<td>609 mm (24 in.)</td>
<td>1780 N (400 lbs.)</td>
<td>$K_1 = 633$ N/mm</td>
</tr>
<tr>
<td>RR24-S2-I2</td>
<td>Rigid</td>
<td>609 mm (24 in.)</td>
<td>1780 N (400 lbs.)</td>
<td>75% $K_1 = 475$ N/mm</td>
</tr>
<tr>
<td>RR24-S2-I3</td>
<td>Rigid</td>
<td>609 mm (24 in.)</td>
<td>1780 N (400 lbs.)</td>
<td>125% $K_1 = 791$ N/mm</td>
</tr>
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</table>
# Table 4.6. Summary of Parametric Study – Corrugated Ties

<table>
<thead>
<tr>
<th>Model ID</th>
<th>EQ Level</th>
<th>Ground Motion</th>
<th>Deflection Mode</th>
<th>Veneer Cracking</th>
<th>Tie Yielding</th>
<th>Tie Pullout</th>
<th>Collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Tie Number</td>
<td>Tie Number</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1 2 3 4 5 6 7</td>
<td>1 2 3 4 5 6 7</td>
<td></td>
</tr>
<tr>
<td>RC-16-S1-I</td>
<td>DBE El Centro</td>
<td>I</td>
<td>No</td>
<td>x x x</td>
<td>No</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-16-S1-I</td>
<td>DBE Sylmar</td>
<td>I</td>
<td>No</td>
<td>x x x</td>
<td>No</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-16-S1-I</td>
<td>DBE Tarzana</td>
<td>I</td>
<td>No</td>
<td>x x x</td>
<td>No</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-16-S1-I</td>
<td>MCE El Centro</td>
<td>I</td>
<td>Yes</td>
<td>x x x x x x x x</td>
<td>x x x x</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>RC-16-S1-I</td>
<td>MCE Sylmar</td>
<td>I</td>
<td>No</td>
<td>x x x x x x x x</td>
<td>x x x x x x</td>
<td>Yes</td>
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</tr>
<tr>
<td>RC-16-S1-I</td>
<td>MCE Tarzana</td>
<td>I</td>
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<td>x x x x x x x x</td>
<td>x x x x x x</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>RC-16-S2-I</td>
<td>DBE El Centro</td>
<td>II</td>
<td>No</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>RC-16-S2-I</td>
<td>DBE Sylmar</td>
<td>II</td>
<td>No</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-16-S2-I</td>
<td>DBE Tarzana</td>
<td>II</td>
<td>Yes</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-16-S2-I</td>
<td>MCE El Centro</td>
<td>II</td>
<td>Yes</td>
<td>No</td>
<td></td>
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<tr>
<td>RC-16-S2-I</td>
<td>MCE Sylmar</td>
<td>II</td>
<td>Yes</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-16-S2-I</td>
<td>MCE Tarzana</td>
<td>II</td>
<td>Yes</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-16-S3-I</td>
<td>DBE El Centro</td>
<td>II</td>
<td>No</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-16-S3-I</td>
<td>DBE Sylmar</td>
<td>II</td>
<td>No</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>RC-16-S3-I</td>
<td>DBE Tarzana</td>
<td>II</td>
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<td></td>
<td></td>
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<tr>
<td>RC-16-S3-I</td>
<td>MCE El Centro</td>
<td>II</td>
<td>Yes</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-16-S3-I</td>
<td>MCE Sylmar</td>
<td>II</td>
<td>Yes</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-16-S3-I</td>
<td>MCE Tarzana</td>
<td>II</td>
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<td>No</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-16-S2-12</td>
<td>DBE El Centro</td>
<td>II</td>
<td>Yes</td>
<td>x x x x x x x x</td>
<td>x x x x</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>RC-16-S2-12</td>
<td>DBE Sylmar</td>
<td>II</td>
<td>Yes</td>
<td>x x x x x x x x</td>
<td>x x x x</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>RC-16-S2-12</td>
<td>DBE Tarzana</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td></td>
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<tr>
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<td>MCE El Centro</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td></td>
</tr>
<tr>
<td>RC-16-S2-12</td>
<td>MCE Sylmar</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
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<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td></td>
</tr>
</tbody>
</table>

Note: Mode I defined by the veneer leaning away from the backing as rigid body and rotating about the wall base.
Mode II defined by the veneer bending and cracking about the wall mid-height.
Table 4.7. Summary of Parametric Study – Rigid Ties

<table>
<thead>
<tr>
<th>Model ID</th>
<th>EQ Level</th>
<th>Ground Motion</th>
<th>Deflection Mode</th>
<th>Veneer Cracking</th>
<th>Tie Yielding</th>
<th>Tie Pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Row Number</td>
<td>Row Number</td>
<td>Collapse</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>---------</td>
<td>----------</td>
<td>---------</td>
<td>----------</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RR-24-S2-I1</td>
<td>MCE El Centro</td>
<td>II</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td>RR-24-S2-I2</td>
<td>MCE El Centro</td>
<td>II</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td></td>
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<tr>
<td>RR-24-S2-I3</td>
<td>MCE El Centro</td>
<td>II</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td></td>
</tr>
</tbody>
</table>

Note: Mode I defined by the veneer leaning away from the backing as rigid body and rotating about the wall base.
Mode II defined by the veneer bending and cracking about the wall mid-height.

Table 4.8. Summary of Parametric Study – Corrugated Ties, Closer Spacing

<table>
<thead>
<tr>
<th>Model ID</th>
<th>EQ Level</th>
<th>Ground Motion</th>
<th>Deflection Mode</th>
<th>Veneer Cracking</th>
<th>Tie Yielding</th>
<th>Pullout / Collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC-8-S1-I1</td>
<td>DBE El Centro</td>
<td>I</td>
<td>No</td>
<td>x x x x x x x x x x x</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td></td>
<td>DBE Sylmar</td>
<td>I</td>
<td>Yes</td>
<td>x x x x x x x x x x x</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td></td>
<td>DBE Tarzana</td>
<td>II</td>
<td>No</td>
<td>- x x x x x x x x x x x</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MCE El Centro</td>
<td>II</td>
<td>Yes</td>
<td>x x x x x x x x x x x</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MCE Sylmar</td>
<td>II</td>
<td>Yes</td>
<td>x x x x x x x x x x x</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MCE Tarzana</td>
<td>II</td>
<td>Yes</td>
<td>x x x x x x x x x x x</td>
<td>No</td>
<td></td>
</tr>
</tbody>
</table>

Note: Mode I defined by the veneer leaning away from the backing as rigid body and rotating about the wall base.
Mode II defined by the veneer bending and cracking about the wall mid-height.
Table 4.9. Summary of Parametric Study – Slack Models

<table>
<thead>
<tr>
<th>Model ID</th>
<th>EQ Level</th>
<th>Ground Motion</th>
<th>Deflection Mode</th>
<th>Veneer Cracking</th>
<th>Tie Yielding Row Number</th>
<th>Tie Pullout Row Number</th>
<th>Collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC-16-S1-II (Slack)</td>
<td>DBE El Centro</td>
<td>I</td>
<td>No</td>
<td>x x x x x x x x x x x x x</td>
<td>1 2 3 4 5 6 7</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td></td>
<td>DBE Sylmar</td>
<td>I</td>
<td>No</td>
<td>x x x x x x x x x x x x x</td>
<td>Yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DBE Tarzana</td>
<td>I</td>
<td>No</td>
<td>x x x x x x x x x x x x x</td>
<td>Yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>MCE El Centro</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td></td>
<td>MCE Sylmar</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td></td>
<td>MCE Tarzana</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td>RC-16-S2-II (Slack)</td>
<td>DBE El Centro</td>
<td>II</td>
<td>Yes</td>
<td>x x x x x x</td>
<td>No</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DBE Sylmar</td>
<td>II</td>
<td>Yes</td>
<td>x x x x x x</td>
<td>No</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DBE Tarzana</td>
<td>II</td>
<td>Yes</td>
<td>x x x x x x x x</td>
<td>Yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>MCE El Centro</td>
<td>II</td>
<td>Yes</td>
<td>x x x x x x x x x x x x x</td>
<td>Yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>MCE Sylmar</td>
<td>II</td>
<td>Yes</td>
<td>x x x x</td>
<td>Yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>MCE Tarzana</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td></td>
</tr>
</tbody>
</table>

Note: Mode I defined by the veneer leaning away from the backing as rigid body and rotating about the wall base.
Mode II defined by the veneer bending and cracking about the wall mid-height.
Figure 4.1. Partial Nail Extraction

Figure 4.2. Nail Pullout

Figure 4.3. Partial Nail Extraction

Figure 4.4. Bed-Joint Cracking

Figure 4.5. Wood 7 after Tarzana 125%

Figure 4.6. Failure Pattern of Wood 7X
Figure 4.7. Extraction of the Top row of Ties

Figure 4.8. Failure Pattern of Wood 8

Figure 4.9. Uplift of Double Top Plate

Figure 4.10. Failure Pattern of Wood 9

Figure 4.11. Tie Pullout

Figure 4.12. Failure Pattern of Wood 10
Figure 4.13. Tie Pullout from Mortar Joint  
Figure 4.14. Anchor Pull-Through  

Figure 4.15. Deformed Tie Hole
Figure 4.16. Crack Patterns of the O-O-P Walls

(a) Solid Walls

(b) Walls with Window Opening

Figure 4.17. Wood 5 (D2mt) – T70

Figure 4.18. Wood 6 (D2mt) – T70
Figure 4.19. Wood 7 (D2et) – T70

Figure 4.20. Wood 8 (D4mt) – T70

Figure 4.21. Wood 9 (D3mt) – T70

Figure 4.22. Wood 10 (D2et) – T70

Figure 4.23. PGA vs. Peak Acceleration Response for the Tested Walls
Figure 4.24. Finite Element Discretization
Figure 4.25. *Concrete02* under Monotonic Compression

Figure 4.26. *Concrete02* under Monotonic Tension
Figure 4.27. Axial Behavior under Reversed Cyclic Loading (Corrugated Anchors)

Figure 4.28. Axial Behavior under Reversed Cyclic Loading (Rigid Anchors)
Figure 4.29. Displacement Profiles under Sylmar 40% (Wood 5)

Figure 4.30. Displacement Profiles under Sylmar 150% (Wood 5)

Figure 4.31. Displacement Profiles under Tarzana 100% (Wood 5)
Figure 4.32. Veneer Acceleration Profiles (Wood 5)

Figure 4.33. Anchor Force Distribution (Wood 5)
Figure 4.34. Displacement Profiles under Sylmar 40% (Wood 7)

Figure 4.35. Displacement Profiles under Tarzana 70% (Wood 7)
Figure 4.36. Veneer Acceleration Profiles (Wood 7)

Figure 4.37. Anchor Force Distribution (Wood 7)
Figure 4.38. Displacement Profiles under Sylmar 40% (Wood 9)

Figure 4.39. Displacement Profiles under Sylmar 80% (Wood 9)

Figure 4.40. Displacement Profiles under Tarzana 70% (Wood 9)
Figure 4.41. Veneer Acceleration Profiles (Wood 9)

Figure 4.42. Anchor Force Distribution (Wood 9)
Figure 4.43. Displacement Profiles under Sylmar 40% (Wood 10)

Figure 4.44. Displacement Profiles under Tarzana 70% (Wood 10)
Figure 4.45. Veneer Acceleration Profiles (Wood 10)

Figure 4.46. Anchor Force Distribution (Wood 10)
Figure 4.47. Wood 5 Tarzana 125%

Figure 4.48. Wood 7 Tarzana 125%

Figure 4.49. Wood 9 Tarzana 100%

Figure 4.50. Wood 10 Tarzana 125%
4.7 Acknowledgement of Publications

Parts of this chapter were published in the following journal papers or conference proceedings in which the author of this dissertation was either the primary or a co-author.


CHAPTER 5

IN-PLANE RESPONSE OF WALL SEGMENTS

5.1 Introduction

This chapter presents the results of the experiments conducted on the in-plane wall assemblies outlined in chapter 3. It starts with a detailed account of the experimental observations and damage progression followed by an evaluation of the displacement and acceleration responses of the walls. The chapter then presents the numerical modeling framework used to simulate the in-plane seismic behavior of masonry veneer. The experimental results of the shaking-table testing program were used for the purpose of model calibration and verification.

5.2 Experimental Observations

This section presents the qualitative observations of damage evolution in the wall specimens. Pictures of damaged specimens are provided with each specimen. A summary of the experimental observations is presented in Table 5.1.

5.2.1 Specimen Wood 1

No visible signs of damage were observed in the veneer or the wood-stud backing in this specimen through the entire testing sequence. Wall response was
characterized by veneer panel rocking. Post-test inspection revealed minor nail extraction for the veneer ties due to the prying action developed as a result of the different displacements of the veneer and the backing. Figure 5.1 shows one of the affected ties at the bottom row.

5.2.2 Specimen Wood 2

Sliding of the veneer was recorded in the Sylmar 80% shaking with no signs of damage in the veneer and the backing. After the first Tarzana 150% shaking, diagonal cracks at the top and the bottom of the window piers of the veneer (Figures 5.2 and 5.3) symmetrically formed but the wall remained integral and sustained the entire ground motion record safely. The veneer wall finally collapsed under the second Tarzana 150% shaking with the complete detachment of the veneer from the backing frame (Figure 5.4), which did not show any signs of distress. Tie pullout from the backing was the main cause of failure, along with minor cases of tie rupture as shown in Figures 5.5.

