SEISMIC PERFORMANCE ASSESSMENT OF CONFINED MASONRY BUILDINGS

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ABSTRACT

SEISMIC PERFORMANCE ASSESSMENT OF CONFINED MASONRY BUILDINGS

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Although it is a known fact that confined masonry construction has many advantages over unreinforced masonry construction, its application has been very limited in Turkish construction practice. One of the reasons for this issue is that the seismic regulations in Turkey do not explicitly enforce or even encourage the use of confined masonry structural systems. Now, being on the verge of releasing a new version of the Turkish seismic code, it seems to be an appropriate time to adapt confined masonry to the Turkish construction practice. This study intents to reveal why such attempts are necessary by comparing the behaviour of unreinforced and confined masonry on the building scale.

In the first phase of the study, standards for unreinforced masonry and confined masonry buildings around the world are given. Studies regarding unreinforced and confined masonry walls are summarized. Idealized tri-linear capacity curves are used to define the wall capacities and pushover analysis is performed to find the storey shear capacities of the critical storey of each model following the theory of Tomazevic. 6 different floor plans are designed as unreinforced masonry and confined masonry buildings with changing geometrical and mechanical properties. Demand is represented by ten real ground motion records of changing PGA values. Capacity spectrum method of FEMA 440 is used to carry out the performance analysis of buildings. The results of the performance analysis reveal the superior behaviour of confined masonry buildings over unreinforced masonry buildings against the seismic action.

Keywords: confined masonry; unreinforced masonry; masonry wall; masonry building; seismic performance; damage index.

ÖZ

KUŞATILMIŞ YIĞMA BİNALARIN PERFORMANS DEĞERLENDİRMESİ

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Kuşatılmış yığma yapıların donatısız yığma yapılara oranla birçok avantajının olduğu bilinen bir gerçek olmasına rağmen kuşatılmış yığma yapılar Türk yapı uygulamasında hiçbir zaman popüler olmamıştır. Bunun nedenlerinden biri de Türk deprem yönetmeliklerinin kuşatılmış yığma duvarlı bina yapımına teşvik etmemesi ve dayatmamasıdır. Şimdi, yeni bir Türk deprem yönetmeliğinin oluşturulması aşamasında kuşatılmış yığma yapı uygulamasının Türk yapım uygulamasına intibak ettirilmesi için uygun zaman gibi görünmektedir. Bu çalışma böyle girişimlerin neden gerekli olduğunu donatısız ve kuşatılmış yığma yapıların davranışlarını karşılaştırarak ortaya çıkarmayı hedeflemektedir.

Çalışmanın ilk aşamasında, dünya genelinde kuşatılmış ve donatısız yığma yapılara ait yapım standartları ile ilgili titiz bir kaynak araştırması yürütülmüştür. Kuşatılmış ve donatısız yığma duvarlarla ilgili çalışmaları özetlenmiştir. Tomazevic'in teorisi doğrultusunda üç performans noktalı idealleştirilmiş üç doğrulu kapasite eğrileri kullanılarak duvar kapasiteleri tanımlanmış ve artımsal itme analizi yöntemi ile toplam kat kayma dayanımları belirlenmiştir. 6 farklı kat planı değişen mekanik ve geometrik özelliklerde kuşatılmış ve donatısız yığma yapı olarak tasarlanmıştır. Binalara gelen yatay yükler, 10 gerçek deprem kaydında yararlanılarak elde edilmiştir. FEMA 440'ta geçen kapasite spektrum metodu kullanılarak performans analizi gerçekleştirilmiştir. Performans analiz sonuçları deprem hareketleri karşısında kuşatılmış yığma yapıların donatısız yığma yapılara olan üstünlüğünü açıkça ortaya koymuştur.

Anahtar Kelimeler: kuşatılmış yığma; donatısız yığma; yığma duvar; yığma bina; sismik performans; hasar indisi.

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a: Ground acceleration

A: Gross floor area

A_m : Gross area of the section

A_w : Horizontal cross section area of wall

AI: Arias Intensity

B : Reduction factor for effective damping

b : Shear stress distribution factor

C_{cr} : Coefficient defined by Tomazevic

C_i : Interaction coefficient

D: Distance between centroid of steel under tension and the fiber under maximum compression

d_i : Displacement in the i'th iteration of pushover analysis.

d_r : Diameter of the tie-column longitudinal bar

E_i : Input energy per unit mass for a 5% damped SDOF system

EI: Energy index

EPA: Effective peak acceleration

e': Eccentricity of the loading

F_{AE} : Factor that depends on the length to height ratio

F_E : Slenderness and eccentricity reduction factor

F_R : Strength reduction factor

f_b: Masonry unit compressive strength

f_c : Compressive strength of concrete

f_d: Design masonry design compressive strength of masonry

f_m : Compressive strength of masonry

f_t : Tensile strength of masonry

f_{vko}: Initial shear strength

f_y : Yield strength of reinforcement bar

H : Effective height of the wall

HI: Housner spectrum intensity

h: Greater clear height of the openings adjacent to the wall

h_{eff}: Effective height of the wall

I: Building importance factor

I_d : Damage index defined by Tomazevic

J : Rotational moment of inertia of the storey

k: Wall effective height factor

K_d : Secant stiffness

K_e : Elastic stiffness

k_x : In-plane stiffness values of wall segments in x direction

k_y : In-plane stiffness values of wall segments in y direction

L : Total length of the floor

l: Length of the wall

L_d: Total length of masonry walls in one of the principal directions

m: Meter

mm: Millimeter

MPa: Mega Pascal

M_{ru} : Flexural resistance moment of the wall

M_0 : Pure bending strength of the wall

N : Number of floors of the building

P: Vertical load acting on the wall

PF: Modal participation factor

PGA: Peak ground acceleration

PGV: Peak ground velocity

PI : Performance index

PS_v : 5% damped pseudo spectral velocity

P_R : Maximum axial load that a wall can resist

P_u : Design axial load on the wall

S_a : Spectral acceleration

S_d : Spectral displacement

t: Wall thickness

t_d : Total record duration

T: Period

T_{eff} : Effective period value

$t_{ef,min}$: Minimum thickness for the walls

T_0 : Initial period value

T_{sec} : Secant period value

TV_x : Total shear force capacity of the critical storey in x direction

TV_y : Total shear force capacity of the critical storey in y direction

V : Base shear force

V_e : Energy equivalent velocity

V/A: Velocity to acceleration ratio

V_c : Shear force resisted by concrete

$V_{d,r}$: Shear resistance contribution of the reinforcement in tie-columns of confined masonry

V_{dt} : Diagonal tension strength

V_{fl} : Flexure strength

v_m^*: Design diagonal compressive strength of masonry

V_{mR} : Diagonal shear strength of the wall

V_p : Shear force that must be resisted by the column in the critical zone

V_{ss} : Sliding shear strength

$V_{u,s}$: Shear resistance contribution of the wall in confined masonry

V_w : Vertical load on the wall

W : Total weight (dead+live loads) of the structure

w_i : Weighing factors

Φ: Main bar diameter

β_0 : Initial damping value

Δ_{roof} : Roof level displacement

Δt_{eff} : Effective duration

$\varphi_{i,roof}$: Amplitude of mode 1 in the roof level

μ : Ductility

$\propto_1$: Modal mass coefficient for the first natural mode

τ_{em} : Allowable shear stress of the wall

τ_0 : Allowable shear stress under zero compression

τ_{em} : Contribution of compressive stress on the wall due to frictional resistance

μ: Friction coefficient

λ : Aspect ratio

α : Parameter depending on the assumed shape and distribution of interaction forces

θ : Rotation angle due to displacement d_i

$Б_d$: Compressive stress on the wall

δ_{max}: Displacement value at the maximum strength

δ_{ult} : Displacement value corresponding to ultimate strength

σ_y : Vertical stress on the wall

f_{vo} : Shear bond strength at zero compression

μ : Coefficient of friction between masonry unit and mortar

α : Constraint parameter

τ_m : Shear strength under zero compression

σ_0 : Average compressive stress on the wall

φ_e : Reduction factor for slenderness

CHAPTER 1

INTRODUCTION

1.1. Masonry Construction in General

Masonry is the oldest construction method for the mankind. First masonry unit production dates back to 6000 years. Sun baked clay bricks involved chopped straw and grass to prevent cracking and distortion. Then people started to use firing instead of sun-baking for production. This improved the durability of bricks. Today's clay brick is composed of clay and shale and it is baked in kilns up to 1100°C. High temperatures enable the clay particles to bond chemically.

Masonry structures can withstand the disasters and environmental factors for hundreds or even thousands of years. Most important examples are the Egyptian Pyramids, the Colosseum in Rome, India's Taj Mahal, the Great Wall of China (Figure 1.1). There are many materials and methods that can be used in masonry construction. Bricks may be made of different materials like stone, clay, concrete, aerated concrete etc. Construction of masonry can be executed with or without mortar. Walls can be single or double leaf walls. Construction method may be unreinforced, confined or reinforced.

Figure 1.1. Eyptian Pyramids constructed between 2630-2611 BC

1.2. Types of Masonry Constructions

Main types of masonry constructions can be listed as unreinforced masonry (URM), confined masonry (CM) and reinforced masonry (RM). These types of masonry constructions are explained briefly in the following sections.

1.2.1. Unreinforced Masonry (URM)

In this type of masonry construction, there are no vertical structural members (i.e. columns) around the load-bearing walls. The walls are not reinforced, but some construction codes may enforce the use of steel ties or reinforcement in the wall connections. Floors can be wooden or reinforced concrete. Tie beams should be used around the reinforced concrete slab according to Turkish Earthquake Code (2007), abbreviated as TEC-07, and some other codes.

This type of construction should be preferred only in regions of low-seismicity and for buildings up to 2-3 stories. Algerian, Peruvian and Chilean seismic codes do not even permit the URM construction. URM walls are constructed by binding masonry units with mortar in general. The wall strength depends on the mechanical properties of masonry units and the mortar used. They can carry vertical loads according to the material and construction quality but they are more vulnerable against lateral loads due to their heterogeneous nature. A typical URM building is shown in Figure 1.2.

Figure 1.2. Unreinforced masonry building under construction

1.2.2. Confined Masonry (CM)

CM construction looks like a reinforced concrete frame construction, because it involves reinforced concrete columns and beams in an integrated manner. But the load carrying members are not the columns and beams, but the masonry walls just like the URM construction. The purpose of using columns and beams in this construction method is to prevent the disintegration of load-bearing walls and increase the stability against lateral loads and out of plane action. Accordingly, they are known as tie beams and tie columns.

The construction method is also different than a reinforced concrete frame construction. In the reinforced concrete frame construction columns and beams are cast in first place, non-structural masonry walls are built afterwards. In CM construction, load-bearing masonry walls are built in the first place, and then the non-structural tie-columns are cast. It is important to make a toothed connection between the masonry wall and tie-column. (Figure 1.3)

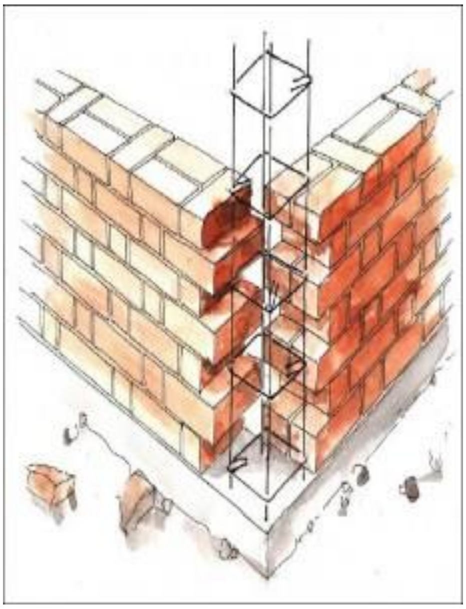


Figure 1.3. CM construction technique. (http://www.world-housing.net)

CM technique is used in countries having high or medium seismic risk as Chile, Peru, Argentina, Slovenia, Mexico and Iran. Low or medium height confined masonry buildings showed very good performance during major earthquakes in Chile. Structural components of a typical two storey confined masonry building is illustrated in Figure 1.4.

1.2.3. Reinforced Masonry (RM)

In RM construction, steel reinforcement or prestressing methods are used to strengthen the masonry walls, piers and beams against flexure and shear. Most common application of RM construction is to place reinforcement bars inside the hollow masonry units and fill it with grout (Figure 1.5.a). Reinforcement may also be embedded in the horizontal mortar joints. (Figure 1.5.b).

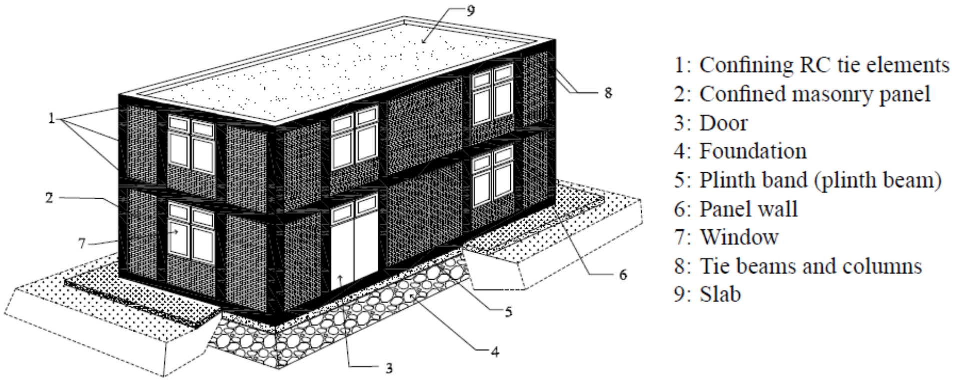


Figure 1.4. Components of a typical two storey CM building (Brzev, 2007).

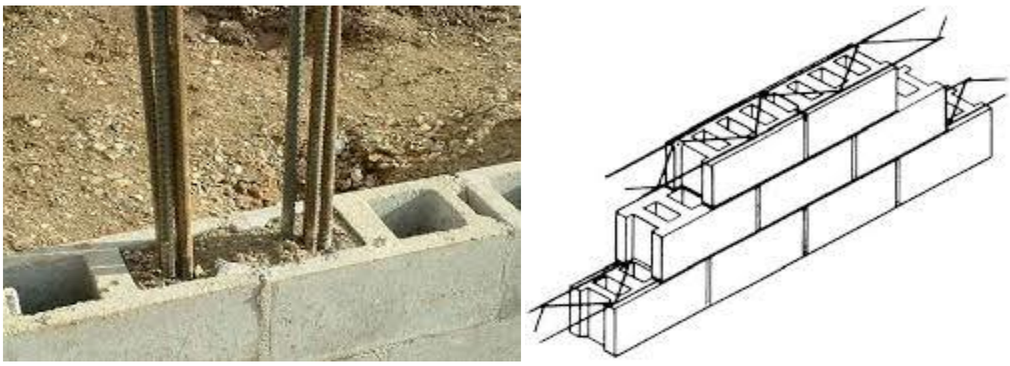


Figure 1.5. Placement of reinforcement in masonry **a)** vertically inside the hollow units, **b)** horizontally in the mortar joints.

1.3. Material Used for Masonry Constructions

This section provides a brief discussion regarding the materials used in different types of masonry construction.

1.3.1. Masonry Units

Material properties of masonry units directly affect the global performance of masonry buildings in terms of total weight, load carrying capacity and cost of the structure. If masonry walls are the main load-bearing mechanism for the structure, masonry units should have the minimum strength or void ratio requirements as defined in the specifications of each national code.

1.3.2. Mortar

In history, mortar is first used to fill the holes when pieces of stone laid on to each other, but it became the binding mixture for the masonry units later on. It is composed of cement, aggregates, water, lime and sometimes admixtures in different proportions. Generally, mortar prisms of standard sizes are tested to determine mechanical properties of mortar. Important properties of mortar are bond strength, workability and water retentivity. Compressive strength of mortar doesn’t have priority since the main function of mortar is to bind the masonry units.

It is important to use mortar in adequate thickness not to reduce the compressive strength of the masonry wall. In general, wall compressive strength reduces as the mortar thickness increases. (Lenczner, 1972).

1.3.3. Concrete and Grout

Grout is used in reinforced masonry, where steel bars are placed inside the voids of hollow masonry units. It is a high slump concrete needed to fill the holes of masonry. The concrete mix is used to cast the RC confining elements and slabs. Additives may be used to avoid water transmission to masonry.

1.3.4. Reinforcing Steel

Reinforcement bars are used for tie-columns and tie-beams in confining elements. It is also used for reinforced masonry between or inside the masonry units.

1.3.5. Masonry Wall Component

Masonry wall behavior is affected by masonry unit properties, mortar properties and the workmanship. In turn, masonry wall behavior determines the seismic resistance of masonry buildings. So it is very important to know the strength and deformation characteristics of masonry wall components. Since masonry quality can vary significantly and is highly dependent of workmanship, strength values of masonry are taken very conservatively in most of the national codes.

1.3.5.1. Compressive Strength of Masonry

Compressive strength of masonry can be determined from empirical formulas, depending on the masonry unit and mortar strength or it can be determined from charts as given in TEC-07. A more precise method is to perform tests on masonry prisms or wallettes (Figure 1.6). In any case, conservative strength values are used in design of masonry structures.

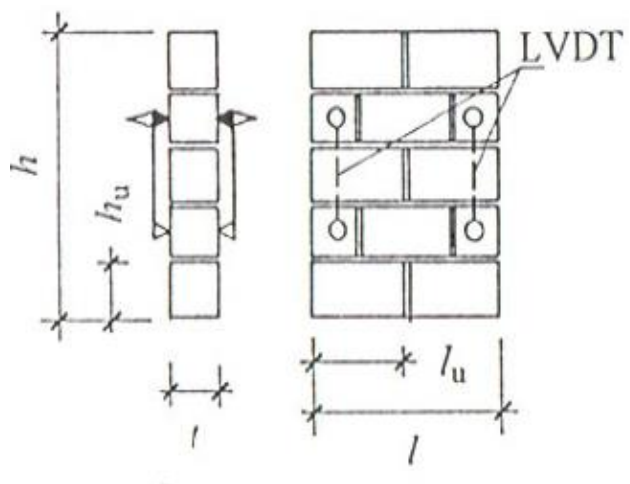


Figure 1.6. Masonry wallette test specimen (Tomazevic, 1999).

According to the test results performed on masonry walls, it is observed that the compressive strength of wall is lower than the compressive strength of single masonry unit. On the other hand, compressive strength of wall can greatly exceed the mortar cube strength.

1.3.5.2. Shear Strength of Masonry

Shear strength of masonry consists of two parts:

- Initial shear strength under zero compression (τ_0)
- Addition of shear strength due to compressive stress on the masonry wall.

Initial shear strength under zero compression (τ_0) is determined by tests on specimens called as triplet tests. The minimum number of specimens to be tested is provided in the related national standard. The characteristic shear strength of masonry (τem) is determined by Equation 1.1. (TEC, 2007)

$$\tau_{em} = \tau_0 + \mu\sigma \tag{1.1}$$

In this equation, σ is the compressive stress on the wall. μ is the friction coefficient and may be taken as 0.5. This equation indicates that the shear strength of a masonry wall increases due to frictional resistance if the wall is under compression. But this trend between compressive stress and shear strength of a wall is not valid after the compressive stress exceeds a certain value, where cracks in the masonry units cause failure of the wall.

1.4. Structural Components of CM Walls

CM walls consist of reinforced concrete elements surrounding the wall called as the confining elements. As mentioned before, vertical confining elements are called as tie-columns and horizontal confining elements are called as tie beams. Tie beams and tie columns should have adequate reinforcement defined in seismic codes of each country. Also tie-columns and tie beams are bonded using appropriate reinforcement detailing. These confining elements improve strength and ductility of masonry walls by confining them. Slabs that can behave like a rigid diaphragm should be used by the designer to distribute the gravity and horizontal loads to the CM walls. Components of CM construction are shown in Figure 1.7.

It is of vital importance to use tie-columns and tie-beams around large openings, because corners of the openings are the primary cause of damage in masonry walls.

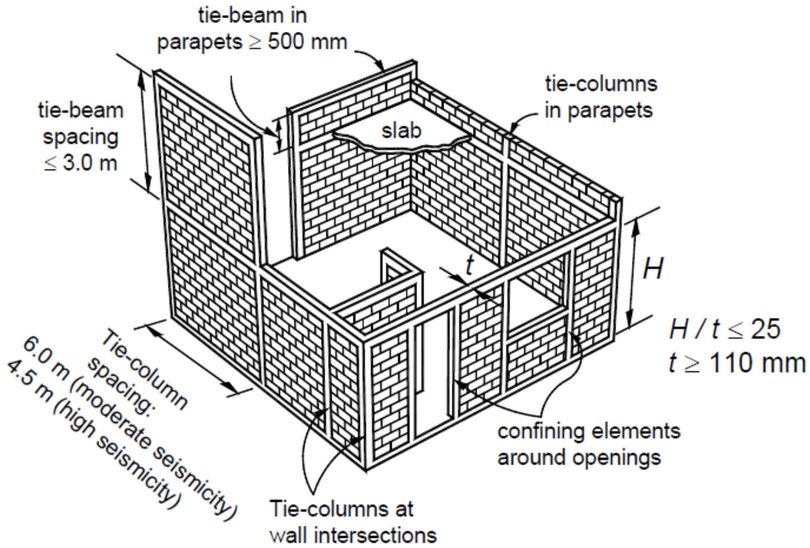


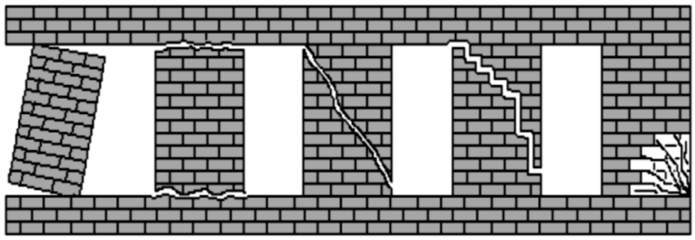
Figure 1.7. Components of a confined masonry building (NTC-M, 2004).

1.5. Failure Modes of Masonry Walls

Most common failure modes of masonry are diagonal tension failure, flexural failure (in the way of toe crushing or rocking) and sliding failure (Figure 1.8). Since masonry walls do not have a homogeneous texture, they show a brittle behaviour. Tensile forces cannot be resisted by masonry. Shear forces can disintegrate the mortar joints. Flexural effects can cause rocking or toe crushing.

1.5.1. Sliding Failure

If masonry wall is squat and has low compressive stress, shear resistance will be lower and sliding failure becomes possible. When the sliding shear resistance is lower than the force needed to create a flexural yielding mechanism or when the wall faces several cycles of inelastic rotation causing the loss of frictional resistance, sliding failure is observed (Centeno et al., 2012) (Figure 1.9).



(a) (b) (c) (d)

Figure 1.8 Failure modes of in-plane masonry walls: **a)** rocking, **b)** sliding, **c)** diagonal tension, **d)** toe crushing (Yi et al., 2006)

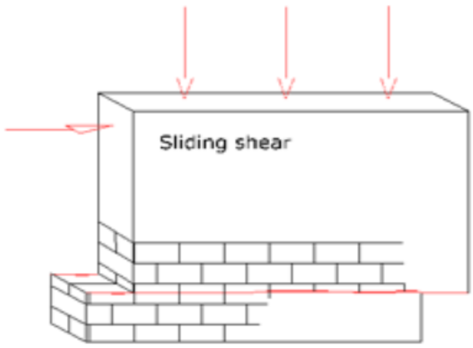


Figure 1.9. Sliding shear mechanism of the wall under loading (Tomazevic, 1999).

1.5.2. Flexural Failure

If the masonry wall has enhanced shear resistance and moment to shear ratio is high, flexural failure may occur in masonry walls. Crushing of compression zones in masonry is observed. In masonry walls with high aspect ratio and low compressive stress, tension cracks at the top and bottom of the wall may cause rocking failure. Flexural failure is rarely observed in masonry structures, because the weight of the structure is higher and deformation capacity is lower, so that moment to shear ratio is usually low.

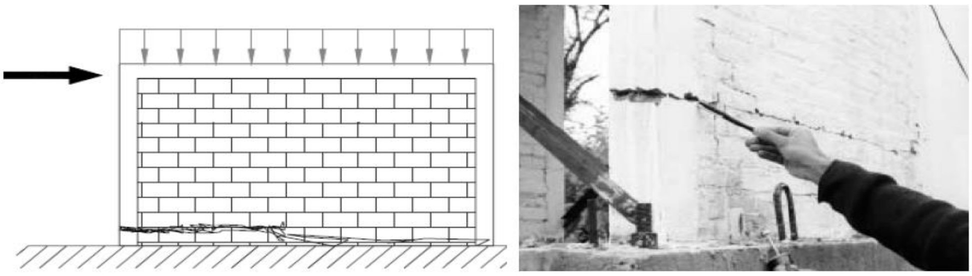


Figure 1.10. Flexural failure mode (Zabala et al., 2004).

1.5.3. Diagonal Shear Failure

It is the most common failure type in masonry structures under seismic loading. Lateral loads combined with vertical loads on the masonry wall causes principal tensile stresses develop perpendicular to the planes cutting the wall diagonally. When the tensile stress reaches the tensile strength of masonry, X-shaped diagonal cracks occur in the wall.

Cracks may follow the mortar joints or pass through the masonry units. In case of strong masonry units and weak mortar, the cracks go through the mortar joints, the wall shows a more ductile behavior. If the cracks go through masonry units, the failure occurs in a brittle manner with sudden strength loss. (Figure 1.10)

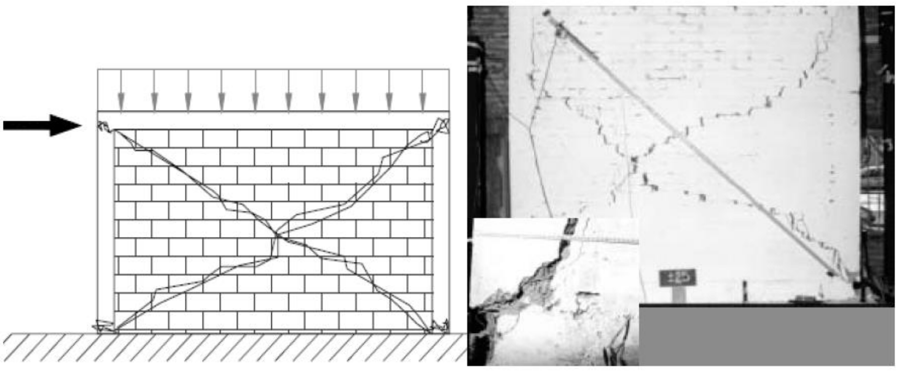


Figure 1.11. Diagonal cracking under cycling loading (Zabala et al., 2004).

1.6. Performance of Masonry Buildings

Masonry is a non-homogeneous material. Different characteristics of mortar and masonry unit makes it vulnerable against lateral forces. Performance tests and analysis on URM structures indicate that after cracking of masonry, structure shows no ductility and faces a sudden brittle failure. On the contrary, confined masonry structures show higher strength, ductility and energy dissipation capacity. (Chourasia et al., 2016). According to the study of Chourasia et al., (2016) results of the test performed on 1 storey CM model show that the first cracks occur in the mortar joints between the masonry unit and mortar due to combined sliding and flexure. At higher deformation levels diagonal shear cracks appear on the walls with the crushing of masonry units and tie column respectively in the compression zones. But the deformed walls can resist longer due to the confining elements. The behavior factor, q, is found to be 1.26 for URM and 3.34 for CM indicating the ductile behavior of CM model.

Observations on the buildings exposed to earthquakes support the test results. According to Doğangün et al. (2008), URM buildings in rural areas of Turkey showed a very poor performance during destructive earthquakes. But the CM buildings; although they are not properly built; showed a better performance than URM buildings. It is also observed in the study that most of the damage is due to unconfined structural or nonstructural elements. Also damages in the walls initiate from the corners of openings most of the time. By using tie-columns around the openings, it is possible to prevent such damages.

1.7. Scope of the Study

This study is concentrated on the performance assessment and comparison of URM and CM buildings under vertical and lateral forces. Total of six generic building models with different floor plans are designed as URM and CM structures with varying material properties and number of stories. In total, 72 building models in their weaker direction are analyzed against 10 real earthquake records with different characteristics. Masonry wall capacities are modeled by the trilinear capacity curve by Tomazevic (1999), which considers three critical limits of performance for masonry walls. Capacity of a storey is determined as the sum of the wall capacities in the critical storey as proposed by Tomazevic (1999). Critical storey of the building models is selected as the ground storey in this study. Capacity spectrum method is used for analysis in order to estimate the performance points of generic building models. Earthquake records and storey shear capacities are converted into Acceleration Displacement Response Spectra (ADRS) format. Performance point of each model is determined as the meeting point of capacity and demand curves. In order to quantify and compare with each other, the results are expressed by single-valued indices. Comparison of URM and CM buildings are realized by using the analysis results.

1.8. Outline of the Study

This study contains six chapters. The content of each chapter is summarized below:

- First chapter starts with general information and brief explanations about masonry construction types, materials used at constructions, structural components of confined masonry, failure types observed in masonry walls and masonry buildings and ends with the scope of the study.
- Second chapter is about the seismic codes or standards about the URM or CM construction all over the world. Specifications for masonry construction of six different countries from different continents are explained in detail. General design rules, geometrical limitations and analysis methods are given. At the end of the chapter, the considered specifications are compared in terms of summary tables.
- Third chapter is about the behavior of masonry walls under vertical and lateral forces. Study of Erköseoğlu (2014), which was intended to be the first phase of this study, is discussed in this chapter. The formulas that were chosen to represent the masonry wall behavior in that study and the results of the parametric study on URM and CM masonry walls were given. Equations used in this study are introduced.
- Fourth chapter is about the proposed approach used to determine the performances of URM and CM buildings in this study. Capacity Spectrum Method procedure introduced by FEMA 440 (ATC, 2005) is given in detail. Quantifi-

cation of seismic demand and capacity in order to estimate the performance point of a structure is also explained.

- Fifth chapter is about the application of the procedure defined in Chapter 4 by using the generic URM and CM building models that have been created in accordance with the code principles discussed in Chapter 2. Models and design rules chosen for the unreinforced and confined masonry buildings are explained. Matlab codes produced to carry out the analysis procedure are introduced. Results of analysis are presented in terms of performance points, limit states and single-valued indices.
- Sixth chapter is the conclusion. The thesis study is briefly summarized, results obtained through the analysis of unreinforced and confined masonry buildings are discussed and future recommendations are made.

CHAPTER 2

REGULATIONS IN SEISMIC CODES FOR URM AND CM BUILDINGS

2.1. General

This chapter provides a detailed discussion regarding the seismic regulations for URM and CM construction techniques. Different codes of practice are discussed and compared throughout this chapter. The seismic codes that are considered in this chapter are the Turkish earthquake code (Turkish Ministry of Public Works and Settlement, 2007), Eurocode (CEN, 2003a and CEN, 2003b), Algerian code (Ministere de L'Habitat et de L'Urbanisme, 2003), Mexican code (Sociedad Mexicana de Ingeniería Estructural, 2001), Chilean code (INN, 1997) and Peruvian code (Ministerio de Vivienda, 2006).

2.2. Turkish Earthquake Code

Based on the empirical design approach, Turkish earthquake code (2007), abbreviated as TEC-07, proposes simple stress calculation methods for seismic design of masonry structures. There are strict geometrical limitations in the code, which impair the flexibility in seismic design of masonry buildings. The following sections explain the code regulations of TEC-07 in a detailed manner.

In TEC-07, masonry construction is not classified according to the construction technique. Hence there is not a specific set of rules for CM construction. Reinforced masonry (RM) construction is not even mentioned in the code. But the code enforces the use of tie-beams and supports the use of tie-columns and defines the dimensions, reinforcement and construction method of confining elements. By the use of tie-columns, it is possible to relax the empirical geometrical limitations dictated for the design of URM construction (like unsupported wall strength, opening size and location, etc.).

2.2.1. General Design Rules

Since TEC-07 is a design code that is based on empirical approach, there exist severe geometrical limitations for masonry buildings. One of the major limitations is related to maximum number of stories in terms of seismic zonation. The allowed number of stories according to each seismic zone is presented in Table 2.1.

Table 2.1. Maximum number of stories according to seismic zone in TEC-07.

Seismic Zone	Maximum Number of Stories
1	2
2,3	3
4	4

According to TEC-07, maximum height from floor level of a storey to floor level of an adjacent storey should be at most 3.00 m in URM buildings. Load bearing walls must be continuous and aligned along the height.

For load bearing masonry walls, minimum thickness values are given in TEC-07. Minimum thickness values of walls are defined separately for 1 to 4 story buildings, for each storey and for each material type as given in Table 2.2.

There exists a simple criterion in TEC-07 regarding minimum total length of load-bearing masonry walls in each principal direction in plan.

$$L_d/A \geq 0.2\ I \tag{2.1}$$

In Equation 2.1, L_d is the total length of masonry walls in one of the principal directions, A is the gross floor area and I is the building importance factor. Although it seems as a simple requirement, field observations after earthquakes have recalled that total length of masonry walls is correlated with the seismic performance of masonry buildings (Bayülke, 1992).

The free length of a load bearing wall segment between two walls intersecting it perpendicularly is defined as the unsupported length of a load bearing wall in TEC-07. This length is limited to 5.5 m in seismic zone 1, and 7.5 m in other zones. If tie-columns are used with 4.0 m intervals, this length can be extended up to 16.0 m.

The wall lengths between any two openings, between a corner and an opening or between an opening and an intersection are also limited in TEC-07. The limit values vary with the seismic zone as shown in Figure 2.1. Solid wall segment between an opening and building corner should be at least 1.5 m in zones 1, 2; and 1.0 m in zones 3, 4. Minimum length can be reduced 20% if tie-columns are used in both sides of the opening.

Table 2.2. Minimum wall thicknesses in TEC-07

Seismic Zone	**Floors**	**Natural stone (mm)**	**Concrete (mm)**	**Clay brick or AAC (unit)**	**Others (mm)**
1,2,3,4	Basement Ground	500 500	250 -	1 1	200 200
1,2,3,4	Basement Ground 1st	500 500 -	250 - -	1.5 1 1	300 200 200
2,3,4	Basement Ground 1st 2nd	500 500 - -	250 - - -	1.5 1.5 1 1	300 300 200 200
4	Basement Ground 1st 2nd 3rd	500 500 - - -	250 - - - -	1.5 1.5 1.5 1 1	300 300 300 200 200

Except the building corners, length of solid wall segment between an opening and wall intersection should be at least 0.50 m. If tie-columns are constructed on both sides of the opening, this limitation can be overruled.

Minimum length of solid wall elements between two openings should be 1.0 m in seismic zones 1 and 2, and 0.80 m for zones 3 and 4. (Figure 2.1) Any opening in plan cannot be longer than 3.0 m.

It is obvious that these strict limitations are enforced to prevent the potential damage that would occur around openings due to stress concentrations. Another restriction in TEC-07 states that length of openings should not be more than 40% of the unsupported wall length. (Figure 2.1) If tie-columns are used along the height of the story on both sides of the opening, maximum length can be increased up to 20%.

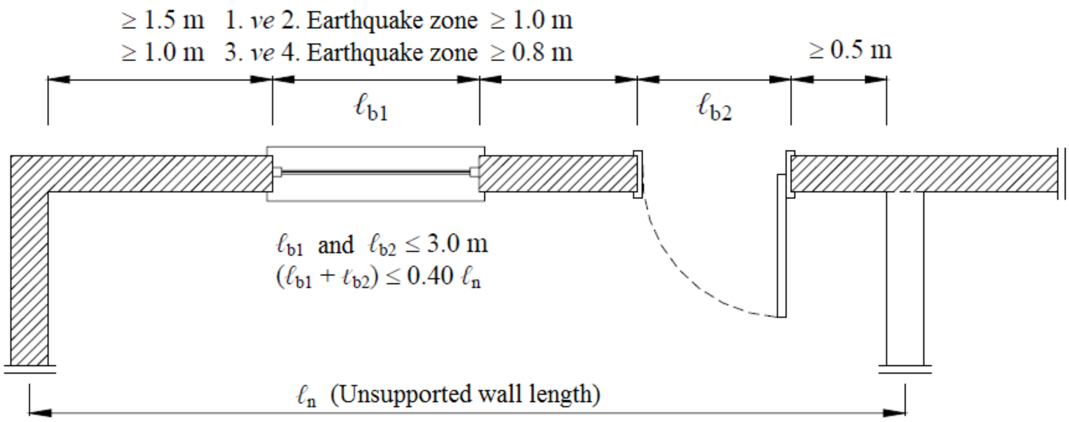


Figure 2.1. Limitations for openings in TEC-07

According to TEC-07, tie-beams must be used in all slab-wall connections including stairway landings. Tie-beams should have the wall thickness and minimum of 200 mm height. Concrete quality must be higher than C16 (concrete compressive strength of 16 MPa). Reinforcement of tie-beams are at least 6 ϕ10 mm bars in stone walls, and 4 ϕ10 mm bars in others. ϕ8 mm stirrups must be installed along the beam with 250 mm intervals.

Method of construction for tie-columns is also defined in the code. Tie-columns should be cast after the masonry walls are built. Tie-columns should have the thickness of wall, width should be at least 200 mm. Concrete quality should be C16. In stone masonry walls, 6 ϕ12 bars should be used whereas in walls with other materials, 4

ϕ12 bars should be used as reinforcement. Longitudinal bars should be surrounded by ϕ8 stirrups with 200 mm intervals.

The material qualifications for the masonry to be used in a load bearing wall must conform to the Turkish standards mentioned in the clause 5.4.1 of TEC-07. For example, if there is no available experimental data, elasticity modulus of masonry can be taken as 200*f_d, where f_d is design masonry design compressive strength of masonry.

TEC-07 defines the minimum unit strength necessary for load-bearing masonry units. Accordingly, minimum compressive strength of masonry units other than adobe should be higher than 5 MPa. Natural stones to be used in basements should have a minimum compressive strength value of 10 MPa. If concrete walls are used in basement, minimum concrete quality should be C16.

2.2.2. Design Calculations

There exist simple stress checks in TEC-07 for design of masonry buildings. Vertical stresses are calculated by dividing the vertical load to the net cross sectional area of the masonry walls. Calculated vertical stresses must be lower than the allowable compressive stress of the wall.

Calculation of the allowable compressive stress should be done by using one of the methods defined in the code. If there are experimental results available for the compressive strength of the wall, allowable compressive stress should be 0.25 times the experimental value. If the mortar type and unit compressive strength are known, allowable compressive strength can be taken from Table 5.2 of the code.

If there exist experimental results available for the masonry unit, but not for the wall component; masonry compressive strength of the wall is 0.5 times the unit strength and the allowable compressive strength is 0.25 times the compressive strength of the wall. If there is no experimental data available for wall, unit and mortar, allowable compressive strength of the wall can be taken from Table 5.3 of TEC-07.

The allowable compressive stress obtained by using one of the methods above is reduced according to the slenderness ratio of the wall to account for eccentric loading conditions. The reduction factor values in terms of slenderness ratio are provided in Table 5.4. of TEC-07.

In addition to vertical stress, shear stress is also checked by using equivalent lateral load approach. In calculation of shear stresses, torsional effects should also be considered calculating the positions of mass center and rigidity center. Story shears are distributed to each wall element according to the relative shear stiffness of each element assuming rigid floor diaphragm action. For each wall axis, shear stiffness of wall segments between two openings are calculated and summed up to find total stiffness of the wall axis.

According to TEC-07, earthquake loads should be applied separately for two orthogonal directions of the building. Shear force for each wall segment should be calculated by distribution of story shears to each wall segment according to its relative stiffness plus the shear force created due to torsional moment on the wall segment. Calculated shear force divided by cross sectional area should be less than the allowable shear stress. Allowable shear stress is calculated according to Equation 1.1.

2.3. Eurocode (CEN, 2003)

The design rules to be applied for earthquake resistant masonry buildings exist in Eurocode 6 (EN 1996) (CEN, 2003a): Design of masonry structures and Eurocode 8 (EN 1998) (CEN, 2003b): Design of structures for earthquake resistance.

2.3.1. General Design Rules

Geometric requirements are defined for masonry walls for different type of unreinforced masonry and confined masonry walls. There is a minimum thickness requirement for each wall. But also minimum effective height of the wall should be less than a specified ratio of the effective wall thickness. This limitation defines the slenderness of the wall. Minimum length of a solid wall segment between two openings was explicitly defined in TEC-07. But in Eurocode 8; it is defined as the ratio of solid wall length to greater height of the adjacent openings. The prescribed requirements are given in Table 2.3.

