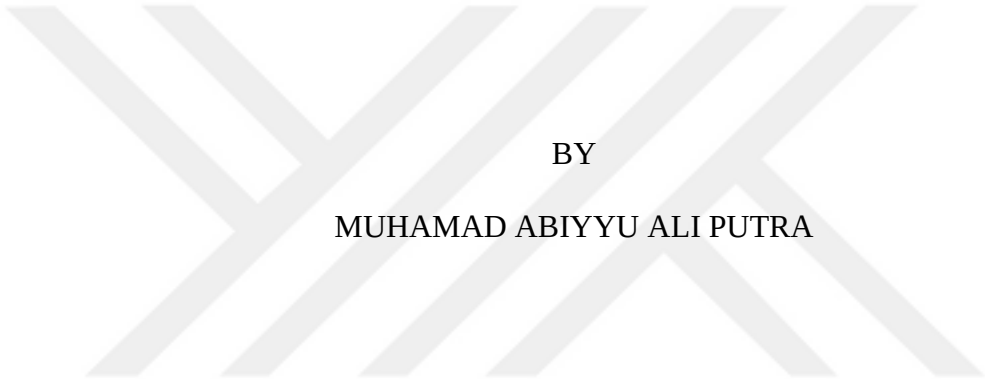


EARTHQUAKE RESISTANCE ASSESSMENT AND IMPROVEMENT FOR
NON-ENGINEERED CONFINED MASONRY HOUSES IN INDONESIA

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY



BY
MUHAMAD ABIYYU ALI PUTRA

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
CIVIL ENGINEERING

APRIL 2024

Approval of the thesis:

**EARTHQUAKE RESISTANCE ASSESSMENT AND IMPROVEMENT
FOR NON-ENGINEERED CONFINED MASONRY HOUSES IN
INDONESIA**

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ABSTRACT

EARTHQUAKE RESISTANCE ASSESSMENT AND IMPROVEMENT FOR NON-ENGINEERED CONFINED MASONRY HOUSES IN INDONESIA

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April 2024, 183 pages

Local architecture and construction techniques differ based on the traditional inheritance in each country and region. Unique construction techniques can be seen in Türkiye, China, Japan, Indonesia, Latin America, Africa, etc. This thesis concentrates on the safe structural design of the selected non-engineered confined masonry houses and contains development of applicable strengthening techniques to those non-engineered houses. The non-engineered houses evaluated in this thesis are in Indonesia, Southeast Asia, where it has a tectonically complex and seismically active area; this region consists of 3 large tectonic plates and 9 smaller plates. These plates with different movements have created subduction and fault zones that are continuously active. Unfortunately, most non-engineered confined masonry houses made from masonry are not appropriately built, and when earthquakes occur, severe damage or even total collapse happens at relatively low magnitude. Thus, this thesis concentrates on evaluation and strengthening techniques that can be applied to the typical traditional masonry house type while considering the out-of-plane bending capacity, crack generation via tensile stresses (such as opening wall at the corners), strengthening methods using additional columns-lintels and post-tensioning, and obtainability of the material which will be used to strengthen masonry houses.

Keywords: Earthquake Resistant Design, Confined Masonry House, Non-Engineered House



ÖZ

ENDONEZYA'DA MÜHENDİSLİK HİZMETİ ALMAMIŞ KUŞATILMIŞ YIĞMA EVLERİN DEPREME KARŞI DAYANIKLI DEĞERLENDİRİLMESİ VE GELİŞTİRİLMESİ

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Nisan 2024, 183 sayfa

Her ülke ve bölgede yerel mimari ve inşaat teknikleri, o ülkenin ve bölgenin geleneksel mimari mirasına dayalı olarak farklılıklar gösterir. Türkiye, Çin, Japonya, Endonezya, Latin Amerika, Afrika gibi farklı ülkelerde benzersiz inşaat teknikleri görülebilir. Bu tez, seçilmiş yerel mühendislik hizmeti almamış kuşatılmış yığma evlerin depremlere dayanıklı güvenli yapısal tasarımı ele almakta ve uygulanabilir güçlendirme tekniklerini geliştirmektedir. Bu kapsamda değerlendirilen yerel evler, aktif fay hatlarına sahip tektonik olarak karmaşık ve sismik olarak aktif bir bölge olan Güneydoğu Asya'da yer alan Endonezya'da bulunmaktadır. Maalesef, çoğu yerel yapı uygun şekilde inşa edilmemekte ve depremler meydana geldiğinde düşük büyüklüklerde bile ciddi hasar veya tam çökme meydana gelmektedir. Bu nedenle, bu tez, tipik geleneksel mimaride tuğla yığma ev türüne uygulanabilecek değerlendirme ve güçlendirme teknikleri üzerine odaklanmaktadır; düzlem-dışı eğilme, duvar köşelerinde açılma, çekme gerilme yoluyla çatlak oluşma, ek duvar içi ince kolon, lento ve ard-germe kullanarak güçlendirme yöntemleri, ve taş evleri güçlendirmek için kullanılacak malzemenin elde edilebilirliği de dikkate alınmaktadır.

Anahtar Kelimeler: Depreme Dayanıklı Tasarım, Çevrelenmiş Tuğla Yığma Ev





To my family, teachers, and friends

ACKNOWLEDGMENTS

The author wishes to express his deepest gratitude to his supervisor Prof. Dr. Ahmet Türer for his guidance, advice, criticism, encouragements, and insight throughout the research.

The author also wishes to express unparalleled gratitude to his father Dr. Ahmad Alamsyah Sp. KFR, his mother Cut Susana S.Psi. Psikolog, his sister Atifa Namiyah Putri, his uncle Teuku Nur Adzan and other family members for their continuous support and encouragement to make the author able to finish this thesis, without them the completion of this thesis will not be possible.

The author also owed a debt of gratitude to Furkan Konkan, Muhammad Juanda Putra, Güralp Aydemir, and Feyza Balcıoğlu for their valuable insights, guidance, and contribution for the completion of this thesis.

The author would also thank Elif Uyar, Kübra Kozan, Zulfiani Ayu, Rabia Tuğçe Şengül, Osman Yazgaç, Akbar Rifqy and other author's friends for their continuous support throughout the writing process of this thesis.

The author would also thank Konkan Engineering workers for their insights and valuable experiences to help the author finish this thesis.



TABLE OF CONTENTS

ABSTRACT.....	v
ÖZ	vii
ACKNOWLEDGMENTS	xi
TABLE OF CONTENTS.....	xiii
LIST OF TABLES	xvii
LIST OF FIGURES	xix
LIST OF SYMBOLS	xxv
CHAPTERS	
1 INTRODUCTION	1
1.1 Masonry House Construction in Indonesia.....	1
1.2 Materials used for Confined Masonry Construction in Indonesia	4
1.2.1 Clay Bricks.....	4
1.2.2 Mortars	6
1.2.3 Tie Columns and Beams	7
1.3 Failure Modes of Masonry Walls.....	7
1.3.1 In-Plane Failure Mechanism	7
1.3.2 Out-of-Plane Failure Mechanism.....	9
1.3.3 Failure Mechanisms of Confined Masonry Walls with No Diaphragm Action.....	11
1.4 Typical Observed Damage for Non-Engineered Masonry House in Indonesia	11

1.5	Objective and Scope	15
2	LITERATURE REVIEW	17
2.1	Strengthening of Masonry Wall Using Bamboo Component.....	17
2.2	Strengthening of Masonry House Wall Using Scrap Tires	18
2.3	In-Plane Seismic Strengthening of Masonry Wall Using Rebars	19
2.4	In-Plane Strengthening of Wall Using Timber.....	20
2.5	Surface Strengthening of Masonry Wall using Mortar Cross Strips.....	23
2.6	Partially Grouted Reinforced Clay Masonry Walls.....	24
2.7	Non-Engineered Masonry House Finite Element Modelling.....	26
2.8	Retrofit of Masonry Walls Using Chicken Wire Mesh	27
2.9	Masonry Wall's Out-of-Plane Strength after Retrofitted with OSB	30
2.10	Tensile Strength in Bending Comparison between Clay Brick and Aerated Brick.....	31
2.11	Evaluation of Confined Masonry Houses Guidelines	32
3	UNSTRENGTHENED CONFINED MASONRY HOUSE MODELING	35
3.1	Modelling of the Unstrengthened Confined Masonry Houses	35
3.2	Load Application	45
3.2.1	Dead Load	45
3.2.2	Wind Load	45
3.2.3	Earthquake Load.....	50
3.3	Load Combination	54
3.4	Analysis for Unstrengthened Models	55
3.4.1	Mesh Sensitivity Analysis	55
3.4.2	Displacement Criteria	61

3.4.3	Analysis of Unstrengthened House 1 Model	67
3.4.4	Analysis of Unstrengthened House 2 Model	77
3.4.5	Analysis of Unstrengthened House 3 Model	84
3.4.6	Yield Line Theory	93
4	STRENGTHENED CONFINED MASONRY HOUSE MODELLING	101
4.1	House 1 Strengthened Model	101
4.1.1	Addition of Columns	101
4.1.2	Addition of Lintel	104
4.1.3	Addition of Lintel + Column	106
4.1.4	Rebars Addition for Post-Tensioning	109
4.2	House 2 Strengthened Model	116
4.2.1	Addition of Columns	116
4.2.2	Addition of Lintel	117
4.2.3	Addition of Lintel + Column	118
4.2.4	Rebars Addition for Post-Tensioning	121
4.3	House 3 Strengthened Model	123
4.3.1	Addition of Columns	123
4.3.2	Addition of Lintel	126
4.3.3	Addition of Lintel + Column	128
4.3.4	Rebars Addition for Post-Tensioning	131
4.1	Displacement criteria for strengthened models	134
5	MINIMUM REQUIREMENTS AND RECOMMENDATIONS	139
5.1	Recommendations and Requirements According to TBDY 2018 and EERI & SDC/HA	139

5.2	Recommendations and Requirements According to Retrofitting Simple Buildings Damaged by Earthquakes Manual	145
6	DISCUSSION OF RESULTS AND CONCLUSIONS	151
6.1	Overall Discussion of Results	151
6.1.1	House 1 Discussion of Results	153
6.1.2	House 2 Discussion of Results	161
6.1.3	House 3 Discussion of Results	167
6.2	General Recommendations for Non-engineered Masonry Houses to improve EQ performance.	175
6.3	Conclusions	176
6.4	Future Studies	179
7	REFERENCES	181

LIST OF TABLES

TABLES

Table 1. Flexural Test Analysis Results [2].	32
Table 2. External Pressure Coefficient for wall. [20]	46
Table 3. External Pressure Coefficient for Roof. [20]	46
Table 4. External Pressure Coefficient for Roof. [20]	47
Table 5. Allowable story drift [23]	62
Table 6. Displacement criteria according to ASCE 7-16.	63
Table 7. Displacement criteria according to Euro Code 8	64
Table 8. Average λ for major period of structures.	65
Table 9 Displacement criteria according to TBDY 2018	66
Table 10. Flexure and Shear Stress Result for Unstrengthened House 1 Model.	75
Table 11. Flexure and Shear Stresses Checked for House 1 Unstrengthened Model	76
Table 12. Flexure and Shear Stresses Results of House 2 Unstrengthened Model.	83
Table 13. Flexure and Shear Stresses Checked for House 2 Unstrengthen Model.	84
Table 14. Flexure and Shear Stresses Results of House 3 Unstrengthened Model	90
Table 15. Flexure and Shear Stresses Checked for House 3 Unstrengthened Model	91
Table 16. House 1 yield line theory and EQ load calculation results.	96
Table 17. House 2 yield line theory and EQ load calculation results.	97
Table 18. House 3 yield line theory and EQ load calculation results.	98
Table 19. Additional Column Effect at House 1 Strengthened Model	103
Table 20. Additional lintel effect at House 1 Strengthened Model.	105
Table 21. Column and Lintel Additions effect at House 1 Strengthened Model.	107
Table 22. Post Tensioning Required for House 1 Strengthened Model.	115
Table 23. Additional Column Effect at House 2 Strengthened Model	117
Table 24. Additional lintel effect at House 2 Strengthened Model.	118

Table 25. Column and Lintel Additions effect at House 2 Strengthened Model. .	120
Table 26. Post Tensioning Required for House 2 Strengthened Model	123
Table 27. Additional column effect at House 3 Strengthened Model.	125
Table 28. Additional lintel effect at House 3 Strengthened Model.	127
Table 29. Column and Lintel Additions effect at House 3 Strengthened Model. .	129
Table 30. Post Tensioning Required for House 3 Strengthened Model	133
Table 31. Displacement criteria for additional column models	134
Table 32. Displacement criteria for additional lintel models	135
Table 33. Displacement criteria for additional column+lintels models	137
Table 34. Comparison between SAP2000 analysis result and Yield line theory ..	151
Table 35. Relative improvement of σ_{M11} for walls which are insufficient for house 1 using different strengthening techniques.	156
Table 36. Relative improvement of σ_{M22} for walls which are insufficient for house 1 using different strengthening techniques.	158
Table 37. Relative improvement of τ_{S12} for walls which are insufficient for house 1 using different strengthening techniques.....	160
Table 38. Relative improvement of σ_{M11} for walls which are insufficient for house 2 using different strengthening techniques.	163
Table 39. Relative improvement of σ_{M22} for walls which are insufficient for house 2 using different strengthening techniques.	165
Table 40. Relative improvement of τ_{S12} levels for walls which are insufficient for house 2 using different strengthening techniques.	167
Table 41. Relative improvement of σ_{M11} levels for walls which are insufficient for house 3 using different strengthening techniques.....	170
Table 42. Relative improvement of σ_{M22} levels for walls which are insufficient for house 3 using different strengthening techniques.....	172
Table 43. Relative improvement of τ_{S12} levels for walls which are insufficient for house 3 using different strengthening techniques.	174
Table 44. Summary of Failing Walls for Modeled Houses.....	176

LIST OF FIGURES

FIGURES

Figure 1.1. Tectonic plate boundaries of Indonesia and its surrounding region using crustal relative motion from GPS data (blue arrows) up to 2016 [1].....	2
Figure 1.2. One of non-engineered masonry house front view.....	4
Figure 1.3. In-Plane failure mechanism of masonry wall [5].	8
Figure 1.4. Bending action type which causes out-of-plane failure [5].	10
Figure 1.5. Roof tiles dislocation [6].	12
Figure 1.6. Separation and cracking of infill walls [6].	12
Figure 1.7. Out of plane failure of the masonry walls [6].....	13
Figure 1.8. Failure at corners of openings [6].....	13
Figure 1.9. Diagonal cracks in walls [6].	14
Figure 1.10. Total collapse at some of the walls [6].	14
Figure 1.11. Total house collapse [6].....	15
Figure 2.1. Test configuration for strip walls where (a). Using STR Chains and (b) Hybrid system [8].	18
Figure 2.2. Ultimate load capacities for different rebar configurations [9].	20
Figure 2.3. (a) Preparation of brick masonry wall; (b) and (c) unstrengthened brick masonry wall side view; (d),(e), and (f) details of strengthened reinforced brick wall using timber; (g),(h), and (i) details of strengthened brick masonry wall using timber and hold down anchor [10].	21
Figure 2.4. Comparison of envelope curve according to horizontal load and drift angle for walls [11].	22
Figure 2.5. Dimensional details of masonry specimens (in mm). Influence of double face strengthening is higher on the peak load than on cracking load [12]..	23
Figure 2.6. Dimensional details of masonry walls and different vertical reinforcement configurations (all dimensions are measured in mm) [13] a W1, b W2, c W3/W4, d W5, e W6/W7, and f W8.	25

Figure 2.7. Experimental setup for the masonry walls [13].	26
Figure 2.8. Steel mesh and 4 cm thick Khorasan mortar application [16].	29
Figure 2.9. Dimension of masonry wall specimens and its connection layout for one-sided and two-sided walls (in mm) [10].	30
Figure 3.1. Masonry material properties.	36
Figure 3.2. Mortar material properties.	37
Figure 3.3. Concrete material properties.	37
Figure 3.4 Zinc material properties.	38
Figure 3.5. Macro modeling of shell Model House 1 3-D View.	39
Figure 3.6. Macro Modeling of Shell Model House 1 Top View	40
Figure 3.7. Macro Modeling of Shell Model House 2 3-D View.	41
Figure 3.8. Macro Modeling of Shell Model House 2 Top View.	42
Figure 3.9. Macro Modeling of Shell Model House 3 3-D View.	43
Figure 3.10. Macro Modeling of Shell Model House 3 Top View.	44
Figure 3.11. Wind load assignment.	48
Figure 3.12. C_p assignment for wind X direction at the incoming side	49
Figure 3.13. Interface of the Indonesian Design Spectra website	51
Figure 3.14. Response spectrum value in SAP2000.	52
Figure 3.15. Load case assignment for EQX load.	53
Figure 3.16. Load case assignment for EQY load.	54
Figure 3.17. Mesh sensitivity analysis of house 1 at one of wall's maximum axial stress.	56
Figure 3.18. Mesh sensitivity analysis of house 1 by comparing frequency at both major axes.	57
Figure 3.19. Mesh sensitivity analysis of house 1 by comparing period at both major axes.	57
Figure 3.20. Mesh sensitivity analysis of house 2 at one of wall's maximum axial stress.	58
Figure 3.21. Mesh sensitivity analysis of house 2 by comparing frequency at both major axes.	59

Figure 3.22. Mesh sensitivity analysis of house 2 by comparing period at both major axes.	59
Figure 3.23. Mesh sensitivity analysis of house 3 at one of wall's maximum axial stress.....	60
Figure 3.24. Mesh sensitivity analysis of house 3 by comparing frequency at both major axes.	60
Figure 3.25. Mesh sensitivity analysis of house 3 by comparing period at both major axes.	61
Figure 3.26. Sign Convention in SAP2000 [27].	68
Figure 3.27. House 1 Brick Wall Numbering	69
Figure 3.28. House 1 Axial Stress X direction (S11) General View (1).	70
Figure 3.29. House 1 Axial Stress X direction (S11) General View (2).	70
Figure 3.30. House 1 Axial stress Y direction (S22) general view (1).	71
Figure 3.31. House 1 Axial stress Y direction (S22) general view (2).	71
Figure 3.32. House 1 Shear Stress (S12) general view.	72
Figure 3.33. Out-of-plane failure caused by M22 (left) and M11 (right) [2]	73
Figure 3.34. House 2 Brick Wall Numbering.	78
Figure 3.35. House 2 Axial Stress X Direction (S11) General View (1).	79
Figure 3.36. House 2 Axial stress X direction (S11) general view (2).	79
Figure 3.37. House 2 Axial stress Y direction (S22) general view (1).	80
Figure 3.38. House 2 Axial stress Y direction (S22) general view (2).	81
Figure 3.39. House 2 Shear Stress (S12) general view.	81
Figure 3.40. House 3 Brick Wall numbering	85
Figure 3.41. House 3 Axial Stress X Direction (S11) General View (1).	86
Figure 3.42. House 3 Axial stress X direction (S11) general view (2).	86
Figure 3.43. House 3 Axial stress Y direction (S22) general view (1).	87
Figure 3.44. House 3 Axial stress Y direction (S22) general view (2).	87
Figure 3.45. House 3 Shear Stress (S12) general view.	88
Figure 3.46. Failure lines for $L < 2H$ case.	93
Figure 3.47. Failure lines for $L > 2H$ case.	93

Figure 4.1. Addition of columns (blue) on masonry walls (red) for House 1.	102
Figure 4.2. Location of existing columns (red) and additional columns (blue) at house 1.....	104
Figure 4.3. Columns and Lintels (blue) and masonry walls (red) for House 1.	105
Figure 4.4. Columns and Lintels (blue) and masonry walls (red) for House 1.	107
Figure 4.5. Bending capacity increases due to post-tensioning [9].	110
Figure 4.6. Post tensioning technique using U-Channel and threaded rebars (1).	111
Figure 4.7. Post tensioning technique using U-Channel and threaded rebars (2).	112
Figure 4.8. Post Tensioning effect on Mohr Circle representation [8].	113
Figure 4.9. Addition of columns (blue) on masonry walls (red) for House 2.	116
Figure 4.10. Addition of lintels (blue) on masonry walls (red) at House 2.....	117
Figure 4.11. Addition of columns and lintels (blue) on masonry walls (red) at house 2	119
Figure 4.12. Location of existing columns (red) and additional columns (blue) at house 2.....	121
Figure 4.13. Addition of columns (blue) on masonry walls (red) at house 3.....	124
Figure 4.14. Addition of lintels (blue) on masonry walls (red) at House 3.....	126
Figure 4.15. Addition of columns and lintels (blue) on masonry walls (red) at house 3.	128
Figure 4.16. Location of existing columns (red) and additional columns (blue) at house 3.....	131
Figure 5.1. Recommended stirrups spacing between tie columns and beams [29].	141
Figure 5.2. L-shaped rebar between columns and beams [29].	142
Figure 5.3. T-shaped rebar between columns and beams [29].	142
Figure 5.4. Top view of the bottom beams connecting to a central column [29]. ..	143
Figure 5.5. Mortar layer reinforcement extension along column reinforcement [29].	144
Figure 5.6. Maximum distances between tie columns and perpendicular walls [26].	144

Figure 5.7. Maximum allowable distance for window and door frames to the edges [26].	145
Figure 5.8. Beam rebar extension through the column at the beam and column connection [6].	147
Figure 5.9. Column rebar extension through the beam at the beam and column connection [6].	147
Figure 5.10. Anchor addition at both walls and foundation. [6].....	149
Figure 6.1. $\sigma M11$ comparison of each strengthening techniques at walls which are insufficient for house 1.	154
Figure 6.2. The normalized $\sigma M11$ as a ratio to the experimental stress / strength for each strengthening techniques for walls which are insufficient for house 1.	155
Figure 6.3. $\sigma M22$ comparison of each strengthening techniques at walls which are insufficient for house 1.	157
Figure 6.4. The normalized $\sigma M22$ as a ratio to the experimental stress / strength for each strengthening techniques for walls which are insufficient for house 1.	157
Figure 6.5. $S12$ comparison of each strengthening techniques at walls which are insufficient for house 1.	159
Figure 6.6. The normalized $\tau S12$ as a ratio to the experimental stress / strength for each strengthening techniques for walls which are insufficient for house 1.	159
Figure 6.7. $\sigma M11$ comparison of each strengthening techniques at walls which are insufficient for house 2.	162
Figure 6.8. The normalized $\sigma M11$ as a ratio to the experimental stress / strength for each strengthening techniques for walls which are insufficient for house 2.	162
Figure 6.9. $\sigma M22$ comparison of each strengthening techniques at walls which are insufficient for house 2.	164
Figure 6.10. . The normalized $\sigma M22$ as a ratio to the experimental stress strength for each strengthening techniques for walls which are insufficient for house 2...	164
Figure 6.11. $S12$ comparison at each strengthening techniques for walls which are insufficient for house 2.	166

Figure 6.12. The normalized τS_{12} as a ratio to the experimental stress strength for each strengthening techniques for walls which are insufficient for house 2.	166
Figure 6.13. σM_{11} comparison of each strengthening techniques at walls which are insufficient for house 3.	169
Figure 6.14. The normalized σM_{11} as a ratio to the experimental stress strength for each strengthening techniques for walls which are insufficient for house 3.	169
Figure 6.15. σM_{22} comparison of each strengthening techniques at walls which are insufficient for house 3.	171
Figure 6.16. The normalized σM_{22} as a ratio to the experimental stress strength for each strengthening techniques for walls which are insufficient for house 3.	171
Figure 6.17. S_{12} comparison of each strengthening techniques at walls which are insufficient for house 3.	173
Figure 6.18. The normalized τS_{12} as a ratio to the experimental stress strength for each strengthening techniques for walls which are insufficient for house 3.	173
Figure 6.19. Improvement percentage for σM_{11}	177
Figure 6.20. Improvement percentage for σM_{22}	178
Figure 6.21. Improvement percentage for τS_{12}	178

LIST OF SYMBOLS

SYMBOLS

P	Design wind pressure (N/m ²)
q	Incoming wind velocity pressure (q_z) measured at height z or other wall/roof velocity pressure (q_h) measured at height h (N/m ²)
q_i	Velocity pressure to evaluate the positive/negative internal pressure on all surfaces. The value is conservatively taken as (q_h) measured at a height h (N/m ²)
G	Coefficient factor due to wind blow (taken as 0.85)
C_p	Coefficient due to external pressure
(GC_{pi})	Coefficient due to internal pressure (for a fully closed building the value is taken as 0.18)
V	Base wind velocity (taken as 40 m/s)
K_d	Coefficient due to wind direction factor (taken as 0.85)
K_{zt}	Coefficient factor due to topography factor
K_z/K_h	Coefficient factor due to velocity pressure
σ_x	Axial stress in X direction (S11) (MPa)
σ_y	Axial stress in Y direction (S22) (MPa)
τ_{xy}	Shear stress (S12) (MPa)
σ_{max}	Max axial stress (MPa)
σ_{min}	Min axial stress (MPa)
τ_{max}	Max principal Shear stress (MPa)
M11	Max bending moment applied to the wall along 1-1 plane (kNm)
M22	Max bending moment applied to the wall along 2-2 plane (kNm)
T	Thickness of wall (m)
L	Length of wall (m)
H	Height of wall (m)
σ_{M11}	Flexural tensile strength at 1-1 Plane (MPa)

σ_{M22}	Flexural tensile strength at 2-2 Plane (MPa)
δ_{xe}	Deflection at the required location based on elastic analysis.
I_e	Importance factor based on section 11.5.1 ASCE 7-16
C_d	Deflection amplification factor based on table 12.2-1 ASCE 7-16



CHAPTER 1

INTRODUCTION

1.1 Masonry House Construction in Indonesia

Indonesia is located between convergence of three major plates- the Sunda Block, Pacific, and Australian Plates while there exist at least four more minor plates that creates a complex network of around 18,000 km length active plate boundaries. The tectonic plate boundaries are shown in Figure 1.1, where black “toothed” curves indicate active subduction zones, and red lines are active faults. This location makes it one of the world’s most seismically active countries in the world while ranking as 4th most populated country in the world. Moreover, most of the houses built in the country were made from masonry as the main structural materials and many of them are considered as non-engineered buildings, which are buildings that are traditionally constructed using locally available materials without any or insufficient interference from professional Architects or Civil Engineer in their design. Considering the continuous use of this practice in Indonesia, reducing the collapse risk after an earthquake is extremely necessary to save lives either by retrofitting methods for the existing masonry houses or applying strengthening techniques for the construction of the new masonry houses.

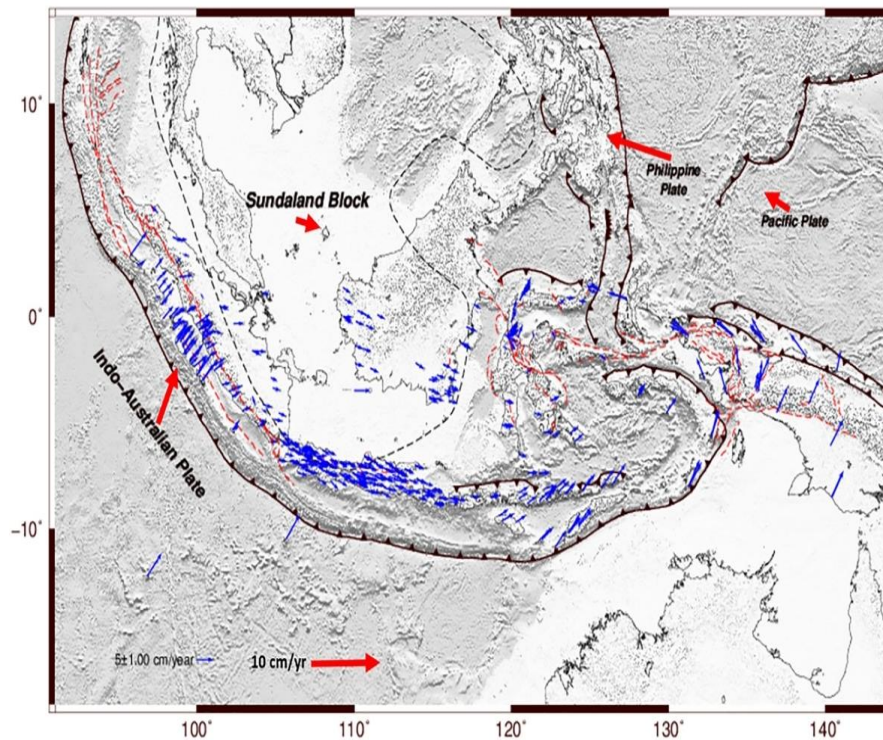


Figure 1.1. Tectonic plate boundaries of Indonesia and its surrounding region using crustal relative motion from GPS data (blue arrows) up to 2016 [1].

Masonry is considered as one of the oldest construction types that are still used in modern construction. The masonry term itself is applied to any building systems that are constructed by stacking small units of various materials with the most common ones are stone, clay, or concrete. These small units are then combined using mortar to form walls, columns, beams, domes, or arches. In addition to this, masonry is known for its fire resistance, durability, thermal insulation, and widespread availability. However, regarding its seismic response and design properties, limited research is available compared to reinforced concrete and steel. Thus, this thesis aims to give additional contribution to masonry-related research by focusing on a

strengthening technique for three diverse types of one-floor non-engineered masonry houses that are located at Aceh Province Indonesia using some strengthening techniques at masonry walls to improve its earthquake-resistant capacity. Moreover, the retrofitting or strengthening technique that will be applied also preferably uses readily available materials with relatively straightforward installation for these methods to be installed properly and effectively due to lack of skilled worker and limited tools availability.

One of the masonry houses is shown in Figure 1.2. It is constructed as a confined masonry structure in which additional tie beams and columns are used and these will increase the structure's lateral capacity while keeping the masonry wall together under earthquakes. However, for confined masonry structure these tie columns and tie beams are assumed to not distribute any load and majority of the loads are resisted by the masonry walls. During earthquakes, the wall perpendicular to the direction of earthquake acceleration is weaker and more flexible than the walls parallel to the direction of shaking. Additionally, both in-plane failure modes and out-of-plane failure modes are possible in which in-plane failure modes occur with shorter walls where the ratio between lateral support distance and wall thickness is relatively small, while out-of-plane failure modes are a concern for slender walls.



Figure 1.2. One of non-engineered masonry house front view.

1.2 Materials used for Confined Masonry Construction in Indonesia

1.2.1 Clay Bricks

Bricks can be made of different materials such as clay, concrete, adobe, or stone and can be hollow or solid. In Indonesia, solid clay bricks are mainly used for one-story confined masonry construction. This brick is produced by mixing clay with water and putting it into

brick mold. The mold has been designed according to the desired dimension (around 20x10x5 cm).

After bricks have been shaped according to the desired dimensions, they are dried under sunlight for around 2-7 days according to the weather and bricks' condition. After it is dry enough, it is heated in a furnace, and it takes around 7 days to heat 50,000 clay bricks. Once heated adequately, it is left outside for approximately 1 day before it is ready to be used.

The mechanical properties of the clay brick consist of the following:

1. Compressive Strength

The compressive strength of masonry bricks varies based on masonry materials, void ratio, aspect ratio, and production method. According to a direct compression test [2], the traditional Clay brick method has the highest compressive strength value of 4.69 MPa.

2. Tensile and Shear Strength

A standard testing procedure for the determination of tensile strength of masonry is not available in Indonesian code. However, an experiment has been done to determine the tensile and shear strength of traditionally made clay bricks [3]. This experiment gives the tensile and shear strength of traditional clay bricks as 2.03 kg/cm² (0.199 MPa) and 3.04 kg/cm² (0.298 MPa), respectively.

3. Flexural Strength

For checking the flexural strength of masonry, point load tests were used on a single clay brick [2] and the average flexure strength is found as 1.803 MPa. Moreover, a wall consisting of bricks and mortar was assessed using a point load test and the average flexural strength found before the wall cracked was found to be 0.408 MPa.

4. Modulus of Elasticity

The modulus of elasticity describes clay brick's resistance to deform when subjected to applied load. For obtaining the modulus of elasticity for clay brick, cylindrical compressometer and dial gauge was used to find longitudinal deformation when clay brick reached 40% of its maximum compressive strength, the result is obtained as 3201.86 MPa [4].

1.2.2 Mortars

Mortar is used to bind bricks together. This material is made from an appropriate ratio of Portland cement, sand, lime, and water. In Indonesia, the sand and Portland cement ratio is 3:1, while 1 bag of mortar will be mixed with 11 liters of water. It should be noted that wet mix makes it easier to apply while dry mix makes it have a stronger bond; thus, experience is also needed when mixing mortar. In addition to that, lime is used to accelerate the hardening of mortars and increase the strength of mortars. In this scenario, the recommended sand, lime, and cement ratio is 6:2:1. One of the standard methods to test whether the mortar is at a suitable viscosity is by taking a fair amount of mortar using a small shovel and putting it at 90° so the mortar mix should not fall. The compressive

strength test was done according to SNI 03-2156-1991 [2], and the mortar compressive strength was found as 8.167 MPa.

1.2.3 Tie Columns and Beams

In addition to bricks and mortars, the main components of confined masonry structures are tie columns and beams. In this thesis, the column and upper beam cross-section is 15 x 15 cm with 4 pieces of 10 mm diameter steel bars for longitudinal reinforcements and 6 mm diameter steel bars with 15 cm spacing for stirrups. Moreover, the bottom beam cross-section is 15 x 20 cm with 4 pieces of 10 mm diameter steel bars for longitudinal reinforcements and 6 mm diameter steel bars with 15 cm spacing for stirrups.

The primary function of having confined columns and beams in this masonry house is to enhance the stability and integrity of masonry walls when subjected to in-plane and out-of-plane earthquake forces while reducing the brittleness of masonry walls. However, considering the damage that occurred after the earthquake still significantly affects this type of masonry structure, further strengthening is deemed necessary.

1.3 Failure Modes of Masonry Walls

1.3.1 In-Plane Failure Mechanism

In terms of in-plane action, there are three failure mechanisms which can be observed:

1. Sliding Shear
2. Diagonal Tension
3. Flexure

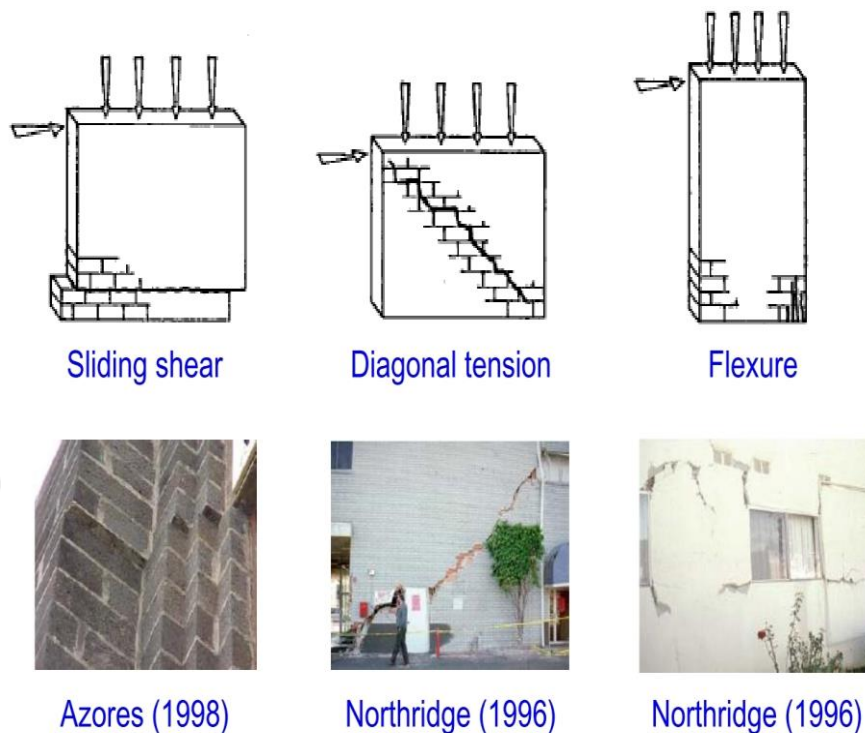


Figure 1.3. In-Plane failure mechanism of masonry wall [5].

Sliding shear usually occurs due to low-quality mortar usage and low magnitude of axial load, which results in the sliding of the upper part of the wall on the horizontal mortar joints. Moreover, it usually occurs at the upper story of a building where the vertical loads are low while lateral loads are high.

Diagonal tension is observed to be the most common failure mechanism of masonry walls under seismic load for in-plane failure. This failure happens due to a combination of vertical and horizontal loads where principal tensile stresses developed in the wall surpass the tensile strength of masonry and it is characterized by an X-shape and can propagate along mortar joints, masonry units, or both.

In-plane flexural failure occurs when shear resistance is improved with a high moment to shear ratio which causes the crushing of compressed zones at the ends of the wall. It is rarely observed for masonry buildings because of the low ratio between tensile and compressive strength of masonry materials and structural characteristics of masonry (low moment to shear ratio), which cause predominant shear-type behavior when subjected to earthquake loads at in-plane direction.

1.3.2 Out-of-Plane Failure Mechanism

The out-of-plane behavior of masonry walls largely depends on floor-wall connections. When the walls are appropriately connected to the floors, out-of-plane behavior is usually not critical. Moreover, depending on the boundary conditions, connection types, and properties, different types of bending action that leads to out-of-plane behavior are as follows:

1. Vertical Bending
2. Horizontal Bending
3. Two-way diagonal bending

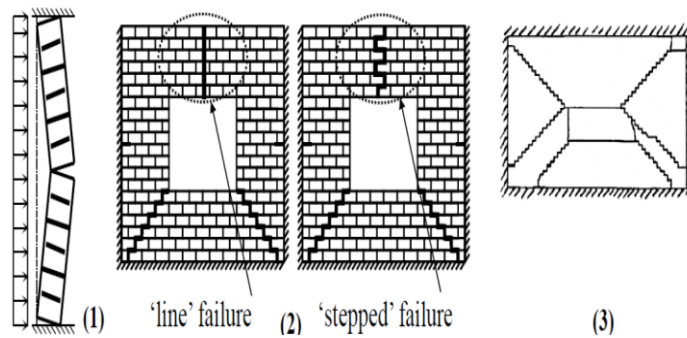


Figure 1.4. Bending action type which causes out-of-plane failure [5].

Vertical bending is the most common type of out-of-plane failure, and it occurs due to poor wall-to-wall connections. Furthermore, it can be assumed that the stress-strain relationship is linear until failure (elastic-brittle behavior) and section is assumed to fail when tensile strength reaches the outermost fiber. During this failure, the crack will start at the mid height and base for the walls supported at both top and bottom whereas walls supported at the bottom only will initially have crack at the base only.

Horizontal bending occurs when a wall-to-floor connection is poor. Moreover, since both units and mortars may fail at their ultimate state, a complex distribution of stresses between brick units and mortars is expected. In horizontal bending, there are two failure modes, shown in Figure 1.4.

Two-way diagonal bending is the most complex out-of-plane behavior in which both wall-to-wall and wall-to-floor connections support masonry walls. In the two-way diagonal bending case, the virtual work method can be the best solution because it performs well for different wall geometries, perforations, etc.) [5].

1.3.3 Failure Mechanisms of Confined Masonry Walls with No Diaphragm Action

The failure mechanism of confined masonry walls without diaphragm action relies heavily on the stiffness and strength of the walls without additional stiffness that is provided by horizontal element such as slab which cause the walls to become more vulnerable when subjected to lateral forces especially in out-of-plane direction which can lead to larger deformation compared to the walls with diaphragm action. Furthermore, confining elements such as tie columns and tie beams have small cross sections (15 x 15 cm) with a relatively long dimension which causes these members to be quite flexible and cannot completely act as lateral support for masonry walls.

As a result, confined masonry walls without diaphragm action can fail due to overturning (crack at the base of the wall and overturning of the wall due to excessive moments associated with lateral loads such as wind or seismic forces) which can lead to partial walls collapse or total collapse. Moreover, lack of anchors between tie columns and walls can cause separation and diagonal crack at the walls at a large earthquake excitation, the observed damage at these confined masonry houses without diaphragm action will be explained in section 1.4.

1.4 Typical Observed Damage for Non-Engineered Masonry House in Indonesia

According to field observations that have been done for the last 35 years of post-earthquake response of non-engineered confined masonry houses in Indonesia, both in-plane and out-of-plane failure of walls cause damage or collapse to the walls. However, according to T. Boen [6], the main cause of

the EQ damage is dominantly the out-of-plane loading. The following typical damages are observed as follows:

1. For small magnitude earthquakes, the walls may stay intact; however, especially for roofs with steep slopes, it might have major roof tiles dislocations as seen in Figure 1.5. Although this damage is not structural, it may still hurt people and cause life losses if the heavy tiles fall on heads of occupants.



KARANGASEM EQ, JANUARY 1, 2004



YOGYAKARTA EQ, MAY 27, 2006

Figure 1.5. Roof tiles dislocation [6].

2. For larger earthquake excitations, it was observed that the infill walls are separated from the confinement columns. This type of failure is dominantly seen when there are no anchors between the infill wall and the columns. The separated infill may also start to form diagonal cracks as it can be seen in Figure 1.6.



PADANG PANJANG EQ, FEBRUARY 16, 2004



WEST SUMATRA EQ, SEPTEMBER 30, 2009

Figure 1.6. Separation and cracking of infill walls [6].

3. For larger magnitude of earthquake, lack of concrete slab at the upper part of the wall causes out of plane failure of the masonry walls as shown in left figure in Figure 1.7 while the right figure show total collapse due to out of plane failure.



HALMAHERA EQ, JANUARY 21, 1994



BENGKULU EQ, SEPTEMBER 12, 2007

Figure 1.7. Out of plane failure of the masonry walls [6].

4. Openings on the walls are necessary for windows and doors addition. However, these openings reduce in-plane and out-of-plane resistance of walls because it reduces the overall walls' cross-section and cause stress concentrations at corners which can initiate formation of diagonal crack as shown in Figure 1.8.



SUKABUMI EQ, 1982



WEST SUMATRA EQ, SEPTEMBER 30, 2009

Figure 1.8. Failure at corners of openings [6].

5. When subjected to both vertical and horizontal loads during earthquake the tensile strength of masonry or mortar might be exceeded. In this case,

diagonal cracks can start to develop at the masonry wall as shown in Figure 1.9.



LIWA EQ, FEBRUARY 16, 1994



WEST-JAVA EQ, SEPTEMBER 2, 2009

Figure 1.9 Diagonal cracks in walls [6].

6. For a large magnitude earthquake, some of the walls which are wider, have inadequate mortar and brick material qualities, lack of concrete slab and anchor connection between tie columns and masonry wall. Some of the walls might collapse completely as shown in Figure 1.10.



BALI EQ, JULY 1976



SUMBAWA, NOVEMBER 25, 2007

Figure 1.10. Total collapse at some of the walls [6].

7. For extremely large magnitude of earthquakes in addition to worse workmanship and structural properties, the houses may experience total house collapse, which is the worst failure mode of them all. The occupants are likely to experience life and property losses as shown in Figure 1.11.



MANOKWARI EQ, JANUARY 4, 2009



WEST SUMATRA EQ, SEPTEMBER 30, 2009

Figure 1.11. Total house collapse [6].

1.5 Objective and Scope

The case studies used in this thesis are specifically the masonry houses built in Banda Aceh region. The proposed strengthening techniques are explained in the subsequent sections and the main objectives of this thesis are summarized as follows:

- Evaluate the behavior of non-engineered confined masonry houses in Indonesia against EQ loads and wind / rain load combinations using SAP2000 as linear models.
- Investigate the effect of strengthening techniques on the non-engineered masonry houses.
 1. Post-tensioning (proposed),
 2. Addition of lintels,
 3. Additional tie columns inside brick walls,
 4. Chicken wire mesh and OSB covering (discussed).
- Develop the most optimal and cost-efficient strengthening technique for non-engineered masonry houses.
- Provide guidelines regarding safer housing for Indonesian citizens that are simple and low-cost to apply.

Furthermore, the scope of the study can be explained as follows:

- Mesh Sensitivity Analyses of 3 house types.
- Finite Element Modeling of houses
 1. Nominal (as built) models.
 2. Addition of one layer of lintels.
 3. Addition of extra in-wall columns for long walls.
 4. Combination of additional lintels and additional columns.
 5. Analytical calculations for vertical post-tensioning of walls.
- Application of yield-line theory to calculate wall failure (M_{cr}).
- Listing minimum requirements and recommendations
- Discussion of results and conclusions.

CHAPTER 2

LITERATURE REVIEW

Around the world, many strengthening techniques have been proposed for both masonry buildings and masonry walls using different materials alternatives which are proven to be effective at increasing load carrying capacity, deformability, energy dissipation capacity etc. Some of the strengthening techniques which have been done are summarize as follows:

2.1 Strengthening of Masonry Wall Using Bamboo Component

A study was done by using different configurations of bamboo to strengthen masonry walls [7]. Bamboo was chosen as the material because bamboo is a sustainable material, and it has been proved to have comparable mechanical properties compared to conventional building materials.

The test consists of 6 unconfined masonry walls with an additional 4 confined masonry walls including reference wall for each of them. Moreover, different configuration of bamboos placement such as bamboo strips, bamboo ties and bamboo mats were applied to the walls while the walls are subjected to pre-compression of 305 kN (equivalent to uniform pressure of 0.6 MPa) was applied to the top of each wall through top beam using hydraulic rams. This load can be assumed as load simulation from structural components above the wall and located at lowermost of six-story building. In addition to this, cyclic lateral load was also applied and increased gradually by 30 kN increments to simulate earthquake loads.

From this test, it was concluded that bamboo grid reinforced cement mortar can enhance energy dissipation capacity, deformability, and lateral load carrying capacity while the wall strengthened with bamboo reinforcement is superior in deformability but has similar lateral strength compared to other wall strengthened with half amount of bamboo reinforcement. Moreover, strengthening techniques using bamboo mats are also proven to enhance energy dissipation capacity and deformability even though it is not as effective as bamboo grid reinforced cement mortar. Lastly, confining a masonry wall having confined concrete end columns with bamboo bars placed in mortar layers also has higher initial stiffness and ductility increase.

2.2 Strengthening of Masonry House Wall Using Scrap Tires

In another research, scrap tires were used to strengthened masonry house wall [8]. in which 43 scrap tire rings from 9 different brands were used and these tire rings were applied to 6 strip walls (4 brick walls and 2 briquette walls) while applying post tensioning loads with both scrap tire tread ring chain and hybrid configurations as shown in Figure 2.1.

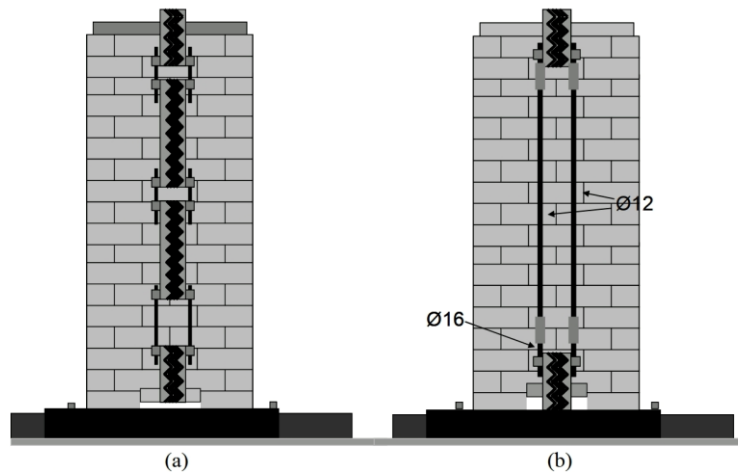


Figure 2.1. Test configuration for strip walls where (a). Using STR Chains and (b) Hybrid system [8].

From the test result, out-of-plane capacity of brick and briquette walls increased 9 and 5 times compared to original capacities, respectively.

In addition to this, two-full scale traditional masonry houses were tested using a tilting table and the experiment showed that capacity of strengthened house increase 75% compares to unstrengthened one.

2.3 In-Plane Seismic Strengthening of Masonry Wall Using Rebars

Moreover, there is another research in which masonry walls were strengthened by rebars for in-plane seismic strengthening [9]. Different configurations of rebars position were applied and in-plane seismic forces were applied using a hydraulic piston. In addition to that, this research tested the effect of using rebars for both 2 m long single brick masonry wall and 6 m long half scale brick masonry house. From the experiment, the ultimate lateral load capacity of 6 m long brick masonry house was increased up to 6 times compares to original value while energy dissipation capacity also increased 10 times more than original house. In addition to that, for the 2 m long brick URM wall, the nominal capacity of walls with combination of vertical, horizontal, and diagonal rebar post-tensioning has an improvement up to 17 times original values. The comparison of ultimate load capacities from different configurations is shown in Figure 2.2.