5.2.3 Specimen Wood 3

No visible signs of damage were observed in the veneer or the wood-stud backing in this specimen through the entire testing sequence. Similar to specimen Wood 1, the wall responded by veneer panel rocking. Minor nail extraction for the veneer ties at the bottom row as shown in Figure 5.6 was observed after the tests, the extraction was so small that it did not jeopardize the wall stability or resistance.
5.2.4 Specimen Wood 4

No signs of damage were observed in this wall through the entire test sequence which ended at the Maximum Considered Earthquake (MCE) level. The test was stopped after the MCE level and the wall was tested out-of-plane as specimen Wood 7.

5.2.5 Summary of the Experimental Observations

The experimental observations presented before are summarized in Table 5.1. All the walls performed satisfactorily under the representative Design Basis and the MCE ground motions. The walls seem to have a significant strength reserve after these levels. The behavior of the slender walls is characterized by veneer panel rocking. The squat walls responded by window pier rocking and sliding along the flashing at the wall base. Only Wood 2 collapsed under the repeated shaking of Tarzana 150%. This may be attributed to the higher flexibility of the window piers region in the wall and the fatigue failure of the veneer ties. A general comparison of the observed behavior with the out-of-plane (O-O-P) walls shows the walls are stronger in their in-plane direction compared to their out-of-plane direction, this is mainly due to the higher resistance to bending and the way the veneer anchors transfer the load between the masonry wall and the wood-stud backing, with tensile and compressive forces in the O-O-P walls and shear forces in the in-plane (I-P) walls.
5.3 Evaluation of the Experimental Results

5.3.1 Dynamic Properties of the Tested Walls

Acceleration time histories from the veneer and the backing under white-noise excitation were analyzed for all walls to determine their fundamental frequencies after they had been progressively damaged. Four levels were chosen for each wall, namely, the initial undamaged state, after Sylmar 80% (DBE), after Sylmar 125% (MCE) and Tarzana 125%. Table 5.2 presents the fundamental frequency as well as the progression of the fundamental frequency and its reduction after each one of the aforementioned tests. The table also provides the percentage reduction from the initial fundamental frequency after the DBE and the MCE and before collapse.

Table 5.2 shows that the I-P walls had an initial frequency approximately two times that of the O-O-P. The walls experienced a maximum reduction in fundamental frequency of 16.70% from the initial value for shaking up to the MCE. This reduction may be attributed to the partial nail extraction from the wood-studs as a result of the prying action generated by the sliding and rocking veneer panel. Specimen Wood 2 which failed under the repeated ground motion level of Tarzana 150% suffered a reduction of the fundamental frequency of 41.40% after the Tarzana 125% shaking. This significant reduction in stiffness is consistent with the observed diagonal cracking at this level and the severe pier rocking at the later ground motion levels.
5.3.2 Displacement and Acceleration Responses

Figure 5.7 presents a plot of the peak acceleration response at the top (accelerometer location) vs. the peak ground acceleration (PGA). The figure shows that the veneer and the backing had the same maximum acceleration at the top for all the specimens during the Sylmar motions. During the Tarzana motions, the acceleration of the wood backing became increasingly higher than that of the veneer for Wood 1 and Wood 2 as the peak ground acceleration increased. The plot also shows that the specimens experienced a dynamic amplification of about 1.00 for the Sylmar record, while it is much higher for the wood backing during the Tarzana motions.

Figure 5.8 presents a plot of the panel drift (%) vs. the peak ground acceleration (PGA). Panel drift is defined as the ratio of the lateral displacement measured by the top displacement transducer to the height of the transducer. In this case, the veneer had much larger drift than the wood backing during the Tarzana motions.

5.4 Analysis of Masonry Veneer Subjected to In-Plane Excitation

This section presents the development and calibration of the modeling scheme for the simulation of the dynamic behavior of the in-plane excited wall assemblies. This modeling scheme is also suitable for modeling wall panels under both in-plane and out-of-plane loads. Two modeling ideas are introduced for both the masonry wall and the wood-stud frame. The masonry panel uses a combination of beam-column and truss elements. The modeling of the wood-frame is a simple innovative approach that takes
advantage of the fact, outlined in the literature review (see Chapter 2), that all the wood shear wall nonlinearities are due to fastener deformations and that all the wood elements remain elastic. A combination of elastic beam-column elements, elastic truss bars, and nonlinear strut and tie model is used to model the entire shear wall assembly.

5.4.1 Description of Masonry Modeling

Figure 5.9 shows detailed schematics of the modeling procedure for the masonry. This modeling scheme provides a “Unit Masonry Element”, which is an assembly of beam and truss elements connected together, proportioned and assigned material models in order to have the overall behavior of the assembly the same as that of the masonry in the tested walls. The proposed model is capable of capturing the response of any unreinforced masonry wall under in-plane axial and shearing loads as well as out-of-plane loads.

The unit masonry element as shown in Figure 5.9b consists of two vertical and two horizontal nonlinear displacement-based beam-column elements, two diagonal nonlinear truss elements and four zero-length elements. The figure shows the node numbers to depict the element connectivity. For the purposes of this numerical study, the overall element dimensions are 203 mm x 203 mm (8 in. x 8 in.), the vertical and horizontal beam elements have fiber cross-sections with dimensions 102 mm x 92 mm (4 in. x 3.625 in.), the diagonal truss elements have a cross-section with dimensions
115 mm x 92 mm (4.525 in. x 3.625 in.). All the elements had the same thickness of 92 mm (3.625 in.) as the actual wall.

The vertical members had half the tributary width of a unit element so that it represents the exact strength and stiffness of the wall under out-of-plane loading while the diagonal trusses had no contribution to the strength and stiffness. The purpose of the diagonal elements is to provide the in-plane shear strength and stiffness of the unit element. The vertical and horizontal members were assigned a concrete material law with linear tension softening (*Concrete02* in OpenSEES), with masonry compressive strength of 27.5 MPa (4 ksi), strain at ultimate strength of 0.00285 and a tensile strength of 0.28 MPa (0.04 ksi). This would result in an initial stiffness of 19.30 GPa (2800 ksi) according to the Kent-Scott-Park formulation of the material model. This modulus was chosen to be consistent with the MSJC formula of the elastic modulus as $E_m=700f'_m$.

In order to avoid the excessive axial tensile strength introduced by the diagonal trusses into the unit masonry element, a concrete material model with zero tensile strength was used for the diagonal members (*Concrete01* in OpenSEES). The compressive strength of model (see Figure 5.9c) was chosen to be 0.70 MPa (0.10 ksi) and the strain at ultimate strength was set to be 0.0000114. This is to provide a shear strength that is equal to the axial tensile strength of the unit masonry element which is 0.28 MPa (0.04 ksi). These model properties result in an elastic modulus of 60.4 GPa (8771 ksi) for the diagonals. It is worth mentioning that for the purpose of calibration of the unit masonry element, a unit assembly was tested under tension and compression, and
shear loads. The boundary conditions and the loading of the unit element are shown in Figure 5.9d. In order to estimate the shear modulus of the unit masonry element based on the properties of the diagonals, a simplified calculation is conducted using the shown loading and boundary conditions in the aforementioned figure as follows:

\[ G = \frac{\tau}{\gamma} \]
\[ \tau = \frac{V}{A_n} \]
\[ \gamma = \frac{\Delta}{h} \]
\[ V = N \cos 45^\circ \]
\[ N = AE \frac{\delta}{L} \]
\[ \delta = \Delta \cos 45^\circ \]
\[ G = E \frac{A}{A_n} \cos^3 45^\circ \]

where,

\( G \) is the shear modulus of the unit masonry element
\( \tau \) is the shear stress in the unit masonry element
\( \gamma \) is the shear strain in the unit masonry element
\( V \) is the shear force
\( A_n \) is the bed joint area of the unit masonry element
\( \Delta \) is the shear deformation of the unit masonry element
\( h \) is the height of the unit masonry element
\( N \) is the axial force in the diagonal element
\( A \) is the cross-sectional area of the diagonal element
\( E \) is the elastic modulus of the diagonal element material
δ is the axial deformation of the diagonal element
L is the initial length of the diagonal element

This calculation resulted in a shear modulus of the unit masonry element of 12.1 GPa (1754 ksi). The high stiffness of the diagonal members led to an increase in the initial elastic modulus of the unit element in the axial direction to 32.80 GPa (4770 ksi) (determined numerically), which would however be reduced to the stresses of the vertical members after the diagonals are crushed. The crushing of diagonals at a low axial stress level sets a limitation on the use of such a model for walls with low axial load ratios, such as veneer walls; otherwise this loss of the diagonal resistance would lead to a loss of the assembly shear resistance.

In order to overcome the additional shear resistance provided by the frame action between the vertical and horizontal members, zero-length elements were used to connect the two as shown in Figure 5.9b. These elements have zero in-plane rotational stiffness making the horizontal beam element essentially a truss element for the in-plane purposes only, while having the full bending stiffness out-of-plane, allowing the element to resist two-way bending actions out-of-plane. Figures 5.9e and 5.9f show the behavior of the unit masonry element under axial compression and axial tension, respectively.
5.4.2 Validation of Masonry Element

In order to test the validity of the masonry element and its ability to capture the global behavior of a masonry wall, a masonry veneer panel was analyzed under a lateral in-plane load and compared with a simplified hand calculation of the strength of the wall. The value of the strain, in tension and shear, at which the stress drops to zero, was chosen to be 0.01, to enhance the solution robustness and avoid numerical convergence issues. The simplified estimation of strength assumed a linear distribution of the tensile stresses with the masonry tensile strength at the extreme tension fiber and zero at the neutral axis. The compressive stress distribution was modeled with an equivalent rectangular stress block. The deformed mesh is shown in Figure 5.10a. The pushover curve is shown in Figure 5.10b along with the simplified estimation of the peak strength. The results show a good correlation between the conservative estimate of the hand calculation and the result of the numerical model.

5.4.3 Description of Wood-Stud Frame Modeling

The modeling of the wood-stud frame follows a similar strategy as that utilized in the masonry modeling. The wood-stud frame unit elements as shown in Figure 5.11 are used to model the entire wood-stud frame assembly. The element consists of two vertical elastic beam elements that represent the vertical wood-studs with an elastic modulus of 10.3 GPa (1500 ksi). The element also has two horizontal truss elements representative of the tributary area of the OSB. In wood-stud frame construction, the assembled frames resist lateral loads, such as wind and earthquake induced forces, through in-plane racking
of the sheathing and transfer of the shear forces to the framing elements through the nailed connections. These fastener deformations are capable of dissipating large amounts of energy. The behavior of the nailed sheathing-to-wall connection is the one that governs the response of the whole shear wall assembly. The load-deformation response of timber shear walls under reversed cyclic loading is characterized by pinched hysteresis with both stiffness degradation and strength deterioration (Dolan and Madsen, 1992, Shenton et al., 1998, Folz and Filiatrault, 2001). In this model, all the nonlinearities that develop in the wood-stud frame due to fastener deformations are lumped into the diagonal truss elements shown in Figure 5.11b. The hysteretic model shown in Figure 5.11c is used for these elements. This model was calibrated versus the results of the racking tests of wood-stud frames with different aspect ratios conducted by Salenikovich (Salenikovich, 2000).

5.4.3.1 Validation of Wood-Stud Frame Element

Salenikovich (Salenikovich, 2000) tested a number of wood shear walls with aspect ratios (ratio of vertical to horizontal dimension) of 2:1, 1:1 and 2:3. The studs in these walls were 38 mm x 89 mm (nominal 2 in. x 4 in.) spruce-pine-fir (SPF) stud grade members. Studs were spaced at 406 mm (16 in.) with the exterior studs doubled in the case of fully anchored wall. All of the wall specimens were tested under monotonic and cyclic racking displacements applied to the top of the shear wall. Figure 5.12 shows the response for each one of the three shear walls under monotonic racking, while Figure 5.13 shows the stabilized envelope of the response for the cyclic racking tests compared to the numerical hysteretic response for one of the walls. A good match is achieved in the
monotonic racking tests. The numerical hysteretic response shows the ability of the model to capture the stiffness degradation and the strength deterioration.

5.4.4 **Hysteretic Models for Veneer Anchors**

Nonlinear truss elements were used to model the corrugated veneer anchors. The hysteretic model used is shown in Figure 5.14. This model is based on those developed and proposed by Zisi (Zisi, 2009) in terms of the shape of the backbone curve and the degree of pinching. The model is also based on the experiments conducted on corrugated sheet metal anchors by Johnson and McGinley (Johnson and McGinley, 2003). The figure presents the force-displacement relationship for the shear resistance of the corrugated anchor. Similar to the out-of-plane model, the model assumes a symmetric behavior in both directions for simplicity though experiments show an unsymmetrical behavior. The model is characterized by a pinched hysteresis but no strength degradation. The strengths and stiffnesses of the model were fine tuned to give the best match possible with the experimental results. The spring was oriented in the direction of horizontal shear resistance with no springs oriented vertically, which is reasonable for corrugated ties.