Table 2.3. Requirements for shear walls from Eurocode 8 (CEN, 2003b).

Masonry type	$t_{ef,min}$ **(mm)**	$(h_{ef}/t_{ef})_{max}$	$(l/h)_{min}$
Unreinforced, with natural stone units	350	9	0.5
Unreinforced, with any other type of units	240	12	0.4
Unreinforced, with any other type of units, in cases of low seismicity	170	15	0.35
Confined masonry	240	15	0.3

In Table 2.3, $t_{ef,min}$ (in mm) is the minimum thickness for the walls, heff is the effective height of the wall, l is the length of the wall and h is the greater clear height of the openings adjacent to the wall.

2.3.1.1. Rules for URM Construction in Eurocode

URM construction in Eurocode is classified into two categories based on Eurocode 8 for seismic resistance as unreinforced masonry construction in accordance with Eurocode 6 only and unreinforced masonry in accordance with Eurocode 8. Eurocede 6 covers the simple masonry structures. There are some additional requirements for unreinforced masonry in accordance with Eurocode 8 such that there should be continuous horizontal ring beams or steel ties in the plane of the wall at each floor level or with a vertical spacing not more than 4 m. In this respect, Eurocode 8 has the same approach with TEC-07. Minimum cross sectional area of longitudinal steel bars placed in the beams should not be less than 200 mm2. Unreinforced masonry construction in accordance with Eurocode 6 can only be constructed in low-seismicity regions a design acceleration value (a_g) not exceeding 0.20g.

2.3.1.2. Rules for CM Construction in Eurocode

Requirements for construction of CM buildings are defined in both Eurocode 6 (rules for reinforced and unreinforced masonry) and Eurocode 8 (design of structures for earthquake resistance). According to these codes, tie-columns should exist at every wall intersection. Openings greater than 1.5 m^2 should have tie-columns on both sides. In any case, distance between tie-columns or tie-beams should not exceed 4 m.

Confining elements should not be smaller than 0.02 m^2 with a minimum dimension of 150 mm in the plan of the wall. According to Eurocode 8, minimum reinforcement cross-sectional area can be 1% of total cross-sectional area and 300 mm^2. Stirrup diameter should be at least 5 mm; stirrup interval cannot be more than 150 mm. Units next to tie-columns should be overlapped according to Eurocode 6.

Evaluation of material properties should be made according to the related standards for those materials. The compressive strength of masonry should be determined by tests. If the experimental results are not available, the equations given in Section 3.6.1.2. of Eurocode 6 can be used. (CEN, 2003a) These equations represent the characteristic compressive strength of masonry by means of compressive strengths of masonry unit and mortar in the form of Equation 2.3.

$$f_k = K f_b^{\alpha} \, f_m^{\beta} \qquad (2.3)$$

The characteristic values of strength should be divided by the specific partial factors mentioned in the code for each material to evaluate the values to be used in design. The characteristic shear strength of masonry should be determined by tests as in the case of compressive strength. Alternatively, Equations 3.5, 3.6 and 3.7 given in Eurocode 6 may be used depending on the initial shear strength (f_{vko}), compressive stress on the wall (f_d), and the masonry unit compressive strength (f_b). Initial shear strength (f_{vko}) should also be determined from tests or taken from Table 3.5 of Eurocode 6. The values of characteristic shear strength cannot be greater than specified values (like $0.045f_b$ or $0.065f_b$ depending on the materials used in masonry) mentioned in the code.

Characteristic flexural strength due to out of plane bending is considered in two ways, which are the failure planes parallel to bed joints and perpendicular to bed joints. Characteristic flexural strength values should be determined by tests or taken from the tables given in Section 3.6.3 of Eurocode 6.

Minimum compressive strength of masonry units and mortar are defined in Eurocode 8. Compressive strength normal to the bed plane is 5 N/mm^2, parallel to the bed plane is 2.5 N/mm2. Minimum strength of mortar is 5 N/mm^2. Reinforcing steel should be of Class B or C in accordance with Eurocode 2 (CEN 2004).

2.3.2. Design Calculations

According to Eurocode, stiffness values of structural elements should be evaluated considering both shear and flexural response of these elements. Uncracked stiffness may be used for analysis, but cracked stiffness should be preferred. If no experimental data about cracked stiffness values are available, it may be taken as one half of the gross section uncracked elastic stiffness. Masonry spandrels can be modeled as coupling beams if they are properly connected to the neighbouring walls, lintel and tie-beams. If spandrels are modeled as coupling beams, a frame analysis can be used to determine the forces on the structural elements. Redistribution of base shear forces is possible within the limitations specified in the code.

Behavior or load reduction factor values used in determination of earthquake loads are defined in the code according to masonry construction type. The values to be used can change between the intervals defined in Table 2.4 according to the national annex of the country.

Table 2.4. Behavior factors according to masonry type

Type of construction	Behavior factor (q)
Unreinforced masonry in accordance with Eurocode 6 alone	1.5
Unreinforced masonry in accordance with Eurocode 8	1.5-2.5
Confined masonry	2.0-3.0

2.3.2.1. Simple Masonry Buildings

For the simple masonry buildings defined in the code, no structural analysis is required. Simple masonry buildings should comply with the rules described in sections 9.2, 9.5, and 9.7.2 of Eurocode 8. Simple masonry buildings should satisfy minimum area of shear walls in plan indicated in Table 9.3 of the same standard. The building should be symmetric in plan and almost rectangular in plan geometry. There should be at least two shear walls in each orthogonal direction of analysis and each of those walls should have at least 30% of the building length in that direction. Shear walls should carry at least 75% of the vertical loads. For unreinforced masonry buildings, walls should be joined with orthogonal walls at a maximum length of 7 m.

2.4. Algerian Code (MHUV, 2003)

Confined masonry is the only method of masonry construction allowed in Algerian Seismic Code. (MHUV, 2003). The details regarding this code are provided in the following sections.

2.4.1. General Design Rules

According to the Algerian Code, buildings should be regularly designed. Length to width ratio of the building should not exceed 3.5. Height and number of floors of the buildings are limited according to seismic zones (Table 2.5 and 2.6).

Behavior factor used to determine the earthquake loads should be taken as 2.5. Walls should be evenly distributed in both directions and total cross sectional area of walls should not be less than 4% of the total floor area. Wall-wall and wall-floor connections should be properly made. Walls should be properly tied to rigid floor diaphragms for the shear forces to be distributed according to the stiffness of the walls. Thickness of lateral load carrying walls should be at least 20 cm. Maximum distance between the walls is also limited according to seismic zones as given in Table 2.7.

Table 2.5. Seismic zones according to the Algerian Code

Seismic Zone	Level of Seismicity
Zone 0	No seismicity
Zone I	Low seismicity
Zone IIa and IIb	Moderate seismicity
Zone III	High seismicity

Table 2.6. Height and number of floors according to seismic zones

		Seismic Zone		
		Zone I	**Zone II**	**Zone III**
Height	in m	17	14	11
Number of flo-ors	N	5	4	3

Table 2.7. Maximum distance between structural walls.

Seismic Zone	**Zone III**	**Zone II**	**Zone I**
Distance (m)	6	8	10

Similar to TEC-07, length of openings and distances between openings are limited according to the Algerian code. Total length of openings in each wall segment should not be more than 50% of the total length of the wall. Solid wall segment between building corners and openings should not be less than 1 m in length. Solid wall segment between openings should not be less than 1 m in length for zone III, and one-third of the total length of openings for zone I and zone II.

In confining elements; tie-beams should be used in foundation level and each floor level. Tie-columns should be used in intersection of walls, around the openings that have a height greater than 1.8 m. No wall element in the structure should have a free end.

Maximum distance between confining elements should be 5 m. Maximum area between confining elements should not be more than 20 m^2. Minimum dimensions of a tie-column must be 15*15 (in cm). Minimum 4φ10 mm bars of longitudinal reinforcement are necessary for the tie-column. Transverse reinforcement with a maximum spacing of 25 cm needs to be installed around the longitudinal bars. Reinforcement should be overlapped by 40×bar diameter in zone I and II and by 50×bar diameter in zone III. Maximum distance between bars should not be more than 20 cm.

Rules of tie-columns for reinforcement are identical for tie-beams. Tie-beams must have a minimum width equal to 15 cm and two-thirds of the wall thickness for the wall cover.

In case of openings; it is already mentioned that tie-columns must be used around the openings that is higher than 1.8 m. Besides, the openings are categorized into 3 groups in the Algerian code. Category G represents the openings that have a dimension greater than 2.5 m. These kinds of openings should be surrounded by a steel, wooden or reinforced concrete frame whose elements are connected to each other. Category M represents the openings that have a dimension greater than 1.5 m but less than 2.5 m. These openings should be surrounded by a frame as described above if the building remains in zone III. If they remain in zone II, they should be framed in case they take place in a wall segment that doesn't have a dimension larger than 3.2 m. Openings that are not included in categories M and G, are considered under category P. For the openings in category P, similar rules to category M is valid except that the elements of the surrounding frame need not to be connected to each other.

2.4.2. Design Calculations

According to Algerian code, masonry walls are accepted as cantilever vertical elements fixed at their base. Masonry and reinforced concrete confining elements are assumed to form a triangular system where the diagonal element is formed by a strut in the masonry panel. The length of this strut is taken as the minimum of 1/6 of the strut length or 4 times the thickness of the wall. The compressive stress on the masonry should be less than the characteristic compressive resistance of the wall divided by the material factor.

2.5. Mexican Code (SMIE, 2004)

Mexican code gives very detailed rules about design and analysis of masonry structures. Unconfined and confined masonry construction rules are clearly separated.

2.5.1. General Design Rules

Design rules enforced by the Mexican code will be separately discussed for URM and CM buildings in the following sections.

2.5.1.1. Rules for URM Construction in the Mexican Code

Unconfined and unreinforced masonry structure defined in Mexican code does contain some reinforcement necessary for structural integrity, but this amount of reinforcement is not enough to consider it as either reinforced or confined masonry. The minimum thickness of walls should be 10 cm. The behavior factor to be used in seismic design is 1.

For the integrity of the structure, vertical and horizontal reinforcement described in the code should be used. Reinforcement should be embedded in concrete elements having a minimum dimension of 50 mm or vertical reinforcement of minimum 2 bars or wires should be used at ends of the wall, wall intersections and with a maximum distance of 4 m. The total area of vertical reinforcement necessary in a wall is calculated from Equation 2.4.

$$A_S = \frac{2V_{mR}}{3F_R f_y} \tag{2.4}$$

where:

$$V_{mR} = F_R(0.5v_m A_T + 0.3P) \leq 1.5F_R v_m A_T \tag{2.5}$$

In Equations 2.4 and 2.5; V_{mR} represents diagonal shear strength of the wall, F_R represents strength reduction factor and P is the vertical load acting on the wall. At least two bars or wires should be used as horizontal reinforcement in wall-floor connections. Total area of horizontal reinforcement to be used in a wall can be calculated by Equation 2.6.

$$A_S = \frac{2V_{mR}}{3F_R f_y}\frac{H}{s_v} \tag{2.6}$$

Stirrups or cross ties with 200 mm intervals should be placed around the longitudinal reinforcement. Reinforcement for unconfined and unreinforced masonry is illustrated in Figure 2.2.

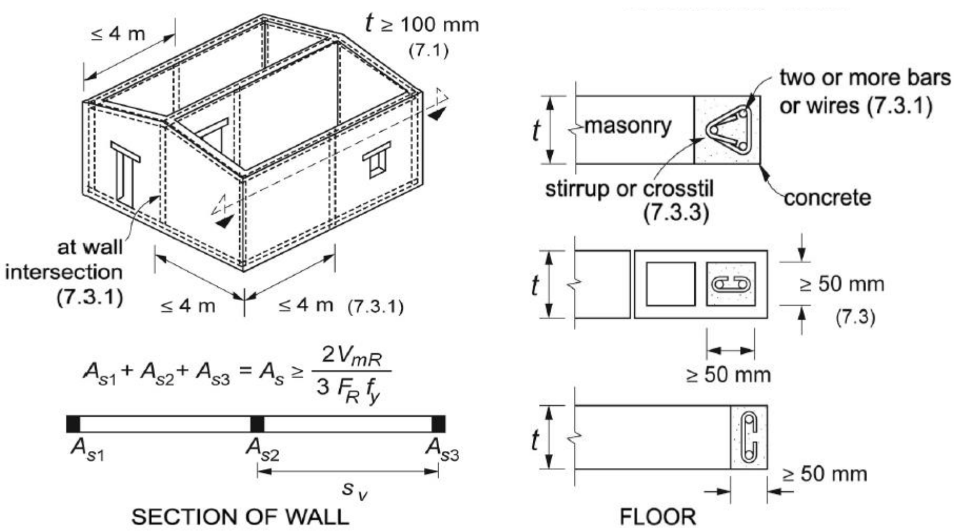


Figure 2.2. Integrity reinforcement in the Mexican Code.

2.5.1.2. Rules for CM Construction in the Mexican Code

Confined masonry walls should be reinforced with tie-columns and tie-beams placed in required locations dictated by the code (Figure 2.3). Tie-columns and tie-beams must be cast after the walls have been built.

Tie-columns should be placed with spacing not more than 1.5 times the story height or 4 m. Bond beams should be placed at every floor level. Space between two bond beams should not be more than 3 m. Bond beams should be placed on top of parapets of which height is greater than 50 cm. Minimum dimensions for tie-columns and tie-beams should be equal to the wall thickness. Minimum wall thickness is allowed to be 10 cm. Height to thickness ratio of walls should not exceed 30. Concrete quality of confining elements must be at least 15 MPa.

Amount of longitudinal reinforcement to be used in tie-columns and tie-beams should be calculated so as to resist the corresponding vertical and horizontal components of the strut that develops in the masonry wall panel. In any case, longitudinal reinforcement should be at least three bars and the total area should not be less than the amount specified in Equation 2.7.

$$A_S = 0.2\frac{f_c'}{f_y}t^2 \tag{2.7}$$

Minimum area of transverse reinforcement is defined by Equation 2.8. Spacing of stirrups should not exceed 1.5 times the wall thickness or 20 cm.

$$A_{sc} = \frac{10000s}{f_y h_c} \tag{2.8}$$

If the design diagonal compressive strength of masonry (v_m*) is greater than 0.6 MPa; additional transverse reinforcement should be used at both ends of the tie-column to prevent the failure of tie-columns. A distance of H_0 from both ends of the tie-column must be reinforced with additional stirrups of specified amount in Equation 2.8. H_0 should be taken the larger of H/6, 40 cm or 2 times the thickness of the confining element. Openings should also be surrounded by confining elements, if their width exceeds the values shown in Figure 2.4.

Materials to be used in masonry structures are classified as solid units and hollow masonry units. Solid units are the ones that have the cross-sectional net area equal or greater than 75% of the gross area. Also the exterior shell of these units should have a minimum thickness of 2 cm. Hollow masonry units are those that have a net cross-sectional area equal or greater than 50% of the gross area. Thickness of outer shell should be 15 mm. For multi-perforated units, interior thickness between holes should be at least 7 mm. Units with horizontal holes are also permitted.

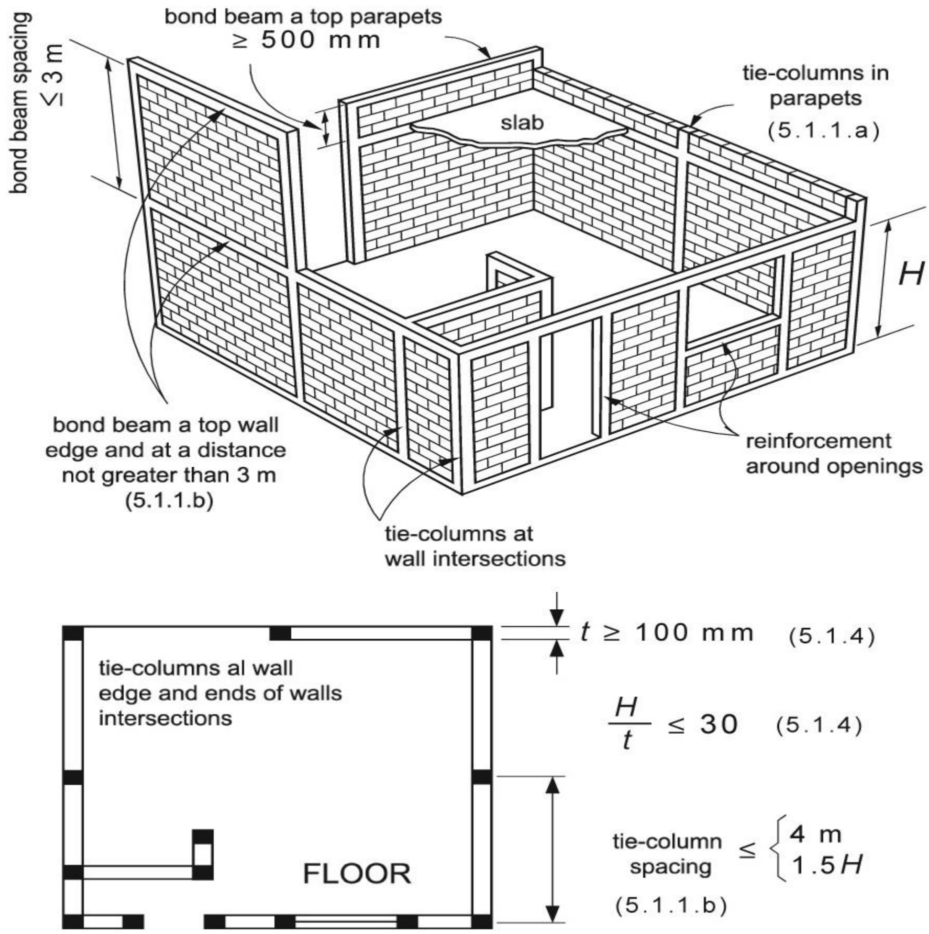


Figure 2.3. Requirements of confined masonry construction

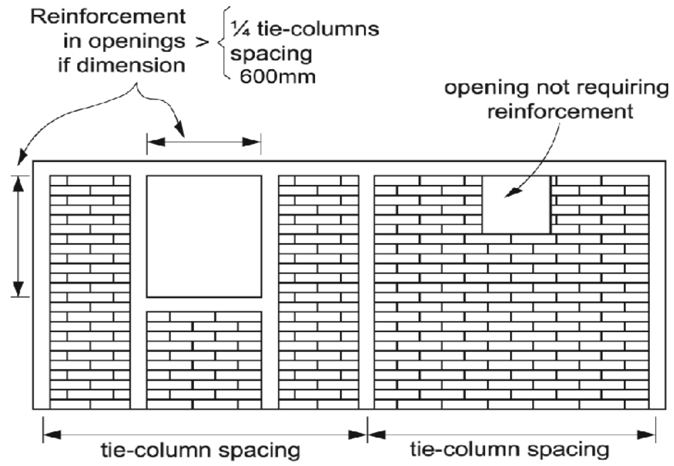


Figure 2.4. Conditions for confinement of openings.

Similar to TEC-07, compressive strength of masonry (f_m) can be determined by using one of the three methods. The direct method is to perform experiments on masonry prisms that have height to width ratio between 2 and 5. But if it is not possible; fm can be determined using the known compressive strength of masonry units and mortar according to type of units given in Tables 2.6 and 2.7 of the code. If there exist no information on prism or units and mortar, Table 2.8 of the code should be used. If a concentrated load is directly applied to masonry, bearing strength should be taken as 0.6 fm. Tensile strength of masonry should be neglected.

Design diagonal compressive strength of masonry is the main parameter in determining the resistance of masonry to lateral loads. Similar to diagonal compressive strength, it can be determined by tests on masonry wallets (Figure 2.5) or from Table 2.9 of the code according to masonry unit and mortar characteristics.

Modulus of elasticity for short term loads and long term loads are separated in the code. Modulus of elasticity for short term loads can be determined from tests on masonry prisms. If this method is used, modulus of elasticity for sustained loading is calculated using short term modulus. Short term modulus is divided by 2.3 for concrete units and 1.7 for clay units. Modulus of elasticity can also be determined form design compressive strength of masonry as follows (see Equations 2.9, 2.10, 2.11 and 2.12):

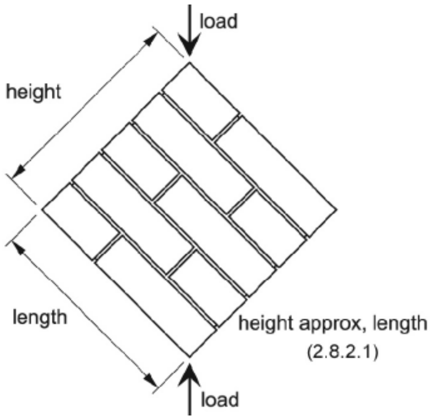


Figure 2.5: Diagonal compression test

Concrete bricks:

$$E_m = 800\ f_m \text{ (for short term loads)} \tag{2.9}$$

$$E_m = 350\ f_m \text{ (for long term loads)} \tag{2.10}$$

Other units:

$$E_m = 600\ f_m \text{ (for short term loads)} \tag{2.11}$$

$$E_m = 350\ f_m \text{ (for long term loads)} \tag{2.12}$$

Shear modulus of masonry can also be determined by tests. If there is no experimental data, it is given by Equation 2.13.

$$G_m = 0.4\ E_m \tag{2.13}$$

2.5.2. Design Calculations

Lateral loads should be distributed to the structural elements considering their shear and flexural stiffnesses. Confined masonry walls can be modeled as wide columns having the same moment of inertia and shear area of the actual wall. If these walls have openings, openings can be modeled as coupling beams in the case that the openings are regularly distributed throughout the height of the building. (Figure 2.6.)

Inelastic lateral drift angles should be limited to the values described in the code according to the type of masonry construction. The drift angle value should not exceed 0.0035 for unreinforced confined masonry and 0.0015 for unconfined and unreinforced masonry.

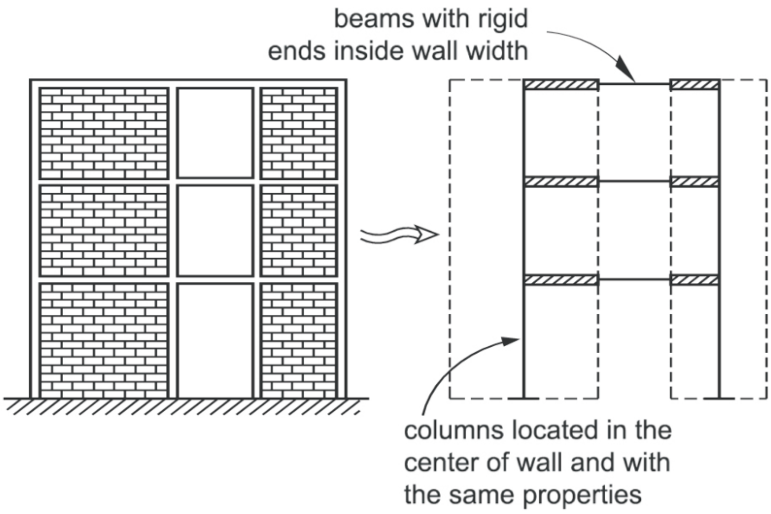


Figure 2.6. Wide column model for confined masonry.

Resistance of unconfined and unreinforced masonry walls are represented by their design strengths in the code. Maximum axial load that a wall can resist (P_R) is determined based on the masonry compressive strength (f_m), strength reduction factor (F_R) and slenderness and eccentricity reduction factor (F_E) (Equation 2.14.).

$$P_R = F_R F_E f_m A_T \tag{2.14}$$

For unconfined and unreinforced masonry walls under axial loading, F_R is 0.3. Reduction factor for eccentricity and slenderness ratio (F_E) depends on the free wall height (H), wall effective height factor (k), wall thickness (t) and eccentricity of the loading (e'). For unconfined and unreinforced masonry walls; F_E is determined by Equation 2.15.

$$F_E = \left(1 - \frac{2e'}{t}\right)\left[1 - \left(\frac{kH}{30t}\right)^2\right] \tag{2.15}$$

Strength under axial load and bending should be determined considering that the masonry does not resist tension. Failure occurs when the stress in the compression zone reaches f_m*.

Shear strength of walls should be determined by Equation 2.5 based on the diagonal compressive strength of the wall (v_m*) and vertical load (P) acting on the wall. In lateral loading case, F_R is taken as 0.4.

Compressive strength of a confined masonry wall is computed by the contribution of reinforcement in the tie-columns into account according to Equation 2.16.

$$P_R = F_R F_E \left(f_m{}^* A_T + \sum A_S f_y\right) \tag{2.16}$$

Reduction factor for slenderness and eccentricity for confined masonry is calculated according to Equation 2.17.

$$F_E = \left(1 - \frac{2e'}{t}\right)\left[1 - \left(\frac{kH}{30t}\right)^2\right]\left(1 - \frac{H}{L'}\right) + \frac{H}{L'} \leq 0.9 \tag{2.17}$$

The second alternative to compute P_R is given by Equation 2.18.

$$P_R = F_R F_E (f_m{}^* + 0.4) A_T \tag{2.18}$$

In Equation 2.18, units should be in MPa and mm^2.

The flexural strength of confined masonry walls is determined considering the same principles as in the unconfined and unreinforced masonry walls. The tension force is resisted only by the reinforcing steel and the section fails when the compressive strain on the masonry reaches 0.003.

An alternative practical equation is presented in the code for the walls that is symmetrically reinforced with identical tie-columns on both ends (Equations 2.19 and 2.20).

$$M_R = F_R M_0 + 0.3 P_u d; \qquad If\ 0 \le P_u \le \frac{P_R}{3} \tag{2.19}$$

$$M_R = (1.5 F_R M_0 + 0.15 P_R d)\left(1 - \frac{P_u}{P_R}\right); \ \text{if}\ P_u \ge \frac{P_R}{3} \tag{2.20}$$

In these equations, d is the distance between centroid of steel under tension and the fiber under maximum compression (Figure 2.11). P_u is design axial load on the wall. M_0 is defined as the pure bending strength of the wall and calculated by Equation 2.21.

$$M_0 = A_s f_y d' \tag{2.21}$$

In Equation 2.21; d' represents the distance between centroids of steel at two ends of the wall. (Figure 2.7)

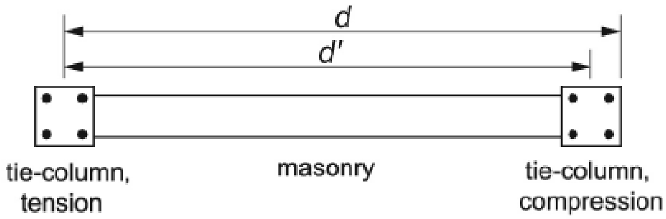


Figure 2.7. Definitions of d and d' for flexural strength of confined masonry.

Strength reduction factor for confined masonry walls are much higher compared to unconfined and unreinforced masonry walls resulting in much higher design strengths. Value of F_R is dependent on the level of design axial load (P_R); it is equal to 0.6 if $P_u \geq P_R/3$ or equal to 0.8 if $0 \leq P_u \leq \frac{P_R}{3}$ (Figure 2.8).

Shear force resisted by a confined masonry wall should be calculated based on Equation 2.5 similar to the unreinforced and unconfined masonry. Tie-columns should be included in the cross-sectional area without any area transformation. It should be reminded that value of F_R is different for confined masonry which gives a higher shear strength value for confined masonry.

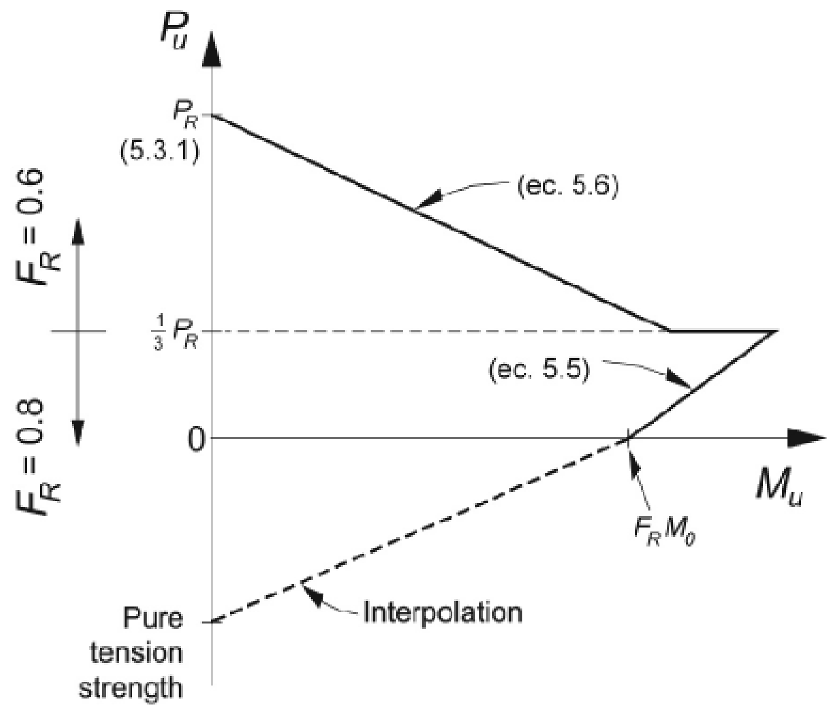


Figure 2.8. Interaction diagram for axial load-design flexural moment and value of F_R.

2.5.2.1. Simplified Method

There exists a simplified method of analysis for the buildings that comply with certain requirements described in the code. To apply the method, on each floor of the building at least 75% of the vertical forces should be resisted by continuous shear walls in elevation. Floors should satisfy the rigid diaphragm behavior. Walls should be symmetrically distributed, which means the torsional eccentricity should not be more than 10% of the total length of the building in that direction.

Similar to TEC-07, stiffnesses of structural walls can be defined as a factor of cross sectional areas called the effective areas. Effective area is the product of cross sectional area of a wall and the factor F_{AE} that depends on the length to height ratio (L/H) of the wall.

$$F_{AE} = 1; \qquad \text{if } H/L \leq 1{,}33$$

$$F_{AE} = 1.33\left(L/H\right)^2; \qquad \text{if } H/L > 1{,}33 \tag{2.22}$$

There has to be two walls in each direction of analysis that have a total length equal to at least one half of the dimension of the building in that direction. The length to width ratio of the building should be less than or equal to 2. To satisfy this criterion, building should be divided into independent parts. The height of the building should not be more than 3.5 times the smaller dimension of building. The maximum height of the building should not be more than 13 m.

If the conditions defined above are satisfied, a very simplified analysis can be applied that the stiffness of load bearing elements are proportional to their effective areas ignoring torsion and overturning moments.

2.6. Chilean Standards (INN, 1997)

There is not a standard for unconfined and unreinforced masonry in Chile. Rules of confined masonry construction are defined in the Chilean code abbreviated as "NCh2123". The code is published in 1997 and modified in 2003. NCh2123 includes definitions for material properties, conditions and limitations of design, construction rules and control of works.

2.6.1. General Design Rules

Confining elements do not increase the shear resistance of the wall. They prevent brittle failure of the wall after diagonal cracking. Construction detailing should be done according to the code so that masonry and confining elements act together to resist the stresses. In seismic zone 2 and 3, all the walls should be confined according to NCh433-Seismic Design of Buildings. In seismic zone 1, all the perimeter walls should be confined, confined walls should withstand at least 70% of story shear, walls that resist 10% or more of the story shear should be confined. Confining elements should provide confinement in the plane of the wall and support the wall against out of plane forces. Allowable stress values can be increased 33% for seismic stresses or for permanent loading.

Confining elements should be cast after the masonry walls are built. Tie-columns and masonry wall should be connected by toothing or bars embedded in the tie-column and horizontal mortar joints.

The thickness of the wall should be equal or more than 1/25 of the smaller of length or height of the wall. In any case, thickness should be equal to or more than 14 cm for fabricated units and 15 for handmade units. Maximum area of a wall panel between tie-beams and tie-columns should be 12.5 m2. Maximum length of wall can be 6 m.

Tie columns should be located in all free-ended walls, all intersections of walls and inside the walls to satisfy the maximum dimension limitations of a wall. Tie-beams should be located on every floor level and roof level. They should also be used inside the wall to satisfy the maximum dimensions of the wall.

Reinforcement of openings can be provided by reinforced concrete elements, reinforced bars placed inside the holes of masonry or horizontal mortar joints. Reinforcement of the walls should be designed according to in-plane and out of plane forces. Minimum area of reinforcement of vertical reinforcement on each border should be 0.8 cm2 and minimum diameter of bars should be 8 mm. Horizontal reinforcement with a minimum area of 0.5 cm^2 should be placed in the first mortar joint above and below the opening. Diameter of bars should be equal or less than the joint thickness.

Reinforcement of openings is not necessary if the following conditions are all satisfied:

- the area of the opening is less than 5% of the wall area,
- dimensions of the opening are less than 60 cm, the distance between the vertical border of the opening and the face of the tie-column is equal or more than 25% of the length of the wall panel,
- The distance between the horizontal borders of the opening and tie-beams inner face is equal or more than one-third of the wall panel height.

Openings with dimensions equal or less than 20 cm can be in any location on the wall. Three openings of this type are allowed in a single wall provided that they are separated with 1 m wall segments.

Tie-columns and tie-beams must have critical zones at both ends. Tie-columns, starting from the internal face of the tie-beam, should have a critical zone greater of two times the width of tie-column or 60 cm. For tie-beams critical zone starting from the inner face of the column is 60cm. If a rigid slab is cast together with the beam, critical zones may be ignored. Thickness of tie-columns and tie-beams should be equal or more than the wall thickness. Width should be equal or more than 20 cm.

Secondary units (units used before) can only be used with the approval of the structural designer of the work. Masonry wall panels must be constructed with the same quality or class of materials. Minimum compressive strength should be 15 MPa for these units. Materials should conform to the related Chilean standards. Minimum compressive strength of mortar should correspond to the compressive strength of the unit. In any case, minimum compressive strength of mortar should not exceed 5 MPa for handmade ceramic bricks; 10 MPa for fabricated units.

Compressive strength of masonry should be determined by tests performed on masonry wall prisms. Compressive strength of masonry should be evaluated by performing 5 prism tests.

If experimental data is not available for masonry, for fabricated masonry units Equation 2.23 and Equation 2.24 can be used. But in this case, mortar used in the masonry should conform to the related standards.

$$f_m = 0.25 f_p; \mathit{for\ ceramic\ units} \tag{2.23}$$

$$f_m = 0.30 f_p; \ \mathit{for\ concrete\ blocks} \tag{2.24}$$

In any case compressive strength of masonry calculated with Equation 2.23 cannot be higher than 6.0 MPa, and calculated with Equation 2.24 cannot be higher than 4.5 MPa. If there is no information about the quality of masonry or the materials; fm shall be taken as 1.5 MPa.

Shear strength of masonry is determined in a similar way to compressive strength if there is experimental data available for shear strength of prisms. If this is not available, shear strength can be determined by considering class of masonry unit and mortar according to NCh2123.

Tensile strength for flexure is determined based on the test procedure described in ASTMC1072. Equation 2.22 is used to obtain the tensile strength F_{bt}. F_{bt} can also be taken from Table 2 of NCh2123. For units not mentioned in the table, experimental results must be obtained.

There are practical equations given in the code for elastic and shear modulus (Equation 2.25, Equation 2.26).

$$E_m = 1000 f_m \tag{2.25}$$

$$G_m = 0.3 E_m \tag{2.26}$$

2.6.2. Design Calculations

There are no specific rules defined for modeling of masonry buildings defined in NCh2123. In case of analysis, calculation of allowable stresses in masonry is defined in detail.

The shear force that each column should resist in critical zones should be the lower value of the allowable shear force (without 33.3% addition) and 4/3 times the shear force acting on the wall. Stirrup area in critical zones should be calculated by Equation 2.27.

$$A_e = \frac{(V_p - V_c)s}{f_y d_p} \tag{2.27}$$

In Equation 2.27, V_p represents the shear force that must be resisted by the column in the critical zone, V_c represents the shear force resisted by concrete (Equation 2.28).

$$V_c = 16.66\sqrt{f_c}\,b d_p \tag{2.28}$$

As a minimum, 4φ10 longitudinal bars should be used in tie-columns or tie-beams. Minimum diameter of stirrups should be 6 mm. Maximum spacing of stirrups should be 20 cm in intermediate zones and 10 cm in critical zones. Lap splicing of longitudinal bars should be outside the critical zones.

The rules given in NCh2123 are valid for walls that masonry unit lapping length is more than one-fourth of unit length. Allowable shear strength of a confined masonry wall for in plane forces is calculated according to Equation 2.29.

$$V_a = (0.23\tau_m + 0.12\sigma_0)A_m \geq 0.35\tau_m A_m \tag{2.29}$$

In this equation, A_m represents the gross area of the section, τ_m represents the shear strength under zero compression and σ_0 represents the average compressive stress on the wall. Allowable compressive strength of a wall is calculated by Equation 2.30.

$$N_a = 0.4 f_m \varphi_e A_m \tag{2.30}$$

Here φ_e represents the reduction factor for slenderness that depends on the wall height (h) and thickness (t) and it is calculated by Equation 2.31.

$$\varphi_e = \left[1 - \left(\frac{h}{40t}\right)^3\right] \tag{2.31}$$

Strength of a wall against pure bending is represented by Equation 2.32.

$$M_{oa} = 0.9\, A_s f_s d' \tag{2.32}$$

When a confined masonry wall is subjected to axial force and moment, moment resistance is calculated by Equations 2.33 and 2.34.

$$M_a = M_{oa} + 0.20Nd;\ if\ N \leq N_a/3 \tag{2.33}$$

$$M_a = (1.5M_{oa} + 0.10N_a d)\left(1 - \frac{N}{N_a}\right);\ if\ N > N_a/3 \tag{2.34}$$

The required resistance of the wall against out of plane forces is also defined. Walls should be considered as plates simply supported by tie-columns and tie-beams. When the seismic forces are applied, the tension force that would result as a combination of axial force and flexure should be 50% or less than the flexural tensile strength F_{bt}. For the flexural compression design of the walls, 50% of the seismic stresses calculated according to NCh433 should be used.

2.7. Peruvian Standards (Ministerio de Vivienda, 2006)

The Peruvian code named as “Norma E.070 Albanileria” covers the design, analysis, materials, construction and control of works for masonry structures. The code does not support the construction of unreinforced and unconfined masonry structures. Design and construction of confined or reinforced masonry is permitted. Seismic design of masonry structures is carried out according to “Norma Tecnica de Edificacion E.030 Diseno Sismoresistence” together with the rules in the masonry code.

2.7.1. General Design Rules

According to the code, configuration of buildings should be simple. Geometrical layouts including L, T or similar shapes should be avoided or these types of buildings should be divided into simple shapes. The ratio between dimensions of the building cannot exceed 4. If the length to width ratio is greater than 4, the structure will behave like a flexible diaphragm, on the other hand if height to length ratio is greater than 4, walls will be slender and flexural forces will cause deformations on the edges of the walls. For confined masonry buildings, maximum height should be 15 m and maximum number of floors should be 5.

Considering the required material properties in the code, steel bars should be able to elongate minimum 9% for ductility. Steel yield stress is taken as 412 MPa in the standard for the calculation of reinforcement amount. Units with a net area less than 70% of the gross area are defined as hollow masonry units. If this ratio is more than 70%, they are called solid masonry units. Tubular masonry units are the ones that have voids parallel to the bed face. The units referred in the standard are clay, lime-silica and concrete blocks or bricks. Type of masonry that can be used in load bearing masonry changes according to seismic zone and number of floors of the building.

For fabricated masonry units, 10 units will be randomly chosen for testing the compressive strength form groups of maximum 50 units. If the results show more than 20% scatter, another sample of 10 specimens is tested; if the dispersion results persist, the lot is rejected. Absorption of clay and lime-silica units cannot be greater than 22%. Mortar should be composed of cement, hydrated lime and fine aggregates.

Concrete compressive strength should be equal or more than 17.15 MPa (175 kg/cm^2). Plain bars can only be used in stirrups or horizontal reinforcement. Construction of confining elements should be made after the masonry walls are built. Connection between the wall and column can be realized by a toothed connection or horizontal reinforcement placed in the wall-column interface (see Figure 2.9). If a toothed connection is preferred, the projected length of a tooth cannot exceed 5 cm. If reinforcement is used, at least a single bar should be embedded 40 cm into the wall and 12.5 cm into the column with a 90 degree hook of 10 cm. Thickness of the mortar joint should not be less than 10 mm and more than 15 mm, or twice the dimensional tolerance of height for the units.