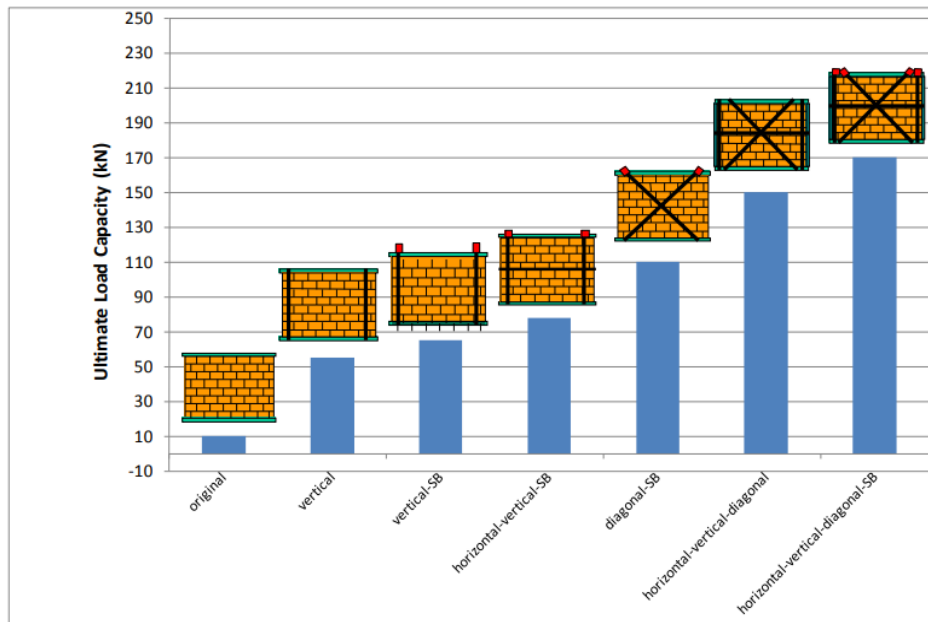


Figure 2.2. Ultimate load capacities for different rebar configurations [9].

2.4 In-Plane Strengthening of Wall Using Timber

In another research, timber was used to enhance in-plane shear strength and deformation capacity [10] by comparing typical brick wall (BW wall), reinforced brick wall with timber (BW-T wall) and reinforced brick wall strengthened with timber wall and addition of hold down anchor (BW-TA wall) as shown in Figure 2.3.



Figure 2.3. (a) Preparation of brick masonry wall; (b) and (c) unstrengthened brick masonry wall side view; (d),(e), and (f) details of strengthened reinforced brick wall using timber; (g),(h), and (i) details of strengthened brick masonry wall using timber and hold down anchor [10].

The combination of static vertical load at the upper part of the masonry wall and cyclic horizontal load at the top corner of the wall was applied with an additional pre-compression stress of 0.2 MPa. It was observed that BW-T wall failed at the lowest part of wall, but it can maintain its original shape compared to BW wall while the horizontal load capacity does not improve.

Moreover, when BW-TA wall was tested, max horizontal load capacity increased by 22% while drift angle calculated from diagonal measurement increased by 4.6 times. Moreover, experimental results show that plywood panels can withstand diagonal tension Forces better than timber Frames. For BW-T wall, first cracks originated at bottom of wall corner and uplift occur between lowest mortar and bottom of lowest brick, it was found that wooden wall did not improve horizontal load of BW-T during cyclic load, but it maintained original shape of brick wall. For BW-TA wall, it was not uplifted until failure mode and recorded displacements in vertical direction were small, as the load increase, stepped diagonal cracks are formed and vertical cracks are formed in mortar joint.

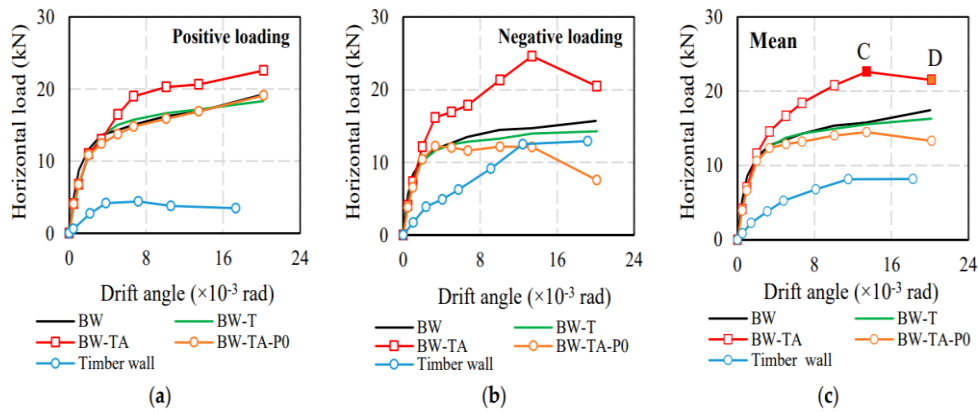


Figure 2.4. Comparison of envelope curve according to horizontal load and drift angle for walls [11].

The horizontal load of BW-TA wall is 14.8% (36.2%) higher than BW wall in positive (negative) loading and it is 22% greater in mean result at maximum load. The combined effect of timber wall and anchor started when flexural fracture emerged. However, it was observed that there is no initial stiffness increase after strengthening while the hold-down anchor in BW-TA wall can decrease the vertical tensile cracking of the wall.

2.5 Surface Strengthening of Masonry Wall using Mortar Cross Strips

Another strengthening technique that can be done is by surface reinforcement, from experimental study on seismic behavior of masonry walls strengthened by reinforced mortar cross strips [12]. The dimensional details of masonry specimens are shown in Figure 2.5. The test consists of three unreinforced masonry walls (URM) as reference walls and eight masonry walls strengthened using reinforced mortar (RM) cross strips. These eight strengthened masonry walls were differed based on mortar grade used, strip thickness, diameter of steel rebars, and reinforcement types (double-face or single-face strengthening).

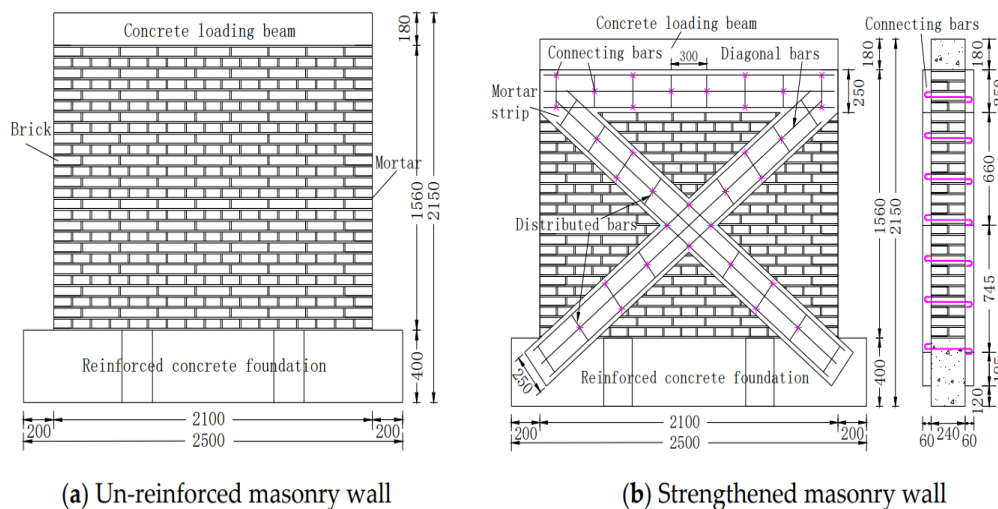


Figure 2.5. Dimensional details of masonry specimens (in mm). Influence of double face strengthening is higher on the peak load than on cracking load [12].

Both reference walls and strengthened walls were subjected to both lateral cyclic load and vertical load on the top face. Initially, vertical load was applied using 1000 kN capacity actuator while the lateral cyclic load was applied using 500 kN capacity actuator according to Chinese Code JGJ/T

101-2015 [12], the control method consists of 4 minutes duration of reversed force with 20 kN increment at each cycle until the first crack was observed and displacement cycles with increment of cracking displacement value. For each increment in displacement cycle, three cycles of same magnitude were applied, and the test terminated when recorded lateral force had decreased to 85% of peak load. By using reinforced mortar cross strips, shear capacity of brick wall increased by 38.2% and reinforcement ratio noted to be the key factor to influence shear capacity. Moreover, mortar cross strips have negligible stiffness and mass compared to the original structure with low cost, minimal impact and easy construction while the load which can initiate crack of strengthened walls increased by 20.1-42% and ultimate load improved by 38.2-65%. In addition to that, the influence of double face strengthening is higher on the peak load than on cracking load while ductility improves 1.6 times for single-face and 2.8 times for double-face shear failure remain the main failure of RM strengthened walls. Additionally, increase of depth-width ratio will decrease shear capacity and depth-width ratio has more significant effect compared to strength grade of masonry mortar.

2.6 Partially Grouted Reinforced Clay Masonry Walls

Moreover, another research explores the possibility of strengthening brick wall by creating holes in the brick and using vertical reinforcement throughout the bricks and mortar [13]. 8 walls were constructed using multi-perforated clay bricks and horizontally reinforced with ladder-type bed-joint reinforcement as shown in Figure 2.6 and the test setup is shown in Figure 2.7. All walls except W3 and W6 were subjected to axial stress of 0.55 MPa and axial stress was kept constant while in-plane cyclic horizontal displacement was applied incrementally until collapse. All tests except W8 were terminated when lateral load of the specimen had dropped to 80% of maximum value. For W8, after reaching its maximum strength, it is

diagonally retrofitted with Carbon Fiber Reinforce Polymer (CFRP) strip to observe its effectiveness in improving shear capacity of wall.

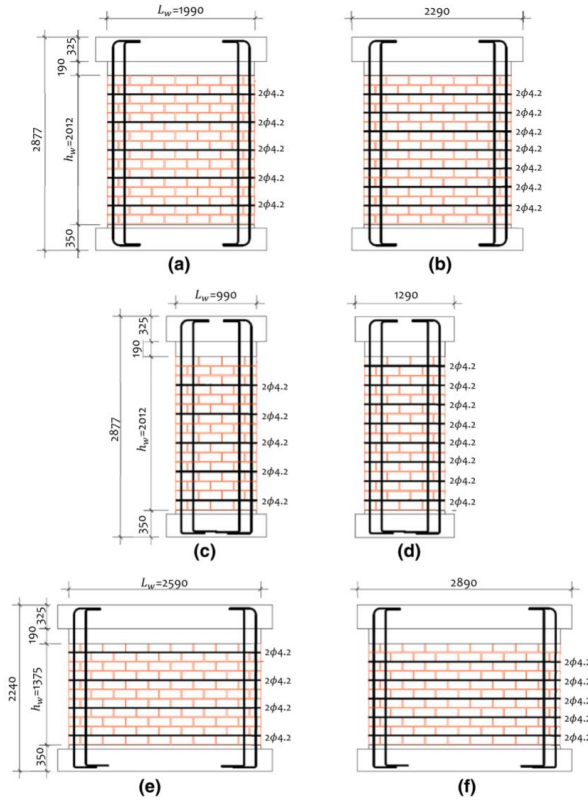


Figure 2.6. Dimensional details of masonry walls and different vertical reinforcement configurations (all dimensions are measured in mm) [13] a W1, b W2, c W3/W4, d W5, e W6/W7, and f W8.

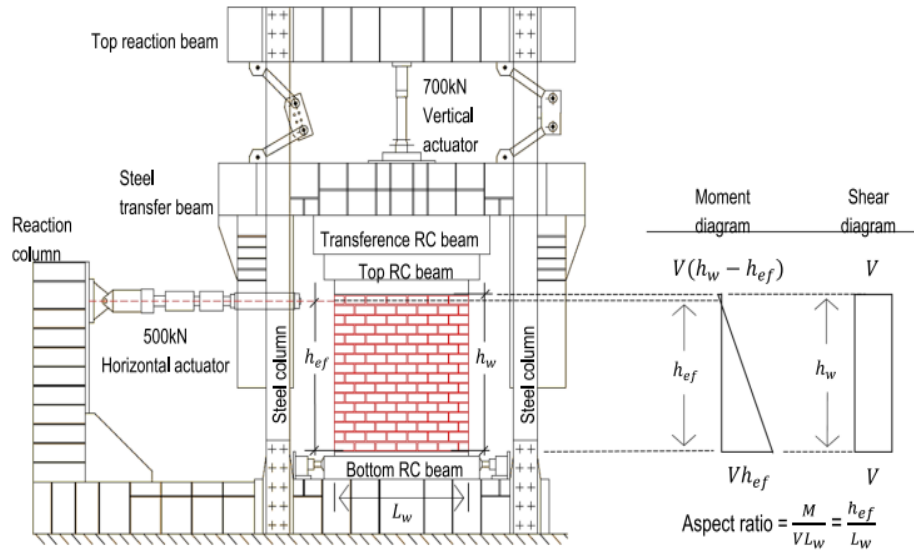


Figure 2.7. Experimental setup for the masonry walls [13].

The study proved that as the horizontal reinforcement ratio increases, lateral capacity of walls increases especially for square and slender walls. Moreover, axial load influence becomes more relevant as aspect ratio decreases. From the test result, it is observed that when the wall has higher horizontal reinforcing bars it will have more distributed crack pattern. Moreover, walls' maximum resistance was negatively impacted by a significant diagonal crack's abrupt growth in size and thickness in one of the load directions. In terms of drift, the average drift at maximum load increases as aspect ratio also increases. While regarding the shear capacity of walls, maximum lateral load decreases when aspect ratio increases. Moreover, aspect ratio of walls was found to be the primary design parameter influencing stiffness degradation.

2.7 Non-Engineered Masonry House Finite Element Modelling

In a thesis regarding one story non-engineered masonry house modeled using different types of bricks (hand-made bricks and factory-made bricks) [14]. The house was modeled using SAP2000 and the analysis results were

compared with the experimental results. The masonry wall was modelled as shell-thin elements and load combinations which include earthquake load, wind load, dead load and live load were applied to the confine masonry house model. From the experimental result it was found that the hand-made clay brick wall has much less compressive and tensile strength compares to factory made brick with comparable shear strength (hand-made brick has 0.832 MPa, 0.203 MPa and 0.304 MPa of compressive strength, tensile strength, and shear strength respectively while factory-made brick has 12 MPa, 1.67 MPa and 0.35 MPa respectively).

From the SAP2000 analysis result it was obtained that the maximum principal stress for hand-made clay brick was 0.136 MPa with maximum shear stress of 0.0542 MPa. These maximum principal stresses and maximum shear stresses are less than hand-made brick tensile and shear capacity. For the factory-made brick, the obtained maximum principal stresses and shear stress were obtained as 0.1922 MPa and 0.07225 MPa respectively. These analyses results are less than factory-made brick's tensile and shear capacity and it was concluded that the building can withstand the earthquake load which might occur in the region.

2.8 Retrofit of Masonry Walls Using Chicken Wire Mesh

Chicken wire mesh is a retrofit technique applicable to strengthen diagonal cracks due to diagonal shear failure after earthquake, this diagonal shear failure occurs because of tensile strength failure at clay brick. Moreover, chicken wire mesh is recommended to use for cracks larger than 5 mm up to 40% of damage at walls [15] ,this retrofit technique is more economical because it can be done without demolishing walls and chicken wire mesh can be easily obtained.

The study was done by testing compressive and tensile strength of brick wall, plaster, and testing the tensile strength of chicken wire mesh. Moreover, the study modeled both a single brick wall with different grid sizes of chicken wire mesh and a 3-story masonry house using a finite element model. As a result, chicken wire mesh was able to regain the initial compressive strength of brick wall by 1.5 times the initial values and regained initial tensile strength of brick wall to its original values. In addition to that, plaster addition does not regain the initial tensile strength of brick wall but improves its compressive strength.

An experimental study was conducted for assessment of retrofit technique using one sided chicken wire mesh and covered by Khorasan Mortar [16]. In this study, 3 unstrengthened walls and 3 walls strengthened with Khorasan mortar + steel mesh on one side were tested and the steel mesh + 4 cm anchor application is shown in Figure 2.8.



Figure 2.8. Steel mesh and 4 cm thick Khorasan mortar application [16].

The displacements are checked using 6 displacement meters (LVDT) (k1 and k2 at horizontal loading side, k3 at the vertical loading side, k4 diagonally while k5 and k6 at the opposite side). From the experimental results, average peak load obtained from failure of unreinforced wall was 33.28 kN while for strengthened wall 105.07 kN which represents 215.73% increase compared to unstrengthen wall. Furthermore, max displacement for unreinforced wall has 23.88 mm displacement whereas the strengthened wall has 35.53 mm displacement which represents a 48.82% increase at the displacement. For the shear strength, the shear strength for unreinforced wall was 0.25 MPa whereas the strengthened wall has 0.54 MPa which represents 118.58% increase in shear strength.

2.9 Masonry Wall's Out-of-Plane Strength after Retrofitted with OSB

A study compared unstrengthen wall with one sided strengthened masonry wall and two-sided strengthened masonry wall using OSB as shown in Figure 2.9. Masonry wall dimension is 1115 x 1115 x 215 mm, and three experimental phases were done: experimental procedures to obtain masonry components properties, out-of-plane flexural bond strength tests and out of plane bending test by four-point loading test on all walls [10].

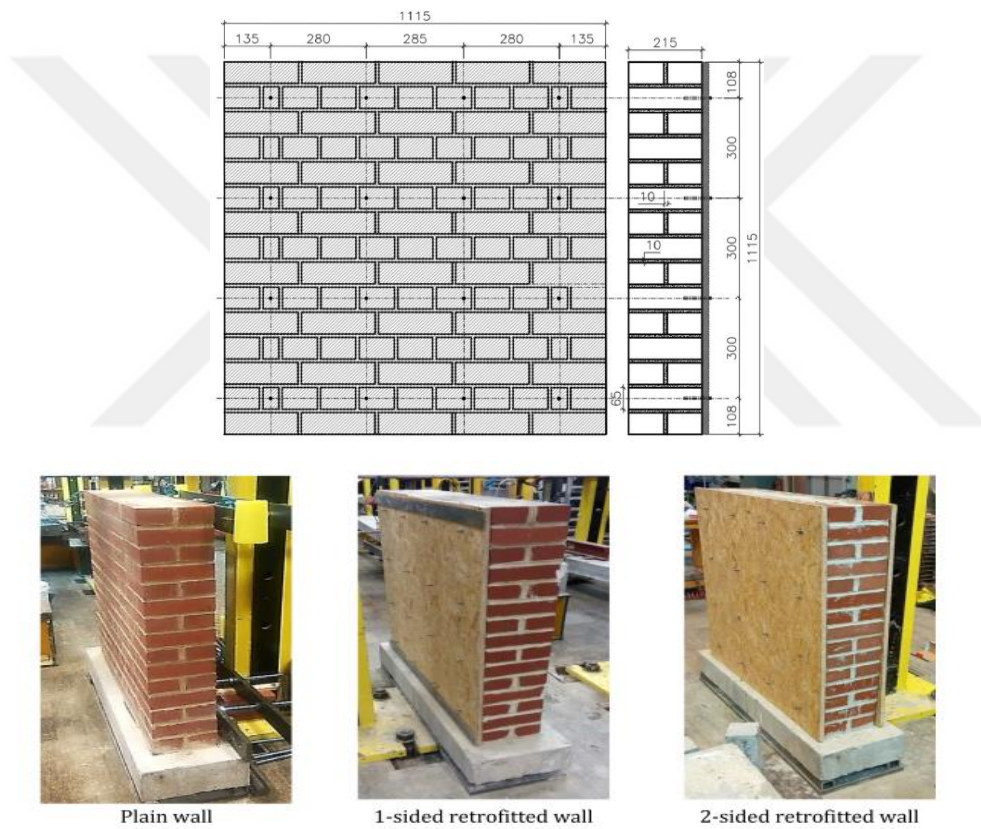


Figure 2.9. Dimension of masonry wall specimens and its connection layout for one-sided and two-sided walls (in mm) [10].

From the experimental results, one sided OSB can increase out-of-plane loading of wall by 1.4 times and two sided of masonry wall can increase out-of-plane loading by 1.8 times before initial crack occurs. Moreover, it is also

observed that deflection capacity of double-sided wall is higher than both unstrengthen and one-sided wall.

For the failure pattern, it is observed that failure of the unstrengthen wall was cracks opening at mortar joint at walls' middle height while the OSB application causes formation of horizontal cracks at middle third of walls after higher load was applied and collapse occurred when the walls reached OSB's ultimate strain. In addition to that, compared to fiber-based retrofit technique, the cost of applying OSB is around 30% cheaper.

2.10 Tensile Strength in Bending Comparison between Clay Brick and Aerated Brick

The research aims to compare the flexural strength of clay made brick wall and aerated brick (made from fly ash, cement, lime, and aluminum powder) using experimental procedures [2]. The experiment was done according to ASTM E-72 which tested the strength of panels that are used in construction. The panel used has both width and height of 120 cm.

The flexural strength test is used to measure walls' behavior towards perpendicular force i.e. out-of-plane failure. Moreover, this research also analyzed the effect of adding galvanized wire horizontally between light weight bricks to both clay bricks and light weight bricks walls.

The flexural strength results are given in Table 1

Table 1. Flexural Test Analysis Results [2].

No	Wall Type	load (kN)	Deflection (mm)	Flexural Strength (MPa)
1	Clay Brick Wall	2.97	1.653	0.408
2	Light Brick Wall	1.718	1.369	0.42
3	Light Brick+1 wire	1.734	1.811	0.424
4	Light Brick+3 wire	2.235	1.875	0.546
5	Light Brick+5 wire	1.768	1.409	0.432

The table shows that clay brick walls can resist the highest load compared to light weight brick and light weight bricks with additional galvanized wire. Moreover, the test result also shows that additional wire can improve deflection capacity of the wall but when there are too many wires added the deflection decreases. The reason for this is excessive addition of galvanized wire can cause a significant decrease in contact area between bricks and mortar and 3 additional wires can be considered as the optimum addition for light weight brick [2].

2.11 Evaluation of Confined Masonry Houses Guidelines

Non-engineered confined masonry houses can be found in many countries with high risk of seismic hazard. These countries include India, Mexico, Colombia, Peru, Argentina, Japan, China, Cambodia, Indonesia etc. [17]. The reasons why confined masonry houses are still used in these countries despite the significant seismic hazards are:

1. Methods and techniques used in confined masonry construction are very similar to techniques used in unreinforced masonry construction which were the common practices before the introduction of confined masonry construction techniques.

2. The confined masonry house construction is only marginally expensive compared to unreinforced masonry houses.
3. Confined masonry houses exhibit better seismic performance than conventional unreinforced masonry houses.

To ensure a proper detailing and determination of each member's cross section in non-engineered confined masonry houses, guidelines have been developed which are based on available codes from different countries and simplifying complex codes' design equations to recommendations and easily applicable rules. However, guidelines are usually region dependent and not applicable outside of the specified region.

Moreover, shortcomings of current confined masonry housing can be categorized into:

1. Seismic design flaws.

Consist of irregular and unsymmetrical distribution of masonry walls at both principal directions which causes torsion under earthquake and increases lateral forces at some in-plane walls. These plan irregularities are observed in Iran, Peru, and Kyrgyzstan. In addition to that, insufficient confining elements which consist of excessive spacing between tie columns and lack of tie columns at some of the wall ends and intersections.

2. Construction and post-construction.

These include material quality far below the defined value in the codes, poor workmanship, and structural components modification without consulting experienced engineers. These shortcomings are largely observed in Chile, Iran, and Kyrgyzstan.

3. Lack of regulation.

Unfortunately, there is a major deficit in regulation regarding the inspection systems for confined masonry houses to assess bad workmanship, material,

and connection detailing which cause houses that are severely damage even in lower magnitude earthquakes.



CHAPTER 3

UNSTRENGTHENED CONFINED MASONRY HOUSE MODELING

3.1 Modelling of the Unstrengthened Confined Masonry Houses

The corresponding houses are modeled based on the engineering drawing provided by Showbitz company, which is located at Banda Aceh city, Aceh province in Indonesia. The first and third houses are 1-story houses with 1200x900 cm dimensions, consisting of 2 symmetrical houses with different wall configurations. The second house is a smaller house with 600x800 cm dimensions with shorter height (3.15 m) compared to the other houses (3.6 m) but using a higher roof slope 20° compared to 10° for houses 1 and 3). Initially, materials that consist of masonry, concrete (for tie columns and tie beams), and roof materials are assigned in SAP2000 as follows:

1. Masonry material properties of typical clay brick in Indonesia is taken from experimental result that is based on Indonesian standard [18].

The image shows a software dialog box titled "Material Property Data". It contains several sections for defining material properties:

- General Data:** Includes "Material Name and Display Color" (set to "Brick Masonry" with a red color swatch), "Material Type" (set to "Other"), "Material Grade", and "Material Notes" with a "Modify/Show Notes..." button.
- Weight and Mass:** Includes input fields for "Weight per Unit Volume" (16,5732) and "Mass per Unit Volume" (1,69).
- Units:** A dropdown menu set to "KN, m, C".
- Isotropic Property Data:** Includes input fields for "Modulus Of Elasticity, E" (3201860), "Poisson, U" (0,254), "Coefficient Of Thermal Expansion, A" (9,000E-06), and "Shear Modulus, G" (1276658,7).
- At the bottom, there is a checkbox "Switch To Advanced Property Display" and "OK" and "Cancel" buttons.

Figure 3.1. Masonry material properties.

2. Mortar material properties that are made from an appropriate ratio of Portland cement, sand, lime, and water. In Indonesia, the sand and Portland cement ratio is 3:1 and the experimental result to determine its properties is shown in Figure 3.2 [18].

Material Property Data

General Data

Material Name and Display Color: MORTAR [Color: Red]

Material Type: Other

Material Grade:

Material Notes: [Modify/Show Notes...]

Weight and Mass

Weight per Unit Volume: 2,220E-03

Mass per Unit Volume: 2,264E-04

Units

KN, m, C

Isotropic Property Data

Modulus Of Elasticity, E: 2000000,

Poisson, U: 0,15

Coefficient Of Thermal Expansion, A: 1,170E-05

Shear Modulus, G: 869565,2

☐ Switch To Advanced Property Display

OK Cancel

Figure 3.2. Mortar material properties.

- Concrete K-175 is used which is based on SNI 2847:2019 [19].

Material Property Data

General Data

Material Name and Display Color: Concrete (K-175) [Color: Green]

Material Type: Concrete

Material Grade: C30/37

Material Notes: [Modify/Show Notes...]

Weight and Mass

Weight per Unit Volume: 23,536

Mass per Unit Volume: 2,4

Units

KN, m, C

Isotropic Property Data

Modulus Of Elasticity, E: 18895747,

Poisson, U: 0,2

Coefficient Of Thermal Expansion, A: 1,400E-05

Shear Modulus, G: 7873228,

Other Properties For Concrete Materials

Specified Concrete Compressive Strength, f_c : 14244,159

Expected Concrete Compressive Strength: 14244,159

☐ Lightweight Concrete

Shear Strength Reduction Factor:

☐ Switch To Advanced Property Display

OK Cancel

Figure 3.3. Concrete material properties.

4. Zinc is the material used for roof covers and zinc will be connected with roof trusses and purlins using nails.

The screenshot shows a software window titled 'Material Property Data' with a close button (X) in the top right corner. The window is divided into several sections:

- General Data:** Includes fields for 'Material Name and Display Color' (set to 'Zinc roof' with a yellow color swatch), 'Material Type' (set to 'Other'), 'Material Grade' (empty), and 'Material Notes' (empty). A 'Modify/Show Notes...' button is located below these fields.
- Weight and Mass:** Includes 'Weight per Unit Volume' (0,0687) and 'Mass per Unit Volume' (7,002E-03). To the right is a 'Units' dropdown menu set to 'KN, m, C'.
- Isotropic Property Data:** Includes 'Modulus Of Elasticity, E' (80000000), 'Poisson, U' (0,331), 'Coefficient Of Thermal Expansion, A' (0,017), and 'Shear Modulus, G' (30052592).
- At the bottom, there is a checkbox labeled 'Switch To Advanced Property Display' (which is unchecked), and 'OK' and 'Cancel' buttons.

Figure 3.4 Zinc material properties.

After the materials are assigned, the houses are modeled using shell elements (for masonry and roof cover) and frame elements (for columns, beams, and truss elements for the roof). Moreover, unstrengthened houses are developed using SAP2000 as follows:

1. Macro Modeling Shell Model of House 1

The model uses a macro modeling assumption in which masonry walls consist of masonry materials while the interaction between mortar and masonry was not considered in this model. The model of house 1 is shown in Figure 3.5 and Figure 3.6.

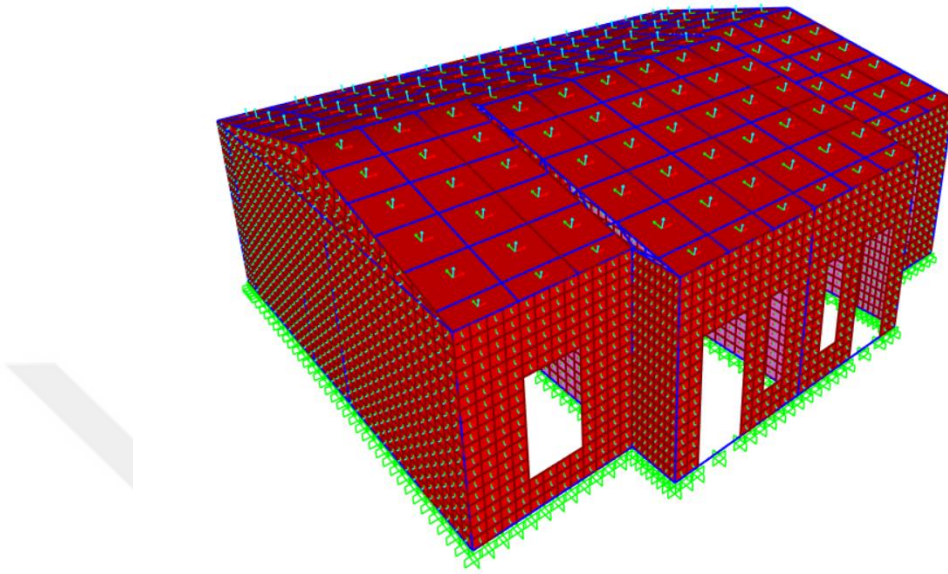


Figure 3.5. Macro modeling of shell Model House 1 3-D View.

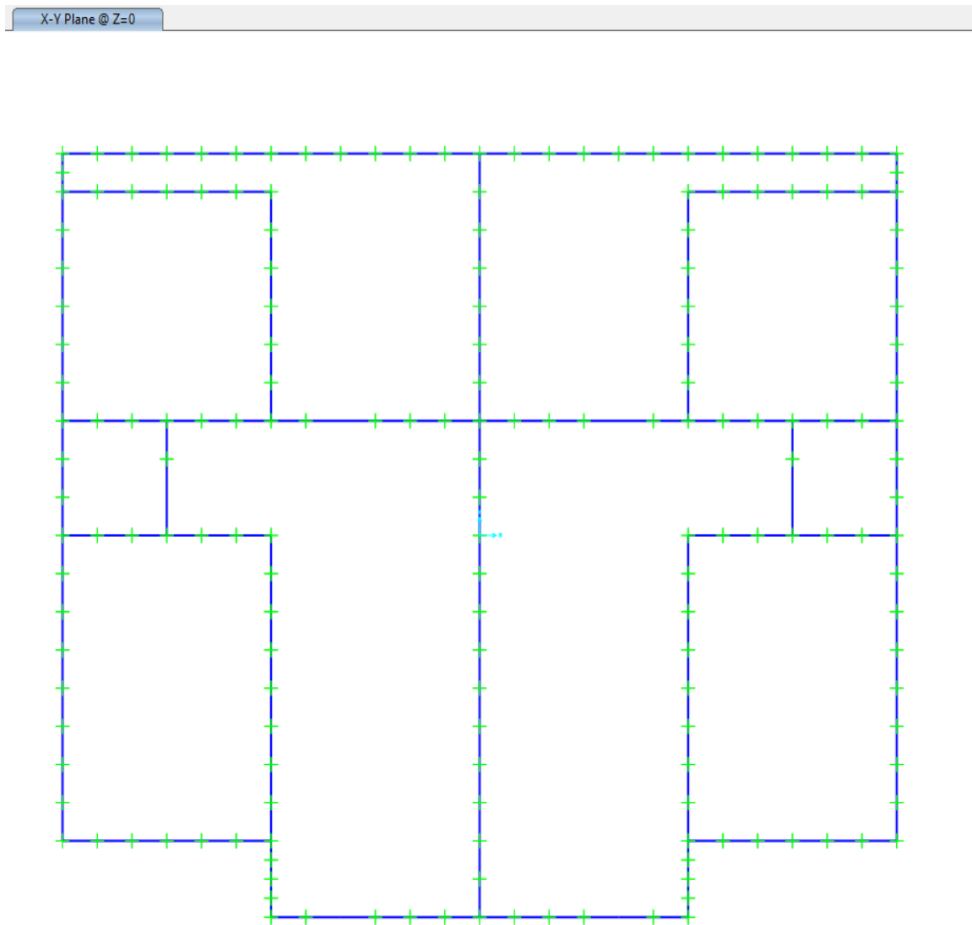


Figure 3.6. Macro Modeling of Shell Model House 1 Top View

2. Macro Modeling Shell Model of House 2

The second house is an unsymmetrical shape house with a shorter height but a higher slope compared to the first and third houses; the model is shown in Figure 3.7 and Figure 3.8:

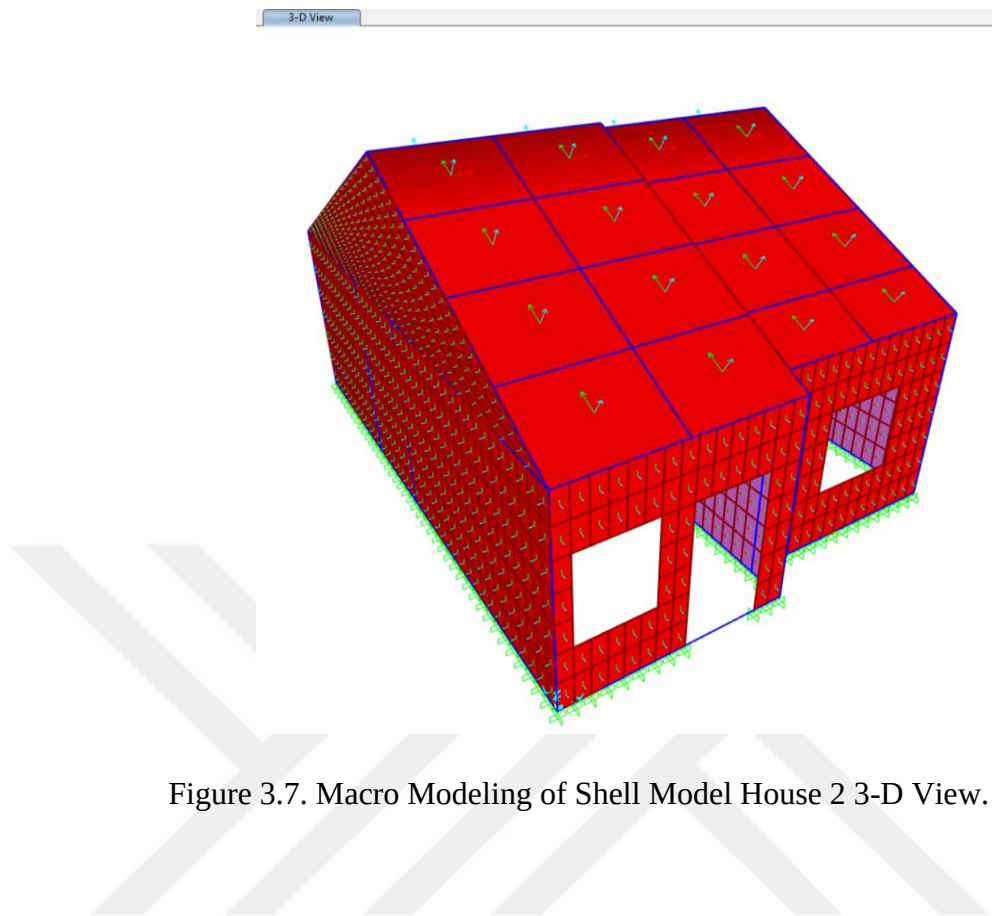


Figure 3.7. Macro Modeling of Shell Model House 2 3-D View.

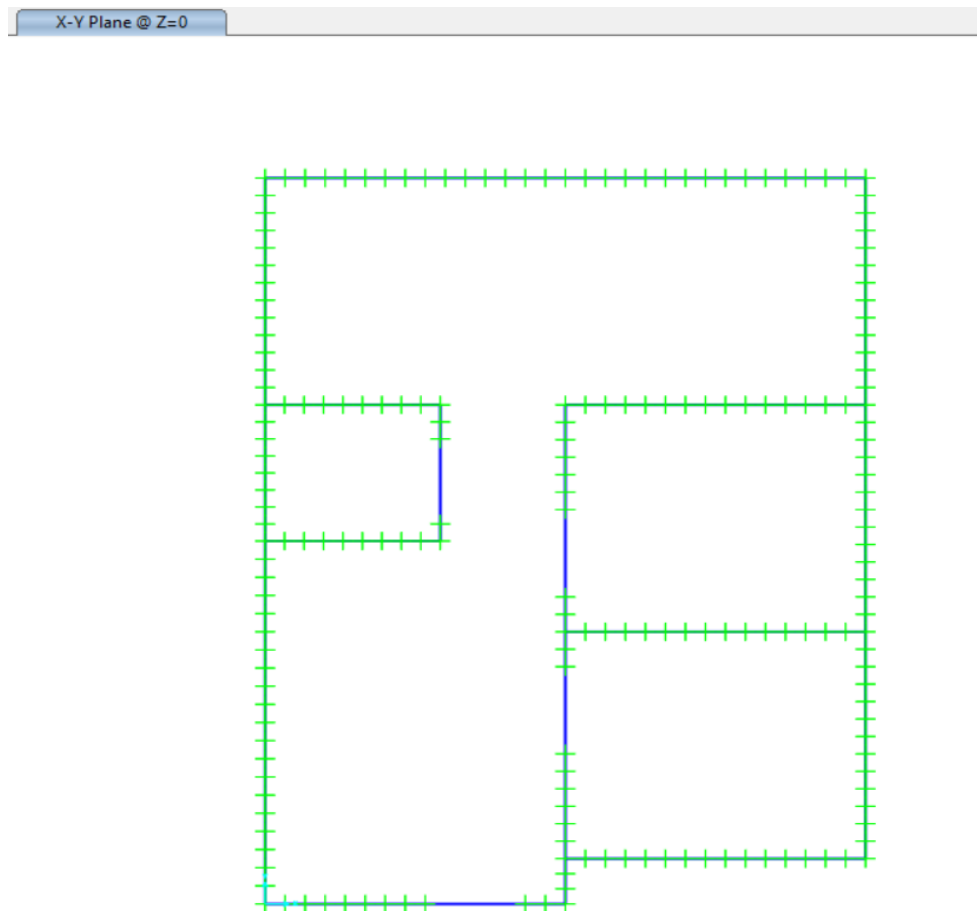


Figure 3.8. Macro Modeling of Shell Model House 2 Top View.

3. Macro Modeling Shell Model of House 3

The third house is similar to house 1 in terms of overall shape but has a different interior wall configuration. The model is shown in Figure 3.8 and Figure 3.9 :

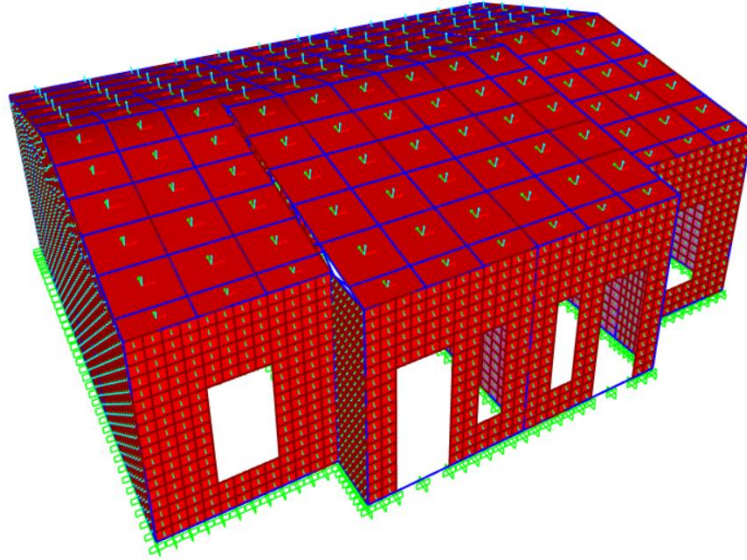


Figure 3.9. Macro Modeling of Shell Model House 3 3-D View

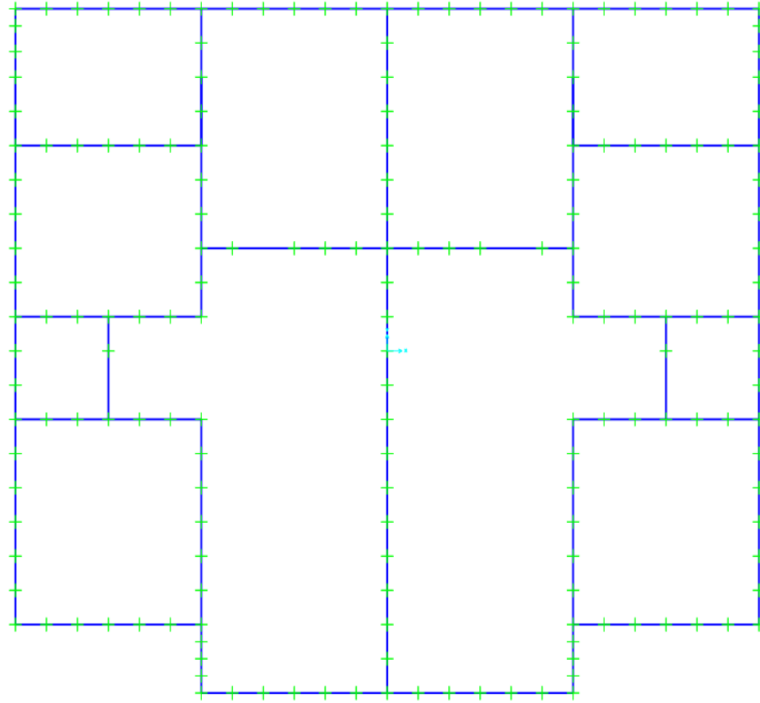


Figure 3.10. Macro Modeling of Shell Model House 3 Top View.

3.2 Load Application

Each model's load combinations are assigned based on SNI 1727:2020 [20], which is an Indonesian code for determining minimum design loads and other required criteria. In this code, the following loads are considered for the house:

3.2.1 Dead Load

Dead load is the self-weight of the house which includes the weight of masonry, mortar, zinc roof, concrete, steel hollow, and steel truss. SAP2000 automatically calculates this weight.

3.2.2 Wind Load

Wind load is calculated according to SNI 1727:2020 [20] which is based on ASCE 7-16 using the following formula:

$$P = q * GC_p - q_i * (GC_{pi})$$

While q is calculated using either formula:

$$q_z = 0.613 * K_d * K_{zt} * K_z * V^2$$

$$q_h = 0.613 * K_d * K_{zt} * K_h * V^2$$

For comparison, TS498 [21] has $q = 0.625 * V^2$; However, SNI code was used for consistency. In SAP2000, the program can automatically calculate the required wind pressure according to ASCE 7-16. However, it is necessary to input the required coefficient due to external pressure on each side of the walls and roofs. This coefficient is obtained from Table 2 taken from SNI 1727:2020 [20]:

Table 2. External Pressure Coefficient for wall. [20]

External Pressure Coefficient, C_p		
surface	L/B	C_p
windward wall	All Values	0.8
Leeward wall	0-1	-0.5
	2	-0.3
	≥ 4	-0.2
Side wall	All Values	-0.7

Table 3. External Pressure Coefficient for Roof. [20]

External Pressure Coefficient for Roof, C_p		windward side							
		angle							
wind direction	h/L	10	15	20	25	30	35	45	≥ 60
perpendicular to roof top for $\geq 10^\circ$	≥ 0.25	-0.7	-0.5	-0.3	-0.2	-0.2	-0.2	0	$0.01 * \theta$
		-0.18	0	0.2	0.3	0.3	0.3	0.4	
	0.5	-0.9	-0.7	-0.4	-0.3	-0.2	-0.2	0	$0.01 * \theta$
		-0.18	-0.18	0	0.2	0.2	0.3	0.4	
	≥ 1	-1.3	-1	-0.7	-0.5	-0.3	-0.2	0	$0.01 * \theta$
		-0.18	-0.18	-0.18	0	0.2	0.2	0.4	

Table 4. External Pressure Coefficient for Roof. [20]

External Pressure Coefficient for Roof, C_p		Leeward side		
		Angle		
wind direction	h/L	10	15	≥ 20
Perpendicular to roof top for $\geq 10^\circ$	≥ 0.25	-0.3	-0.5	-0.6
	0.5	-0.5	-0.5	-0.6
	≥ 1	-0.7	-0.6	-0.6
		Horizontal distance from windward side		C_p
Parallel to roof top for all θ	≤ 0.5	0-h		-0.9
		h-2h		-0.5
		$> 2h$		-0.3
	≥ 1	0-h/2		-1.3
		$> h/2$		-0.7

Where 'L' is the length of the house parallel to the wind direction, 'B' is the length of the house perpendicular to the wind direction and 'h' is the height from bottom to top half of the roof.

As an example, house 1 and house 3 wind load coefficients are assigned as follows: In the models, both winds in X and Y directions are considered in the model. Wind loads are assigned at the load pattern option as shown in Figure 3.11:

ASCE 7-10 Wind Load Pattern

Exposure and Pressure Coefficients

- ☐ Exposure from Extents of Rigid Diaphragms
- ☒ Exposure from Frame and Area Objects
 - ☒ Include Area Objects
 - ☐ Include Frame Objects (Open Structure)

Wind Exposure Parameters

Wind Direction Angle:

Windward Coeff, C_p :

Leeward Coeff, C_p :

Case (ASCE 7-10 Fig. 27.4-8):

e1 Ratio (ASCE 7-10 Fig. 27.4-8):

e2 Ratio (ASCE 7-10 Fig. 27.4-8):

Wind Coefficients

Wind Speed (mph):

Exposure Type:

Topographical Factor, K_{zt} :

Gust Factor:

Directionality Factor, K_d :

Solid / Gross Area Ratio:

Exposure Height

- ☒ Program Calculated
- ☐ User Specified
 - Maximum Global Z:
 - Minimum Global Z:

Figure 3.11. Wind load assignment.

In addition to that, the coefficients due to external pressure (C_p) are applied according to Figure 3.11. For the wind in X direction. C_p value is taken as 0.8 for the wall side where wind direction is coming to the wall. Since all walls' local 3 directions are directed towards outside, and the wind direction is directed towards the wall for the incoming side. Thus, the C_p value for the incoming wall side is taken as -0.8, and it is assigned in SAP2000 as shown in Figure 3.12 :

Figure 3.12. C_p assignment for wind X direction at the incoming side

Moreover, for the perpendicular side of the wind direction, the value is taken as 0.7 in the outward direction. Thus, the C_p value is taken as 0.7.

In addition to that, for the outgoing side of the wall, L/B should be calculated. L/B value for X direction is calculated as:

$$\frac{L}{B} = \frac{12}{10} = 1.2$$

Since 1.2 is not given in the table from Figure 21. Interpolation is done to find the required C_p value, and the value is calculated as:

$$\frac{(C_p - (-0.5))}{(-0.3 - (-0.5))} = \frac{(1.2 - 1)}{(2 - 1)}$$

$$C_p = -0.46$$

This value is directed toward the outside. Thus, the C_p value is taken as 0.46.

For the roof, considering the X direction of a rooftop is parallel to the wind direction, h/L is calculated as:

$$\frac{h}{L} = \frac{3.6 + 0.5}{10} = 0.4$$

The horizontal distance of the roof (12 m) is more than 2*h (8.2 m). Thus, C_p for wind X is taken as 0.3. For wind Y, the roof top is perpendicular to wind direction and the roof angle is 15° and h/L is calculated as:

$$\frac{h}{L} = \frac{3.6 + 0.5}{10} = 0.4$$

Thus, C_p for wind Y at the windward and leeward sides is taken as 0.7 and 0.5, respectively.