5.4.5 **Boundary Conditions and Time History Analyses**

The veneer at the base was supported on two kinds of springs. The first is an elasto-plastic (see Figure 5.15) spring having a very high stiffness and a strength consistent with a coefficient of friction of 0.50 multiplied by the wall weight. The spring was located at a bottom corner of the veneer panel. The second kind was a set of vertical
springs located at all the nodes of the veneer base. The spring had a stiff elastic compression branch and zero tensile strength and stiffness (see Figure 5.16); this was to simulate the rocking of the veneer panel under lateral loading. No restraint was applied to the top of the wood-stud frame since the steel restraint frame in the test was slotted, and allowed for in-plane displacements, and it only restrained the out-of-plane motion. The gravity load of the wall assembly was applied at the nodes of the veneer wall and the wood-stud frame models.

Time history analyses were conducted with a sequence of the table records similar to that experienced by the wall assemblies in the tests. Similar to the out-of-plane models, Rayleigh damping was used with the damping ratios specified to be 5% of the critical for the fundamental and the 12th modes in order to minimize the damping effect for the high-frequency axial modes that would be excited by veneer cracking. The constant-average-acceleration method was used to integrate the equations of motion for the system with an integration time step of 0.0025 second. It is worth mentioning that as part of the research project, some of the wall assemblies were tested quasi-statically under in-plane racking displacements applied to the top of the wood-stud frame. The experiments were conducted at North Carolina A&T University.

5.4.6 Results of the Analytical Study

Quasi-static and dynamic analyses were conducted with the wall models and the results were compared with the test results. Figure 5.17a shows a very good match of the
hysteretic response of the backing frame for the wall with window opening. The response of the veneer (Figure 5.17b) showed a very close match as well. The pinching of the hysteresis curve for veneer is not as severe as that of the actual wall specimen but the overall strength and stiffnesses are well captured by the model.

The dynamic analyses showed a very good match with the test results. The results of slender and squat walls are shown in Figure 5.18. The model seems to capture the sliding of the veneer panel as well as the wood-stud frame lateral drifts and differential movement between the veneer and the backing with a high degree of accuracy.

The numerical results for the slender wall match the experimental results very well through the entire shaking sequence. On the other hand, there was a discrepancy between the experimental and analytical veneer displacements under severe shaking (Tarzana 150%) for the wall with window opening. This discrepancy may be attributed to the inability of the model to capture a second-order phenomenon that is associated with the in-plane displacement of the veneer panel. Once the veneer starts to slide, the veneer anchors start to move from their initial perpendicular position. Since these anchors have a specific length, which is the width of the air gap, they tend to force the veneer panel to bend out-of-plane to preserve their length. This induces out-of-plane acceleration under dynamic shaking and out-of-plane bending moments on the bed-joints (weak axis), leading to the cracking of the mortar joints on top and bottom of the window piers (reduced bed joint area) for specimen Wood 2. This explanation was confirmed by the experimental observations from the quasi-static and dynamic tests. Peak out-of-plane
accelerations of 0.52g, 0.63g, 0.80g and 1.07g where recorded in Specimen Wood 2 under the tests of Sylmar 150%, Tarzana 70%, Tarzana 100% and Tarzana 125%, respectively. The model is unable to capture this phenomenon due to ignoring the large displacement effect. Furthermore, the tie model does not account for the interaction of the axial and shear responses of the veneer anchors.

5.4.7 Demand on Veneer Anchors

Figure 5.19 shows the anchor deformation distribution under Sylmar 150% ground shaking for specimen Wood 1 while Figure 5.20 shows the results for Wood 2 under Sylmar 150% and Tarzana 125%. The plots were generated at the instant of peak top veneer displacement. The numerical results showed that all the anchors at any specific row experienced the same deformation. This may be attributed entirely to the sliding response of Wood 2 and the combined sliding and rocking of the slender wall (Wood 1). Numerical results from specimen Wood 1 show that the bottom rows experienced higher anchor deformation than the top rows. The ties at the bottom rows are the first to extract and exhibit load degradation and transfer the forces to the higher levels. This may be visualized as a result of the fixation of the wood-stud frame at the base while it is free to move at the top with the veneer sliding under its own inertia (see Figure 5.21). This results in higher relative displacements at the lower rows and consequently higher anchor deformations. The same behavior is observed more severely in specimen Wood 2 which may be attributed to the dominant sliding response of this squat wall. Figure 5.22 shows the results of the quasi-static specimen with similar design to Wood 2. The plot shows an
Interestingly different distribution in which opposite but almost equal deformations are developed at the top and bottom rows of veneer anchors. This is mainly attributed to the fact that in the quasi-static test, the racking displacement was applied to the top of the wood-stud frame. In this setup, the wood-frame drags the veneer panel from the top, and since the panel is squat, it slides rather than rocks. This makes the frame lead at the top half of the wall assembly and lags at the bottom half due to its fixed base, thus producing the aforementioned distribution (see Figure 5.21). Analysis of specimen Wood 2 with the addition of a tributary roof mass at the top of the wood-stud frame showed similar distribution as that without roof mass for Sylmar 150%. Under stronger shaking, the distribution is characterized by less anchor deformation at the top of the wood frame and more at the base of the wall. This may be explained as a result of the increased deformation at the top of the wood-stud frame due to the added mass which reduces the relative displacements between the wood-stud frame and the sliding veneer (see Figure 5.21).

5.5 Summary and Conclusions

The shaking-table tests conducted on wall assemblies excited in their in-plane direction showed satisfactory performance under ground motions representative of the DBE and MCE for SDC D and E. Wall response is governed by sliding for squat segments and rocking for slender ones. The walls showed superior performance to the out-of-plane walls. The seismic performance and collapse mechanism of these walls were analyzed using computational models developed and calibrated with the results of the
quasi-static and shaking-table experiments conducted on wall assemblies. The model offers an innovative modeling scheme for wood-stud shear walls which was calibrated with the racking experiments conducted on walls with different aspect ratios. The calibrated models were able to simulate the in-plane dynamic response of the veneer and the backing. The model can also be used for out-of-plane analyses. A hysteretic model for veneer anchors subjected to shear loading was proposed and verified with the shaking-table test results.
Table 5.1. Summary of Experimental Observations

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Shaking Direction</th>
<th>Failure Motion</th>
<th>Max. PGA</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood 1</td>
<td>I-P</td>
<td>T 150%</td>
<td>3.07 g</td>
<td>Sliding, Rocking</td>
</tr>
<tr>
<td>Wood 2</td>
<td>I-P</td>
<td>T 150%</td>
<td>3.10 g</td>
<td>Pier Diagonal Cracks, Tie Pullout, Tie Rupture</td>
</tr>
<tr>
<td>Wood 3</td>
<td>I-P</td>
<td>T 150%</td>
<td>3.10 g</td>
<td>Sliding, Rocking</td>
</tr>
<tr>
<td>Wood 4</td>
<td>I-P</td>
<td>S 125%</td>
<td>1.35 g</td>
<td>No Visible Signs of Damage</td>
</tr>
</tbody>
</table>

Note: Under PGA, “S” denotes the Sylmar record and “T” denotes the Tarzana record with the acceleration scaling indicated. PGA values reflect actual table acceleration.

Table 5.2. Change of Fundamental Mode Frequency (Hz)

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Initial</th>
<th>Sylmar 80%</th>
<th>Sylmar 125%</th>
<th>Tarzana 125%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood 1D</td>
<td>15.92</td>
<td>15.33 (3.70%)</td>
<td>14.45 (9.20%)</td>
<td>No Data</td>
</tr>
<tr>
<td>Wood 2D</td>
<td>16.99</td>
<td>16.31 (4.00%)</td>
<td>14.16 (16.70%)</td>
<td>9.96 (41.40%)</td>
</tr>
<tr>
<td>Wood 3D</td>
<td>17.68</td>
<td>17.58 (0.56%)</td>
<td>17.29 (2.20%)</td>
<td>No Data</td>
</tr>
<tr>
<td>Wood 4D</td>
<td>25.30</td>
<td>25.10 (0.79%)</td>
<td>24.71 (2.33%)</td>
<td>No Data</td>
</tr>
</tbody>
</table>
Figure 5.1. Prying Action and Partial Nail Extraction (Wood 1)

Figure 5.2. Diagonal Cracks at Pier Bottom (Wood 2)

Figure 5.3. Diagonal Cracks at Pier Top (Wood 2)

Figure 5.4. Failure of Wood 2

Figure 5.5. Tie Pullout and Rupture

Figure 5.6. Minor Nail Extraction
Figure 5.7. PGA vs. Peak Response Acceleration

Figure 5.8. PGA vs. Panel Drift %
Figure 5.9. Equivalent Frame-Truss Model for Masonry (1 MPa = 0.145 ksi)
Figure 5.10. Masonry Model Validation (1 kN = 0.225 kip; 1 m = 39.4 in.)

Figure 5.11. Equivalent Frame-Truss Model for Wood-Stud Walls
(c) Hysteretic Model for the Nonlinear Truss Element

Figure 5.11. (Continued) Wood-Stud Frame Element Details

Figure 5.12. Response of Fully Anchored Wall under Monotonic Racking
Figure 5.13. Cyclic Racking Response vs. Experimental Stabilized Envelope

Figure 5.14. Hysteretic Model for the Horizontal Shear Strength of Veneer Anchors
Figure 5.15. Sliding Spring Model

Figure 5.16. Rocking Spring Model
Figure 5.17. Load Deflection Response for the Top of the Stud-Wall from Quasi-Static Test and Analytical Model (1 m = 39.4 in.)
Figure 5.18. Response History – Sylmar 150 % (for sensor location see Figure 3.3)
Figure 5.19. Tie Deformation Distribution - Wood 1

Figure 5.20. Tie Deformation Distribution - Wood 2
Figure 5.21. Mechanism of Tie Deformation Distribution

Figure 5.22. Tie Deformation Distribution - Wood 2 – Quasi-Static
5.6 Acknowledgement of Publications

Parts of this chapter were published in the following journal papers or conference proceedings in which the author of this dissertation was either the primary or a co-author.


CHAPTER 6

SEISMIC PERFORMANCE OF FULL-SCALE ONE-STORY
MASONRY VENEER WOOD-STUD HOUSE

6.1 Introduction

This chapter presents the results of the shaking-table tests conducted on a full-scale one-story masonry veneer house with wood-stud backing. The chapter provides a detailed description of the building in terms of design details, used materials, construction process, testing protocol, test setup, and instrumentation scheme. The experimental observations are summarized with a detailed account of the damage progression and veneer collapse mechanism. Analysis of the collected data including veneer and backing displacements and accelerations is presented and discussed in detail to give an insight into the system’s seismic behavior.

6.2 The Test Structure

6.2.1 General Description, Design and Construction of the Prototype Building

A 3D schematic of the prototype building is shown in Figure 6.1. The shaking direction is the east-west direction, the east and west walls were subjected to out-of-plane (O-O-P) excitation and will be designated the O-O-P walls, while the north and south walls were subjected to in-plane (I-P) excitation and hence will be termed the I-P walls.
The walls of the test structure were designed for Seismic Design Categories (SDC) D and E of ASCE 7-05 (ASCE, 2005) using the provisions of the Masonry Standards Joint Committee (MSJC, 2008), the International Residential Code for One and Two Family Dwellings (ICC, 2006), and the serviceability requirements of the Brick Industry Association (BIA, 2002) Technical Notes. The walls were constructed by professional masons in a manner conforming to common practice. The design details of the test structure are shown in Figures 6.2 and 6.3.

The north and south walls were 6.33 m (20.7 ft) long and 2.47 m (8 ft) high, each having a 1.22-m x 1.22-m (4 ft x 4 ft) window and a 1.22-m x 2.00-m (4 ft x 6½ ft) door. The veneer on these walls was attached to the wood studs with 22-gage (0.80-mm) corrugated sheet metal ties using electro-galvanized 8d (10-1/4 gage, 2.87-mm diameter, and 64-mm long) nails. In the north wall, the ties were mechanically attached to 9-gage (4-mm) wire joint reinforcement embedded in bed-joints through a built-in hook. On the other hand, the south wall did not have joint reinforcement. The ties were spaced at 406 mm (16 in.) horizontally and 400 mm (15 3/4 in.) vertically (6 brick courses) in both walls. The top row of veneer ties was located at 206 mm (8 in.) (3 brick courses) from the veneer top. Additional veneer anchors were provided around the perimeter of the window and the door as shown in the design details in Figure 6.2.

The details of the east wall are shown in Figure 6.3. The wall was 6.33 m (20.7 ft) long and 2.47 m (8 ft) high and had two 1.22 m x 1.22 m (4 ft x 4 ft) windows. The veneer on this wall was attached to the wood studs with the 16-gage (1.60-mm) rigid ties.
using #10 (nominal shank diameter of 4.76 mm and 64 mm long) screws. Joint reinforcement was used and attached to the ties through a built-in hook. The ties were spaced at 406 mm (16 in.) horizontally and 600 mm (23 5/8 in.) (9 brick courses) vertically. The top row of ties was located at 260 mm (10 in.) (4 brick courses) from the veneer top. Similar to the I-P walls, additional veneer anchors were provided around the perimeter of the windows. This wall was connected at the edges to the north and south walls. The west wall was 6.12 m (20 ft) long and 2.47 m (8 ft) high and had two 1.22 m x 1.22 m (4 ft x 4 ft) windows. The veneer on these walls was attached to the wood studs with 22-gage (0.80-mm) corrugated sheet metal tie using electro-galvanized 8d nails (10-1/4 gage, 2.87-mm diameter, and 64-mm long). This wall did not have joint reinforcement. The ties were spaced at 406 mm (16 in.) horizontally and 400 mm (15 3/4 in.) (6 brick courses) vertically. The top row of ties was located at 194 mm (7 5/8 in.) (3 brick courses) from the veneer top. Once again, additional veneer anchors were provided around the perimeter of the windows. This wall was separated from the north and south walls with vertical joints. This is intended to simulate the veneer on the external face of a building unit that has a common wall with the neighboring buildings.