Overlap of the vertical or horizontal reinforcement should be at least 45 times the largest diameter of the overlapped bars. The minimum concrete cover on the reinforcement should be 2 cm for elements subjected to atmosphere, and 3 cm for elements covered with earth.

According to the Peruvian code, for a single building, the compressive strength and shear strength of masonry walls can be determined by two methods. Determination of masonry strength from brick and mortar strength is called as “method A”, and determination by performing experiments on masonry prisms and diagonal compression tests on masonry wallets is named as “method B”. Methods that can be used according to seismic zone and number of floors is indicated in Table 7 of the code. As the number of floors increase and the seismic zone is more severe, method B should be used to determine safer results. 28 day strength values should be considered for the tests performed. If the strength values are going to be determined by method A, values should be taken from Table 9 of the code. In any case, value of diagonal shear strength cannot be greater than $0.319\sqrt{f_m}$.

Figure 2.9. Reinforcement for wall-column connection

2.7.1.1. Specific Rules for CM Construction in the Peruvian Code

For a confined masonry wall, maximum distance between tie-columns should be two times the distance between tie-beams. The distance between tie-columns cannot exceed 5 m. The first condition is to protect the wall against torsional effects. The second rule is for not losing the confinement effect in the mid portion of the wall. Minimum thickness of columns and beams should be equal to effective wall thickness. Minimum height for the beam should be equal to the height of the slab. Minimum width of tie-columns should be 15 cm.

2.7.2. Design Calculations

According to the code, slabs should be designed as rigid diaphragms that are able to transfer the loads directly into surrounding walls. Greater dimension of a slab should not be more than 4 times the shorter dimension to satisfy this condition.

Load bearing walls and masses should be distributed equally and symmetrically in the two main directions of the building to avoid torsional effects. Load bearing walls should be continuous from foundation to the roof level. Walls that have a length equal or greater than 1.2 m should be considered as a load bearing wall. Window sills should be isolated from the adjacent walls by an isolation joint, because the stress concentrated on the corners of the opening causes cracking of the wall. Also, it reduces the effective height of the wall and increases rigidity so that lateral loads resulting from an earthquake increases for that wall. Control joints should be provided at every 8 m in concrete unit walls and 25 m in clay brick walls. Walls should be braced according to the conditions described in the code to transfer the out of plane effects into the surrounding structural elements. Tie-columns with toothed connection and tie-beams are accepted as a bracing. Load bearing walls should have a minimum effective thickness of h/20 for seismic zones 2 and 3 and h/25 for seismic zone 1.

Maximum axial compressive load that can be applied to a load bearing wall is defined by Equation 2.35. Maximum gravity loads on service and live loads should be added to determine the axial load acting on the wall.

$$\sigma_m = \frac{P_m}{L.t} \leq 0.2\, f_m \left[1 - \left(\frac{h}{35t}\right)^2\right] \leq 0.15 f_m \tag{2.35}$$

Equation 2.34 prevents the buckling failure of the wall. The limit 0.15 fm prevents the reduction of ductility when the wall is subjected to severe seismic loads. For the confined masonry, the axial force can be calculated using transformed section. Equivalent area of concrete can be calculated by multiplying concrete area by Ec/Em. In this way, axial stress is reduced on the section; but in any case $\frac{\mathrm{P_m}}{\mathrm{L.t}} \leq 0.15\mathrm{f_m}$ condition must be satisfied.

If there is a concentrated load acting on the wall, the axial stress produced by the concentrated load should not exceed 0.375fm in order to prevent crushing. To determine the axial stress caused by the concentrated loads, in the case of a beam passing through the midspan of the wall, the local stress created by the beam can be calculated considering the load is concentrated over the length of the beam added by two times the thickness of the wall on each side. All load bearing walls that carry more than 10% of the seismic force and walls in the perimeter of the building in seismic zone 2 and 3 should be “reinforced” which means they should be either fully grouted reinforced masonry or confined masonry walls. In seismic zone 1, at least perimeter walls should be reinforced. Minimum density of reinforced walls in a building is defined with Equation 2.36. Equation 2.36 represents only the minimum density of walls to be “reinforced”, the actual amount should be determined according to seismic analysis.

$$\frac{\text{Shear area of reinforced walls}}{\text{Plan area}} = \frac{\sum \text{Lt}}{\text{Ap}} \geq \frac{\text{Z.U.S.N}}{56} \quad (2.36)$$

In Equation 2.35, N is the number of floors of the building, L is the total length of the floor, t is the effective thickness of the wall; Z, U and S represents the seismic zone, building importance, and soil class factors, respectively.

2.8. Comparison of Code Regulations

In this section, a brief review of examined code regulations regarding URM and CM construction is carried out. The regulations will be compared under the headings of building dimensions, material quality, wall dimensions, placement of confining elements and detailing of confining elements. In TEC07, URM construction is permitted, where there is not a specific chapter for CM construction. In Eurocode and Mexican code, URM and CM design rules are given in detail on spate sections. In Algerian, Peruvian and Chilean codes only URM construction is permitted.

Table 2.8. Masonry types in code regulations.

The Code	Masonry Type
TEC07	URM, CM partially included in URM
Eurocode	URM, CM, RM
Algerian	CM
Mexican	URM, CM, RM
Chilean	CM, RM
Peruvian	CM, RM

2.8.1. Building Dimensions

TEC-07, the Algerian and the Peruvian codes enforce limitations for building dimensions. Number of stories and building height is limited due to highly nonlinear and brittle nature of masonry components. Length to width ratio of the building is also limited to prevent irregularities. In Eurocode, the Mexican and the Chilean codes, there is no strict limitation on building dimensions. But there are limitations in Eurocode for the so called "simple masonry buildings" in the code, for which no safety verification is needed. The Mexican code also puts forward a simplified analysis, which can be used for simple buildings that have certain limitations. The limitations defined in the codes are compared in Table 2.9.

Table 2.9. Comparison of the regulations regarding building dimensions.

The Code	Max. height	Number of stories	Minimum wall	Length/ Width
TEC07	3m (storey height) (for adobe, 2.7 m)	2 at zone 1; 3 at zone 2,3; 4 at zone 4	≥0,2I	-
Eurocode	-	-	-	-
Algerian	17 m at zone 1; 14 m at zone 2; 11 m at zone 3	5 at zone 1; 4 at zone 2; 3 at zone 3	≥4% of total floor area	<3.5
Mexican	-	-	-	-
Chilean	-	-	-	-
Peruvian	15 m	5	-	<4

2.8.2. Material Quality

Minimum required compressive strength of some materials used as masonry units are given in Table 2.10. It is observed that similar values are given by different codes in terms of material strength.

Table 2.10. Comparison of strength values (in MPa) in different codes

The Code	Masonry unit	Mortar	Concrete
TEC-07	5 10 (for nat. stones)	-	C16
Eurocode	5 (vertical) 2.5 (horizontal)	5	-
Algerian	-	5	15
Mexican	6	4	15
Chilean	Related norms	5	Related norms
Peruvian	-	-	13.72

2.8.3. Wall Properties

Parameters like thickness/height ratio, minimum thickness, maximum area, length or height of wall panels for confined masonry and maximum unsupported length of unconfined masonry are defined in all the codes. Mexican code accepts any wall as a load bearing wall that has a thickness of 10 cm. Other codes have varying values. In Eurocode and the Chilean code, there is a minimum length criterion for a wall to be considered as a load bearing wall. A full comparison is carried out in Table 2.11.

Wall stiffnesses are calculated according to their shear capacity in TEC07 by a simple formula. In the other codes considered in this study, it is stated that both the shear and flexural properties of the wall should be considered in calculation of the wall stiffnesses. Effect of openings to the wall stiffness is also an important issue.

Table 2.11. Wall Dimensions

The Code	**Thickness(cm)**	**Height(m)**	**Length(m)**
TEC-07	20-30 (Table 5.6) (URM)	3 m, 2.7 for adobe (URM)	5.5 at zone 1 (URM) 7.5 in others (URM) 4 m (with tie col.)
Eurocode	35 cm (stone) 24 cm (URM/CM) 17 cm (URM with low seismicity)	$\leq 9\ t_{ef}$ (stone) $\leq 12\ t_{ef}$ (URM) $\leq 15\ t_{ef}$ (CM and URM with low seismicity)	≥0.5h (stone) ≥0.4h (URM) ≥0.35h (URM with low seismicity) ≥0.3h (CM)
Algerian	20 (CM)	Height×Length≤20 m^2	
Mexican	10 (URM, CM)	≤30t and 3 m (CM)	≤4 m (CM)
Chilean	14 (CM)	≤25t (CM)	$\left.\begin{array}{l}\leq 25t \\ \leq 6 \\ \text{area} \leq 12.5\ m^2\end{array}\right\}$ (CM)
Peruvian	h/20 (zones 2,3) (CM) h/25 (zone 1) (CM)	-	$\left.\begin{array}{l}\geq 1.20 \\ L<2h\end{array}\right\}$ (CM)

2.8.4. Locations of Confining Elements

Confining elements should be cast around the wall panels for confined masonry buildings. But TEC-07, Eurocode and the Mexican code also force the use of tie-beams in unconfined masonry buildings. The maximum distance between the tie-columns and tie-beams are limited in codes to have the desired confinement effect and reduce the slenderness of the walls. The rules regarding the placement of confining elements are indicated in Table 2.12.

2.8.5. Dimensions and Reinforcement Details of Confining Elements

Ideally, the thickness of tie-columns should not be less than the wall thickness. The only exception is the Algerian code, in which it is allowed to be two-thirds of the wall thickness accounting for a wall cover. Minimum dimensions of confining elements should not be less than 150 mm in Eurocode, the Algerian and the Peruvian code, and not less than 200 mm in TEC-07 and the Chilean code. The Mexican code is the most relaxed code and with the minimum wall thickness 10 cm, confining elements can be at the same size.

Table 2.12. Comparison of the rules regarding the placement of confining elements

The Code	Tie-columns	Tie-beams
TEC-07	4 m intervals (total wall length<16m)	3 m 2.7 (adobe)
Eurocode	4 m intervals (CM) Opening area >1.5 m^2	4 m (URM) 4 m (CM)
Algerian	5 m intervals Openings height>1.8 m	5 m
Mexican	4 m (URM) ≤ 4 m or 1.5 h (CM) (around openings)	<3 m (for CM) around openings
Chilean	6 m intervals (around openings)	area≤12.5 m^2
Peruvian	5 m intervals L<2h	-

For detailing of the reinforcement, generally 4 bars of minimum reinforcement are required in most of the codes for either tie-columns or tie-beams. In Eurocode, the amount of reinforcement is defined as a ratio of cross sectional area, whereas in the Mexican code; it is defined by an equation and at least three bars. Stirrups are generally used with maximum 200 mm intervals. Amount of transverse reinforcement is enforced by an equation in the Mexican code and with a maximum interval of 200 mm. The rules in different codes are compared in Table 2.13.

Overall, it can be stated that basic geometrical limitations are dictated by all the codes for design and analysis of masonry structures. The reason is that it is very difficult to standardize the design and analysis of masonry structures since the mechanical behavior is unpredictable due to its highly nonlinear, non-homogeneous and anisotropic nature and also due to variable and local conditions in the production of materials, construction of the structure, etc. Hence, in order to keep the design and analysis rules simple yet reliable, conservative simplifications and assumptions are used for masonry construction. However, this impairs the flexibility in design and analysis of this structural type since the rules dictate prototype masonry construction where almost no additional architectural features allowed. This is the main issue that halts the construction of new masonry structures in most parts of the world.

The regulations are more strict for URM construction since its structural performance is known to be inferior when compared to CM construction. However the number of regulations or limitations is more for CM construction due to the additional elements that have to be considered during design or analysis, i.e. tie beams and tie columns. Following the codes of the South American countries, it is seen that they have nearly abandoned the vulnerable URM construction, at least in earthquake prone regions.

They seem to have established a complete recipe for the construction of CM buildings. However, this is not the case for the Turkish code. It still promotes the URM construction, giving unsatisfactory consideration to CM construction with only few rules to relax the strict regulations of URM construction. However, the discussions in this chapter indicate that more emphasis should be given to CM construction in high seismicity zones whereas URM can still be used for low-seismicity regions. The following chapters aim to exhibit the superiority of CM construction over URM construction through analysis, starting from the component level and extending to the structural level.

Table 2.13. Comparison of the rules regarding the dimensions and reinforcement details of the confining elements.

The Code	Tie-columns		Tie-beams	
	Dimensions mm	**Reinf.**	**Dimensions mm**	**Reinf.**
TEC-07	200t	4ϕ12 6ϕ12 (stone) ϕ8/200 mm stirrups	200t	4ϕ10 6ϕ10 (stone) ϕ8/250 mm stirrups
Eurocode	≥0.02 m^2 ≥150 mm	≥%1 of area ≥300 mm^2 ϕ5/150 mm stirrups	≥0.02 m^2 ≥150 mm	≥%1 of area ≥300 mm^2 ϕ5/150 mm stirrups
Algerian	150x150	4ϕ10 Stirrup/250	≥150 ≥2t/3	4ϕ10 Stirrup/250
Mexican	URM:50x50 CM:100x100	URM:2 bars $\geq \frac{2V_{mR}}{3F_R f_y}$ Stirrup: $\geq \frac{2V_{mR}}{3F_R f_y} \frac{H}{S}$ CM:3 bars $\geq 0.2 \frac{f_c'}{f_y} t^2$ Stirrup/200 mm Or $\geq \frac{10000s}{f_y h_c}$	URM:50x100 b CM:100x100	URM:2 bars $\geq \frac{2V_{mR}}{3F_R f_y}$ Stirrup: $\geq \frac{2V_{mR}}{3F_R f_y} \frac{H}{S}$ CM:3 bars $\geq 0.2 \frac{f_c'}{f_y} t^2$ Stirrup/200 mm or 1.5t Or $\geq \frac{10000s}{f_y h_c}$
Chilean	200t	4ϕ10 Stirrup: ϕ6/200/100	200t	4ϕ10 Stirrup: ϕ6/200/100
Peruvian	150t	-	txt_f	-

CHAPTER 3

SUMMARY OF FINDINGS FOR THE SEISMIC BEHAVIOR OF INDIVIDUAL MASONRY WALLS

3.1. Introduction

This study is intended to be a complement of the research work by Erköseoğlu (2014), which concentrated on the seismic performance of individual URM and CM walls. The study by Erköseoğlu conducted a literature survey about experimental data on URM and CM wall specimens. In addition, the empirical formulas found in literature were collected. The behavior of URM and CM walls were idealized according to the piece-wise linear model of Tomazevic (1999) as seen in Figure 3.1. That model defines the behavior of URM and CM walls in terms of three limit states which are the cracking, maximum resistance and ultimate deformation. The aim of Erköseoğlu's study was to gather the experimental data on wall specimens together with the formulas available in literature used to define the three limit states and then find the best matching formulas that defines the three limit states compared to the experimental data. Finally a parametrical study on URM and CM walls was carried out with the selected formulas to represent the three limit states. The selected parameters were material properties, geometrical properties and loading on the wall. Erköseoğlu's study is summarized in this chapter since it forms the basis of this study with the seismic performance information of CM and URM walls.

In this study, the formulations proposed by Tomazevic (1999) were used to evaluate the three limit states of URM and CM walls. The validity of these formulas have been justified in different studies. (Salmanpour et. al. (2013), Ahmad et al.(2010), Brzev (2007), Abo-El-Ezz et al. (2013))

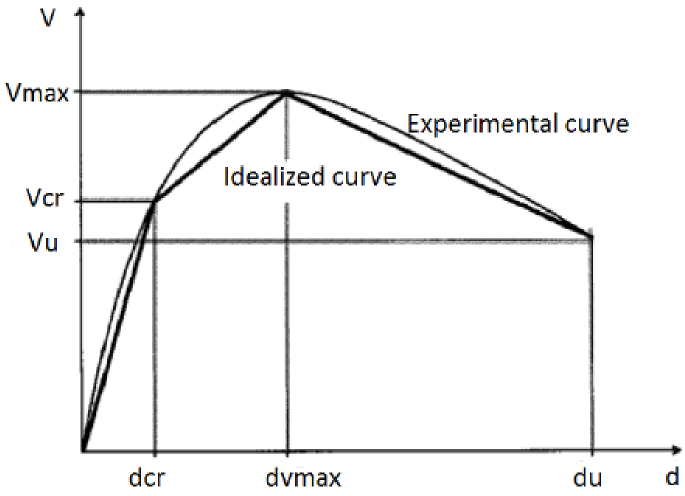


Figure 3.1 Trilinear idealization of capacity curve for a masonry wall (Tomazevic, 1999)

3.2. Data Collection and Analysis Regarding Single Masonry Walls

In Erköseoğlu’s study (2014), first literature survey was carried out to obtain experimental data regarding the tests on individual wall specimens with lateral loading applied for both URM and CM walls. The specimens of the selected tests did not have any openings or reinforcement inside the masonry. Specimens were tested under monotonic or cyclic lateral loads. The data inconvenient with the trilinear capacity curve of Tomazevic (1999) was eliminated. Finally, 20 unreinforced and 23 confined masonry test results were used in the study by Erköseoğlu (2014) as seen in Tables 3.1 and 3.2.

Table 3.1 Reference list for URM walls used in Erköseoğlu's study (2014)

WALL CODE IN	REFERENCE STUDY	WALL CODE IN
URM-W1	Tomazevic et al. (1997)	B1
URM-W2	Tomazevic et al. (1997)	B2
URM-W3	Tomazevic et al. (1997)	B3
URM-W4-1	ESECMASE (2007)	7.1a.2
URM-W5	ESECMASE (2007)	7.1a.3
URM-W6-1	ESECMASE (2007)	7.1a.8
URM-W7	ESECMASE (2007)	7.1a.1 (-)
URM-W4-2	ESECMASE (2007)	7.1a.2 (-)
URM-W8	ESECMASE (2007)	7.1a.6 (-)
URM-W6-2	ESECMASE (2007)	7.1a.8 (-)
URM-W9	Yoshimura et al. (2004)	3D-L0-H0VO-48-1
URM-W10	Yoshimura et al. (2004)	2D-L0-H0VO-84-1
URM-W11	Yoshimura et al. (2004)	3D-L0-H0VO-84-1
URM-W12-1	Magenes et al. (2008)	CL04 (+)
URM-W12-2	Magenes et al. (2008)	CL04 (-)
URM-W13-1	Magenes et al. (2008)	CL05 (+)
URM-W13-2	Magenes et al. (2008)	CL05 (-)
URM-W14-1	Magenes et al. (2008)	CL07 (+)
URM-W14-2	Magenes et al. (2008)	CL07 (-)
URM-W15	Magenes et al. (2008)	CL09 (+)

Table 3.2 Reference list for CM walls used in Erköseoğlu's study (2014)

WALL CODE IN ERKÖSE-OĞLU'S STUDY	**REFERENCE STUDY**	**WALL CODE IN REFERENCE**
CM-W1	Tomazevic et al. (1997)	A1
CM-W2	Tomazevic et al. (1997)	A2
CM-W3	Tomazevic et al. (1997)	A3
CM-W4	Aguilar et al. (1996)	M0
CM-W5-1	Perez-Gavilan et al. (2012)	M1
CM-W5-2	Perez-Gavilan et al. (2012)	M2
CM-W6	Yanez et al. (2004)	1-1(+) (concrete)
CM-W7	Yanez et al. (2004)	1-2(+) (concrete)
CM-W8-1	Yanez et al. (2004)	1-1(+) (clay)
CM-W8-2	Yanez et al. (2004)	1-2(-) (clay)
CM-W9-1	Yanez et al. (2004)	1-1(+) (clay)
CM-W9-2	Yanez et al. (2004)	1-2(-) (clay)
CM-W10	Gouveia et al. (2007)	W2.4.
CM-W11	Yoshimura et al. (1996)	4-HOVO
CM-W12	Bourzam et al. (2008)	CM30J-1
CM-W13	Bourzam et al. (2008)	CM30J-2
CM-W14-1	Marinilli et al. (2004)	M1 (+)
CM-W14-2	Marinilli et al. (2004)	M1 (-)
CM-W15	Yoshimura et al. (2004)	2D-L1-H0VO-48-1
CM-W16	Yoshimura et al. (2004)	3D-L1-H0VO-48-1
CM-W17	Yoshimura et al. (2004)	2D-L1-H0VO-84-1
CM-W18	Yoshimura et al. (2004)	2D-H1-H0VO-84-2
CM-W19	Yoshimura et al. (2004)	3D-L1-H0VO-84-1

Then the test results were compared with the empirical formulas found in literature. For URM walls, only formulas by Tomazevic were able to represent the required three limit states for the capacity curve. As stated in Erköseoğlu (2014), the first step in the determination of three limit states for an URM wall is the estimation of maximum strength. Maximum strength of an URM wall is evaluated as the minimum of three failure modes: diagonal tension (V_{dt}), sliding shear (V_{ss}) and flexure (V_{fl}).

$$V_{dt} = A_W \frac{f_t}{b} \sqrt{\frac{\sigma_y}{f_t} + 1} \tag{3.1}$$

$$V_{ss} = A_W \left(f_{vo} + \mu \sigma_y \right) \tag{3.2}$$

$$V_{fl} = \frac{M_{ru}}{\alpha h} \tag{3.3}$$

In these formulations, A_w is the cross-sectional area of the wall, h is the height of the wall, α is the constraint parameter, b is the shear stress distribution factor, f_t is the tensile strength of masonry, μ is the coefficient of friction between masonry unit and mortar, σ_y is the vertical stress on the wall and f_{vo} is the shear bond strength at zero compression. M_{ru} represents the flexural resistance moment of the wall and can be defined by Equation 3.4.

$$M_{ru} = \frac{\sigma_y L^2 t}{2}\left(1 - \frac{\sigma_y}{f_m}\right) \tag{3.4}$$

In Equation 3.4, L is the wall length, t is the wall thickness and f_m is the compressive strength of masonry. The next step is the determination of the cracking strength in relation to the maximum strength.

$$V_{cr} = C_{cr} V_{max} \tag{3.5}$$

C_{cr} is a coefficient defined by Tomazevic (1999) and taken as 0.7 in the study of Erköseoğlu (2014). Ultimate strength is obtained in a similar way.

$$V_u = C_{sr} V_{max} \tag{3.6}$$

The value of C_{sr} is taken as 0.8. This means that when the strength of the URM wall drops below 80% of the maximum strength as the deformation increases, the load bearing capacity of the wall is no longer reliable, and the wall can be assumed to have failed. After the strength values are obtained, the displacement values corresponding to three limit states should also be found. Up to cracking strength level, wall is assumed to behave elastically, so the elastic stiffness (K_e) is calculated for this region to determine the displacement value corresponding to the cracking strength.

$$K_e = \left(\frac{H^3}{\beta E_m I_w} + \frac{\kappa H}{G_m A_w} \right)^{-1} \tag{3.7}$$

Displacement value can be obtained by dividing cracking strength to elastic stiffness.

$$\delta_{cr} = \frac{V_{cr}}{K_e} \tag{3.8}$$

After cracking of the wall, behavior of masonry wall is no longer in the elastic range. Thus stiffness degradation initiates in the wall. Tomazevic (1999) related secant stiffness values at maximum and ultimate displacements to elastic stiffness by an empirical formula.

$$K_d = K_e - \sqrt{c_1 I_d - c_2} \tag{3.9}$$

In Equation 3.9, Kd is the secant stiffness. Parameter Id is a damage index defined by Tomazevic (1999). Value of the damage index is taken as 0.5 for maximum strength and 0.75 for ultimate strength by Erköseoğlu (2014). Coefficients c_1 and c_2 were adjusted according to the experimental results. Erköseoğlu used the values of 1.28 and 0.32 for these constants respectively for URM walls considering the experimental data used in the study. After the calculation of secant stiffness values at the maximum and ultimate strength levels, determination of displacement values corresponding to these points are carried out in a similar way.

$$\delta_{\max} = \frac{V_{\max}}{K_d} \tag{3.10}$$

$$\delta_{ult} = \frac{V_{ult}}{K_d} \tag{3.11}$$

In these equations, δ_{max} is the displacement value at the maximum strength and δ_{ult} is the displacement value corresponding to ultimate strength. For the CM walls, several different formulas found in literature were compared with the considered experimental results. The closest match with the experimental values was obtained by the formulation of the masonry seismic code of Argentina (INPRES-CIRSOC 1983). Accordingly, the maximum strength of CM wall in this code is given as

$$V_{\max} = \left(0.6 v_m + 0.3 \sigma_y\right) A_w \tag{3.12}$$

where v_m denotes the basic shear strength of the masonry. For the cracking and ultimate strength, Equations 3.5 and 3.6 were used. C_{cr} and C_{sr} values were taken as 0.7 and 0.8 again.

Displacement values corresponding to the three limit states were found in a similar way by Equations 3.7, 3.8, 3.9, 3.10 and 3.11. The most appropriate values for coefficients c_1 and c_2 were found to be 0.52 and 0.38, respectively.

3.3. Parametrical Study on URM and CM Wall Behavior

After the most convenient formulas that represent the three limit states of Tomazevic's model had been determined, a parametrical study on behavior of URM and CM walls were conducted by Erköseoğlu (2014). Parameters used in the study were compressive strength of masonry (f_m), vertical compressive stress to masonry compressive strength ratio (σ_0/f_m) and the aspect ratio ($h/l= \lambda$). Values used in the parametrical study were f_m =2 MPa (low strength), 5 MPa (moderate strength), 8 MPa (high strength) for masonry compressive strength; λ=0.5 (squat wall), 1.0 (square wall), 1.5 (slender wall) for aspect ratio; and σ_0/f_m=0.05 (low stress), 0.10 (moderate stress) and 0.20 (high stress) for axial compressive stress to masonry compressive strength ratio.

Parametric analyses were assumed to be carried out on a typical wall that is 2 m long and has a 30 cm thickness. Sample results are provided in Figures 3.2-3.4. First, the effect of masonry compressive strength (f_m) was considered. It was observed that increase in maximum strength is proportional to masonry compressive strength for both URM and CM walls. Ultimate displacement values show a similar trend. If URM and CM walls are compared, load capacities are close to each other for low axial stress, but in other cases CM walls have higher strength capacity than URM walls. The strength capacity of CM walls increase as the axial stress level increases. Failure of URM walls seems to occur by flexure under low axial stress but in other cases diagonal tension governs the failure mode. CM walls are all assumed to fail in diagonal tension under specific conditions used in the parametrical analysis. Charts for λ=1.0 and σ_0/f_m=0.05, 0.10, 0.20 are given in Figure 3.2.

If effect of axial stress level is considered, increase in axial stress level increases the maximum stress and ultimate displacement. Rate of increase is higher for CM walls compared to URM walls. This shows that CM walls provide a greater contribution to strength and displacement as the axial stress increases. CM walls have greater strength capacity than URM walls except for the squat walls (λ=0.5). Deformation capacities of CM walls are always greater than URM walls but the difference between URM and CM walls increase significantly with higher axial stress. Charts for f_m=5 MPa and λ=0.5, 1.0, 1.5 are given in Figure 3.3.

Finally, the effect of aspect ratio is considered. Accordingly, variation in the aspect ratio does not make a sensible change in the load capacity for squat (λ=0.5) and square walls (λ=1.0), but when it comes to slender walls (λ=1.5), a significant reduction in load capacity occurs. This reduction is much greater for URM walls. CM walls can keep their load capacity much higher with respect to URM walls. In the range of axial stress used in the parametric analysis, axial stress level does not seem to cause a significant effect on the reduction of load capacity of slender walls. The drift ratios are not really affected by the aspect ratio for both URM and CM walls, since the ultimate displacement values seem to increase proportionally with the increasing height due to the increase in aspect ratio. The failure mode is flexure for square and slender walls under low axial stress, whereas all other walls fail in diagonal tension. Charts for f_m=5 MPa and σ_0/f_m=0.05, 0.10, 0.20 are given in Figure 3.4.

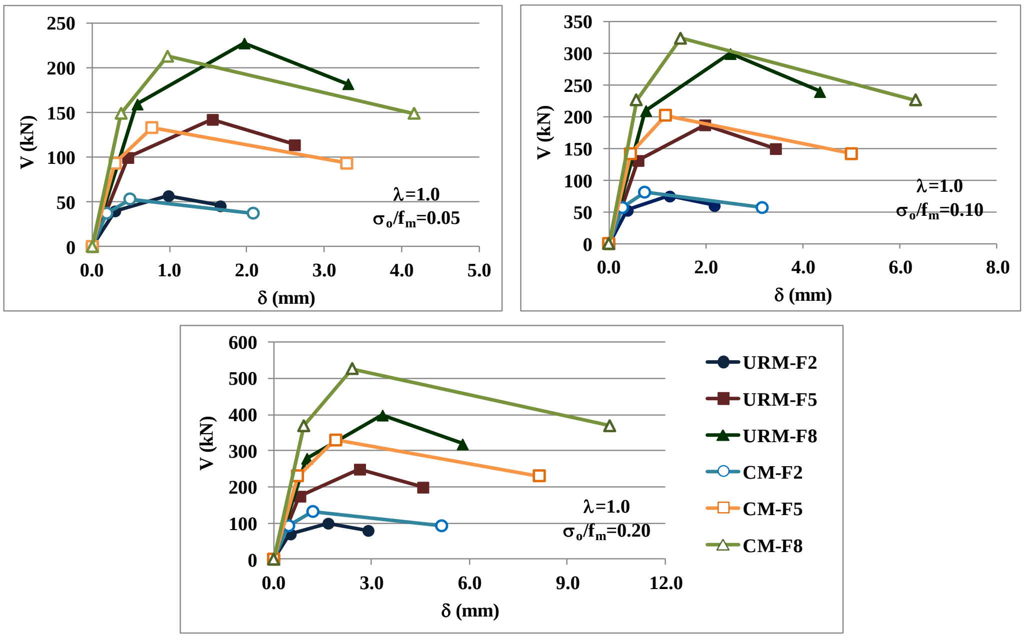


Figure 3.2 Effect of f_m on URM and CM walls for λ=1.0 and σ_0/f_m=0.05, 0.10, 0.20.

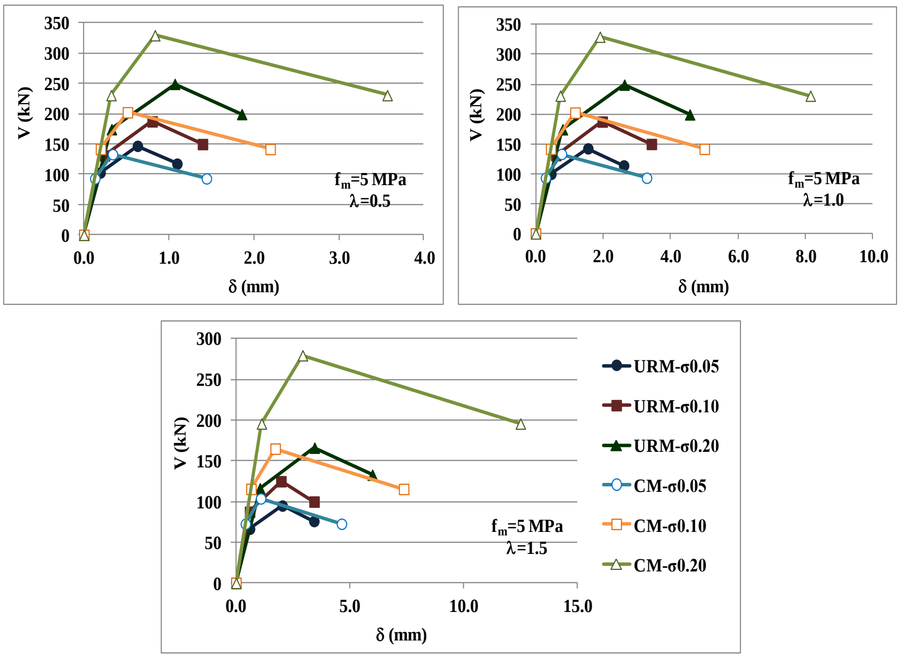


Figure 3.3 Effect of σ_0/f_m on unreinforced and confined masonry walls for f_m=5 MPa and λ=0.5, 1.0, 1.5

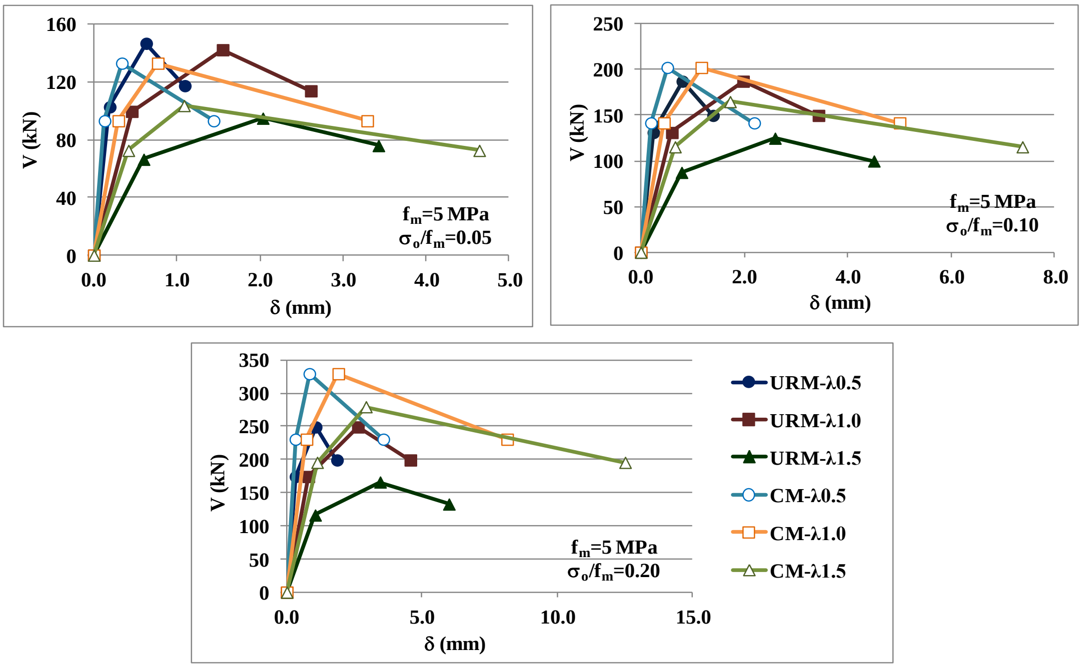


Figure 3.4 Effect of λ on unreinforced and confined masonry walls for f_m=5 MPa and σ_0/f_m=0.05, 0.10, 0.20.

3.4. Wall Behavior Definition for the Building Models

The main goal of this study is to determine the performance state of URM and CM buildings at the critical storey to make a detailed comparison between the two masonry wall types. According to the approach proposed by Tomazevic (1999); storey shear capacity is evaluated as the sum of shear capacities of each wall segment in the storey. At this point, it is important to simulate the behavior of single URM and CM walls in a reliable manner to have a good estimate in building level.

3.4.1. Capacity of an URM Wall

Capacity of an URM wall is determined according to Section 3.1 and theory of Tomazevic's model (1999) and Equations 3.1 to 3.11 that were proposed by Erköseoğlu (2014) as discussed in Section 3.1 are used. Values of c_1 and c_2 are taken as 1.28 and 0.32 as of the study by Erköseoğlu. Exactly the same method and formulations are used.

3.4.2. Capacity of a CM Wall

According to the parametric analysis of Erköseoğlu, it was observed that the selected formula for CM walls; i.e. the formulation enforced by the masonry seismic code of Argentina (INPRES-CIRSOC 1983); did not reflect the contribution of confinement under low axial stress (σ_0/f_m=0.05). The formula itself is only dependent on axial stress and shear strength of masonry. Since the building models that are intended to be analyzed in this study are 2 and 3 storey low-rise buildings, the axial stress levels are even below 0.05 in general. Hence another formulation considered in the study of Erköseoğlu, the one proposed by Tomazevic for CM walls is used. According to Tomazevic, maximum resistance of a CM wall is composed of two parts: contribution of masonry wall and the dowel effect of tie-column reinforcement. Contribution of masonry wall is formulated as

$$V_{u,s} = \frac{f_t \times A_w}{C_i \times b} \left[1 + \sqrt{C_i^2 \left(1 + \frac{V_w}{f_t \times A_w}\right) + 1}\right] \tag{3.13}$$

$$C_i = 2\alpha b \frac{1}{H} \tag{3.14}$$

where $V_{u,s}$ is the shear resistance contribution of the wall in confined masonry, C_i is the interaction coefficient, α is a parameter depending on the assumed shape and distribution of interaction forces and H is the effective height of the wall, V_w is the vertical load on the wall, A_w is the horizontal cross section area of wall, and b is the shear stress distribution coefficient. The dowel effect of the reinforcement in tie-columns is added to the wall panel resistance to find the maximum resistance of a CM wall (Tomazevic, 1999).

$$V_{d,r} = \sum_1^n 0.8059\, d_r^2 \sqrt{f_c\, f_y} \tag{3.15}$$

$$V_{max} = V_{u,s} + V_{d,r} \tag{3.16}$$

In Equations 3.15 and 3.16, $V_{d,r}$ is the shear resistance contribution of the reinforcement in tie-columns of confined masonry, d_r is the diameter of the tie-column longitudinal bar, f_c and f_y are the compressive strength of concrete and yield strength of reinforcement bar.

The cracking resistance is determined in the same manner as a URM wall, it is assumed to be 0.7 times the resistance of the wall panel only, because up to the cracking point the dowel effect of reinforcement is not pronounced.

$$V_{cr} = 0.7V_{u,s} \tag{3.17}$$

The ultimate resistance is related to V_{max} and it is calculated by Equation 3.6. Displacement values of CM walls are calculated exactly in the same way as URM walls.

3.4.2.1. CM Walls with Interior Columns

As tie-columns are inserted into intersections of CM walls, some walls have either one or two interior columns. Behavior of CM walls with interior columns should differ from the ones without any interior columns. Such a comparison was done by Marinilli and Castilla (2004). Models shown in Figure 3.5 were analyzed under constant vertical loads and cyclic lateral loading.

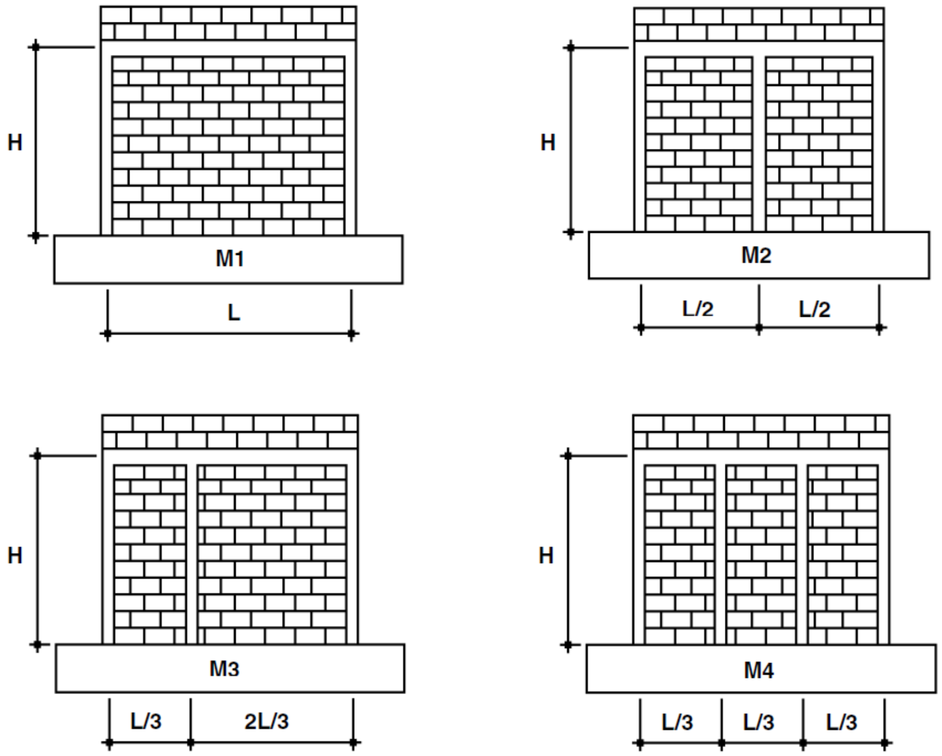


Figure 3.5 Models used in the study of Marinilli and Castilla (2004).

The results given in Table 3.3 were obtained for the cracking, maximum resistance and ultimate displacement states of the CM walls.

Table 3.3 Wall capacity results obtained in the study of Marinilli and Castilla (2004).