3.2.3 Earthquake Load

Earthquake load is determined according to SNI 1726:2019 [22], based on ASCE 7-17. To obtain the earthquake response spectrum of the house location, the Ministry of Infrastructure of Indonesia has provided a website to obtain the required response spectrum. The interface of the website is shown in Figure 3.13:

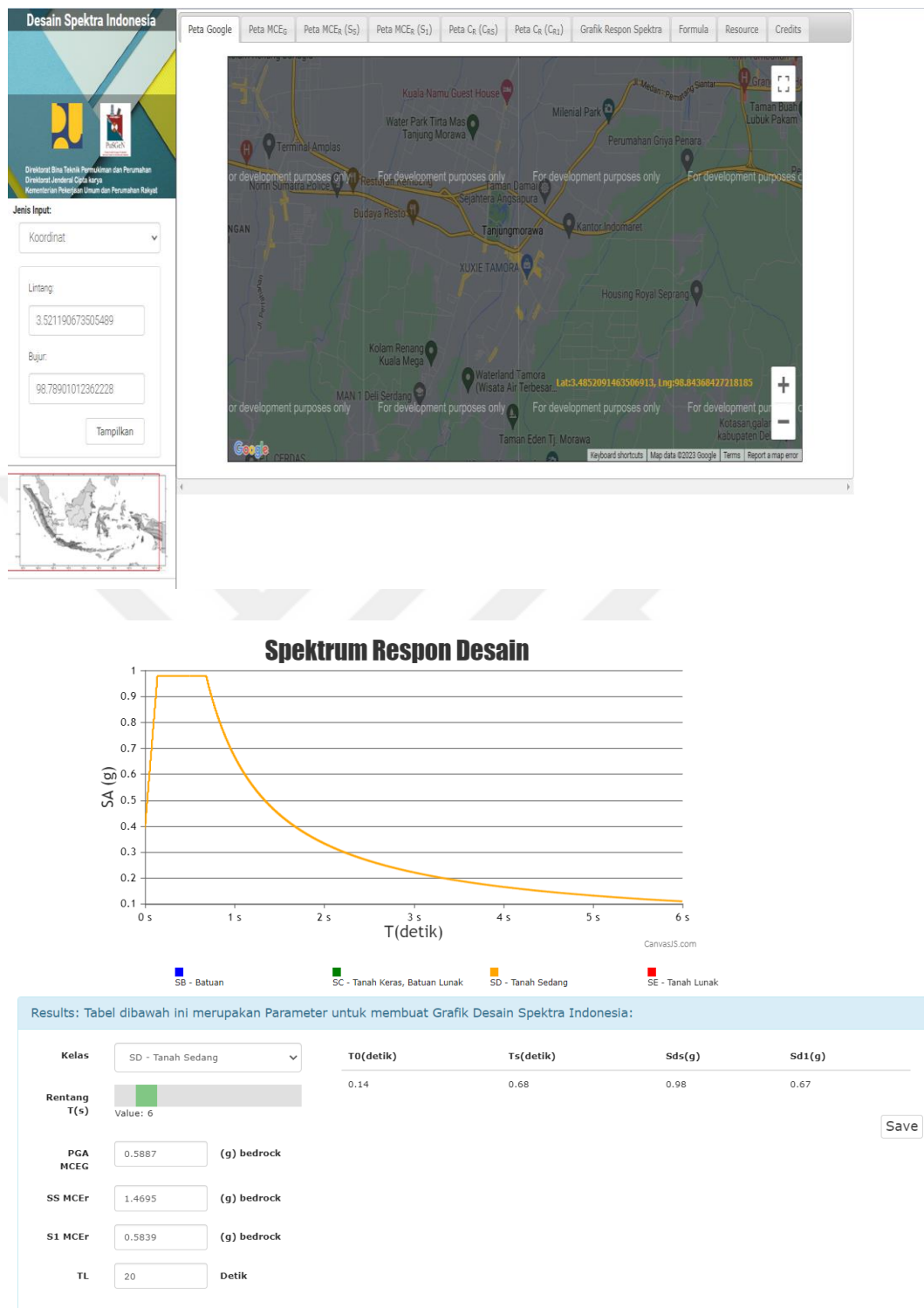


Figure 3.13. Interface of the Indonesian Design Spectra website

The earthquake load analysis of non-engineered masonry houses uses data from the Ministry of Infrastructure of Indonesia, and the response spectrum is added to SAP2000. The obtained S_s , S_1 , and T values are being input into SAP2000 as shown in Figure 3.14:

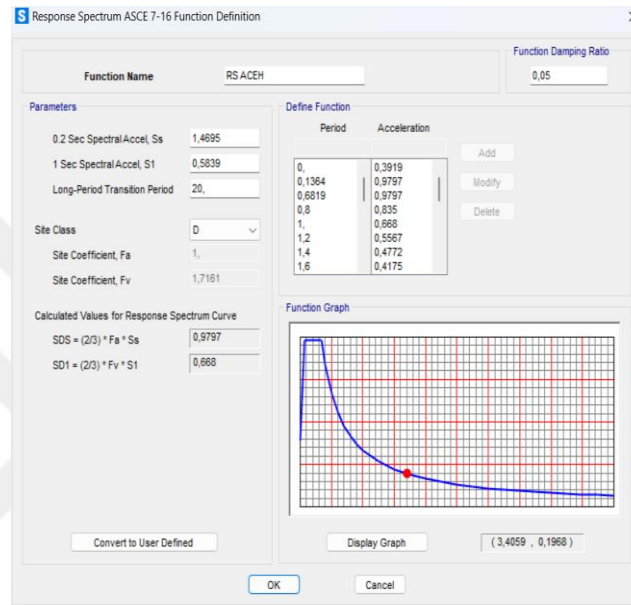


Figure 3.14. Response spectrum value in SAP2000

According to SNI 1726:2019 [22], modal combinations calculation can be done by either SRSS (Square Root of Sum of Squares) or CQC (Complete Quadratic Combination) method. The CQC method is used for soil that has spectral acceleration differences less than 15% for close periods whereas the SRSS method is used for spectral acceleration differences of more than 15% [3]. In Aceh province, the following spectral acceleration data is obtained:

T_1 (1 sec): 0.667 g

T_2 (2 sec): 0.335 g

T_3 (3 sec): 0.223 g

T_4 (4 sec): 0.167 g

Spectral acceleration differences are calculated as:

$$T_1 - T_2 = (0.667 - 0.335) / 0.667 \times 100\% = 49.775 > 15$$

$$T_2 - T_3 = (0.335 - 0.223) / 0.335 \times 100\% = 33.43 > 15$$

$$T_3 - T_4 = (0.223 - 0.167) / 0.167 \times 100\% = 33.53 > 15$$

Considering the spectral acceleration differences are more than 15%, the SRSS method is preferable. Thus, the load cases are assigned as shown in Figure 3.15 and Figure 3.16:

Load Case Data - Response Spectrum

Load Case Name: EQX [Set Def Name] [Modify/Show...]

Load Case Type: Response Spectrum [Design...]

Modal Combination

☐ CQC ☒ SRSS ☐ Absolute ☐ GMC ☐ NRC 10 Percent ☐ Double Sum

GMC f1: 1, GMC f2: 0, Periodic + Rigid Type: SRSS

Modal Load Case

Use Modes from this Modal Load Case: MODAL

☒ Standard - Acceleration Loading ☐ Advanced - Displacement Inertia Loading

Loads Applied

Load Type	Load Name	Function	Scale Factor
Accel	U1	RS ACEH	9.81
Accel	U1	RS ACEH	9.81

[Add] [Modify] [Delete]

☐ Show Advanced Load Parameters

Other Parameters

Modal Damping: Constant at 0.05 [Modify/Show...]

Diaphragm Eccentricity

Eccentricity Ratio: 0, [Override...]

[OK] [Cancel]

Figure 3.15. Load case assignment for EQX load.

Load Case Data - Response Spectrum

Load Case Name: EQY [Set Def Name] [Modify/Show...]

Load Case Type: Response Spectrum [Design...]

Modal Combination

☐ CQC ☒ SRSS ☐ Absolute ☐ GMC ☐ NRC 10 Percent ☐ Double Sum

GMC #1: 1, GMC #2: 0, Periodic + Rigid Type: SRSS

Modal Load Case

Use Modes from this Modal Load Case: MODAL [v]

☒ Standard - Acceleration Loading ☐ Advanced - Displacement Inertia Loading

Loads Applied

Load Type	Load Name	Function	Scale Factor
Accel	U2	RS ACEN	9.81
Accel	U2	RS ACEN	9.81

[Add] [Modify] [Delete]

☐ Show Advanced Load Parameters

Other Parameters

Model Damping: Constant at 0.05 [Modify/Show...]

[OK] [Cancel]

Figure 3.16. Load case assignment for EQY load.

3.3 Load Combination

The load combinations considering earthquake effect and wind loads are based on SNI 2847:2019 [19] and it is given as follows:

Combination 1: 1.4D

Combination 2: 1.2D+1.6L

Combination 3: 1.2D+0.5W_x

Combination 4: 1.2D+0.5W_y

Combination 5: 1.2D+W_x+L

Combination 6: 1.2D+W_y+L

Combination 7: 1.39D+L+EQ_x+0.3EQ_y

Combination 8: 1.39D+L+EQ_x-0.3EQ_y

Combination 9: 1.39D+L-EQ_x+0.3EQ_y

Combination 10: 1.39D+L-EQ_x-0.3EQ_y

Combination 11: 1.39D+L+EQ_y+0.3EQ_x

Combination 12: 1.39D+L+EQ_y-0.3EQ_x

Combination 13: $1.39D+L-EQ_y+0.3EQ_x$

Combination 14: $1.39D+L-EQ_y-0.3EQ_x$

Combination 15: $0.7D+EQ_x+0.3EQ_y$

Combination 16: $0.7D+EQ_x-0.3EQ_y$

Combination 17: $0.7D-EQ_x+0.3EQ_y$

Combination 18: $0.7D-EQ_x-0.3EQ_y$

Combination 19: $0.7D+EQ_y+0.3EQ_x$

Combination 20: $0.7D+EQ_y-0.3EQ_x$

Combination 21: $0.7D-EQ_y+0.3EQ_x$

Combination 22: $0.7D-EQ_y-0.3EQ_x$

In addition to the 22 load combinations, Envelope combinations which include the combinations of all 22 combinations are also added to find the maximum forces and stresses.

To calculate the coefficient of dead load from load combination 7 to load combination 14, the following formula is used according to SNI 2847:2019 [19]:

$$1.2 + 0.2 * S_{DS} = 1.2 + 0.2 * 0.97 = 1.39$$

To calculate the coefficient of dead load from load combination 15 to load combination 22, the following formula is used according to SNI 2847:2019:

$$0.9 - 0.2 * S_{DS} = 0.9 - 0.2 * 0.97 = 0.7$$

3.4 Analysis for Unstrengthened Models

3.4.1 Mesh Sensitivity Analysis

In finite element analysis, determining mesh size is one of the fundamental aspects since large mesh size can lead to inaccurate results while small mesh

size can lead to extended computing time. Thus, mesh sensitivity analysis is required to determine an ideal mesh size that is small enough to give accurate results while being large enough to shorten computing time required. To do the mesh sensitivity analysis, several mesh sizes at each house were modeled and maximum axial stress at one of the walls in addition to frequency, and period of the houses at all mesh sizes are plotted. In house 1, six different mesh sizes are modelled and the results are plotted as shown in Figure 3.17, Figure 3.18, and Figure 3.19.

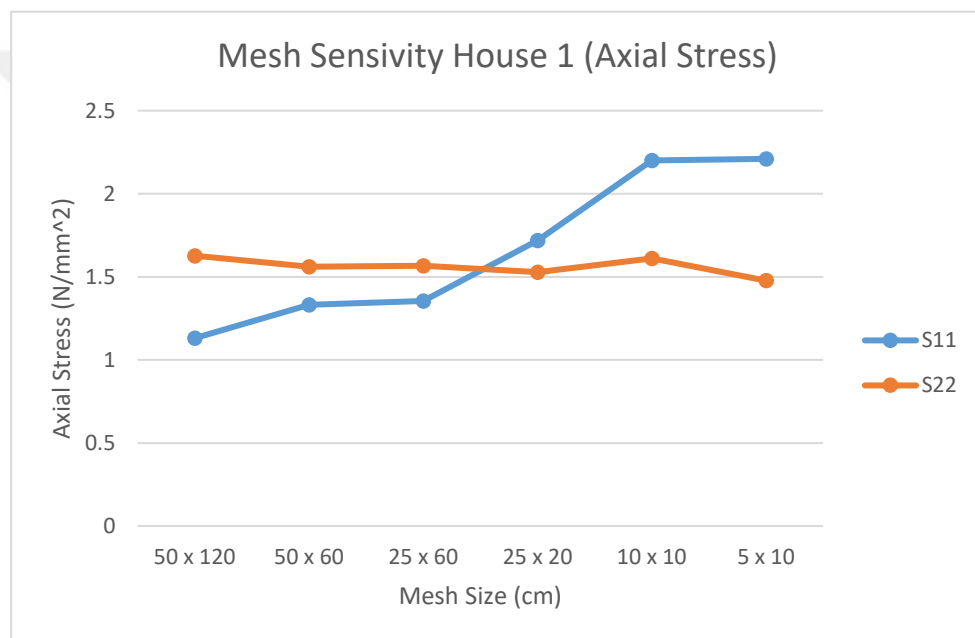


Figure 3.17. Mesh sensitivity analysis of house 1 at one of wall's maximum axial stress.

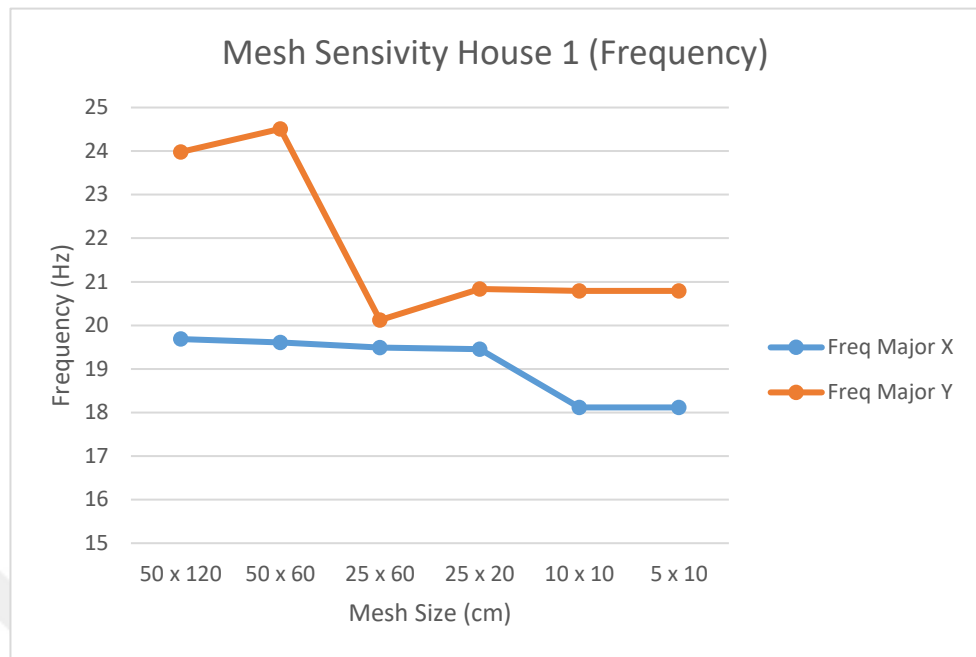


Figure 3.18. Mesh sensitivity analysis of house 1 by comparing frequency at both major axes.

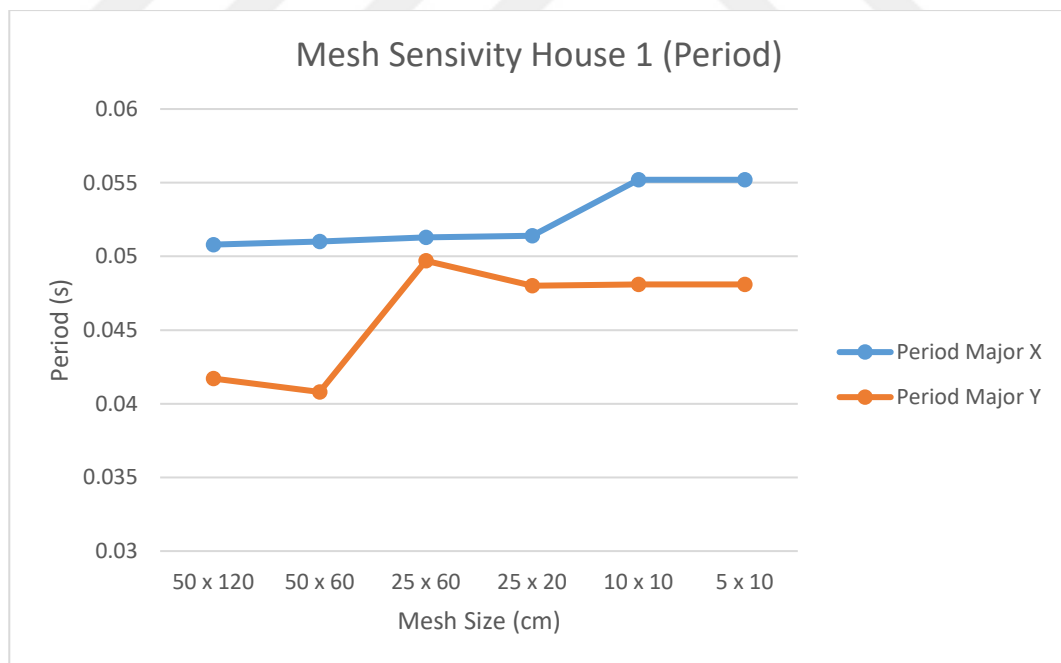


Figure 3.19. Mesh sensitivity analysis of house 1 by comparing period at both major axes.

From the graphs, it can be observed that the results for house 1 reach convergence at 10 x 10 cm mesh size since the frequency, period, and maximum axial stress results do not change significantly by reducing the mesh size to 5 x 10 cm while computing time increase significantly. Thus, mesh size of 10 x 10 cm will be used for house 1 at the next step of the analysis.

In house 2, four different mesh sizes are modeled and the results are plotted as shown in Figure 3.20, Figure 3.21, and Figure 3.22.

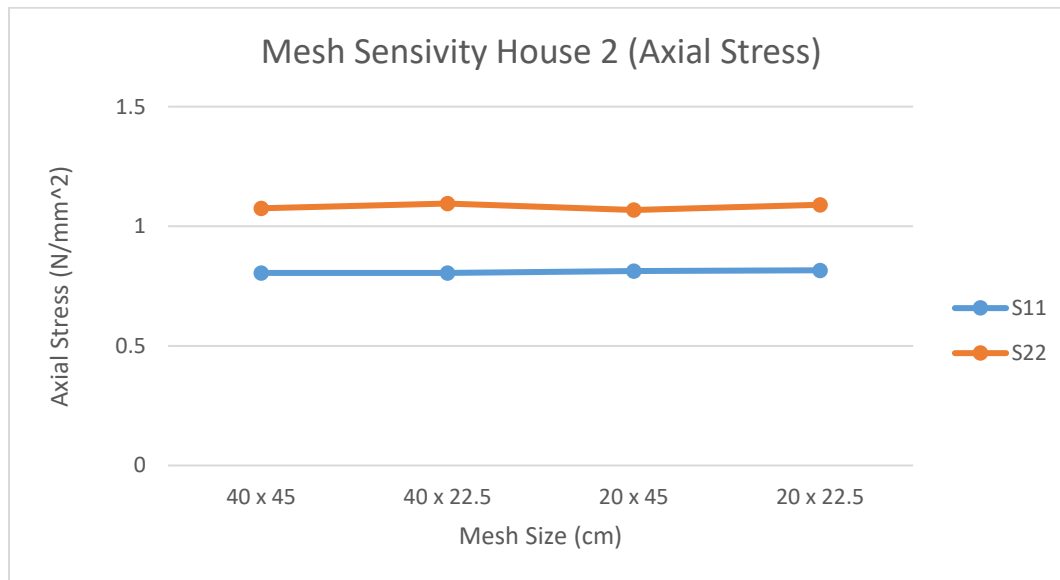


Figure 3.20. Mesh sensitivity analysis of house 2 at one of wall's maximum axial stress.

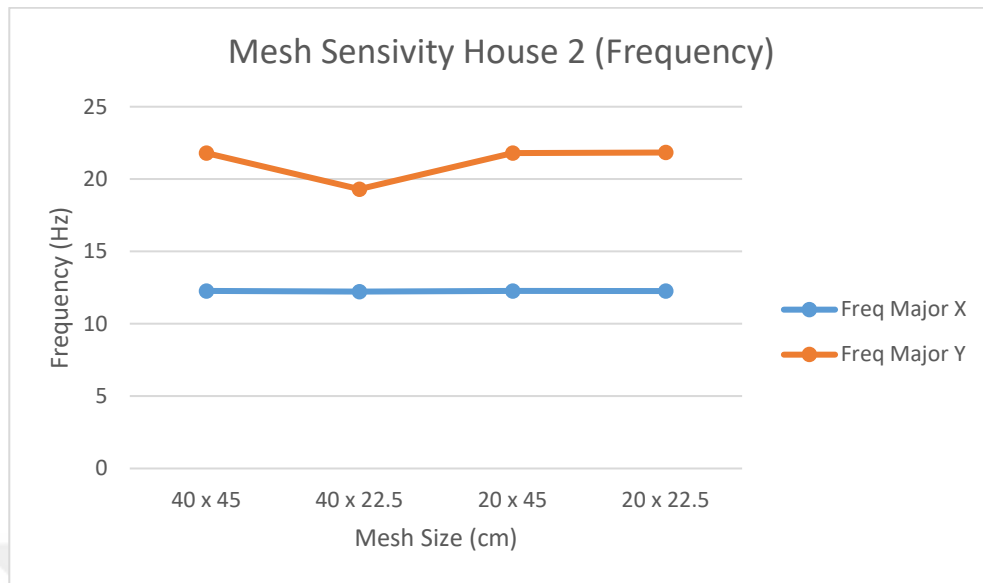


Figure 3.21. Mesh sensitivity analysis of house 2 by comparing frequency at both major axes.

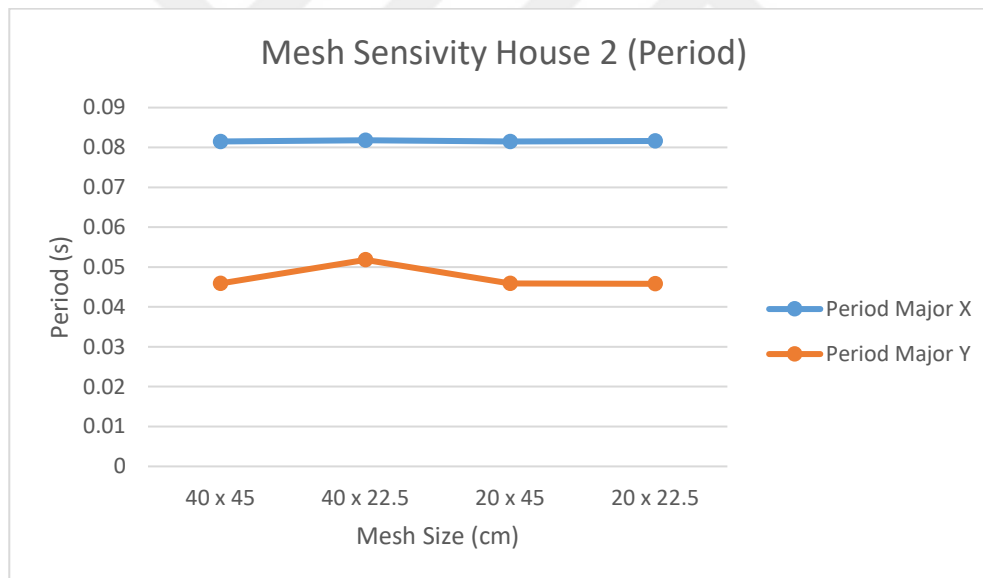


Figure 3.22. Mesh sensitivity analysis of house 2 by comparing period at both major axes.

From the graphs, it can be observed that the results for house 2 reach convergence at 20 x 45 cm mesh size since the frequency, period, and maximum axial stress results do not change significantly by reducing the mesh size to 20 x 22.5 cm while

computing time increase significantly. Thus, mesh size of 20 x 45 cm will be used for house 2 at the next step of the analysis.

In house 3, five different mesh sizes are modeled and the results are plotted as shown in Figure 3.23, Figure 3.24, and Figure 3.25.

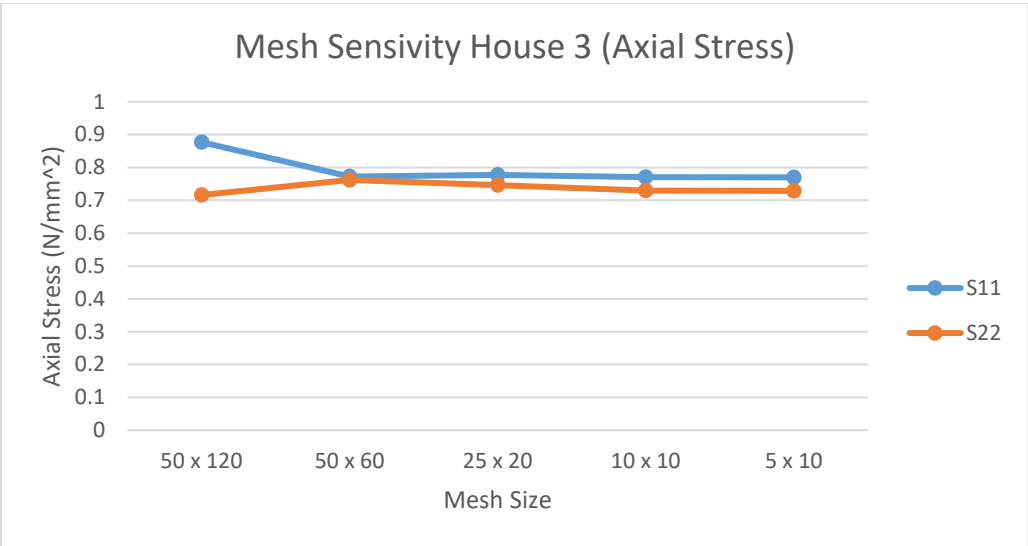


Figure 3.23. Mesh sensitivity analysis of house 3 at one of wall’s maximum axial stress.

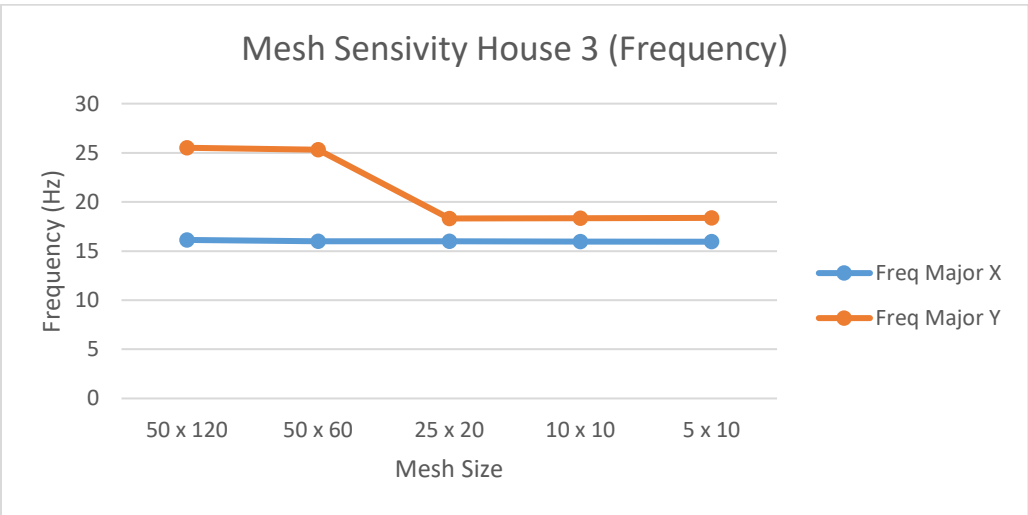


Figure 3.24. Mesh sensitivity analysis of house 3 by comparing frequency at both major axes.

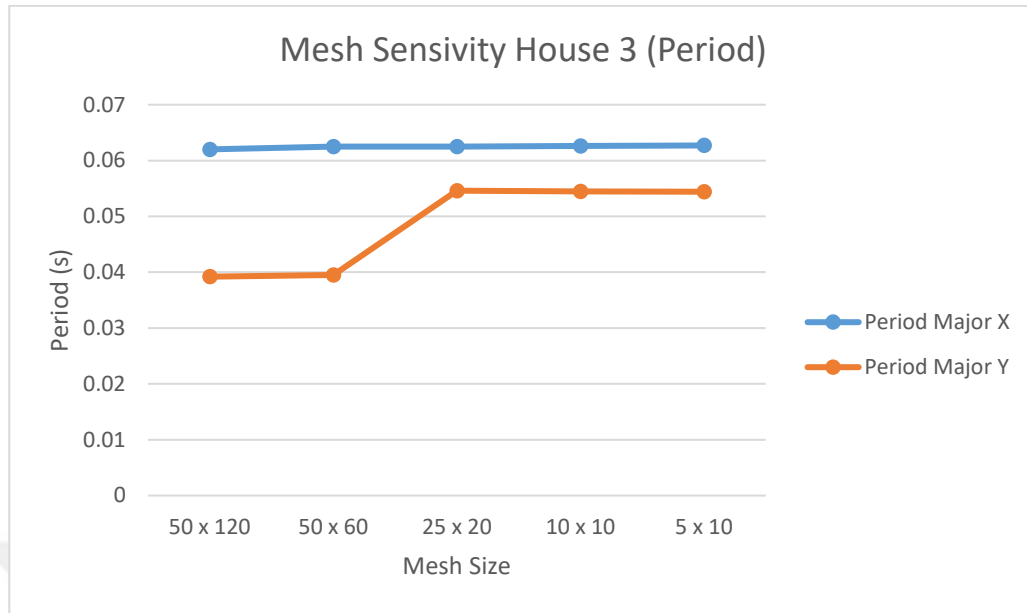


Figure 3.25. Mesh sensitivity analysis of house 3 by comparing period at both major axes.

From the graphs, it can be observed that the results for house 3 reach convergence at 25 x 20 cm mesh size since the frequency, period, and maximum axial stress results do not change significantly by reducing the mesh size to 10 x 10 cm and 5 x 10 cm while computing time increase significantly. Thus, mesh size of 25 x 20 cm will be used for house 3 at the next step of the analysis.

3.4.2 Displacement Criteria

3.4.2.1 ASCE 7-16

According to ASCE 7-16 [23], the deflection formula used for story drift is calculated as:

$$\delta_x = \frac{C_d * \delta_{xe}}{I_e}$$

According to table 12.2-1 from ASCE 7-16, the wall can be categorized as ordinary plain masonry shear walls and C_d value can be taken as 5/4 while the residential house is considered at risk category 1 which has earthquake importance factor of 1 according to table 1.5-2 ASCE 7-16.

Moreover, allowable story drift is calculated according to table 12.12-1 from ASCE 7-16 given in Table 5.

Table 5. Allowable story drift [23]

Structure	Risk Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{xx}^c$	$0.020h_{xx}$	$0.015h_{xx}$
Masonry cantilever shear wall structures ^d	$0.010h_{xx}$	$0.010h_{xx}$	$0.010h_{xx}$
Other masonry shear wall structures	$0.007h_{xx}$	$0.007h_{xx}$	$0.007h_{xx}$
All other structures	$0.020h_{xx}$	$0.015h_{xx}$	$0.010h_{xx}$

^a h_{xx} is the story height below level x.

^bFor seismic force-resisting systems solely comprising moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.

^cThere shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.

^dStructures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are so constructed that moment transfer between shear walls (coupling) is negligible.

For unstrengthen house 1 model, the max displacements calculated are 4.92 mm and 2.93 mm in X and Y direction, respectively. Thus, according to ASCE 7-16:

In X direction:

$$\delta_x = \frac{C_d * \delta_{xe}}{I_e} = \frac{\left(\frac{5}{4}\right) * 4.92}{1} = 6.15 \text{ mm}$$

The story drift result is compared with Table 5 for masonry cantilever shear wall with risk category of 1 for residential house.

$$0.01 * h_x = 0.01 * 3600 = 36 \text{ mm}$$

$$\delta_x = 6.15 \text{ mm} \leq 36 \text{ mm OK}$$

In Y direction:

$$\delta_y = \frac{C_d * \delta_{ye}}{I_e} = \frac{\left(\frac{5}{4}\right) * 2.93}{1} = 3.66 \text{ mm}$$

The story drift result is compared with Table 5 for masonry cantilever shear wall with risk category of 1 for residential house.

$$0.01 * h_x = 0.01 * 3600 = 36 \text{ mm}$$

$$\delta_y = 3.66 \text{ mm} \leq 36 \text{ mm OK}$$

The same calculation is done for all the unstrengthened house models and the result is tabulated in Table 6.

Table 6. Displacement criteria according to ASCE 7-16

ASCE	X Axis (mm)			
Unstrengthened	Max Disp	Story drift	Allowable	
House 1	4.92	6.15	36	OK
House 2	3.45	4.3125	31.5	OK
House 3	4.74	5.925	36	OK
ASCE	Y Axis (mm)			
Unstrengthened	Max Disp	Story drift	Allowable	
House 1	2.93	3.6625	36	OK
House 2	2.19	2.7375	31.5	OK
House 3	1.95	2.4375	36	OK

It can be concluded from Table 6 that all houses' maximum displacements are below the allowable limit defined by ASCE 7-16.

3.4.2.2 Euro Code 8

According to sections 4.3.4 and 4.4.3.2 in Eurocode 8 regarding the limitation of inter-story drift [24], the displacement of the structure induced by design seismic action can be calculated as:

$$d_s = q_d * d_c$$

Where d_c is the displacement based on linear analysis from the design response spectrum and q_d is a behavior factor given as 1.5 [25]. Moreover, the non-engineered confined masonry houses can be considered as buildings that have brittle materials connected to the structure and it is calculated as:

$$d_s * v \leq 0.005 * h$$

v value depends on the importance classes of a building which is given at Table 4.3 in Euro Code 8 and the confined masonry houses can be categorized as importance class II. For importance class II, v value is recommended as 0.5.

For unstrengthen house 1 model, the max displacements calculated are 4.92 mm and 2.93 mm in X and Y direction, respectively. Thus, according to Euro Code 8:

In X direction:

$$d_s = q_d * d_c = 1.5 * 4.92 = 7.38 \text{ mm}$$

$$d_s * v \leq 0.005 * h$$

$$7.38 * 0.5 \leq 0.005 * 3600$$

$$3.69 \text{ mm} \leq 18 \text{ mm OK}$$

In Y direction:

$$d_s = q_d * d_c = 1.5 * 2.93 = 4.395 \text{ mm}$$

$$d_s * v \leq 0.005 * h$$

$$4.395 * 0.5 \leq 0.005 * 3600$$

$$2.19 \text{ mm} \leq 18 \text{ mm OK}$$

The same calculations are done for all the unstrengthen house models and the result is tabulated in Table 7.

Table 7. Displacement criteria according to Euro Code 8

Euro Code 8	X Axis (mm)			
Unstrengthened	Max Disp	Story drift	Allowable	
House 1	4.92	3.69	18	OK
House 2	3.45	2.5875	15.75	OK
House 3	4.74	3.555	18	OK
Euro Code 8	Y Axis (mm)			
Unstrengthened	Max Disp	Story drift	Allowable	
House 1	2.93	2.1975	18	OK
House 2	2.19	1.6425	15.75	OK
House 3	1.95	1.4625	18	OK

It can be concluded from Table 7 that all houses' max displacements are below the allowable limit defined by Eurocode 8.

3.4.2.3 Turkish Code (TBDY 2018)

According to section 4.9.1.3 from TBDY 2018 [26], inter-story drift can be calculated for a case when hollow or solid fill walls made of brittle materials are not connected with any flexible joint as follows:

$$\lambda * \frac{R * \Delta}{I * h} \leq 0.008 * \kappa$$

R value is given as 3 for confined masonry structure and importance factor (I) of the structure is taken as 1 for residential house. Furthermore, κ value is given as 1 for reinforce concrete and 0.5 for structural steel. Since the value for masonry structure is not given, κ is assumed to be the same as reinforce concrete. In addition to that, λ is calculated as ratio of spectral acceleration at DD-3 and DD-2 for major period of structures at X and Y axis.

$$\lambda = \frac{S_{ae}(T)(DD - 3)}{S_{ae}(T)(DD - 2)}$$

Considering the buildings location are not in Türkiye, 5 different locations in Türkiye with different spectral accelerations are used and the average λ is used to calculate the maximum allowable inter-story drift.

Table 8. Average λ for major period of structures.

		DD2	DD3	λ
S_{ae}	Tekirdağ	1.106	0.58	0.524412
	Adana	0.73	0.304	0.416438
	Trabzon	0.668	0.219	0.327844
	Muğla	0.998	0.528	0.529058
	Hatay	1.139	0.511	0.448639
			AVG	0.449278

For unstrengthen house 1 model, the max displacements calculated are 4.92 mm and 2.93 mm in X and Y direction, respectively. Thus, according to TBDY 2018:

In X direction:

$$\lambda * \frac{R * \Delta}{I * h} \leq 0.008 * \kappa$$

$$(0.449) * \frac{3 * 4.92}{1 * 3600} \leq 0.008 * 1$$

$$0.00184 \leq 0.008 \text{ OK}$$

In Y direction:

$$\lambda * \frac{R * \Delta}{I * h} \leq 0.008 * \kappa$$

$$(0.449) * \frac{3 * 2.93}{1 * 3600} \leq 0.008 * 1$$

$$0.00109 \leq 0.008 \text{ OK}$$

The same calculations are done for all the unstrengthen house models and the result is tabulated in Table 9.

Table 9 Displacement criteria according to TBDY 2018

TBDY 2018	X Axis			
Unstrengthened	Max Disp (mm)	Calculated	Allowable	
House 1	4.92	0.002105	0.008	OK
House 2	3.45	0.001476	0.008	OK
House 3	4.74	0.001775	0.008	OK
TBDY 2018	Y Axis			
Unstrengthened	Max Disp (mm)	Calculated	Allowable	
House 1	2.93	0.001254	0.008	OK
House 2	2.19	0.000937	0.008	OK
House 3	1.95	0.00073	0.008	OK

It can be concluded from Table 9 that all houses' max displacements are below the allowable limit defined by TBDY 2018.

3.4.3 Analysis of Unstrengthened House 1 Model

After the models have been designed, the required loads and load combinations have been assigned, and the appropriate mesh size has been determined. Analysis is done to check whether the modeled houses, especially masonry walls as the main structural elements, has enough capacity to prevent out-of-plane failures which is the main cause of damage in non-engineered confined masonry house in Indonesia. The following checks are done:

1. Displacement check.
2. Shear Stress check.
3. Flexural strength check at 1-1 Axis.
4. Flexural strength check at 2-2 Axis.

Displacement check was done in section 3.4.2, and other checks will be done in this section. Furthermore, the following sign conventions are used in SAP2000 [27] for the analysis.

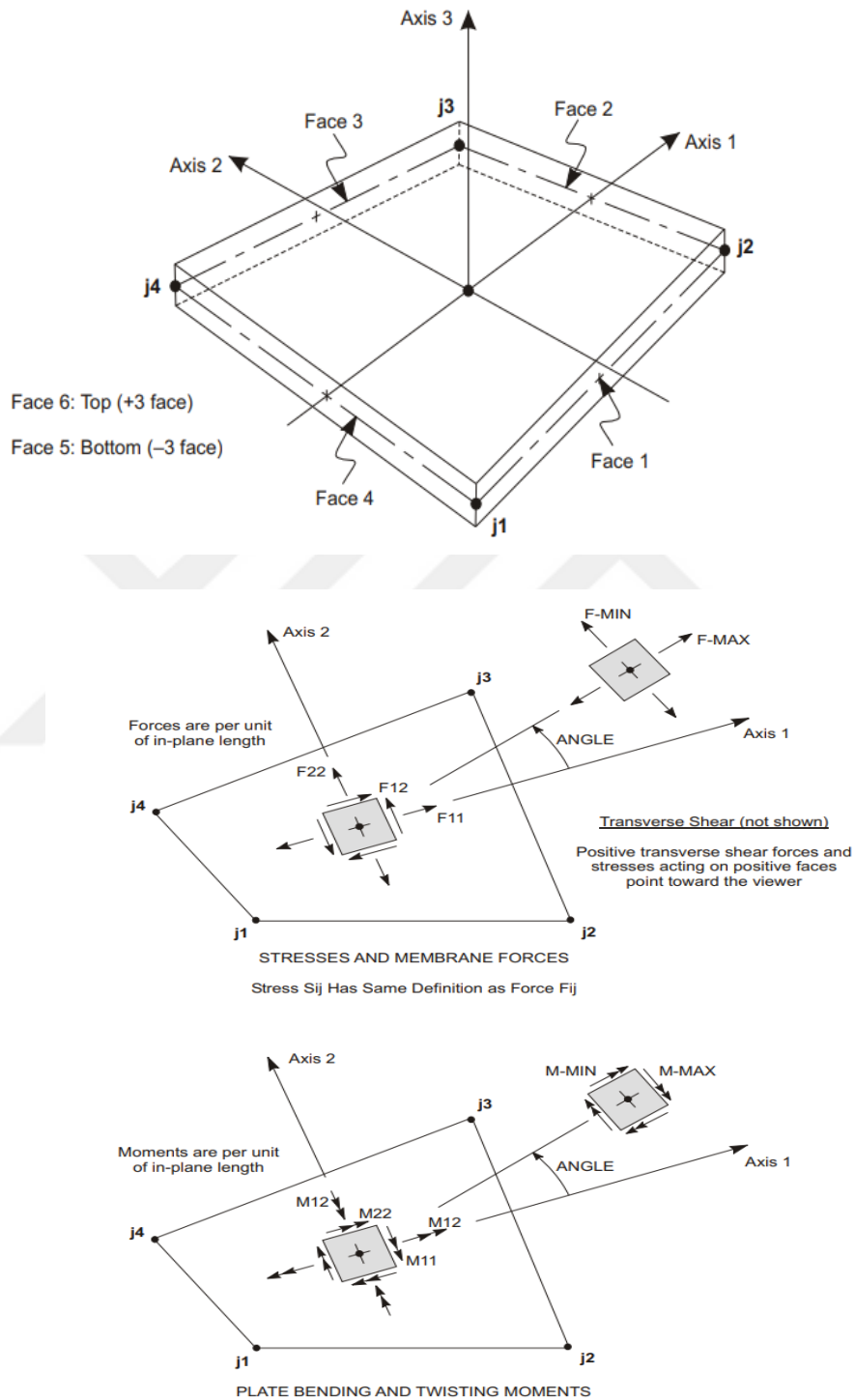


Figure 3.26. Sign Convention in SAP2000 [27].

Furthermore, each wall is numbered to differentiate the location of each wall better; the wall numbering for house 1 is given in Figure 3.27:

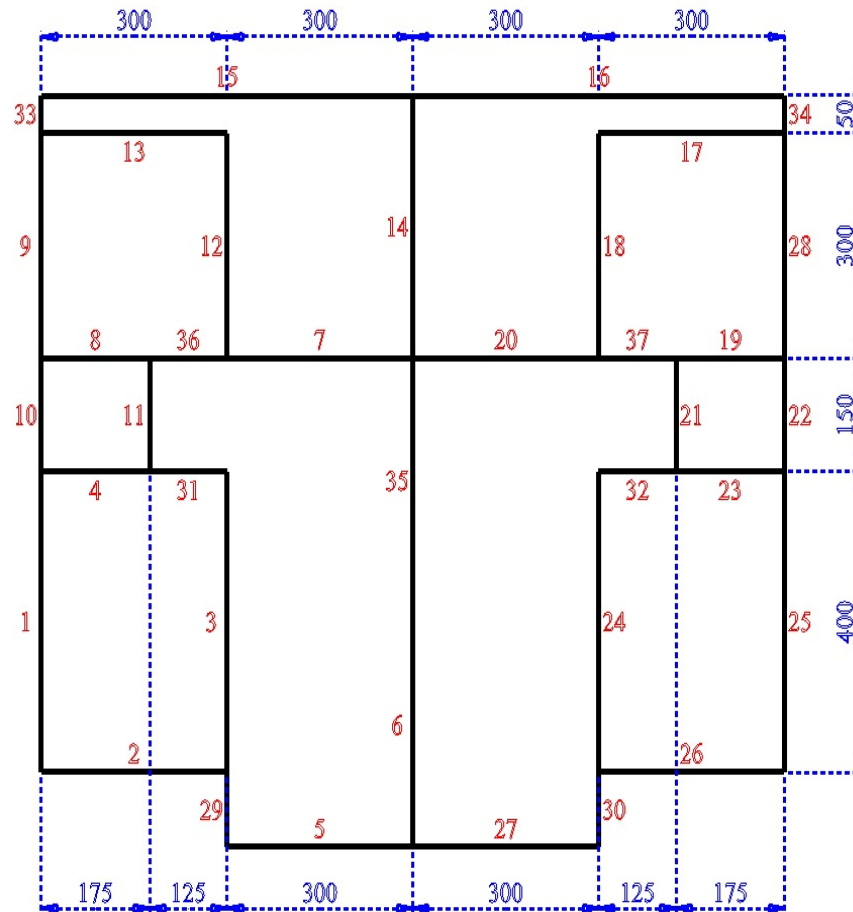


Figure 3.27. House 1 Brick Wall Numbering

All stresses and bending moments are considered for the envelope combinations. It takes the maximum values from all the maximum values at each load combination determined in section 3.3.

Axial stress in X direction (S11) is calculated as:

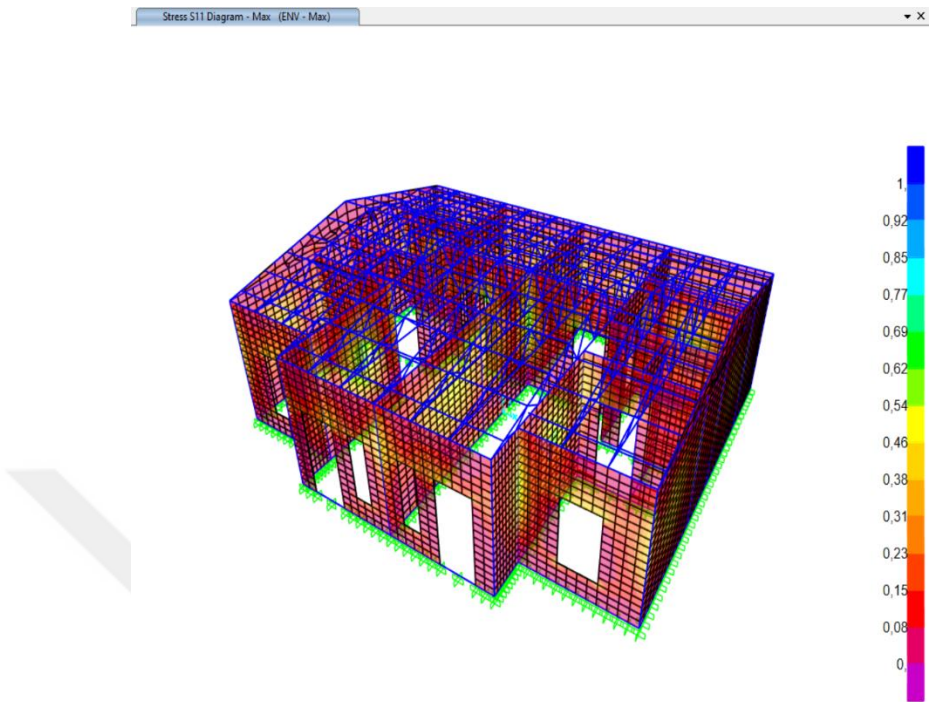


Figure 3.28. House 1 Axial Stress X direction (S11) General View (1).

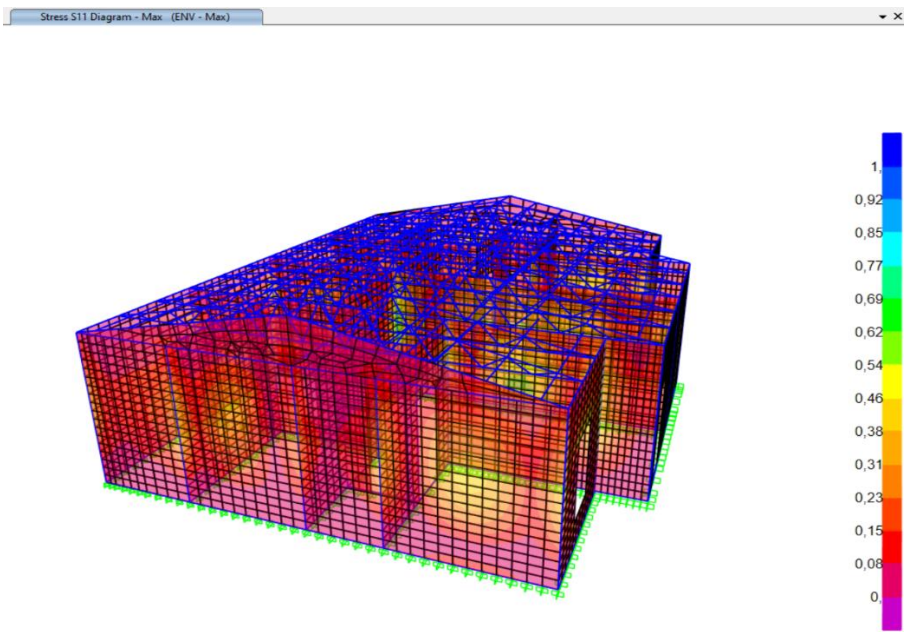


Figure 3.29. House 1 Axial Stress X direction (S11) General View (2).