6.2.2 Wood-Stud Frames

The wood-stud backing walls were constructed with 38-mm x 89-mm (nominal 2 x 4-in.) No.2 Douglas fir studs according to the IRC specifications. The frames were separated from each other with no connection around the building corners. The only means of connection was provided through the roof diaphragm (Figure 6.4). The vertical
studs in each frame were spaced at 406 mm (16 in.) on center. A double top plate and a sole plate were nailed to the vertical studs using two 16d end nails. On the exterior (veneer) side of the wood frame, a 12-mm (1/2-in.) thick oriented strand board (OSB) was nailed to the edge studs using 6d nails spaced at 152 mm (6 in.) on center, and to the interior studs at 304 mm (12 in.) on center. Twelve-mm (1/2 in.) thick gypsum wall board was attached to the interior of the wood frame using #8 drywall screws spaced at 203 mm (8 in.) on center along the studs. The sole plate was connected to the reinforced concrete foundation using 16-mm (5/8-in.) Simpson SSTB16 anchor bolt pre-embedded in the foundation. In addition, the exterior double vertical studs of the designated wood shear walls were connected to the foundation using Simpson HDU4-SDS2.5 hold-downs (Figure 6.5). The hold-downs were connected to the vertical studs and to the foundation using 16-mm (5/8-in.) Simpson SSTB16 anchor bolts pre-embedded in the concrete foundation, providing approximately 1 kN (225 lb.) of hold-down force at each location. These segmental shear walls and hold-down configurations met or exceeded the bracing requirements for one or two-story wood-framed structures defined in the IRC.

6.2.3 Wood-Joist Roof Diaphragm

The roof diaphragm was constructed with 38-mm x 292-mm (nominal 2 x 12-in.) No.2 Douglas fir joists. The joists were spaced at 406 mm (16 in.) on center and spanned along the north-south direction and were vertically supported on the north and south wood-stud walls as shown in Figure 6.6. The joists were sheathed on the exterior by 12-mm (1/2-in.) thick oriented strand board (OSB) nailed to the edge joists using 6d nails
spaced at 152 mm (6 in.) on center, and to the interior joists at 304 mm (12 in.) on center. Twelve-mm (1/2-in.) thick gypsum wallboard was attached to the interior side of the joist to form a ceiling using #8 drywall screws spaced at 203 mm (8 in.) on center. A 38-mm x 292-mm (nominal 2 x 12-in.) rim joist was nailed to the end of the roof joists using four 16d nails. In addition, each joist was attached to the rim joist with two Simpson A34 clip angles to ensure that the full shear capacity of the diaphragm could be transferred between the joists and the shear wall segments (Figure 6.7). Furthermore, to provide adequate shear transfer to each of the segmental shear wall elements, Simpson LTP4 plates were also used to connect the outer side of the rim joist to the side of the double top plate every 609 mm (2 ft) along the building perimeter (Figure 6.8).

The east and west rim joists were connected to the adjacent roof joists, with 38-mm x 89-mm (nominal 2 x 4-in.) blocking spaced at 406 mm (16 in.) on center. These wood blocking elements are not required by the IRC provisions but are commonly used to support the free edge of the ceiling gypsum wallboard. This blocking has the unintended benefit of transferring out-of-plane wall reaction forces to the diaphragm without localized failure at the diaphragm edges. The 38-mm x 89-mm (nominal 2 x 4-in.) blockings were end nailed to the joists using two 16d nails. Since the blocking end nailing connection is weak in tension, diagonal metal braces were introduced between the bottom of the rim joists on the east and west sides and the top of the next interior joist in the roof system (Figure 6.9) to provide a load path for the tensile forces and guard against a premature connection failure during the testing. These Simpson LTB-20 metal strap braces are typically used to provide bridging for wood floor and roof framing and were
proved at a 609 mm (2 ft) spacing in an effort to transfer the lateral load from the top of the out of plane walls to the level of the roof sheeting and to prevent the rotation of the rim joist. It should be noted that these braces are not specifically required by the IRC provisions but were provided to reduce the possibility of premature failure of this connection.

6.2.4 Clay Masonry Veneer

The clay masonry veneer wythe was separated from the wood-stud backing wall by a specified 25-mm (1-in.) air gap. The veneer was vertically supported on a concrete slab covered by 30-mil (0.76 mm) flashing made of Ethylene Propylene Diene Monomer (EPDM). The flashing was stapled to the wood-stud backing near the base. Weep holes were introduced at the bottom veneer course by removing the mortar from the head joints at intervals less than 0.90 m (3 ft). The masonry veneer wythe was constructed in running bond using nominal 101-mm x 68-mm x 203-mm (4 x 2-5/8 x 8-in.) standard modular clay masonry units conforming to ASTM Standard C 216 (ASTM, 2006). Type N masonry cement mortar was used conforming to the single-bag proportion specification of ASTM C 270 (ASTM, 2006) with a volumetric masonry cement-to-sand ratio of 1:3. The structure was cured for at least 28 days before testing. Material tests showed that the masonry had an average compressive strength of 20 MPa (3 ksi) and average flexural tensile strength of (118 psi) based on the bond wrench test. The type N masonry cement mortar had an average compressive strength of 4.10 MPa (0.60 ksi).
Vertical movement joints were placed at both corners of the west wall. These movement joints were formed by stopping the west veneer approximately 10 mm (3/8 in) from the interior face of the north and south walls and allowing independent movement of the veneer on these three walls. This detail is typical of multifamily construction where short panels of veneer are used. Flashing and steel angle lintels were provided over all door and window openings and the window sills were formed using partial brick units in a rowlock configuration on top of flashing.

### 6.2.5 Veneer Ties and Joint Reinforcement

The veneer on the west, north, and south walls was attached to the wood studs with 22-gage (0.80-mm) corrugated sheet metal ties, as shown in Chapter 3, using electro-galvanized 8d (10-1/4 gage, 2.87-mm diameter, 64-mm long) nails. In the north wall, the ties were mechanically attached to 9-gage (4-mm) wire joint reinforcement. The veneer on the east wall was attached to the wood studs with 16-gage (1.60-mm) rigid ties similar to those used in the wall assembly tests but using #10 screws (nominal shank diameter of 4.76 mm, 64 mm long), and were mechanically attached to 9-gage wire joint reinforcement in the veneer.

### 6.3 Instrumentation Schemes

Displacement transducers and accelerometers were used to monitor the veneer and backing displacements with respect to the table and the absolute accelerations at the positions of the veneer anchors. The displacement transducers were mounted on steel
reference frames located on the outside (east and west) and the inside of the building. The steel reference frames had very small mass and were stiff enough that their deformations were negligible as compared to those of the walls. Figure 6.10 shows the instrumentation scheme for the building. The instruments were oriented parallel to the direction of the table motion, i.e., perpendicular to the O-O-P walls and parallel to the I-P walls. The possible sliding of a brick wall along its base was monitored with linear potentiometers. Prior to the anticipated collapse run (Sylmar 200%) of the shaking table, most of the instruments on the veneer side of the system were removed to avoid possible damage from falling veneer.

6.4 Test Setup

The structure was tested on the outdoor shaking table at the University of California at San Diego. The reinforced concrete foundation was tied to the table by post-tensioned steel rods. Figure 6.10 shows the building specimen and sensor support systems on the shaking table. The table has plan dimensions of 7.60 m x 12.20 m (25 ft x 40 ft) and is capable of carrying a maximum payload of 20 MN (4500 kips). The two hydraulic actuators controlling the table motion have a displacement stroke of ± 0.75 m (29.5 in.) and are capable of driving the table to a maximum velocity of 1.80 m/sec (70 in. /sec). Figure 6.11 shows the completed structure on the shaking table on the testing day.
6.5 Testing Protocols

Shaking-table tests were conducted using the Sylmar – 6-story County Hospital Parking Lot record (360-degree component) from the 1994 Northridge (California) Earthquake. The record was obtained from the Center for Engineering Strong Motion Data (www.strongmotioncenter.org). The record is identical to the one used in the wall tests as shown again for convenience in Figure 6.12. The structure was subjected to a sequence of Sylmar ground motion histories with the acceleration scaled to different levels. Table 6.1 summarizes the test sequence with the actual Peak Ground Acceleration (PGA) measured from the table. The acceleration scaling used design basis response spectra that are representative of ASCE 7-05 (ASCE, 2005) SDC D and E as a reference. The upper-bound spectrum for Seismic Design Category (SDC) E, lower-bound spectrum for SDC D, and intermediate spectra for SDC D and E shown in the figure are determined from the seismic maps of California. The last two are more or less the mean of the upper and lower-bound spectral curves and are representative of the seismic intensity in many areas in California. As shown in Figure 6.13, the Sylmar response spectra with 80% acceleration scale factor matches the spectral coordinate of the representative design spectra at the estimated initial fundamental period of the structure of 0.10 second. The Sylmar record was selected because it has a long-duration acceleration pulse, which is characteristic of a near-fault ground motion, and its spectral shape, as shown in Figure 6.13, peaks between 0.2 and 0.5 sec. This spectral characteristic would impose a severe demand on the structure as its period increased with damage. White-noise excitation was used in between the earthquake runs to assess the dynamic properties of the wall system.
and to track the progression of damage. The white noise had a root-mean-squared acceleration of 0.03g and swept a frequency range of 1 – 33 Hz.

### 6.6 Experimental Observations

This section presents the qualitative observations of damage evolution in the test. Descriptions of damage for various levels of shaking are presented with pictures.

#### 6.6.1 Behavior under Low Level Ground Motions

Except for the extraction of the veneer anchors in the west wall, the building sustained the Sylmar ground motions scaled to 25% and 50% with no visible signs of damage. There was no evidence of obvious nail extraction or tie damage on the east (out-of-plane) wall, or veneer sliding in the north and south (in-plane) walls. However, nails in the top row of veneer anchors of the west wall were extracted from the wood studs by as much as 11.4 mm (0.45 in.) during Sylmar 50%. With the exception of the nail extraction in the west wall, the performance of this wall system is consistent with the expectation for such levels of ground shaking.

#### 6.6.2 Behavior under the Design Basis and Maximum Considered Earthquakes

During Sylmar 80% (DBE), the top half of the veneer on the west wall (fastened with corrugated ties and 8d nails) peeled off the structure (See Figure 6.14). This veneer
failure was caused by complete nail extraction from the wood studs (See Figure 6.15). Based on the observations from the tests conducted earlier on individual wall segments and the analytical studies conducted prior to the building tests (Chapter 4), this mode of failure was not expected until much higher levels of ground shaking. The rest of the structure, in contrast, showed no signs of distress under this level of shaking. During Sylmar 120% (MCE), the rest of the veneer on the West Wall was detached from the backing wall as the remaining nails were extracted from the wooden studs. The rest of the structure showed no signs of obvious distress under this level of shaking.

### 6.6.3 Behavior under Severe Ground Shaking

No major additional damage was observed during testing with Sylmar 150%. The east wall (which had rigid ties) responded in the out-of-plane direction, with noticeable two-way bending but had no visible cracks. Relative motion between the rim joist and the double top plate was observed on the east wall, coupled with minor deformation of the metal plates connecting the outside of the rim joist and the top plate.

During the first test using Sylmar 200%, the top portion of the veneer on the east wall collapsed (Figure 6.16). Failure was characterized by bed-joint cracking and pullout of most of the anchors from the mortar joints (Figure 6.17). A few anchors were detached when screw heads were pulled through deformed holes in the rigid ties. Diagonal cracking was also observed at the top and bottom of the smaller veneer segments on the north and south walls; due to rocking of those segments (Figures 6.18 and 6.19). On the
east wall, joint reinforcement attached to the top row of ties was pulled out of the mortar joints and was not able to hold the dislodged pieces of veneer together (Figure 6.20).

During the second test under Sylmar 200%, more of the veneer on the east wall peeled off. The veneer over the entire south wall was loosen from the backing frame and collapsed out-of-plane as shown in Figure 6.21. The veneer collapse was initiated by the collapse of the lintel and veneer sections above the door which were already diagonally cracked on either sides of the door during the previous test. After the lintel collapse, all the veneer anchors were pried out of the backing after experiencing severe cyclic shearing deformations (Figures 6.22 and 6.23). The veneer on the north wall was on the verge of collapse. It suffered excessive sliding (Figure 6.24) and most of the anchors were pried out of the wood-stud backing, due to in-plane shearing deformations, as shown in Figures 6.25 and 6.26. The lintel beam over the door opening also suffered significant diagonal cracking similar to that observed on the south wall. The joint reinforcement did not seem to arrest the propagated cracks (Figure 6.27).

During this level of excitation, the end nails in the blockings next to the east wall were pulled out as shown in Figure 6.28. The magnitude of the pullout along the length of the east side suggests the bowing of the rim joist due to the two-way bending action of the entire east wall (veneer and backing). After the test, the gypsum board of the ceiling at this region was removed, and visual inspection showed a significant separation
between the blockings and the rim joist as well as pullout of the metal braces connecting the rim joist to the first interior joist (Figure 6.29).