Specimen	Cracking strength (kN)	Cracking displacement (mm)	Maximum strength (kN)	Displacement at maximum strength (mm)	Ultimate displacement (mm)
M1	14.71	3.05	20.67	6.08	8.03
M2	16.53	2.77	26.48	10.74	12.15
M3	15.94	2.68	23.31	6.61	7.90
M4	14.73	1.85	29.57	11.08	14.09

Results show that interior columns increase the initial stiffness reducing cracking displacement. Interior columns increase the maximum strength and ultimate displacement thus the energy dissipation capacity.

According to Table 3.3, for a single interior column in the middle of the wall; cracking displacement can be taken as 0.907 times, cracking strength is 1.124 times the one without interior columns. Thus the initial stiffness is 1.24 times greater. Similarly, cracking strength is 1.12 times; maximum resistance is 1.28 times, displacement at maximum resistance is 1.77 times greater than the simple CM wall.

If there are two interior columns equally spaced in the wall, there is not a significant change in cracking strength, but cracking displacement becomes 0.61 times smaller. Thus the initial stiffness is 1.65 times greater. Similarly, maximum resistance is 1.43 times, displacement at maximum resistance is 1.82 times greater than the initial values.

In this study, strength and deformation capacities of CM walls with interior columns are modified according to the test results given above.

CHAPTER 4

BASICS OF THE PERFORMANCE EVALUATION PROCEDURE FOR URM & CM BUILDINGS

4.1. General

This chapter focuses on the details of the performance evaluation procedure employed for URM and CM buildings. The steps of the procedure are explained below with the help of simple flow chart in Figure 4.1.

1) Selection of the critical storey for the building and labeling the wall segments of the critical storey in both principal directions.
2) Determination of the geometrical properties of the wall segments in the critical storey.
3) Determination of the capacity curve for the critical storey in both principal directions by using the methodology proposed by Tomazevic (1999).
4) Conversion of the capacity curve into Acceleration Displacement Response Spectrum (ADRS) format.
5) Identification of response spectrum to represent seismic demand.
6) Conversion of the demand curve into ADRS format.
7) Determination of the performance point according to the Procedure C of FEMA 440 (ATC, 2005)
8) Assessment of the overall performance of the critical storey by using a performance index that considers the states of the wall segments at the previously calculated performance point.

4.2. Selection of the Critical Storey and Labeling of the Wall Segments

The first step in the proposed procedure is the selection of the critical storey of the buildings, for which all the calculations are intended to be carried out. For masonry buildings, this is usually the ground storey due to maximum shear force demand. However, in some cases, intermediate stories can also be regarded as critical stories if there is an abrupt change in stiffness, strength or structural characteristics of the building at this floor level. In this case, the selection may require some engineering experience and judgment, or both stories can be considered as candidate critical storey for which the results will show the more critical one.

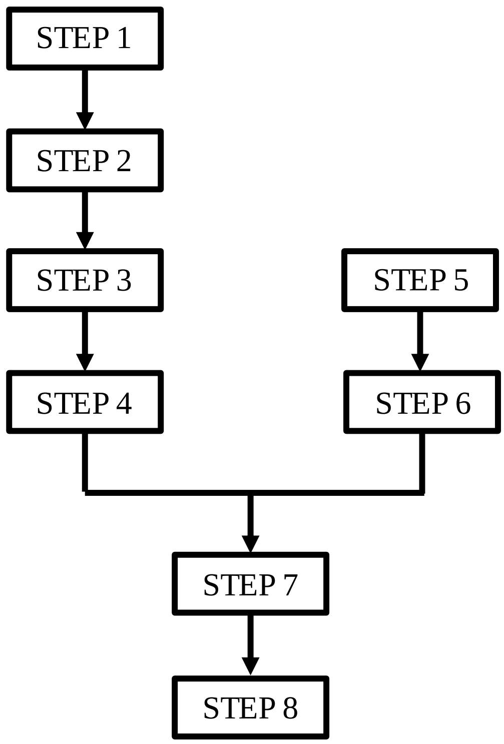


Figure 4.1 Simple flowchart that represents the performance evaluation procedure.

After selecting the critical storey, the structural layout of the storey is used to distinguish the wall segments to be considered in performance assessment analysis in both principal directions. Figure 4.2 shows a sample structural layout in which the wall segments are labeled in both directions. It should be noted that only pier elements between two openings or an opening and a corner are considered as wall segments in this procedure. The wall sections below and above openings are omitted.

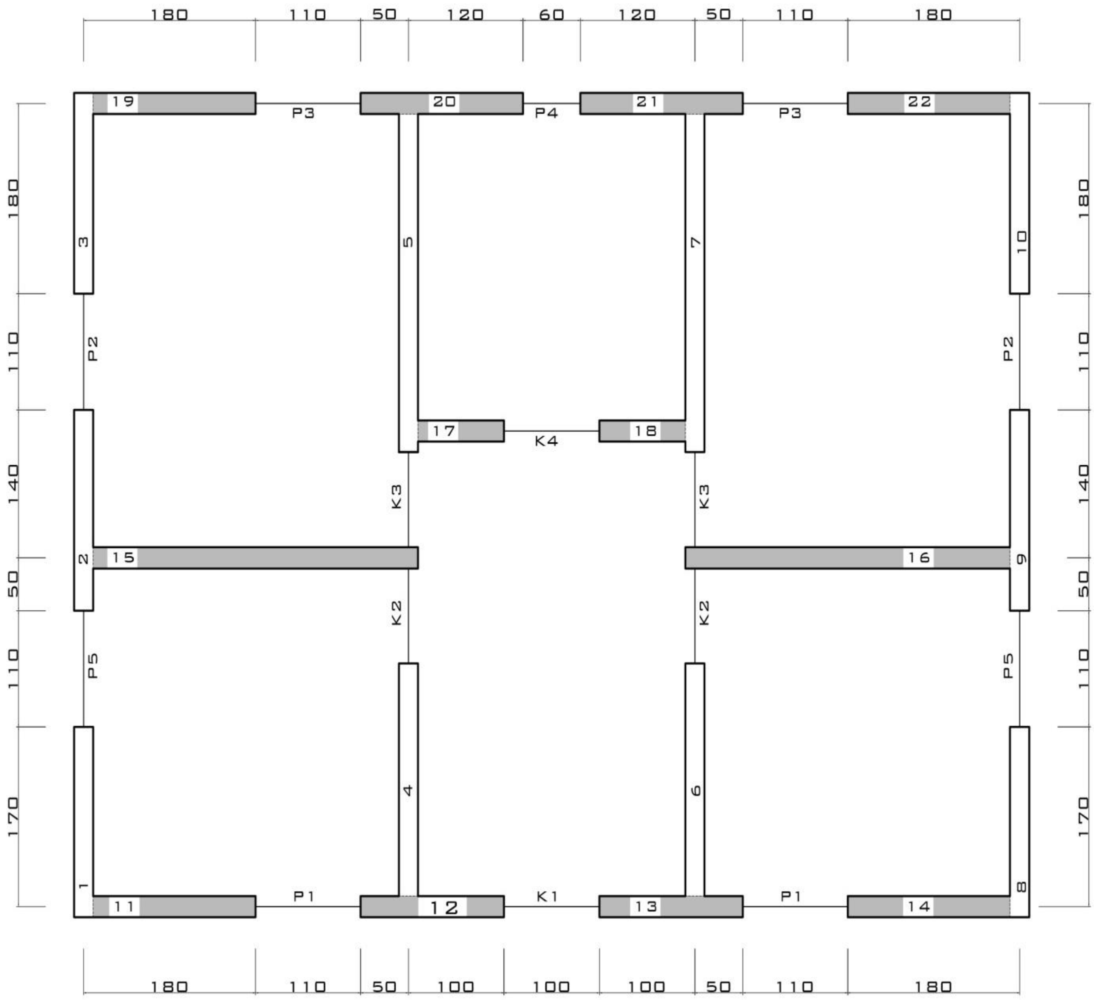
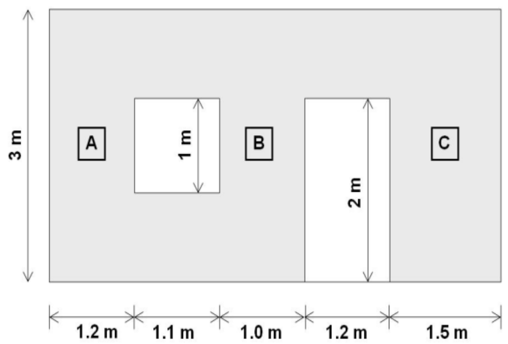


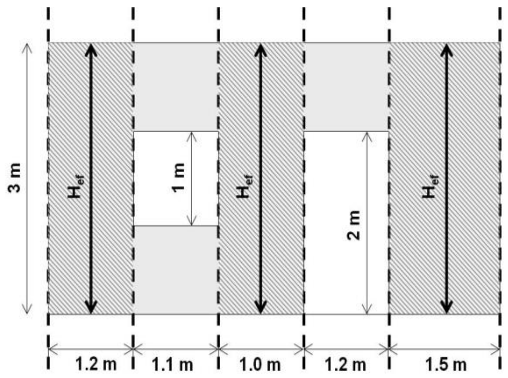
Figure 4.2. Sample structural layout with labeled wall segments.

4.3. Determination of Geometrical Properties of the Wall Segments

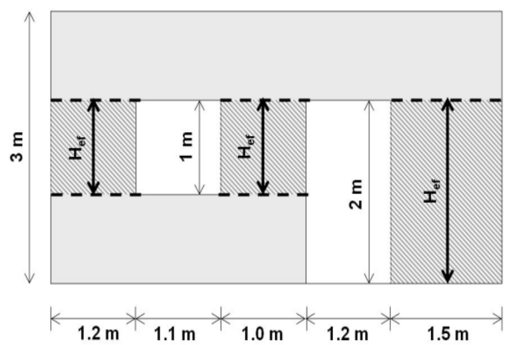
Each wall segment in the critical storey is represented by its length, height and thickness as the major geometrical properties. Among these, thickness can be directly determined if the walls are single or multi leaf walls. For ribbed, cavity and fin walls, it may be necessary to calculate an equivalent thickness. As the second parameter, length of the wall can be easily determined by calculating the horizontal projection between any two openings or an opening and a corner. The most challenging parameter is the height of the wall due to complex arrangement of window and door openings in perforated masonry walls. Different definitions exist in literature for the determination of the equivalent height of the wall segment as can be seen in Figure 4.3. The three alternatives given in this figure consider the full vertical projection between openings (Figure 4.3.b), the shortest height of the adjacent openings, which has also been employed by the current Turkish earthquake code (Figure 4.3.c) and diagonal projections form the corners of openings (Figure 4.3.d) Among those alternatives, the last one seems to be the most appropriate method to define equivalent height of a wall segment. It was proposed by Dolce in 1989, hence generally known as Dolce method. This method suggests that for a solid wall segment between two openings, diagonal projections of 30 degrees at maximum with the horizontal are drawn from the corners of the opening to find the equivalent height as shown in Figure 4.4.



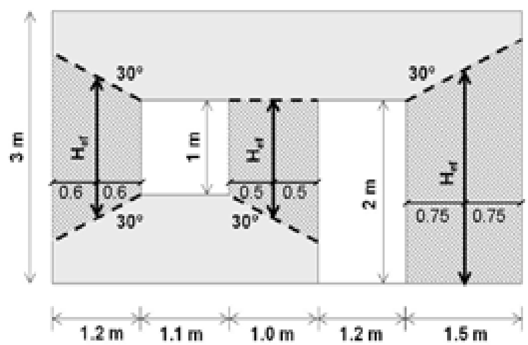
(a)



(b)



(c)



(d)

Figure 4.3 Calculation of equivalent wall height by considering different approaches.

4.4. Construction of the Capacity Curve for the Critical Storey

In this step, using the idealized capacity curves of individual wall segments as explained in Chapter 3, the pushover (or the capacity) curve of the critical storey of a URM or a CM building is constructed by using the method proposed by Tomazevic (1999). According to this method, the capacity curves of individual wall segments in a certain direction of the critical storey are superimposed to obtain the overall envelope. The details of the method are presented in the following paragraphs. In this study, the ground storey is assumed to be the critical storey for the case study masonry buildings, the resulting curves are obtained as base shear force versus roof displacement.

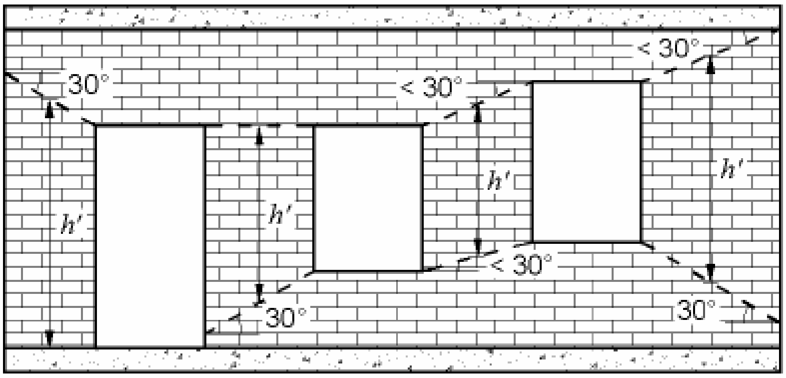


Figure 4.4 The determination of equivalent height according to Dolce (1989) method.

There exist three gross assumptions while applying the method proposed by Tomazevic. First of all, the floors are assumed to be rigid in their own planes so that the lateral forces are distributed to the wall segments in proportion to their relative stiffnesses. In addition, the floor is assumed to go rigid body translation and rotation. These are generally valid in the case of RC floors, but questionable in the case of wooden floors for masonry buildings. In this study, the floors are assumed to be rigid RC floors. The second assumption is that the first mode shape dominates, or in other words, lateral floor displacements can be calculated by using this mode shape. A further simplification for the method is to use inverse triangular distribution instead of the first mode shape since they can be assumed to be very close to each other. In this way, it becomes very easy to relate ground storey displacement to the roof displacement. The third assumption is that the flange effect of orthogonal walls connecting to a load bearing wall is neglected.

A displacement based pushover procedure is applied to the building under consideration to construct the capacity curve. The process is carried out by gradually pushing the building, namely; increasing the displacement by a small increment in each iteration and calculating the storey shear capacity. Displacement in the direction of analysis (denoted as x or y hereafter) is augmented in small increments. In each step, shear capacities of considered wall segments are summed up to yield the storey shear capacity. Torsional moments due to the difference between mass and rigidity centers can be calculated as

$$\boldsymbol{M = TV_X * ymr} \tag{4.1}$$

$$\boldsymbol{M = TV_y * xmr} \tag{4.2}$$

In above equations, TV_x, TV_y are the total shear force capacities of the critical storey, *ymr* and *xmr* are the distance vectors from mass center to rigidity center. Then angle of rotation due to moment is calculated as

$$\boldsymbol{\theta = M/J} \tag{4.3}$$

Where θ is the rotation angle due to displacement d_i, J is the rotational moment of inertia of the storey calculated by

$$J = \sum\left(k_x y^2 + k_y x^2\right) \tag{4.4}$$

k_x and k_y are the in-plane stiffness values of wall segments in the x and y directions. The change in deformation due to the rotation is calculated for each wall. The capacities of wall segments are recalculated according to their final displacement values. By using the final displacements, total shear capacity of the critical storey is obtained. Displacement is increased up to a point that one or more wall segments reach their ultimate displacements, i.e. the point corresponding to 80% of maximum strength. At that point pushover procedure restarts with zero displacement, but the wall segments that have reached to their ultimate displacement in the previous pushover analysis are not taken into account. Multiple capacity curves are plotted and they are combined such that each curve starts at the point where the previous curve ends. At the end a saw-tooth shaped storey shear capacity curve is obtained as described in FEMA 440 (ATC, 2005) as seen in Figure 4.5.

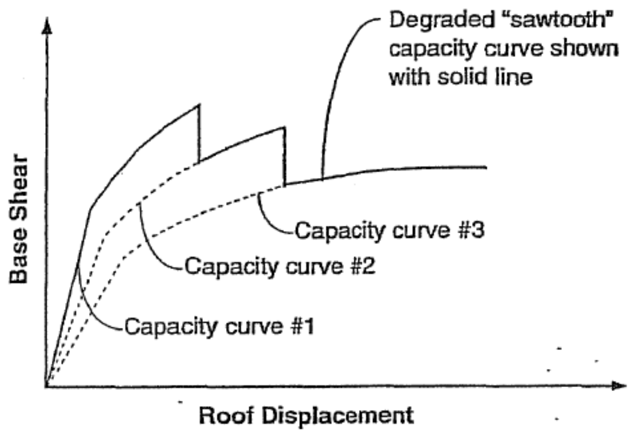


Figure 4.5 Sawtooth shaped capacity curve as defined in FEMA 440 (ATC, 2005)

4.5. Conversion of Capacity Curve to ADRS Spectra

In order to compare seismic capacity and demand, capacity curve needs to be converted into ADRS format (Acceleration Displacement Response Spectra). This format requires using spectral displacement and spectral acceleration values.

To convert the capacity curve into ADRS format, base shear (V) should be converted into spectral acceleration (S_a) and roof displacement should be converted into spectral displacement (S_d) (ATC, 1996). Equations 4.5 and 4.6 are used for this purpose.

$$S_{a1} = \frac{V/W}{\alpha_1} \qquad (4.5)$$

$$S_d = \frac{\Delta_{roof}}{PF * \varphi_{i,roof}} \tag{4.6}$$

In these equations, V represents the base shear force, W represents the total weight (dead+live loads) of the structure, $\propto_1$ represents the modal mass coefficient for the first natural mode, PF represents the modal participation factor, Δ_{roof} represents the roof level displacement, $\varphi_{i,roof}$ represents the amplitude of mode 1 in the roof level. Figure 4.6 shows an illustration for the conversion from capacity curve to ADRS format.

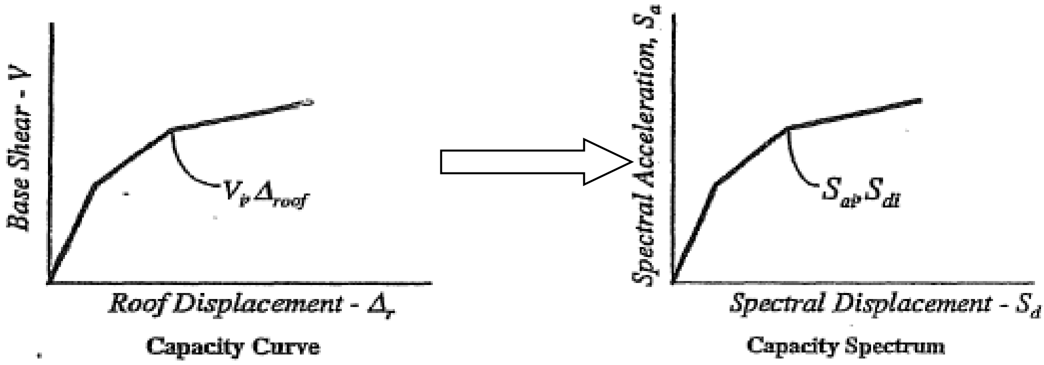


Figure 4.6 Conversion from capacity curve to ADRS format. (ATC, 1996)

4.6. Identification of Response Spectrum for Seismic Demand

Seismic demand can be represented by earthquake response spectra either as presented in seismic design codes or by real earthquake records. Typical design and response spectra are presented in Figure 4.7. The design spectrum is obtained from the statistical analysis of response spectra for an ensemble of ground motion records with specific characteristics; therefore it is composed of a set of smooth curves or a series of straight lines. On the other hand, the response spectrum of an individual ground motion is inherently in jagged (or erratic) nature with random peaks and valleys in the period range. For this reason, finding the location of the performance point with earthquake response spectrum is more challenging in the capacity spectrum method.

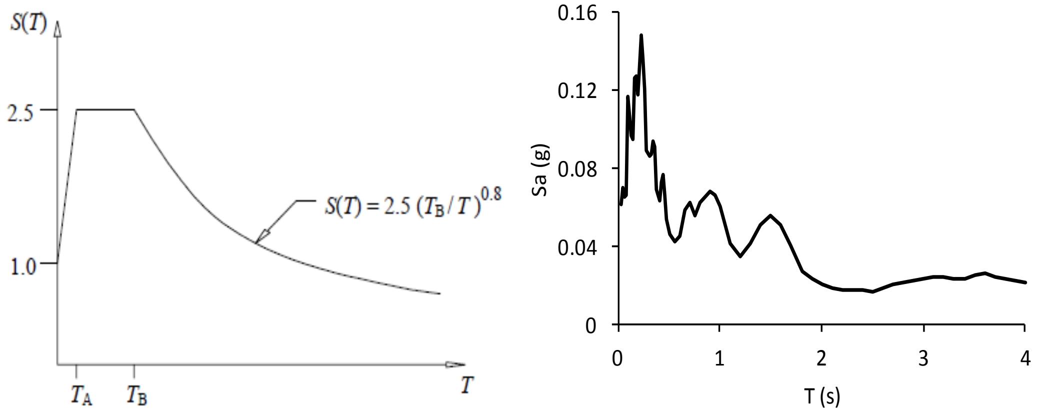


Figure 4.7 Identification of seismic demand in the form of **a)** design spectrum (Turkish Earthquake Code, 2007), **b)** earthquake response spectrum

When design spectrum in seismic codes is employed, seismic zone and soil characteristics for the building site are the main parameters to be identified. According to the Turkish Earthquake Code (2007), elastic spectral acceleration coefficient (A_0) varies from 0.1 to 0.4 with the seismic zone. The corner periods T_A and T_B of the acceleration amplification region (i.e. the plateau) are determined according to the site-soil classes. These soil classes are named as Z1, Z2, Z3 or Z4, where Z4 represents very soft soil conditions. In the case of poor soil conditions, a wider range of structures are subjected to the maximum spectral acceleration as seen in Table 4.1.

Table 4.1 Periods T_A and T_B values according to site class (Turkish Earthquake Code, 2007)

Site Soil Class	T_A (sec)	T_B (sec)
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

4.7 Conversion of Response Spectrum to ADRS Format

For the capacity spectrum method to be implemented, both capacity and demand curves need to be converted into ADRS format. (Figure 4.8).

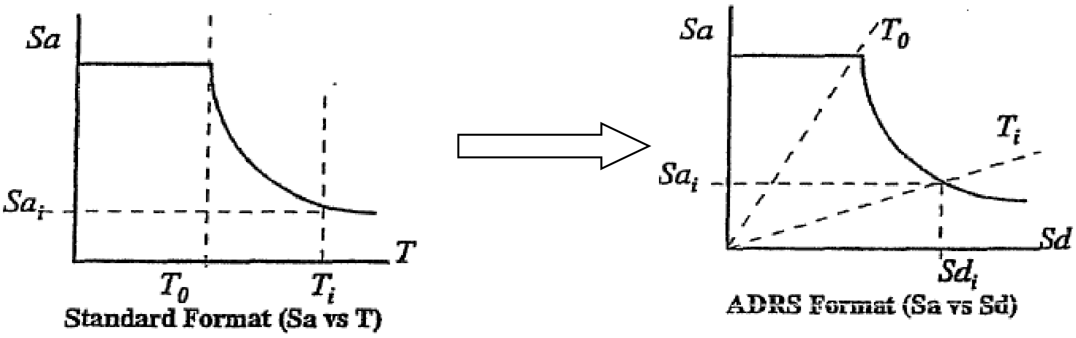


Figure 4.8 Conversion of demand spectrum to ADRS format. (ATC, 1996)

As mentioned in the previous section, seismic demand is represented by earthquake design or response spectrum. The standard format of this spectrum is spectral acceleration (S_a) as ordinate and period (T) as abscissa. To convert the demand spectra into ADRS format, spectral displacement values (S_d) for each point on the S_a vs T curve should be obtained. For any point S_{ai} vs T_i on the curve, S_{di} values can be evaluated by Equation 4.7.

$$S_{d_i} = \frac{{T_i}^2}{4\pi^2} S_{a_i} g \tag{4.7}$$

Spectral acceleration values of ADRS demand spectra should be reduced for the viscous damping inherent in the structure. Initial ADRS demand values are calculated for 5% viscous damping. Foundation effects that can change the behavior of structure in an earthquake are ignored in this study.

4.8. Determination of the Performance Point

For the determination of performance point, Procedure C of FEMA 440 (ATC, 2005) is used. The procedure is based on adjusting initial ADRS (earthquake spectrum) for different levels of ductility (μ). To make this adjustment, first effective damping

(β_{eff}) is calculated according to Equations 4.8, 4.9 and 4.10. For $1.0<\mu<4.0$; Equation 4.8 is used.

$$\beta_{eff} = [4.9 * (\mu - 1)^2 - 1.1 * (\mu - 1)^3 + \beta_0] \tag{4.8}$$

For $4.0<\mu<6.5$; Equation 4.9 is used.

$$\beta_{eff} = [14.0 + 0.32 * (\mu - 1) + \beta_0] \tag{4.9}$$

For $\mu>6.5$; Equation 4.10 is used.

$$\beta_{eff} = 19 * \left[\frac{0.64 * (\mu - 1) - 1}{[0.64 * (\mu - 1)]^2}\right] * \left[\frac{T_{eff}}{T_0}\right]^2 + \beta_0 \tag{4.10}$$

Effective damping (β_{eff}) depends on the hysteretic behavior of the structure and shape of the capacity curve. But the formulas given above are generalized for all kind of hysteretic behavior, so they represent the mean values of different kind of structures and may include some kind of error in this respect.

Effective period values can be calculated using Equations 4.11, 4.12 and 4.13. For $1.0<\mu<4.0$; Equation 4.11 is used.

$$T_{eff} = [0.20 * (\mu - 1)^2 - 0.038 * (\mu - 1)^3 + 1] * T_0 \tag{4.11}$$

For $4.0<\mu<6.5$; Equation 4.12 is used.

$$T_{eff} = [0.28 + 0.13 * (\mu - 1) + 1] * T_0 \tag{4.12}$$

For $\mu>6.5$; Equation 4.13 is used.

$$T_{eff} = \left\{0.89 * \left[\sqrt{\frac{(\mu - 1)}{1 + 0.05 * (\mu - 2)}} - 1\right] + 1\right\} * T_0 \tag{4.13}$$

Equation 4.10 is used for T_0 values between 0.2 and 2.0 sec. Spectral reduction of earthquake spectral acceleration values are determined by Equation 4.14.

$$B = \frac{4}{5.6 - \ln \beta_{eff}\ (in\ \%)} \tag{4.14}$$

where μ is ductility, B is the reduction factor for effective damping, β_0 is the initial damping value. T_0, T_{eff} and T_{sec} are initial, effective and secant period values.

The next step is based on modifying the obtained initial ADRS (earthquake spectra) by the modification factor M to obtain MADRS (Modified Acceleration Displacement Response Spectra). M depends on the initial (T_0), effective (T_{eff}) and secant (T_{sec}) period values. The initial period (T_0) is defined as the period of the structure that is observed in the elastic region until the cracking limit. T_0 can be found by bilinearization method described in FEMA 440. The secant period T_{sec} can be calculated by Equation 4.15.

$$\left(\frac{T_0}{T_{sec}}\right)^2 = \frac{1 + \alpha * (\mu - 1)}{\mu} \tag{4.15}$$

where α is defined as the post elastic stiffness value and can be calculated by Equation 4.16.

$$\alpha = \frac{\left(\frac{a_{p_i} - a_y}{d_{p_i} - d_y}\right)}{\left(\frac{a_y}{d_y}\right)} \tag{4.16}$$

Initial ADRS is reduced by β and modified by M to obtain MADRS. Then T_{sec} line is plotted on the graph. Intersection of MADRS and T_{sec} line is defined as the locus point. Locus points for different levels of ductility are obtained. Finally, a line connecting the locus points are plotted. Intersection of this line with the capacity curve of the structure is denoted as the performance point (Figure 4.9).

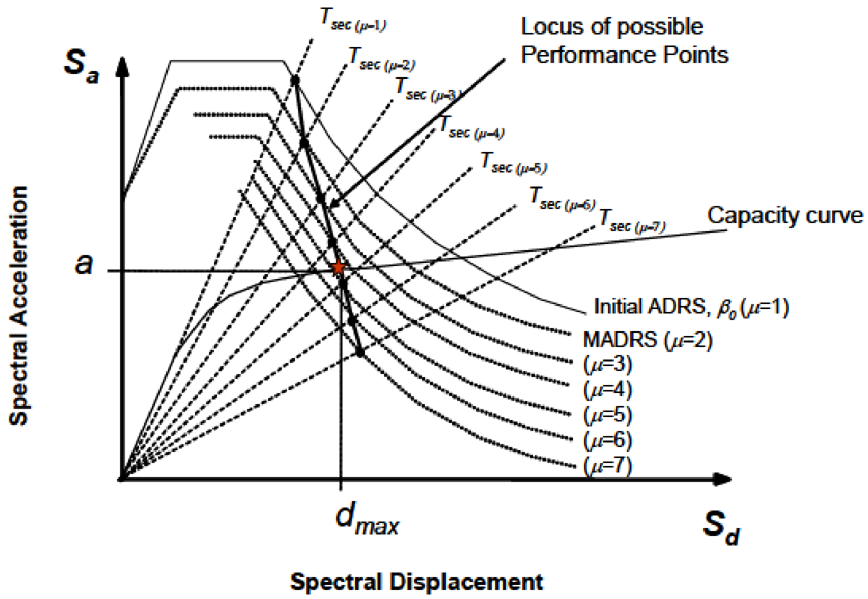


Figure 4.9 Locus of possible performance points using MADRS (ATC, 2005)

4.9 Assessment of Overall Performance by Using an Index

Once the performance points on the storey shear capacity curves are obtained, performance of all the load carrying wall segments in the critical storey level can be obtained. Performances of wall segments are assessed according to the trilinear idealization of Tomazevic (1999) as explained in Chapter 3. For each wall segment, 3 different limit states (LS_i), and therefore, 4 different performance states (PS_i) are defined as shown in Figure 4.10. It should be noted that the limit state definitions are coincident with the displacement limits (i.e. δ_{cr}, δ_{max} and δ_{ult}) discussed in Chapter 3.

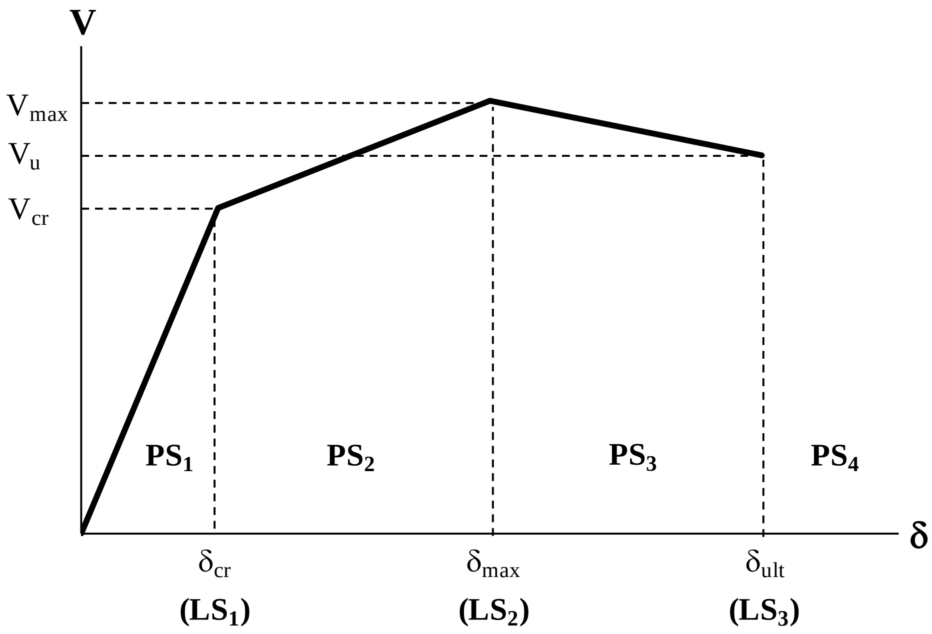


Figure 4.10 Performance states defined for an individual wall segment.

The first performance state (PS_1 in Figure 4.10) represents the behavior of the masonry wall segment before cracking initiates. In this state, the masonry wall behaves in the elastic range. The second performance state (PS_2 in Figure 4.10) represents the behavior of the masonry wall segment that has reached the cracking load, but not the maximum shear resistance. In this state, the masonry wall segment starts to show inelastic behavior but the crack width and propagation is not significant, therefore the damage level is low. The third performance state (PS_3 in Figure 4.10) represents the behavior of the masonry wall segment that has reached its maximum resistance but it has not been pushed to its ultimate deformation capacity yet. Although the crack distribution and damage level is significant, the wall segment is still able to maintain more than 80% of its maximum resistance. And finally, the fourth performance state (PS_4 in Figure 4.10) represents the failure state. Increasing deformations cause the wall segment to lose its integrity. The resistance of the wall segment drops to less than 80% of the maximum resistance. In this state, wall segment is no longer a lateral load carrying member and the loads carried by that member should be distributed to other members.

In order to evaluate the overall performance of the critical story in the building, the information regarding the existing states of all the walls in that story should be gathered. The most appropriate way to determine the overall performance state of the critical storey of a building is to employ a damage index (DI) with weighing factors. For this purpose, the weighing factors (w_i) are assigned to each performance state as indicated in Table 4.2.

Table 4.2 Weighing factors defined for the performance states

Performance state	Weighing factors	
PS_1	w_1	0
PS_2	w_2	1/3
PS_3	w_3	2/3
PS_4	w_4	1

Hence the damage index can be defined as seen in Equation 4.17.

$$DI = \frac{\sum_{i=1}^{n} w_i k_i}{\sum_{i=1}^{n} k_i} \tag{4.17}$$

In this equation, n is the number of wall segments in the direction of analysis; and k_i is the stiffness of the wall segment as given in Equation 3.7. The significance of using the in-plane stiffness of the walls in Equation 4.17 is based on the assumption that the story shear is distributed with respect to the relative in-plane stiffnesses of the walls in that story. Hence as the wall gets stiffer, it attracts larger forces. This increases the importance of that individual wall within that story, which should be reflected in the damage index in an explicit manner.

At the final stage, a single damage score is obtained for the critical story of the considered building as a result of the proposed performance evaluation procedure for URM and CM buildings. This damage score takes values between 0 and 1, for which the lower bound means that all the walls in the critical story behave in their elastic range under the specified seismic action whereas the upper bound means that all the walls in the critical story have failed during the seismic action. Generally, depending on the performance states of the walls, the index takes values in between the bounds.

The procedure explained in this chapter is used to compare the relative performances of generic URM and CM buildings subjected to different levels of ground motion records in the next chapter.

CHAPTER 5

APPLICATIONS OF THE PROPOSED PROCEDURE TO ASSESS SEISMIC PERFORMANCE OF CM AND URM BUILDINGS

5.1 General

This chapter presents the results obtained from the proposed seismic performance evaluation procedure in order to compare the relative performances of URM and CM buildings. In order to achieve this task, 3 generic story plans for each construction type (in total 6 story plans) are developed, in which the geometry and distribution of walls, openings, vertical and horizontal ties are determined in accordance with the seismic design principles as discussed in Chapter 2. The other variable parameters considered for the buildings are the number of stories and compressive strength of masonry (f_m). These story plans represent the critical story of the buildings, for which the procedure is applied. Ten ground motion records from different earthquakes with varying peak ground acceleration (PGA) values are employed to represent different levels of seismic demand for the capacity spectrum method. The results are presented in terms of the damage index (PI) that has been introduced in Chapter 4.

5.2 Generic Story Plans for Building Models

The application of the procedure described in Chapter 4 has been carried out on 12 different masonry building models. The model names are coded in two parts. The first letter stands for the type of construction, i.e. U for URM models and C for CM models, followed by a hyphen. After the hyphen, the next four parameters are introduced in an alpha-numerical way, i.e. each with a letter and a number. The first parameter is number of stories, abbreviated with “N”. In this study, 2 and 3 story building models have been studied, so the coding for number of stories becomes N2 or N3.

Next parameter is related to the wall distribution of the building in plan, classifying models as having “regular” or “irregular” wall distribution. This is abbreviated with “R”. Regular wall distribution means having the masonry walls distributed evenly in plan. Irregular wall distribution means having the masonry walls distributed unevenly in plan, which may create significant torsional effects within the story. As shown in Appendix A, the regular story plans, represented by “R1” in this study, satisfy the above criteria. On the other hand, the irregular story plans, represented by “R2” in this study, have rectangular shapes but the distribution of masonry walls within the story is not uniform, which creates a torsional effect for that building model.

The third parameter is the compressive strength of masonry walls (f_m), which is abbreviated by “F”. In this study, three levels of strength have been considered in accordance with the common construction practice. The values considered are f_m=2 MPa (low strength), f_m=5 MPa (moderate strength) and f_m=8 MPa (high strength). These three levels are classified as “F2”, “F5” and “F8” for the building models.

The final parameter is related to the required length of walls in any principal direction and size and distribution of openings in walls in the critical story. This parameter is abbreviated by “W” and it is considered in three different classes as “W1”, “W2” and “W3”. The definitions of these sub-classes are based on the criteria given in Section 5.4 of TEC-07. The major criterion is the L_d/A ratio (Section 5.4.4 in TEC-07), which is defined as the ratio of the total length of masonry load bearing walls in any of the orthogonal directions in plan to the gross area. A certain limit is enforced in the code for this criterion, which is 0.2 for residential buildings in TEC-07. Other criteria are minor in the sense that they are related to required wall lengths between openings or between openings and corners, etc. in a local sense (Section 5.4.6 in TEC-07). Accordingly, W1 stands for the story plans which do not violate the code principles and possess adequate lateral wall resistance whereas W3 represents the story plans which do not obey most of the code principles and possess inadequate lateral wall resistance.

The story plans given in Appendix A are for all the variants of R and W classes (i.e. R [2 classes] × W [3 classes] = 6 variants for each construction type). The other parameters (N and F) are not directly related with the story plans. In total, there exist 2 (U or C) × 2 (N2 or N3) × 2 (R1 or R2) × 3 (F2, F5 or F8) × 3 (W1, W2 or W3)=72 building models to be analyzed. The building models are coded as (U or C)-N(2 or 3)R(1 or 2)F(2, 5 or 8)W(1, 2 or 3). For instance, a 2-story regular URM building model having f_m=5 MPa and conforming to wall length and distribution criteria given in the code is abbreviated as U-N2R1F5W1.

Each model is analyzed only in their weak direction (x or y). Wall thicknesses are taken as 20 cm for all load-bearing walls in order to make a direct comparison between the models. The weight per unit area (live loads + dead loads) is assumed to be 15 kN/m^2 for typical story and 10 kN/m^2 for the roof.

5.2.1. Generation of URM Building Models

Generation of URM building models fully conform to the standards of masonry design according to TEC-07. The limitations for openings, minimum wall lengths in critical zones, unsupported wall length and other requirements are all satisfied. The only exception is that the ground storey wall thicknesses for the three storey buildings are 20 cm instead of 30 cm, which is used for a direct comparison of performances. It is possible to use 20 cm wall thickness in ground storey walls according to the Mexican, the Algerian, the Chilean or the Peruvian codes.

Eventually, models with wall irregularity do not conform to the standards. For all the models, concrete slabs and tie beams are used for rigid diaphragm action. Tie beams have a width equal to wall thickness and height of 15 cm according to Eurocode 6. Floor height is taken as 2.80 m, which satisfies 3 m criterion in TEC-07. For models with R2W3 combination, it is necessary to design some of the wall as reinforced concrete members.

5.2.2. Generation of CM Building Models

In generation of CM building models, it is important to place the tie columns in appropriate locations. In all of the codes analyzed in Chapter 2, it is a common practice to put the tie-columns on the corners of the plan and at all the intersections of walls. For the openings, the Mexican and the Chilean codes enforce to use tie columns around all the openings, whereas in the Algerian code and the Eurocode, it is necessary to use confining elements for openings having larger dimensions than a prescribed limit.

In this study, tie columns are placed principally at the corners of all the openings, but for some small wall segments that remain on the corners and intersections of walls, they are not used because those small wall segments do not make a significant contribution to the lateral load capacity and not considered as load bearing walls. Hence these walls are not taken into account in the lateral load capacity evaluation of the critical storey.

Tie columns should have the same thickness with the wall according to all considered codes. The only exception is the Algerian code where the thickness can be two-thirds of the wall thickness allowing for a brick facade. In this study, tie column dimensions are 20x20 cm, since the wall thickness is 20 cm and most of the tie columns are on the intersections of walls.