Axial stress in Y direction (S22) is calculated as:

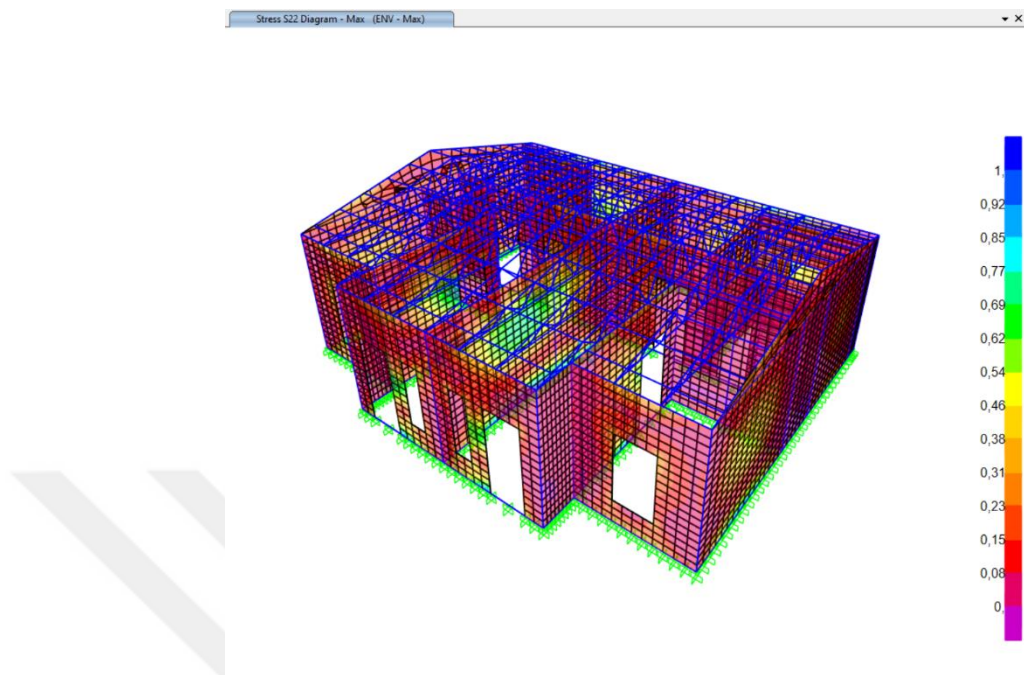


Figure 3.30. House 1 Axial stress Y direction (S22) general view (1).

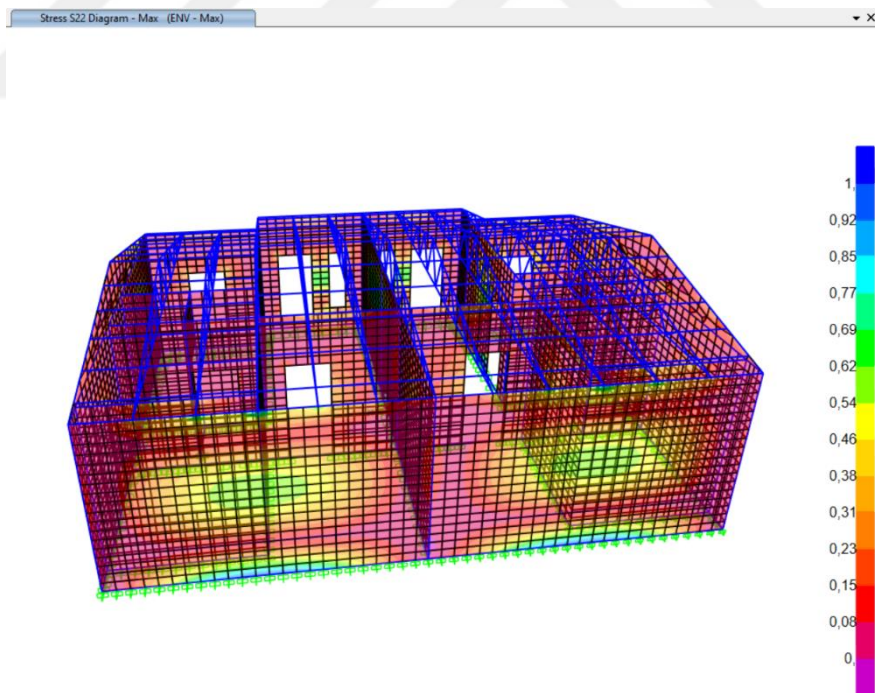


Figure 3.31. House 1 Axial stress Y direction (S22) general view (2).

Shear stress (S12) is calculated as:

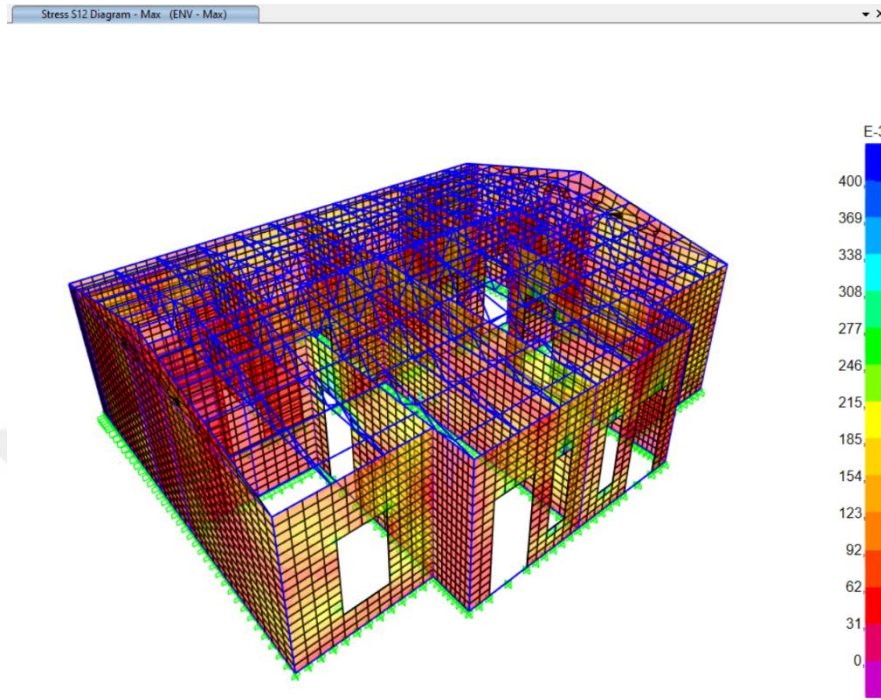


Figure 3.32. House 1 Shear Stress (S12) general view.

From the analysis result, maximum tension stress in X direction (S11) and Y direction (S22) are tabulated at each wall.

As an example, at wall number 1 the following values are obtained.

Maximum tension stress in X direction (S11) is calculated as 0.459 MPa.

Maximum tension stress in Y direction (S22) is calculated as 0.685 MPa.

Maximum and minimum shear stress (S12) is calculated as 0.175 MPa.

To find the maximum shear stresses applied to the masonry walls, the following formulas are used, which are based on the Mohr Circle stress concept:

$$\tau_{max} = \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2}$$

At wall 1, the maximum shear stress is calculated as:

$$\tau_{max} = \sqrt{\left(\frac{0.459 - 0.685}{2}\right)^2 + 0.175^2} = 0.208 \text{ MPa}$$

Using the same calculation explained above, maximum shear stress at all walls is tabulated in Table 10. Moreover, considering the out-of-plane failure mechanism in which the brick wall can fail due to vertical bending, the maximum bending moment that occur at each wall is obtained from SAP2000 analysis result and the flexural stress at each wall is calculated based on the following formula:

$$\sigma_{flex} = \frac{M * y}{I}$$

M11 and M22 are checked for out-of-plane failure mechanisms when calculating flexure stress. Figure 3.33 explains different failure mechanisms caused by M11 and M22 bending moments.

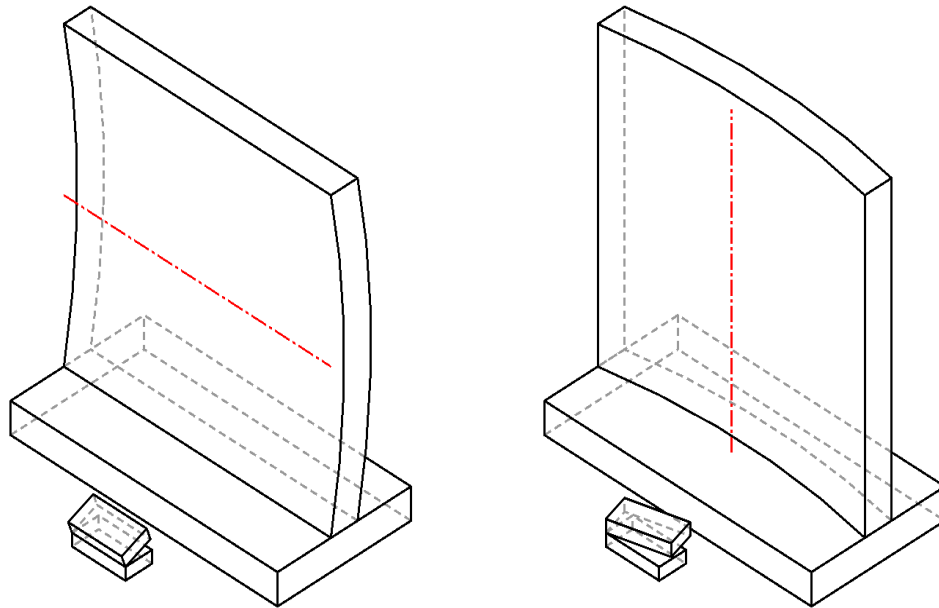


Figure 3.33. Out-of-plane failure caused by M22 (left) and M11 (right) [2]

For M11, flexural stress is calculated by:

$$\sigma_{M11} = \frac{M11 * (\frac{t}{2})}{(\frac{h * t^3}{12})}$$

For M22, flexural stress is calculated by:

$$\sigma_{M22} = \frac{M22 * (\frac{t}{2})}{(\frac{l * t^3}{12})}$$

for example, at wall number 1, the following moment values are obtained from SAP2000:

M11: 0.764 kNm/m

M22: 1.242 kNm/m

Moments given from analysis are given per unit of in-plane length, thus:

$$M11 = 0.764 * h = 0.764 * 3.6 = 2.7504 \text{ kNm}$$

$$\sigma_{M11} = \frac{M11 * (\frac{t}{2})}{(\frac{h * t^3}{12})} = \frac{2.7504 * (\frac{0.1}{2})}{(\frac{3.6 * 0.1^3}{12})} = 458 \text{ kPa} = 0.458 \text{ MPa}$$

$$M22 = 1.242 * l = 1.242 * 4 = 4.968 \text{ kNm}$$

$$\sigma_{M22} = \frac{M22 * (\frac{t}{2})}{(\frac{l * t^3}{12})} = \frac{4.968 * (\frac{0.1}{2})}{(\frac{4 * 0.1^3}{12})} = 745 \text{ kPa} = 0.745 \text{ MPa}$$

The same calculations are done to all brick walls and the results are tabulated in Table 10:

Table 10. Flexure and Shear Stress Result for Unstrengthened House 1 Model.

wall no	length (m)	height (m)	S11 Max (MPa)	S22 Max (MPa)	S12 Max (MPa)	S12 Princ. (MPa)	M11 (kNm/m)	M22 (kNm/m)	σ_{M11} (MPa)	σ_{M22} (MPa)
1	4	3.6	0.459	0.685	0.175	0.501	0.764	1.242	0.458	0.745
2	3	3.6	0.433	0.611	0.288	0.527	0.664	0.7533	0.398	0.452
3	4	3.6	1.117	1.623	0.384	1.135	1.8878	2.7646	1.133	1.659
4	1.75	3.6	0.237	0.148	0.063	0.955	0.3428	0.2963	0.206	0.178
5	3	3.6	0.711	0.958	0.313	0.772	1.271	1.691	0.763	1.015
6	4	3.6	0.555	1.191	0.224	0.590	0.95	1.98	0.570	1.188
7	3	3.6	0.644	0.633	0.257	0.710	1.062	1.211	0.637	0.727
8	1.5	3.6	0.203	0.186	0.068	1.069	0.3222	0.1323	0.193	0.079
9	3	3.6	0.372	0.319	0.088	0.478	0.6147	0.6016	0.369	0.361
10	1.5	3.6	0.214	0.083	0.046	1.072	0.5201	0.1007	0.312	0.060
11	1.5	3.6	0.264	0.31	0.074	1.083	0.2594	0.1651	0.156	0.099
12	3	3.6	0.403	0.492	0.151	0.502	0.674	0.862	0.404	0.517
13	3	3.6	0.387	0.407	0.189	0.490	0.608	0.729	0.365	0.437
14	3.5	3.6	0.618	0.462	0.121	0.620	0.8386	0.823	0.503	0.494
15	6	3.6	0.598	0.984	0.177	1.341	1.039	1.787	0.623	1.072
16	6	3.6	0.597	0.986	0.174	1.340	1.038	1.791	0.623	1.075
17	3	3.6	0.331	0.415	0.164	0.447	0.529	0.727	0.317	0.436
18	3.5	3.6	0.378	0.452	0.144	0.381	0.644	0.814	0.386	0.488
19	1.5	3.6	0.199	0.165	0.068	1.069	0.3206	0.1338	0.192	0.080
20	3	3.6	0.612	0.641	0.238	0.682	1.08	1.14	0.648	0.684
21	1.5	3.6	0.203	0.208	0.065	1.069	0.252	0.1499	0.151	0.090
22	1.5	3.6	0.281	0.076	0.052	1.087	0.4598	0.1301	0.276	0.078
23	1.5	3.6	0.32	0.142	0.094	1.098	0.4968	0.2919	0.298	0.175
24	4	3.6	0.746	1.167	0.366	0.772	1.1539	1.9397	0.692	1.164
25	4	3.6	0.523	0.91	0.189	0.560	0.882	1.5683	0.529	0.941
26	3	3.6	0.482	0.474	0.283	0.568	0.658	0.7605	0.395	0.456
27	3	3.6	0.722	0.975	0.321	0.782	1.277	1.712	0.766	1.027
28	3	3.6	0.336	0.3103	0.097	0.450	0.535	0.6458	0.321	0.387
29	1	3.6	0.386	0.099	0.077	1.356	0.669	0.1948	0.401	0.117
30	1	3.6	0.302	0.156	0.053	1.335	0.5452	0.1459	0.327	0.088
31	1.5	3.6	0.378	0.148	0.08	1.116	0.647	0.2963	0.388	0.178
32	1.5	3.6	0.277	0.12	0.112	1.086	0.4949	0.2919	0.297	0.175
33	0.5	3.6	0.394	0.084	0.043	1.599	0.613	0.153	0.368	0.092
34	0.5	3.6	0.39	0.078	0.053	1.598	0.6055	0.1529	0.363	0.092
35	2.5	3.6	0.632	1.026	0.202	0.838	1.028	1.481	0.617	0.889
36	1.5	3.6	0.276	0.236	0.147	1.086	0.3345	0.255	0.201	0.153
37	1.5	3.6	0.325	0.256	0.105	1.099	0.3041	0.1345	0.182	0.081

The calculated shear stresses and flexural stresses are compared with shear strength and flexural strength test results [2], which are the following values:

Flexural Strength of Wall: 0.408 MPa

Shear Stress of Clay Brick: 0.298 MPa

If both shear and flexural stresses obtained at each axis are larger or equal to flexural strength test results, the wall is considered safe; otherwise, it is considered not safe and further strengthening will be required. The result for house 1 is given in Table 11.

Table 11. Flexure and Shear Stresses Checked for House 1 Unstrengthened Model

Wall No	S12 Princ. (MPa)	σ_{M22} (MPa)	σ_{M11} (MPa)	Flexure Check (F22)	Flexure Check (F11)	Shear Check (S12 Prin.)
1	0.501	0.745	0.458	FAILURE	FAILURE	OK
2	0.527	0.452	0.398	FAILURE	OK	FAILURE
3	1.135	1.659	1.133	FAILURE	FAILURE	FAILURE
4	0.955	0.178	0.206	OK	OK	OK
5	0.772	1.015	0.763	FAILURE	FAILURE	FAILURE
6	0.590	1.188	0.570	FAILURE	FAILURE	FAILURE
7	0.710	0.727	0.637	FAILURE	FAILURE	OK
8	1.069	0.079	0.193	OK	OK	OK
9	0.478	0.361	0.369	OK	OK	OK
10	1.072	0.060	0.312	OK	OK	OK
11	1.083	0.099	0.156	OK	OK	OK
12	0.502	0.517	0.404	FAILURE	OK	OK
13	0.490	0.437	0.365	FAILURE	OK	OK
14	0.620	0.494	0.503	FAILURE	FAILURE	OK
15	1.341	1.072	0.623	FAILURE	FAILURE	OK
16	1.340	1.075	0.623	FAILURE	FAILURE	OK
17	0.447	0.436	0.317	FAILURE	OK	OK
18	0.381	0.488	0.386	FAILURE	OK	OK
19	1.069	0.080	0.192	OK	OK	OK
20	0.682	0.684	0.648	FAILURE	FAILURE	OK
21	1.069	0.090	0.151	OK	OK	OK
22	1.087	0.078	0.276	OK	OK	OK
23	1.098	0.175	0.298	OK	OK	OK
24	0.772	1.164	0.692	FAILURE	FAILURE	FAILURE

Table 11 (continued)

25	0.560	0.941	0.529	FAILURE	FAILURE	OK
26	0.568	0.456	0.395	FAILURE	OK	OK
27	0.782	1.027	0.766	FAILURE	FAILURE	FAILURE
28	0.450	0.387	0.321	OK	OK	OK
29	1.356	0.117	0.401	OK	OK	OK
30	1.335	0.088	0.327	OK	OK	OK
31	1.116	0.178	0.388	OK	OK	OK
32	1.086	0.175	0.297	OK	OK	OK
33	1.599	0.092	0.368	OK	OK	OK
34	1.598	0.092	0.363	OK	OK	OK
35	0.838	0.889	0.617	FAILURE	FAILURE	OK
36	1.086	0.153	0.201	OK	OK	OK
37	1.099	0.081	0.182	OK	OK	OK

From Table 11, it can be said that wall numbers 1, 2, 3, 5, 6, 7, 12, 13, 14, 15, 16, 17, 18, 20, 24, 25, 26, 27, and 35 need to be strengthened and the strengthened model will be discussed more detail in Chapter 4.

3.4.4 Analysis of Unstrengthened House 2 Model

Each wall is numbered as shown in Figure 3.34:

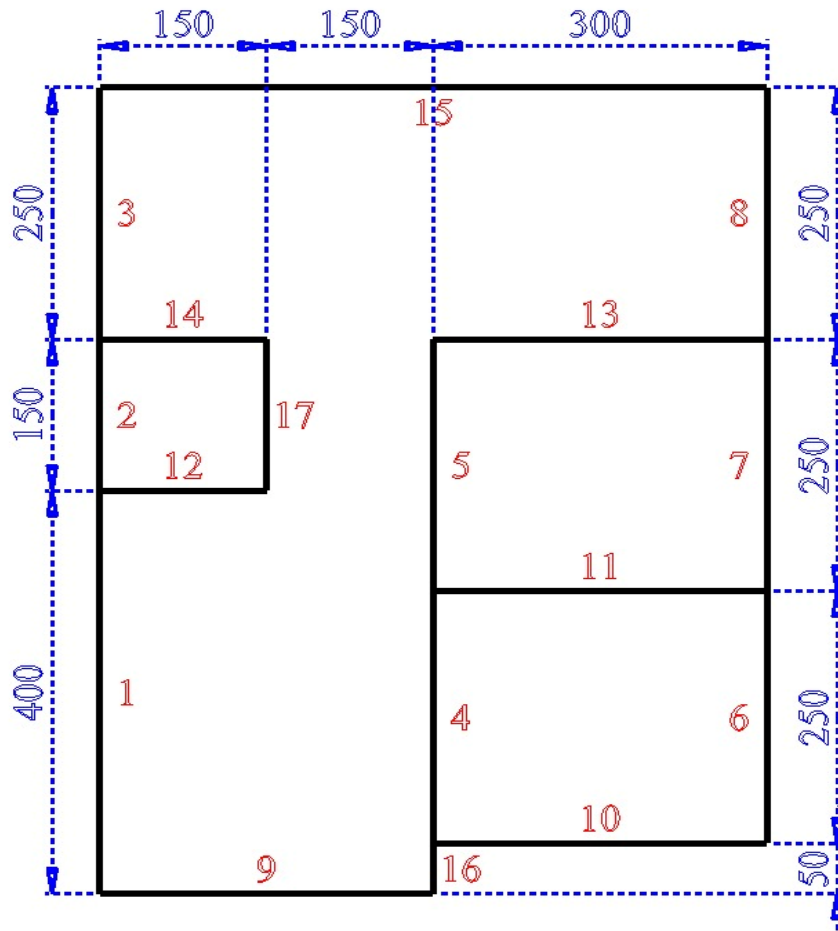


Figure 3.34. House 2 Brick Wall Numbering.

All stresses and bending moments are considered for the envelope combinations. It takes the maximum values from all the maximum values at each load combination determined in section 3.3.

Axial stress in X direction (S11) is calculated as:

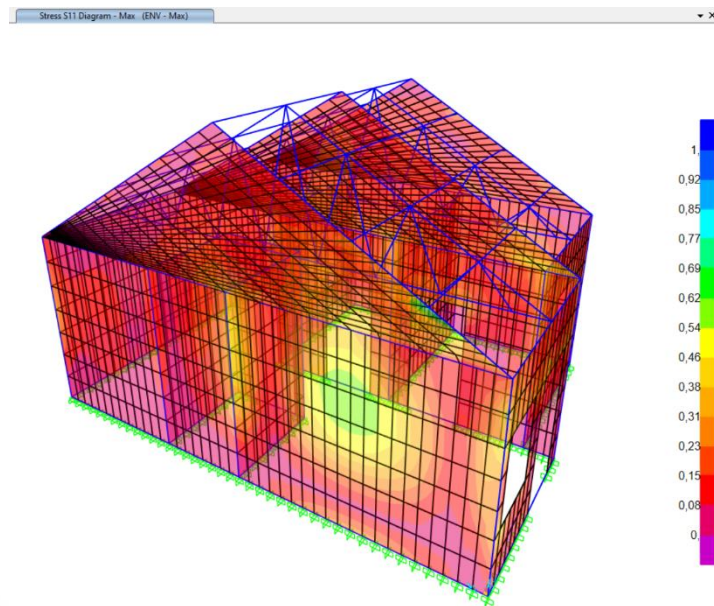


Figure 3.35. House 2 Axial Stress X Direction (S11) General View (1).

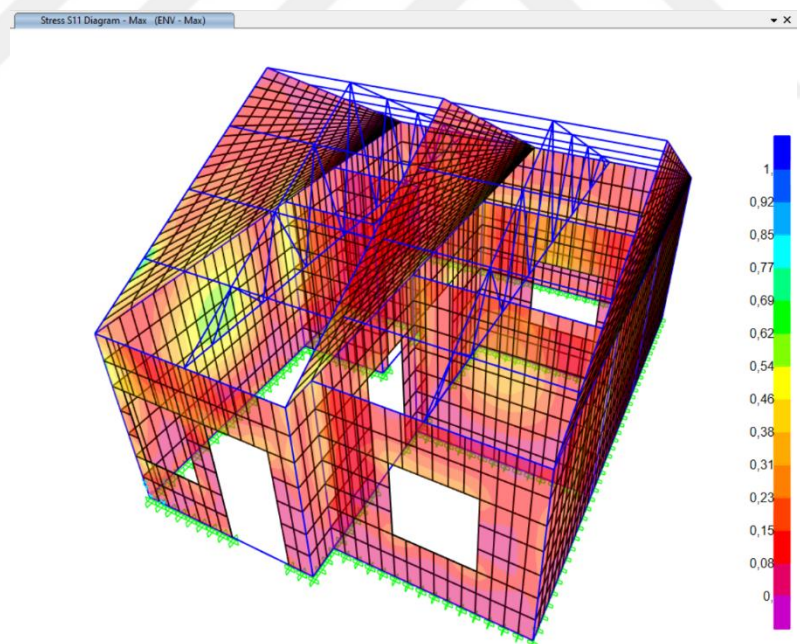


Figure 3.36. House 2 Axial stress X direction (S11) general view (2).

Axial stress in Y direction (S22) is calculated as:

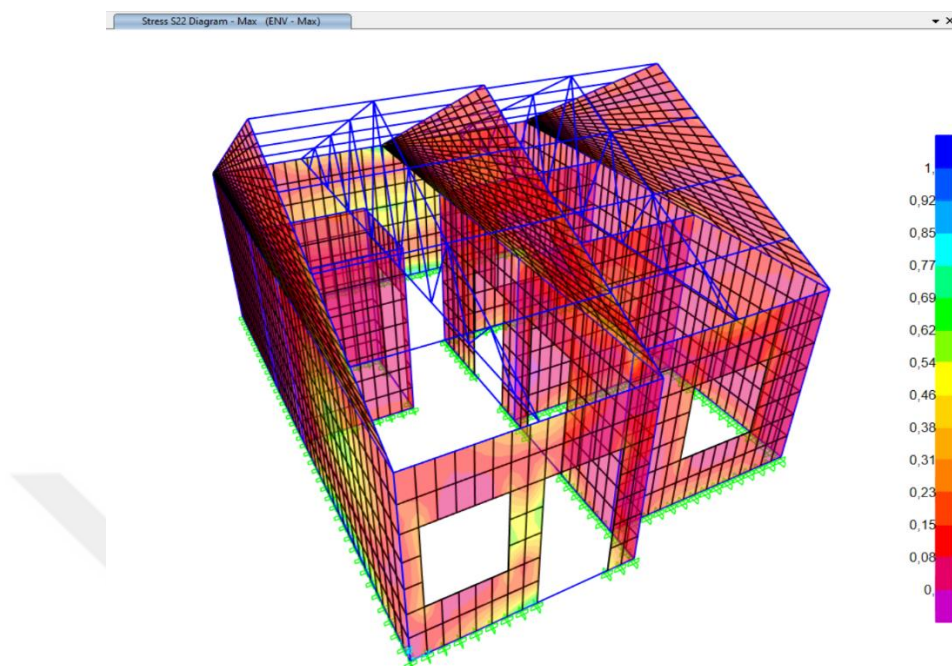


Figure 3.37. House 2 Axial stress Y direction (S22) general view (1).

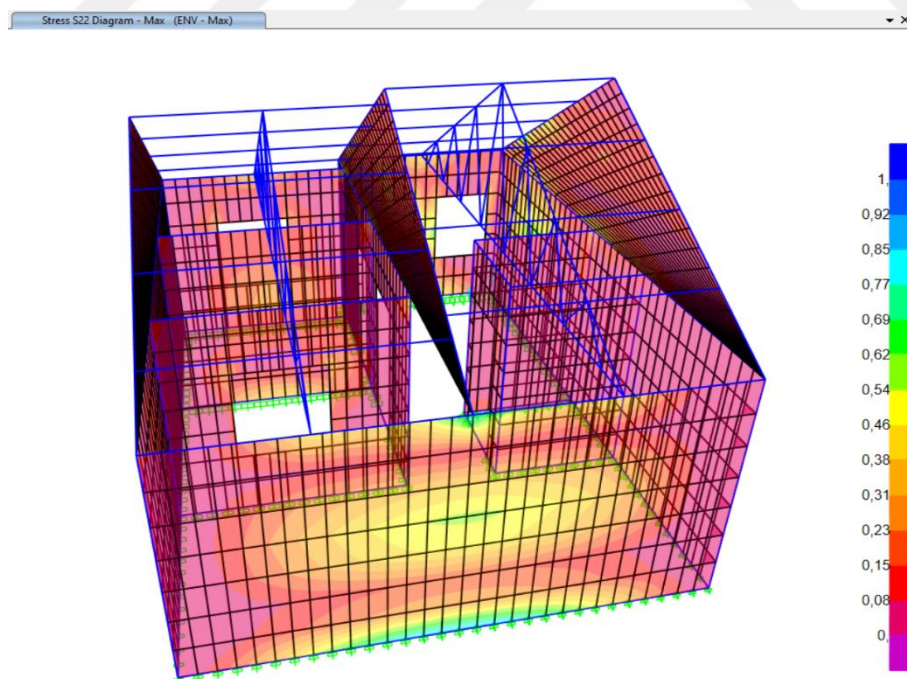


Figure 3.38. House 2 Axial stress Y direction (S22) general view (2).

Shear stress (S12) is calculated as:

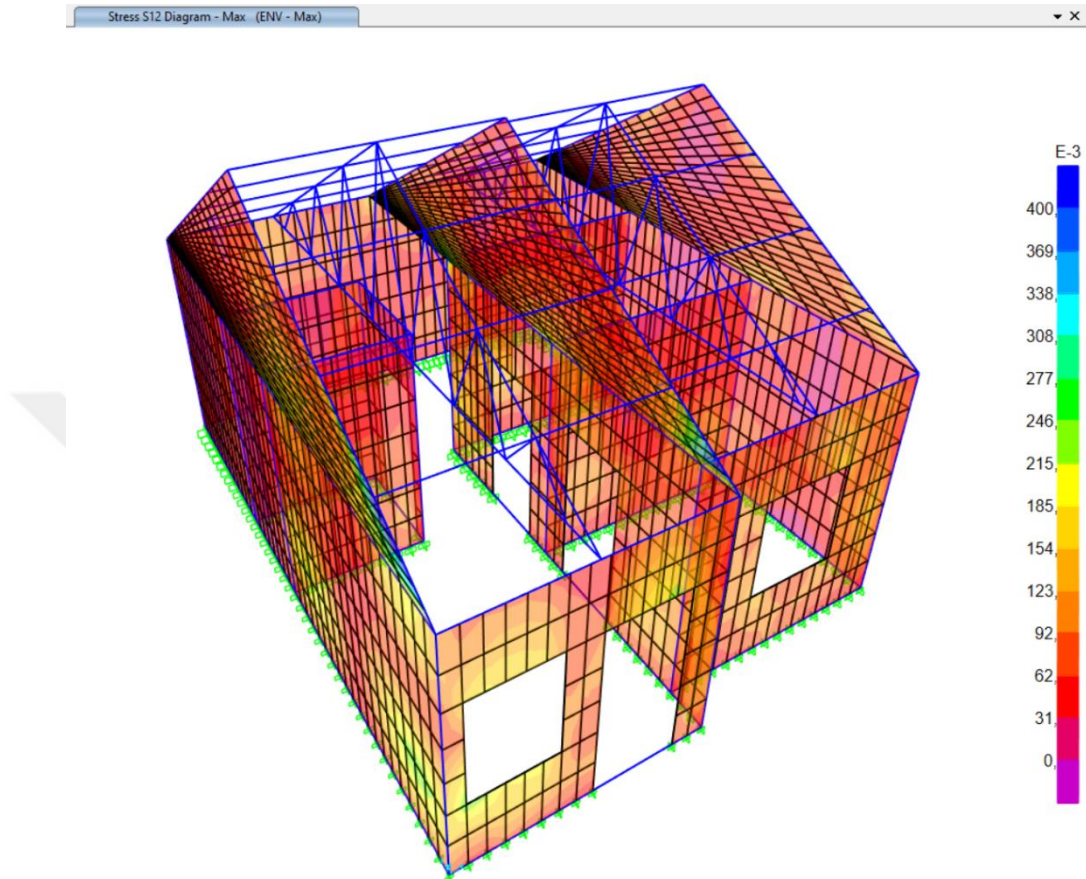


Figure 3.39. House 2 Shear Stress (S12) general view.

From the analysis result, maximum tension stress in X direction (S11) and Y direction (S22) are tabulated at each wall.

As an example, at wall number 2 the following values are obtained.

Maximum tension stress in X direction (S11) is calculated as 0.358 MPa.

Maximum tension stress in Y direction (S22) is calculated as 0.114 MPa.

Maximum and minimum shear stress (S12) is calculated as 0.079 MPa.

To find the maximum shear stresses applied to the masonry walls, the following formulas are used, which are based on the Mohr Circle stress concept:

$$\tau_{max} = \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2}$$

At wall 6, the maximum shear stress is calculated as:

$$\tau_{max} = \sqrt{\left(\frac{0.358 - 0.114}{2}\right)^2 + 0.079^2} = 0.145 \text{ MPa}$$

Moreover, considering the out-of-plane failure mechanism in which the brick wall can fail due to vertical bending, the maximum bending moment that occurs at each wall is obtained from SAP2000 analysis result and the flexural stress at each wall is calculated based on the following formula:

$$\sigma_{flex} = \frac{M * y}{I}$$

For M11, flexural stress is calculated by:

$$\sigma_{M11} = \frac{M11 * (\frac{t}{2})}{(\frac{h * t^3}{12})}$$

For M22, flexural stress is calculated by:

$$\sigma_{M22} = \frac{M22 * (\frac{t}{2})}{(\frac{l * t^3}{12})}$$

M11: 0.6 kNm/m

M22: 0.2197 kNm/m

Moments given from analysis are given per unit of in-plane length, thus:

$$M11 = 0.6 * h = 0.6 * 3.15 = 1.89 \text{ kNm}$$

$$\sigma_{M11} = \frac{M11 * (\frac{t}{2})}{(\frac{h * t^3}{12})} = \frac{1.89 * (\frac{0.1}{2})}{(\frac{3.15 * 0.1^3}{12})} = 360 \text{ kPa} = 0.36 \text{ MPa}$$

$$M22 = 0.2197 * l = 0.2197 * 1.5 = 0.32955 \text{ kNm}$$

$$\sigma_{M22} = \frac{M22 * \left(\frac{t}{2}\right)}{\left(\frac{l * t^3}{12}\right)} = \frac{0.32955 * \left(\frac{0.1}{2}\right)}{\left(\frac{1.5 * 0.1^3}{12}\right)} = 131.82 \text{ kPa} = 0.132 \text{ MPa}$$

The same calculations are done to all the brick walls; the results are tabulated in Table 12.

Table 12. Flexure and Shear Stresses Results of House 2 Unstrengthened Model.

wall no	Length (m)	Height (m)	S11 Max (MPa)	S22 Max (MPa)	S12 Max (MPa)	S12 Princ (MPa)	M11 (kNm/m)	M22 (kNm/m)	σ_{M11} (MPa)	σ_{M22} (MPa)
1	4	3.15	0.813	1.06	0.235	0.265	1.384	1.868	0.830	1.121
2	1.5	3.15	0.358	0.114	0.079	0.145	0.600	0.220	0.360	0.132
3	2.5	3.15	0.197	0.135	0.06	0.068	0.356	0.290	0.214	0.174
4	2.5	3.15	0.388	0.409	0.177	0.177	0.512	0.830	0.307	0.498
5	2.5	3.15	0.637	0.447	0.184	0.207	1.046	0.909	0.627	0.545
6	2.5	3.15	0.291	0.257	0.079	0.081	0.493	0.496	0.296	0.298
7	2.5	3.15	0.319	0.202	0.072	0.093	0.534	0.405	0.320	0.243
8	2.5	3.15	0.271	0.227	0.097	0.099	0.490	0.451	0.294	0.271
9	3	3.15	0.62	0.811	0.229	0.248	0.990	1.426	0.594	0.856
10	3	3.15	0.364	0.355	0.239	0.239	0.534	0.522	0.321	0.313
11	3	3.15	0.439	0.429	0.122	0.122	0.739	0.795	0.443	0.477
12	1.75	3.15	0.384	0.105	0.139	0.197	0.559	0.246	0.335	0.148
13	3	3.15	0.509	0.398	0.229	0.236	0.821	0.716	0.493	0.429
14	1.75	3.15	0.218	0.109	0.102	0.116	0.395	0.261	0.237	0.157
15	6	3.15	0.344	0.946	0.137	0.331	0.609	1.596	0.365	0.958
16	0.5	3.15	0.578	0.204	0.126	0.143	0.931	0.345	0.559	0.207
17	1.5	3.15	0.13	0.281	0.081	0.111	0.277	0.164	0.166	0.099

The calculated shear and flexural stresses are compared with shear strength and flexural strength test results [2]. Which are the following values:

Flexural Strength of Wall: 0.408 MPa

Shear Stress of Clay Brick: 0.298 MPa

If both shear stresses and flexural stresses obtained at each axis are larger or equal to flexural strength test results, the wall is considered safe; otherwise, it is considered not safe and further strengthening will be required. The result for house 2 is given in Table 13.

Table 13. Flexure and Shear Stresses Checked for House 2 Unstrengthen Model

Wall No	S12 Princ. (MPa)	σ_{M22} (MPa)	σ_{M11} (MPa)	Flexure Check (F22)	Flexure Check (F11)	Shear Check (S12 Prin.)
1	0.265	0.830	1.121	FAILURE	FAILURE	OK
2	0.145	0.360	0.132	OK	OK	OK
3	0.068	0.214	0.174	OK	OK	OK
4	0.177	0.307	0.498	FAILURE	OK	OK
5	0.207	0.627	0.545	FAILURE	FAILURE	OK
6	0.081	0.296	0.298	OK	OK	OK
7	0.093	0.320	0.243	OK	OK	OK
8	0.099	0.294	0.271	OK	OK	OK
9	0.248	0.594	0.856	FAILURE	FAILURE	OK
10	0.239	0.321	0.313	OK	OK	OK
11	0.122	0.443	0.477	FAILURE	FAILURE	OK
12	0.197	0.335	0.148	OK	OK	OK
13	0.236	0.493	0.429	FAILURE	FAILURE	OK
14	0.116	0.237	0.157	OK	OK	OK
15	0.331	0.365	0.958	FAILURE	OK	FAILURE
16	0.143	0.559	0.207	OK	OK	OK
17	0.111	0.166	0.099	OK	OK	OK

From Table 13, it can be said that wall numbers 1, 4, 5, 9, 11, 13, and 15 need to be strengthened and the strengthened model will be discussed in detail in Chapter 4.

3.4.5 Analysis of Unstrengthened House 3 Model

Each wall is numbered as shown in Figure 3.40:

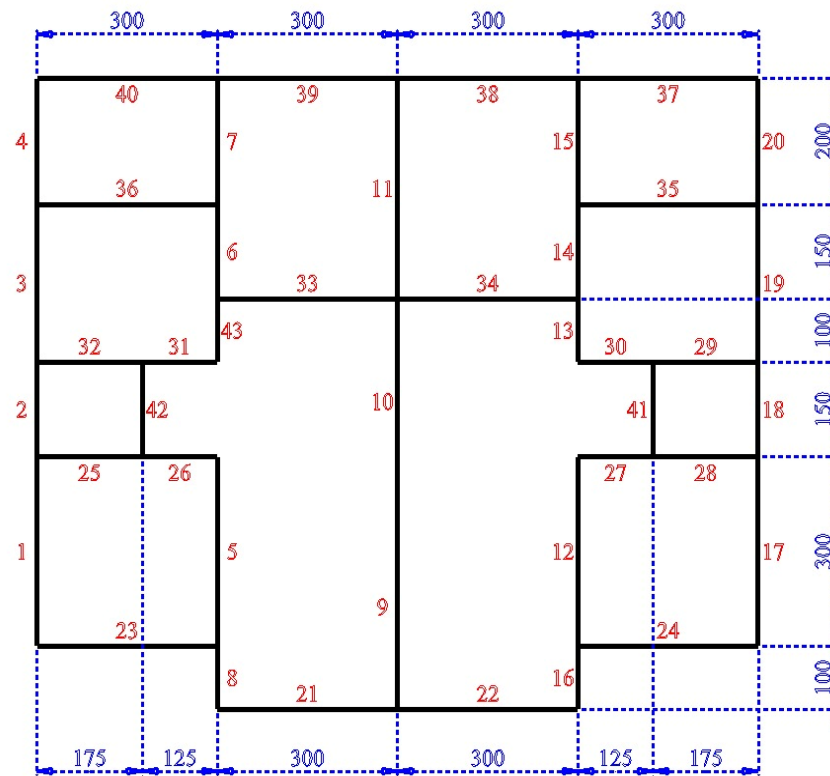


Figure 3.40. House 3 Brick Wall numbering

All stresses and bending moments are considered for the envelope combinations. It takes the maximum values from all the maximum values at each load combination determined in the previous section.

Axial stress in X direction (S11) is calculated as:

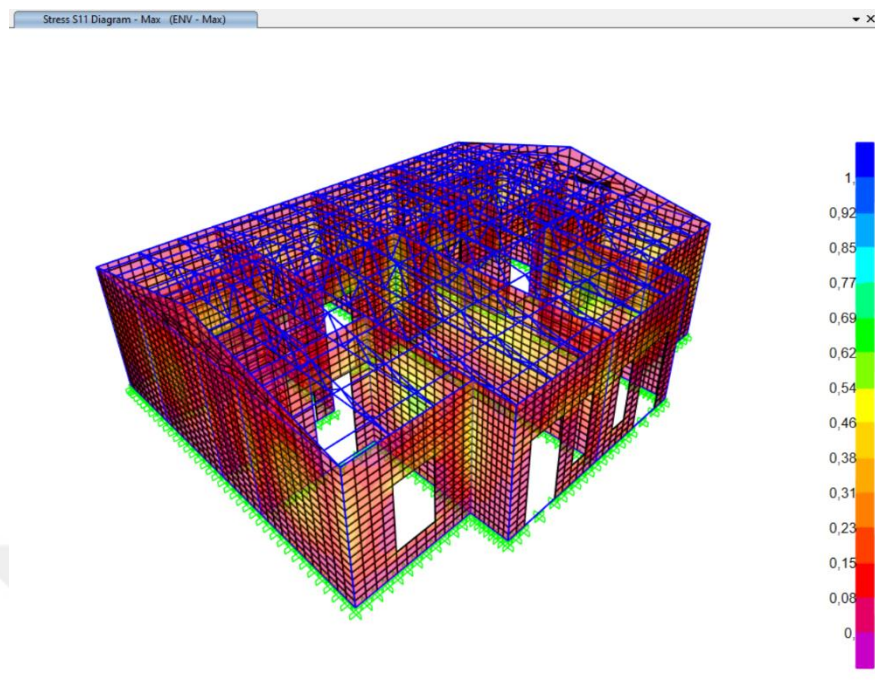


Figure 3.41. House 3 Axial Stress X Direction (S11) General View (1).

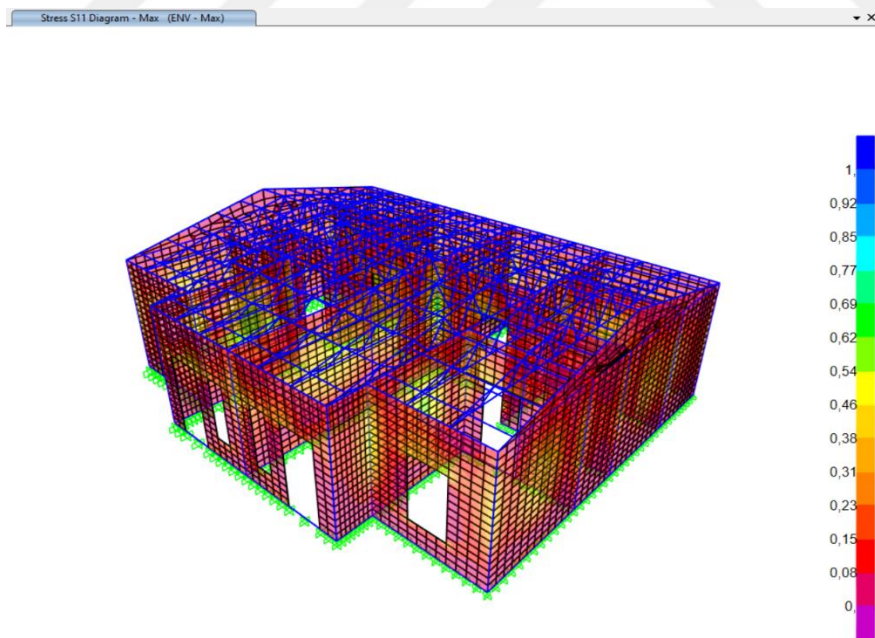


Figure 3.42. House 3 Axial stress X direction (S11) general view (2).

Axial stress in Y direction (S22) is calculated as:

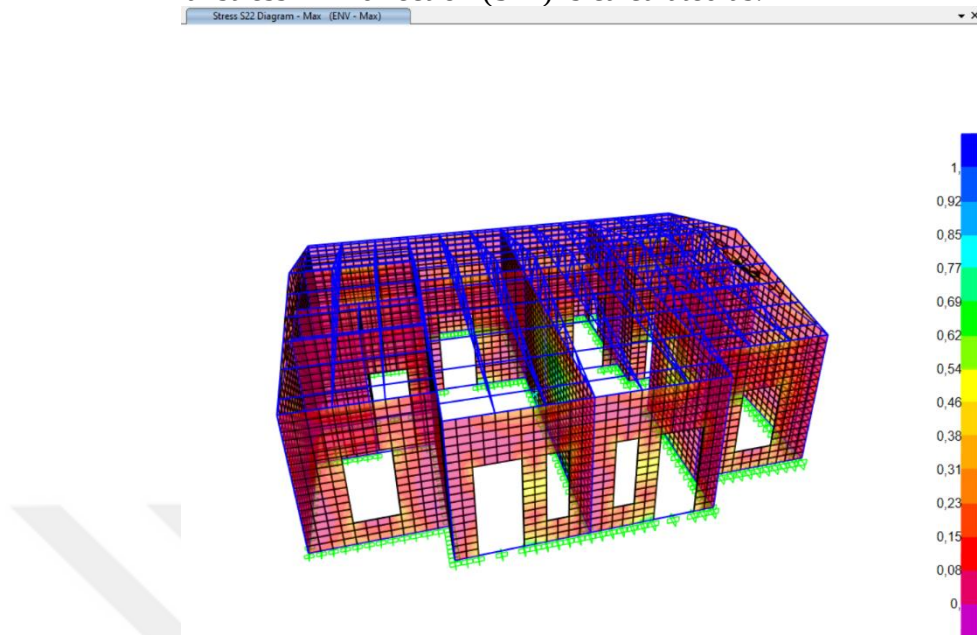


Figure 3.43. House 3 Axial stress Y direction (S22) general view (1).

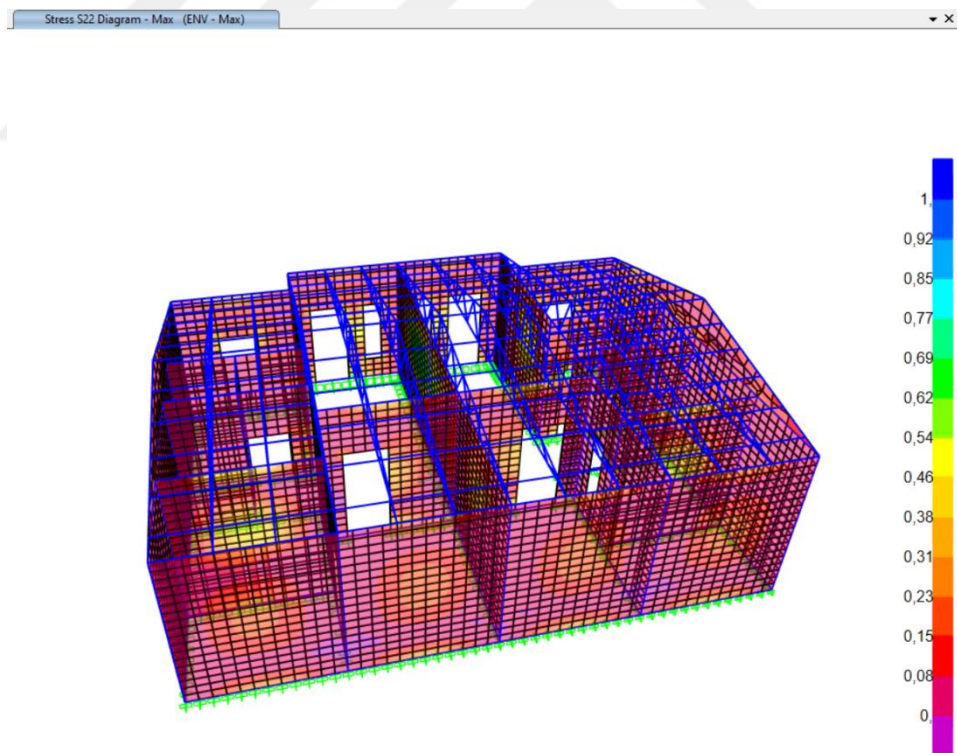


Figure 3.44. House 3 Axial stress Y direction (S22) general view (2).

Shear stress (S12) is calculated as:

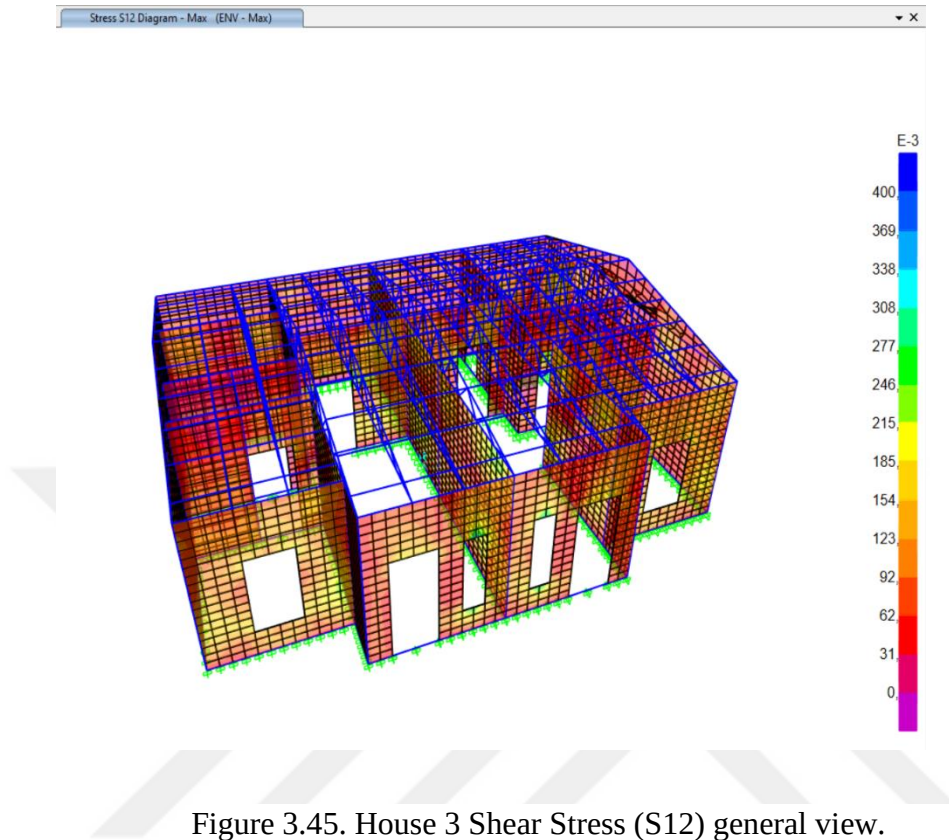


Figure 3.45. House 3 Shear Stress (S12) general view.