### 6.7 Analysis of the Experimental Results

Acceleration time histories of the veneer and backing walls subjected to the white-noise excitation applied before the first and after each earthquake ground motion test were analyzed. The results shown in table 6.2 show that the initial fundamental frequency of the west wall (7.85 Hz) was significantly different from that of the east wall (9.90 Hz). This striking difference may be attributed to the discontinuity in the veneer around the northwest and southwest corners of the building. The initial slack in the corrugated ties may have also contributed to this difference in frequency. The results also show that the west wall experienced a decrease in fundamental frequency to below 6 Hz right before failure. The east wall of the structure showed no signs of distress until the Sylmar 200% test, with failure again occurring after a decrease in fundamental frequency to about 6 Hz. The north and south walls and the roof showed no significant changes in fundamental frequency during the entire test sequence.

Figure 6.30 presents the plots of the average peak absolute roof acceleration and the average dynamic amplification (ratio of the average peak roof acceleration to the peak base acceleration) versus the peak ground acceleration (PGA). The latter plot shows that the structural response experienced a dynamic amplification in the range of 1.2 to 2.5 with a jump occurring at a PGA that corresponds to the Sylmar 80% test, probably due to
the sudden shedding of the veneer mass caused by the out-of-plane collapse of the west veneer. Similar jumps can be observed for the two Sylmar 200% runs, during which the east wall veneer collapsed. It is worth mentioning that the recorded accelerations by all the accelerometers on the roof were almost identical. Displacement transducer DR02 (see Figure 6.10d) showed a peak displacement of 2 mm (0.08 in.) under the second run of Sylmar 200%, revealing that the diaphragm was practically rigid in its plane.

Figure 6.31 presents a plot of the absolute base sliding of the veneer in the north and south walls versus PGA. This figure shows that no veneer sliding occurred until a PGA of 1.20 g, which corresponds to the Sylmar 120% test. It also shows that the north and south veneer walls behaved similarly under in-plane shaking but the south wall slid more. Both walls showed less sliding at the smaller west segments due to the flange effect provided by their connection to the east wall, and also to their tendency to rock.

The degree of rocking motion of the veneer walls is indicated by the net in-plane veneer displacement as determined by the difference between the top displacement of the veneer and the base sliding. Figure 6.32 shows the variation of this quantity with PGA. The plots show no significant motion until the Sylmar 120% test (MCE level). The small veneer segment on the south wall exhibited much more significant rocking than the other segments. Figure 6.33 shows the absolute relative in-plane top displacement between the veneer and the backing. Once again, the plots show no major relative displacement till
Sylmar 120% with the small veneer segment on the south wall exhibiting the most severe relative displacement.

Figures 6.34 and 6.35 present the absolute peak out-of-plane accelerations at the top of the veneer and the backing plotted against the PGA for the west and east walls, respectively. The figures show that, under Sylmar 50%, the west veneer, which had corrugated ties, experienced a peak acceleration of 2.07g, about twice that experienced by the east wall, which had rigid ties. This high acceleration corresponds to a local dynamic amplification of about 4.2 for the west veneer. This higher acceleration can be attributed to the initial slack in the corrugated ties, which induced a shock to the veneer as they got straightened. The discontinuous corners at the edges of the west wall provided no restraint for the wall motion.

Prior to the next shaking (Sylmar 80%), visual inspection of the top row of ties in the west wall revealed partial extraction of the nails from the wood-stud backing. The top half of the west veneer collapsed during Sylmar 80% with a peak out-of-plane acceleration of about 6.5 g near the top of the wood wall. The figures show that the east wall experienced more than twice the maximum acceleration recorded on the west wall before failure. This can be attributed to 1) the rigid ties used on this wall; 2) the use of screws, which had significantly higher extraction capacities than nails; and 3) the bonded corners, which restrained the wall segments adjacent to the window from out-of-plane motion.
Figure 6.36 presents the absolute peak in-plane top displacements of the backing wall. The plots for different positions along the shorter and longer segments of the wall show a linear relation up to Sylmar 120% (MCE level), and the increase in displacement is more rapid afterwards. This indicates a reduction of the structural stiffness. This is likely attributable to the initiation of the in-plane veneer sliding, as well as the softening of the wood-stud shear walls, which exhibited some cracking and crushing of the gypsum wallboard at the corners of the doors and windows.

Figure 6.37 shows a plot of the absolute peak out-of-plane displacement of the east wall backing versus the PGA. The figure clearly shows that there was more deformation towards the center of the wall and less near the edges. This is consistent with the two-way bending deformation visually observed in the test and the bowing of the rim joist as evidenced by the differential nail extraction from the blockings (see Figure 6.28). Much of this variation in out-of-plane deformation can be attributed to the restraint provided by the continuous corners of the veneer at the northeast and northwest corners of the building. Figure 6.38 shows the maximum difference between the out-of-plane displacements of the veneer and the backing (maximum tie elongation or extraction) at the same locations. To determine the effect of the in-plane veneer segments on the behavior of the wood-stud structure, a relative velocity index, which is defined as \((Velocity_{Wood} - Velocity_{Veneer}) \times \text{sign}(Velocity_{Wood})\), is used. A positive value of the index means that the wood wall was moving faster than the veneer and, thereby, the veneer was providing a restraint, and a negative value means that the veneer was driving the wood
wall. Figure 6.39 shows the time histories of the index for the short and long veneer segments on the south wall. Figures 6.39a and 6.39b show the results obtained at Location 2 (see Figure 6.36) of the long wall segment for the Sylmar 80% test (prior to veneer sliding) and Sylmar 150% test (after veneer sliding). The plots suggest that the long veneer segment was restraining the wood wall and helped to reduce the drift of the wood-stud shear wall for both ground motions. Figures 6.39c and 6.39d show similar plots for the short segments of veneer. Under Sylmar 80%, the short veneer segment restrained the wood wall. However, under the higher ground motion level, the veneer began to rock and became a driving force. This can be partially attributed to the continuous veneer corners that transferred the inertia force from the out-of-plane wall (east wall) to the short veneer segment. This was not observed on the longer segments because they were constructed discontinuous from the west veneer and also had less tendency to rock.

6.8 Design Implications based on Test Results

The test results clearly underscore the vulnerability of corrugated ties to nail extraction from wood studs, which caused the premature failure of the west veneer at DBE. The tests presented in the previous chapter on the out-of-plane performance of wall segments showed a much higher nail extraction capacity, which met the code expectation, than what was observed in the building test. This difference can be attributed to the higher moisture content of the wood studs in the present study. The test structure considered here was built and tested outdoor during rainy days in December and January,
while the wall segments in the prior study were built and tested in dry and hot summer months. It is well known that the extraction capacity of nails is highly dependent on the moisture content of the wood. Based on results of the tests presented in this chapter, it is highly recommended that corrugated ties should be attached to wood studs with either ring-shank nails or screws, which provide a higher extraction capacity with less sensitivity to the moisture content of wood. This is supported by the striking difference in performance of the east and west walls of the tested structure. The east wall, having screw fasteners for the rigid ties, sustained more than twice the ground motion level before veneer collapsed when compared to the west wall with nailed anchors. Furthermore, the straightening of possible slack in corrugated ties could introduce a shock effect during dynamic shaking and, thereby, amplify the tie forces.

No ovaling of holes in OSB and gypsum wallboard was observed. This implies that the wood frame did not experience much in-plane deformation. There is strong evidence from experimental data that this is a result of the restraint provided by the in-plane veneer. The interior gypsum wallboard did show some local distress, including local crushing at panel edges, and local cracking around door and window openings in the in-plane walls, but this only happened under levels of excitation in excess of the MCE. Hence, in-plane masonry veneer should be considered as an integral part of the lateral load resisting and energy dissipating mechanisms, and should not be treated as added mass only.
The north and south walls provided the opportunity for direct comparison of the effect of joint reinforcement. The veneer on the south wall, which did not have joint reinforcement, did fail first, but the veneer on the north wall, which had joint reinforcement, was at the state of eminent collapse under the same ground motion level. In spite of the fact that the short veneer segment on the south wall rocked more, the north and south walls had similar cracking patterns with no significant difference in the ultimate performance that might be attributed to joint reinforcement. Also the joint reinforcement on the east wall did not appear to be able to keep the veneer pieces together once failure occurred.

One aspect of observed performance which merits further studies is the behavior of wood diaphragms. The wood frame was designed and constructed in accordance with current prescriptive requirements for SDC D. In addition, metal straps and blocking elements were added. The blocking elements were in conformance to common construction practice. These elements are not required by code but enhanced the roof system behavior. The OSB sheathing on the roof top showed little deformation in the tests, and the most vulnerable roof components were the rim joists perpendicular to the shaking direction. Even with the aforementioned strengthening, considerable amounts of nail extraction were observed at the transverse blocking elements connected to the rim joist on the east side of the structure. Hence, this detail merits further attention.
6.9 Summary and Conclusions

A single-story, full-scale, wood-framed structure with clay masonry veneer was tested on a shaking table to evaluate its seismic performance. The structure had plan dimensions of 6.33 m x 6.33 m (20.7 ft x 20.7 ft) and had 2.44-m (8-ft) tall walls. The structure was designed and constructed according to current US code provisions. Except for the west wall of the structure, both the wood frame and the masonry veneer performed well under severe ground motions, which far exceeded a maximum considered earthquake representative (intermediate) of Seismic Design Category D according to ASCE 7-05 (ASCE, 2005).

The premature collapse of the west veneer at a design level earthquake can be attributed to the low extraction capacity of the nails attaching the corrugated ties to the wood-stud backing. The high moisture content in the wood is probably the main cause of this low extraction capacity. Hence, it is recommended that either ring-shank nails or screws be used to attach veneer ties as for the case of the rigid ties considered here, which were attached by screws and performed well. There is strong evidence from the test data that the veneer walls parallel to the direction of shaking helped to restrain the motion of the wood structure. However, short veneer walls with a high height-to-length ratio tended to rock and induce additional seismic force on the wood structure under severe excitation.

There is no evidence that joint reinforcement in veneer can improve the seismic performance and safety of such structures. Its benefit is negligible and cannot be relied upon. The detailing of the wood roof diaphragm requires special attention in
consideration of the inertia force of the veneer that can be transmitted through the top plate of the wood-stud wall to the rim joist when subjected to out-of-plane shaking. Further experimental and analytical studies are needed to understand the performance of these structures with different wall heights and subjected to bi-directional ground motions.

### Table 6.1. Test Sequence for the One-Story House

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<tr>
<th>Step</th>
<th>Level</th>
<th>PGA</th>
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<tr>
<td>1</td>
<td>Sylmar 25 %</td>
<td>0.23 g</td>
</tr>
<tr>
<td>2</td>
<td>Sylmar 50 %</td>
<td>0.49 g</td>
</tr>
<tr>
<td>3</td>
<td>Sylmar 80 %</td>
<td>0.71 g</td>
</tr>
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<td>4</td>
<td>Sylmar 120 %</td>
<td>1.20 g</td>
</tr>
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<td>6</td>
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</tr>
<tr>
<td>7</td>
<td>Sylmar 200 %</td>
<td>2.00 g</td>
</tr>
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</table>

### Table 6.2. Evolution of the Fundamental Mode Frequency (Hz)

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<th>Specimen ID</th>
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<th>S80</th>
<th>S120</th>
<th>S150</th>
<th>S200</th>
</tr>
</thead>
<tbody>
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<td>9.90</td>
<td>9.90</td>
<td>9.90</td>
<td>9.38</td>
<td>7.68</td>
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<td>9.90</td>
<td>9.90</td>
<td>9.38</td>
<td>7.68</td>
<td>6.21</td>
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<td>West Wall Veneer (AWV08)</td>
<td>7.85</td>
<td>5.57</td>
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<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td>West Wall Wood (AWW08)</td>
<td>7.85</td>
<td>5.57</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
</tbody>
</table>
Figure 6.1. One-Story Test Building with Wood-Stud Walls and Masonry Veneer
(1 inch = 25.4 mm)
Figure 6.2. Design Details of North and South Walls
(dimensions in mm, 1 m = 39.4 in.)
Figure 6.3. Design Details of East Wall (west wall had similar design but had different vertical tie spacing and had discontinuous veneer corners; dimensions in mm, 1 m = 39.4 in)
Figure 6.4. Erection of Separate Frames

Figure 6.5. Hold-Downs

Figure 6.6. Orientation of Roof Joists

Figure 6.7. Joist to Rim Connection

Figure 6.8. Rim to Top Plate Connection

Figure 6.9. Blocking and Bracing
Sensor Nomenclature Scheme

(b) Out-of-Plane Walls (East and West)

(d) In-Plane Walls (North and South)

Figure 6.10. Locations of Displacement Transducers and Accelerometers in the Structure (Numbers without designation represent sensor locations in both veneer and wood walls)
Figure 6.10. (Continued) Locations of Displacement Transducers and Accelerometers in the Structure (Numbers without designation represent sensor locations in both veneer and wood walls)

Figure 6.11. Shaking-Table Test Setup
Figure 6.12. Original Sylmar Ground Motion Record

Figure 6.13. Pseudo-Acceleration Response Spectra (5\% Damping)
Figure 6.14. Collapse of Veneer on West Wall

Figure 6.15. Typical Nail Extraction from the West Wall Wood-Studs

Figure 6.16. Collapse of Veneer on East Wall (1 x Sylmar 200 %)

Figure 6.17. Anchor Pullout (Mortar Joints) and Pull-Through

Figure 6.18. Diagonal Cracking at the South-East Top Corner

Figure 6.19. Pier Diagonal Cracking (South Wall)
Figure 6.20. Joint Reinforcement from the East Wall

Figure 6.21. Collapse of Veneer on the South Wall

Figure 6.22. Typical Veneer Anchor from the Collapsed South Wall

Figure 6.23. South Wall Veneer Anchor Subjected to Severe Shear Deformations

Figure 6.24. Sliding of the In-Plane Veneer

Figure 6.25. Nail Extraction by In-Plane Sliding (North Wall)
Figure 6.26. North Wall on the verge of Collapse

Figure 6.27. Incipient Collapse of the Lintel (North Wall)

Figure 6.28. Extraction of End Nails from Blockings (East Wall)

Figure 6.29. Extraction of Diagonal Brace (Roof - East Side)
Figure 6.30. Average Peak Roof Acceleration and Dynamic Amplification versus PGA

Figure 6.31. Peak In-Plane Veneer Sliding versus PGA (1 m = 39.4 in.)
Figure 6.32. Top In-Plane Veneer Displacements Subtracted by Base Sliding versus PGA (1 m = 39.4 in.)