The storey height (floor to floor) is 2.80 m. Tie-beam dimensions are 15x20 cm. The width is equal to wall thickness and the height is 15 cm. These numbers satisfy the condition for minimum area of tie beams in Eurocode.

For building models with R2W3 combination, it is necessary to consider some of the walls as reinforced concrete members, similar to the URM models.

5.3. Ground Motion Records

Ten ground motion records from different earthquakes have been used in this study in order to estimate the performance points of the developed building models through the Capacity Spectrum Method. The list of the records and their detailed information are provided in Appendix B. In the selection of these records, the main criterion has been peak ground acceleration (PGA) since it is a known fact that rigid masonry structures are highly influenced from this ground motion parameter. Peculiar ground motion records due to near field effects, rupture directivity or significant site amplification effects are not taken into consideration. Hence the records have been selected in such a way that their PGA values cover all levels of ground motion intensity, from 0.06g to 0.75g. Ground motions are abbreviated and listed in an ascending order in terms of PGA (see Table B.1 in Appendix B). The other parameters listed in Appendix B are peak ground velocity (PGV), velocity to acceleration (V/A) ratio, effective peak acceleration (EPA), Housner spectrum intensity (HI), Arias Intensity (AI), Energy index (EI), effective duration (Δt_{eff}) and spectral acceleration values at different periods (T=0.2, 0.5 and 1.0 seconds). PGV and V/A ratio are straight-forward and self-explanatory. EPA is defined as that ground acceleration causing an average acceleration response amplification of 2.5 in the 5% damped short period SDOF systems with $0.1<T<0.5$ seconds, where T is the oscillation period (Kramer, 1996). The value of 2.5 is accepted as a global acceleration response amplification factor for 5% damped SDOF systems in the acceleration range of earthquake spectra. Therefore acceleration response spectrum has to be calculated first in this period range in order to obtain EPA. The ratio of EPA to PGA generally varies from 0.60 to 1.2 for recorded accelerograms, which also holds for the records in this study. HI, in which spectral velocity has been used as the response quantity, can be given as follows (Housner 1952)

$$HI = \int_{0.1}^{2.5} PS_v\,(T)dT \tag{5.1}$$

where PS_v is the 5% damped pseudo spectral velocity. PS_v is related to the maximum kinetic energy stored by the associated SDOF system, and HI is approximately simulating the energy stored by structures having vibration periods and damping ratios within the practical range of interest. In terms of HI, GM8, GM9 and GM10 seem to have significantly higher values than the other records in the data set.

AI is defined with the integral expression (Arias,1970)

$$IA = \frac{\pi}{2g}\int_0^{t_d} a^2(t)\,dt \tag{5.2}$$

where "*a*" is the ground acceleration and t_d is the total record duration. This parameter accounts for the duration of the accelerogram as well as its complete time history. For undamped SDOF systems, AI is closely related to the root mean square acceleration and it corresponds to the area under the spectrum of total energy absorbed by undamped SDOF systems at the end of ground excitation. In the selected list, GM09 has the highest AI value, followed by GM06.

EI employs energy equivalent velocity V_e

$$EI = \frac{1}{4}\int_0^{4} V_e\,(T)dt \tag{5.3}$$

where V_e is defined as

$$V_e = \sqrt{\frac{2E_i(T)}{m}} \tag{5.4}$$

E_i is the input energy per unit mass for a 5% damped SDOF system (Sucuoglu and Nurtug, 1995). EI is a more involved intensity parameter than HI because it is a direct measure of average energy imparted by the ground motion into structures throughout its entire duration. For the ground motion list in this study, GM08 has the highest EI value, followed by GM09 and GM10.

The duration of ground motion which contributes to the significant part of the response of SDOF systems is called the effective duration, Δt_{eff}. A practical way of calculating this duration employs Equation 5.1. It is defined as the duration between the instants when the integral in Equation 5.1 reaches the 5% and 95% of its final value (Trifunac and Brady, 1975). For the selected records, it is surprising to see that the longest effective duration belongs to GM03 followed by GM01, which is completely opposite of the previous trends. Ground motion records with high PGA values seem to be shorter in duration.

Spectral values at different periods are also presented in Table B.1. These values provide some information about the spectral amplifications due to site conditions where the records have been taken. However more information can be obtained from the acceleration spectra of the records for the whole period range up to 4 seconds as given in the spectral plots in Appendix B. The ground acceleration traces are also present in the appendix.

The disordered values obtained from different ground motion parameters reveal that it is not possible to identify the damaging potential of a ground motion record by a single parameter. Different parameters should be considered together in order to make such a judgement.

5.4. Application of the Proposed Procedure and the Obtained Results

The proposed procedure involves the determination of seismic demand and capacity and then the estimation of the performance point through the Capacity Spectrum Method as defined in FEMA 440 (ATC, 2005). Seismic capacity is quantified by using the procedure of Tomazevic (1999) as explained in Chapter 4. A Matlab (The Mathworks Ins., 2009) code is produced to construct the capacity curves. The sample codes for URM and CM models are provided in Appendix C. As stated before, a displacement based pushover analysis is carried out to construct the storey shear capacity curve. Capacity curves for the models used in this study are given in Appendix D.

For the quantification of seismic demand, the elastic spectra of the selected 10 ground motion records have been used (see the spectral plots in Appendix B). These records have varying levels of ground motion intensity in terms of their PGA values, hence GM01 can be assumed to exhibit the lowest level of seismic intensity whereas GM10 is the record with the highest intensity.

The superposition of seismic demand with capacity yields the performance points of the 72 building models subjected to 10 ground motion records of different seismic intensity levels, summing up to 720 cases of performance analysis for URM and CM building models. A sample performance analysis is shown in Figure 5.1. In this figure, performance point of model C-N2R1F8W1 subjected to ground motion GM10 is represented graphically. Series named with "μ=1-6" represent the reduced response spectrum of GM10 for ductility levels from 1 to 6, series named with "TSEC1-6" are the secant lines defined in Capacity Spectrum Method (ATC, 2005) drawn for the corresponding ductility levels and series named with "Capacity" represent the capacity curve of the building. Intersection of reduced response spectrum curves with the corresponding secant lines for each ductility level gives the locus points that construct the locus curve. Performance point is defined as the intersection of capacity curve with the locus curve.

Hence spectral displacement (S_d), spectral acceleration (S_a), roof displacement (d_{roof}) and base shear to weight ratio (V/W) values of all the calculated performance points are obtained through analysis. However, it is difficult to both interpret and display this huge amount of data with many variable parameters. Instead, a single-valued index, named as damage index (DI) in Chapter 4, is employed for the interpretation of results. The details and formulation of DI have been discussed in Section 4.9. The DI values obtained for each performance analysis are given in tabular form in Appendix E.

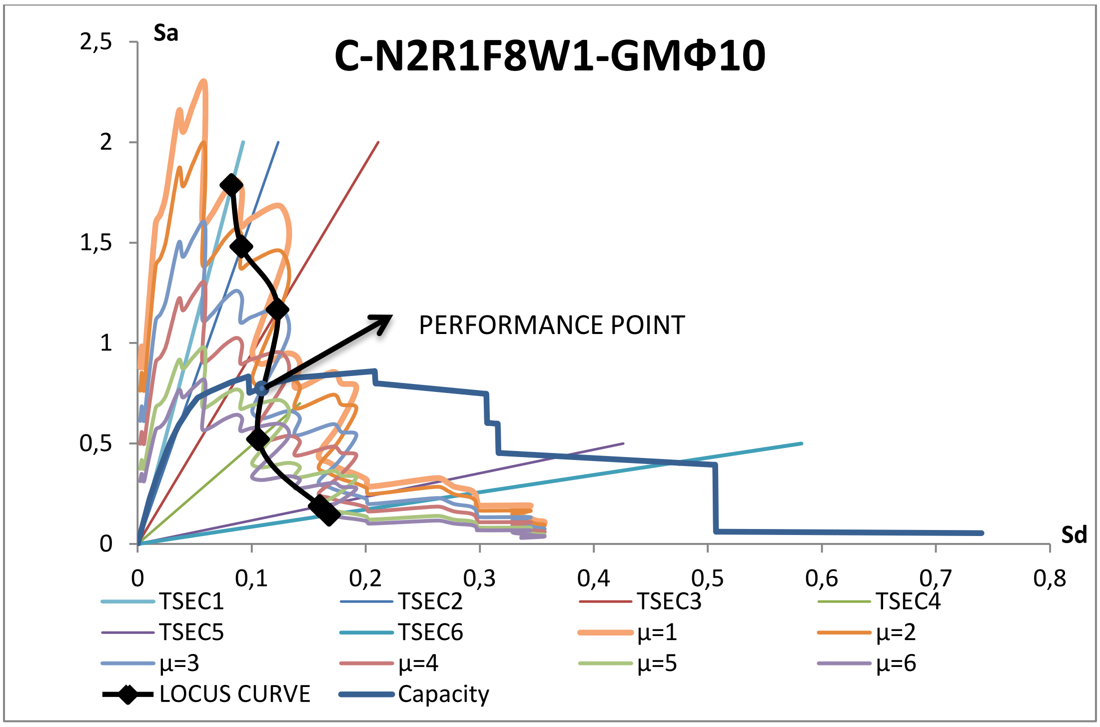


Figure 5.1 Sample performance analysis.

5.5. Analysis of Results

The following trends can be observed by examining the results:

- When compared with each other, the DI values clearly show the superiority of CM models over URM models for almost all cases. There are exceptions that DI values of two similar building models of different construction type are equal to each other. These cases occur either for the lowest levels of ground motion intensity where both models behave in the elastic range or for the highest levels of ground motion intensity where DI=1.0 (collapse state), especially for low levels of strength (f_m=2 MPa). This observed trend is demonstrated in Figures 5.2.a (left) and 5.2.b (right) for 2-story and 3-story URM and CM building models with the most favorable (regular plan geometry, high strength, adequate masonry walls with even distribution, i.e. R1F8W1) and unfavorable (irregular plan geometry, low strength, inadequate masonry walls with uneven distribution, i.e. R2F2W3) parameters. Even in the case of building models with totally opposite parameters, CM variants seem to be performing better than URM variants with lower DI values and higher PGA values at DI=1.0.

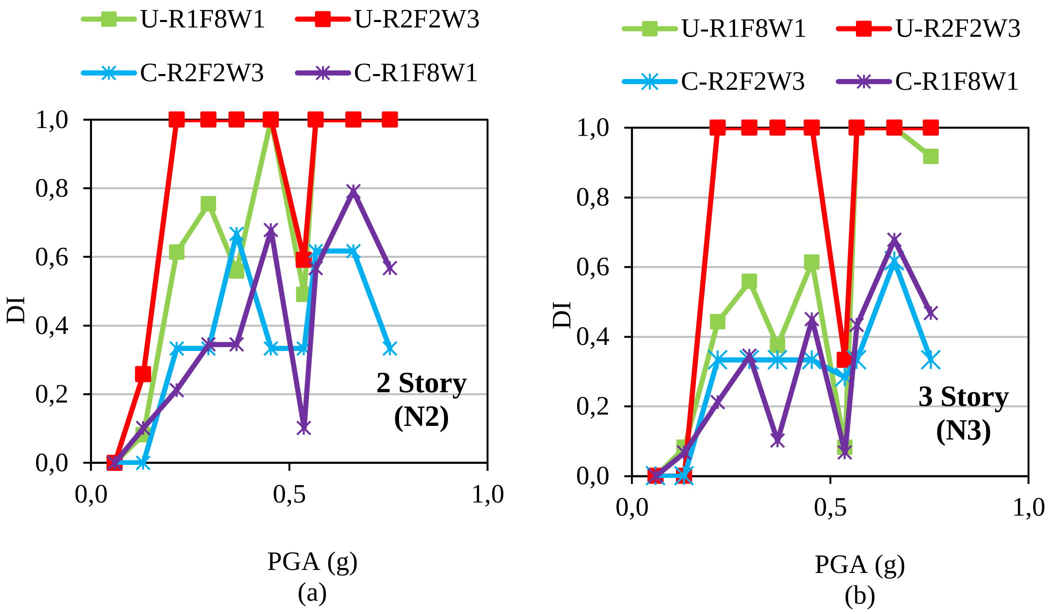


Figure 5.2 DI-PGA relationship for building models with 2 and 3 stories

- It is also possible to comment on the effect of number of stories (N2-N3) from Figures 5.2.a and 5.2.b. It can be stated that DI is sensitive to number of stories in the case of URM models, especially in the case of most favorable parameters (i.e. R1-F8-W1). In the case of most unfavorable parameters (i.e. R2-F2-W3), N-curves get closer to each other. The main reason is that both curves reach to the state DI=1.0 at low levels of PGA and preserve this value at higher levels of seismic intensity. In the case of CM models, DI is slightly sensitive to number of stories for R1-F8-W1 whereas it does not seem to be sensitive for R2-F2-W3 as the N-curves overlap for nearly all levels of PGA. It is also important to note that curves for CM models (either with favorable or unfavorable parameters) never reach the state DI=1.0. This verifies the good seismic performance of CM models over URM models.
- The influence of total length of walls and their distribution within the story plan (i.e. W1-2-3 cases) on the seismic performance of masonry buildings is

examined in Figures 5.3.a (left) and 5.3.b (right) for regular and irregular 3-story models with high or low strength (i.e. N3-R1-F8 and N3-R2-F2, respectively) as the most unfavorable and favorable parameters for this comparison. In the case of N3-R1-F8, W2 and W3 curves for URM (U-W2 and U-W3 in Figure 5.3.a) reach the state DI=1.0 at PGA values of nearly 0.6g whereas for CM models, none of the W-curves reach this state. In the case of N3-R2-F2, it is expected that W1-2-3 cases should have a significant effect on the seismic performance of masonry buildings. However, Figure 5.3.a shows that the expected trend is only slightly observed for URM models (U-W3 model reaching DI=1 at a lower PGA value, U-W1 model never reaching DI=1) and not observed for CM models at all (all models have similar trends). So it can be stated that of the effect this parameter is not very significant on the overall seismic response of CM buildings. This may be due to the presence of vertical tie-columns.

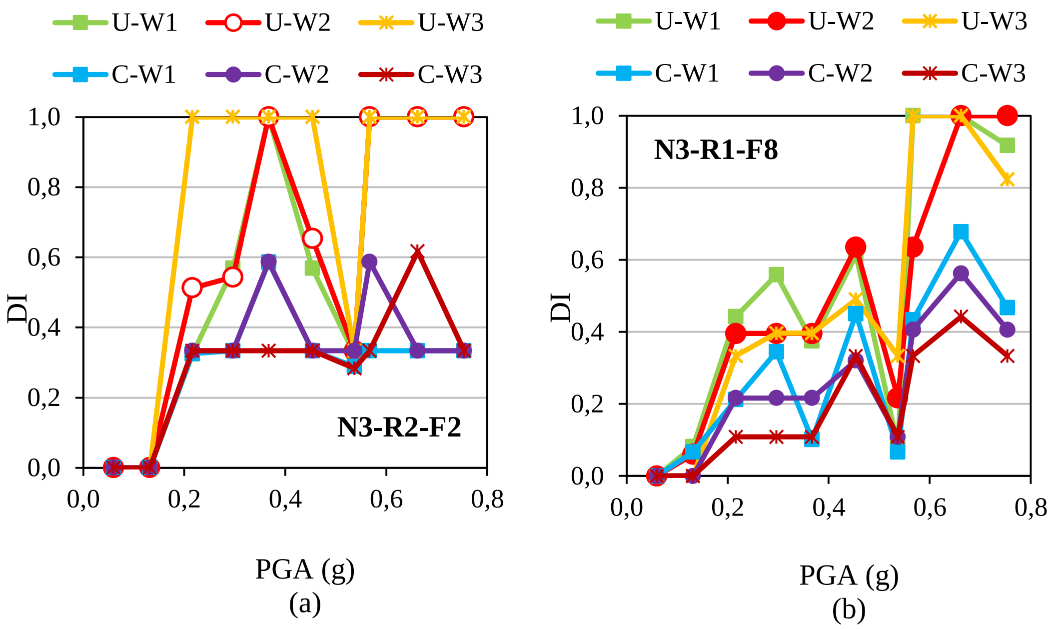
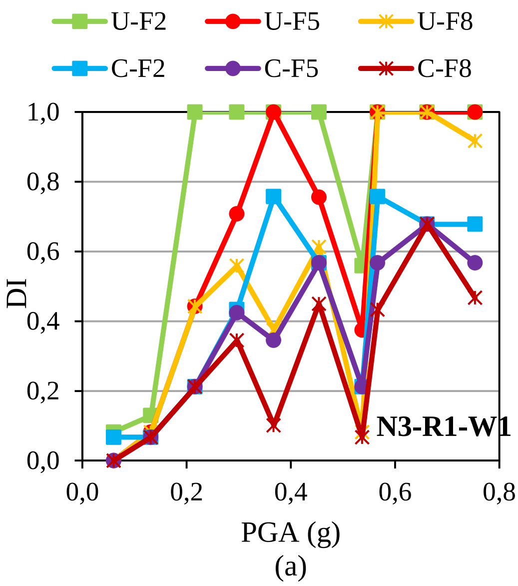


Figure 5.3 Sample DI-PGA relationships for the comparison of W-parameter

- The influence of material strength (i.e. F2-5-8 cases) on the seismic performance of masonry buildings is examined in Figures 5.4.a (left) and 5.4.b (right) for regular and irregular 3-story models with adequate and inadequate length and distribution of walls in the critical story (i.e. N3-R1-W1 and N3-R2-W3, respectively) as the most unfavorable and favorable parameters for this comparison. For both cases, URM models with low strength reach to collapse state (DI=1.0) at low levels of PGA (i.e. around 0.2g), followed by models with moderate strength reaching DI=1.0 at approximately 0.4g. The URM models with high strength take the value of DI=1.0 when PGA~0.6g. This trend shows the importance of strength parameter on the seismic performance of URM models. CM models (either for N3-R1-W1 or N3-R2-W3) never reach to the value of DI=1.0. This trend shows that CM models behave better than URM models under similar conditions. Furthermore, the strength parameter is relatively less pronounced for CM models. This is due to the fact that there are different parameters in CM construction that play important roles for seismic performance other than the material strength.



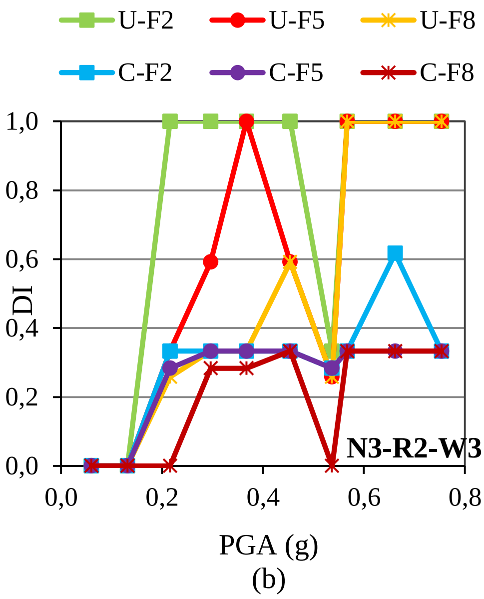


Figure 5.4 Sample DI-PGA relationships for the comparison of F-parameter

- The influence of wall distribution in plan (i.e. R1-2 cases) on the seismic performance of masonry buildings is examined in Figures 5.5.a (left) and 5.5.b (right) for 3-story models with high strength and adequate length and distribution of walls in the critical story (i.e. N3-F8-W1) and with low strength and inadequate length and distribution of walls in the critical story (i.e. N3-F2-W3) as the most unfavorable and favorable parameters for this comparison. In the case of URM models, irregularity in wall distribution yields relatively higher values of DI than the cases with regular plan, but the difference is not significant. In addition, the URM model with unfavorable parameters (i.e. N3-F2-W3) reaches to the collapse state earlier than the URM model with favorable parameters (i.e. N3-F8-W1). In the case of CM models, the trends are very close to each other while never reaching to collapse state. This reveals that wall distribution in plan is not a major parameter for the seismic performance of CM building models.

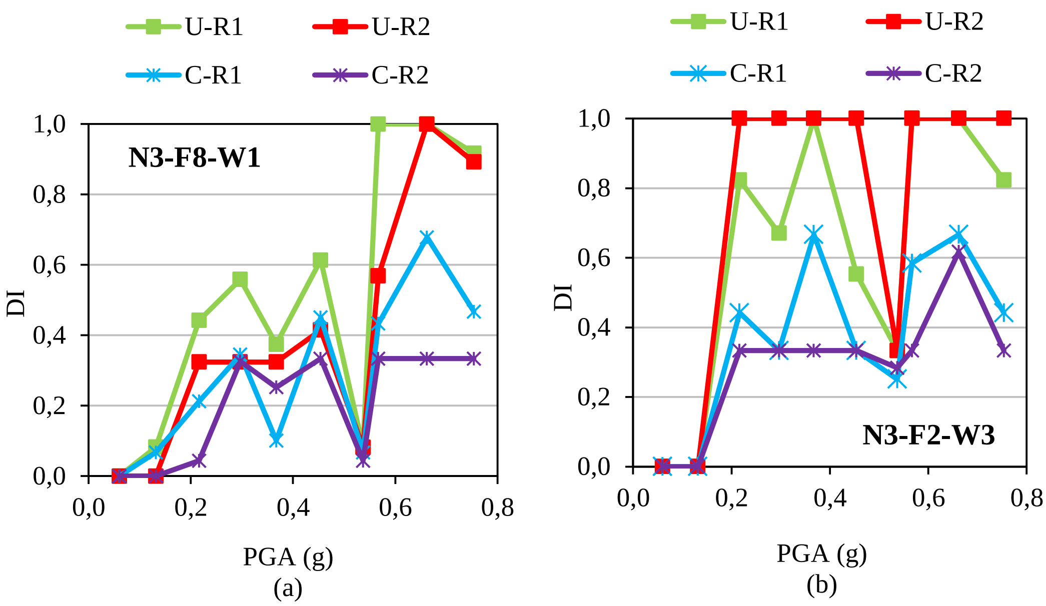


Figure 5.5 Sample DI-PGA relationships for the comparison of R-parameter

From Figure 5.5 it can also be observed that CM building plans with irregular wall distribution in plan has slightly lower damage index especially in the case of high PGA values whereas URM models with irregular wall distribution in plan has higher DI values in W3 plans. This may be the result of contribution of long walls to the storey shear resistance. In Figure 5.6, contribution of the long wall in N2R2F5W1 models of both URM and CM types are compared. It is clearly seen that contribution of the long wall in URM model declines as the PGA level increases whereas the corresponding long wall in CM model makes even more contribution to the capacity as the PGA level increases.

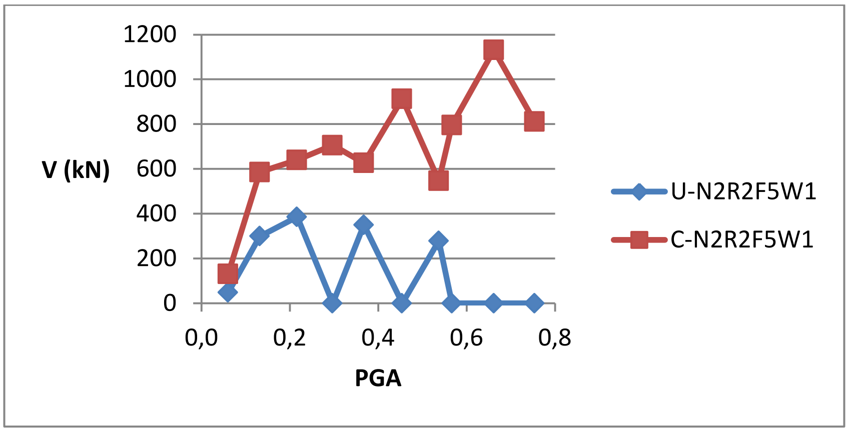


Figure 5.6 Comparison of long wall contribution to the storey shear resistance.

Since ground motions are numbered and listed in the ascending order of PGA values, it is expected that the DI values should also be increasing in a consistent manner. However, the results (see Figures 5.2-5.5) reveal that the record named as GM07 (with a PGA=0.54g) yields lower values of DI than the other records with lower PGA. Although this trend is present in Figures 5.2-5.5, it is more clearly observed in Figure 5.6 in the sample case for 2-story regular URM and CM building models with proper length and distribution of load bearing walls in plan (i.e. N2-R1-W1) for varying strength levels. In all curves, there is a drastic drop at 0.54g, the PGA level that belongs to GM07.

This may seem to be controversial, but it reveals that PGA alone may not be adequate to express the seismic vulnerability of these building structures; other parameters should also be taken into account. Accordingly, although PGA value is high for this record, other parameters like PGV, HI and EI seem to be lower than the values of the same parameters for GM05 and GM06, which may be an indicator of low damageability of this record. The relationships of other ground motion intensity measures (i.e. EPA, PGV, HI and EI) with DI for building models with 2 and 3 stories are presented in Figure 5.8. The distributions seem to be different from the ones with PGA, but the general trends of the building models seem to prevail.

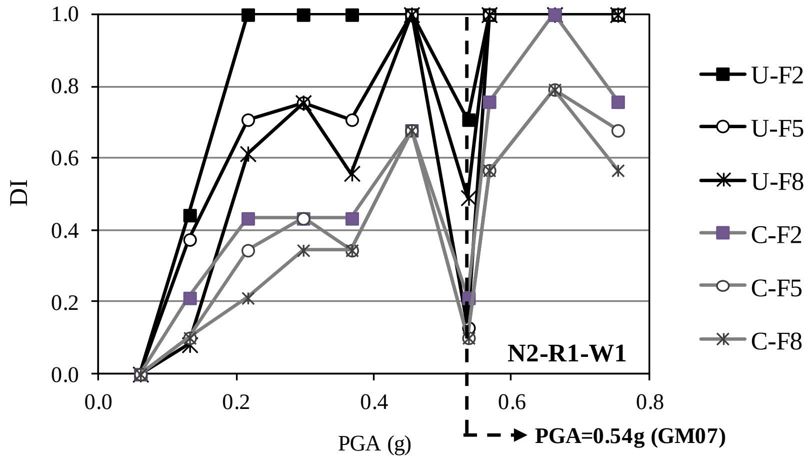


Figure 5.7 DI-PGA relationship demonstrating the drastic drop at PGA=0.54g (due to GM07)

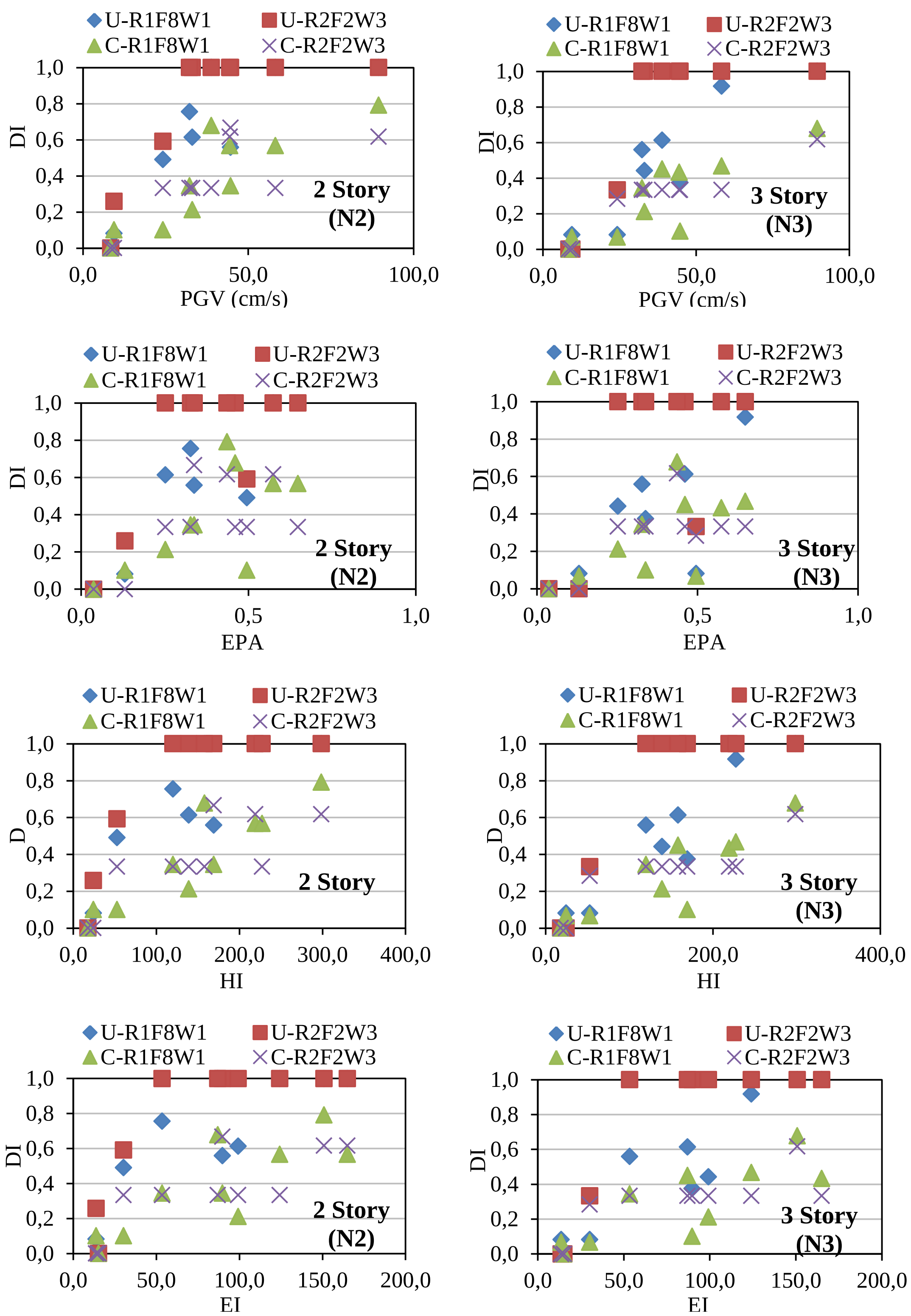


Figure 5.8 Relationship of earthquake parameters with DI for building models with 2 and 3 stories

Another important parameter to assess the seismic performance of URM and CM models is the PGA value at collapse (DI=1.0), i.e. PGA_C. This parameter is defined as the acceleration value when DI=1.0 for the first time. PGAc values can be examined in Tables E.1-E.6 at Appendix E, which are in the cells shown by dark color. Examining the PGA_c values for URM and CM models, it is observed all URM models (i.e. 83% of the URM models) reach at the collapse state at some PGA value whereas only 1 out of 36 CM models (i.e. 3% of the URM models) reaches at the collapse state. This shows the overwhelming superiority of CM models over URM models in terms of reaching the collapse state during seismic action. CM buildings are more immune to collapse due to their modular structure, where the vertical tie-columns, horizontal tie-beams and the wall segment enclosed by these RC non-structural elements constitute the modules of the building. Each module behaves in an independent manner since the damage and crack formation cannot propagate from one wall to the other (as in the case of URM buildings) due to confining elements in between. This increases the energy dissipation capacity of the CM buildings, which in turn decreases the possibility of experiencing collapse during seismic action. Although some of the wall segments (or modules) confined by tie-beams and tie-columns can be heavily damaged during shaking, this may not induce total collapse since the structure is still stable due to non-damaged or slightly damaged wall segments.

In order to evaluate the PGA values at collapse, the URM models are classified into three sub-classes according to their quality of construction as “good”, “moderate” and “poor”. URM models with good quality of construction represent regular or nearly regular buildings with good material quality and strength, having adequate amount of and evenly distributed masonry load-bearing walls in plan. Such buildings are generally located in urban regions, made of massive stone, good quality brick or new technology materials like autoclave aerated concrete (AAC). In this study, such structures are represented by the following eight URM building models: N2-R1-F8-W1, N3-R1-F8-W1, N2-R1-F5-W1, N3-R1-F5-W1, N2-R1-F8-W2, N3-R1-F8-W2, N2-R2-F8-W1 and N3-R2-F8-W1. Figure 5.9 illustrates PGA_C values of these eight models. The average PGA_C value of the remaining six building models is 0.54g±0.10g (i.e. COV=0.19).

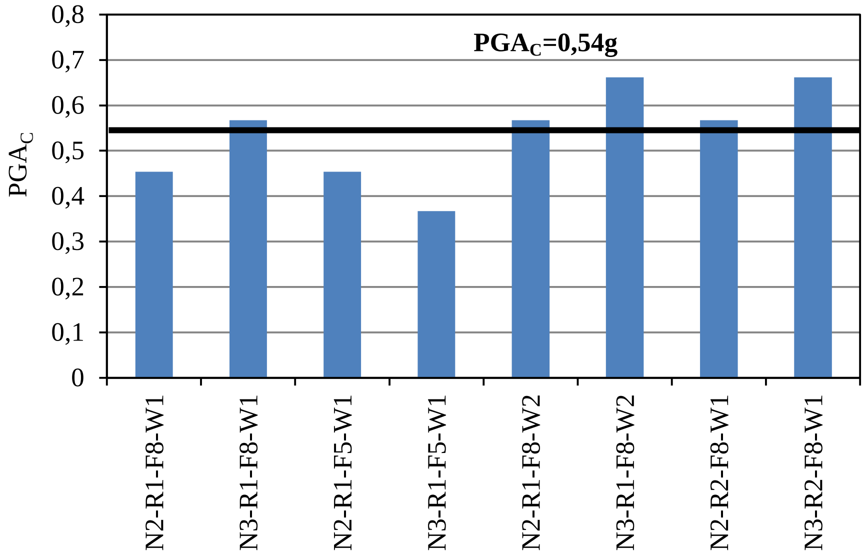


Figure 5.9 PGA_C values of URM models classified as "good quality construction"

URM models with moderate quality of construction represent regular or irregular buildings with variable (but on the average moderate) material quality and strength. These building may have deficiencies in relation with the amount or story plan distribution of masonry load-bearing walls. Such buildings constitute the majority of the URM building stock, either in urban or rural regions. In this study, such structures are represented by the following twenty URM building models: N2-R1-F2-W1, N3-R1-F2-W1, N2-R1-F2-W2, N3-R1-F2-W2, N2-R1-F5-W2, N3-R1-F5-W2, N2-R1-F5-W3, N3-R1-F5-W3, N2-R1-F8-W3, N3-R1-F8-W3, N2-R2-F2-W1, N3-R2-F2-W1, N2-R2-F5-W1, N3-R2-F5-W1, N2-R2-F5-W2, N3-R2-F5-W2, N2-R2-F8-W2, N3-R2-F8-W2, N2-R2-F8-W3 and N3-R2-F8-W3. Figure 5.10 illustrates PGA_C values of these twenty models. The average PGA_C value of the remaining sixteen building models is 0.42g±0.15g (i.e. COV=0.34).

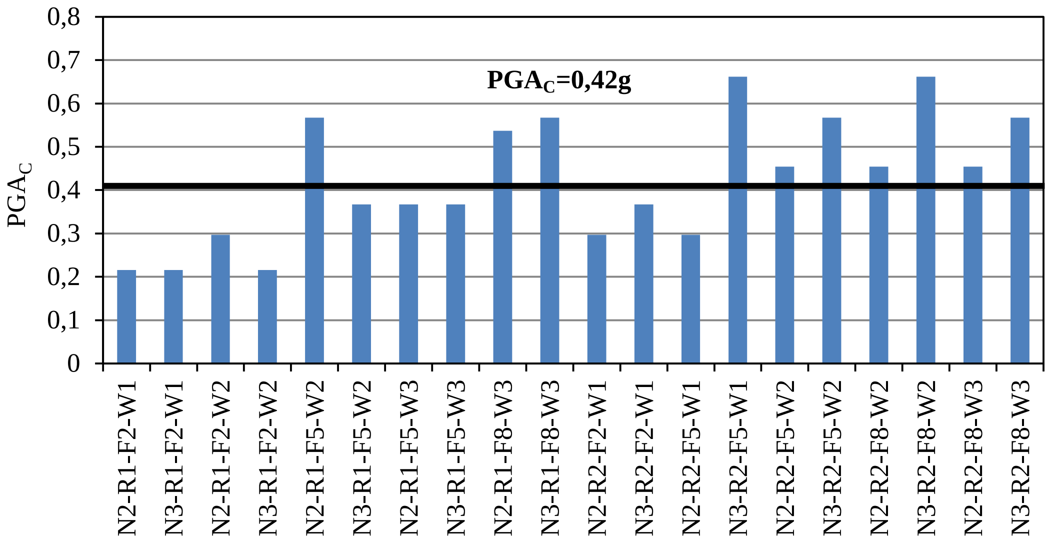


Figure 5.10 PGA_C values of URM models classified as "moderate quality construction"

Finally, URM models with poor quality of construction represent irregular buildings with low material quality and strength. These building have major deficiencies in relation with the amount or story plan distribution of masonry load-bearing walls. Such buildings are generally located in rural regions or in suburb regions of cities. This group of buildings is known to exhibit low seismic performance even in moderate size earthquakes. In this study, such structures are represented by the following eight URM building models: N2-R1-F2-W3, N3-R1-F2-W3, N2-R2-F2-W2, N3-R2-F2-W2, N2-R2-F2-W3, N3-R2-F2-W3, N2-R2-F5-W3 and N3-R2-F5-W3. Figure 5.11 illustrates PGA_C values of these eight models. The average PGA_C value of these eight building models is 0.30g±0.10g (i.e. COV=0.32).

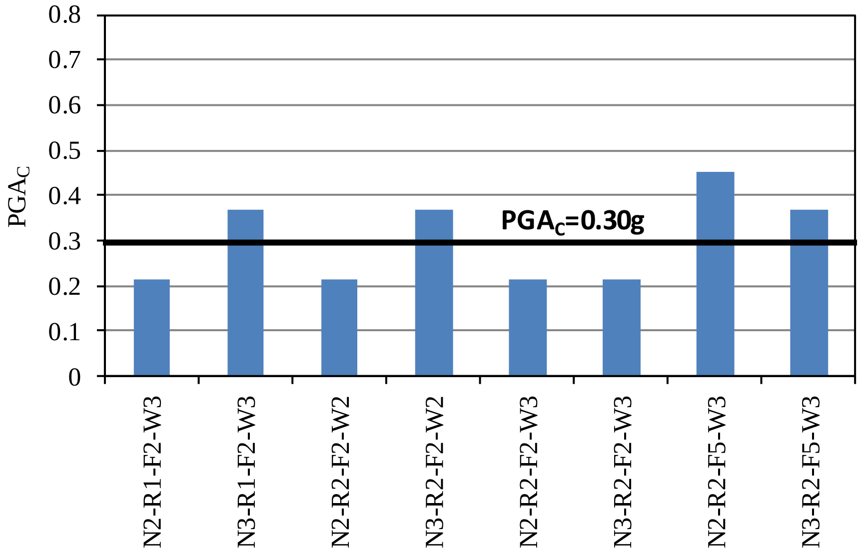


Figure 5.11 PGA_C values of URM models classified as "poor quality construction"

The above information is valuable in the sense that it gives a crude estimation of the range of PGA values that would cause the collapse of the URM building class under consideration. As expected, poor quality and deficient URM structures are the most vulnerable ones with a PGA value range 0.20g to 0.40g to induce collapse whereas well-constructed and code-compliant URM buildings have much higher PGA values at collapse state (0.40g to 0.62g). The moderate quality URM buildings that constitute the majority of the building stock have a PGA range of 0.29g to 0.57g, depending on the structural properties. The high COV value of this class of URM buildings arise from the wide range of properties inherent in this group. Hence it can be concluded that deficient URM structures exhibit heavy damage or collapse even under moderate levels of ground motion intensity (i.e. PGA=0.2g). As the quality of construction increases and structural deficiencies decrease, URM structures perform better and PGA values at collapse rise up to values of 0.5g-0.6g. On the other hand, CM buildings have much more better behavior than URM structures and exhibit almost no collapse under the levels of seismic action considered in this study.

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1. SUMMARY

In this study, seismic performances of CM buildings are compared to that of URM buildings. Story plans of six generic building models are employed for this comparison. The story plans have been generated by using the common construction practice and typical dimensions in Turkey in addition to the rules and limitations enforced by some of the national and international seismic design codes. Three of them have a regular plan whereas the other three have a more irregular distribution of load-bearing masonry walls in plan. Seismic performances of the building models are evaluated by using the Capacity Spectrum method “Procedure C” described in FEMA440 (ATC, 2005).