From the analysis result, maximum tension stress in X direction (S11) and Y direction (S22) are tabulated at each wall.

As an example, at wall number 5 the following values are obtained.

Maximum tension stress in X direction (S11) is calculated as 0.775 MPa.

Maximum tension stress in Y direction (S22) is calculated as 0.744 MPa.

Maximum and minimum shear stress (S12) is calculated as 0.185 MPa.

To find the maximum shear stresses applied to the masonry walls, the following formulas are used, which are based on the Mohr Circle stress concept:

$$\tau_{max} = \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2}$$

At wall 5, the maximum shear stress is calculated as:

$$\tau_{max} = \sqrt{\left(\frac{0.775 - 0.744}{2}\right)^2 + 0.185^2} = 0.186 \text{ MPa}$$

Moreover, considering the out-of-plane failure mechanism in which the brick wall can fail due to vertical bending, the maximum bending moment that occur at each wall is obtained from SAP2000 analysis result and the flexural stress at each wall is calculated based on the following formula:

$$\sigma_{flex} = \frac{M * y}{I}$$

For M11, flexural stress is calculated by:

$$\sigma_{M11} = \frac{M11 * \left(\frac{t}{2}\right)}{\left(\frac{h * t^3}{12}\right)}$$

For M22, flexural stress is calculated by:

$$\sigma_{M22} = \frac{M22 * \left(\frac{t}{2}\right)}{\left(\frac{l * t^3}{12}\right)}$$

For example, at wall number 5, the following moment values are obtained from SAP2000:

M11: 1.271 kNm/m

M22: 1.283 kNm/m

Moments given from analysis are given per unit of in-plane length, thus:

$$M11 = 1.4771 * h = 1.271 * 3.60 = 4.5756 \text{ kNm}$$

$$\sigma_{M11} = \frac{M11 * \left(\frac{t}{2}\right)}{\left(\frac{h * t^3}{12}\right)} = \frac{4.5756 * \left(\frac{0.1}{2}\right)}{\left(\frac{3.60 * 0.1^3}{12}\right)} = 763 \text{ kPa} = 0.763 \text{ MPa}$$

$$M22 = 1.2381 * l = 1.283 * 3 = 3.849 \text{ kNm}$$

$$\sigma_{M22} = \frac{M22 * \left(\frac{t}{2}\right)}{\left(\frac{l * t^3}{12}\right)} = \frac{3.849 * \left(\frac{0.1}{2}\right)}{\left(\frac{3 * 0.1^3}{12}\right)} = 770 \text{ kPa} = 0.770 \text{ MPa}$$

The same calculations are done to all the brick walls and the results are tabulated in Table 14.

Table 14. Flexure and Shear Stresses Results of House 3 Unstrengthened Model

wall no	length (m)	height (m)	S11 Max (MPa)	S22 Max (MPa)	S12 Max (MPa)	S12 Princ (MPa)	M11 (kNm/m)	M22 (kNm/m)	σ_{M11} (MPa)	σ_{M22} (MPa)
1	3	3.6	0.539	0.545	0.153	0.153	0.912	0.956	0.547	0.574
2	1.5	3.6	0.214	0.162	0.06	0.065	0.366	0.292	0.220	0.175
3	2.5	3.6	0.343	0.256	0.086	0.096	0.581	0.5	0.349	0.300
4	2	3.6	0.288	0.101	0.077	0.121	0.495	0.226	0.297	0.136
5	3	3.6	0.775	0.744	0.185	0.186	1.271	1.283	0.763	0.770
6	1.5	3.6	0.405	0.183	0.137	0.176	0.665	0.252	0.399	0.151
7	2	3.6	0.437	0.338	0.132	0.141	0.68	0.509	0.408	0.305
8	1	3.6	0.412	0.154	0.07	0.147	0.679	0.259	0.407	0.155
9	4	3.6	0.584	1.255	0.236	0.410	0.964	1.877	0.578	1.126
10	2.5	3.6	0.726	1.055	0.196	0.256	1.211	1.534	0.727	0.920
11	3.5	3.6	0.452	0.522	0.141	0.145	0.758	0.942	0.455	0.565
12	3	3.6	0.745	0.689	0.188	0.190	1.259	1.238	0.755	0.743
13	1	3.6	0.184	0.06	0.061	0.087	0.2881	0.098	0.173	0.059
14	2.5	3.6	0.38	0.22	0.215	0.229	0.53	0.284	0.318	0.170
15	2	3.6	0.311	0.247	0.159	0.162	0.495	0.576	0.297	0.346
16	1	3.6	0.398	0.203	0.061	0.115	0.66	0.336	0.396	0.202
17	3	3.6	0.556	0.555	0.147	0.147	0.94	0.976	0.564	0.586
18	1.5	3.6	0.222	0.165	0.062	0.068	0.378	0.293	0.227	0.176
19	2.5	3.6	0.463	0.34	0.115	0.130	0.6765	0.472	0.406	0.283
20	2	3.6	0.325	0.127	0.082	0.129	0.561	0.27	0.337	0.162
21	3	3.6	0.795	0.758	0.363	0.363	1.173	1.341	0.704	0.805
22	3	3.6	0.797	0.762	0.33	0.330	1.18	1.354	0.708	0.812
23	3	3.6	0.72	0.659	0.243	0.245	0.836	0.655	0.502	0.393
24	3	3.6	0.735	0.644	0.283	0.287	0.887	0.839	0.532	0.503
25	1.5	3.6	0.246	0.174	0.149	0.153	0.373	0.256	0.224	0.154
26	1.5	3.6	0.772	0.266	0.243	0.351	0.713	0.693	0.428	0.416
27	1.5	3.6	0.772	0.266	0.243	0.351	0.713	0.693	0.428	0.416
28	1.5	3.6	0.256	0.209	0.17	0.172	0.393	0.324	0.236	0.194
29	1.5	3.6	0.23	0.137	0.102	0.112	0.405	0.241	0.243	0.145
30	1.5	3.6	0.494	0.663	0.15	0.172	0.622	0.514	0.373	0.308
31	1.5	3.6	0.164	0.14	0.071	0.072	0.233	0.186	0.140	0.112
32	1.5	3.6	0.17	0.113	0.076	0.081	0.297	0.205	0.178	0.123
33	3	3.6	0.999	0.909	0.365	0.368	1.751	1.616	1.051	0.970

Table 14 (continued)

34	3	3.6	0.965	0.89	0.319	0.321	1.674	1.55	1.004	0.930
35	3	3.6	0.705	0.751	0.203	0.204	1.177	1.325	0.706	0.795
36	3	3.6	0.582	0.62	0.172	0.173	0.973	1.105	0.584	0.663
37	3	3.6	0.36	0.313	0.091	0.094	0.618	0.59	0.371	0.354
38	3	3.6	0.438	0.325	0.095	0.111	0.757	0.622	0.454	0.373
39	3	3.6	0.451	0.332	0.097	0.114	0.781	0.634	0.469	0.380
40	3	3.6	0.375	0.315	0.094	0.099	0.644	0.6	0.386	0.360
41	1.5	3.6	0.212	0.364	0.087	0.116	0.314	0.219	0.188	0.131
42	1.5	3.6	0.214	0.261	0.089	0.092	0.292	0.139	0.175	0.083
43	1	3.6	0.173	0.057	0.068	0.089	0.272	0.112	0.163	0.067

The calculated shear and flexural stresses are compared with shear strength and flexural strength test results [2]. Which are the following values:

Flexural Strength of Wall: 0.408 MPa

Shear Stress of Clay Brick: 0.298 MPa

If both shear and flexural stresses obtained at each axis are larger or equal to flexural strength test results, then the wall is considered safe; otherwise, it is considered not safe and further strengthening will be required. The result for house 3 is given in Table 15.

Table 15. Flexure and Shear Stresses Checked for House 3 Unstrengthened Model

Wall No	S12 Princ. (MPa)	σ_{M22} (Mpa)	σ_{M11} (Mpa)	Flexure Check (F22)	Flexure Check (F11)	Shear Check (S12 Prin.)
1	0.153	0.547	0.574	FAILURE	FAILURE	OK
2	0.065	0.220	0.175	OK	OK	OK
3	0.096	0.349	0.300	OK	OK	OK
4	0.121	0.297	0.136	OK	OK	OK
5	0.186	0.763	0.770	FAILURE	FAILURE	OK
6	0.176	0.399	0.151	OK	OK	OK
7	0.141	0.408	0.305	OK	OK	OK
8	0.147	0.407	0.155	OK	OK	OK
9	0.410	0.578	1.126	FAILURE	FAILURE	FAILURE
10	0.256	0.727	0.920	FAILURE	FAILURE	OK
11	0.145	0.455	0.565	FAILURE	FAILURE	OK

Table 15 (continued)

12	0.190	0.755	0.743	FAILURE	FAILURE	OK
13	0.087	0.173	0.059	OK	OK	OK
14	0.229	0.318	0.170	OK	OK	OK
15	0.162	0.297	0.346	OK	OK	OK
16	0.115	0.396	0.202	OK	OK	OK
17	0.147	0.564	0.586	FAILURE	FAILURE	OK
18	0.068	0.227	0.176	OK	OK	OK
19	0.130	0.406	0.283	OK	OK	OK
20	0.129	0.337	0.162	OK	OK	OK
21	0.363	0.704	0.805	FAILURE	FAILURE	FAILURE
22	0.330	0.708	0.812	FAILURE	FAILURE	FAILURE
23	0.245	0.502	0.393	OK	FAILURE	OK
24	0.287	0.532	0.503	FAILURE	FAILURE	OK
25	0.153	0.224	0.154	OK	OK	OK
26	0.351	0.428	0.416	FAILURE	FAILURE	FAILURE
27	0.351	0.428	0.416	FAILURE	FAILURE	FAILURE
28	0.172	0.236	0.194	OK	OK	OK
29	0.112	0.243	0.145	OK	OK	OK
30	0.172	0.373	0.308	OK	OK	OK
31	0.072	0.140	0.112	OK	OK	OK
32	0.081	0.178	0.123	OK	OK	OK
33	0.368	1.051	0.970	FAILURE	FAILURE	FAILURE
34	0.321	1.004	0.930	FAILURE	FAILURE	FAILURE
35	0.204	0.706	0.795	FAILURE	FAILURE	OK
36	0.173	0.584	0.663	FAILURE	FAILURE	OK
37	0.094	0.371	0.354	OK	OK	OK
38	0.111	0.454	0.373	OK	FAILURE	OK
39	0.114	0.469	0.380	OK	FAILURE	OK
40	0.099	0.386	0.360	OK	OK	OK
41	0.116	0.188	0.131	OK	OK	OK
42	0.092	0.175	0.083	OK	OK	OK
43	0.089	0.163	0.067	OK	OK	OK

From Table 13, it can be said that wall numbers 1, 5, 9, 10, 11, 12, 17, 21, 22, 23, 24, 26, 27, 33, 34, 35, 36, 38, and 39 need to be strengthened and the strengthened model will be discussed in detail at Chapter 4.

3.4.6 Yield Line Theory

The failure line at each wall can be considered using the yield line theory. For modeled houses, all walls are supported at both ends while the top parts are considered free because of the unavailability of a concrete slab at the top. Two failure modes are considered in this model which are shown in Figure 3.46 and Figure 3.47.

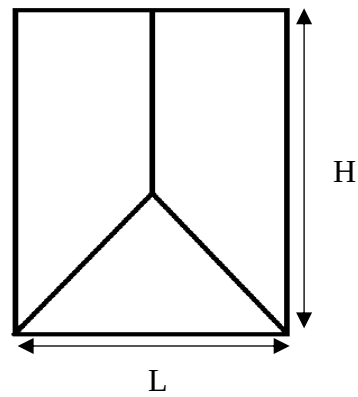


Figure 3.46. Failure lines for $L < 2H$ case.

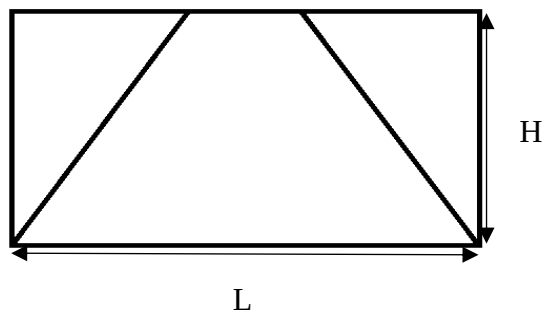


Figure 3.47. Failure lines for $L > 2H$ case.

Both failure modes are calculated using yield line theory as follows:

For $L > 2H$ case, using yield line theory:

External energy = Internal energy

$$w * h * l * \frac{\delta}{2} = M_{cr} * l * \frac{\delta}{h} + M_{cr} * h * \frac{\delta}{h} * 2 + M_{cr} * h * \sqrt{2} * \frac{\delta}{h} * \sqrt{2} * 2$$

$$w * l * h^2 * \frac{\delta}{2} = M_{cr} * \frac{\delta}{h} * (l + 6h)$$

$$w = M_{cr} * \left(\frac{l + 6h}{l * h^2} \right) * 2$$

Where M_{cr} is the crack moment, which is calculated by:

$$\frac{M_{cr} * y}{I} = \sigma_{tenscr}$$

$$\frac{M_{cr} * \frac{t}{2}}{1 * \frac{t^3}{12}} = \sigma_{tenscr}$$

$$M_{cr} = \sigma_{tenscr} * \frac{t^2}{6}$$

Thus, w for $L > 2H$ case is calculated as:

$$w = \sigma_{tenscr} * \frac{t^2}{6} * \left(\frac{l + 6h}{l * h^2} \right) * 2$$

For $L < 2H$ case, using yield line theory:

External energy = Internal energy

$$w * h * l * \frac{\delta}{2} = M_{cr} * 2h * \frac{\delta}{l} + M_{cr} * \left(h - \frac{l}{2} \right) * \frac{\delta}{l} * 2 + M_{cr} * l * \frac{\delta}{l} + M_{cr} * 2 * \frac{\sqrt{2}l}{2} * \frac{\delta}{l} * \sqrt{2}$$

$$w * l * h * \frac{\delta}{2} = M_{cr} * \frac{\delta}{l} * (4h + 2l)$$

$$w = 4 * M_{cr} * \left(\frac{4h + 2l}{h * l^2} \right)$$

Thus, w for $L < 2H$ case is calculated as:

$$w = 4 * \sigma_{tenscr} * \frac{t^2}{6} * \left(\frac{4h + 2l}{h * l^2} \right)$$

Furthermore, the minimum force required to initiate cracking at the wall can be calculated as follows:

$$F = w * l * h$$

This minimum force will be compared with earthquake force applied at each wall calculated based on TBDY 2018 [26] as follows:

$$V = \frac{W * S_{ae} * I}{R} = \frac{t * l * h * \rho * g * S_{ae} * I}{R}$$

S_{ae} is spectral acceleration and assuming that the structure is located at Hatay province which is one of the most earthquake-prone regions in Türkiye, elastic design spectrum can be obtained from AFAD website to determine variables (T_A , T_B , S_s , S_1 , PGA , PGV , S_{DS} , and S_{D1}) which are required to obtain spectral acceleration value. Furthermore, for a single-story masonry house the period of the structure is expected to be around 0.1 s and from the elastic design spectrum, this period corresponds to S_{DS} value. Thus, S_{ae} can be taken as S_{DS} value which is 1.134.

Moreover, density of clay brick is taken as 1700 kg/m^3 , importance factor I is taken as 1 for residential house, and R is taken as 1 for the most critical case. As an example, consider a wall at house 1 which has 2.5 m length, 3.6 m height, and 0.1 m thickness. In this wall the length is smaller than $2 * \text{height}$ ($L < 2H$) and the cracking stress is taken as 0.408 MPa [2]. The minimum force required to initiate cracking for $L < 2H$ case is calculated as:

$$w = 4 * \sigma_{tenscr} * \frac{t^2}{6} * \left(\frac{4h + 2l}{h * l^2} \right)$$

$$w = 4 * 408 * \frac{0.1^2}{6} * \left(\frac{4 * 3.6 + 2 * 2.5}{3.6 * 2.5^2} \right) = 2.345 \frac{kN}{m^2}$$

$$F = w * l * h = 2.345 * 2.5 * 3.6 = 21.105 \text{ kN}$$

Earthquake force applied is calculated as:

$$V = \frac{W * S_{ae} * I}{R} = \frac{t * l * h * \rho * g * S_{ae} * I}{R}$$

$$V = \frac{0.1 * 2.5 * 3.6 * 1700 * 9.81 * 1.134 * 1}{1} = 17.02 \text{ kN}$$

The wall is considered safe since the earthquake force applied is less than the minimum force required to initiate cracking. The same calculation is done for each wall at all houses and the results are tabulated below.

Failure mode 1 is for L<2H case and failure mode 2 is for L>2H case.

Table 16. House 1 yield line theory and EQ load calculation results.

Wall no	Length (m)	Height (m)	Failure Mode	w (kPa)	F (kN)	EQ Load TBDY (kN)	Check
1	4	3.6	1	1.058	15.232	27.233	FAILURE
2	3	3.6	1	1.713	18.496	20.425	FAILURE
3	4	3.6	1	1.058	15.232	27.233	FAILURE
4	1.75	3.6	1	4.416	27.822	11.914	OK
5	3	3.6	1	1.713	18.496	20.425	FAILURE
6	4	3.6	1	1.058	15.232	27.233	FAILURE
7	3	3.6	1	1.713	18.496	20.425	FAILURE
8	1.5	3.6	1	5.843	31.552	10.212	OK
9	3	3.6	1	1.713	18.496	20.425	FAILURE
10	1.5	3.6	1	5.843	31.552	10.212	OK
11	1.5	3.6	1	5.843	31.552	10.212	OK
12	3	3.6	1	1.713	18.496	20.425	FAILURE
13	3	3.6	1	1.713	18.496	20.425	FAILURE
14	3.5	3.6	1	1.320	16.631	23.829	FAILURE
15	6	3.6	1	0.554	11.968	40.849	FAILURE
16	6	3.6	1	0.554	11.968	40.849	FAILURE
17	3	3.6	1	1.713	18.496	20.425	FAILURE
18	3.5	3.6	1	1.320	16.631	23.829	FAILURE
19	1.5	3.6	1	5.843	31.552	10.212	OK
20	3	3.6	1	1.713	18.496	20.425	FAILURE
21	1.5	3.6	1	5.843	31.552	10.212	OK
22	1.5	3.6	1	5.843	31.552	10.212	OK
23	1.5	3.6	1	5.843	31.552	10.212	OK
24	4	3.6	1	1.058	15.232	27.233	FAILURE
25	4	3.6	1	1.058	15.232	27.233	FAILURE
26	3	3.6	1	1.713	18.496	20.425	FAILURE

Table 16 (continued)

27	3	3.6	1	1.713	18.496	20.425	FAILURE
28	3	3.6	1	1.713	18.496	20.425	FAILURE
29	1	3.6	1	12.391	44.608	6.808	OK
30	1	3.6	1	12.391	44.608	6.808	OK
31	1.5	3.6	1	5.843	31.552	10.212	OK
32	1.5	3.6	1	5.843	31.552	10.212	OK
33	0.5	3.6	1	46.542	83.776	3.404	OK
34	0.5	3.6	1	46.542	83.776	3.404	OK
35	2.5	3.6	1	2.345	21.107	17.021	OK
36	1.5	3.6	1	5.843	31.552	10.212	OK
37	1.5	3.6	1	5.843	31.552	10.212	OK

Table 17. House 2 yield line theory and EQ load calculation results.

Wall no	Length (m)	Height (m)	Failure Mode	w (kPa)	F (kN)	EQ Load TBDY (kN)	Check
1	4	3.15	1	1.112	14.008	23.829	FAILURE
2	1.5	3.15	1	5.987	28.288	8.936	OK
3	2.5	3.15	1	2.432	19.149	14.893	OK
4	2.5	3.15	1	2.432	19.149	14.893	OK
5	2.5	3.15	1	2.432	19.149	14.893	OK
6	2.5	3.15	1	2.432	19.149	14.893	OK
7	2.5	3.15	1	2.432	19.149	14.893	OK
8	2.5	3.15	1	2.432	19.149	14.893	OK
9	3	3.15	1	1.785	16.864	17.872	FAILURE
10	3	3.15	1	1.785	16.864	17.872	FAILURE
11	3	3.15	1	1.785	16.864	17.872	FAILURE
12	1.75	3.15	1	4.540	25.024	10.425	OK
13	3	3.15	1	1.785	16.864	17.872	FAILURE
14	1.75	3.15	1	4.540	25.024	10.425	OK
15	6	3.15	1	0.590	11.152	35.743	FAILURE
16	0.5	3.15	1	46.974	73.984	2.979	OK
17	1.5	3.15	1	5.987	28.288	8.936	OK

Table 18. House 3 yield line theory and EQ load calculation results.

Wall no	Length (m)	Height (m)	Failure Mode	w (kPa)	F (kN)	EQ Load TBDY (kN)	Check
1	3	3.6	1	1.713	18.496	20.425	FAILURE
2	1.5	3.6	1	5.843	31.552	10.212	OK
3	2.5	3.6	1	2.345	21.107	17.021	OK
4	2	3.6	1	3.476	25.024	13.616	OK
5	3	3.6	1	1.713	18.496	20.425	FAILURE
6	1.5	3.6	1	5.843	31.552	10.212	OK
7	2	3.6	1	3.476	25.024	13.616	OK
8	1	3.6	1	12.391	44.608	6.808	OK
9	4	3.6	1	1.058	15.232	27.233	FAILURE
10	2.5	3.6	1	2.345	21.107	17.021	OK
11	3.5	3.6	1	1.320	16.631	23.829	FAILURE
12	3	3.6	1	1.713	18.496	20.425	FAILURE
13	1	3.6	1	12.391	44.608	6.808	OK
14	2.5	3.6	1	2.345	21.107	17.021	OK
15	2	3.6	1	3.476	25.024	13.616	OK
16	1	3.6	1	12.391	44.608	6.808	OK
17	3	3.6	1	1.713	18.496	20.425	FAILURE
18	1.5	3.6	1	5.843	31.552	10.212	OK
19	2.5	3.6	1	2.345	21.107	17.021	OK
20	2	3.6	1	3.476	25.024	13.616	OK
21	3	3.6	1	1.713	18.496	20.425	FAILURE
22	3	3.6	1	1.713	18.496	20.425	FAILURE
23	3	3.6	1	1.713	18.496	20.425	FAILURE
24	3	3.6	1	1.713	18.496	20.425	FAILURE
25	1.5	3.6	1	5.843	31.552	10.212	OK
26	1.5	3.6	1	5.843	31.552	10.212	OK
27	1.5	3.6	1	5.843	31.552	10.212	OK
28	1.5	3.6	1	5.843	31.552	10.212	OK
29	1.5	3.6	1	5.843	31.552	10.212	OK
30	1.5	3.6	1	5.843	31.552	10.212	OK
31	1.5	3.6	1	5.843	31.552	10.212	OK
32	1.5	3.6	1	5.843	31.552	10.212	OK
33	3	3.6	1	1.713	18.496	20.425	FAILURE
34	3	3.6	1	1.713	18.496	20.425	FAILURE
35	3	3.6	1	1.713	18.496	20.425	FAILURE
36	3	3.6	1	1.713	18.496	20.425	FAILURE

Table 18 (continued)

37	3	3.6	1	1.713	18.496	20.425	FAILURE
38	3	3.6	1	1.713	18.496	20.425	FAILURE
39	3	3.6	1	1.713	18.496	20.425	FAILURE
40	3	3.6	1	1.713	18.496	20.425	FAILURE
41	1.5	3.6	1	5.843	31.552	10.212	OK
42	1.5	3.6	1	5.843	31.552	10.212	OK
43	1	3.6	1	12.391	44.608	6.808	OK

In summary, out of 37 walls in house 1, 20 of the walls failed under the given earthquake response. In house 2, 6 out of 17 walls failed while in house 3, 18 walls are considered failure out of 43 walls.



CHAPTER 4

STRENGTHENED CONFINED MASONRY HOUSE MODELLING

After all unstrengthened models have been modeled and the analysis results were obtained, it is deemed necessary for the non-engineered houses to be strengthened. In this thesis, several techniques for strengthening are evaluated, and these strengthened houses are modeled in SAP2000 and compared to the unstrengthened models. The following strengthening techniques are evaluated:

1. Addition of columns.
2. Addition of lintel.
3. Post-tensioning using rebars.

4.1 House 1 Strengthened Model

4.1.1 Addition of Columns

In the current walls' configuration, the available columns are insufficient to prevent flexural failures at some walls. Tie columns are mainly used to maintain the masonry wall's integrity, especially when an earthquake occurred. Initially, columns are added at the walls with insufficient flexural strength (σ_{11} or $\sigma_{22} > \sigma_{\text{flexural capacity}}$) and walls with no opening.

The new column configurations with added columns are shown in Figure 4.1.

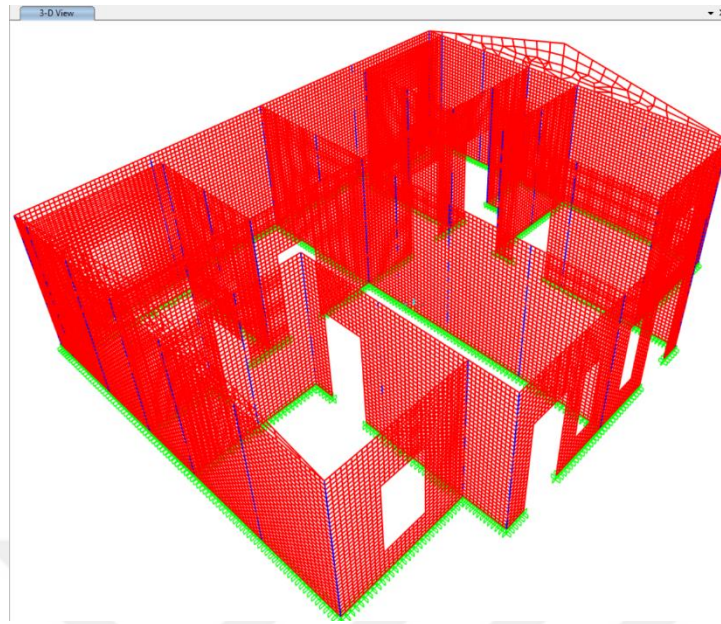


Figure 4.1. Addition of columns (blue) on masonry walls (red) for House 1.

The analysis result of the addition of columns is given in Table 19. From the analysis result, it is observed that wall 12, 14, and 18 passed the flexural strength check at both σ_{M11} and σ_{M22} . However, other walls which need to be strengthened are still insufficient. Thus, other proposed strengthening techniques are applied to the model.

Table 19. Additional Column Effect at House 1 Strengthened Model

Wall No	Length (m)	Height (m)	S12max (MPa)	σ_{M11} (MPa)	σ_{M22} (MPa)	Flexure Check (F22)	Flexure Check (F11)	Shear Check
1a	2	3.6	0.128	0.377	0.458	FAILURE	OK	OK
1b	2	3.6	0.118	0.304	0.476	FAILURE	OK	OK
3a	2	3.6	0.223	0.557	0.750	FAILURE	FAILURE	OK
3b	2	3.6	0.197	0.757	0.948	FAILURE	FAILURE	OK
6a	2	3.6	0.177	0.421	0.611	FAILURE	FAILURE	OK
6b	2	3.6	0.338	0.440	1.023	FAILURE	FAILURE	FAILURE
12a	1.5	3.6	0.137	0.391	0.406	OK	OK	OK
12b	1.5	3.6	0.135	0.404	0.403	OK	OK	OK
14a	1.5	3.6	0.092	0.350	0.369	OK	OK	OK
14b	2	3.6	0.076	0.346	0.380	OK	OK	OK
15a	3	3.6	0.187	0.408	0.671	FAILURE	OK	OK
15b	3	3.6	0.171	0.444	0.671	FAILURE	FAILURE	OK
16a	3	3.6	0.187	0.405	0.707	FAILURE	OK	OK
16b	3	3.6	0.177	0.444	0.707	FAILURE	FAILURE	OK
18a	1.5	3.6	0.119	0.357	0.394	OK	OK	OK
18b	1.5	3.6	0.127	0.346	0.397	OK	OK	OK
24a	2	3.6	0.245	0.733	0.545	FAILURE	FAILURE	OK
24b	2	3.6	0.199	0.377	0.645	FAILURE	OK	OK
25a	2	3.6	0.187	0.403	0.674	FAILURE	OK	OK
25b	2	3.6	0.178	0.397	0.674	FAILURE	OK	OK
35a	1.5	3.6	0.236	0.3972	0.7926	FAILURE	OK	OK
35b	1	3.6	0.253	0.4746	0.2562	OK	FAILURE	OK

Moreover, the location of existing columns and additional columns are given in Figure 4.2 where red columns are existing columns and blue columns are additional columns.

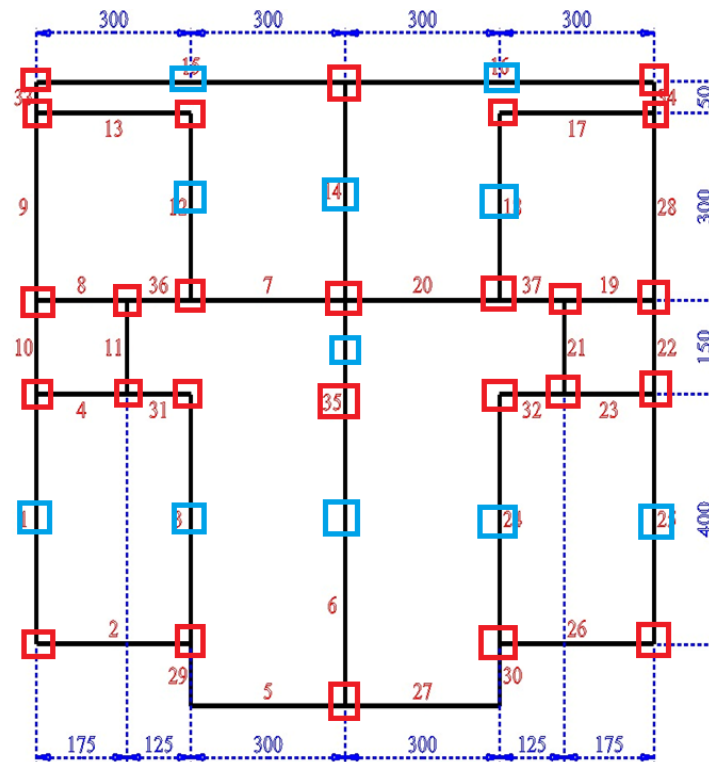


Figure 4.2. Location of existing columns (red) and additional columns (blue) at house 1.

4.1.2 Addition of Lintel

A lintel is an additional beam separating a masonry wall into two different walls while preventing the deflection of a window or door frame when placed on top of the frames. In addition to that, the lintel should be connected between 2 columns and should not end at the middle of the walls so that the load can be transferred to the foundation from the columns.

In the strengthened model, the mid-height lintels are added at the outer parts and inner parts of the structure. The strengthened model is shown in Figure 4.3.

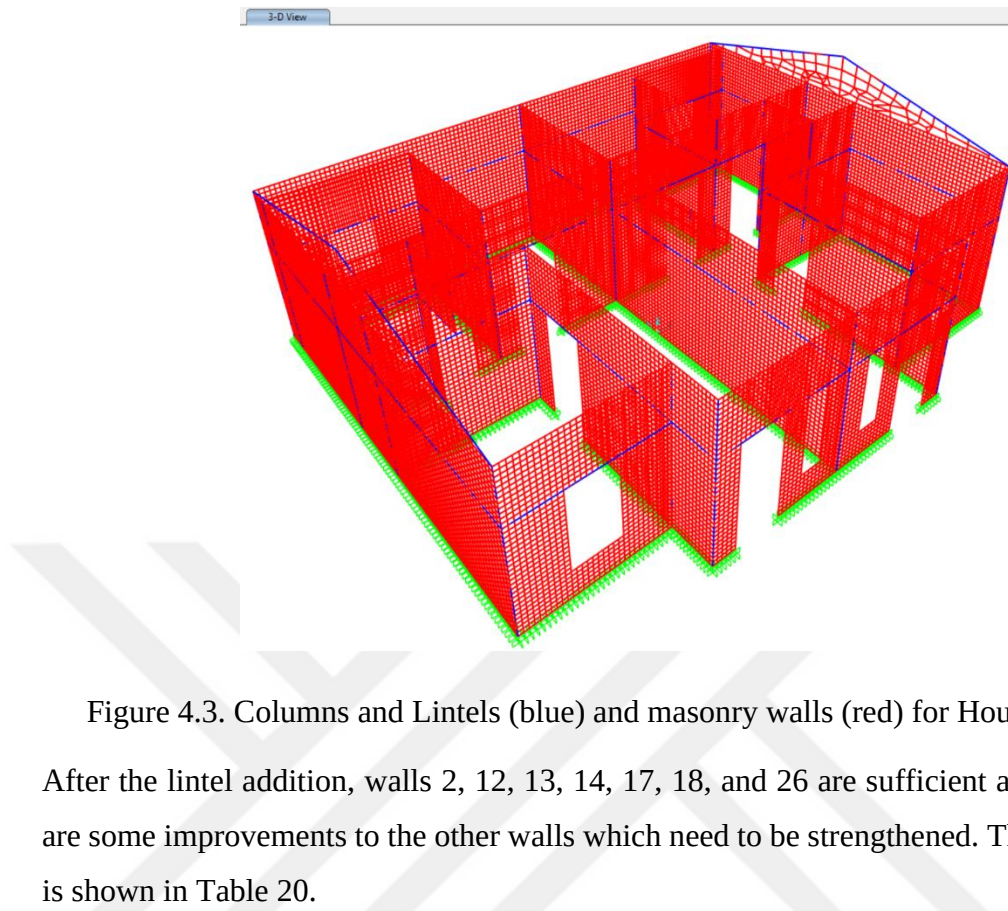


Figure 4.3. Columns and Lintels (blue) and masonry walls (red) for House 1.

After the lintel addition, walls 2, 12, 13, 14, 17, 18, and 26 are sufficient and there are some improvements to the other walls which need to be strengthened. The result is shown in Table 20.

Table 20. Additional lintel effect at House 1 Strengthened Model.

Wall No	Length (M)	Height (M)	S12max (MPa)	σ_{M11} (MPa)	σ_{M22} (MPa)	Flexure Check (F22)	Flexure Check (F11)	Shear Check
1a	4	2.4	0.186	0.3246	0.63	FAILURE	OK	OK
1b	4	1.2	0.151	0.3258	0.3606	OK	OK	OK
2a	3	2.4	0.189	0.405	0.3396	OK	OK	OK
2b	3	1.2	0.133	0.3336	0.2682	OK	OK	OK
3a	4	2.4	0.377	0.5922	1.1916	FAILURE	FAILURE	FAILURE
3b	4	1.2	0.232	0.6558	0.6894	FAILURE	FAILURE	OK
5a	3	2.4	0.391	0.3354	0.792	FAILURE	OK	FAILURE
5b	3	1.2	0.164	0.3114	0.3072	OK	OK	OK
6a	4	2.4	0.295	0.4926	0.95616	FAILURE	FAILURE	OK
6b	4	1.2	0.238	0.4746	0.8616	FAILURE	FAILURE	OK
7a	3	2.4	0.193	0.3138	0.5232	FAILURE	OK	OK
7b	3	1.2	0.154	0.2742	0.2754	OK	OK	OK
12a	3	2.4	0.125	0.399	0.3672	OK	OK	OK

Table 20 (continued)

12b	3	1.2	0.141	0.3882	0.2928	OK	OK	OK
13a	3	2.4	0.112	0.3306	0.3408	OK	OK	OK
13b	3	1.2	0.144	0.2754	0.348	OK	OK	OK
14a	3.5	2.4	0.116	0.30672	0.4038	OK	OK	OK
14b	3.5	1.2	0.112	0.2988	0.2826	OK	OK	OK
15a	6	2.4	0.26	0.55332	1.0506	FAILURE	FAILURE	OK
15b	6	1.2	0.192	0.56676	0.8526	FAILURE	FAILURE	OK
16a	6	2.4	0.252	0.55038	0.9138	FAILURE	FAILURE	OK
16b	6	1.2	0.192	0.56364	0.5928	FAILURE	FAILURE	OK
17a	3	2.4	0.125	0.3162	0.3366	OK	OK	OK
17b	3	1.2	0.117	0.2406	0.354	OK	OK	OK
18a	3	2.4	0.132	0.3954	0.387	OK	OK	OK
18b	3	1.2	0.134	0.3786	0.2934	OK	OK	OK
20a	3	2.4	0.178	0.3036	0.4698	FAILURE	OK	OK
20b	3	1.2	0.154	0.2838	0.348	OK	OK	OK
24a	4	2.4	0.291	0.3414	1.0068	FAILURE	OK	OK
24b	4	1.2	0.154	0.6414	0.5448	FAILURE	FAILURE	OK
25a	4	2.4	0.221	0.387	0.6102	FAILURE	OK	OK
25b	4	1.2	0.158	0.3858	0.399	OK	OK	OK
26a	3	2.4	0.163	0.357	0.342	OK	OK	OK
26b	3	1.2	0.134	0.3408	0.258	OK	OK	OK
27a	3	2.4	0.334	0.2544	0.501	FAILURE	OK	FAILURE
27b	3	1.2	0.163	0.3288	0.3186	OK	OK	OK
35a	2.5	2.4	0.22	0.3984	0.7836	FAILURE	OK	OK
35b	2.5	1.2	0.173	0.46362	0.54372	FAILURE	FAILURE	OK

4.1.3 Addition of Lintel + Column

A combination of lintels and columns are also applied to evaluate the flexural and shear strength improvement of the house 1. The strengthened model is shown in Figure 4.4.

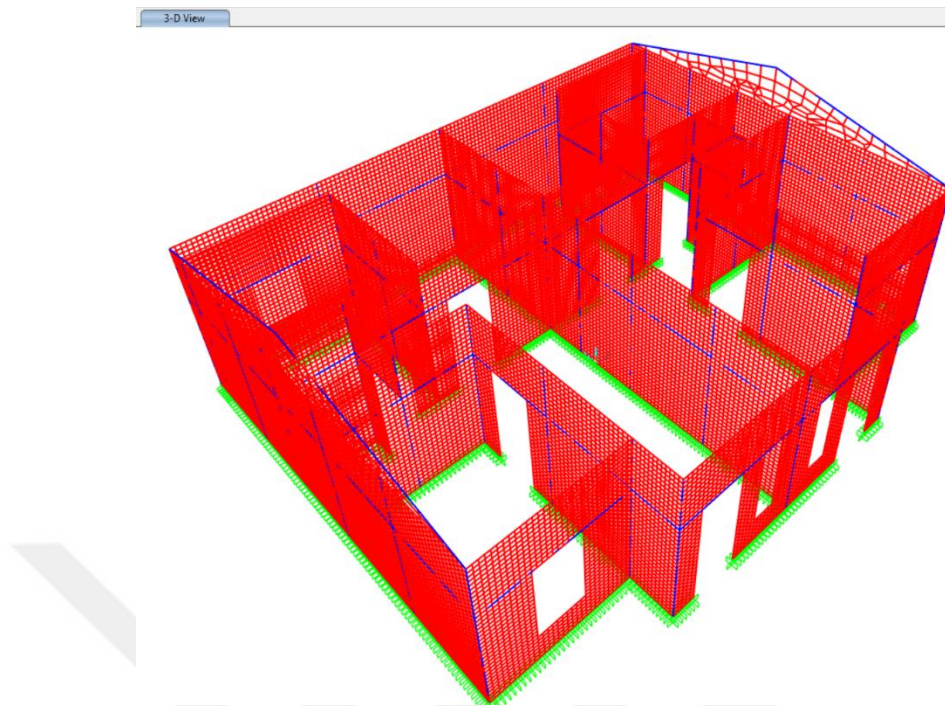


Figure 4.4. Columns and Lintels (blue) and masonry walls (red) for House 1.

After lintel and column additions, walls 2, 7, 12, 13, 14, 17, 18, 20, and 26 are considered sufficient. The detailed result is shown in Table 21.

Table 21. Column and Lintel Additions effect at House 1 Strengthened Model.

Wall no	Length (m)	Height (m)	S12max (MPa)	σ_{M11} (MPa)	σ_{M22} (MPa)	Flexure Check (F22)	Flexure check (F11)	Shear check
1a	2	2.4	0.146	0.2706	0.4896	FAILURE	OK	OK
1b	2	2.4	0.135	0.2814	0.4938	FAILURE	OK	OK
1c	2	1.2	0.142	0.3276	0.3024	OK	OK	OK
1d	2	1.2	0.107	0.2856	0.2952	OK	OK	OK
2a	3	2.4	0.189	0.3378	0.3774	OK	OK	OK
2b	3	1.2	0.133	0.3336	0.3048	OK	OK	OK
3a	2	2.4	0.157	0.402	0.5598	FAILURE	OK	OK
3b	2	2.4	0.233	0.3999	0.729	FAILURE	OK	OK
3c	2	1.2	0.143	0.387	0.5286	FAILURE	OK	OK
3d	2	1.2	0.136	0.39342	0.44466	FAILURE	OK	OK
5a	3	2.4	0.283	0.3354	0.792	FAILURE	OK	OK
5b	3	1.2	0.164	0.3114	0.3072	OK	OK	OK
6a	2	2.4	0.131	0.2946	0.55692	FAILURE	OK	OK

Table 21 (continued)

6b	2	2.4	0.179	0.29838	0.74928	FAILURE	OK	OK
6c	2	1.2	0.147	0.29652	0.42984	FAILURE	OK	OK
6d	2	1.2	0.208	0.3978	0.6864	FAILURE	OK	OK
7a	3	2.4	0.155	0.3138	0.4056	OK	OK	OK
7b	3	1.2	0.154	0.3096	0.2754	OK	OK	OK
12a	1.5	2.4	0.15	0.366	0.3918	OK	OK	OK
12b	1.5	2.4	0.139	0.3588	0.3636	OK	OK	OK
12c	1.5	1.2	0.16	0.3852	0.3138	OK	OK	OK
12d	1.5	1.2	0.147	0.3672	0.3186	OK	OK	OK
13a	3	2.4	0.113	0.3306	0.3408	OK	OK	OK
13b	3	1.2	0.136	0.2754	0.348	OK	OK	OK
14a	1.5	2.4	0.102	0.32898	0.25764	OK	OK	OK
14b	2	2.4	0.101	0.30636	0.27036	OK	OK	OK
14c	1.5	1.2	0.112	0.33222	0.2226	OK	OK	OK
14d	2	1.2	0.095	0.30774	0.25152	OK	OK	OK
15a	3	2.4	0.193	0.399	0.69858	FAILURE	OK	OK
15b	3	2.4	0.178	0.3954	0.68226	FAILURE	OK	OK
15c	3	1.2	0.157	0.363	0.4518	FAILURE	OK	OK
15d	3	1.2	0.128	0.384	0.4344	FAILURE	OK	OK
16a	3	2.4	0.181	0.4062	0.68286	FAILURE	OK	OK
16b	3	2.4	0.187	0.3786	0.69918	FAILURE	OK	OK
16c	3	1.2	0.16	0.3624	0.4476	FAILURE	OK	OK
16d	3	1.2	0.165	0.3894	0.4698	FAILURE	OK	OK
17a	3	2.4	0.125	0.3162	0.3366	OK	OK	OK
17b	3	1.2	0.11	0.2406	0.354	OK	OK	OK
18a	1.5	2.4	0.15	0.366	0.3918	OK	OK	OK
18b	1.5	2.4	0.139	0.3588	0.3636	OK	OK	OK
18c	1.5	1.2	0.159	0.3852	0.3138	OK	OK	OK
18d	1.5	1.2	0.147	0.3672	0.3186	OK	OK	OK
20a	3	2.4	0.141	0.3036	0.4074	OK	OK	OK
20b	3	1.2	0.154	0.3096	0.282	OK	OK	OK
24a	2	2.4	0.212	0.3951	0.5022	FAILURE	OK	OK
24b	2	2.4	0.273	0.26796	0.6726	FAILURE	OK	OK
24c	2	1.2	0.165	0.31494	0.3102	OK	OK	OK
24d	2	1.2	0.164	0.31374	0.29802	OK	OK	OK
25a	2	2.4	0.138	0.3066	0.4722	FAILURE	OK	OK
25b	2	2.4	0.136	0.3006	0.4974	FAILURE	OK	OK
25c	2	1.2	0.136	0.315	0.3162	OK	OK	OK
25d	2	1.2	0.109	0.2982	0.3144	OK	OK	OK

Table 21 (continued)

26a	3	2.4	0.163	0.357	0.342	OK	OK	OK
26b	3	1.2	0.134	0.2322	0.258	OK	OK	OK
27a	3	2.4	0.292	0.28614	0.8226	FAILURE	OK	OK
27b	3	1.2	0.163	0.29166	0.3186	OK	OK	OK
35a	1.5	2.4	0.282	0.399	0.7938	FAILURE	OK	OK
35b	1.5	1.2	0.198	0.393	0.3054	OK	OK	OK
35c	1	2.4	0.19	0.405	0.609	FAILURE	OK	OK
35d	1	1.2	0.18	0.3822	0.1872	OK	OK	OK

4.1.4 Rebars Addition for Post-Tensioning

Post-tensioning is a technique in which additional compression in the vertical plane is introduced using rebars placed vertically on each side of the wall at certain spacing. This technique will increase the resistance of the same masonry to bending actions as compression can abolish tensile stresses due to bending. Moreover, post-tensioning reduces principal tensile stresses and improves the shear resistance of masonry walls.

An experiment was done to test the effectiveness of applying post-tensioning to masonry walls [9], and it was proven that vertical in-plane post-tensioning of masonry walls would enhance the initial cracking strength of walls in the linear range and postpone the formation of tension cracks at the base. Moreover, post-tensioning is an economical and easy way to apply vertical forces. However, slight wall crushing can result in an abrupt loss of vertical post-tensioning effect and premature failure when the walls reach their cracking capacity.

Experimental and analytical studies on walls have shown that walls with a low vertical load (which is the case for this non-engineered house), and high aspect ratio (H/B) have tendency to fail due to:

- a. Corner Brick Crushing
- b. Diagonal Cracking (Split + Shear generated shear stresses)

Theoretically, the effects of post-tensioning on masonry walls can be explained in Figure 4.5:

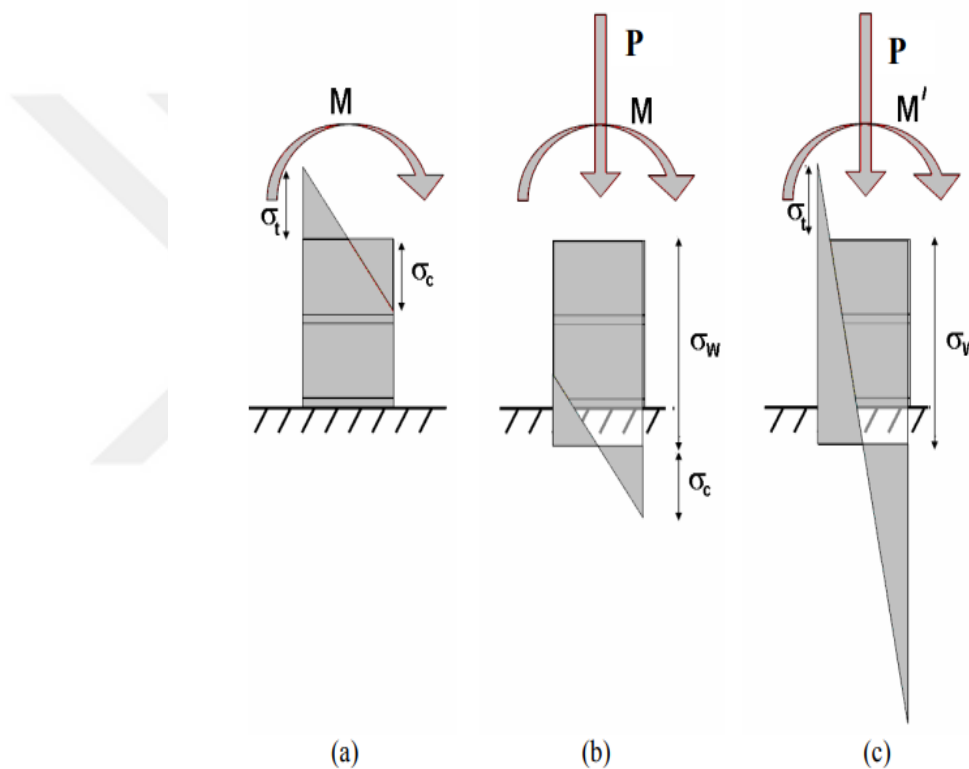


Figure 4.5. Bending capacity increases due to post-tensioning [9].