Figure 6.33. Relative Top In-Plane Displacements between Veneer and Wood Wall versus PGA (1 m = 39.4 in.)
Figure 6.34. Peak Out-of-Plane Accelerations versus PGA for West Wall

Figure 6.35. Peak Out-of-Plane Accelerations versus PGA for East Wall
Figure 6.36. Peak In-Plane Wood Wall Displacements versus PGA (1 m = 39.4 in.)

Figure 6.37. Peak Out-of-Plane Displacements versus PGA for East Wood Wall (1 m = 39.4 in.)
Figure 6.38. Peak Axial Tie Deformations versus PGA for East Wall (1 m = 39.4 in.)

Figure 6.39. Relative Velocity Index Time Histories (displacement transducer locations identified in Figure 6.36; 1 m = 39.4 in.)
6.10 Acknowledgement of Publications

Parts of this chapter were published in the following journal papers or conference proceedings in which the author of this dissertation was either the primary or a co-author.

CHAPTER 7

PARAMETRIC STUDIES ON SYSTEM PERFORMANCE AND DESIGN RECOMMENDATIONS

7.1 Summary

This chapter presents an extension of the numerical study conducted in the Chapters 4 and 5. The developed models are used to assess the behavior of untested masonry veneer building conditions. The parametric studies first investigate the influence of masonry veneer, oriented parallel to the direction of motion, on the seismic performance of wood-stud shear walls with different aspect ratios. The second study involves the analyses of two-story masonry veneer under out-of-plane seismic excitation. In addition, a numerical simulation of the prototype building was conducted using a simplified three-dimensional model. The outcome of these studies along with the information obtained from the experimental and analytical studies are used to formulate a set of design recommendations for the masonry veneer in wood-stud buildings.

7.2 Influence of In-Plane Veneer on the Seismic Performance of Wood Shear Walls

7.2.1 Outline of Analytical Study

The test results presented in the previous chapter showed that masonry veneer oriented parallel to the direction of shaking can have a beneficial restraining effect on the
wood-stud frame. In order to further investigate the nature of this influence, a numerical study was conducted on solid wood-stud shear walls with different aspect ratios. Three aspect ratios were considered, namely ratio of vertical to horizontal dimension of 1.25, 1.875 and 3.75. The walls were 120 in. high. The walls had lengths of 96 in., 64 in. and 32 in., respectively. The models represented three variations, 1) bare wood-stud shear wall, 2) bare wood-stud shear wall with the mass of the veneer lumped to the wood-stud model nodes, and 3) wood-stud shear wall connected to masonry veneer with veneer anchors.

7.2.2 Seismic Mass Representation

In order to determine the tributary seismic mass for the wood-stud shear walls to be analyzed a prototype one-story structure of 12.20 m by 12.20 m (40 ft. by 40 ft.) in plan dimensions as shown in Figure 7.1 was considered. The wood-shear walls along the perimeter are responsible for resisting the lateral loads. The dead load of the roof due to its own weight was 0.34 kN/m$^2$ (7 psf). A live load of 0.96 kN/m$^2$ (20 psf) was assumed, according to ASCE 7-05 (ASCE, 2005). It is assumed that for such one-story buildings a reduction factor of 30% will be appropriate for the live load contribution in the calculations of the seismic mass. Assuming that half of the roof seismic mass will be carried by each of the two parallel shear walls, one can calculate the weight per unit length of a shear wall as follows:

Area of roof supported by frame = 12.20 m x 6.10 m = 74.42 m$^2$ (800 ft.$^2$)
Weight per unit area of roof = 0.34 kN/m² + 0.30 x 0.96 kN/m² = 0.628 kN/m² (13 psf)
Total seismic weight per frame = 74.42 m² x 0.628 kN/m² = 46.74 kN (10.40 kips)
Seismic weight per unity length of wood shear wall = 46.74 kN / 12.20 m = 3.83 kN/m (0.26 kips/ft)

7.2.3 Time History Analyses

Time history analyses were conducted on the developed models with three ground motion records, namely, the Sylmar, Tarzana and El Centro ground motion records. The acceleration time histories were scaled to reflect the Design Basis and Maximum Considered earthquakes for such structures. Scaling factors of 80 and 120%, 36 and 54% and 150 and 225% were applied to each one of the three records to arrive at the DBE and MCE, respectively. Analysis time step of 0.0005 was used for the numerical integration of the equations of motion. Similar to the analyses presented before in this dissertation, Rayleigh damping was used to simulate the energy dissipation mechanism with 5% of the critical damping applied at the first and the 12th natural modes. Numerical results are designated by the condition it represents in that (W) represents the bare wood-stud shear wall model, (W+AM) represents the bare wood-stud frame with the added veneer mass and (W+V) represents the model considering both the veneer and the wood frame explicitly. It should be noted that a coefficient of friction of 0.50 was used for the veneer panel, a value that was shown to provide good results.
7.2.4 Discussion of Analytical Results

The fundamental frequency of each of the model variations plotted against the panel aspect ratio in Figure 7.2. The figures show that veneer stiffens the considered wood-stud shear wall, with this effect more pronounced for the more squat panels. The case with veneer as added mass was shown to underestimate the wall frequency. This leads to an over estimation of the frame lateral drifts.

Figures 7.3 and 7.4 show the plots of the peak top displacement of the wood-stud frame versus the panel aspect ratio for the three variations under the DBE and MCE of the Sylmar record, respectively. The figures show that for lower aspect ratios the wood-stud frame with veneer (W+V) has either the same or slightly higher top peak displacement than the bare frame. For the aspect ratio of 3.75, the veneer addition will lead to increased frame lateral drifts. This is due to the rocking of the veneer which tends to transfer more load and overturning moments to the wood backing as shown in the experimental results in Chapter 6. The model considering the veneer simply as added mass shows highly overestimated frame displacements as compared to the wood/veneer model.

Figures 7.5 and 7.6 show similar results for the Tarzana record. Under the design basis earthquake, the wood/veneer model shows less wood-stud frame top displacements up to an aspect ratio of 2.75. This shows that the veneer actually restrains the wood-stud frame and helps to reduce its lateral drifts. The squat veneer panel attracts a huge portion of the seismic load and dissipates a lot of energy by base sliding, leading to less demands
on the backing frame and hence a reduction in the top displacements. This effect is reduced under the MCE level due to the occurrence of the base sliding of the veneer and merely reduces its effectiveness in load resistance. Under the MCE, the restraining effect is only experienced up to an aspect ratio of about 1.70. Once again, the added mass model is shown to be overly conservative as it highly overestimates the frame drifts. Figures 7.7 and 7.8 show similar results for the El Centro ground motion.

7.2.5 Evaluation of the Relative Velocity Index

In Chapter 6, the Relative Velocity Index (RVI) was developed as a measure of the restraining effect of the veneer. To further assess the validity of the developed index and to confirm the observations outlined above, the RVI is evaluated with the results of the wood/veneer model. Figures 7.9 and 7.10 show the relative velocity index time history for the walls with aspect ratios 3.75 and 1.25, respectively. The plots were generated under the El Centro design basis level. Similar to the observations in Chapter 6, the slender wall has negative values of R.V.I. in many time instances showing that the veneer is driving the wood backing. On the other hand, the squat panel has mostly positive values of R.V.I. This observation is also replicated under the other ground motion records.

7.2.6 Conclusions of the Study

The results of the numerical study outlined above suggest that masonry veneer oriented parallel to the direction of shaking can have a favorable restraining effect on the
wood-stud backing. The effect is dependant on the panel aspect ratio. Veneers with aspect ratios about 1.25 - 1.50 and lower tend to restrain the wood backing against lateral drifts. The relative velocity index developed based on the results of the shaking-table tests of the prototype building was confirmed to be a good measure of the restraining effect of the veneer.

7.3 Out-of-Plane Seismic Performance of Two-Story Masonry Veneer

7.3.1 Outline of Analytical Study

This phase of the analytical study investigates the performance of masonry veneer in two-story buildings. The analyses deal specifically with the out-of-plane response of such veneers. The study assesses the adequacy of the current MSJC (MSJC, 2008) prescriptive code provisions for these structures. The study considered two different anchor types (corrugated and rigid ties), different anchor vertical spacing (406 mm (16 in.) and 203 mm (8 in.) for corrugated ties and 609 (24 in.) for rigid ties), and different anchor strengths. The strengths considered for the corrugated ties are 1) average pullout strength (667 N (150 lbs)) reported by previous researchers and deduced in this research, 2) upper bound strength (1780 N (400 lbs)) corresponding to the use of screws instead of nails and 3) a lower bound strength of (222 N (50 lbs)) corresponding to reduced nail pullout capacity due to higher wood moisture content.
7.3.2 Finite Element Discretization

Figure 7.11 shows a schematic of the finite element discretization of the two-story building model. The masonry veneer and the wood-stud backing were modeled in a way similar to that presented in Chapter 4. The out-of-plane wall assembly is connected with stiff springs to a rigid beam with a rotational spring at the base. The rigid beam was used to simulate the behavior of the in-plane wood-stud shear wall. Pinned supports were assigned to the base of the out-of-plane wood-stud frame and masonry veneer walls. The model used the prototype building shown in Figure 7.1 as a reference for the calculation of the seismic masses and cross-sectional dimensions. For the simulation of the worst case scenario of the building, the contribution of the in-plane masonry veneer of the first story to the wall resistance is ignored. Furthermore, the mass of the in-plane masonry veneer in the two stories is lumped and combined with the floor masses of the two stories. It is worth mentioning that for the purpose of comparison of the results, a similar one-story model was developed using the same procedure. Two different cases of the two-story models were analyzed, namely, continuous and discontinuous veneers over the entire building height. The discretization of the discontinuous veneer models is shown in Figure 7.11.

7.3.3 Modeling of In-Plane Wood-Stud Shear wall

In order to simplify the model and to cut down on the computational effort, the in-plane wood-stud shear wall was represented with a rigid beam with a nonlinear rotational spring at the base. The properties of this spring were chosen such that the response of the
beam model is similar to the response of the two-dimensional wood-stud frame model of the shear walls analyzed with the detailed modeling scheme developed in Chapter 5. For this reason the hysteretic relation in OpenSees (OpenSees ID: `uniaxialMaterial Hysteretic`) is used with a tri-linear backbone curves only. Figures 7.12 and 7.13 show a comparison of the monotonic and cyclic force-displacement responses of the simplified and detailed models for the one- and two-story buildings, respectively. The figures show a perfect match under low displacements and a very close match under higher displacement levels.

### 7.3.4 Incremental Dynamic Analysis

The numerical models for the one- and two-story models were analyzed under three ground motion records, namely, El Centro, Sylmar and Tarzana records. The analyses were conducted in an incremental manner by subjecting the model to the aforementioned motions scaled by 60%, 80%, 100%, 125%, 150%, 175% and 200%, respectively. The relative displacement between the veneer and the wood-stud backing, reflecting the anchor deformation, was monitored for all the ties. The spectral acceleration of the scaled ground motion was plotted against the maximum tie deformation. Furthermore, the spectral accelerations were plotted against the peak veneer accelerations. The spectral acceleration was determined at the fundamental period of each of the buildings determined by eigenvalue analysis. The following section presents and discusses in detail the results of the incremental dynamic analyses. The figures uses the notation of "E", "S", "T" to denote the three ground motions records, El Centro, Sylmar
and Tarzana, respectively. Notation "1S1" denotes the results of the one-story model and "2S1" and "2S2" are used for the results of the first and second floors for the two-story model.