Lateral resistance capacity of the models is determined according to the approach proposed by Tomazevic (1999), who states that the storey shear resistance of a masonry structure in a certain direction is the sum of lateral resistances of all the structural walls in that direction in the considered storey. Accordingly, the strength capacities in the critical stories are determined for the principal directions in plan. The masonry wall capacities are modeled according to the trilinear idealization of Tomazevic (1999). Trilinear idealization model requires determining the three limit states (cracking, maximum strength and ultimate strength) of each wall. Formulas for the three limit states are determined according to the study of Erköseoğlu (2014) and Tomazevic (1999). For URM walls, three failure modes (sliding, flexure, diagonal shear) are compared to find the maximum strength. For CM walls, it is assumed that the governing mode of failure is diagonal shear under the vertical stresses applied on the walls for the scope of this study. Besides, all the walls are assumed to resist against the possible out of plane effects.

The shear resistance of the critical storey (ground storey for this study) is determined step by step through displacement-based pushover analysis. Changes in displacements due to torsional effects are considered in each step. Displacement versus shear force data obtained from the pushover analysis is converted into ADRS format according to ATC-40 (1996). The roof displacement values that have been required to make this conversion are determined by assuming an inverse triangular distribution, analogous to the fundamental mode shape for the building models.

For seismic demand quantification, ten actual earthquake records given in Appendix B are used. Earthquake records are converted into ADRS format. 5% damped spectral acceleration values are used as the initial ADRS demand curve in the Capacity Spectrum method of analysis. Foundation damping that can change the seismic demand on the structure is out of the scope of this study.

Demand and capacity curves are plotted on the same chart. Reduced response spectrum curves are replotted for different ductility levels. Locus points and locus curve is defined and the performance point is obtained. Analysis is performed only in the weaker direction of each model. Each model has variable parameters as construction type (URM or CM), number of stories and masonry strength. 72 models are analyzed with 10 earthquake records, producing 720 performance points. Limit state of each wall for each earthquake is determined. Then a damage index (DI) is introduced to represent the overall damage distribution of each building model by a single value for the sake of comparison. PGA values at which complete failure occurs (i.e. DI=1.0) is abbreviated as PGA_C and these values can be examined in Appendix D.

When the PGA values of ten earthquakes are plotted against the DIs of each model, it is observed that a CM model even with the most unfavorable parameters shows a relatively better performance than the corresponding URM model with most favorable parameters. Number of stories does not seem to have a regular trend on the seismic performance of CM models whereas 3 storey URM models have a slightly lower DI in the most favorable model. Wall length and distribution does not seem to affect the performance of CM models but in the case of URM models, the model with most unfavorable distribution has a higher DI. The influence of plan geometry is not significant for either URM or CM models. Masonry strength seems to be very important for URM models, low quality masonry leads to earlier collapse in those models. CM models with lower material quality have higher DI, but only one CM model reaches to the collapse state. So the effect of material strength is less critical for CM buildings. Overall, CM models could perform well despite the changing parameters, whereas URM models are sensitive to unfavorable design conditions.

PGA_C values showed clearly the superiority of CM construction that all URM models reaching collapse damage state compared to 1 out of 36 models in CM buildings. URM models are grouped into poor, moderate and good quality construction to see the vulnerability level of these models. It is observed that even good quality construction URM models are vulnerable against earthquakes with high PGA values. In the case of poor quality construction buildings commonly observed in rural regions or in suburbs; those models are even vulnerable against earthquakes with moderate PGA values.

6.2. CONCLUSIONS

There are some gross assumptions and limitations in this study in order to evaluate the seismic performance of URM and CM buildings. Eventually this is a must since modeling and analyzing masonry structures are challenging tasks due to highly nonlinear, orthotropic and nonhomogeneous nature of masonry as a construction material and the non-standard techniques used in construction with variable material properties, which makes the estimation of actual behavior rather difficult. Hence, it is important to point out that the results obtained and the conclusions drawn in this study are based on these assumptions and simplifications. Accordingly, the following conclusions are drawn in this study:

- Low-rise CM buildings have very good seismic performance even under strong seismic action. They are more immune to collapse due to their modular structure, where the vertical tie-columns, horizontal tie-beams and the wall segment enclosed by these RC non-structural elements constitute the modules of the building. Each module behaves in an independent manner since the damage and crack formation cannot propagate from one wall to the other (as in the case of URM buildings) due to confining elements in between. This increases the energy dissipation capacity of the CM buildings, which in turn decreases the possibility of experiencing collapse during seismic action. Although some of the wall segments (or modules) confined by tie-beams and tie-columns can be heavily damaged during shaking, this may not induce total collapse since the structure is still stable due to non-damaged or slightly damaged wall segments.
- On the other hand, URM buildings seem to be vulnerable to seismic action even under moderate levels of seismicity, especially in the presence of unfavorable structural parameters like low strength, inadequate amount and uneven distribution of masonry walls, etc.
- The structural parameters considered in this study have some influence on seismic performance in the case of URM buildings. Especially, masonry strength has a major effect. However, in the case of CM buildings, they do not seem to have a considerable effect on the seismic performance. This may be due to the fact that there are different parameters in CM construction that play important roles for seismic performance other than the ones considered in this study.
- Due to its good seismic performance observed in this study, CM construction should be encouraged in Turkey, especially for small-to-medium sized low-rise residential buildings. The only way to achieve this is to promote this type of construction in the new version of the Turkish Earthquake Code. For instance, the new code may enforce the construction of only CM buildings in regions of high-seismicity whereas URM buildings are allowed to be constructed in regions of low-to-moderate seismicity. In this way, it may be quite pos-

sible to save lives of many in rural and suburb regions only by obeying some simple construction principles regarding CM buildings. The discussion in Chapter 2 shows that this has been done in most of the earthquake-prone South American countries by enforcing the construction of CM buildings through legislation and shifting the content of the building stock from more vulnerable URM buildings to CM buildings. Besides according to a study by Marques and Lourenço (2014), for a typical two storey house, cost of CM structure is 16% less than the RC structure. This is another reason why low-rise CM construction should be encouraged.

- PGA alone may not be adequate to express the seismic vulnerability of these building structures; other parameters (like PGV, HI and EI) should also be taken into account for the complete consideration of ground motion damageability.
- Simple methods and approaches are very appropriate for the seismic analysis of masonry structures when compared to the classical time-history analysis. In this way, it becomes possible to conduct a large number of analyses (which is not possible to do with the classical methods) and carry out parametric studies on masonry behavior. However, as such methods and approaches possess many assumptions and simplifications (as also present in this study), the results obtained in this study should be supported by laboratory tests and the actual performance of URM and CM buildings in the field.

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APPENDIX A

BUILDING MODELS

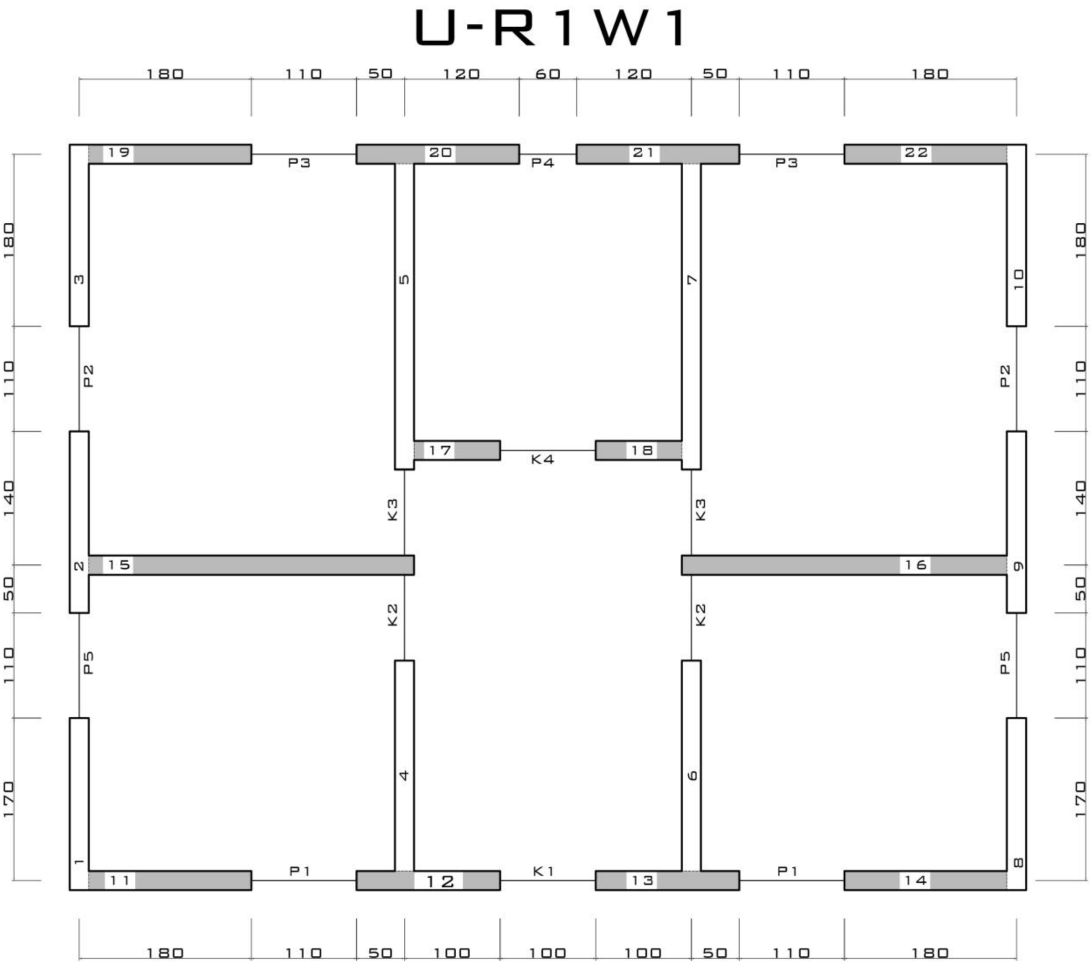

Figure A.1 The floor plan of building model U-R1W1 (all dimensions in cm)

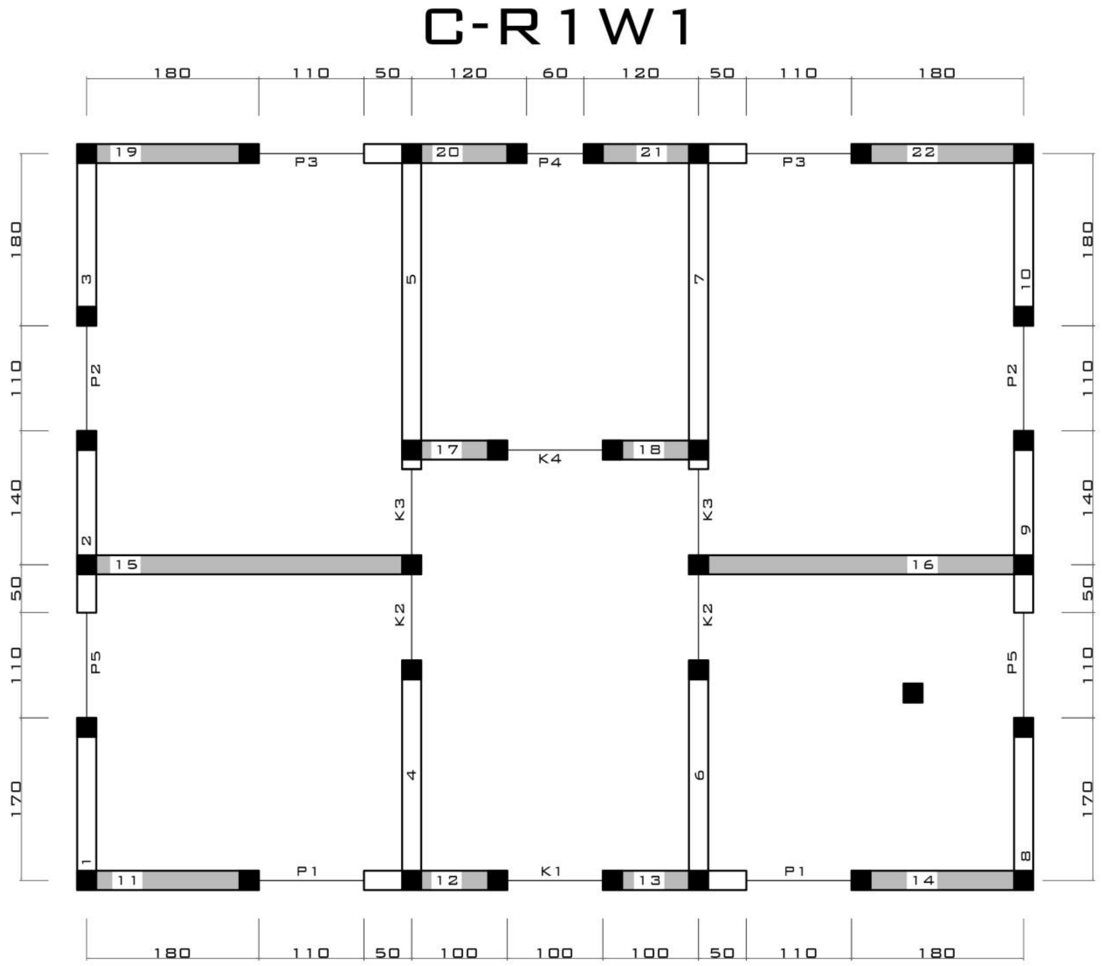

Figure A.2 The floor plan of building model C-R1W1 (all dimensions in cm)

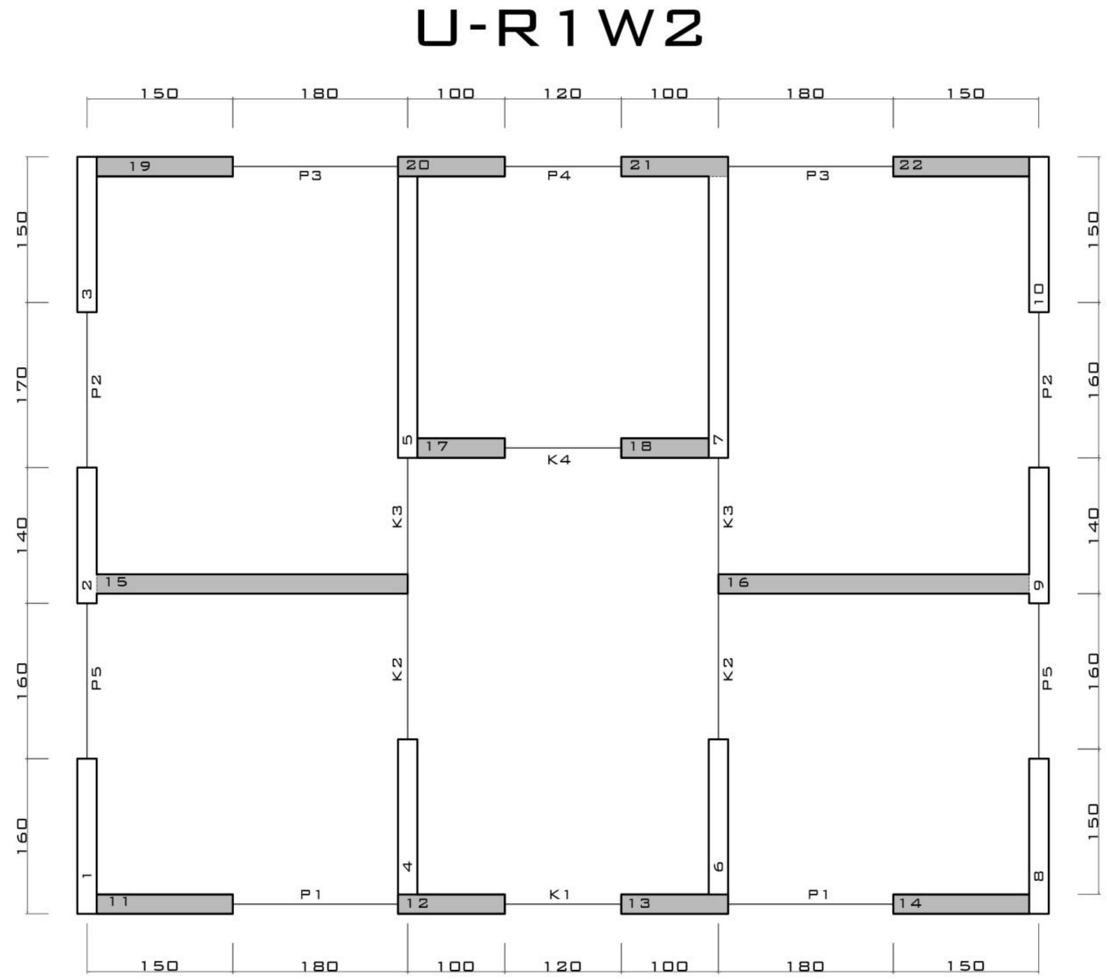

Figure A.3 The floor plan of building model U-R1W2 (all dimensions in cm)

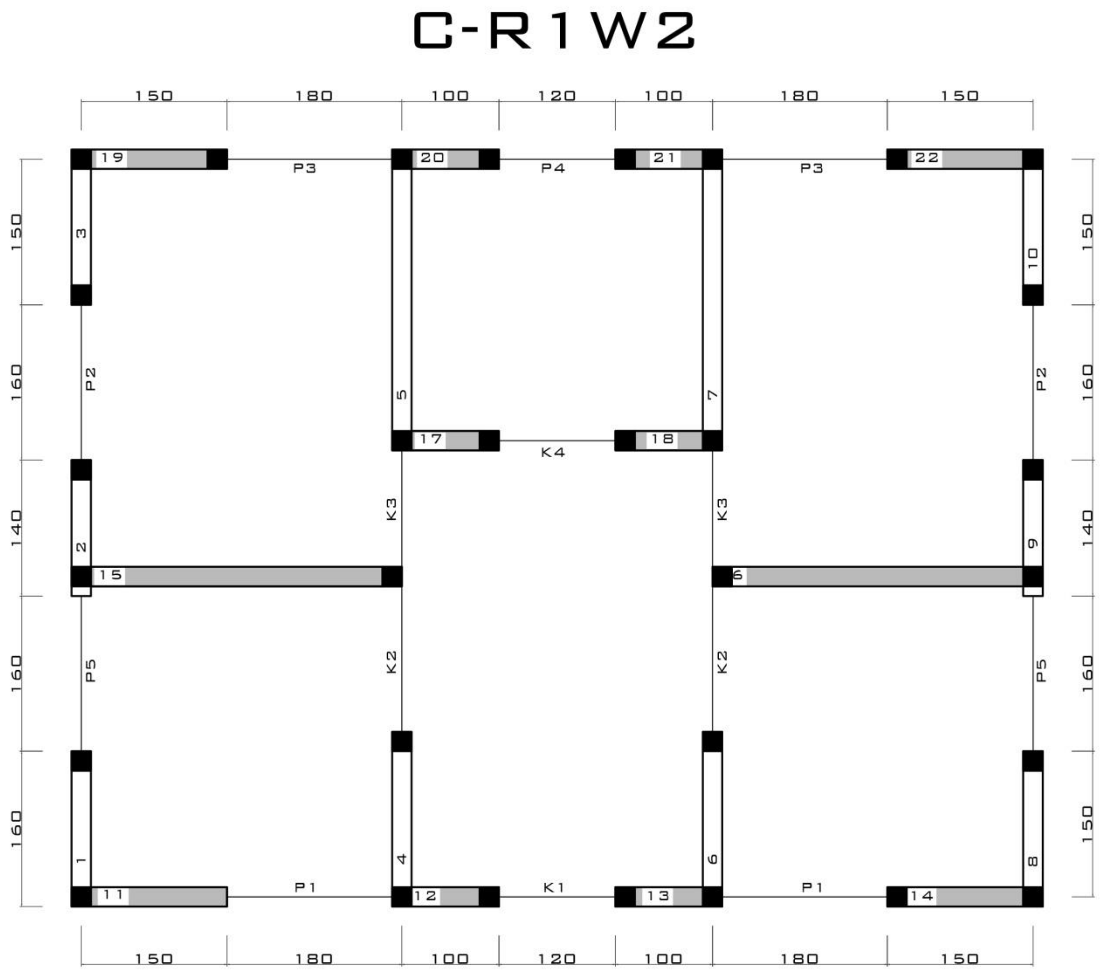

Figure A.4 The floor plan of building model C-R1W2 (all dimensions in cm)

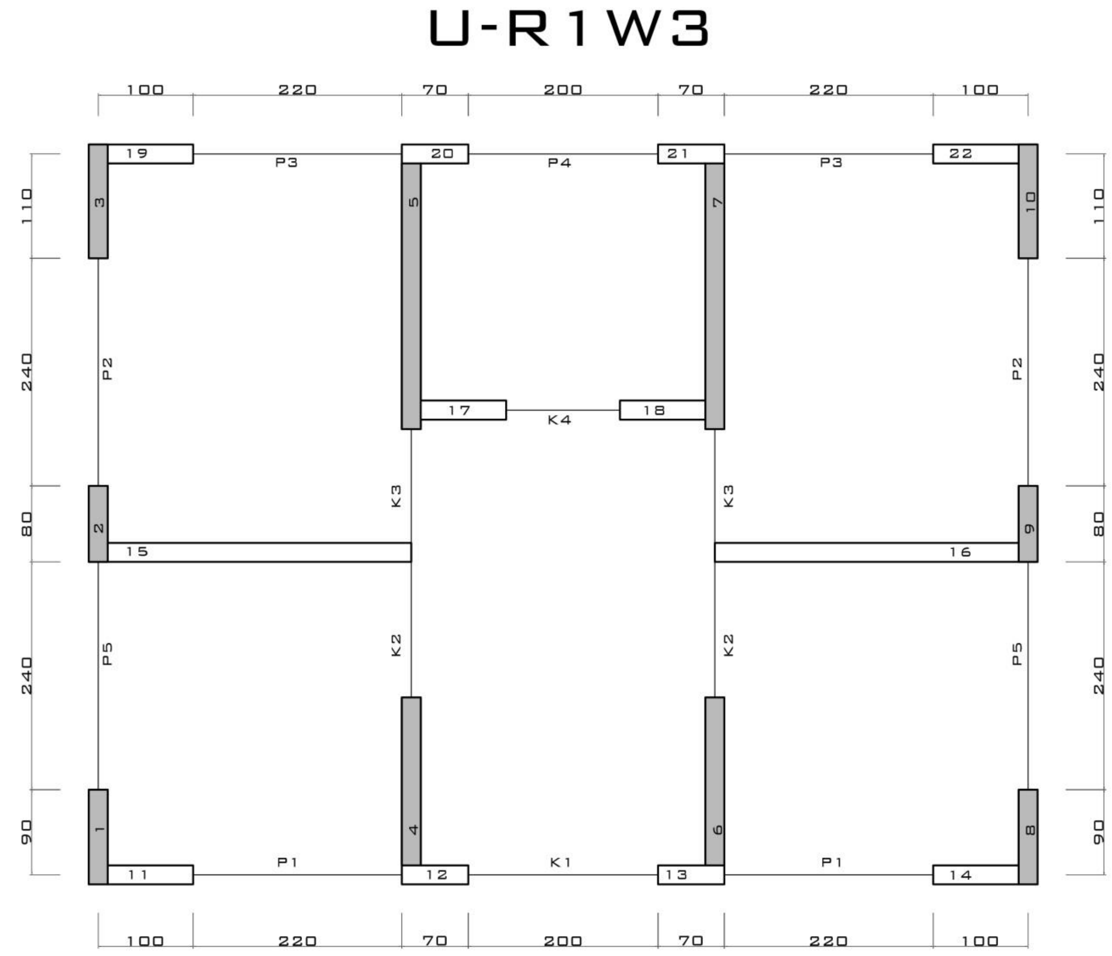

Figure A.5 The floor plan of building model U-R1W3 (all dimensions in cm)

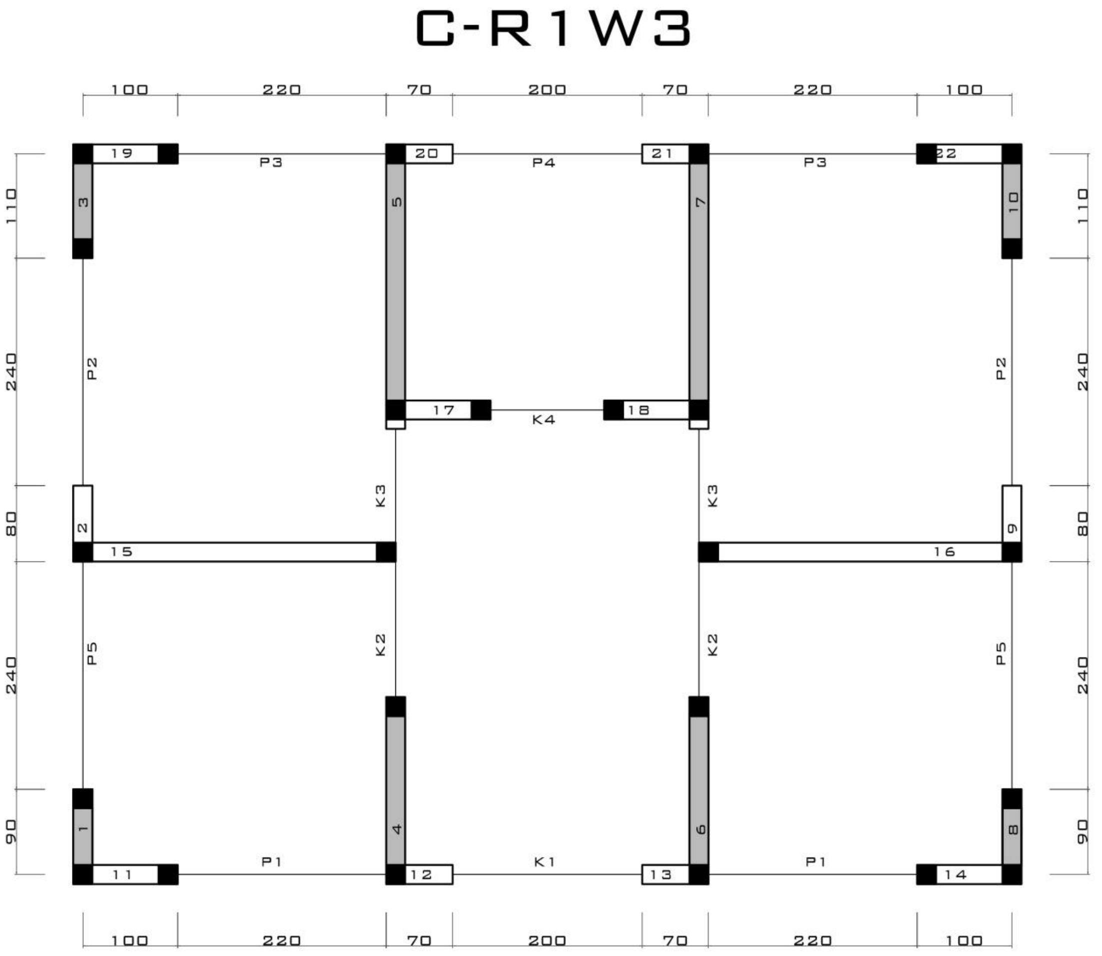

Figure A.6 The floor plan of building model C-R1W3 (all dimensions in cm)

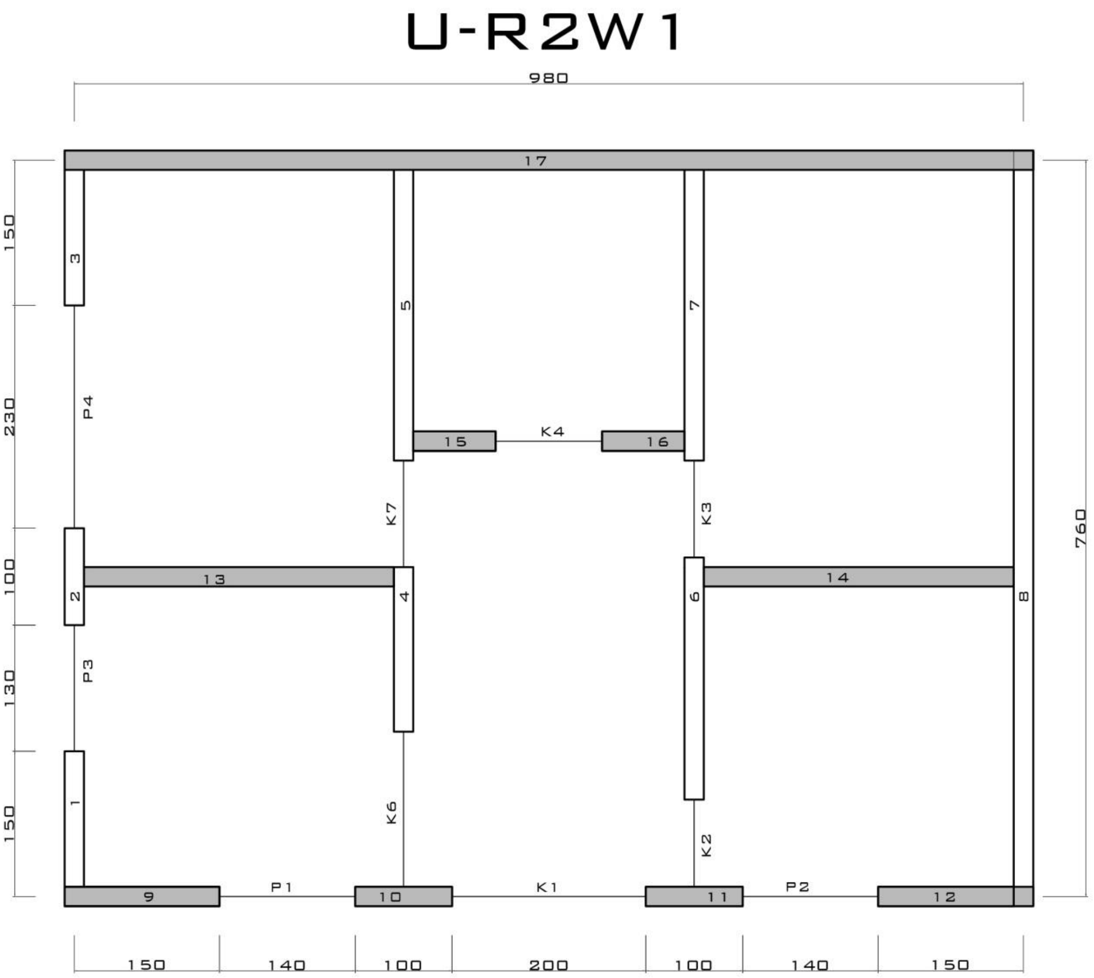

Figure A.7 The floor plan of building model U-R2W1 (all dimensions in cm)

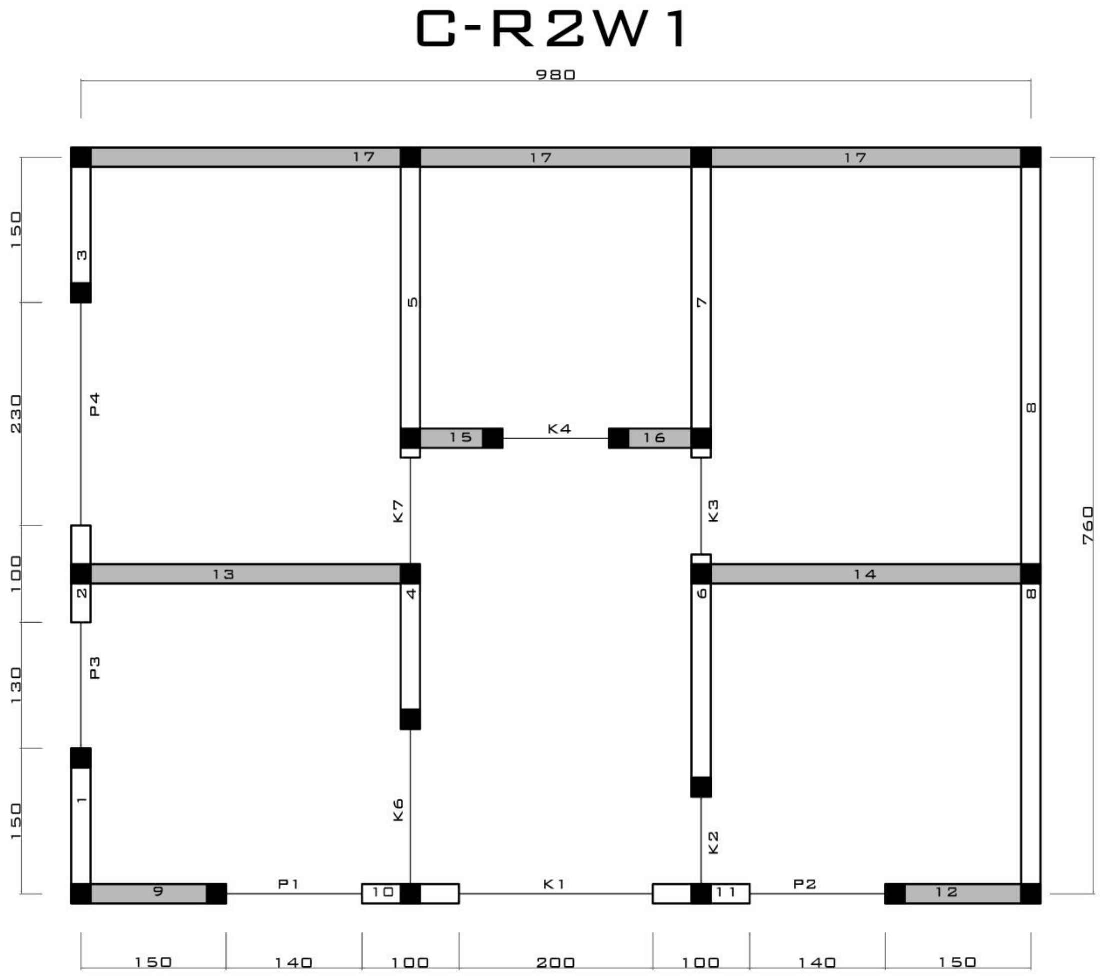

Figure A.8 The floor plan of building model C-R2W1 (all dimensions in cm)

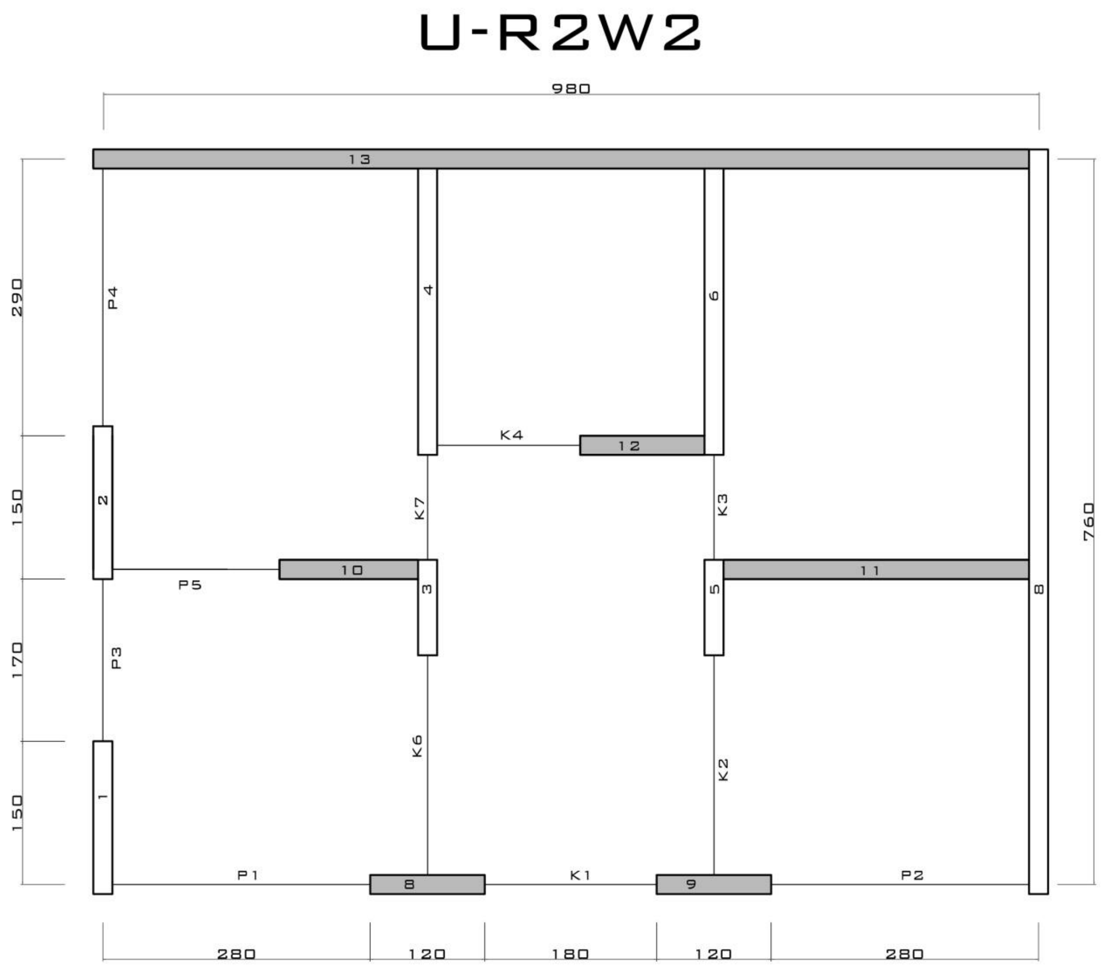

Figure A.9 The floor plan of building model U-R2W2 (all dimensions in cm)

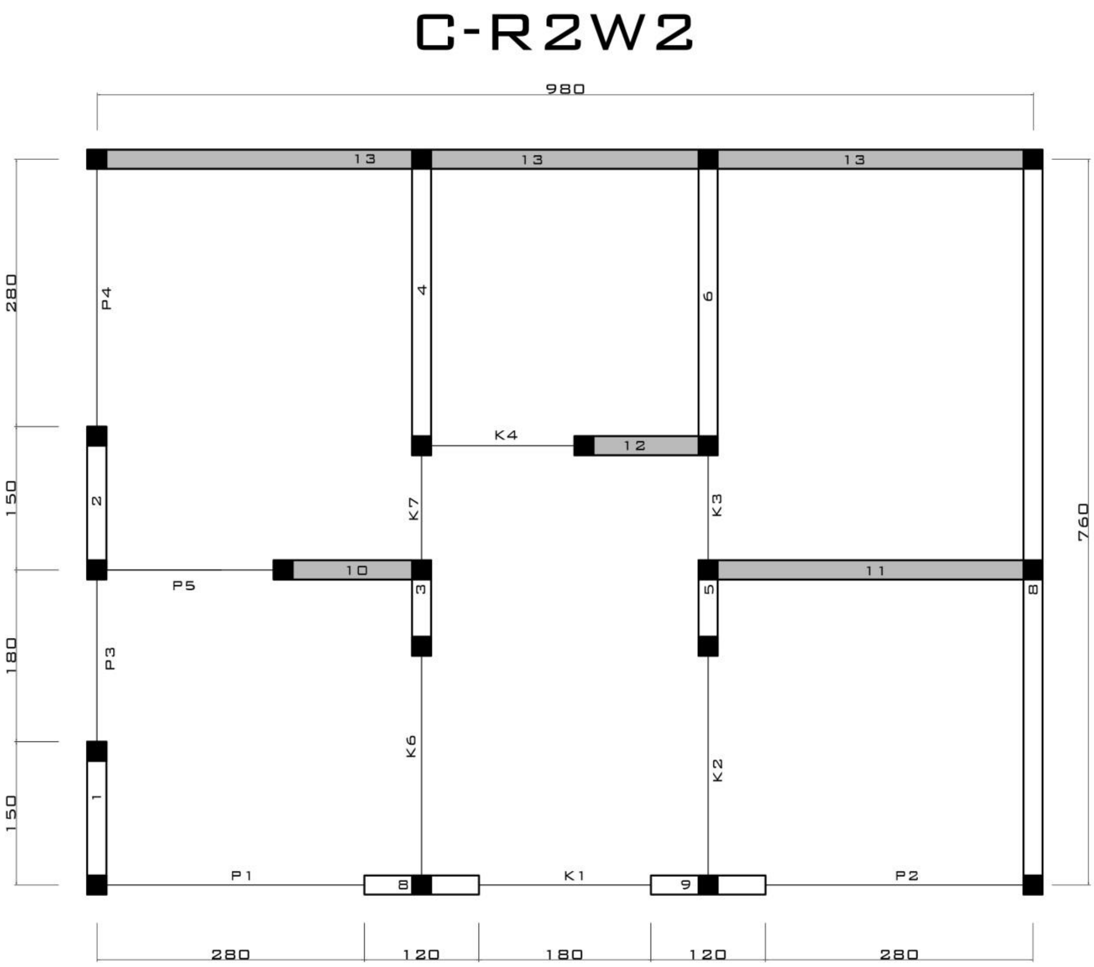

Figure A.10 The floor plan of building model C-R2W2 (all dimensions in cm)