From Figure 4.5, σ_t shows the initial tensile strength capacity, and the compression stress due to bending is σ_c . When post-tensioning force is applied on the wall, the compression stress increases to σ_w , and tension stress is reduced by σ_w . As a result, the tensile capacity of the wall under bending moment in an out-of-plane direction increases to $\sigma_t + \sigma_w$. On the other hand, it is important to note that the combination

of compressive stress applied by post-tensioning (σ_w) and bending related compressive stress (σ_c) cannot exceed the total compressive strength of a masonry wall material.

At the masonry house, Since the column and beam size are 15 x 15 cm and the masonry brick thickness is 10 cm, there will be 5 cm space between the outer column and beam with the bricks (2.5 cm at the inner and 2.5 cm at the exterior). These spaces will be used to apply post-tensioning bars at both sides of the bricks and determination of rebar diameter and spacing will be done in the following section. Moreover, U-Channel will be used at the top of the beams where post-tensioning is applied to increase the stiffness of the masonry wall and the sketch of the intended post-tensioning technique is shown in Figure 4.6 and Figure 4.7.

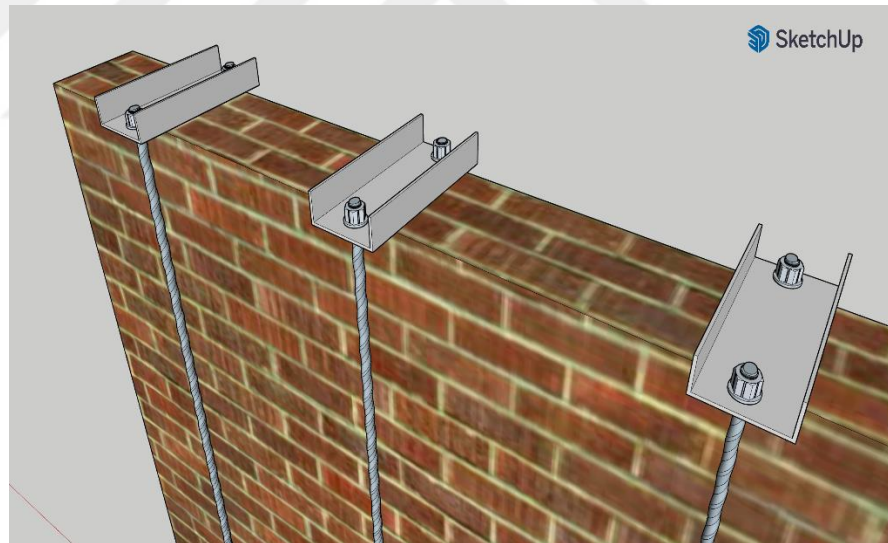


Figure 4.6. Post tensioning technique using U-Channel and threaded rebars (1).

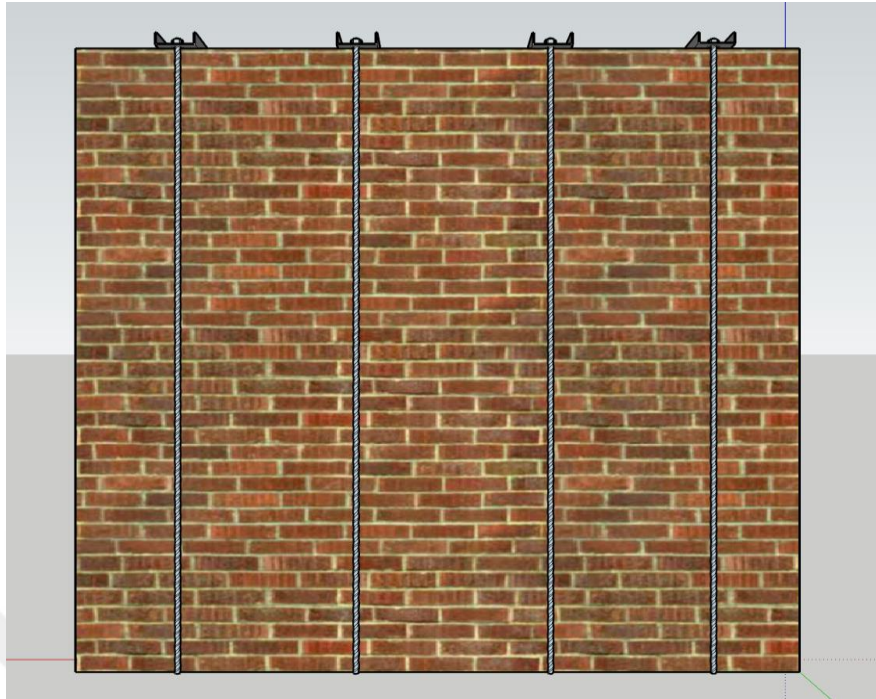


Figure 4.7. Post tensioning technique using U-Channel and threaded rebars (2).

Additionally, after spacing and rebar diameter have been determined at the location where rebars will be placed, there should be 15 cm length with 10mm diameter holes at the beam, which can be created by putting PVC circular hollow pipe before pouring the concrete. Furthermore, nuts can be used to apply the post-tensioning after the concrete has hardened and rebars have been placed.

Regarding the out-of-plane and in-plane effect of post-tensioning on masonry walls, the applied post-tensioning loads alter the normal stress values of point A and point B towards A^1 and B^1 , generating compression fields that reduce tensile stresses. As shear stresses of masonry walls are increased, points A^1 and B^1 are shifted towards A^2 and B^2 until the tensile stress capacity of materials reaches the

principal direction at the Mohr circle. The shift in the Mohr-circle is shown in Figure 4.8

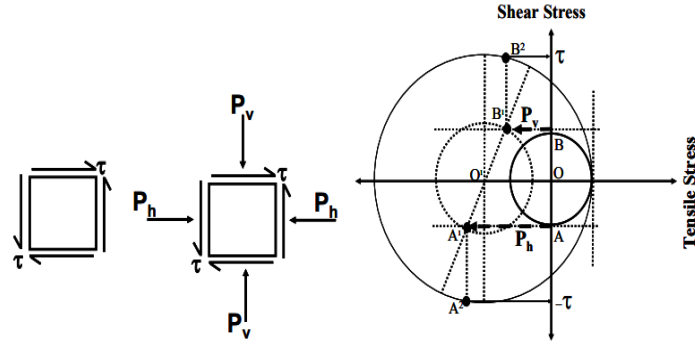


Figure 4.8. Post Tensioning effect on Mohr Circle representation [8].

To determine the amount of post-tensioning required for the walls that need to be post-tensioned and the diameter of rebars that will be used for post-tensioning. The following formula is used:

$$\frac{M * y}{I} - \sigma_{post-tensioning} = \sigma_{tensile\ capacity}$$

The tensile capacity is obtained from the flexural strength test [2] and it is taken as 0.408 MPa.

For example, after adding a lintel and column in the middle, wall number 1 is separated into 4 different walls: wall 1a, wall 1b, wall 1c and wall 1d. At wall 1a, the bending moment M22 is calculated as 1.632 kNm. The required post-tensioning load and diameter of rebar that will be used are calculated as follows:

$$\begin{aligned} \frac{M * y}{I} - \sigma_{post-tensioning} &= \sigma_{tensile\ capacity} \\ \frac{1.632 * 0.05 * 0.001}{\frac{2 * 0.1^3}{12}} - \sigma_{post-tensioning} &= 0.408 \\ \sigma_{post-tensioning} &= 0.0816\ MPa \end{aligned}$$

Consider 0.5 x 0.1 m surface area, the amount of post-tensioning load required for each bar is calculated as:

$$81.6 \text{ kPa} * 0.5 \text{ m} * 0.1 \text{ m} = 4.08 \text{ kN}$$

Since there are 2 bars on each side at every 0.5 m spacing, the required post-tensioning load at each bar is calculated as:

$$\frac{4.08}{2} \text{ kN} = 2.04 \text{ kN/bar}$$

According to SNI 2052:2017 [28] the allowable tensile strength of rebar is 350 MPa. Thus, the rebar diameter required is calculated as:

$$\begin{aligned} \sigma_{allow \text{ tensile}} &= \frac{P_{post-tensioning}}{A} \\ 350 \text{ MPa} &= \frac{2040 \text{ N}}{A} \\ A &= 5.82 \text{ mm}^2 \end{aligned}$$

Using 6 mm diameter rebar ($A_s = 28.27 \text{ mm}^2$) is sufficient.

The above calculations are applied to other walls that must be post-tensioned and the result is given in Table 22.

Table 22. Post Tensioning Required for House 1 Strengthened Model

Wall No	σ_{M22} (MPa)	Flexure Check (σ_{M22})	Tensile Strength Required (MPa)	Force/Bar (KN/Bar)	Area Required (mm ²)	Rebar Diameter Used/0.5 m
1a	0.4896	FAILURE	0.082	2.040	5.829	6
1b	0.4938	FAILURE	0.086	2.145	6.129	6
1c	0.3024	OK				
1d	0.2952	OK				
3a	0.5598	FAILURE	0.152	3.795	10.843	6
3b	0.729	FAILURE	0.321	8.025	22.929	6
3c	0.5286	FAILURE	0.121	3.015	8.614	6
3d	0.44466	FAILURE	0.037	0.916	2.619	6
5a	0.792	FAILURE	0.384	9.600	27.429	6
5b	0.3072	OK				
6a	0.55692	FAILURE	0.149	3.723	10.637	6
6b	0.74928	FAILURE	0.341	8.532	24.377	6
6c	0.42984	FAILURE	0.022	0.546	1.560	6
6d	0.6864	FAILURE	0.278	6.960	19.886	6
15a	0.69858	FAILURE	0.291	7.264	20.756	6
15b	0.68226	FAILURE	0.274	6.857	19.590	6
15c	0.4518	FAILURE	0.044	1.095	3.129	6
15d	0.4344	FAILURE	0.026	0.660	1.886	6
16a	0.68286	FAILURE	0.275	6.872	19.633	6
16b	0.69918	FAILURE	0.291	7.280	20.799	6
16c	0.4476	FAILURE	0.040	0.990	2.829	6
16d	0.4698	FAILURE	0.062	1.545	4.414	6
24a	0.5022	FAILURE	0.094	2.355	6.729	6
24b	0.6726	FAILURE	0.265	6.615	18.900	6
24c	0.3102	OK				
24d	0.29802	OK				
25a	0.4722	FAILURE	0.064	1.605	4.586	6
25b	0.4974	FAILURE	0.089	2.235	6.386	6
25c	0.3162	OK				
25d	0.3144	OK				
27a	0.8226	FAILURE	0.415	10.365	29.614	8
27b	0.3186	OK				
35a	0.7938	FAILURE	0.415	10.365	29.614	6
35b	0.3054	OK				
35c	0.609	FAILURE	0.201	5.025	14.357	6

4.2 House 2 Strengthened Model

4.2.1 Addition of Columns

Initially, columns are added at the walls where flexural strength are insufficient (σ_{11} or $\sigma_{22} > \sigma_{\text{flexural capacity}}$) and walls with no opening.

The new column configurations with added columns are shown in Figure 4.9.

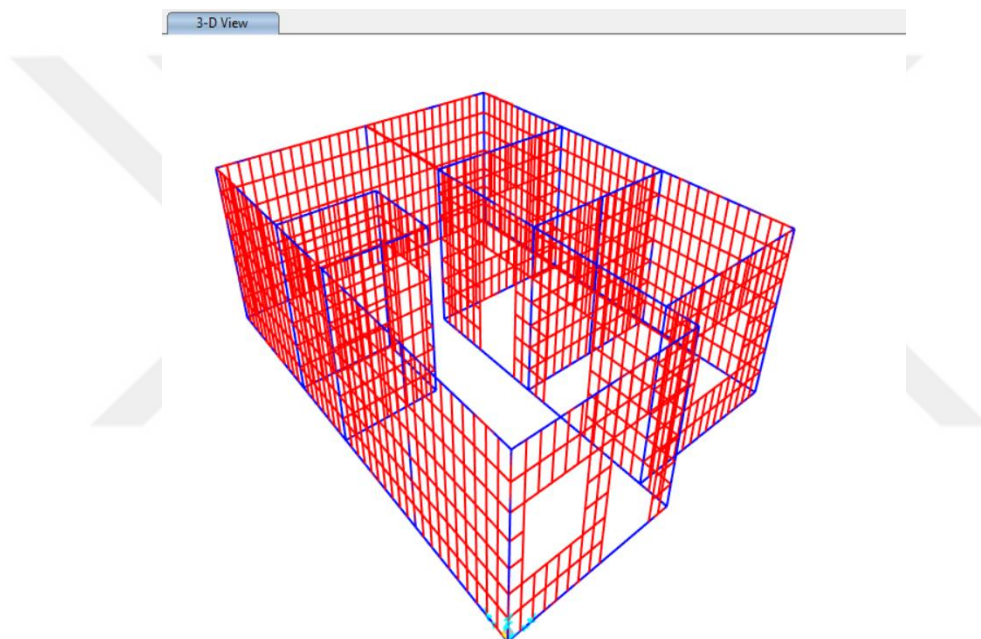


Figure 4.9. Addition of columns (blue) on masonry walls (red) for House 2.

The analysis result of the addition of columns is given in Table 23. The result shows that wall 11 is sufficient and there are improvements at walls 1 and 15.

Table 23. Additional Column Effect at House 2 Strengthened Model

Wall No	Length (m)	Height (m)	S12max (MPa)	σ_{M11} (MPa)	σ_{M22} (MPa)	Flexure Check (F22)	Flexure Check (F11)	Shear Check
1a	2	3.15	0.230	0.572	0.836	FAILURE	FAILURE	OK
1b	2	3.15	0.196	0.777	0.836	FAILURE	FAILURE	OK
11a	1.5	3.15	0.109	0.238	0.384	OK	OK	OK
11b	1.5	3.15	0.103	0.259	0.376	OK	OK	OK
15a	3	3.15	0.173	0.326	0.629	FAILURE	OK	OK
15b	3	3.15	0.177	0.365	0.664	FAILURE	OK	OK

4.2.2 Addition of Lintel

In the strengthened model, mid-height lintels are added to the walls' outer and inner parts. The strengthened model is shown in Figure 4.10.

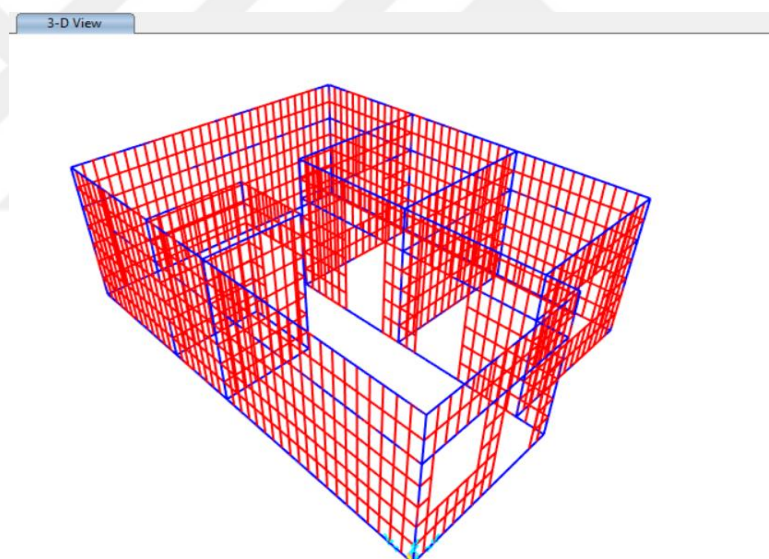


Figure 4.10. Addition of lintels (blue) on masonry walls (red) at House 2.

After the lintel addition, walls 4 and 13 are sufficient and there are improvements on walls 1, 5, and 9. The result is shown in Table 24.

Table 24. Additional lintel effect at House 2 Strengthened Model.

Wall No	Length (m)	Height (m)	S12max (MPa)	σ_{M11} (MPa)	σ_{M22} (MPa)	Flexure Check (F22)	Flexure Check (F11)	Shear Check
1a	4	2.15	0.258	0.744	1.053	FAILURE	FAILURE	OK
1b	4	0.9	0.218	0.676	0.688	FAILURE	FAILURE	OK
4a	3	2.15	0.181	0.191	0.346	OK	OK	OK
4b	3	0.9	0.142	0.287	0.136	OK	OK	OK
5a	4	2.15	0.199	0.322	0.431	FAILURE	OK	OK
5b	4	0.9	0.147	0.325	0.139	OK	OK	OK
9a	3	2.15	0.258	0.400	0.729	FAILURE	OK	OK
9b	3	0.9	0.207	0.407	0.445	FAILURE	OK	OK
11a	4	2.15	0.121	0.399	0.464	FAILURE	OK	OK
11b	4	0.9	0.118	0.408	0.231	OK	OK	OK
13a	3	2.15	0.194	0.362	0.347	OK	OK	OK
13b	3	0.9	0.132	0.380	0.155	OK	OK	OK
15a	3	2.15	0.309	0.366	0.929	FAILURE	OK	OK
15b	3	0.9	0.443	0.520	0.888	FAILURE	OK	OK

4.2.3 Addition of Lintel + Column

A combination of lintel and column are also applied to observe the flexural and shear strength improvement for house 2. The strengthened model is shown in Figure 4.11.

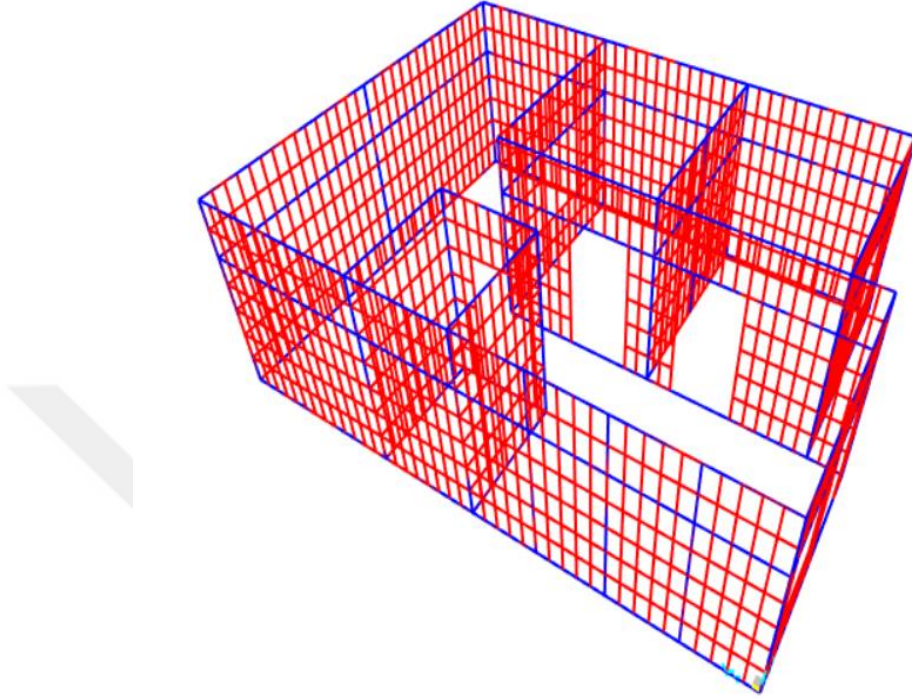


Figure 4.11. Addition of columns and lintels (blue) on masonry walls (red) at house 2

Initially, the column spacing at wall 1 was 2 m. Even after the addition of both a column and lintel, the wall still failed due to σ_{M11} . To improve it, an additional column with 1 m spacing is used and the result is tabulated in Table 25.

Table 25. Column and Lintel Additions effect at House 2 Strengthened Model.

Wall No	Length (m)	Height (m)	S12max (MPa)	σ_{M11} (MPa)	σ_{M22} (MPa)	Flexure Check (F22)	Flexure Check (F11)	Shear Check
1a	1	2.15	0.166	0.380	0.507	FAILURE	FAILURE	OK
1b	1	2.15	0.161	0.386	0.706	FAILURE	FAILURE	OK
1c	1	2.15	0.205	0.385	0.706	FAILURE	FAILURE	OK
1d	1	2.15	0.178	0.391	0.447	FAILURE	FAILURE	OK
1e	1	0.9	0.158	0.245	0.365	FAILURE	OK	OK
1f	1	0.9	0.058	0.387	0.410	OK	OK	OK
1g	1	0.9	0.09	0.401	0.4215	OK	OK	OK
1h	1	0.9	0.139	0.238	0.25908	OK	OK	OK
5a	4	2.15	0.214	0.231	0.490	FAILURE	OK	OK
5b	4	0.9	0.126	0.310	0.143	OK	OK	OK
9a	3	2.15	0.278	0.378	0.563	OK	OK	OK
9b	3	0.9	0.186	0.356	0.402	OK	OK	OK
15a	3	2.15	0.210	0.352	0.766	FAILURE	OK	OK
15b	3	2.15	0.212	0.269	0.803	FAILURE	OK	OK
15c	3	0.9	0.244	0.277	0.604	FAILURE	OK	OK
15d	3	0.9	0.239	0.297	0.609	FAILURE	OK	OK

Moreover, the location of existing columns and additional columns are given in Figure 4.12 where red columns are existing columns and blue columns are additional columns.

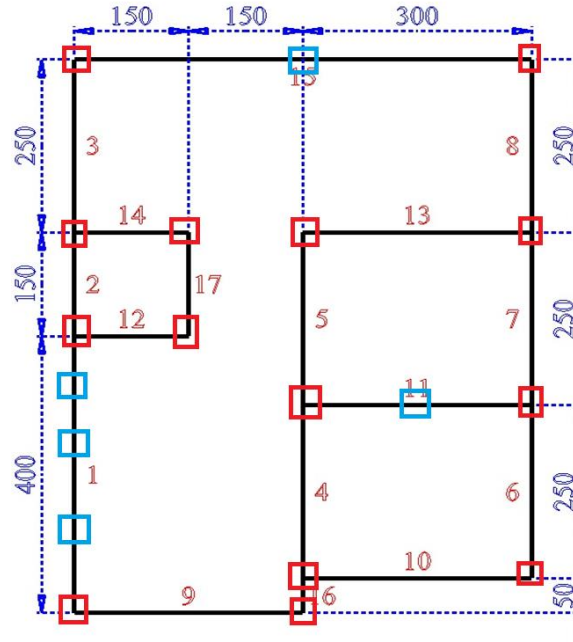


Figure 4.12. Location of existing columns (red) and additional columns (blue) at house 2.

4.2.4 Rebars Addition for Post-Tensioning

To determine the amount of post-tensioning required for the walls that need to be post-tensioned and the diameter of rebars that will be used for post-tensioning. The following formula is used:

$$\frac{M * y}{I} - \sigma_{post-tensioning} = \sigma_{tensile\ capacity}$$

The tensile capacity is obtained from the flexural strength test [2] and it is taken as 0.408 MPa.

For example, after adding lintel and column at the middle, wall number 15 is separated into 4 different walls: wall 15a, wall 15b, wall

15c, and wall 15d. At wall 15a, the bending moment M22 is calculated as 3.8286 kNm. The required post-tensioning load and diameter of rebar that will be used are calculated as follows:

$$\frac{M * y}{I} - \sigma_{post-tensioning} = \sigma_{tensile\ capacity}$$

$$\frac{3.8286 * 0.05 * 0.001}{\frac{3 * 0.1^3}{12}} - \sigma_{post-tensioning} = 0.408$$

$$\sigma_{post-tensioning} = 0.3577\ MPa$$

Consider 0.5 x 0.1 m surface area, the amount of post-tensioning load required for each bar is calculated as:

$$357.7\ kPa * 0.5\ m * 0.1\ m = 17.885\ kN$$

Since there are 2 bars at each side at every 0.5 m spacing, the required post-tensioning load at each bar is calculated as:

$$\frac{17.885}{2}\ kN = 8.943\ kN/bar$$

According to SNI 2052:2017 [28], the allowable tensile strength of the rebar is 350 MPa. Thus, the rebar diameter required is calculated as:

$$\sigma_{allow\ tensile} = \frac{P_{post-tensioning}}{A}$$

$$350\ MPa = \frac{8943\ N}{A}$$

$$A = 25.551\ mm^2$$

Using 6 mm diameter rebar ($A_s = 28.274\ mm^2$) is sufficient.

The above calculations are applied to other walls that must be post-tensioned. The result is given in Table 26.

Table 26. Post Tensioning Required for House 2 Strengthened Model

Wall no	F22 (MPa)	FLEXURE CHECK (F22)	Tensile strength required (MPa)	Force/bar (kN/bar)	Area required (mm ²)	Rebar diameter used/0.5 m
1a	0.507	FAILURE	0.099	2.465	7.041	6
1b	0.706	FAILURE	0.298	7.440	21.257	6
1c	0.706	FAILURE	0.298	7.440	21.257	6
1d	0.447	FAILURE	0.039	0.982	2.807	6
1e	0.365	OK				
1f	0.410	FAILURE				
1g	0.422	FAILURE	0.014	0.337	0.964	6
1h	0.259	OK				
5a	0.490	FAILURE	0.082	2.057	5.876	6
5b	0.143	OK				
9a	0.563	FAILURE	0.155	3.875	11.070	6
9b	0.402	OK				
15a	0.766	FAILURE	0.358	8.943	25.551	6
15b	0.803	FAILURE	0.395	9.879	28.226	8
15c	0.604	FAILURE	0.196	4.896	13.989	6
15d	0.609	FAILURE	0.201	5.015	14.327	6

4.3 House 3 Strengthened Model

4.3.1 Addition of Columns

Initially, columns are added at the walls where flexural strength are insufficient (σ_{11} or $\sigma_{22} > \sigma_{\text{flexural capacity}}$) and walls with no opening.

The new column configurations with added columns are shown in Figure 4.13.

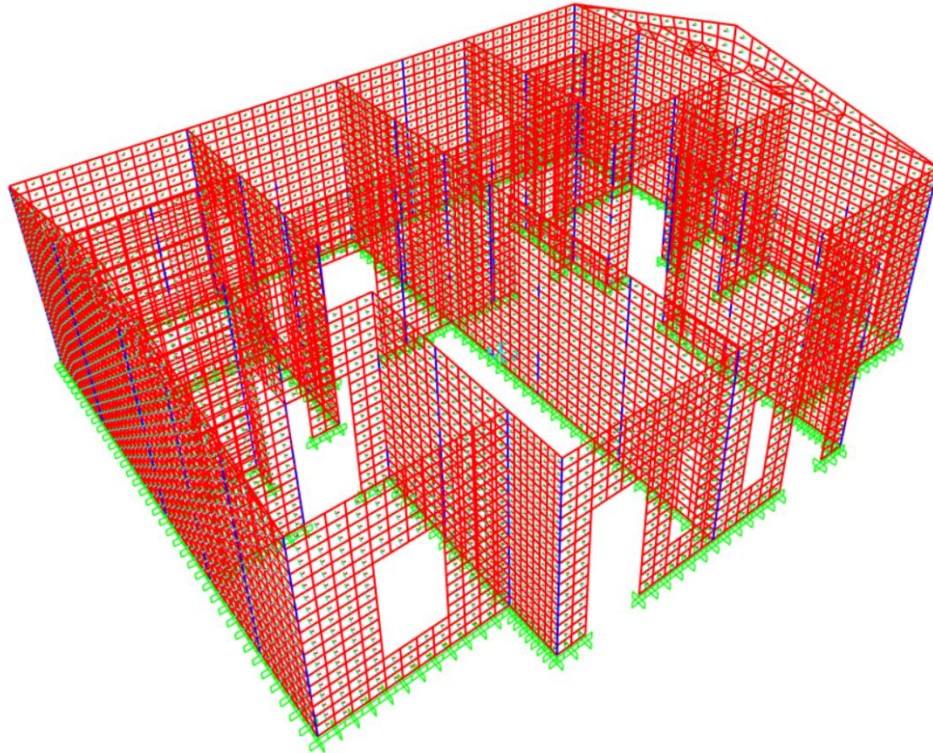


Figure 4.13. Addition of columns (blue) on masonry walls (red) at house 3.

The analysis result of the addition of columns is given in Table 27. The result shows that walls 1, 17, 37, and 40 are sufficient and there are improvements on other walls which need to be strengthened.

Table 27. Additional column effect at House 3 Strengthened Model.

Wall No	Length (m)	Height (m)	S12max (MPa)	σ_{M11} (MPa)	σ_{M22} (MPa)	Flexure Check (F22)	Flexure Check (F11)	Shear Check
1a	1.5	3.6	0.126	0.322	0.386	OK	OK	OK
1b	1.5	3.6	0.110	0.323	0.368	OK	OK	OK
5a	1.5	3.6	0.167	0.649	0.560	FAILURE	FAILURE	OK
5b	1.5	3.6	0.150	0.494	0.560	FAILURE	FAILURE	OK
9a	2	3.6	0.263	0.488	0.858	FAILURE	FAILURE	OK
9b	2	3.6	0.304	0.452	0.932	FAILURE	FAILURE	FAILURE
10a	1.5	3.6	0.317	0.441	0.824	FAILURE	FAILURE	FAILURE
10b	1	3.6	0.225	0.769	0.519	FAILURE	FAILURE	OK
11a	1.5	3.6	0.187	0.747	0.662	FAILURE	FAILURE	OK
11b	2	3.6	0.202	0.610	0.681	FAILURE	FAILURE	OK
12a	1.5	3.6	0.154	0.617	0.523	FAILURE	FAILURE	OK
12b	1.5	3.6	0.142	0.477	0.523	FAILURE	FAILURE	OK
17a	1.5	3.6	0.128	0.404	0.389	OK	OK	OK
17b	1.5	3.6	0.108	0.393	0.373	OK	OK	OK
33a	1	3.6	0.166	0.435	0.300	OK	FAILURE	OK
33b	2	3.6	0.227	0.607	0.504	FAILURE	FAILURE	OK
34a	1	3.6	0.157	0.411	0.284	OK	FAILURE	OK
34b	2	3.6	0.201	0.572	0.475	FAILURE	FAILURE	OK
35a	1.5	3.6	0.135	0.407	0.400	OK	OK	OK
35b	1.5	3.6	0.133	0.486	0.469	FAILURE	FAILURE	OK
36a	1.5	3.6	0.194	0.671	0.669	FAILURE	FAILURE	OK
36b	1.5	3.6	0.202	0.607	0.669	FAILURE	FAILURE	OK
37a	1.5	3.6	0.091	0.328	0.232	OK	OK	OK
37b	1.5	3.6	0.109	0.408	0.233	OK	OK	OK
38a	1.5	3.6	0.109	0.393	0.368	OK	OK	OK
38b	1.5	3.6	0.133	0.509	0.368	OK	FAILURE	OK
39a	1.5	3.6	0.141	0.547	0.421	FAILURE	FAILURE	OK
39b	1.5	3.6	0.127	0.467	0.421	FAILURE	FAILURE	OK
40a	1.5	3.6	0.092	0.374	0.210	OK	OK	OK
40b	1.5	3.6	0.074	0.288	0.218	OK	OK	OK

4.3.2 Addition of Lintel

In the strengthened model, mid-height lintels are added to the walls' outer and inner parts. The strengthened model is shown in Figure 4.14.

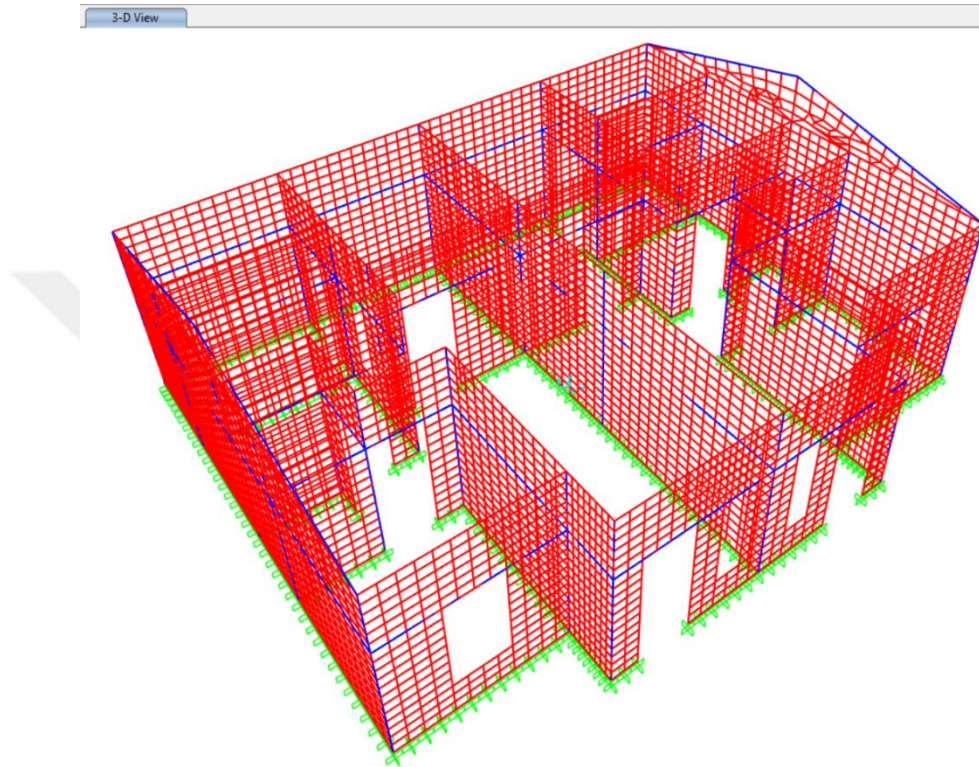


Figure 4.14. Addition of lintels (blue) on masonry walls (red) at House 3

After the lintel addition, walls 21 and 22 are sufficient and there are also improvements on other walls which need to be strengthened. The result is shown in Table 28.

Table 28. Additional lintel effect at House 3 Strengthened Model.

Wall No	Length (m)	Height (m)	S12max (MPa)	σ_{M11} (MPa)	σ_{M22} (MPa)	Flexure Check (F22)	Flexure Check (F11)	Shear Check
1a	3	2.4	0.133	0.524	0.590	FAILURE	FAILURE	OK
1b	3	1.2	0.175	0.476	0.374	OK	FAILURE	OK
5a	3	2.4	0.144	0.617	0.630	FAILURE	FAILURE	OK
5b	3	1.2	0.186	0.581	0.367	OK	FAILURE	OK
9a	4	2.4	0.407	0.600	1.039	FAILURE	FAILURE	FAILURE
9b	4	1.2	0.303	0.574	0.912	FAILURE	FAILURE	FAILURE
10a	3	2.4	0.296	0.676	0.956	FAILURE	FAILURE	OK
10b	3	1.2	0.183	0.682	0.697	FAILURE	FAILURE	OK
11a	3.5	2.4	0.143	0.444	0.554	FAILURE	FAILURE	OK
11b	3.5	1.2	0.152	0.418	0.344	OK	FAILURE	OK
12a	3	2.4	0.165	0.626	0.621	FAILURE	FAILURE	OK
12b	3	1.2	0.178	0.571	0.377	OK	FAILURE	OK
17a	3	2.4	0.134	0.531	0.590	FAILURE	FAILURE	OK
17b	3	1.2	0.163	0.484	0.377	OK	FAILURE	OK
21a	3	2.4	0.423	0.530	0.926	FAILURE	FAILURE	FAILURE
21b	3	1.2	0.179	0.538	0.303	OK	FAILURE	OK
22a	3	2.4	0.410	0.546	0.960	FAILURE	FAILURE	FAILURE
22b	3	1.2	0.171	0.550	0.326	OK	FAILURE	OK
23a	3	2.4	0.190	0.320	0.236	OK	OK	OK
23b	3	1.2	0.162	0.307	0.244	OK	OK	OK
24a	3	2.4	0.233	0.316	0.266	OK	OK	OK
24b	3	1.2	0.164	0.327	0.233	OK	OK	OK
26a	1.5	2.4	0.233	0.368	0.328	OK	OK	OK
26b	1.5	1.2	0.136	0.358	0.173	OK	OK	OK
27a	1.5	2.4	0.218	0.362	0.325	OK	OK	OK
27b	1.5	1.2	0.171	0.350	0.161	OK	OK	OK
33a	3	2.4	0.184	0.302	0.408	OK	OK	OK
33b	3	1.2	0.141	0.289	0.370	OK	OK	OK
34a	3	2.4	0.145	0.314	0.351	OK	OK	OK
34b	3	1.2	0.143	0.293	0.378	OK	OK	OK
35a	3	2.4	0.209	0.653	0.790	FAILURE	FAILURE	OK
35b	3	1.2	0.200	0.566	0.639	FAILURE	FAILURE	OK
36a	3	2.4	0.159	0.537	0.647	FAILURE	FAILURE	OK
36b	3	1.2	0.175	0.464	0.517	FAILURE	FAILURE	OK
37a	3	2.4	0.103	0.488	0.310	OK	FAILURE	OK
37b	3	1.2	0.118	0.492	0.305	OK	FAILURE	OK
38a	3	2.4	0.083	0.379	0.361	OK	OK	OK

Table 28 (continued)

39a	3	2.4	0.082	0.365	0.344	OK	OK	OK
39a	3	2.4	0.082	0.365	0.344	OK	OK	OK
39b	3	1.2	0.089	0.347	0.238	OK	OK	OK
40a	3	2.4	0.099	0.409	0.303	OK	FAILURE	OK
40b	3	1.2	0.088	0.413	0.294	OK	FAILURE	OK

4.3.3 Addition of Lintel + Column

A combination of lintel and column is also applied to observe the flexural and shear strength improvement of the non-engineered masonry house 3. The strengthened model is shown in Figure 4.15.

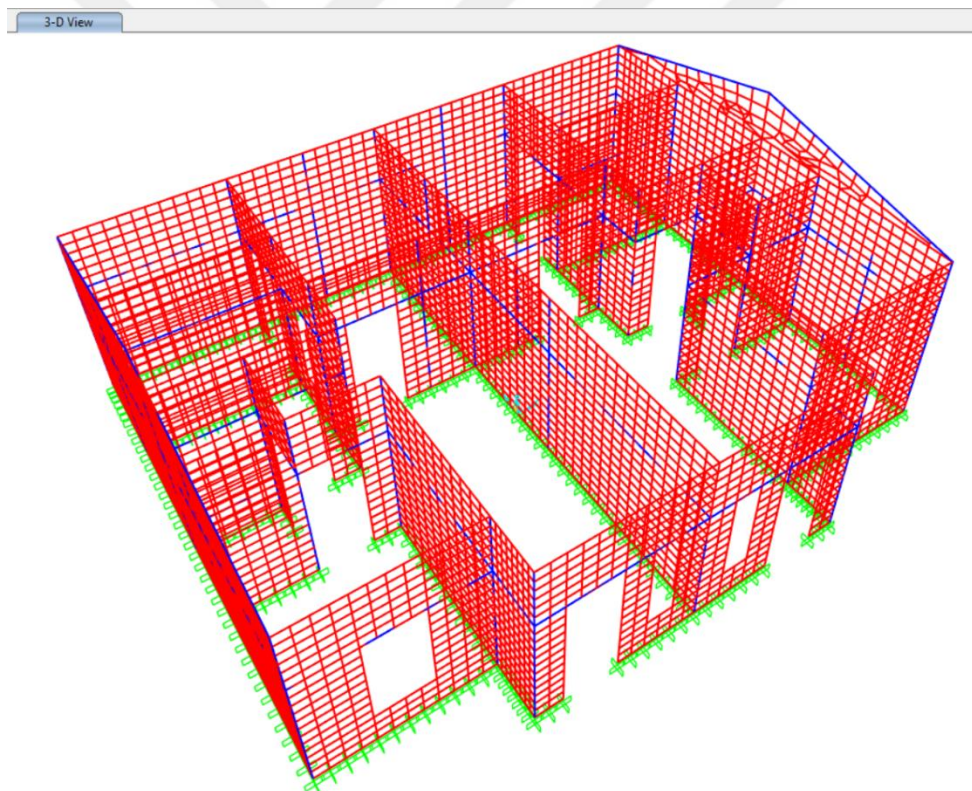


Figure 4.15. Addition of columns and lintels (blue) on masonry walls (red) at house 3.

Initially, at walls 5, 12, 35, and 36 the column spacing was 1.5 m and after the addition of both column and lintel, the wall still failed due to σ_{M11} . To improve the walls, additional columns with 1 m spacing is used and the result is tabulated in Table 29. In addition to that, at wall 9 column spacing was 2 m and after the addition of both column and lintel, the wall still failed due to σ_{M11} . To improve it, an additional column with 1 m spacing is used, and the result is tabulated in Table 29.

After additional lintel + column, walls 11, 12, 38 and 39 are sufficient.

Table 29. Column and Lintel Additions effect at House 3 Strengthened Model.

Wall No	Length (m)	Height (m)	S12max (MPa)	σ_{M11} (MPa)	σ_{M22} (MPa)	Flexure Check (F22)	Flexure Check (F11)	Shear Check
5a	1.5	2.4	0.128	0.388	0.395	OK	OK	OK
5b	1.5	2.4	0.109	0.388	0.575	FAILURE	OK	OK
5c	1.5	1.2	0.148	0.382	0.532	FAILURE	OK	OK
5d	1.5	1.2	0.136	0.386	0.232	OK	OK	OK
5e	1	1.2	0.107	0.388	0.298	OK	OK	OK
5f	1	1.2	0.156	0.383	0.206	OK	OK	OK
9a	2	2.4	0.190	0.375	0.473	FAILURE	OK	OK
9b	2	2.4	0.275	0.389	0.955	FAILURE	OK	OK
9c	2	1.2	0.292	0.406	1.032	FAILURE	OK	OK
9d	2	1.2	0.298	0.400	1.067	FAILURE	OK	OK
9e	1	1.2	0.249	0.387	0.277	OK	OK	OK
9f	1	1.2	0.254	0.384	0.673	FAILURE	OK	OK
9g	1	1.2	0.226	0.392	0.810	FAILURE	OK	OK
9h	1	1.2	0.235	0.398	0.784	FAILURE	OK	OK
10a	1.5	2.4	0.270	0.397	1.037	FAILURE	OK	OK
10b	1	2.4	0.212	0.403	0.527	FAILURE	OK	OK
10c	1.5	1.2	0.206	0.391	0.643	FAILURE	OK	OK
10d	1	1.2	0.251	0.406	0.323	OK	OK	OK
11a	1.5	2.4	0.153	0.399	0.408	OK	OK	OK
11b	2	2.4	0.161	0.390	0.406	OK	OK	OK
11c	1.5	1.2	0.160	0.399	0.338	OK	OK	OK
11d	2	1.2	0.184	0.407	0.408	OK	OK	OK
12a	1.5	2.4	0.126	0.381	0.309	OK	OK	OK
12b	1.5	2.4	0.065	0.403	0.348	OK	OK	OK
12c	1.5	1.2	0.120	0.252	0.326	OK	OK	OK
12d	1.5	1.2	0.119	0.255	0.341	OK	OK	OK
12e	1	1.2	0.062	0.405	0.398	OK	OK	OK

Table 29 (continued)

12f	1	1.2	0.165	0.385	0.325	OK	OK	OK
21a	3	2.4	0.297	0.381	0.851	FAILURE	OK	OK
21b	3	1.2	0.165	0.408	0.315	OK	OK	OK
22a	3	2.4	0.280	0.389	0.949	FAILURE	OK	OK
22b	3	1.2	0.165	0.405	0.325	OK	OK	OK
35a	1.5	2.4	0.145	0.392	0.386	OK	OK	OK
35b	1.5	2.4	0.089	0.396	0.579	FAILURE	OK	OK
35c	1.5	1.2	0.156	0.402	0.419	FAILURE	OK	OK
35d	1.5	1.2	0.172	0.382	0.232	OK	OK	OK
35e	1	1.2	0.093	0.384	0.330	OK	OK	OK
35f	1	1.2	0.165	0.401	0.349	OK	OK	OK
36a	1.5	2.4	0.182	0.404	0.461	FAILURE	OK	OK
36b	1.5	2.4	0.183	0.406	0.683	FAILURE	OK	OK
36c	1.5	1.2	0.176	0.390	0.402	OK	OK	OK
36d	1.5	1.2	0.189	0.399	0.280	OK	OK	OK
36e	1	1.2	0.100	0.402	0.387	OK	OK	OK
36f	1	1.2	0.188	0.408	0.392	OK	OK	OK
38a	1.5	2.4	0.076	0.321	0.308	OK	OK	OK
38b	1.5	2.4	0.074	0.331	0.310	OK	OK	OK
38c	1.5	1.2	0.134	0.340	0.344	OK	OK	OK
38d	1.5	1.2	0.107	0.330	0.356	OK	OK	OK
39a	1.5	2.4	0.075	0.339	0.319	OK	OK	OK
39b	1.5	2.4	0.095	0.331	0.317	OK	OK	OK
39c	1.5	1.2	0.108	0.348	0.354	OK	OK	OK
39d	1.5	1.2	0.108	0.340	0.367	OK	OK	OK

Moreover, the location of existing columns and additional columns are given in Figure 4.16 where red columns are existing columns and blue columns are additional columns.

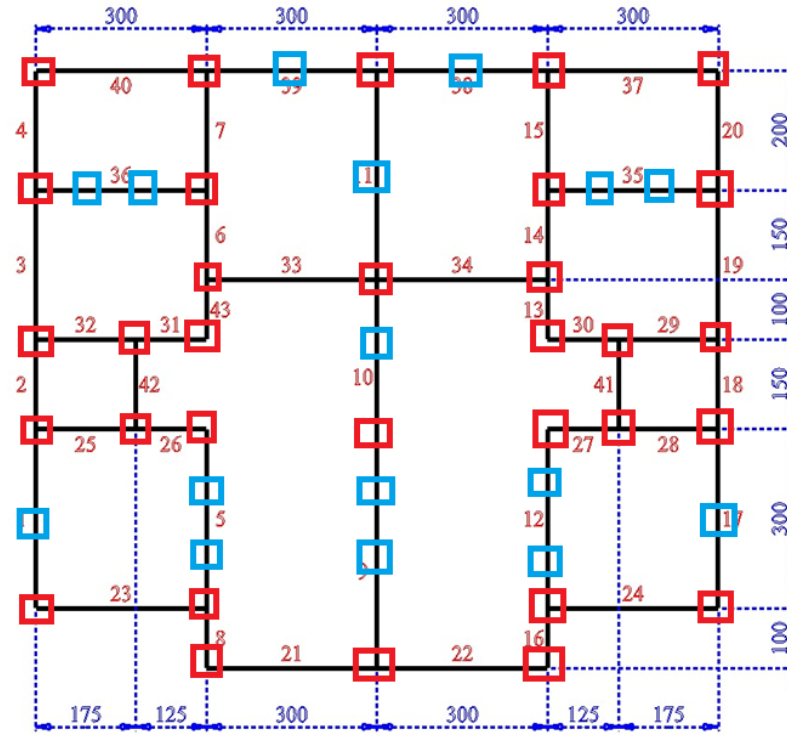


Figure 4.16. Location of existing columns (red) and additional columns (blue) at house 3.

4.3.4 Rebars Addition for Post-Tensioning

To determine the amount of post-tensioning required for the walls which are necessary to be post-tensioned and the diameter of rebars that will be used for post-tensioning. The following formula is used:

$$\frac{M * y}{I} - \sigma_{post-tensioning} = \sigma_{tensile\ capacity}$$

The tensile capacity is obtained from the flexural strength test[2] and it is taken as 0.408 MPa.

For example, after adding a lintel and column in the middle, wall number 10 is separated into 4 different walls: wall 10a, wall 10b, wall 10c, and wall 10d. At wall 10a, the bending moment M22 is calculated

as 2.5935 kNm. The required post-tensioning load and diameter of rebar that will be used are calculated as follows:

$$\frac{M * y}{I} - \sigma_{post-tensioning} = \sigma_{tensile\ capacity}$$

$$\frac{2.5935 * 0.05 * 0.001}{\frac{1.5 * 0.1^3}{12}} - \sigma_{post-tensioning} = 0.408$$

$$\sigma_{post-tensioning} = 0.6294\ MPa$$

Considering 0.5 x 0.1 m surface area, the amount of post-tensioning load required for each bar is calculated as follows:

$$629.4\ kPa * 0.5\ m * 0.1\ m = 31.47\ kN$$

Since there are 2 bars on each side at every 0.5 m spacing, the required post-tensioning load at each bar is calculated as:

$$\frac{31.47}{2}\ kN = 15.735\ kN/bar$$

According to SNI 2052:2017 [28], the allowable tensile strength of the rebar is 350 MPa. Thus, the rebar diameter required is calculated as:

$$\sigma_{allow\ tensile} = \frac{P_{post-tensioning}}{A}$$

$$350\ MPa = \frac{15735\ N}{A}$$

$$A = 44.957\ mm^2$$

Using 8 mm diameter rebar ($A_s = 50.265\ mm^2$) is sufficient.