7.3.5 Discussion of Analytical Results for Discontinuous Veneer

Figures 7.14 through 7.16 present the results for the corrugated-tie walls with 406 mm (16 in.) vertical spacing for the three different anchor strengths. The results show that walls with the average anchor capacity can sustain ground motions representative of the intermediate SDC D and E safely. It is interesting to observe that the two-story building can resist a much higher spectral acceleration than the one-story one. Figures 7.17 and 7.18 show the base shear time histories of the in-plane shear walls of the one- and two-story buildings under El Centro 200% for the case of the average anchor capacity. Figure 7.17 suggest that the one-story wall was still below its first yield point while Figure 7.18 suggests that the two-story wall had already yielded. This means that the one-story wall still preserved its high initial stiffness while the two-story wall suffered a significant reduction in stiffness. This resulted in higher backing deformation for the out-of-plane wall in the two-story model leading more or less to be rigid body rotation of the veneer and the backing about the hinged base; this deformation mode relives the demand on the ties. On the contrary, the one-story model with the high stiffness provides a firmer backing support which increases the demand on the veneer anchors.
The walls with the upper bound anchor strength showed superior performance than that with the average strength with the difference between the one- and two-story models being minimal. On the other hand, the models with the lower bound strength showed extremely low resistance compared to the previous cases. One-story walls with this strength would probably collapse under ground motions lower than the DBE. Two-story models would be highly vulnerable to collapse under ground motions close to the upper-bound DBE for SDC E and definitely collapse under the MCE.

Figure 7.19 through 7.21 show similar plots for the corrugated tie walls with a closer spacing of 203 mm (8 in.). The plots show an overall similar behavior to the wider spacing models but with a higher anchor resistance. This is can be clearly seen by examining Figures 7.15 and 7.20. Figure 7.21 shows a less reduction in the performance compared to the wider spacing model shown in figure 7.16. Once again the one-story model would collapse at a motion lower than the DBE but the closer anchor spacing would save the two-story building under higher ground motion levels.

Figure 7.22 shows the results for the rigid tie walls. Once again the higher pullout strength of these ties due to the use of screws results in a superior performance. The observations of this figure are consistent with the upper bound models presented above as well as the results of the shaking-table tests outlined earlier in this dissertation.

Figure 7.23 through 7.29 show the plots for the peak veneer acceleration versus the spectral acceleration for all the analyzed variations discussed previously. The plots
once again confirm the observed higher strength of the two-story models due to the yielding of the in-plane shear walls.

### 7.3.6 Discussion of Analytical Results for Continuous Veneer

Figures 7.30 through 7.36 present the results of the analyses with continuous veneer over the two-story height. Almost identical response is observed in this case when compared to the discontinuous veneer case. Hence, the effect of the separation at the second story will not be influential since the veneer acts as an added mass carried by the wood-stud backing and forced to follow its response.

### 7.3.7 Conclusions of the Study

The results of the numerical study outlined above suggest that the prescriptive requirements of the MSJC (MSJC, 2008) are adequate for low-rise masonry veneer buildings with wood-stud backing. Additional analyses with a spectrum of ground motion records are necessary to draw a more solid conclusion. The results highlight the necessity of a minimum anchor strength requirement to assure a minimum level of performance under ground motions representative of the design basis and maximum considered earthquakes. Anchor strength of 667 N (150 lbs) was shown to be sufficient for acceptable performance. This strength would be acceptable provided that it can be maintained under the entire life span of the anchor and does not vary with environmental conditions.
7.4 Simplified Three-Dimensional Modeling of Prototype Building

7.4.1 Model Description and Finite Element Discretization

This section describes the simplified numerical model used to simulate the behavior of the prototype building. Figure 7.37 shows the finite element discretization of the west wall in the building. The veneer and the wood-stud frame were modeled using the same procedure outlined in Chapter 5. The corrugated sheet metal anchors on this wall were assigned strength of 222 N (50 lbs). This low strength was chosen to simulate the reduced nail withdrawal capacity due to the high moisture content of the wood at the time of construction. It was shown later to yield good results in comparison with the experimental data. The in-plane walls (veneer and wood-stud backing) were analyzed under cyclic lateral load applied to the top of the wood-stud frame. Results of this analysis were used to deduce the properties of the rotational spring for the beam element model of this wall.

The wood diaphragm was modeled as a stiff beam joining the top of the two in-plane walls. This stiff idealization is motivated by the fact that the diaphragm in the tests did not show any significant deformation under the entire shaking sequence. The influence of the blockings is modeled with a spring connecting the nodes of the top plate in the wood-stud backing and the stiffer beam simulating the wood diaphragm. The blocking is modeled with a similar relationship as that for the nail extraction in the corrugated ties. This latter idealization is due to the fact that in compression the blocking
transmits the force while in tension it can only transfer a load no more than the extraction capacity of the nails.

7.4.2 Comparison of Analytical and Experimental Results

Figures 7.38 and 7.39 show a comparison of the displacement time histories of the veneer and the backing, respectively, from both the test and the numerical simulation. The plots show the results for different sensor locations as depicted by Figure 7.37. The plots show a very close match between the experimental and analytical results under the ground motion record of Sylmar 50%. Under Sylmar 80%, this wall collapsed in the actual experiment by the total pullout of the veneer anchors from the wood-stud backing. This mode of failure was captured by the model as shown by the deformed mesh in Figure 7.40. The time histories of the displacement for a position in the top row of ties, as shown in Figure 7.41, under this level of excitation showed very close match between the experimental and analytical results. The model did capture the exact moment of failure as well as the deformations associated with it.

7.5 Design Recommendations

The shaking-table tests conducted in this research suggest that masonry veneer designed and constructed according to the current MSJC (MSJC, 2008) prescriptive provisions can resist ground motion records corresponding to the DBE and MCE safely with minimum or no damage. However, the tests pointed out the vulnerability of the corrugated veneer anchors to premature failure. This was mainly attributed to the
reduction of the nail pullout strength due to the moisture conditions. Furthermore, the conclusions of the previous analytical studies as well as the results of the numerical models outlined in previous chapters pointed out the need for a minimum required pullout strength for the veneer anchor to assure an acceptable performance and to prevent premature collapse. These two arguments formulate the important considerations of the veneer anchor choice and specification. It is recommended that an anchor has minimum pullout strength of 667 N (150 lbs) that can be sustained under variable environmental conditions, more specifically variable moisture content of the wood-studs at the time of construction. Ring-shank nails or screws offer such desired properties and are therefore highly recommended.

The study conducted in the research outlined in this dissertation addresses only the structural aspects of the response. Effects such as anchor corrosion, loss of galvanization due to cyclic expansion and contraction of the wall system, differential settlement, poor workmanship and insufficient or lack of embedment of veneer anchor are among the factors that were not addressed by this study. Therefore, it is highly recommended that an additional uncertainty factor be applied to the required strength for the veneer anchors. The value of this uncertainty factor is recommended to be 20%.

The test results did not show any beneficial effect of the joint reinforcement under either in-plane or out-of-plane seismic excitation. Under out-of-plane loading the wires embedded in the mortar joint would be totally ineffective. The only benefit one can think of is the distribution of the anchor force on a longer veneer length. This might no be as
influential because the peak values of the anchor forces, which are governed by the anchor capacity, are not high enough to cause veneer distress. Test results showed that the anchor would start pulling out from the backing before the joint reinforcement gets engaged. For the rigid ties, the failure mode was governed by mortar joint fracture and pullout of the ties from the mortar joints. Joint reinforcement did not show any influence that could mitigate that mode of failure. Under in-plane seismic excitation, joint reinforcement did not alter the mode for failure for the wall. It might have helped in restraining the rocking of the slender segments but the observed global response of walls with and without joint reinforcement was essentially the same. The failure mode was also the same and governed by anchor pullout from the wood-studs under reversed cyclic shear deformations. Therefore it is recommended that the requirements for the inclusion of joint reinforcement be eliminated from the MSJC (MSJC, 2008).

Experimental and analytical results suggested that masonry veneer oriented parallel to the direction of shaking can help the wood-stud frame by attracting a huge portion of the lateral load and restrain the wood-stud shear walls. Veneer panels of aspect ratios of 1.25 to 1.50 and lower can effectively restrain and stiffen the wood-stud backing. The treatment of masonry veneer as added mass was shown to be overly conservative and leads to unrealistic assessment of the wood-stud frame deformations. The masonry veneer panel can dissipate a significant amount of energy through sliding along the flashing. It is highly recommended that the synergy between the veneer and the backing be revisited and the contribution of the masonry veneer to the lateral load resisting mechanism should be acknowledged in the design process.
Figure 7.1. Prototype Building Models (1 m = 39.4 in.)
Figure 7.2. Variation of Fundamental Frequency

Figure 7.3. Peak Top Wood Displacement (Sylmar – DBE) (1 m = 39.4 in.)
Figure 7.4. Peak Top Wood Displacement (Sylmar – MCE) (1 m = 39.4 in.)

Figure 7.5. Peak Top Wood Displacement (Tarzana – DBE) (1 m = 39.4 in.)
Figure 7.6. Peak Top Wood Displacement (Tarzana – MCE) (1 m = 39.4 in.)

Figure 7.7. Peak Top Wood Displacement (El Centro – DBE) (1 m = 39.4 in.)
Figure 7.8. Peak Top Wood Displacement (El Centro – MCE) (1 m = 39.4 in.)

Figure 7.9. Relative Velocity Index – Aspect Ratio 3.75 (El Centro – DBE)
Figure 7.10. Relative Velocity Index – Aspect Ratio 1.25 (El Centro – DBE)

Figure 7.11. Finite Element Discretization – Two-Story Model
Figure 7.12. Monotonic Response of In-Plane Shear Walls

Figure 7.13. Cyclic Response of In-Plane Shear Walls
Figure 7.14. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 406 mm, Average Strength, Discontinuous Veneer for Two-Story Model)

Figure 7.15. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 406 mm, Upper Bound Strength, Discontinuous Veneer for Two-Story Model)
Figure 7.16. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 406 mm, Lower Bound Strength, Discontinuous Veneer for Two-Story Model)

Figure 7.17. Base Shear Time History (One-Story In-Plane Shear Wall) – El Centro 200%
Figure 7.18. Base Shear Time History (Two-Story In-Plane Shear Wall) – El Centro 200%

Figure 7.19. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 203 mm, Average Strength, Discontinuous Veneer for Two-Story Model)
Figure 7.20. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 203 mm, Upper Bound Strength, Discontinuous Veneer for Two-Story Model)

Figure 7.21. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 203 mm, Lower Bound Strength, Discontinuous Veneer for Two-Story Model)
Figure 7.22. Spectral Acceleration vs. Maximum Tie Deformation (Rigid Ties @ 609 mm, Average Strength, Discontinuous Veneer for Two-Story Model)

Figure 7.23. Spectral Acceleration vs. Peak Veneer Acceleration (Corrugated Ties @ 406 mm, Average Strength, Discontinuous Veneer for Two-Story Model)
Figure 7.24. Spectral Acceleration vs. Peak Veneer Acceleration (Corrugated Ties @ 406 mm, Upper Bound Strength, Discontinuous Veneer for Two-Story Model)

Figure 7.25. Spectral Acceleration vs. Peak Veneer Acceleration (Corrugated Ties @ 406 mm, Lower Bound Strength, Discontinuous Veneer for Two-Story Model)
Figure 7.26. Spectral Acceleration vs. Peak Veneer Acceleration (Corrugated Ties @ 203 mm, Average Strength, Discontinuous Veneer for Two-Story Model)

Figure 7.27. Spectral Acceleration vs. Peak Veneer Acceleration (Corrugated Ties @ 203 mm, Upper Bound Strength, Discontinuous Veneer for Two-Story Model)
Figure 7.28. Spectral Acceleration vs. Peak Veneer Acceleration (Corrugated Ties @ 203 mm, Lower Bound Strength, Discontinuous Veneer for Two-Story Model)

Figure 7.29. Spectral Acceleration vs. Peak Veneer Acceleration (Rigid Ties @ 609 mm, Average Strength, Discontinuous Veneer for Two-Story Model)
Figure 7.30. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 406 mm, Average Strength, Continuous Veneer for Two-Story Model)

Figure 7.31. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 406 mm, Upper Bound Strength, Continuous Veneer for Two-Story Model)
Figure 7.32. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 406 mm, Lower Bound Strength, Continuous Veneer for Two-Story Model)

Figure 7.33. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 203 mm, Average Strength, Continuous Veneer for Two-Story Model)
Figure 7.34. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 203 mm, Upper Bound Strength, Continuous Veneer for Two-Story Model)

Figure 7.35. Spectral Acceleration vs. Maximum Tie Deformation (Corrugated Ties @ 203 mm, Lower Bound Strength, Continuous Veneer for Two-Story Model)
Figure 7.36. Spectral Acceleration vs. Maximum Tie Deformation (Rigid Ties @ 609 mm, Average Strength, Continuous Veneer for Two-Story Model)

Figure 7.37. Finite Element Idealization
Figure 7.38. Veneer Displacement Response (Sylmar 50%)

(a) Veneer 1  
(b) Veneer 4  
(c) Veneer 7  
(d) Veneer 3  
(e) Veneer 6  
(f) Veneer 9
Figure 7.39. Backing Displacement Response (Sylmar 50 %)
Figure 7.40. Failure Pattern (Sylmar 80 %)

Figure 7.41. Veneer Displacement Time History (Veneer 7 - Sylmar 80 %)
CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 Summary

This dissertation aims at studying the seismic performance of clay masonry veneer in wood-stud framed structures. The seismic performance of clay masonry veneer backed by wood-stud shear walls was experimentally and analytically evaluated by shaking-table tests and finite element models. The study was a part of the NSF NEES research project “Performance Based Design of New Masonry Structures”. The experimental program involved the testing of a total of eleven wall segments (clay masonry veneer and a wood-stud shear wall) dynamically on the NEES outdoor shaking table at the University of California at San Diego. Seven of the walls were tested in their out-of-plane direction and four were tested under in-plane base excitation. In addition, a full-scale one-story prototypical masonry veneer building was tested on the shaking table under uni-directional shaking. All the wall assemblies and the prototype building were designed and constructed in full compliance with the prescriptive requirements of the MSJC code (MSJC, 2008) for masonry veneer, and also satisfied the design and construction requirements of the IRC (ICC, 2006) for timber structures and followed the serviceability requirements of the Brick Industry Association Technical Notes (BIA, 2002).
In the wall segments testing phase of the research, design parameters investigated included anchor types, anchor spacing, wall aspect ratio, and presence and absence of mortar joint reinforcement and window opening. All the tested variations corresponded to the MSJC (MSJC, 2008) requirements for Seismic Design Categories D and E. The shaking-table tests showed that the out-of-plane dynamic response is mostly governed by the properties of the veneer anchors under axial loading, namely the anchor pullout strength. Corrugated ties connected to the wood-studs with nails failed due to progressive nail pullout while the rigid ties connected to the backing with screws failed due to sudden anchor pullout from the mortar joints. The ground motions at which the wall collapsed correspond to levels of excitation that are far in excess of representative Design Basis and Maximum Considered Earthquakes for Seismic Design Categories D and E. Walls with and without joint reinforcement did not show major difference in behavior or seismic resistance. The in-plane dynamic response is characterized by veneer sliding along the base flashing for the squat veneer walls and a combined sliding/rocking mode for the slender panels. The in-plane walls could sustain higher level of ground motions without collapse as compared to the out-of-plane walls. One of the walls with window opening collapsed due to the formation of diagonal cracks at the top and bottom of the window piers and the ties failed by pullout from the backing, pullout from mortar joints and tie rupture and tearing due to fatigue.