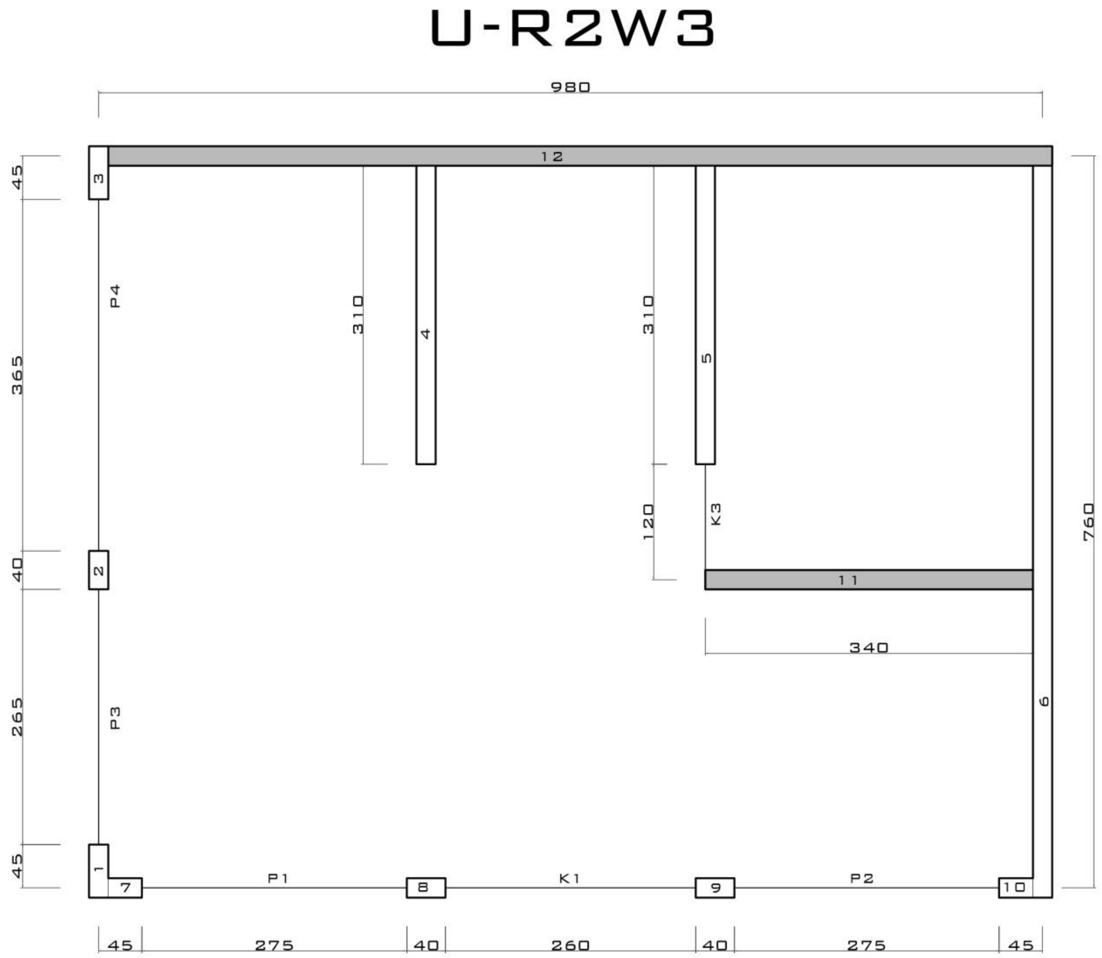

Figure A.11 The floor plan of building model U-R2W3 (all dimensions in cm)

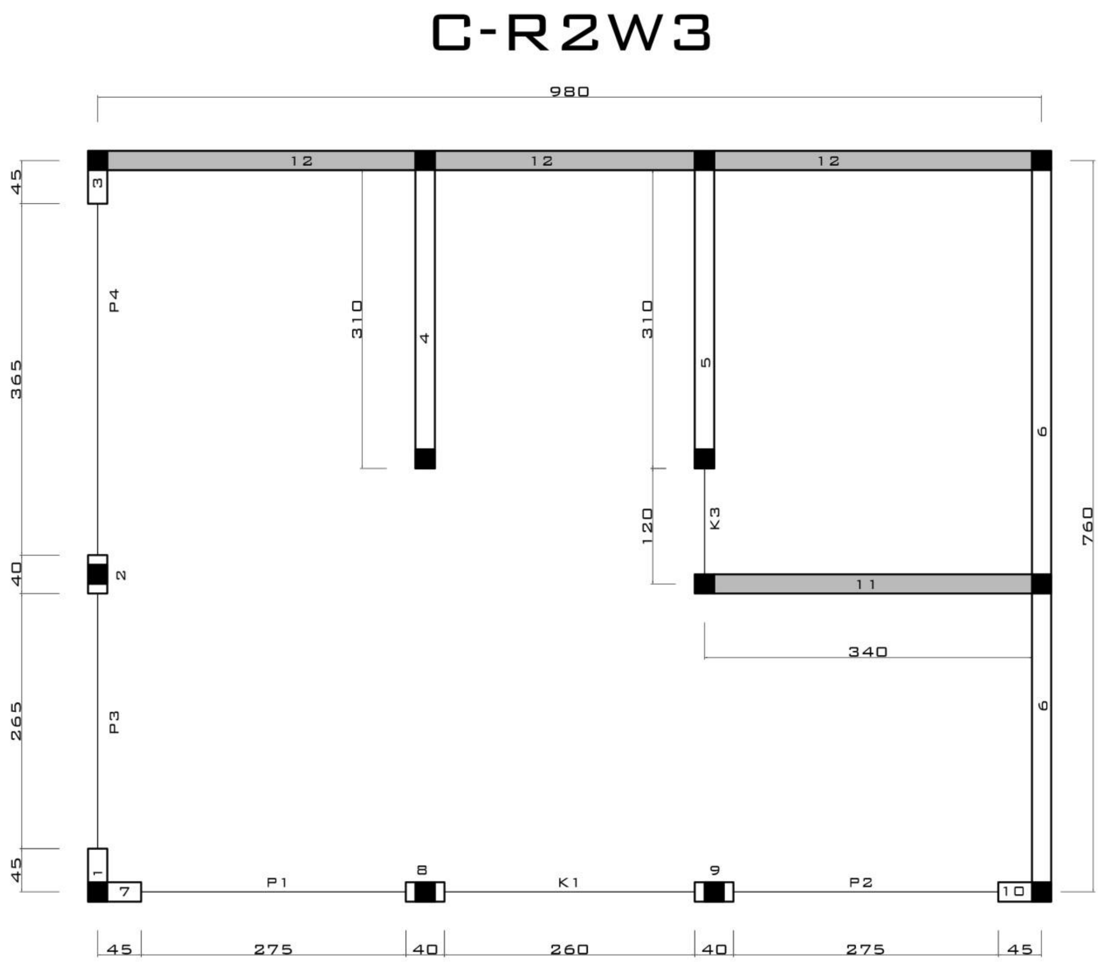

Figure A.12 The floor plan of building model C-R2W3 (all dimensions in cm)

APPENDIX B

GROUND MOTION DATA

B.1 List of Earthquake Records Used in the Performance Analysis.

Table B.1 Earthquake records list.

ECODE	Earthquake	Country	Date	Location	Site Geology	Comp	M_s	Dist. (km)	Depth (km)	F. Type
GM01	Marmara	Turkey	17.08.1999	Istanbul	Stiff Soil	L	7,8	79	17	Strike Slip
GM02	Lazio Abruz-zo	Italy	07.05.1984	Scafa	Rock	NS	5,8	60	8	Normal
GM03	Campano-Lucano	Italy	23.11.1980	Sturno	Rock	NS	6,9	32	16	Normal
GM04	Kalamata	Greece	13.09.1986	Kalamata-Prefecture	Stiff Soil	N355	5,8	9	8	Normal
GM05	Loma Prieta	USA	18.10.1989	Gilroy Array #3	USGS (C)	90	7,1	14,4	17,6	Oblique
GM06	Montenegro	Form. Yugoslavia	15.04.1979	Petrovac, Hotel Oliva	Stiff Soil	NS	7,0	25	12	Thrust
GM07	Whittier Nar-rows	USA	01.10.1987	Cedar Hill Nur-sery, Tarzana	Alluvium / Silts-tone	90	5,8	41,1	14,7	Thrust/Reverse
GM08	Chi Chi	Taiwan	20.09.1999	TCU071	Class D (UBC97)	EW	7,6	4,9	10	Thrust/Reverse
GM09	Cape Men-docino	USA	25.04.1992	Petrolia, General Store	Alluvium	90	7,1	15,9	15	Thrust / Reverse
GM10	Düzce	Turkey	12.11.1999	Bolu	Soil	NS	7,3	5,5	10	Strike Slip

125

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Table B.1 Earthquake records list.(continued)

ECODE	PGA (g)	PGV (cm/s)	V/A (s)	EPA (g)	HI (cm)	AI (cm/s)	EI (cm/s)	Δt_{eff} (s)	Sa(T=0.2) (g)	Sa(T=0.5) (g)	Sa(T=1.0) (g)
GM01	0,060	8,46	0,143	0,038	17,81	3,89	15,27	38,12	0,118	0,046	0,060
GM02	0,132	9,47	0,073	0,131	24,48	21,14	13,74	11,00	0,455	0,407	0,064
GM03	0,216	33,06	0,156	0,252	138,98	130,40	99,31	40,02	0,628	0,594	0,254
GM04	0,297	32,27	0,111	0,328	120,03	86,74	53,54	7,08	0,812	0,906	0,348
GM05	0,367	44,67	0,124	0,338	169,18	134,69	89,78	11,37	0,919	0,753	0,405
GM06	0,454	38,82	0,087	0,461	158,21	452,76	87,07	12,00	1,012	1,334	0,621
GM07	0,537	24,22	0,046	0,496	52,69	241,83	30,32	5,03	0,564	0,286	0,208
GM08	0,567	44,45	0,080	0,575	219,24	932,62	165,03	24,54	1,533	1,076	0,747
GM09	0,662	89,45	0,138	0,437	298,53	382,63	150,92	16,11	1,010	1,457	0,992
GM10	0,754	58,25	0,079	0,649	227,51	386,60	124,25	8,55	1,596	1,623	0,777

B.2 Acceleration-Time Graphs of Ground Motion Records.

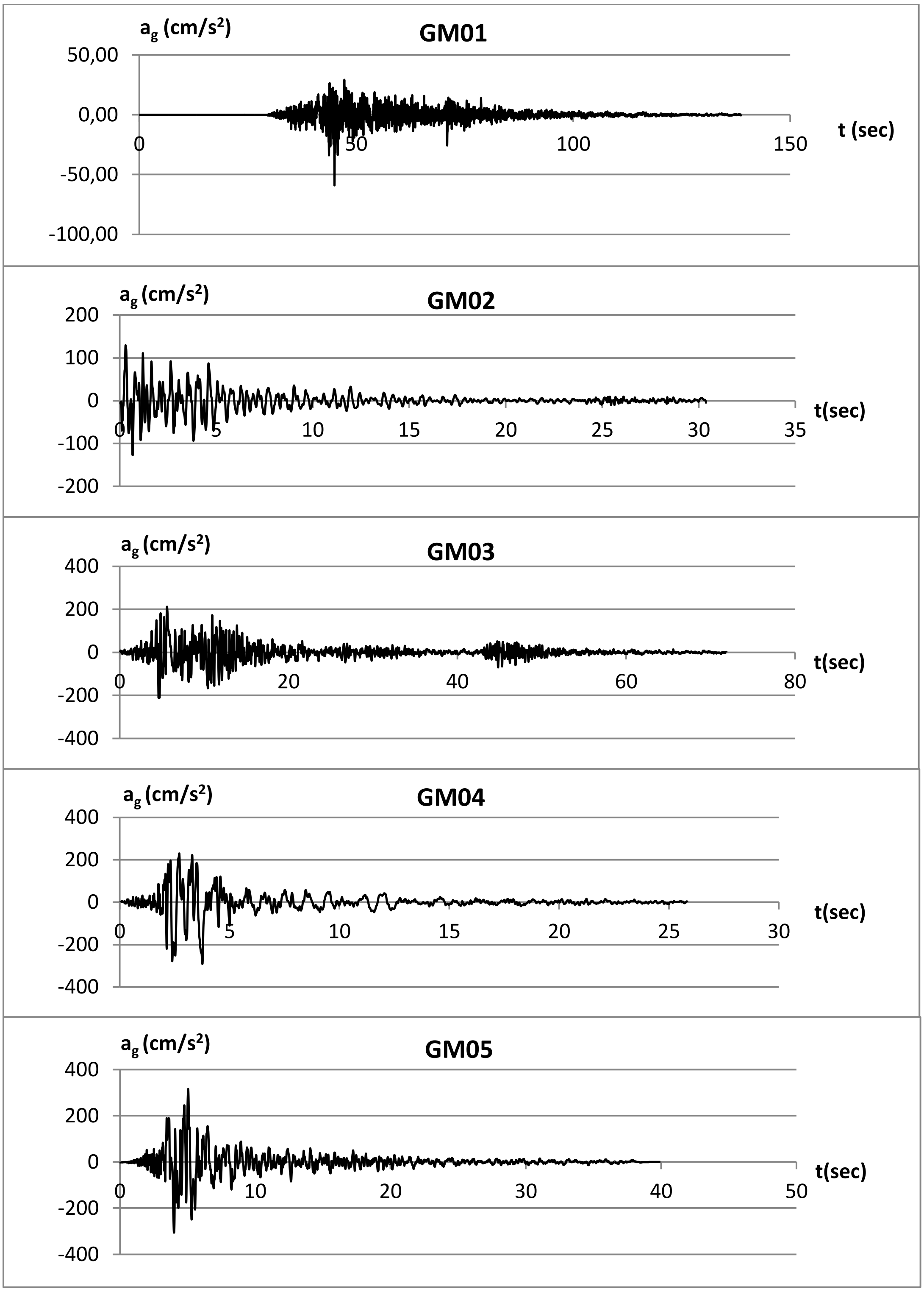
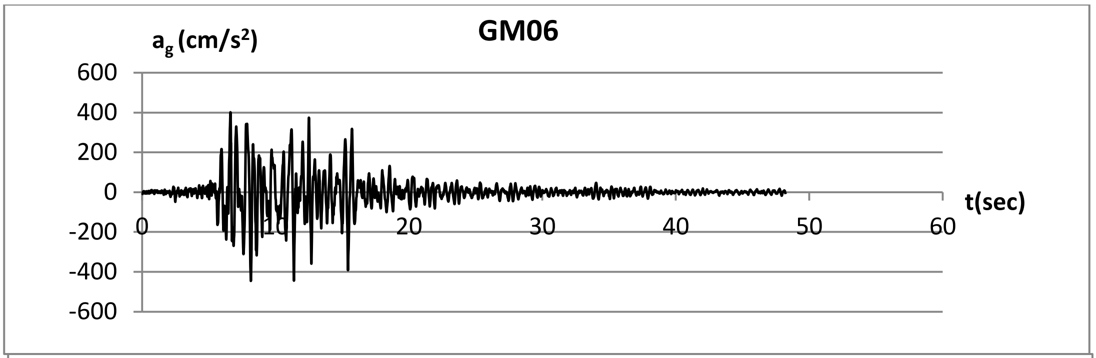
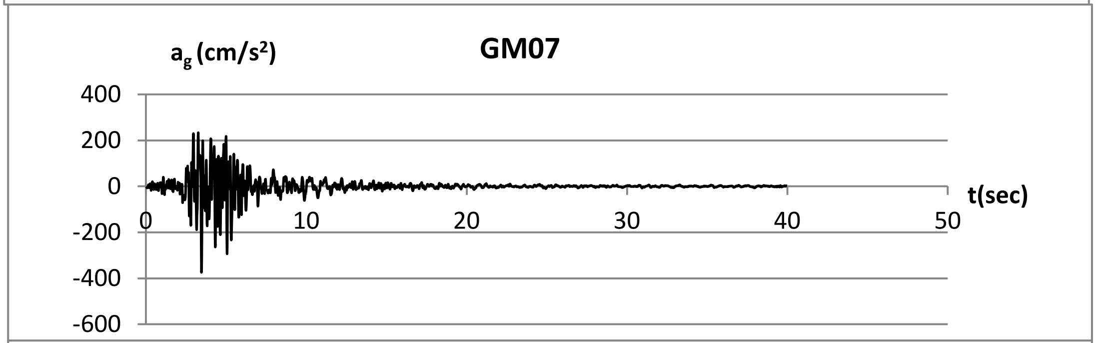
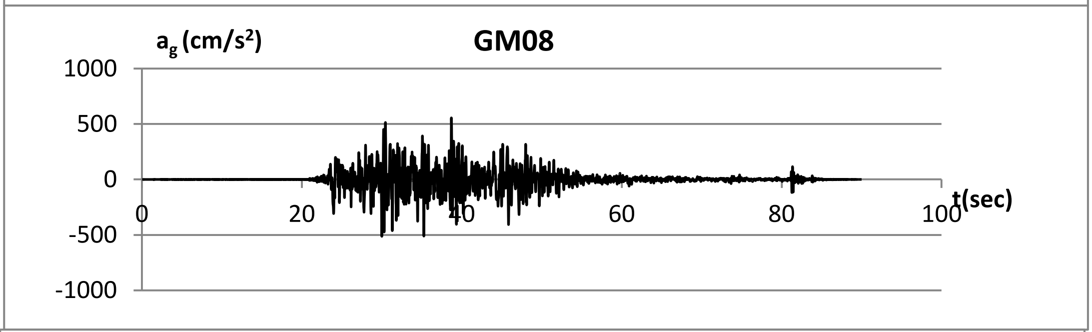
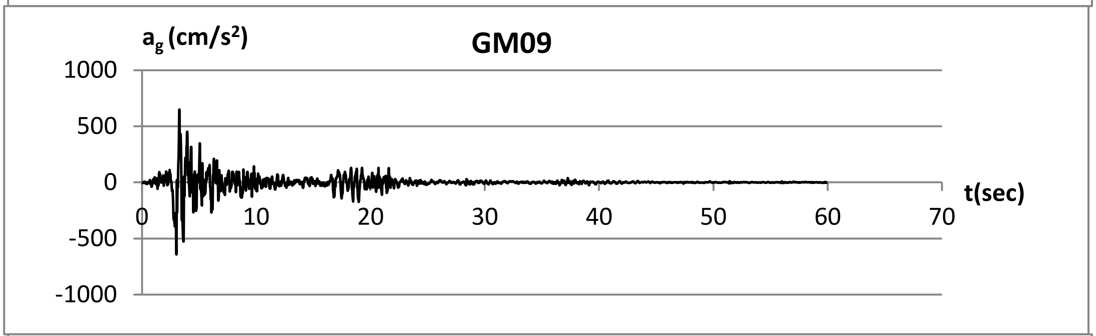


Figure B.1 Acceleration-time graphs of ground motion records









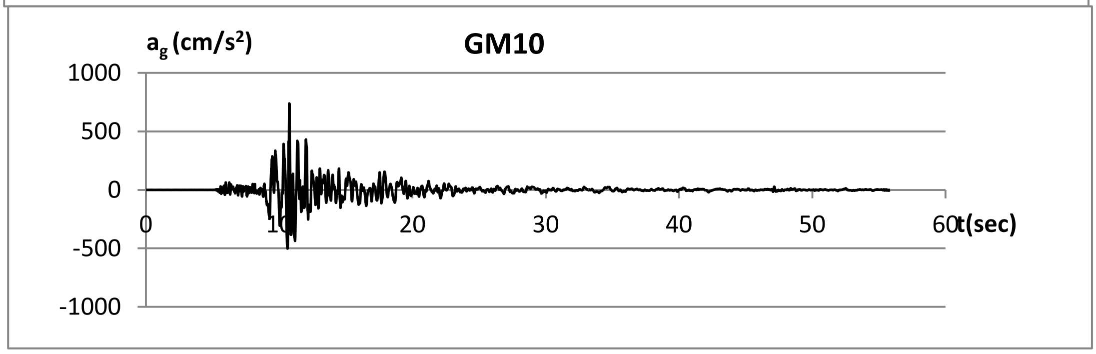


Figure B.1 Acceleration-time graphs of ground motion records (continued)

2. Elastic Spectra for the Ground Motion Records

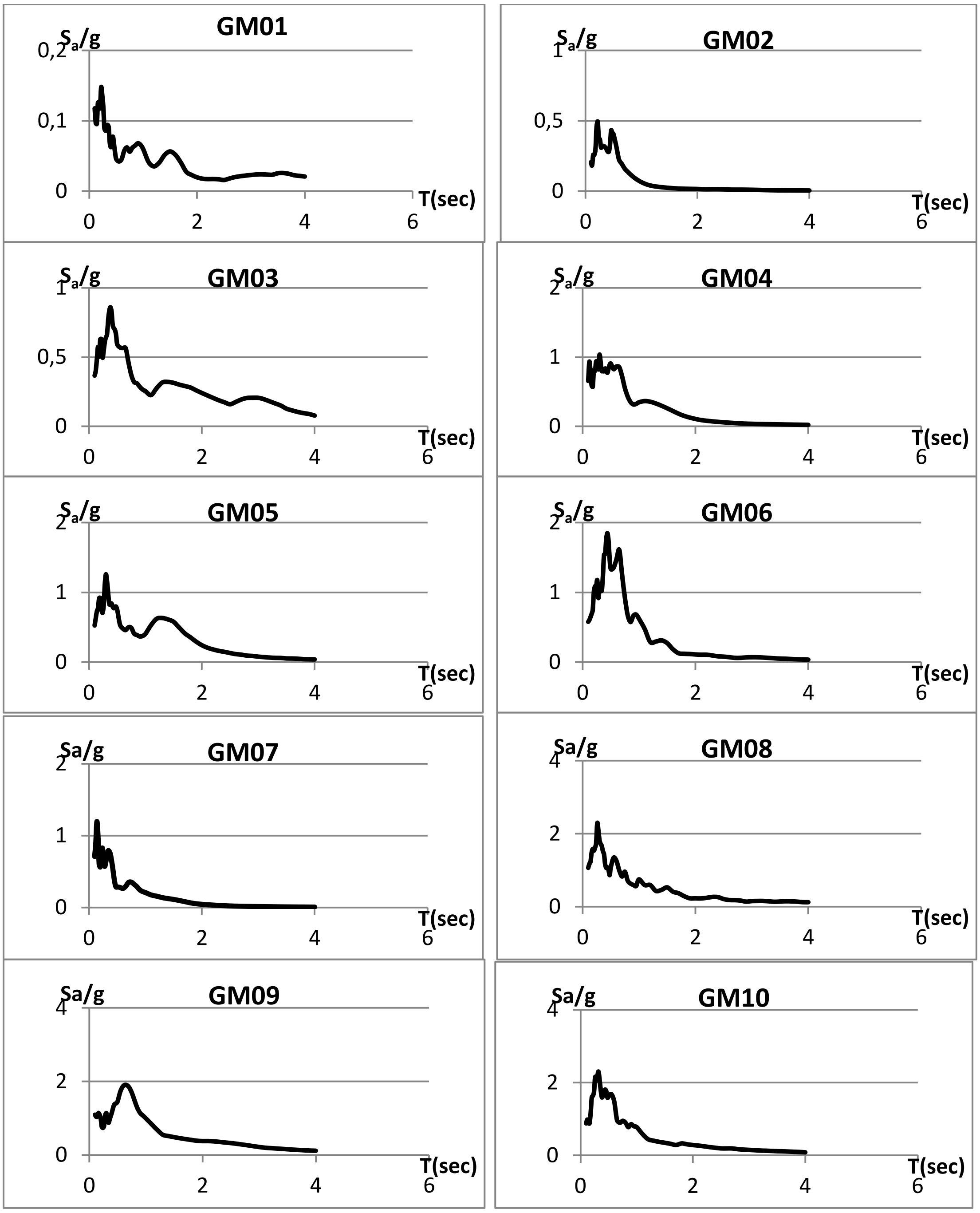


Figure B.2 Elastic spectra for the ground motion records.

APPENDIX C

MATLAB CODE

C.1 Sample Text of the Matlab Code for URM Buildings

```
clear all
clc
  %reading wall data from specified rows of excel
[P, text, alldata]=xlsread('tezplan','R2U12st','B2:L18');
N=size(P,1);
disp(P);
TA=0;
TXA=0;
TYA=0;
for i=1:1:N
    A=P(i,4)*P(i,6)+P(i,5)*P(i,6);
    TA=TA+A;
    XA=A*P(i,2);
    YA=A*P(i,3);
    TXA=TXA+XA;
    TYA=TYA+YA;
end
   %calculation of x and y coordinates of mass center
CMX=TXA/TA;
CMY=TYA/TA;
WRITE1=['X Coordinate of Mass Center=', num2str(CMX)];
WRITE2=['Y Coordinate of Mass Center=', num2str(CMY)];


disp(WRITE1);
disp(WRITE2);

  %enter fm of masonry
fm=2000;
 %determine unknown parameters
Em=1300*fm^0.5;
Gm=0.4*Em;
ft=0.03*fm;
Tkx=0;
Tky=0;
Txky=0;
Tykx=0;
 %calculation of x and y coordinates of rigidity center
for i=1:1:N
    %relative rigidities of walls
    Lx=P(i,4);
    Ly=P(i,5);
    if  Lx==0
        kx=0;
```

```
    else
    kx=1/(P(i,10)^3/P(i,4)+3*P(i,10)/P(i,4));
    end
    if Ly==0
        ky=0;
    else
    ky=1/(P(i,10)^3/P(i,5)+3*P(i,10)/P(i,5));
    end

 Tkx=Tkx+kx;
 Tky=Tky+ky;
 ykx=P(i,3)*kx;
 xky=P(i,2)*ky;
 Txky=Txky+xky;
Tykx=Tykx+ykx;
end
 %x and y coordinates of rigidity center
CRX=Txky/Tky;
CRY=Tykx/Tkx;

disp(CRX);
disp(CRY);
 %distance of mass center to rigidity center ymr
          ymr=CMY-CRY;
          xmr=CMX-CRX;

%calculation of limit states of stiffness, displacement and shear
for each wall
for i=1:1:N
  Lx=P(i,4);
    Ly=P(i,5);
    A=P(i,4)*P(i,6)+P(i,5)*P(i,6);
    if  P(i,10)/(P(i,4)+P(i,5))<=1.0
        b=1;
    elseif  P(i,10)/(P(i,4)+P(i,5))>=1.5
         b=1.5;
    else
        b=P(i,10)/(P(i,4)+P(i,5));
    end
    Vds=A*ft/b*(P(i,11)/ft+1)^0.5;
    Vss=A*(250+0.50*P(i,11));
    Vfl=(P(i,11)*(P(i,4)+P(i,5))^2*P(i,6)/2*(1-
P(i,11)/fm))/0.5*P(i,10);
    Vmax=[Vds Vss Vfl];
    if  Lx==0
        Vymax(i)=min(Vmax);
        Vycr(i)=0.7*Vymax(i);
        Vyu(i)=0.8*Vymax(i);

Kye(i)=(P(i,10)^3/(12*Em*P(i,6)*P(i,5)^3/12)+1.2*P(i,10)/(Gm*A))^
-1;
    Kyvmax(i)=Kye(i)*(1-(1.28*0.5-0.32)^0.5);
    Kyu(i)=Kye(i)*(1-(1.28*0.75-0.32)^0.5);
    dycr(i)= Vycr(i)/Kye(i);
    dyvmax(i)=Vymax(i)/Kyvmax(i);
    dyu(i)=Vyu(i)/Kyu(i);

    Vxmax(i)=0;
```

```
 Vxcr(i)=0;
        Vxu(i)=0;
        Kxe(i)=0;
    Kxvmax(i)=0;
    Kxu(i)=0;
    dxcr(i)=0;
    dxvmax(i)=0;
    dxu(i)=0;

          else
Vxmax(i)=min(Vmax);
 Vxcr(i)=0.7*Vxmax(i);
        Vxu(i)=0.8*Vxmax(i);

Kxe(i)=(P(i,10)^3/(12*Em*P(i,6)*P(i,4)^3/12)+1.2*P(i,10)/(Gm*A))^
-1;
    Kxvmax(i)=Kxe(i)*(1-(1.28*0.5-0.32)^0.5);
    Kxu(i)=Kxe(i)*(1-(1.28*0.75-0.32)^0.5);
    dxcr(i)= Vxcr(i)/Kxe(i);
    dxvmax(i)=Vxmax(i)/Kxvmax(i);
    dxu(i)=Vxu(i)/Kxu(i);
        Vymax(i)=0;
 Vycr(i)=0;
        Vyu(i)=0;
        Kye(i)=0;
    Kyvmax(i)=0;
    Kyu(i)=0;
    dycr(i)=0;
    dyvmax(i)=0;
    dyu(i)=0;

    end
end


      %calculation of storey resistance
          %calculation of storey resistance
      TKx=0;
    TKy=0;
    Ix=0;
    Iy=0;

    x=0;
    y=0;
    c=0;
TVmax=0;
LPmax=0;
    w=N;
a=zeros([1,N]);
TVy=zeros([1600,9]);
    while w>8
         j=0;
            x=x+1;
    for  dx=0:0.0005:0.8
      j=j+1;

    y=y+1;
    d(j,1)=dx;
```

```
        TVx(j,x)=0;
    TKx=0;
    TKy=0;
    Ix=0;
    Iy=0;
        for i=1:1:N
            Lx=P(i,4);
    Ly=P(i,5);
    Ky(i)=0;
 if a(i)==1
     Vx(i)=0;
 elseif Lx==0
                  Kx(i)=0;
                  Ky(i)=Kye(i);
                  Vx(i)=0;
 elseif dx<=dycr(i)
                Vx(i)=  Kxe(i)*dx;
               Kx(i)=Kxe(i);

                elseif dx<=dxvmax(i)
                    Vx(i)=                     Vxcr(i)+(Vxmax(i)-
Vxcr(i))/(dxvmax(i)-dxcr(i))*(dx-dxcr(i));
                    Kx(i)=Vx(i)/dx;

                elseif dx<=dxu(i)
                    Vx(i)=    Vxmax(i)+(Vxu(i)-Vxmax(i))/(dxu(i)-
dxvmax(i))*(dx-dxvmax(i));
                        Kx(i)=Vx(i)/dx;
                else
         Vx(i)= 0;
         a(i)=1;
         c=c+1;
                  end


         %distance of wall to mass center yg
         yg(i)=P(i,3)-CMY;
         xg(i)=P(i,2)-CMX;


          %calculation of Ix and Iy
          Ix=Ix+yg(i)^2*Kx(i)-ymr^2* Kx(i);
          Iy=Iy+xg(i)^2*Ky(i)-xmr^2*Ky(i);
          %calculation of total resistance and stiffness
                TVx(j,x)=TVx(j,x)+Vx(i);
                 TKx=TKx+Kx(i);
  end

         %calculation of rotational moment
         M=TVx(j,x)*ymr;
         J=Ix+Iy;
         Q=M/J;
        TVx(j,x)=0;

         for i=1:1:N
             dxf=dx+Q*(P(i,3)-CRY);
              Lx=P(i,4);
```

```
    Ly=P(i,5);
     Ky(i)=0;

        if a(i)==1
     Vx(i)=0;
 elseif  Lx==0
                  Kx(i)=0;
                  Ky(i)=Kye(i);
                  Vx(i)=0;
              else
                  if dxf<=dxcr(i)
                Vx(i)=  Kxe(i)*dxf;
               Kx(i)=Kxe(i);
                elseif dxf<=dxvmax(i)
                    Vx(i)=                     Vxcr(i)+(Vxmax(i)-
Vxcr(i))/(dxvmax(i)-dxcr(i))*(dxf-dxcr(i));
                    Kx(i)=Vx(i)/dxf;

                elseif dxf<=dxu(i)
                    Vx(i)=    Vxmax(i)+(Vxu(i)-Vxmax(i))/(dxu(i)-
dxvmax(i))*(dxf-dxvmax(i));
                        Kx(i)=Vx(i)/dxf;
          else
         Vx(i)= 0;
                  end
                  end


          %calculation of total resistance and stiffness
                TVx(j,x)=TVx(j,x)+Vx(i);
                 TKx=TKx+Kx(i);

         end

      k= max(TVx);
         if k> TVmax
TVmax=k;
end
 if c>0
             w=w-c;
             c=0;
             ducr(x)=d(j-1,1);
             break
        end
    end

    end

display ducr;
  display (ducr) ;

  %finding yield and performance level for changing ductility
    b=0;
     R=[10,10,10,10,10,10,10];
    for n=2:1:4
        b=b+1;

        for j=20:1:75
```

```
            A1=0;
            A2=0;
dyield=d(j,1);
dpi=dyield*n;
for y=1:1:x
    if dpi<=ducr(y)
        s(y)=TVx(5,y)/d(5,1);
        k=j*n;
       A1=dyield^2*s(y)/2+(dyield*s(y)+TVx(k,y))/2*(dpi-dyield);
        for a=2:1:k
            A=(TVx(a,y)+TVx(a-1,y))/2*(d(a,1)-d(a-1,1));
            A2=A2+A;
        end
        if A1>A2
        RF=A1/A2-1;
        else
             RF=A2/A1-1;
        end
        if RF<R(b)
            R(b)=RF;
            dyfl(b)=d(j,1);
            dpif=n*dyfl;
        end
         break
    end

end
        end
    end
    disp dyfl;
    disp(dyfl);
    disp R;
    disp (R);
    disp s;
    disp (s);
     %writing results to excel
NAME= 'XURMF2R2U12ST'  ;
    x1range='B2';
    sheet='Sheet1';
     xlswrite('tezsonucxR2U1.xlsx',TVx,sheet,x1range)
     x1range='A2';
    sheet='Sheet1';
     xlswrite('tezsonucxR2U1.xlsx',d,sheet,x1range)

           x1range='A1';
    sheet='Sheet1';
     xlswrite('tezsonucxR2U1.xlsx',{NAME},sheet,x1range)

      x1range='M2';
    sheet='Sheet1';
     xlswrite('tezsonucxR2U1.xlsx',s,sheet,x1range)
      x1range='M20';
    sheet='Sheet1';
     xlswrite('tezsonucxR2U1.xlsx',dyfl,sheet,x1range)

     x1range='M21';
    sheet='Sheet1';
     xlswrite('tezsonucxR2U1.xlsx',R,sheet,x1range)
```