The above calculations are applied to other walls that must be post-tensioned. The result is given in Table 30.

Table 30. Post Tensioning Required for House 3 Strengthened Model

Wall No	σ_{M22} (MPa)	Flexure Check (F22)	Tensile Strength Required (MPa)	Force/Bar (KN/Bar)	Area Required (Mm ²)	Rebar Diameter Used/0.5 m
5a	0.395	OK				
5b	0.575	FAILURE	0.1668	4.17	11.91429	6
5c	0.532	FAILURE	0.1242	3.105	8.871429	6
5d	0.232	OK				
5e	0.298	OK				
5f	0.206	OK				
9a	0.473	FAILURE	0.0654	1.635	4.671429	8
9b	0.955	FAILURE	0.5466	13.665	39.04286	8
9c	1.032	FAILURE	0.624	15.6	44.57143	8
9d	1.067	FAILURE	0.6594	16.485	47.1	8
9e	0.277	OK				
9f	0.673	FAILURE	0.2652	6.63	18.94286	8
9g	0.810	FAILURE	0.402	10.05	28.71429	8
9h	0.784	FAILURE	0.3756	9.39	26.82857	8
10a	1.037	FAILURE	0.6294	15.735	44.95714	8
10b	0.527	FAILURE	0.1188	2.97	8.485714	8
10c	0.643	FAILURE	0.2346	5.865	16.75714	8
10d	0.323	OK				
21a	0.851	FAILURE	0.4428	11.07	31.62857	8
21b	0.315	OK				
22a	0.949	FAILURE	0.5412	13.53	38.65714	8
22b	0.325	OK				
35a	0.386	OK				
35b	0.579	FAILURE	0.171	4.275	12.21429	6
35c	0.419	FAILURE	0.0114	0.285	0.814286	6
35d	0.232	OK				
35e	0.330	OK				
35f	0.349	OK				
36a	0.461	FAILURE	0.0528	1.32	3.771429	6
36b	0.683	FAILURE	0.2748	6.87	19.62857	6
36c	0.402	OK				
36d	0.280	OK				
36e	0.387	OK				
36f	0.392	OK				

4.1 Displacement criteria for strengthened models

Using displacement criteria explained in section 3.4.2 and maximum displacements obtained at both X and Y axis for all strengthened models, all strengthened models' maximum allowable displacements according to ASCE, Euro code 8, and TBDY 2018 are calculated. The summary of the results for each strengthening technique is tabulated in Table 31, Table 32, Table 33. It can be concluded that all of the strengthened models' maximum displacements are below the maximum allowable displacements determined according to ASCE 7-16, Euro code 8, and TBDY 2018.

Table 31. Displacement criteria for additional column models

ASCE	X Axis (mm)			
Additional column	Max Disp	Story drift	Allowable	
House 1	4.26	5.325	36	OK
House 2	3.25	4.0625	31.5	OK
House 3	4.25	5.3125	36	OK
ASCE	Y Axis (mm)			
Additional column	Max Disp	Story drift	Allowable	
House 1	2.15	2.6875	36	OK
House 2	2.56	3.2	31.5	OK
House 3	1.99	2.4875	36	OK
Euro Code 8	X Axis (mm)			
Additional column	Max Disp	Story drift	Allowable	
House 1	4.26	3.195	18	OK
House 2	3.25	2.4375	15.75	OK
House 3	4.25	3.1875	18	OK
Euro Code 8	Y Axis (mm)			
Additional column	Max Disp	Story drift	Allowable	
House 1	2.15	1.6125	18	OK
House 2	2.56	1.92	15.75	OK
House 3	1.99	1.4925	18	OK

Table 31 (continued)

TBDY 2018	X Axis			
Additional column	Max Disp (mm)	Calculated	Allowable	
House 1	4.26	0.001595	0.008	OK
House 2	3.25	0.001391	0.008	OK
House 3	4.25	0.001591	0.008	OK
TBDY 2018	Y Axis			
Additional column	Max Disp (mm)	Calculated	Allowable	
House 1	2.15	0.000805	0.008	OK
House 2	2.56	0.000958	0.008	OK
House 3	1.99	0.000745	0.008	OK

Table 32. Displacement criteria for additional lintel models

ASCE	X Axis (mm)			
Additional lintel	Max Disp	Story drift	Allowable	
House 1	5.31	6.6375	36	OK
House 2	3.42	4.275	31.5	OK
House 3	5.12	6.4	36	OK
ASCE	Y Axis (mm)			
Additional lintel	Max Disp	Story drift	Allowable	
House 1	3.15	3.9375	36	OK
House 2	2.43	3.0375	31.5	OK
House 3	1.89	2.3625	36	OK
Euro Code 8	X Axis (mm)			
Additional lintel	Max Disp	Story drift	Allowable	
House 1	5.31	3.9825	18	OK
House 2	3.42	2.565	15.75	OK
House 3	5.12	3.84	18	OK

Table 32 (continued)

Euro Code 8	Y Axis (mm)			
Additional lintel	Max Disp	Story drift	Allowable	
House 1	3.15	2.3625	18	OK
House 2	2.43	1.8225	15.75	OK
House 3	1.89	1.4175	18	OK
TBDY 2018	X Axis			
Additional lintel	Max Disp (mm)	Calculated	Allowable	
House 1	5.31	0.001988	0.008	OK
House 2	3.42	0.001463	0.008	OK
House 3	5.12	0.001917	0.008	OK
TBDY 2018	Y Axis			
Additional lintel	Max Disp (mm)	Calculated	Allowable	
House 1	3.15	0.001179	0.008	OK
House 2	2.43	0.00091	0.008	OK
House 3	1.89	0.000708	0.008	OK

Table 33. Displacement criteria for additional column+lintels models

ASCE	X Axis			
lintel+column	Max Disp	Story drift	Allowable	
House 1	4.82	6.025	36	OK
House 2	3.01	3.7625	31.5	OK
House 3	4.56	5.7	36	OK
ASCE	Y Axis			
Lintel+column	Max Disp	Story drift	Allowable	
House 1	2.414	3.0175	36	OK
House 2	1.97	2.4625	31.5	OK
House 3	1.87	2.3375	36	OK
Euro Code 8	X Axis			
Lintel+column	Max Disp	Story drift	Allowable	
House 1	4.82	3.615	18	OK
House 2	3.01	2.2575	15.75	OK
House 3	4.56	3.42	18	OK
Euro Code 8	Y Axis			
Lintel+column	Max Disp	Story drift	Allowable	
House 1	2.414	1.8105	18	OK
House 2	1.97	1.4775	15.75	OK
House 3	1.87	1.4025	18	OK
TBDY 2018	X Axis			
Lintel+column	Max Disp	Calculated	Allowable	
House 1	4.82	0.001805	0.008	OK
House 2	3.01	0.001288	0.008	OK
House 3	4.56	0.001707	0.008	OK
TBDY 2018	Y Axis			
Lintel+column	Max Disp	Calculated	Allowable	
House 1	2.414	0.000904	0.008	OK
House 2	1.97	0.000738	0.008	OK
House 3	1.87	0.0007	0.008	OK



CHAPTER 5

MINIMUM REQUIREMENTS AND RECOMMENDATIONS

In addition to the proposed strengthening techniques that have been applied in this thesis, some recommendations and minimum requirements that are available in both Türkiye and Indonesia are discussed in this chapter to further enhance the earthquake-resistant capacity of non-engineered confined masonry houses in Indonesia.

5.1 Recommendations and Requirements According to TBDY 2018 and EERI & SDC/HA

Swiss Agency for Development and Cooperation / Human Aid (SDC/HA) ¹ and Earthquake Engineering Research Institute (EERI) ² have jointly prepared a guideline in Turkish in June 2018. According to the construction guideline for confined masonry houses [29], it is recommended that the maximum distance between columns is 4.5 m while the maximum height of the wall recommended is 3 m. Considering that the wall heights in the analyzed masonry houses are 3.6 m and 3.15 m, both exceed the recommended maximum height. Initially, all walls at house 1 had a distance between columns less than 4.5 m, except for walls number 15 and 16, which had a distance of 6 m. These discrepancies have been addressed in the strengthening process. Moreover, all the walls at house 2 and house 3 had a distance between columns less than 4.5 m as recommended in the guideline.

¹ <https://www.eda.admin.ch/sdc>

² <https://www.eeri.org/>

Additionally, the manual recommends that the minimum longitudinal reinforcements required are $\phi 10$ mm and $\phi 12$ mm are preferred; whereas, minimum stirrups requirements are $\phi 6$ mm and $\phi 8$ mm are preferred. In the modeled and actual non-engineered confined masonry houses, the longitudinal reinforcements are used at both tie beams and tie columns as $\phi 10$ mm; whereas, stirrup reinforcements used at both tie beams and tie columns are $\phi 6$ mm. Thus, it satisfies the minimum requirements defined in the manual.

Regarding stirrup spacing, the guideline recommends to have 10 cm spacing at $h/6$ distance from both ends of a column and a beam; whereas, it is recommended to have 20 cm spacing for the rest of the column and beam lengths, as shown in Figure 5.1.

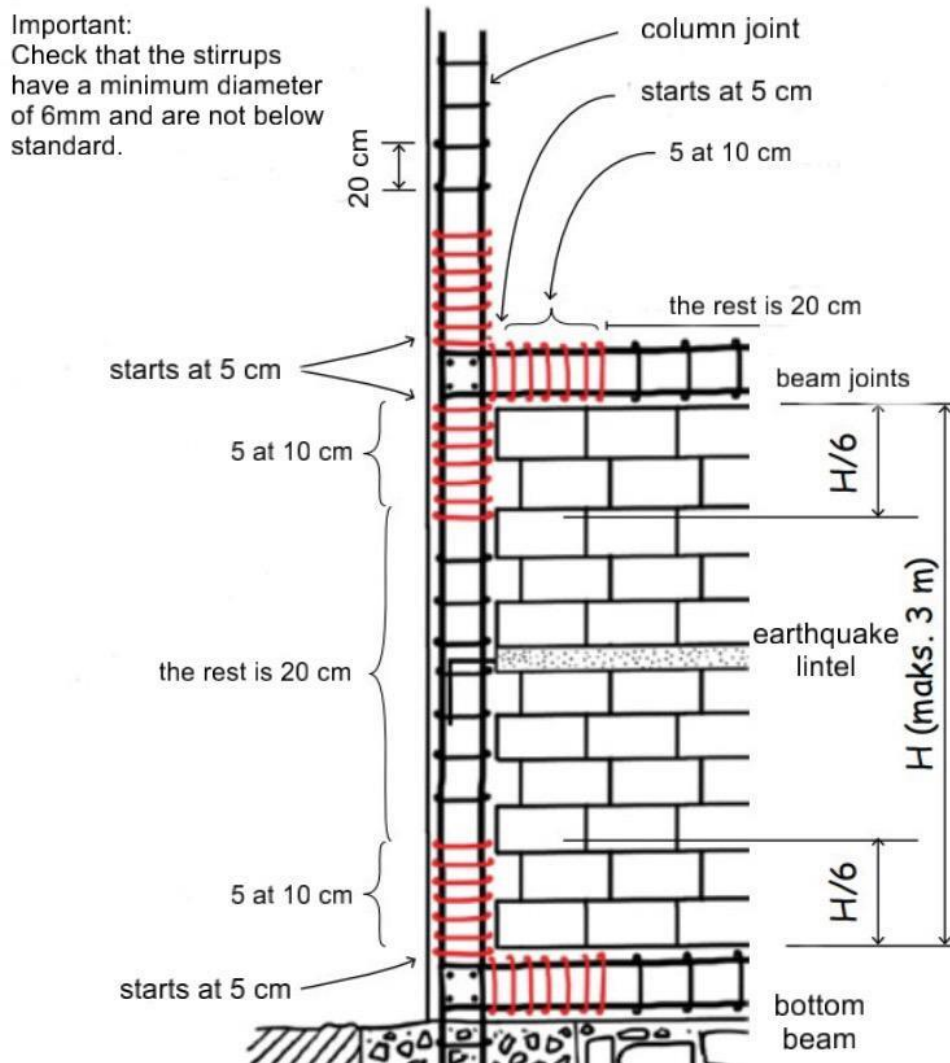


Figure 5.1. Recommended stirrups spacing between tie columns and beams [29].

At the modeled and actual houses, the stirrups' spacing is 15 cm throughout the height, which is sufficient for the middle part but insufficient for $h/6$ from both connection ends. Thus, it is recommended to have 10 cm spacing along 60 cm distance from the joints for beams and columns, while having either 20 cm or 15 cm spacing for the rest of the columns and beams is enough. On the other hand, possible errors and complexity may be eliminated by considering an even 10 cm spacing throughout the lengths of the beams and columns is a simpler approach without precise measurement and complexity.

Furthermore, at the connections between columns and bottom beams, it is recommended to have additional rebar (either L-shaped as shown in Figure 5.2, T-shaped as shown in Figure 5.3, or X (or “+”) shaped as shown in Figure 5.4) with a length of $50 \times$ rebar diameter (i.e. 500 mm). This recommendation is missing in the currently available (already constructed) non-engineered masonry houses, and this can be recommended for future houses to be constructed.

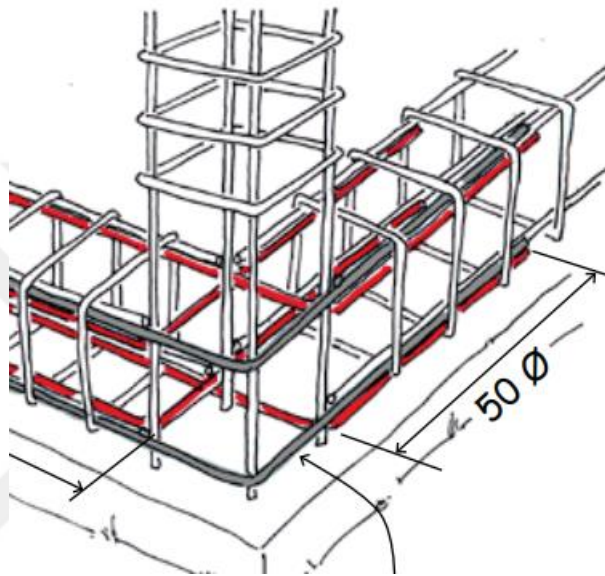


Figure 5.2. L-shaped rebar between columns and beams [29].

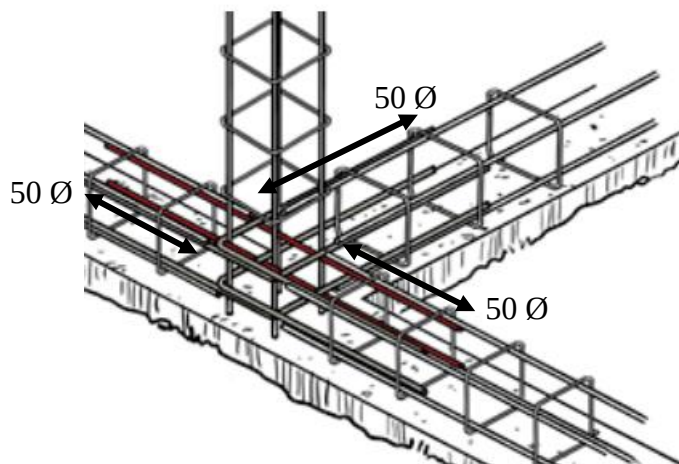


Figure 5.3. T-shaped rebar between columns and beams [29].

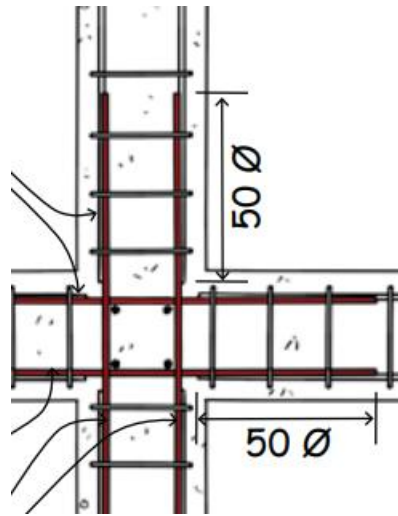


Figure 5.4. Top view of the bottom beams connecting to a central column [29].

Regarding the suitable bricks that should be used, the manual recommends [29] that full bricks should be used instead of bricks with holes. Alternatively, holes' areas should be less than 50% of the total area of the bricks if bricks with holes are used. Moreover, it is recommended to have a minimum brick width of 11 cm. If the brick wall width is less than 12.5 cm, double-layered bricks must be used for the wall. The full bricks, typically produced in Indonesia with traditional techniques, are used in the houses that were modelled in this thesis. Thus, the type of the bricks is aligned with the recommendations in the manual. On the other hand, although the 10 cm width used bricks were suitable and in the range of 7.5-15 cm as described in the guideline, double layer bricks (20 cm thick wall) is necessary since the single layer brick wall width is 10 cm and smaller than 12.5 cm requirement.

In addition to that, it is recommended to have a 30 cm length rebar extension at the mortar reinforcement – column connection along the column, which can increase connection continuity between tie columns and mortar layer reinforcement. This rebar extension is not available in the modelled masonry houses and this application can be recommended for new houses to be built in the future. The application is shown in Figure 5.5.

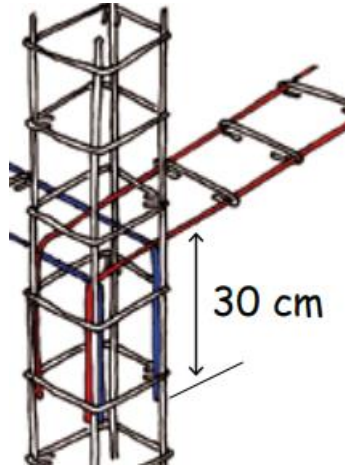


Figure 5.5. Mortar layer reinforcement extension along column reinforcement [29].

In Turkish earthquake code (TBDY 2018) [26], the unsupported wall length limits are given for maximum allowable distance between tie columns (≤ 4 m) and between perpendicular walls (≤ 16 m) in confined masonry houses as shown in Figure 5.6.

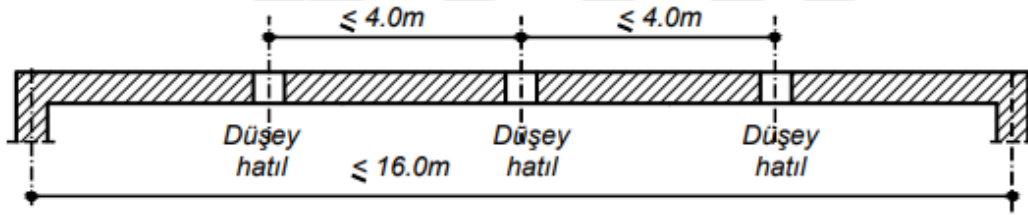


Figure 5.6. Maximum distances between tie columns and perpendicular walls [26].

In the current design of confined masonry houses, most of the walls have distances between tie columns less than or equal to 4 m, except for walls 15 and 16 in house 1, which have distances of 6 m between tie columns. The longest wall being 6 m long also satisfies the 16 m length requirement. The addition of a new tie columns to the mid length of walls 15 and 16 would make the walls comply with the maximum allowable distance stated in TBDY 2018 [26]. Moreover, the maximum allowable distance between window and door frames to the edges for both T-shaped and L-shaped joint is given in Figure 5.7.

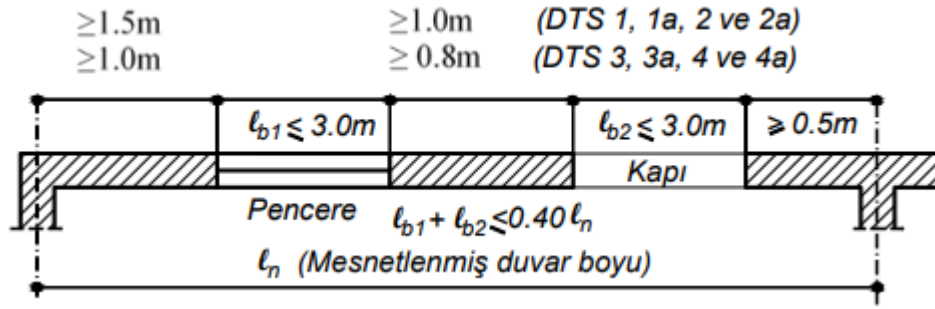


Figure 5.7. Maximum allowable distance for window and door frames to the edges [26].

In the current design of confined masonry houses, most of the openings in walls have sufficient distances to the edges except walls at the front part of the houses, where there are large window and door openings in the same wall (walls 5 and 27 in house 1, wall 9 in house 2 in addition to walls 21 and 22 in house 3). Thus, it is recommended to have smaller size of window and door openings for these walls to satisfy the requirements stated in TBDY 2018 [26]. Door to the edge is $40 \text{ cm} < 50 \text{ cm}$ and walls on either side of the window are only $65 \text{ cm} < 100 \text{ cm}$. The window may be removed, a larger door with a transparent window may be a solution. Alternatively, the wall may be elongated, and the house plan area can be larger.

5.2 Recommendations and Requirements According to Retrofitting Simple Buildings Damaged by Earthquakes Manual

The manual by Prof. T. Boen [6] provides practical and experience-based information on both strengthening and retrofitting techniques for typical non-engineered masonry houses in Indonesia and referred as “Retrofitting Simple Buildings Damaged by Earthquakes Manual”. Initially, it is suggested to have a symmetrical shape in the house's overall plan view and appearance. If the shape is not symmetrical, it is recommended to have dilatation to adjust the shape of buildings into several symmetrical buildings instead of a single block, which was not possible for the houses studied in this thesis. Furthermore, the openings of doors and windows must be positioned as symmetrically as possible, particularly with respect to the main

axis. The symmetrical shape of the structure reduce vulnerability due to asymmetric loads and minimize torsional effects when earthquakes happen. Moreover, all building components including tie columns, tie beams, and masonry walls should be tied to each other so that the buildings will function as one integral unit when earthquakes occur, which is not possible when dilatations exist. For tie columns and beams, rebar extension can be used; whereas, tie columns and masonry walls can be connected using anchor and tothing techniques.

The minimum performance level for earthquake-resistant non-engineered buildings is as follows:

1. Sufficient to withstand wind load and gravity load from both dead and live loads without any damage to structural components and non-structural components.
2. Resist minor earthquakes (43 years return period) without any damage.
3. Resist moderate earthquakes (72 years return period) without structural damage but only non-structural damage.
4. During a major earthquake, it should not suffer partial or total collapse. Even if damage occurs, it should be able to be retrofitted with lower costs compared to constructing a new building and re-function as soon as possible.

T. Boen [6] recommends rebar extension between column and beam connections with an extension length of $40 \times$ rebar diameter, as shown in Figure 5.8 and Figure 5.9 which is similar to construction manual for confined masonry houses by EERI [29]. However, manual by EERI recommends $50 \times$ rebar diameter extension length with various detailing recommendation depending on tie columns and tie beams connections shapes (T-shape, X-shape, or L-shape).

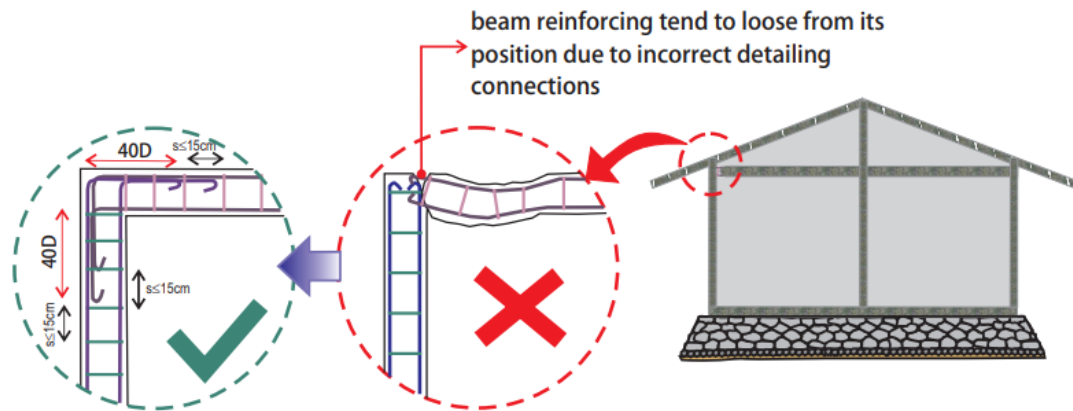


Figure 5.8. Beam rebar extension through the column at the beam and column connection [6].

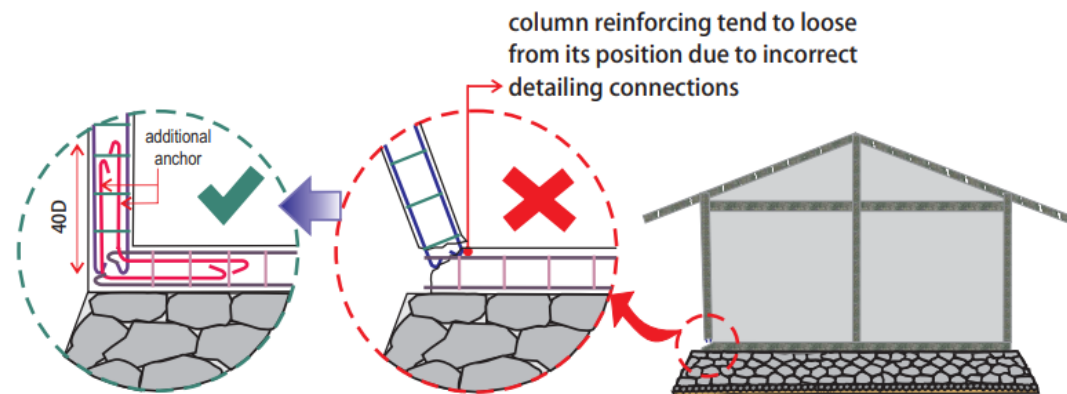


Figure 5.9. Column rebar extension through the beam at the beam and column connection [6].

According to the manual [6], the minimum longitudinal reinforcements for beams and columns are $4\phi 10$ mm with minimum stirrups of $\phi 8$ mm with 150 mm spacing. Thus, the actual and modeled non-engineered houses which have longitudinal reinforcements of $4\phi 10$ mm with stirrups reinforcements of $\phi 6$ mm with 150 mm satisfy the longitudinal reinforcements requirements, but the stirrups reinforcement should have $\phi 8$ mm instead of $\phi 6$ mm.

Regarding retrofitting of confined masonry houses, right after a damaging earthquake, emergency repairs & temporary support need to be carried out so that it can re-function and not collapse due to aftershocks. Moreover, after the situation has settled down, three types of action can be taken:

1. Repair.

Repair the architectural shape of a building so that the building can re-function as soon as possible. These include re-plastering walls, patching defects such as cracks, rebuilding non-structural walls, checking/repairing gas pipes, water pipes, and plumbing services.

2. Restoration.

Restoration is a structural repair to restore structural components' original strength. These include injecting epoxy, which has a high tensile capacity, into cracks in walls, columns, beams, etc., and the addition of reinforcing mesh on both sides of the cracked wall.

3. Strengthening.

Strengthening techniques can be used to make the building stronger than before. These include avoiding brittle failure by re-arranging, adding reinforcing bars, and improving detailing according to ductility requirements. In addition to that, lateral strength can be increased by adding walls, columns, etc.

When constructing a wall, it is recommended to provide wall anchoring with $\phi 10$ mm rebar and a minimum length of 40 cm for every 6 layers of bricks, as shown in Figure 5.10. These anchors will help to improve connection between masonry walls and tie columns which will prevent them from separating or shifting excessively during seismic event.

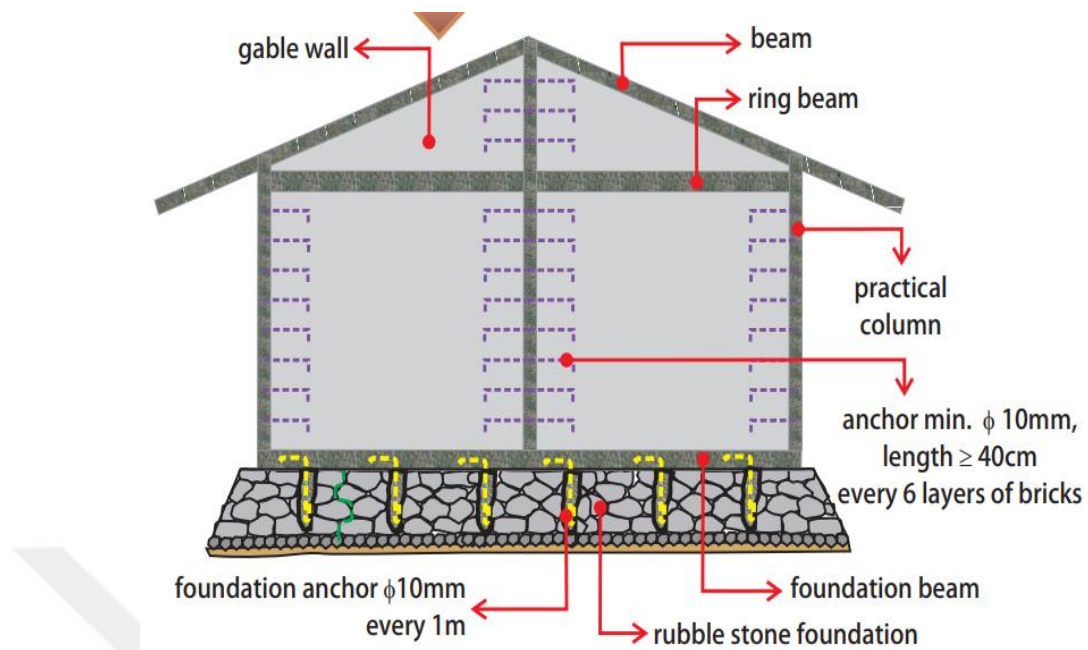


Figure 5.10. Anchor addition at both walls and foundation. [6]

The manual written by T. Boen [6] provides various retrofitting techniques that can be applied depending on the damage after an earthquake, such as buckled columns with deformed reinforcement at the top, wall failures at the corners, ceilings collapses, and roof trusses dislocations. It offers a classification system to determine which retrofit technique should be used based on the structural damage, a step-by-step guide to apply each retrofitting technique, and explanations of how each technique will improve the confined masonry houses' overall earthquake resistant capacity.



CHAPTER 6

DISCUSSION OF RESULTS AND CONCLUSIONS

6.1 Overall Discussion of Results

In this thesis, 3 different non-engineered confined masonry houses are modelled using SAP2000 and yield line theory is used to estimate the failure line and capacity of each wall, the comparison between SAP2000 model and analytical solution from yield line theory is given in Table 34. From the table it can be concluded that yield line theory can provide reasonable accuracy to determine which walls fail during earthquake.

Table 34. Comparison between SAP2000 analysis result and Yield line theory

House 1		
Total wall	37	
Unstrengthened	SAP	Yield Line
Fail wall	19	20
Percentage	51%	54%
House 2		
Total Wall	17	
Unstrengthened	SAP	Yield Line
Fail wall	7	6
Percentage	41%	35%
House 3		
Total wall	43	
Unstrengthened	SAP	Yield Line
Fail wall	19	18
Percentage	44%	42%

Analysis for masonry wall consists of linear and nonlinear analysis. Linear analysis considers a linear relationship between applied loads and displacement before yielding using small displacement and linear material properties assumption whereas

nonlinear analysis considers material's nonlinear post-yielding stress-strain relationship, geometric imperfections, and nonlinear effects such as joint opening and mortar-brick interaction. In masonry wall, linear stress-strain relationship is observed until the first crack starts to occur. Once the crack begins, the wall starts to behave nonlinearly while resisting the additional applied loads until it reaches its maximum load carrying capacity and larger cracks observe. In this study, linear analysis is done to analyze the capacity of each wall before initial cracks occur at different non-engineered confined masonry houses and obtain the improvement in the capacity of each wall after strengthening techniques using additional lintels, additional columns, combination of lintels and columns, and post-tensioning.

Initially, unstrengthened models of houses are modelled and both of their flexural strength at both in in-plane axes and shear stresses are compared with experimental results. The weakest walls are strengthened using a) columns only, b) lintels only, c) both columns and lintels and d) post-tensioning. The improvements are discussed and plotted in the following sections.

Most of the walls are strengthened adequately using the lintel and column additions. The limitation caused by window and door openings prohibited addition of extra columns. Addition of lintels at the mid height of the walls are sometimes prevented by openings as well. Adding lintels at the top level of windows and doors left a larger portion without lintels at the lower part of walls. A minimum spacing of 1m for the columns is considered as a limit since more columns would make the confined masonry wall unreasonably crowded with columns. When a wall is not sufficiently strengthened using lintels and columns, post-tensioning was utilized. The design is not checked again using FEM after post-tensioning since it is well known fact to burry stress field into compression. The mortar and bricks have a compression strength larger than 4.98 MPa, therefore, an even stress field to match 0.4 MPa tension capacity would not cause compression failure.

6.1.1 House 1 Discussion of Results

At house 1 there are 37 walls and the unstrengthen model shows that 18 of them are considered safe for both shear failure and out-of-plane flexural failure. Moreover, 19 walls failed due to either flexural failure at horizontal 1-1 axis (M22) or vertical 2-2 axis (M11) or both. To improve this, initially columns are added to 10 walls which do not have large openings such as window or door. From the additional columns only, walls 12, 14, and 18 are considered sufficient; whereas, insufficient improvements are observed at walls 1, 3, 6, 15, 16, 24, 25, and 35.

In addition to that, additional lintels are also modelled without adding columns to obtain improvement. From additional lintels only, walls 2, 12, 13, 14, 17, 18, and 26 are considered sufficient; whereas some improvements are observed at other walls which need to be further strengthened. Furthermore, both lintels and columns are added to the model as the third strengthened model to obtain additional improvement. From additional lintels and columns walls 2, 7, 14, 20 and 26 are also considered as sufficient whereas improvements are observed at other strengthened walls which need to be strengthened.

Additionally, considering that addition of both columns and lintels do not improve σ_{M22} to sufficient strength at walls 1, 3, 5, 6, 15, 16, 24, 25, 27, and 35. These walls are strengthened further using post-tensioning utilizing the proper rebar diameter and using the force obtained from Table 22.

The comparison of strength improvements at both flexural and shear strengths with different strengthening techniques are plotted in Figure 6.2, Figure 6.4, and Figure 6.6 considering a tensile strength of 400 kPa

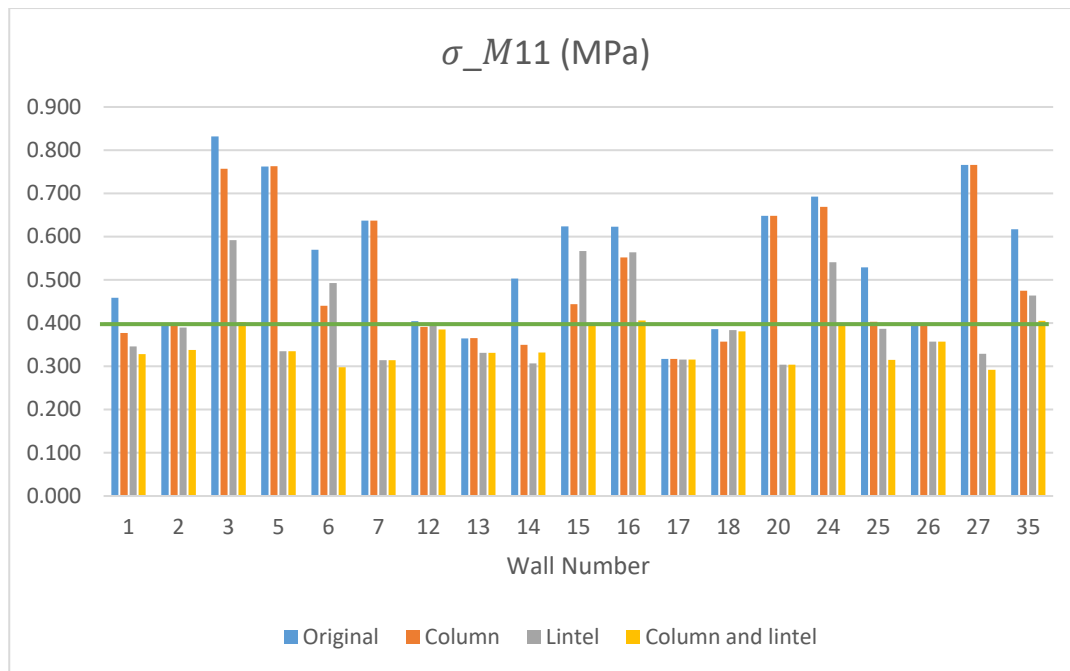


Figure 6.1. σ_{M11} comparison of each strengthening techniques at walls which are insufficient for house 1.

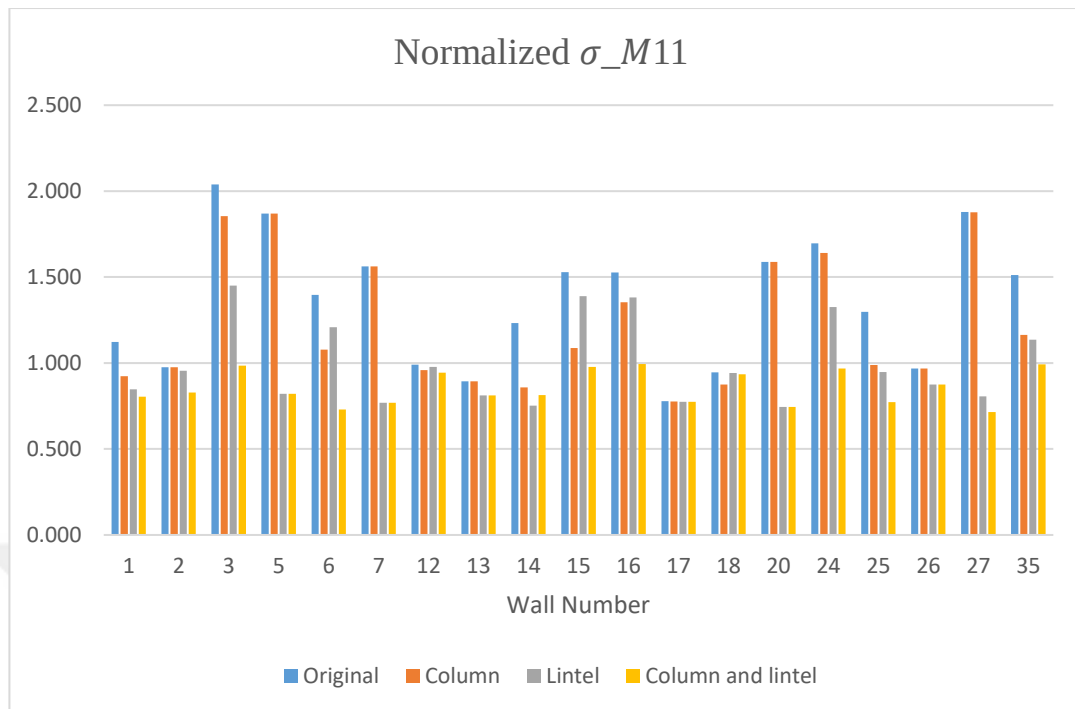


Figure 6.2. The normalized σ_{M11} as a ratio to the experimental stress / strength for each strengthening techniques for walls which are insufficient for house 1.

Some of the walls such as 2, 12, 13, 17, 18, 26 are safe for M11 but listed in Figure 6.1 and Figure 6.2 because these walls have M22 based stresses exceeding the stress capacity.

Table 35. Relative improvement of σ_{M11} for walls which are insufficient for house 1 using different strengthening techniques.

WALL	σ_{M11}			
	ORIGINAL	COLUMN	LINTEL	COLUMN & LINTEL
1	1.124	0.924	0.848	0.804
2	0.975	0.975	0.956	0.828
3	2.039	1.855	1.451	0.985
5	1.869	1.870	0.821	0.821
6	1.397	1.078	1.208	0.730
7	1.562	1.561	0.770	0.770
12	0.991	0.958	0.978	0.944
13	0.894	0.895	0.811	0.811
14	1.233	0.858	0.752	0.814
15	1.528	1.088	1.389	0.978
16	1.526	1.354	1.381	0.995
17	0.778	0.777	0.775	0.775
18	0.946	0.875	0.941	0.934
20	1.588	1.588	0.745	0.745
24	1.697	1.640	1.326	0.968
25	1.297	0.988	0.949	0.772
26	0.968	0.968	0.875	0.875
27	1.878	1.877	0.806	0.716
35	1.512	1.164	1.136	0.993
AVG	1.358	1.226	0.996	0.856

In Table 35, the stress levels are normalized using the accepted flexural capacity of 0.408 MPa [2]. It can be concluded that additional column would improve σ_{M11} by 9.7%, additional lintel would improve σ_{M11} by 26.7% and combination of column + lintel would improve σ_{M11} by 37%.

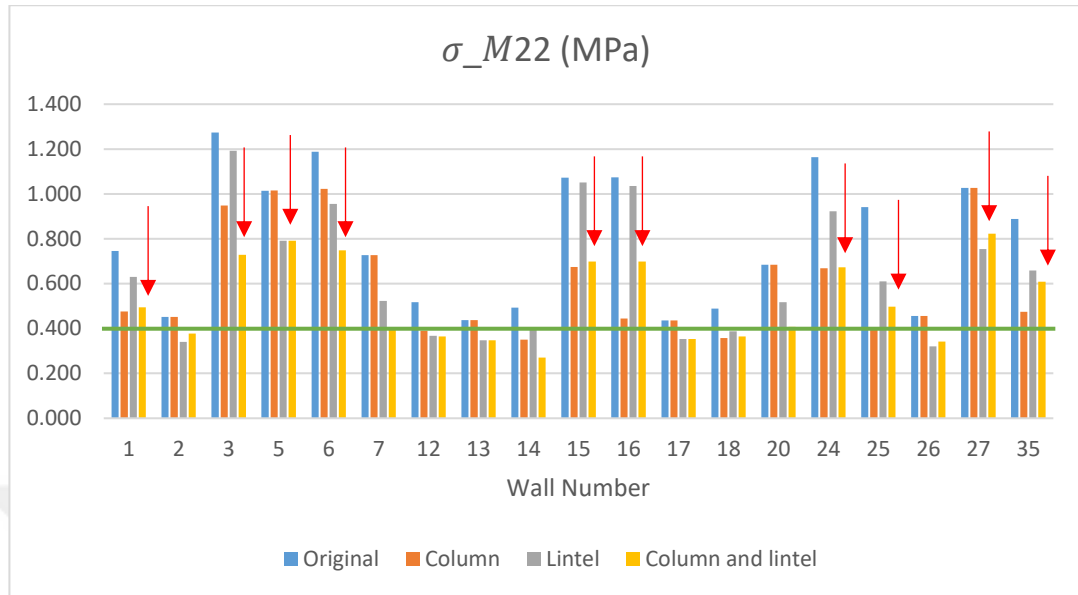


Figure 6.3. σ_{M22} comparison of each strengthening techniques at walls which are insufficient for house 1.

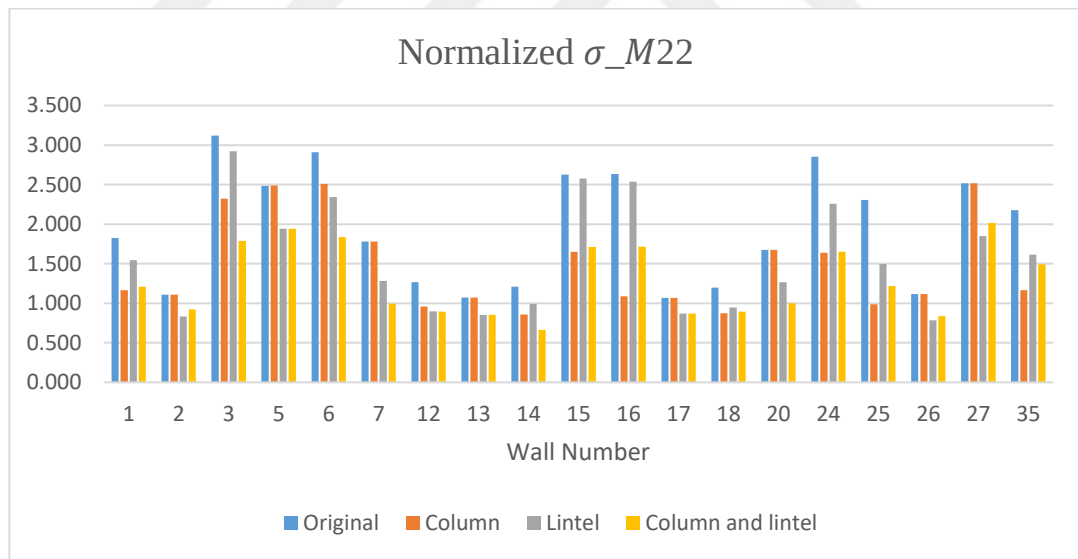


Figure 6.4. The normalized σ_{M22} as a ratio to the experimental stress / strength for each strengthening techniques for walls which are insufficient for house 1.

Table 36. Relative improvement of σ_{M22} for walls which are insufficient for house 1 using different strengthening techniques.

σ_{M22}				
WALL	ORIGINAL	COLUMN	LINTEL	COLUMN & LINTEL
1	1.826	1.167	1.544	1.211
2	1.108	1.108	0.833	0.924
3	3.120	2.324	2.922	1.787
5	2.487	2.488	1.941	1.941
6	2.912	2.507	2.344	1.836
7	1.781	1.782	1.282	0.995
12	1.268	0.958	0.900	0.892
13	1.072	1.071	0.853	0.853
14	1.210	0.858	0.990	0.663
15	2.628	1.652	2.576	1.712
16	2.634	1.088	2.537	1.714
17	1.069	1.069	0.868	0.868
18	1.197	0.875	0.949	0.892
20	1.676	1.676	1.267	0.998
24	2.853	1.640	2.260	1.650
25	2.306	0.988	1.495	1.218
26	1.118	1.118	0.784	0.838
27	2.518	2.517	1.848	2.017
35	2.179	1.164	1.615	1.493
AVG	1.945	1.476	1.569	1.290

In Table 36, the stress levels are normalized using the accepted flexural capacity of 0.408 MPa [2]. It can be concluded that additional column would improve σ_{M22} by 24.1%, additional lintel would improve σ_{M22} by 19.4% and combination of column + lintel would improve σ_{M22} by 33.7%.

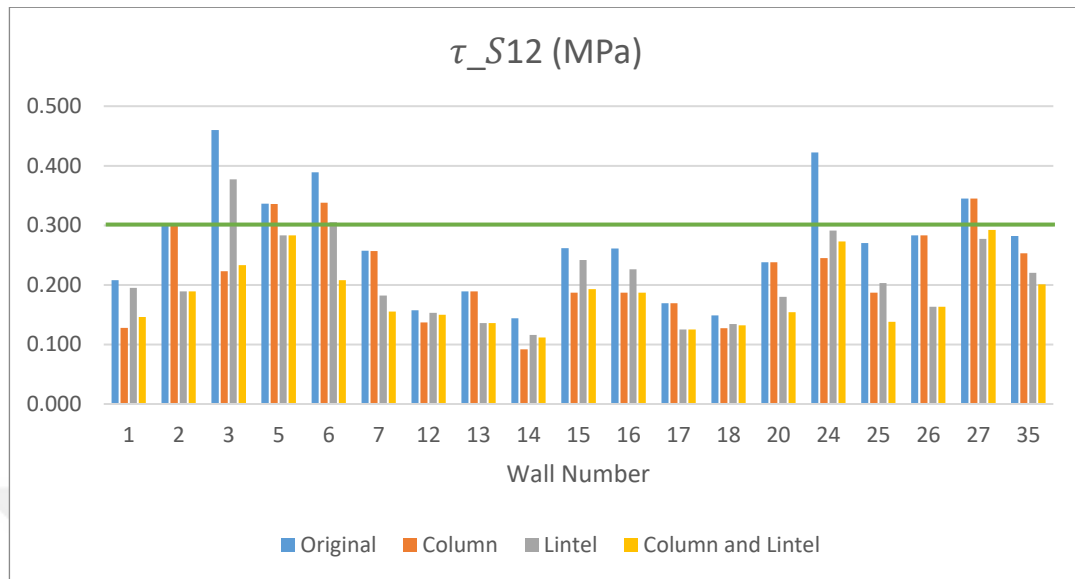


Figure 6.5. S12 comparison of each strengthening techniques at walls which are insufficient for house 1.