The tested prototype building had one of the out-of-plane walls with corrugated ties while the other with rigid ties. The in-plane walls had corrugated ties but one had joint reinforcement and the other did not. Except for the out-of-plane wall with
corrugated ties, both the wood frame and the masonry veneer performed well under severe ground motions, which exceeded a representative maximum considered earthquake and an upper-bound design basis earthquake for Seismic Design Category D. The premature collapse of the west veneer at a design level earthquake was attributed to the low extraction capacity of the nails attaching the corrugated ties to the wood-stud backing. A combination of the high moisture content in the wood and the initial slack in the tie corrugations is probably the main cause of this observed failure. The rigid ties with screw attachment performed well. Test data showed that the veneer walls parallel to the direction of shaking helped to restrain the motion of the wood structure. However, short walls with a high height-to-length ratio tended to rock and induce additional seismic force on the wood structure under severe excitation. Continuous veneer corners helped supporting the out-of-plane veneer segments. There is no evidence that the joint reinforcement in veneer can improve the seismic performance. The tests pointed attention towards the detailing of wood roof diaphragms which requires special consideration of the out-of-plane inertia force of the veneer that can be transmitted through the top plate of the wood-stud wall to the rim joist.

The analytical phase of the research incorporated the development of nonlinear finite element models to simulate the dynamic behavior of masonry veneer. The first model was related to the analysis of the out-of-plane specimens. It consisted of nonlinear displacement-based beam-column elements to model the veneer wythe, elastic beam elements to model the wood-studs and nonlinear truss elements with an appropriate hysteretic rule to model the veneer anchors. Two rules were proposed for the two kinds of
used anchors. The analytical results showed a very close match with the experimental data. The results gave an insight to the mechanism of anchor force distribution along the wall height. The anchor force distribution is dependant on the state of veneer cracking. For uncracked veneer, the distribution is highly affected by the backing flexibility and the boundary conditions of the wall, but once the veneer cracks, higher anchor forces are localized around the developed cracks. Depending on the anchor ductility, redistribution and spreading of the anchor yielding can occur. This behavior was experienced by the corrugated ties but was totally absent in the rigid anchors.

A parametric study on the factors affecting the out-of-plane response revealed the sensitivity of the wall response to the anchor capacity. Lower anchor strength leads to premature collapse of the wall in a deformation mode that is characterized by the veneer leaning away from the wood backing. With increasing the anchor capacity, the deformation changed to a bending mode with cracks forming around the mid-height of the wall. This allowed for re-distribution of anchor forces. Closer anchor spacing was shown to force similar deformation pattern. Anchor initial stiffness did not have too much influence on the response. Slack found in corrugated ties showed a negative effect on the wall performance with the generation of high acceleration spikes, leading to a higher localized dynamic amplification and premature collapse.

Another numerical model was developed to simulate the behavior of the in-plane wall panels. The model used a combination of nonlinear beam-column and truss elements to model the behavior of masonry. These combination formed the “Masonry Element”
that is capable of simulating the behavior of the masonry in unreinforced walls. The model was also validated against hand calculated estimates of the wall strength. The model also introduced an innovative modeling approach for the wood-stud frames. The developed wood-stud frame element is a combination of elastic beam and truss elements and nonlinear diagonal trusses. All the nonlinearities that develop in the wood shear wall due to fastener deformations are lumped in these diagonal elements. An appropriate hysteretic rule was proposed and calibrated against the results of racking tests on wood shear walls with different aspect ratios. The assembled model showed a very good match with the test data from the wall segments. This modeling scheme is also applicable for out-of-plane analyses and modeling of three dimensional veneer structures.

The developed model was then used to conduct a parametric study on the effect of in-plane masonry veneer on the seismic performance of wood-stud shear walls. The study showed that squat veneer panels (aspect ratios of 1.25 and lower) can have a favorable restraining effect on the wood-stud frame. Slender rocking segments tend to have an negative effect. The results of this study confirm the deduction made from the prototypical building tests. Furthermore, the study showed that the treatment of veneer as added mass is not appropriate. It is considered as a highly conservative estimation which leads to an over-estimation of the lateral drifts of the shear wall.

Another parametric study was conducted on the behavior of two-story masonry veneer under out-of-plane seismic excitation. Two-story models, using the same modeling scheme as that used for the out-of-plane wall assembly modeling, was used to
model the wall panel. The contribution of an intersecting in-plane wood-stud shear wall was taken into account as well. These models were analyzed for the cases of continuous and discontinuous two-story veneers. The results of the study also showed the adequacy of the current prescriptive requirements of the MSJC (MSJC, 2008) for either case. The results highlighted the need for a minimum anchor strength requirement to assure a minimum level of performance under ground motions representative of the design basis and maximum considered earthquakes. Anchor strength of 667 N (150 lbs) was shown to be sufficient for accepted performance.

Finally the model was used to simulate the behavior of the prototype building with a simplified three dimensional model. The model for the west wall in the building was built with the two-dimensional modeling scheme discussed earlier and connected to vertical beam elements with nonlinear rotational springs at the base to simulate the behavior of the north and south walls. Influence of the pullout of the nails from the blockings was included in the model. The model showed an excellent match with the experimental results and was able to capture the failure mode of the wall. The strength for the corrugated ties was 222 N (50 lbs) which is deduced from the experimental results.

8.2 Conclusions

8.2.1 Testing

The shaking-table tests on masonry veneer wall assemblies showed that veneers constructed over wood-stud backing and designed in accordance with current MSJC code
provisions (MSJC, 2008) can sustain ground motions far in excess of the representative design basis and maximum considered earthquakes, with minor or no damage at these levels. The in-plane behavior is governed by base sliding for squat veneer panels and a combination of sliding and rocking for slender ones. The out-of-plane behavior is highly affected by the capacity of the veneer anchor under axial tension. Walls with corrugated tie collapse due to nail pullout from the wood-studs. Walls with rigid ties are characterized by a sudden dramatic collapse mostly due to the anchor pullout from the mortar joints. Joint reinforcement, used with both kinds of veneer anchors, does not seem to have any added value to the global response or seismic resistance of a wall.

The shaking-table tests conducted on a prototypical masonry veneer building with wood-stud backing confirmed most of the previously concluded facts in the wall assembly tests. The tests revealed the vulnerability of the corrugated ties to lower extraction capacities due to the higher timber moisture content. Veneer that previously sustained very high ground motions collapsed under the design basis earthquake with no previous warning and by the complete pullout of all the veneer anchors from the wood-studs. Once again, joint reinforcement does not seem to cause any difference in the observed global response of the walls. Continuous veneer corners helps engaging the masonry panels in two-way bending action and restraining the veneer against out-of-plane failure. The vulnerability of wood diaphragms and rim joists connections to damage under moderate excitation merits attention. The tests confirm the superior performance of the rigid ties due to the higher fastener extraction capacity. The tests
highlight the favorable effect of in-plane squat veneer panels in restraining the wood-stud shear wall against lateral drifts.

### 8.2.2 Analytical Modeling

Finite element models were developed and calibrated in light of the experimental results of the wall assembly tests. An analytical model used to simulate the behavior of out-of-plane masonry veneer panels is able to closely capture the displacement and acceleration response as well as the failure pattern of the tested walls. Hysteretic models for veneer anchors under axial loading are proposed. The model shows the effect of anchor ductility on the distribution of anchor forces along the wall height.

A parametric study on the influential parameters revealed the importance of the anchor ultimate capacity, whether nail pullout strength for corrugated ties or anchor pullout strength from mortar joint in the case of rigid ties, on the seismic resistance of a wall. The initial slack and flexibility of corrugated ties have a negative effect on the performance of a wall. Reducing anchor spacing helps the wall sustain higher loads and force a more favorable deformation pattern under lateral loading.

A two-dimensional analytical model was proposed to model masonry veneer under in-plane excitation that is also adaptable for out-of-plane loading and three-dimensional dynamic simulations. The model introduces an innovative approach to account for the nonlinearity of the wood-stud frames. The model is able to capture the
A parametric study on the in-plane masonry veneer showed the dependency of the veneer restraining effect on the panel aspect ratio. Wall panels with aspect ratios (ratio of vertical to horizontal dimension) of 1.25 and lower restrain the wood-stud backing and reduce the lateral drifts. Treating veneer as added mass showed to be overly conservative and leads to an over-estimation of the backing lateral drifts.

A parametric study on the out-of-plane performance of two-story masonry veneer buildings showed that the flexibility of the in-plane intersecting wood shear walls can reduce the demands on the ties in the out-of-plane wall.

**8.2.3 Design Recommendations**

Masonry veneer should not be treated as added mass when dealing with in-plane dynamic loading. Its contribution is dependant on the panel aspect ratio. If masonry veneer is secured against out-of-plane collapse, it enhances the wood-stud shear wall dynamic response by reducing the wall drifts and dissipating energy by sliding. The mass of the in-plane squat veneer panels should be excluded from all wood-stud frame drift calculations.
Attention should be given to fastener withdrawal capacities from the wood-studs. The fastener should be able to provide a minimum required pullout capacity of 667 N (150 lbs) as well as the ability to maintain this capacity under different environmental conditions. Ring shank nails or screws are examples of such fasteners. The capacity was mainly derived taking into account all the structural aspects of the response. This capacity should be further increased by an uncertainty factor with a recommended value of 20%. This factor takes into account the untested conditions such as fastener corrosion, insufficient anchor embedment, anchor corrosion and other environmental and workmanship related reasons.

The requirement in the MSJC code (MSJC, 2008) for the inclusion of joint reinforcement for Seismic Design Category D and above is not necessary and can be eliminated. Test results did not show any favorable effect for the joint reinforcement that can lead to an enhanced seismic performance.

8.3 Recommendations for Future Research

Extensive experimental and analytical studies are needed to study veneers subjected to multi-directional shaking. Properties of different veneer anchors under combined axial and shear loading and the nature of their interaction needs evaluation in a sub-assembly scale in order to be used in detailed numerical simulations.
Experimental evaluation of the seismic performance of multi-story masonry veneer in low-rise wood construction is needed to validate the analytical models.

Application of the developed analytical models to the dynamic simulation of full-scale three-dimensional masonry veneer structures is needed. Analyses using ground motion records of different characteristics such as the duration of strong motion is another area that needs further exploration.
REFERENCES

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APPENDIX A

CONSTRUCTION OF PROTOTYPE BUILDING

The following pictures show the construction of the wood-stud prototype building presented in Chapter 6. It shows the construction sequence and the different components of the building specimen that might have not been clear in the specimen description in Chapter 6.

Figure A.1. Assembled East and West Wood-Stud Frames
Figure A.2. Assembled North and South Wood-Stud Frames

Figure A.3. Footing’s Form Work and Reinforcement Cage Ready for Concrete Pour
Figure A.4. Casting of Reinforced Concrete Footing

Figure A.5. Anchored Erected Wood-Stud Frames
Figure A.6. Installation of Clip Angles at desired Spacing of Joists

Figure A.7. Roof Joists Nailed to Rim and Connected by Two Clip Angles
Figure A.8. 2 x 4 Blockings and Metal Braces

Figure A.9. Stapling of EPDM Flashing at the Frame Base
Figure A.10. Installation of Frames OSB Sheathing

Figure A.11. Installation of Roof OSB Sheathing
Figure A.12. OSB Sheathed Frames Ready for Veneer Construction

Figure A.13. Sheathed Frames, Blockings, Braces and Ceiling Board
Figure A.14. Interior of Structure after Installation of Gypsum Wall Board

Figure A.15. Construction of Rigid Tie Wall with Joint Reinforcement
Figure A.16. Construction of Corrugated Tie Wall with Joint Reinforcement

Figure A.17. Construction of Window Sill Blocks with Flashing and Anchor Rotation
Figure A.18. Completed Structure