C.2 Sample Text of the Matlab Code for CM Buildings

```
clear all
clc
  %reading wall data from specified rows of excel
[P, text, alldata]=xlsread('tezplan','R1U12stCM','B2:N23');
N=size(P,1);
disp(P);
TA=0;
TXA=0;
TYA=0;
for i=1:1:N
    A=P(i,4)*P(i,6)+P(i,5)*P(i,6);
    TA=TA+A;
    XA=A*P(i,2);
    YA=A*P(i,3);
    TXA=TXA+XA;
    TYA=TYA+YA;
end
   %calculation of x and y coordinates of mass center
CMX=TXA/TA;
CMY=TYA/TA;
WRITE1=['X Coordinate of Mass Center=', num2str(CMX)];
WRITE2=['Y Coordinate of Mass Center=', num2str(CMY)];


disp(WRITE1);
disp(WRITE2);

 %enter fm of masonry
fm=2000;
fc=16000;
fy=220000;
 %determine unknown parameters
Em=1300*fm^0.5;
Gm=0.4*Em;
ft=0.03*fm;
Tkx=0;
Tky=0;
Txky=0;
Tykx=0;
 %calculation of x and y coordinates of rigidity center
for i=1:1:N
    %relative rigidities of walls
    Lx=P(i,4);
    Ly=P(i,5);
    if  Lx==0
        kx=0;
    else
    kx=1/(P(i,10)^3/P(i,4)+3*P(i,10)/P(i,4));
    end
    if Ly==0
        ky=0;
    else
    ky=1/(P(i,10)^3/P(i,5)+3*P(i,10)/P(i,5));
```

```
    end

 Tkx=Tkx+kx;
 Tky=Tky+ky;
 ykx=P(i,3)*kx;
 xky=P(i,2)*ky;
 Txky=Txky+xky;
Tykx=Tykx+ykx;
end
 %x and y coordinates of rigidity center
CRX=Txky/Tky;
CRY=Tykx/Tkx;

disp(CRX);
disp(CRY);
 %distance of mass center to rigidity center ymr
          ymr=CMY-CRY;
          xmr=CMX-CRX;

%calculation of limit states of stiffness, displacement and shear
for each wall
for i=1:1:N
  Lx=P(i,4);
    Ly=P(i,5);
    A=P(i,4)*P(i,6)+P(i,5)*P(i,6);

    if  P(i,10)/(P(i,4)+P(i,5))<=1.0
        b=1;
    elseif  P(i,10)/(P(i,4)+P(i,5))>=1.5
         b=1.5;
    else
        b=P(i,10)/(P(i,4)+P(i,5));
    end
    C=2*b*5/4/P(i,10);
    vm=ft/b*(P(i,11)/ft+1)^0.5;
    Vcr=ft/(C*b)*(1+(C^2*(1+P(i,11)/ft)+1)^0.5)*A*0.7;

Vmax=ft/(C*b)*(1+(C^2*(1+P(i,11)/ft)+1)^0.5)*A+4*0.8059*0.010^2*(
fc*fy)^0.5;
    if  Lx==0
        Vymax(i)=Vmax;
        Vycr(i)=Vcr;
        Vyu(i)=0.8*Vymax(i);

Kye(i)=(P(i,10)^3/(12*Em*P(i,6)*P(i,5)^3/12)+1.2*P(i,10)/(Gm*A))^
-1;
    Kyvmax(i)=Kye(i)*(1-(1.52*0.5-0.38)^0.5);
    Kyu(i)=Kye(i)*(1-(1.52*0.75-0.38)^0.5);
    dycr(i)= Vycr(i)/Kye(i);
    dyvmax(i)=Vymax(i)/Kyvmax(i);
    dyu(i)=Vyu(i)/Kyu(i);
    x=P(i,13);
    if x==1
        Vymax(i)=1.28*Vmax;
        Vycr(i)=Vcr;
        Vyu(i)=0.8*Vymax(i);

    dycr(i)=0.83*dycr(i);
```

```
    dyvmax(i)=1.76*dyvmax(i);
    dyu(i)=1.51* dyu(i);
    elseif x==2
     Vymax(i)=1.43*Vmax;
        Vycr(i)=Vcr;
        Vyu(i)=0.8*Vymax(i);

    dycr(i)=0.53*dycr(i);
    dyvmax(i)=1.82*dyvmax(i);
    dyu(i)=1.75* dyu(i);
    Vxmax(i)=0;
    elseif x==3
        Vymax(i)=0;
 Vycr(i)=0;
        Vyu(i)=0;
        Kye(i)=0;
    Kyvmax(i)=0;
    Kyu(i)=0;
    dycr(i)=0;
    dyvmax(i)=0;
    dyu(i)=0;
    end
 Vxcr(i)=0;
        Vxu(i)=0;
        Kxe(i)=0;
    Kxvmax(i)=0;
    Kxu(i)=0;
    dxcr(i)=0;
    dxvmax(i)=0;
    dxu(i)=0;

          else
Vxmax(i)=Vmax;
 Vxcr(i)=Vcr;
        Vxu(i)=0.8*Vxmax(i);

Kxe(i)=(P(i,10)^3/(12*Em*P(i,6)*P(i,4)^3/12)+1.2*P(i,10)/(Gm*A))^
-1;
    Kxvmax(i)=Kxe(i)*(1-(1.52*0.5-0.38)^0.5);
    Kxu(i)=Kxe(i)*(1-(1.52*0.75-0.38)^0.5);
    dxcr(i)= Vxcr(i)/Kxe(i);
    dxvmax(i)=Vxmax(i)/Kxvmax(i);
    dxu(i)=Vxu(i)/Kxu(i);
    if x==1
        Vxmax(i)=1.28*Vmax;
        Vxcr(i)=Vcr;
        Vxu(i)=0.8*Vxmax(i);

    dxcr(i)=0.83*dxcr(i);
    dxvmax(i)=1.76*dxvmax(i);
    dxu(i)=1.51* dxu(i);
    elseif x==2
     Vxmax(i)=1.43*Vmax;
        Vxcr(i)=Vcr;
        Vxu(i)=0.8*Vxmax(i);

    dxcr(i)=0.53*dxcr(i);
    dxvmax(i)=1.82*dxvmax(i);
```

```
    dxu(i)=1.75* dxu(i);
    Vxmax(i)=0;
    elseif x==3
        Vxmax(i)=0;
 Vxcr(i)=0;
        Vxu(i)=0;
        Kxe(i)=0;
    Kxvmax(i)=0;
    Kxu(i)=0;
    dxcr(i)=0;
    dxvmax(i)=0;
    dxu(i)=0;
    end

        Vymax(i)=0;
 Vycr(i)=0;
        Vyu(i)=0;
        Kye(i)=0;
    Kyvmax(i)=0;
    Kyu(i)=0;
    dycr(i)=0;
    dyvmax(i)=0;
    dyu(i)=0;

    end
end


    %calculation of storey resistance
         %calculation of storey resistance
    TKx=0;
    TKy=0;
    Ix=0;
    Iy=0;

    x=0;
    y=0;
    c=0;
TVmax=0;
LPmax=0;
    w=N;
a=zeros([1,N]);
TVy=zeros([1600,9]);
    while w>10
         j=0;
            x=x+1;
    for  dx=0:0.0005:0.8
      j=j+1;

    y=y+1;
    d(j,1)=dx;

        TVx(j,x)=0;
    TKx=0;
    TKy=0;
    Ix=0;
    Iy=0;
        for(i=1:1:N)
```

```
            Lx=P(i,4);
    Ly=P(i,5);
    Ky(i)=0;
 if a(i)==1
     Vx(i)=0;
 elseif Lx==0
                  Kx(i)=0;
                  Ky(i)=Kye(i);
                  Vx(i)=0;
 elseif dx<=dxcr(i)
                Vx(i)=  Kxe(i)*dx;
               Kx(i)=Kxe(i);

                elseif dx<=dxvmax(i)
                    Vx(i)=                    Vxcr(i)+(Vxmax(i)-
Vxcr(i))/(dxvmax(i)-dxcr(i))*(dx-dxcr(i));
                    Kx(i)=Vx(i)/dx;

                elseif dx<=dxu(i)
                    Vx(i)=    Vxmax(i)+(Vxu(i)-Vxmax(i))/(dxu(i)-
dxvmax(i))*(dx-dxvmax(i));
                        Kx(i)=Vx(i)/dx;
                else
         Vx(i)= 0;
         a(i)=1;
         c=c+1;
                  end


         %distance of wall to mass center yg
         yg(i)=P(i,3)-CMY;
         xg(i)=P(i,2)-CMX;


          %calculation of Ix and Iy
          Ix=Ix+yg(i)^2*Kx(i)-ymr^2* Kx(i);
          Iy=Iy+xg(i)^2*Ky(i)-xmr^2*Ky(i);
          %calculation of total resistance and stiffness
                TVx(j,x)=TVx(j,x)+Vx(i);
                 TKx=TKx+Kx(i);
   end

         %calculation of rotational moment
         M=TVx(j,x)*ymr;
         J=Ix+Iy;
         Q=M/J;
        TVx(j,x)=0;

         for(i=1:1:N)
             dxf=dx+Q*(P(i,3)-CRY);
              Lx=P(i,4);
    Ly=P(i,5);
     Ky(i)=0;

        if a(i)==1
     Vx(i)=0;
 elseif  Lx==0
                  Kx(i)=0;
```

```
                  Ky(i)=Kye(i);
                  Vx(i)=0;
              else
                  if dxf<=dxcr(i)
                Vx(i)=  Kxe(i)*dxf;
               Kx(i)=Kxe(i);
                elseif dxf<=dxvmax(i)
                    Vx(i)=                          Vxcr(i)+(Vxmax(i)-
Vxcr(i))/(dxvmax(i)-dxcr(i))*(dxf-dxcr(i));
                    Kx(i)=Vx(i)/dxf;

                elseif dxf<=dxu(i)
                    Vx(i)=     Vxmax(i)+(Vxu(i)-Vxmax(i))/(dxu(i)-
dxvmax(i))*(dxf-dxvmax(i));
                        Kx(i)=Vx(i)/dxf;
          else
         Vx(i)= 0;
                  end
                  end


          %calculation of total resistance and stiffness
                TVx(j,x)=TVx(j,x)+Vx(i);
                 TKx=TKx+Kx(i);

         end

      k= max(TVx);
         if k> TVmax
TVmax=k;
end
 if c>0
             w=w-c;
             c=0;
             ducr(x)=d(j-1,1);
             break
        end
    end

end
display ducr;
  display (ducr) ;

  %finding yield and performance level for changing ductility
    b=0;
     R=[10,10,10,10,10,10,10];
    for n=2:1:8
        b=b+1;

        for j=20:1:130

            A1=0;
            A2=0;
dyield=d(j,1);
dpi=dyield*n;
for y=1:1:x
    if dpi<=ducr(y)
        s(y)=TVx(5,y)/d(5,1);
```

```
        k=j*n;
    A1=dyield^2*s(y)/2+(dyield*s(y)+TVx(k,y))/2*(dpi-dyield);
        for a=2:1:k
            A=(TVx(a,y)+TVx(a-1,y))/2*(d(a,1)-d(a-1,1));
            A2=A2+A;
        end
        if A1>A2
        RF=A1/A2-1;
        else
             RF=A2/A1-1;
        end
        if RF<R(b)
            R(b)=RF;
            dyfl(b)=d(j,1);
            dpif=n*dyfl;
        end
         break
    end

end
        end
    end
    disp dyfl;
    disp(dyfl);
    disp R;
    disp (R);
    disp s;
    disp (s);
     %writing results to excel
NAME= 'XCMF2R1U12ST'  ;
    x1range='B2';
    sheet='Sheet2';
     xlswrite('tezsonucxR1U1.xlsx',TVx,sheet,x1range)
     x1range='A2';
    sheet='Sheet2';
     xlswrite('tezsonucxR1U1.xlsx',d,sheet,x1range)

           x1range='A1';
    sheet='Sheet2';
     xlswrite('tezsonucxR1U1.xlsx',{NAME},sheet,x1range)

x1range='M2';
    sheet='Sheet2';
     xlswrite('tezsonucxR1U1.xlsx',s,sheet,x1range)
      x1range='M20';
    sheet='Sheet2';
     xlswrite('tezsonucxR1U1.xlsx',dyfl,sheet,x1range)

       x1range='M21';
    sheet='Sheet2';
     xlswrite('tezsonucxR1U1.xlsx',R ,sheet,x1range)
```

APPENDIX D

CAPACITY CURVES

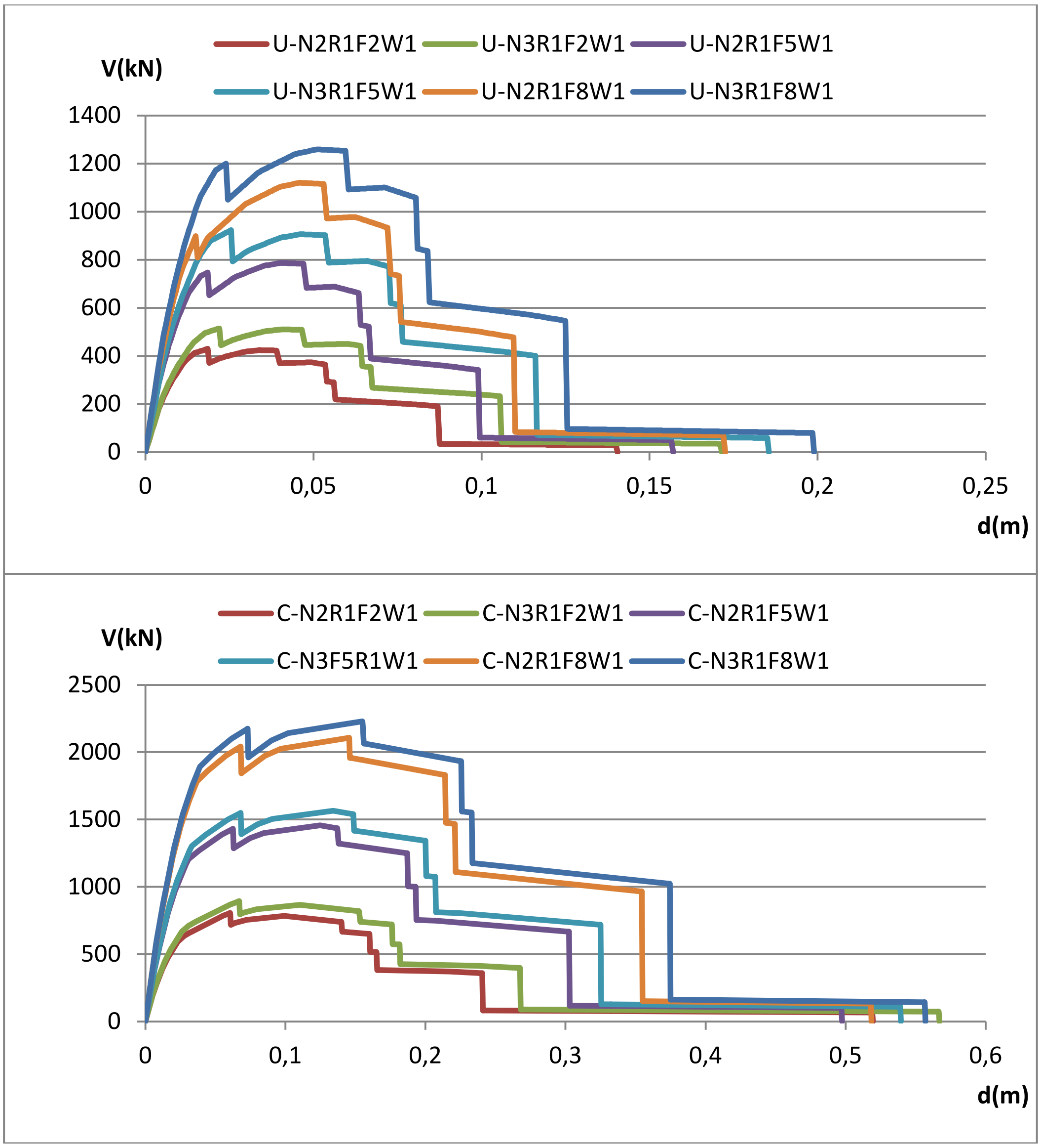


Figure D.1 Capacity curves for the R1W1 building models.

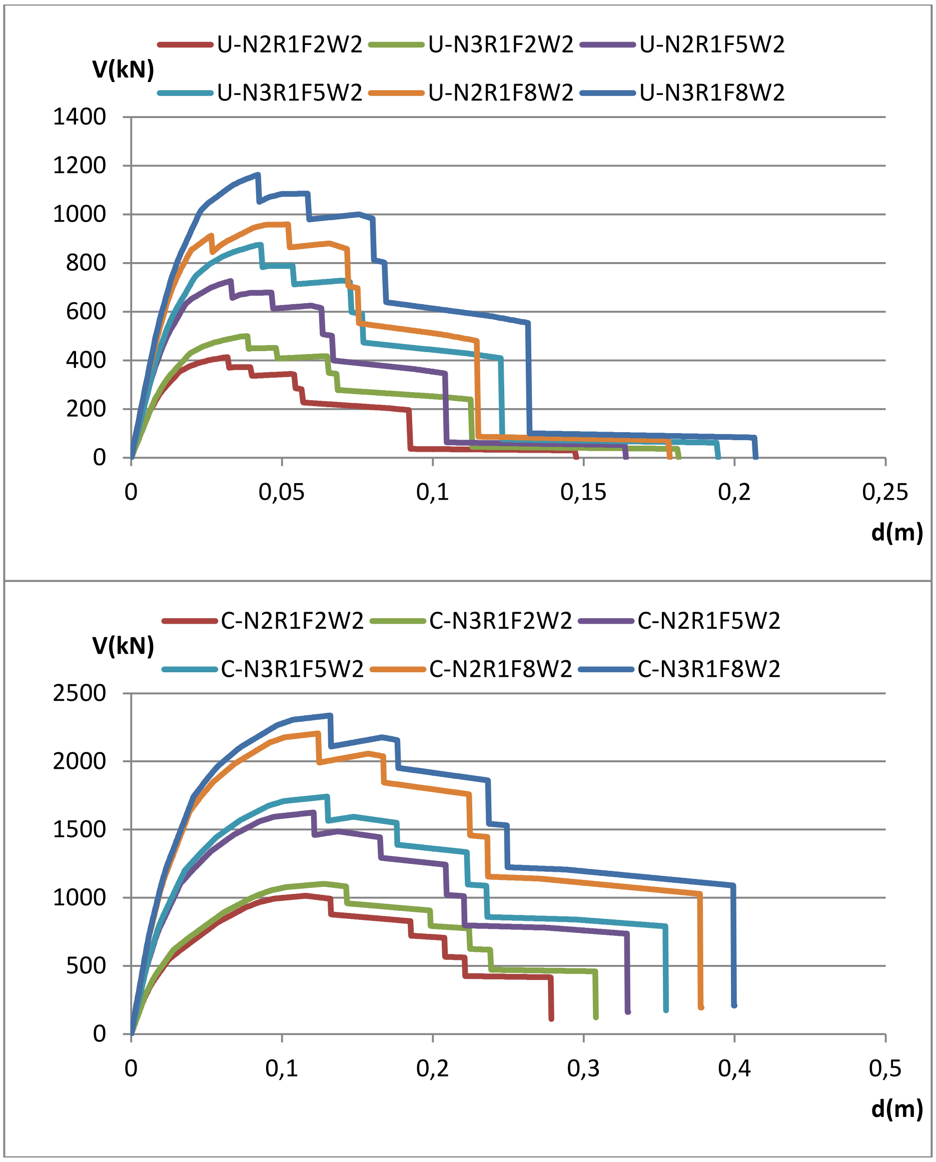


Figure D.2 Capacity curves for the R1W2 building models.

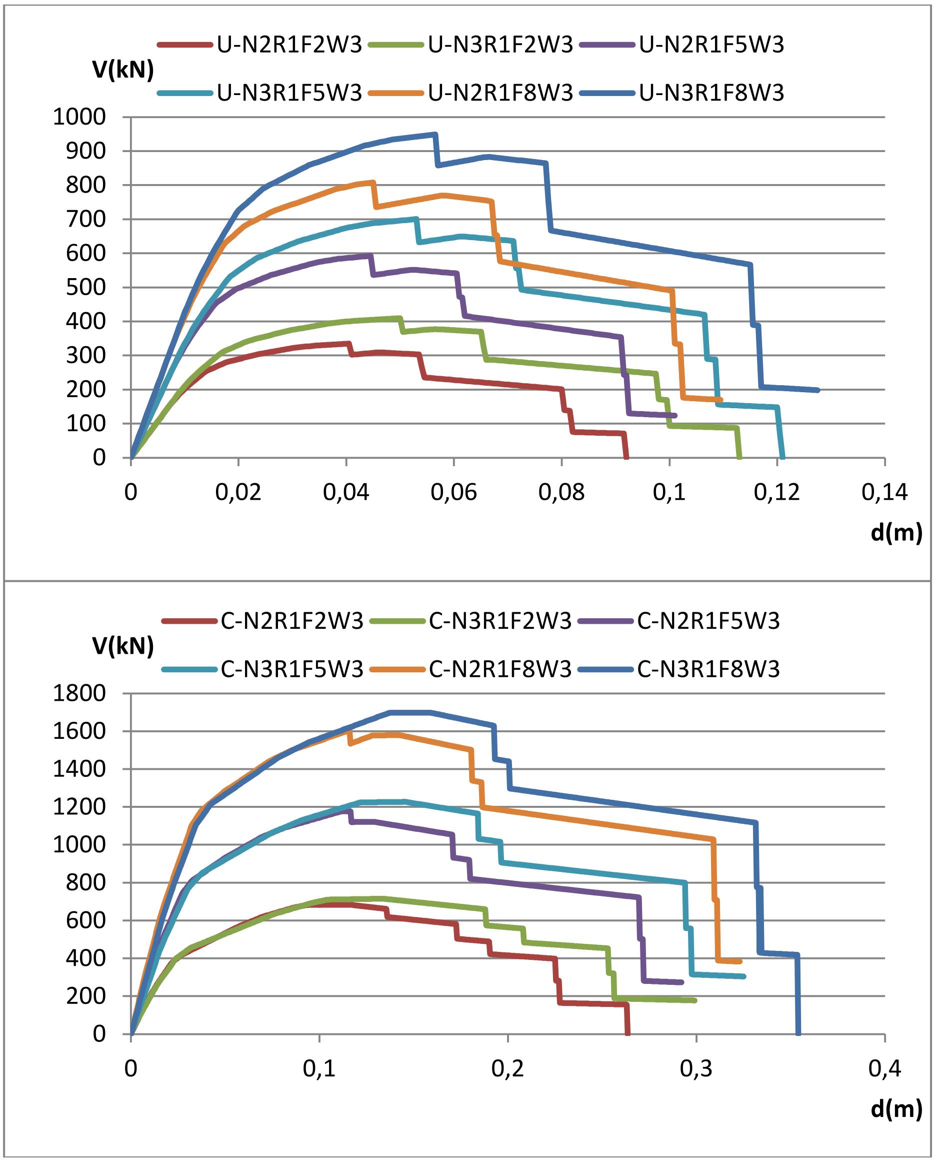


Figure D.3 Capacity curves for the R1W3 building models.

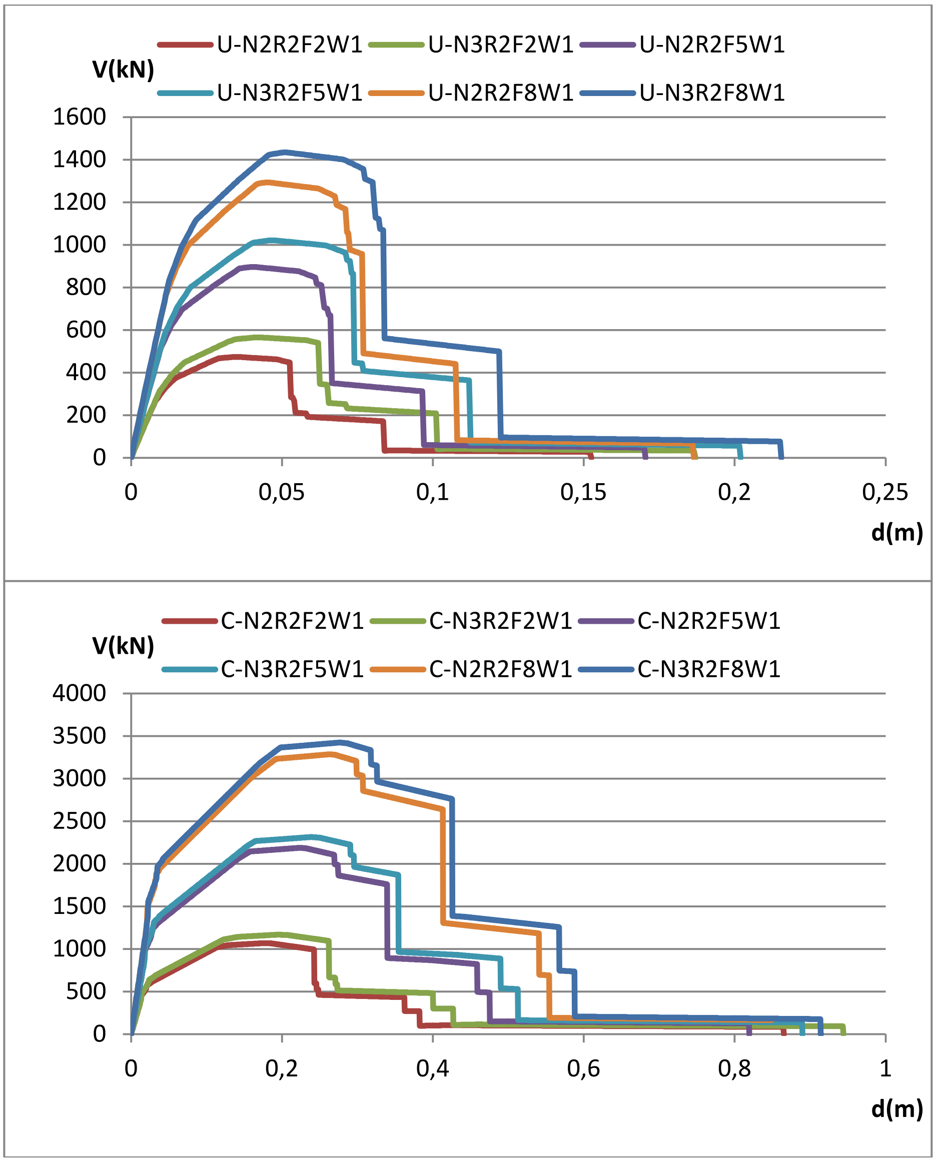


Figure D.4 Capacity curves for the R2W1 building models.

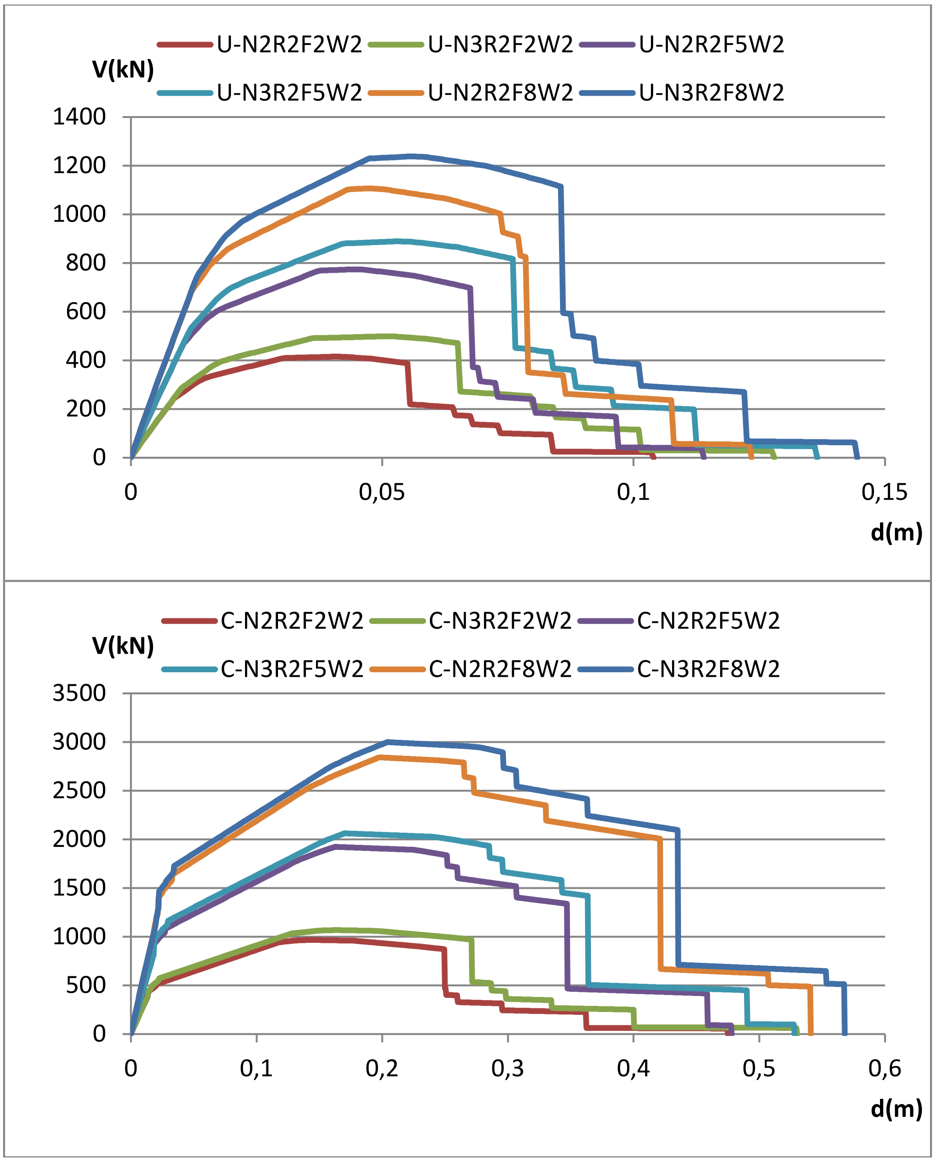


Figure D.5 Capacity curves for the R2W2 building models.

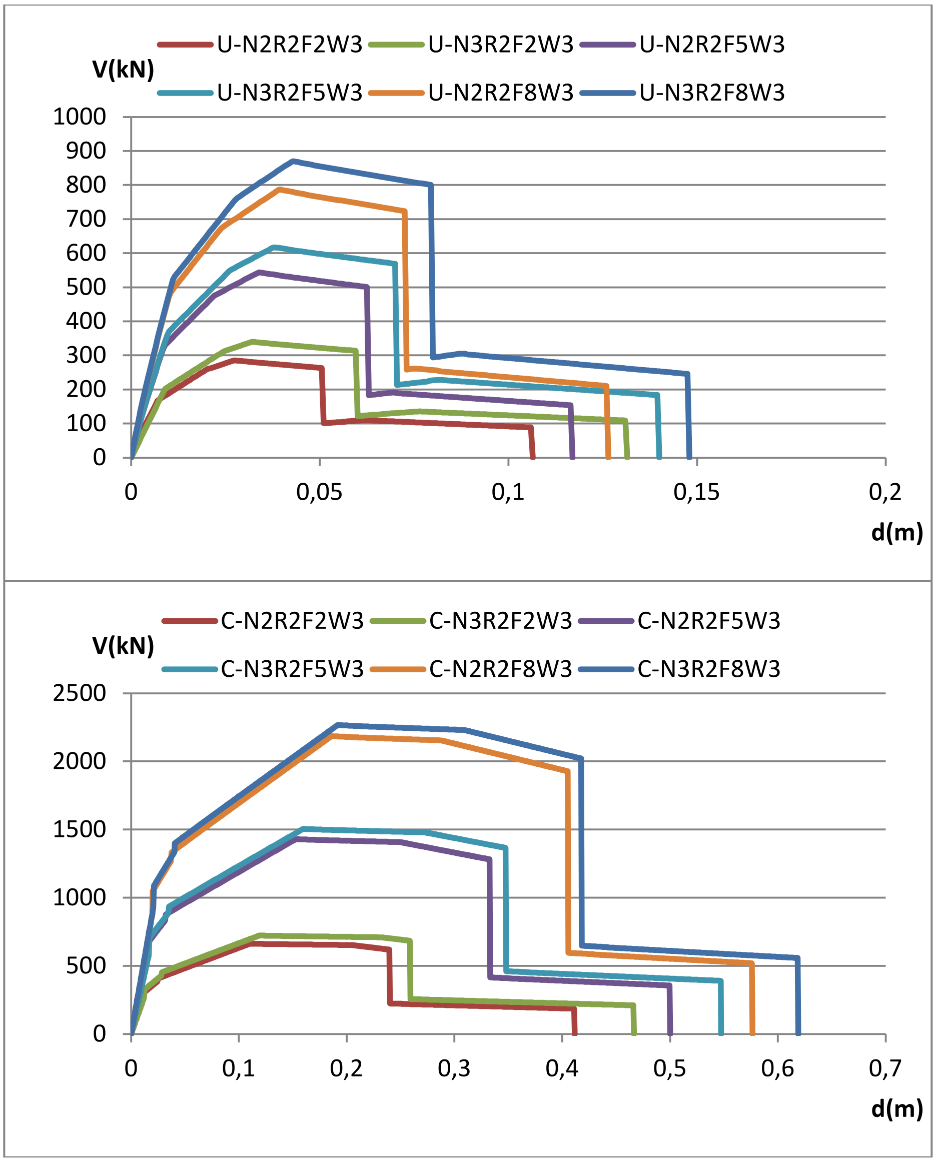


Figure D.6 Capacity curves for the R2W3 building models.

APPENDIX E

RESULTS OF ANALYSIS

1. Damage Index Tables for the R1W1 Building Models.

Table E.1 Damage index tables for the R1W1 building models.

DAMAGE INDEX	PGA(g)	U-N2R1F2W1	U-N3R1F2W1	U-N2R1F5W1	U-N3R1F5W1	U-N2R1F8W1	U-N3R1F8W1
GM01	0.060	0.000	0.082	0.000	0.000	0.000	0.000
GM02	0.132	0.442	0.129	0.375	0.082	0.082	0.082
GM03	0.216	1.000	1.000	0.708	0.442	0.613	0.442
GM04	0.297	1.000	1.000	0.755	0.708	0.755	0.559
GM05	0.367	1.000	1.000	0.708	1.000	0.559	0.375
GM06	0.454	1.000	1.000	1.000	0.755	1.000	0.613
GM07	0.537	0.708	0.559	0.129	0.375	0.491	0.082
GM08	0.567	1.000	1.000	1.000	1.000	1.000	1.000
GM09	0.662	1.000	1.000	1.000	1.000	1.000	1.000
GM10	0.754	1.000	1.000	1.000	1.000	1.000	0.917
DAMAGE INDEX	**PGA(g)**	**C-N2R1F2W1**	**C-N3R1F2W1**	**C-N2R1F5W1**	**C-N3R1F5W1**	**C-N2R1F8W1**	**C-N3R1F8W1**
GM01	0.060	0.000	0.067	0.000	0.000	0.000	0.000
GM02	0.132	0.212	0.067	0.101	0.067	0.101	0.067
GM03	0.216	0.433	0.212	0.345	0.212	0.212	0.212
GM04	0.297	0.433	0.433	0.433	0.424	0.345	0.345
GM05	0.367	0.433	0.758	0.345	0.345	0.345	0.101
GM06	0.454	0.678	0.567	0.678	0.567	0.678	0.450
GM07	0.537	0.212	0.212	0.101	0.212	0.101	0.067
GM08	0.567	0.758	0.758	0.567	0.567	0.567	0.433
GM09	0.662	1.000	0.678	0.792	0.678	0.792	0.678
GM10	0.754	0.758	0.678	0.678	0.567	0.567	0.467

2. Damage Index Tables for the R1W2 Building Models.

Table E.2 Damage index tables for the R1W2 building models.

DAMAGE INDEX	PGA(g)	U-N2R1F2W2	U-N3R1F2W2	U-N2R1F5W2	U-N3R1F5W2	U-N2R1F8W2	U-N3R1F8W2
GM01	0.060	0.000	0.060	0.000	0.000	0.000	0.000
GM02	0.132	0.216	0.060	0.216	0.060	0.302	0.060
GM03	0.216	0.683	1.000	0.454	0.310	0.527	0.395
GM04	0.297	1.000	0.776	0.635	0.443	0.885	0.395
GM05	0.367	1.000	0.885	0.454	1.000	0.468	0.395
GM06	0.454	0.885	0.776	0.885	0.635	0.885	0.635
GM07	0.537	0.635	0.395	0.216	0.310	0.108	0.216
GM08	0.567	1.000	1.000	1.000	0.885	1.000	0.635
GM09	0.662	1.000	1.000	1.000	1.000	1.000	1.000
GM10	0.754	1.000	1.000	1.000	1.000	1.000	1.000
DAMAGE INDEX	**PGA(g)**	**C-N2R1F2W2**	**C-N3R1F2W2**	**C-N2R1F5W2**	**C-N3R1F5W2**	**C-N2R1F8W2**	**C-N3R1F8W2**
GM01	0.060	0.000	0.000	0.000	0.000	0.000	0.000
GM02	0.132	0.216	0.000	0.108	0.000	0.108	0.000
GM03	0.216	0.321	0.321	0.310	0.216	0.216	0.216
GM04	0.297	0.321	0.406	0.406	0.216	0.310	0.216
GM05	0.367	0.321	0.729	0.216	0.216	0.216	0.216
GM06	0.454	0.562	0.321	0.216	0.406	0.562	0.321
GM07	0.537	0.321	0.216	0.108	0.216	0.060	0.108
GM08	0.567	0.562	0.454	0.454	0.454	0.454	0.406
GM09	0.662	0.729	0.562	0.776	0.562	0.635	0.562
GM10	0.754	0.656	0.406	0.454	0.454	0.454	0.406

3. Damage Index Tables for the R1W3 Building Models.

Table E.3 Damage index tables for the R1W3 building models.

DAMAGE INDEX	PGA(g)	U-N2R1F2W3	U-N3R1F2W3	U-N2R1F5W3	U-N3R1F5W3	U-N2R1F8W3	U-N3R1F8W3
GM01	0.060	0.000	0.000	0.000	0.000	0.000	0.000
GM02	0.132	0.157	0.000	0.157	0.000	0.157	0.000
GM03	0.216	1.000	0.823	0.490	0.333	0.396	0.333
GM04	0.297	1.000	0.670	0.553	0.490	0.553	0.396
GM05	0.367	1.000	1.000	1.000	1.000	0.396	0.396
GM06	0.454	1.000	0.553	1.000	0.730	1.000	0.490
GM07	0.537	0.490	0.333	0.396	0.333	0.157	0.333
GM08	0.567	1.000	1.000	1.000	1.000	1.000	1.000
GM09	0.662	1.000	1.000	1.000	1.000	1.000	1.000
GM10	0.754	1.000	0.823	1.000	1.000	1.000	0.823
DAMAGE INDEX	**PGA(g)**	**C-N2R1F2W3**	**C-N3R1F2W3**	**C-N2R1F5W3**	**C-N3R1F5W3**	**C-N2R1F8W3**	**C-N3R1F8W3**
GM01	0.060	0.000	0.000	0.000	0.000	0.000	0.000
GM02	0.132	0.109	0.000	0.000	0.000	0.000	0.000
GM03	0.216	0.333	0.442	0.333	0.109	0.333	0.109
GM04	0.297	0.394	0.333	0.333	0.333	0.333	0.109
GM05	0.367	0.775	0.667	0.333	0.584	0.109	0.109
GM06	0.454	0.442	0.333	0.333	0.333	0.442	0.333
GM07	0.537	0.333	0.251	0.109	0.109	0.000	0.109
GM08	0.567	0.667	0.584	0.442	0.394	0.442	0.333
GM09	0.662	0.667	0.667	0.667	0.442	0.667	0.442
GM10	0.754	0.667	0.442	0.513	0.333	0.442	0.333

4. Damage Index Tables for the R2W1 Building Models

Table E.4 Damage index tables for the R2W1 building models.

DAMAGE INDEX	PGA(g)	U-N2R2F2W1	U-N3R2F2W1	U-N2R2F5W1	U-N3R2F5W1	U-N2R2F8W1	U-N3R2F8W1
GM01	0.060	0,000	0,000	0,000	0,000	0,000	0,000
GM02	0.132	0,235	0,000	0,324	0,000	0,235	0,000
GM03	0.216	0,568	0,324	0,568	0,324	0,324	0,324
GM04	0.297	1,000	0,568	1,000	0,324	0,568	0,324
GM05	0.367	1,000	1,000	0,324	0,324	0,324	0,324
GM06	0.454	0,892	0,568	0,892	0,568	0,739	0,415
GM07	0.537	0,568	0,324	0,235	0,324	0,054	0,082
GM08	0.567	1,000	1,000	1,000	0,657	1,000	0,568
GM09	0.662	1,000	1,000	1,000	1,000	1,000	1,000
GM10	0.754	1,000	1,000	1,000	1,000	1,000	0,892
DAMAGE INDEX	**PGA(g)**	**C-N2R2F2W1**	**C-N3R2F2W1**	**C-N2R2F5W1**	**C-N3R2F5W1**	**C-N2R2F8W1**	**C-N3R2F8W1**
GM01	0.060	0.000	0.000	0.000	0.000	0.000	0.000
GM02	0.132	0.044	0.000	0.044	0.000	0.021	0.000
GM03	0.216	0.333	0.326	0.326	0.252	0.252	0.044
GM04	0.297	0.333	0.333	0.333	0.326	0.326	0.326
GM05	0.367	0.333	0.586	0.326	0.289	0.252	0.252
GM06	0.454	0.333	0.333	0.333	0.333	0.333	0.333
GM07	0.537	0.333	0.289	0.044	0.252	0.000	0.044
GM08	0.567	0.377	0.333	0.333	0.333	0.333	0.333
GM09	0.662	0.659	0.333	0.377	0.333	0.377	0.333
GM10	0.754	0.586	0.333	0.333	0.333	0.333	0.333

5. Damage Index Tables for the R2W2 Building Models

Table E.5 Damage index tables for the R2W2 building models.

DAMAGE INDEX	PGA(g)	U-N2R2F2W2	U-N3R2F2W2	U-N2R2F5W2	U-N3R2F5W2	U-N2R2F8W2	U-N3R2F8W2
GM01	0.060	0.000	0.000	0.000	0.000	0.000	0.000
GM02	0.132	0.268	0.000	0.268	0.000	0.268	0.000
GM03	0.216	1.000	0.513	0.364	0.333	0.333	0.320
GM04	0.297	0.935	0.543	0.653	0.333	0.601	0.333
GM05	0.367	1.000	1.000	0.574	0.333	0.333	0.333
GM06	0.454	1.000	0.653	1.000	0.653	1.000	0.601
GM07	0.537	0.543	0.333	0.000	0.333	0.000	0.320
GM08	0.567	1.000	1.000	1.000	1.000	0.667	0.574
GM09	0.662	1.000	1.000	1.000	1.000	1.000	1.000
GM10	0.754	1.000	1.000	1.000	1.000	0.667	1.000
DAMAGE INDEX	PGA(g)	C-N2R2F2W2	C-N3R2F2W2	C-N2R2F5W2	C-N3R2F5W2	C-N2R2F8W2	C-N3R2F8W2
GM01	0.060	0.000	0.000	0.000	0.000	0.000	0.000
GM02	0.132	0.023	0.000	0.023	0.000	0.000	0.000
GM03	0.216	0.333	0.333	0.333	0.277	0.023	0.023
GM04	0.297	0.333	0.333	0.333	0.333	0.333	0.333
GM05	0.367	0.333	0.587	0.333	0.333	0.289	0.277
GM06	0.454	0.333	0.333	0.333	0.333	0.333	0.333
GM07	0.537	0.333	0.333	0.000	0.277	0.000	0.000
GM08	0.567	0.587	0.587	0.333	0.333	0.333	0.333
GM09	0.662	1.000	0.333	0.357	0.333	0.357	0.333
GM10	0.754	0.587	0.333	0.333	0.333	0.333	0.333

6. Damage Index Tables for the R2W3 Building Models

Table E.6 Damage index tables for the R2W3 building models.

DAMAGE INDEX	PGA(g)	U-N2R2F2W3	U-N3R2F2W3	U-N2R2F5W3	U-N3R2F5W3	U-N2R2F8W3	U-N3R2F8W3
GM01	0.060	0.000	0.000	0.000	0.000	0.000	0.000
GM02	0.132	0.259	0.000	0.259	0.000	0.259	0.000
GM03	0.216	1.000	1.000	0.333	0.333	0.333	0.259
GM04	0.297	1.000	1.000	0.592	0.592	0.592	0.333
GM05	0.367	1.000	1.000	0.592	1.000	0.333	0.333
GM06	0.454	1.000	1.000	1.000	0.592	1.000	0.592
GM07	0.537	0.592	0.333	0.333	0.259	0.000	0.259
GM08	0.567	1.000	1.000	1.000	1.000	1.000	1.000
GM09	0.662	1.000	1.000	1.000	1.000	1.000	1.000
GM10	0.754	1.000	1.000	1.000	1.000	1.000	1.000

DAMAGE INDEX	PGA(g)	C-N2R2F2W3	C-N3R2F2W3	C-N2R2F5W3	C-N3R2F5W3	C-N2R2F8W3	C-N3R2F8W3
GM01	0.060	0.000	0.000	0.000	0.000	0.000	0.000
GM02	0.132	0.000	0.000	0.000	0.000	0.000	0.000
GM03	0.216	0.333	0.333	0.333	0.284	0.284	0.000
GM04	0.297	0.333	0.333	0.333	0.333	0.333	0.284
GM05	0.367	0.667	0.333	0.333	0.333	0.284	0.284
GM06	0.454	0.333	0.333	0.333	0.333	0.333	0.333
GM07	0.537	0.333	0.284	0.000	0.284	0.000	0.000
GM08	0.567	0.617	0.333	0.333	0.333	0.333	0.333
GM09	0.662	0.617	0.617	0.617	0.333	0.333	0.333
GM10	0.754	0.333	0.333	0.333	0.333	0.333	0.333