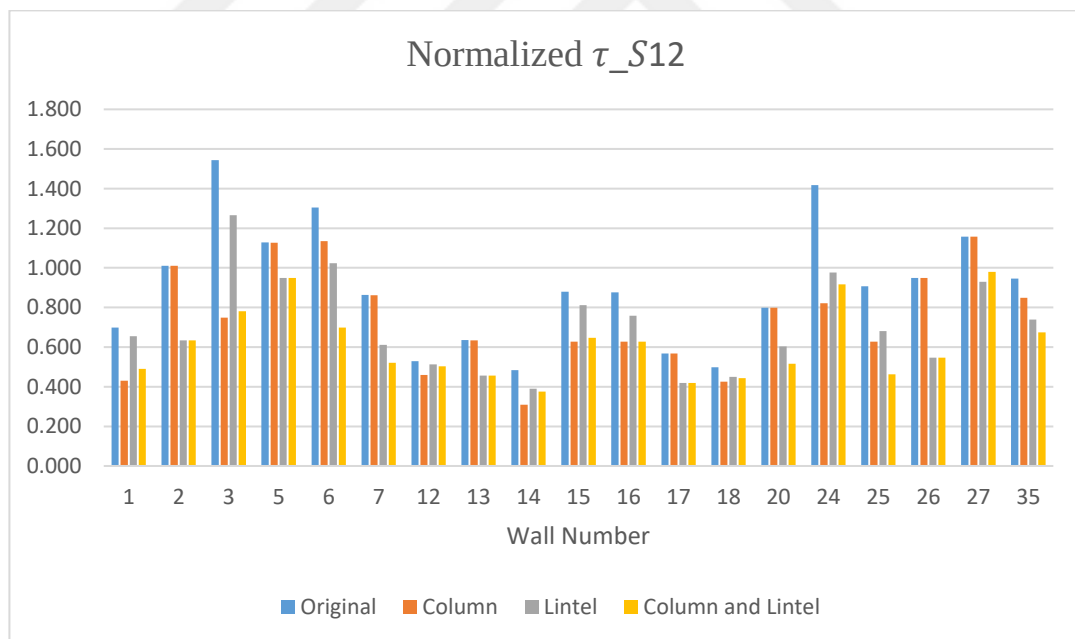


Figure 6.6. The normalized τ_{S12} as a ratio to the experimental stress / strength for each strengthening techniques for walls which are insufficient for house 1.

Table 37. Relative improvement of τ_{S12} for walls which are insufficient for house 1 using different strengthening techniques.

S12				
WALL	ORIGINAL	COLUMN	LINTEL	COLUMN & LINTEL
1	0.698	0.430	0.654	0.490
2	1.010	1.010	0.634	0.634
3	1.544	0.748	1.265	0.782
5	1.129	1.128	0.950	0.950
6	1.305	1.134	1.023	0.698
7	0.863	0.862	0.611	0.520
12	0.528	0.460	0.513	0.503
13	0.635	0.634	0.456	0.456
14	0.483	0.309	0.389	0.375
15	0.879	0.628	0.812	0.648
16	0.876	0.628	0.758	0.628
17	0.568	0.567	0.419	0.419
18	0.499	0.426	0.450	0.443
20	0.799	0.799	0.604	0.517
24	1.417	0.822	0.977	0.916
25	0.908	0.628	0.681	0.463
26	0.950	0.950	0.547	0.547
27	1.158	1.158	0.930	0.980
35	0.946	0.849	0.738	0.674
AVG	0.905	0.746	0.706	0.613

In Table 37, the stress levels are normalized using the accepted shear stress capacity of 298 kPa [2]. it can be concluded that additional column would improve τ_{S12} by

17.6%, additional lintel would improve τ_{s12} by 22% and combination of column + lintel would improve τ_{s12} by 32.3%.

6.1.2 House 2 Discussion of Results

At house 2 there are 17 walls and the unstrengthen model shows that 10 of them are considered safe for both shear failure and out of plane flexural failure. On the other hand, 7 walls failed due to either flexural failure at horizontal 1-1 axis (M22) or vertical 2-2 axis (M11) or both. To improve this, columns are added to 4 walls which do not have large openings such as window or door. From the additional columns only, wall 11 is considered sufficient; whereas, insufficient improvements are observed at walls 1.

In addition to that, additional lintels are also modelled without adding columns to obtain improvement. From additional lintels only, walls 4 and 13 are considered sufficient; whereas some improvements are observed at wall 1, 5, and 11. Furthermore, both lintels and columns are added to the third strengthened model to obtain walls' improvement. However, at wall 1 initially the column spacing was 2 m and after the addition of both column and lintel the wall still fails due to σ_{M11} . To improve it, additional columns with 1 m spacing are used. From additional lintels and columns, improvements are observed at wall 1 and 9.

Lastly, considering that addition of both columns and lintels do not improve σ_{M22} to sufficient strength at walls 1, 5, 9 and 15. These walls are strengthened further using post-tensioning utilizing the proper rebar diameter using the force obtained from Table 26.

The comparison of strength improvements at both flexural strengths and shear strength with different strengthening techniques are plotted in Figure 6.8, Figure 6.10, and Figure 6.12.

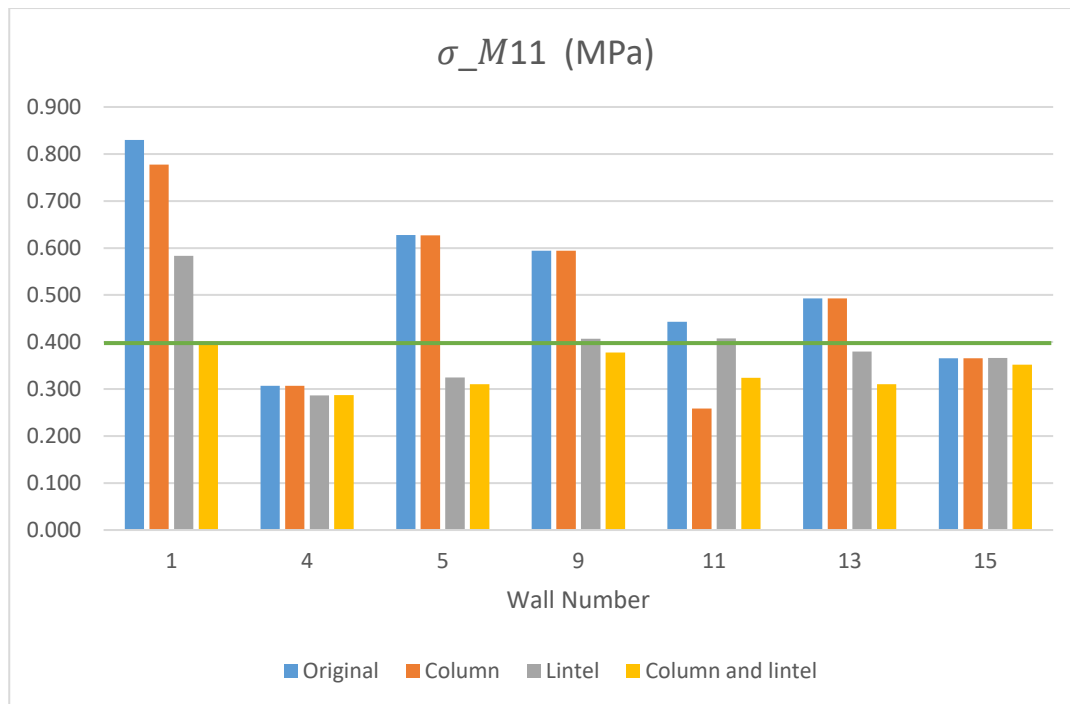


Figure 6.7. σ_{M11} comparison of each strengthening techniques at walls which are insufficient for house 2.

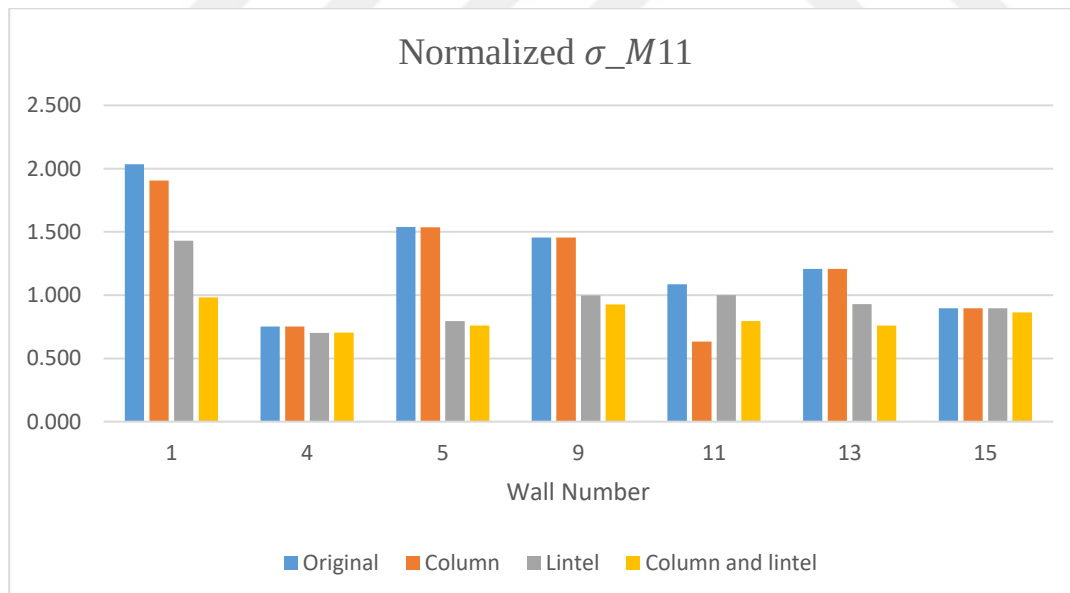


Figure 6.8. The normalized σ_{M11} as a ratio to the experimental stress / strength for each strengthening techniques for walls which are insufficient for house 2.

Some of the walls such as 4 and 15 are safe for M11 but listed in Figure 6.7 and Figure 6.8 because these walls have M22 based stresses exceeding the stress capacity.

Table 38. Relative improvement of σ_{M11} for walls which are insufficient for house 2 using different strengthening techniques.

σ_{M11}				
WALL	ORIGINAL	COLUMN	LINTEL	COLUMN & LINTEL
1	2.035	1.905	1.429	0.983
4	0.752	0.752	0.703	0.703
5	1.538	1.537	0.796	0.761
9	1.456	1.456	0.998	0.926
11	1.086	0.634	1.000	0.794
13	1.208	1.208	0.930	0.760
15	0.895	0.895	0.897	0.863
AVG	1.282	1.198	0.965	0.827

From Table 38, the stress levels are normalized using the accepted flexural capacity of 0.408 MPa [2]. It can be concluded that additional column would improve σ_{M11} by 6.5%, additional lintel would improve σ_{M11} by 24.7% and combination of column + lintel would improve σ_{M11} by 35.5%.

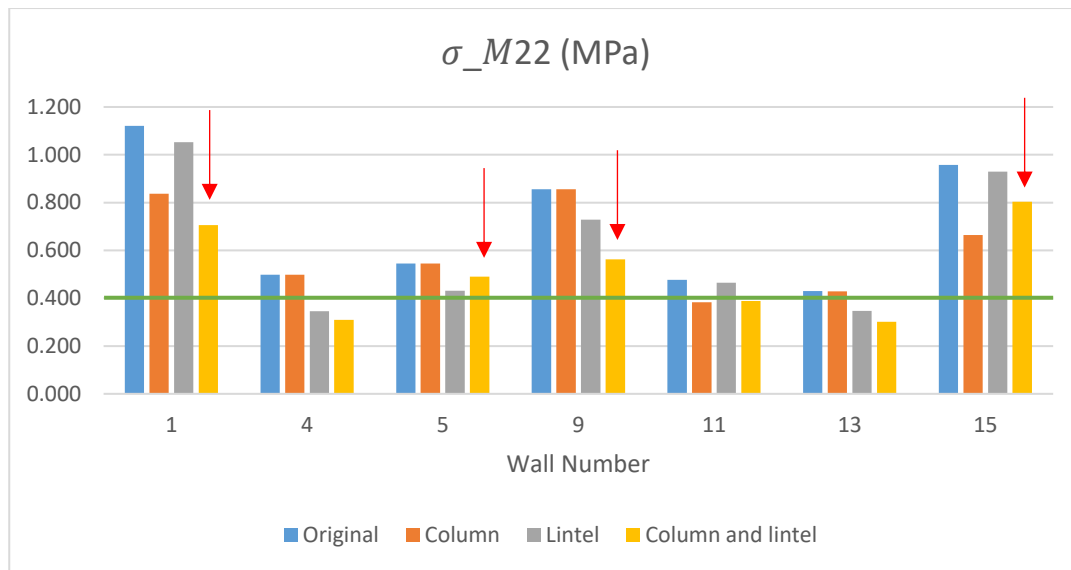


Figure 6.9. σ_{M22} comparison of each strengthening techniques at walls which are insufficient for house 2.

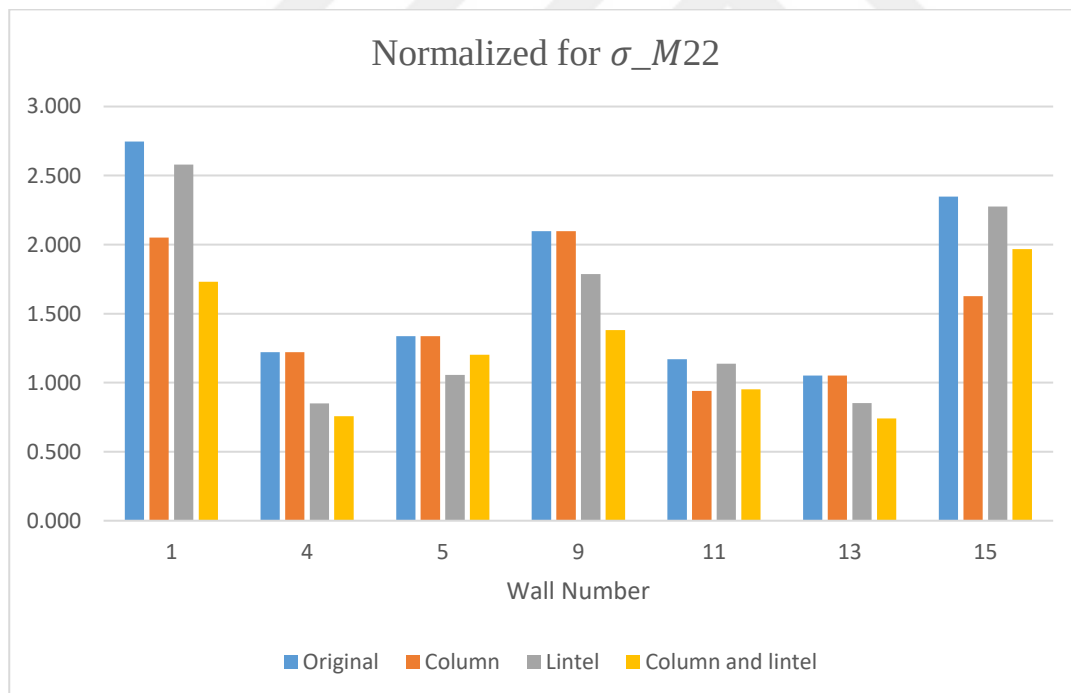


Figure 6.10. . The normalized σ_{M22} as a ratio to the experimental stress strength for each strengthening techniques for walls which are insufficient for house 2.

Table 39. Relative improvement of σ_{M22} for walls which are insufficient for house 2 using different strengthening techniques.

σ_{M22}				
WALL	ORIGINAL	COLUMN	LINTEL	COLUMN % LINTEL
1	2.748	2.050	2.580	1.730
4	1.221	1.221	0.849	0.757
5	1.336	1.336	1.056	1.202
9	2.097	2.098	1.786	1.380
11	1.169	0.941	1.138	0.951
13	1.052	1.051	0.852	0.740
15	2.347	1.628	2.277	1.969
AVG	1.710	1.475	1.505	1.247

From Table 39, the stress levels are normalized using the accepted flexural capacity of 0.408 MPa [2]. it can be concluded that additional column would improve σ_{M22} by 13.8%, additional lintel would improve σ_{M22} by 12% and combination of column + lintel would improve σ_{M22} by 27.1%.

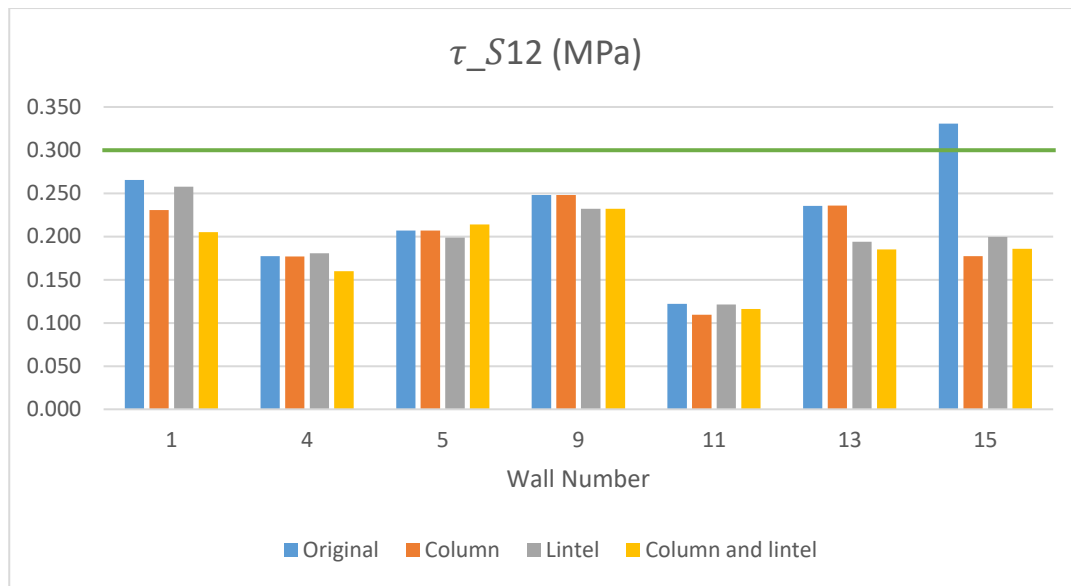


Figure 6.11. S12 comparison at each strengthening techniques for walls which are insufficient for house 2.

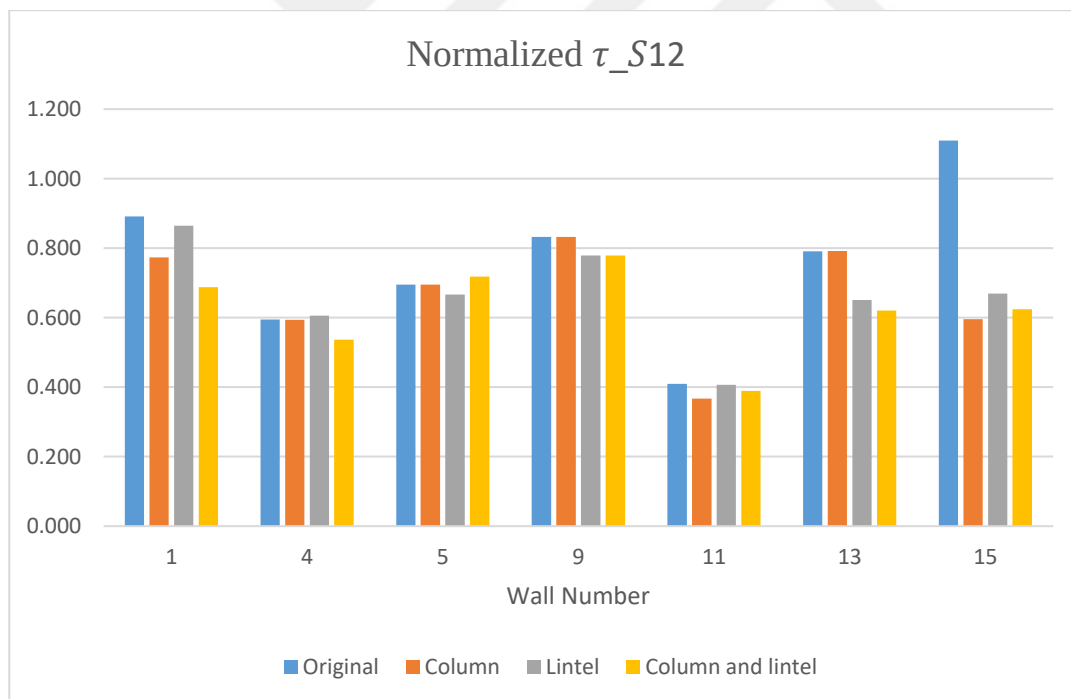


Figure 6.12. The normalized τ_{S12} as a ratio to the experimental stress strength for each strengthening techniques for walls which are insufficient for house 2.

Table 40. Relative improvement of τ_{s12} levels for walls which are insufficient for house 2 using different strengthening techniques.

τ_{s12}				
WALL	ORIGINAL	COLUMN	LINTEL	COLUMN & LINTEL
1	0.891	0.773	0.864	0.688
4	0.595	0.594	0.606	0.537
5	0.695	0.695	0.667	0.718
9	0.833	0.832	0.779	0.779
11	0.410	0.367	0.407	0.389
13	0.791	0.792	0.651	0.621
15	1.110	0.596	0.670	0.624
AVG	0.761	0.664	0.663	0.622

In Table 40, the stress levels are normalized using the accepted shear capacity of 298 kPa [2]. It can be concluded that additional column would improve τ_{s12} by 12.7%, additional lintel would improve τ_{s12} by 12.8% and combination of column + lintel would improve τ_{s12} by 18.2%.

6.1.3 House 3 Discussion of Results

At house 3 there are 43 walls and the unstrengthen model shows that 24 of them are considered safe for both shear failure and out of plane flexural failure. Moreover, 19 walls failed due to either flexural failure at horizontal 1-1 axis (M22) or vertical 2-2 axis (M11) or both. To improve this, initially columns are added to 15 walls which do not have large openings such as windows or doors. From the additional columns only, walls 1, 17, 38 and 39 are considered sufficient; whereas, insufficient improvements are observed at walls 5, 9, 10, 11, 12, 33, 34, 35, and 36.

In addition to that, additional lintels are also modeled without adding columns to obtain improvement. From additional lintels only, walls 23, 24, 26, 27, 33 and 34 are considered sufficient; whereas some improvements are observed at other walls which need to be further strengthened.

Furthermore, both lintels and columns are added to the model as the third strengthened model to obtain the additional improvement. However, at wall 5, 12, 35 and 36 initially the column spacing was 1.5 m and after the addition of both column and lintel the wall still fails due to σ_{M11} , to improve it, additional column with 1 m spacing is used. Similarly, at wall 9, initially the column spacing was 2 m and after the addition of both column and lintel the wall still fails due to σ_{M11} , to improve it, additional column with 1 m spacing is used and the result is tabulated in Table 29. From additional lintels and columns, walls 11, 12, 38, and 39 are considered sufficient. Whereas improvements are observed on other walls which need to be further strengthened.

Lastly, considering that addition of both columns and lintels do not improve σ_{M22} to sufficient strength at walls 5, 9, 10, 21, 22, 35 and 36. These walls are strengthened further using post-tensioning utilizing the proper rebar diameter and using the force obtained from Table 30.

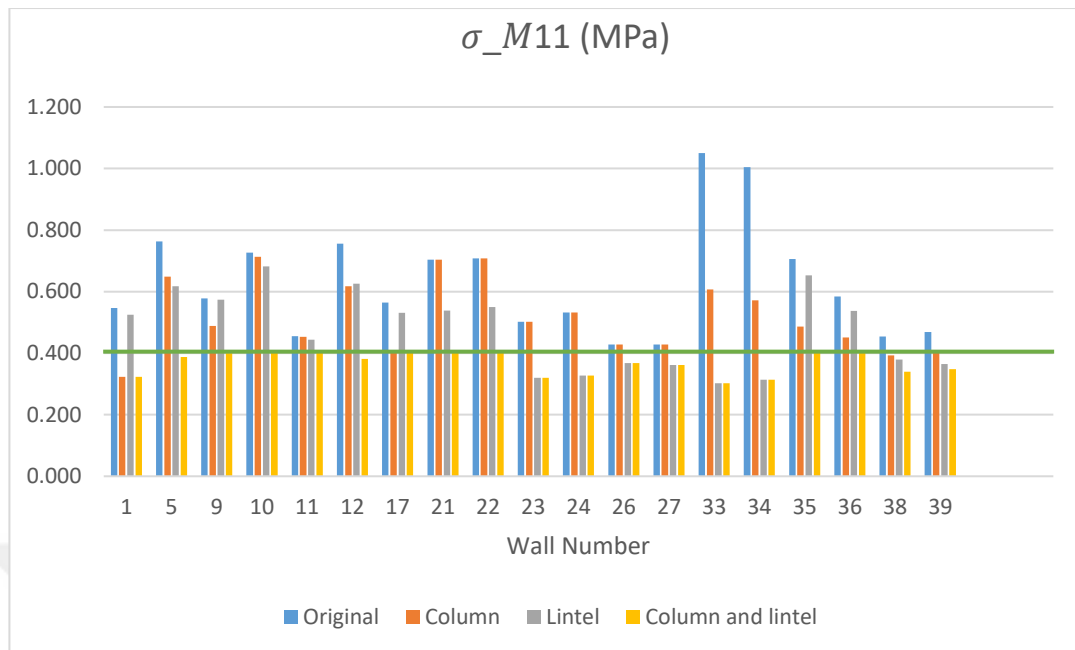


Figure 6.13. σ_{M11} comparison of each strengthening techniques at walls which are insufficient for house 3.

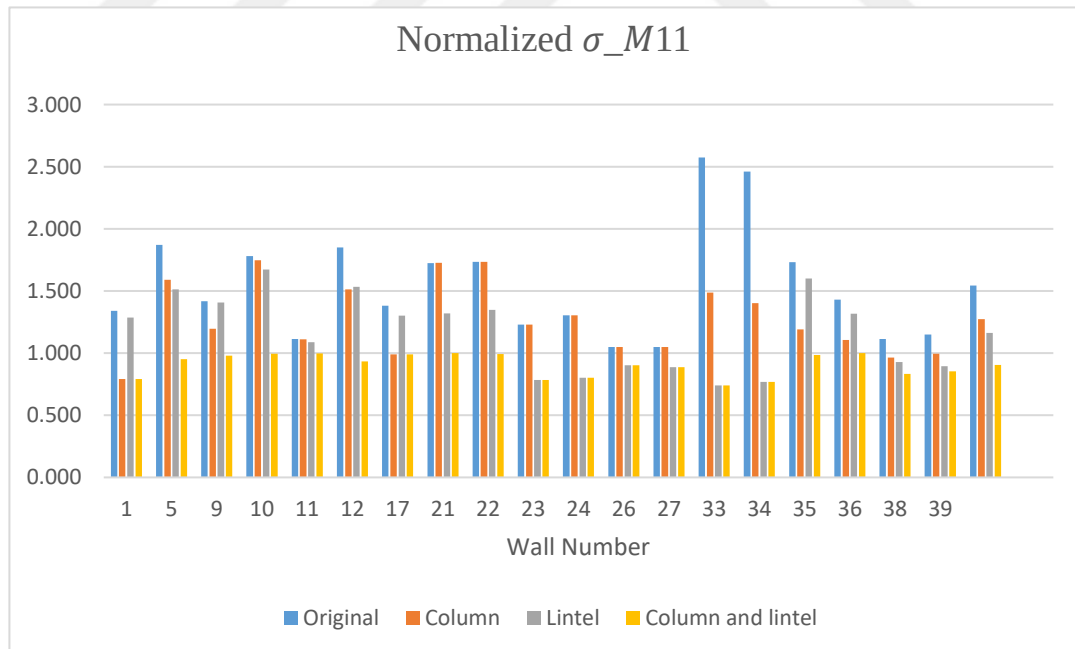


Figure 6.14. The normalized σ_{M11} as a ratio to the experimental stress strength for each strengthening techniques for walls which are insufficient for house 3.

Table 41. Relative improvement of σ_{M11} levels for walls which are insufficient for house 3 using different strengthening techniques.

σ_{M11}				
WALL	ORIGINAL	COLUMN	LINTEL	COLUMN & LINTEL
1	1.341	0.792	1.286	0.792
5	1.870	1.591	1.512	0.951
9	1.418	1.196	1.407	0.980
10	1.781	1.748	1.672	0.995
11	1.115	1.110	1.088	0.998
12	1.851	1.512	1.534	0.934
17	1.382	0.990	1.301	0.990
21	1.725	1.725	1.319	1.000
22	1.735	1.735	1.348	0.993
23	1.229	1.230	0.784	0.784
24	1.304	1.304	0.801	0.801
26	1.049	1.049	0.902	0.902
27	1.049	1.049	0.887	0.887
33	2.575	1.488	0.740	0.740
34	2.462	1.402	0.770	0.770
35	1.731	1.191	1.600	0.985
36	1.431	1.105	1.316	1.000
38	1.113	0.963	0.929	0.832
39	1.149	0.995	0.895	0.854
AVG	1.543	1.272	1.163	0.905

From Table 41, the stress levels are normalized using the accepted flexural capacity of 0.408 MPa [2]. It can be concluded that additional column would improve σ_{M11} by 17.5%, additional lintel would improve σ_{M11} by 24.6%, and combination of column + lintel would improve σ_{M11} by 41.4%.

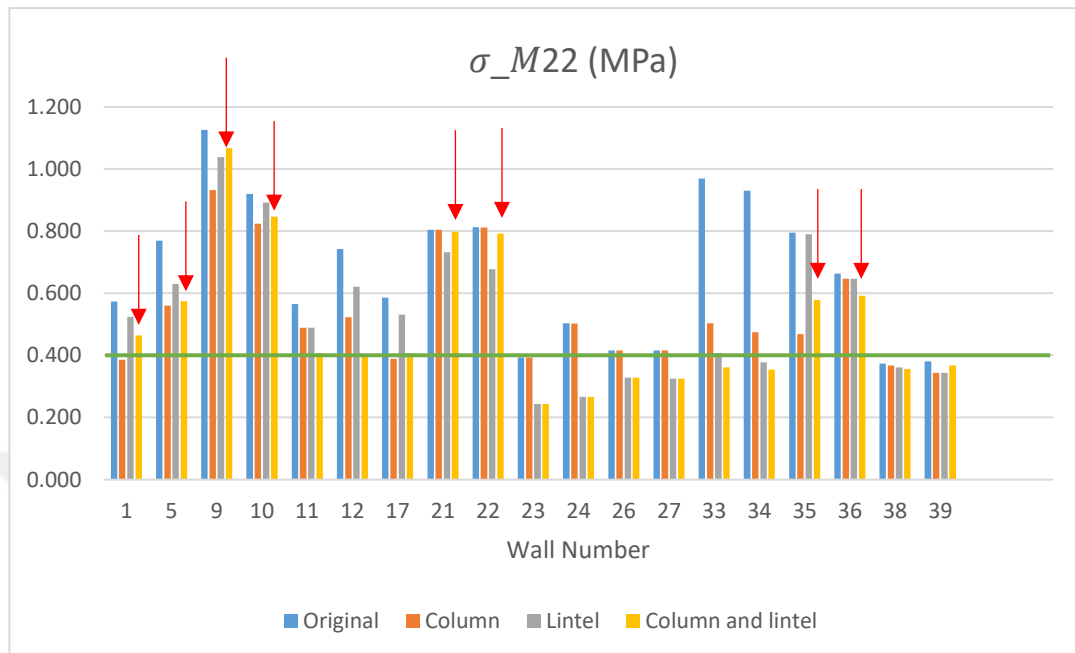


Figure 6.15. σ_{M22} comparison of each strengthening techniques at walls which are insufficient for house 3.

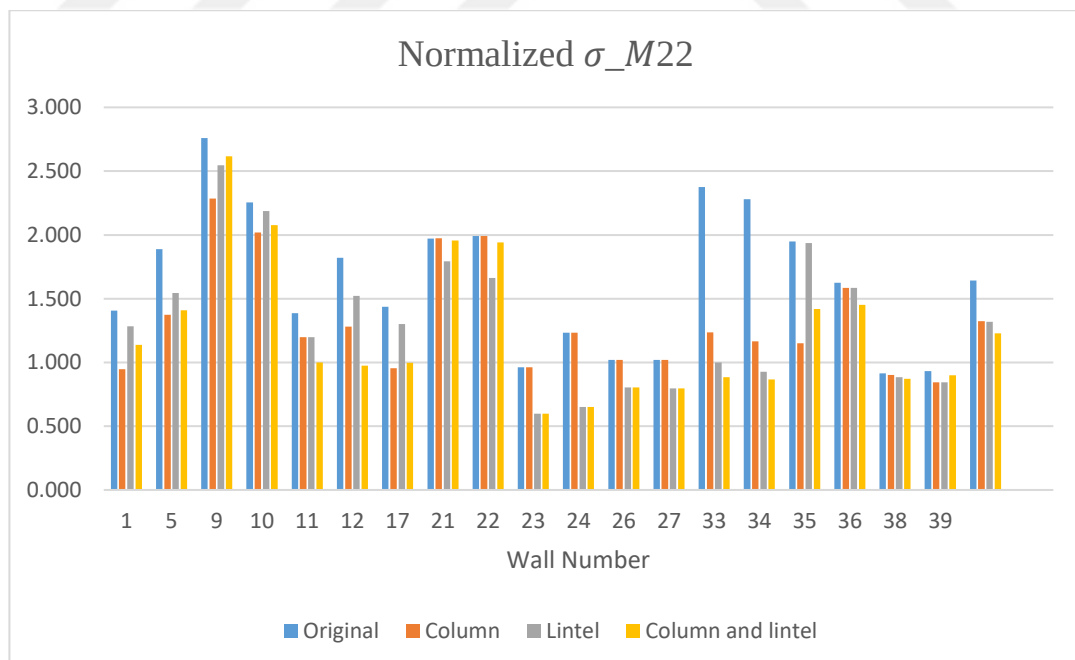


Figure 6.16. The normalized σ_{M22} as a ratio to the experimental stress strength for each strengthening techniques for walls which are insufficient for house 3.

Table 42. Relative improvement of σ_{M22} levels for walls which are insufficient for house 3 using different strengthening techniques.

σ_{M22}				
WALL	ORIGINAL	COLUMN	LINTEL	COLUMN & LINTEL
1	1.407	0.946	1.284	1.137
5	1.887	1.373	1.544	1.409
9	2.760	2.284	2.547	2.615
10	2.256	2.020	2.186	2.076
11	1.385	1.199	1.199	1.000
12	1.821	1.282	1.522	0.974
17	1.436	0.953	1.301	0.998
21	1.972	1.973	1.794	1.956
22	1.991	1.990	1.662	1.941
23	0.963	0.963	0.598	0.598
24	1.234	1.233	0.652	0.652
26	1.020	1.020	0.804	0.804
27	1.020	1.020	0.797	0.797
33	2.376	1.235	1.000	0.885
34	2.279	1.164	0.926	0.868
35	1.949	1.150	1.936	1.419
36	1.625	1.586	1.586	1.451
38	0.915	0.902	0.885	0.873
39	0.932	0.843	0.843	0.900
AVG	1.644	1.323	1.319	1.229

From Table 42, the stress levels are normalized using the accepted flexural capacity of 0.408 MPa [2]. It can be concluded that additional column would improve by σ_{M22} 19.5%, additional lintel would improve σ_{M22} by 19.7% and combination of column + lintel would improve σ_{M22} by 25.2%.

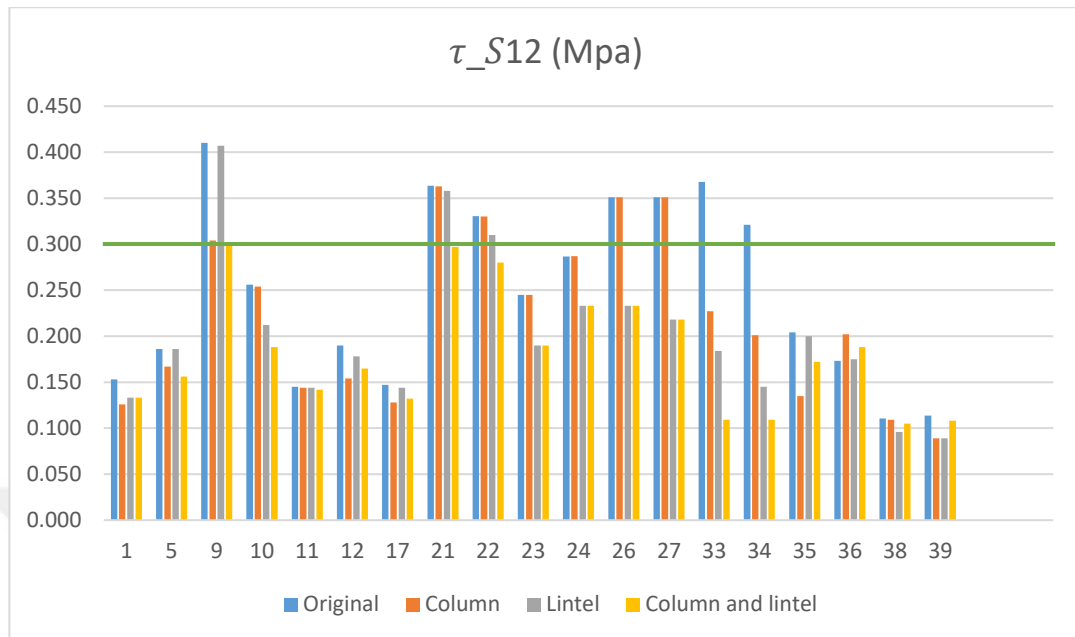


Figure 6.17. S12 comparison of each strengthening techniques at walls which are insufficient for house 3.

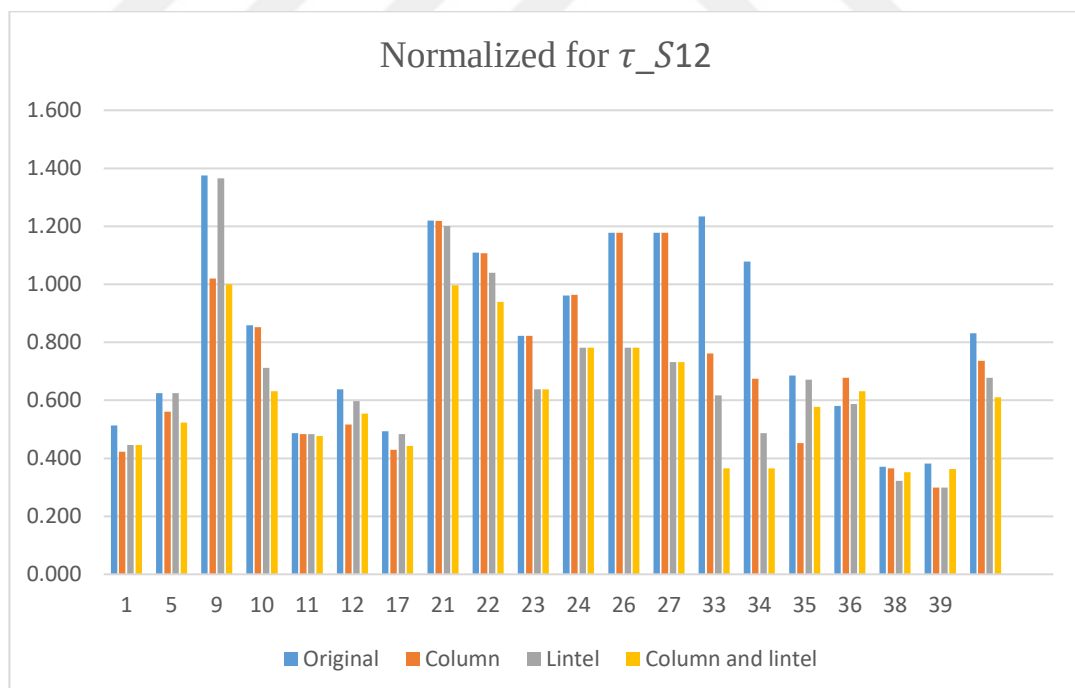


Figure 6.18. The normalized τ_{S12} as a ratio to the experimental stress strength for each strengthening techniques for walls which are insufficient for house 3.

Table 43. Relative improvement of τ_{S12} levels for walls which are insufficient for house 3 using different strengthening techniques.

WALL	τ_{S12}			
	ORIGINAL	COLUMN	LINTEL	COLUMN & LINTEL
1	0.513	0.423	0.446	0.446
5	0.624	0.560	0.624	0.523
9	1.376	1.020	1.366	1.000
10	0.859	0.852	0.711	0.631
11	0.487	0.483	0.483	0.476
12	0.638	0.517	0.597	0.554
17	0.493	0.430	0.483	0.443
21	1.220	1.218	1.201	0.997
22	1.109	1.107	1.040	0.940
23	0.822	0.822	0.638	0.638
24	0.962	0.963	0.782	0.782
26	1.178	1.178	0.782	0.782
27	1.178	1.178	0.732	0.732
33	1.234	0.762	0.617	0.366
34	1.078	0.674	0.487	0.366
35	0.686	0.453	0.671	0.577
36	0.581	0.678	0.587	0.631
38	0.371	0.366	0.322	0.352
39	0.382	0.299	0.299	0.363
AVG	0.831	0.736	0.677	0.610

From Table 43, the stress levels are normalized using the accepted shear capacity of 298 kPa [2]. It can be concluded that additional column would improve τ_{S12} by 11.4%, additional lintel would improve τ_{S12} by 18.5% and combination of column + lintel would improve S12 by 26.5%.

6.2 General Recommendations for Non-engineered Masonry Houses to improve EQ performance.

From recommendations and requirements for masonry house manual according to Earthquake Engineering Research Institute (EERI) [29] and retrofitting simple building damaged by earthquake manual by Prof. T. Boen [6] some general recommendations can be summarized for non-engineered masonry houses as follows:

1. Houses should have symmetrical shape in its overall appearance.
2. Full bricks or bricks with holes' area less than 50% of the total area of the bricks should be used.
3. The Minimum recommended width of brick is 11 cm and if the width of brick is less than 12.5 cm, double-layered bricks should be used.
4. The maximum distance between tie columns is 4.5 m and the maximum recommended wall height is 3 m.
5. The minimum longitudinal reinforcement for tie columns and beams is 4 ϕ 10 mm but 4 ϕ 12 mm is preferred.
6. The minimum required stirrup's diameter is 6 mm with 150 mm spacing but 8 mm with 150 mm spacing is preferred. In addition to that, it is recommended to have 100 mm spacing at h/6 distance from the connections between column and beam at both ends.
7. At the connection joints between columns and beams, it is recommended to have additional confinements and rebar extension with length of either 50 * rebar diameter in mm [29] or 40 * rebar diameter in mm [6] as shown in Figure 5.2, Figure 5.3, Figure 5.4, and Figure 5.5. This prevents connection failure and improves structural integrity during earthquakes.
8. It is recommended to provide wall anchoring with 10 mm diameter rebar and minimum length of 40 cm at every 6 layers of bricks (Figure 5.10).

6.3 Conclusions

This thesis investigates the strengthening of three typical Indonesian non-engineered confined masonry house designs using four different techniques (adding columns, adding lintels, and adding both lintels and columns and post tensioning). Moreover, mesh sensitivity analyses were conducted to determine the most appropriate mesh sizes for each house. Each house evaluated using the correct mesh sizes; the maximum displacements at both x and y axes were checked according to ASCE 7-16, Euro code 8, and Turkish earthquake code (TBDY 2018) as well as wall bending stress comparisons against available experimental results. Additionally, analytical calculations using yield line theory were also conducted to determine failure load to initiate cracking. From the comparison between SAP2000 analysis result and yield line theory calculation given in Table 34, it can be concluded that simplified hand calculations give relatively similar result to determine which wall might crack during a possible earthquake. Thus, yield line theory can be used as preliminary calculation when determining wall failures. A total of 12 FE models were developed in SAP2000 and the summary of failing walls compare to total amount of walls at each strengthening technique is given in Table 44.

Table 44. Summary of Failing Walls for Modeled Houses.

	House 1	House 2	House 3
Original	19/37	7/17	19/43
Add. Column	16/37	6/17	15/43
Add. Lintel	12/37	5/17	13/43
Column+lintel	10/37	4/17	7/43
Column+lintel+post tensioning	0/37	0/17	0/43

the analysis results are concluded as follows:

1. Additional columns improve σ_{M11} by 11.2 %, σ_{M22} by 19.1% and τ_{S12} by 13.9%.
2. Additional lintels improve σ_{M11} by 25.3%, σ_{M22} by 17% and τ_{S12} by 17.8%.
3. Additional columns + lintels improve σ_{M11} by 37.9%, σ_{M22} by 28.7% and τ_{S12} by 25.7%.
4. Post tensioning applications improve σ_{M22} by 38.3 %.

From the analysis result, combination of additional columns and lintels gives the highest improvement for both flexural and shear strength. Furthermore, post-tensioning using U-channels and tension rebars at 0.5 m spacing (applied using simple and low cost conventional tools) will also improve σ_{M22} bending capacity of the walls and the improvement percentage for contribution of each strengthening techniques to the flexural strength and shear strength of the non-engineered confined masonry houses are given in Figure 6.19, Figure 6.20, and Figure 6.21. If the walls can be strengthened using columns and lintels, post-tensioning application is deemed unnecessary and left out of the strengthening scope.

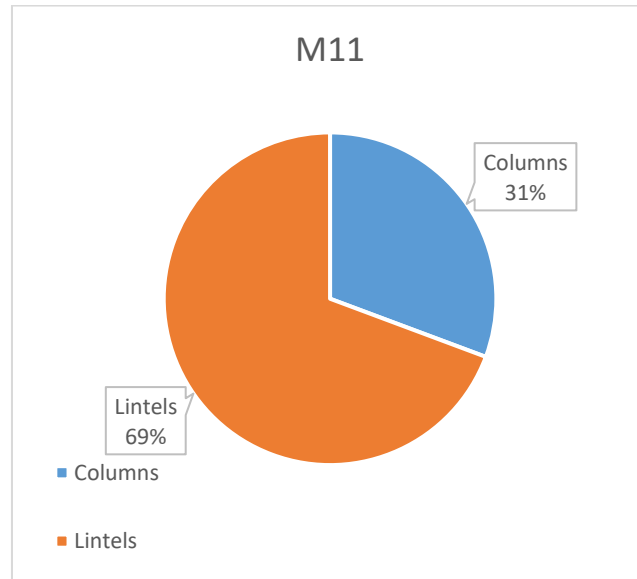


Figure 6.19. Improvement percentage for σ_{M11} .

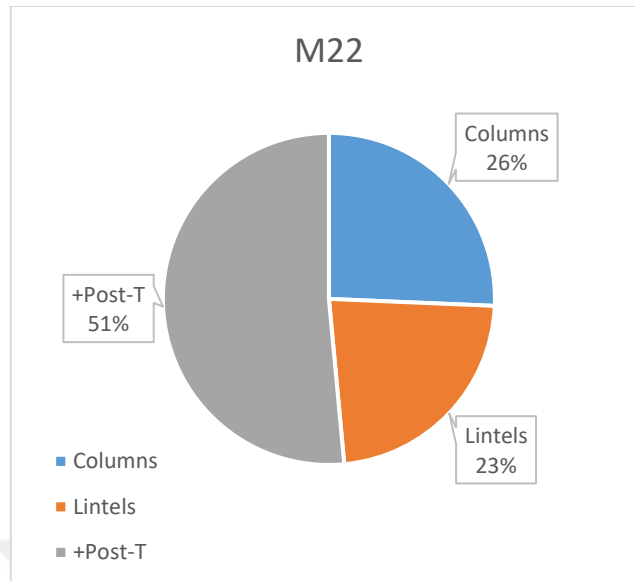


Figure 6.20. Improvement percentage for σ_{M22} .

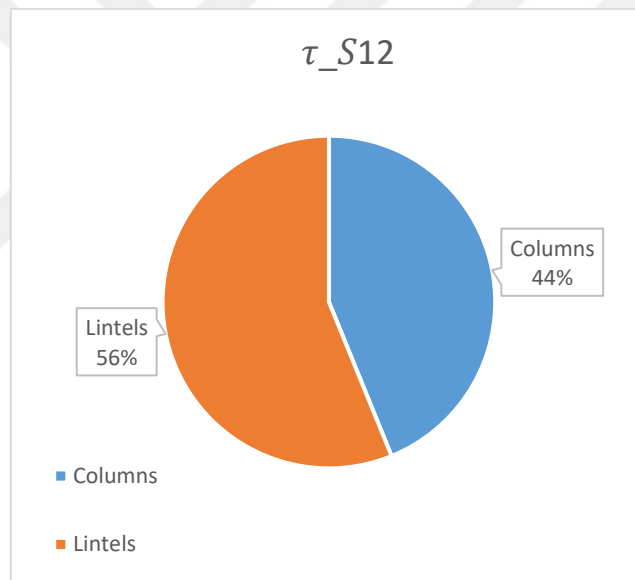


Figure 6.21. Improvement percentage for τ_{S12} .

From Figure 6.19, Figure 6.20, and Figure 6.21 it can be concluded that addition of lintels contributes with higher overall improvement percentage compare to addition of columns only. Thus, addition of lintels is a significant strengthening technique especially when combined with columns. Post-tensioning is shown to be a very effective technique in out-of-plane bending especially when lintel + column addition could not achieve the required improvement level; without using complex assembling techniques. Moreover, by considering the minimum values and

recommendations summarized in section 6.2, it is also possible to improve the overall capacity of the structure even further. Combining recommendations with strengthening techniques presented in this study, the typical non-engineered confined masonry houses in Indonesia will be much stronger and safer for people to live in, considering the earthquake loading expected in Indonesia.

6.4 Future Studies

Future studies in this topic which can increase the accuracy and effectivity of the strengthening techniques are as follows:

- Create experimental setup of the full-scale or smaller scale of non-engineered houses and test it on shaking table.
- Consider the economic aspect of the strengthening techniques.
- Changing materials for lintel and finding the most effective materials.
- Create micro modeling of the model both for solid and shell models.



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