SEISMIC RESISTANT AND DUCTILE RESPONDING RAMMED EARTH WALLS REINFORCED WITH STEEL BARS AND HOOPS

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SEISMIC RESISTANT AND DUCTILE RESPONDING RAMMED EARTH WALLS REINFORCED WITH STEEL BARS AND HOOPS

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ABSTRACT

SEISMIC RESISTANT AND DUCTILE RESPONDING RAMMED EARTH WALLS REINFORCED WITH STEEL BARS AND HOOPS

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In this thesis, improving the strength research and development work was done on rammed-earth wall based structures, which are not common in Turkey. Ramming a thin layer of the earth by using low cost materials like clay has many advantages. Rammed earth wall (REW) examples can be seen in Africa, South America, and India. On the other hand, in Turkey, there is limited researches on rammed earth wall. However, researches were mostly concentrated on rammed earth materials. Within the scope of this thesis, earthquake effect and performance of REW will be investigated. Besides, there is research and development work conducted on reinforced REW by using natural materials in buildings; such as steel hoop confinements and steel bars inside of the REW. Cement replacement materials like pozzolan was also used in the mixture of REW. In this way, economical, sustainable, and nature friendly REW construction techniques are planned for the Aegean, Marmara, and Black Sea regions, where earth is rich in clayey content and at the same time within highly seismic zones. As this low cost, nature friendly, and earthquake-proof technique becomes widespread, it would be an important step on housing need of people in economical, safe, and healthy structures by using natural materials.

In this thesis, REW samples were produced. There was horizontal loading test on unreinforced base model. The behavior and performance of base model on quasi-static reversal lateral load was investigated. The optimum water content of the rammed earth was investigated. Performance improvements by using different reinforcement materials so that it may be suitable to be constructed even in the first seismic zone and/or other seismic zones.

First REW with vertical and diagonal reinforcement has shown the best strength performance. However, the last REW with confinement hoops in addition to vertical and diagonal reinforcement has shown the best ductile behavior.

Keywords: Nature Friendly, Reinforced Rammed Earth, Confinement Hoop, Steel Bar, Cement Replacement

ÖZ

ÇELİK ÇUBUK VE HALKALARLA GÜÇLENDİRİLMİŞ DEPREME DAYANIKLI VE SÜNEK SIKIŞTIRILMIŞ TOPRAK DUVARLAR

Yıldız, Dinçer

Yüksek Lisans, İnşaat Mühendisliği Bölümü

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Bu tezde, ülkemizde kullanımı yaygın olmayan fakat kerpiç evlere oranla daha dayanıklı olan sıkıştırılmış-toprak-duvarlar ile ilgili araştırma ve geliştirme çalışmaları yapılmıştır. Kil, kireç gibi doğal ve düşük maliyetli malzemelerin ince katmanlar halinde sıkıştırılarak kullanıldığı Sıkıştırılmış Toprak Duvarlar (STD) ile ilgili dünyada güney Amerika, Afrika, Hindistan benzeri bölgelerde çalışmalar yapılmakta olup ülkemizde çok sınırlı ve daha çok malzeme bilimi açısından çalışma yapılmıştır. Bu tez kapsamında, STD'nin deprem performansı açısından incelemesi yapılmış ve çimento kullanılmadan, puzzolan gibi doğal malzemeler ile güçlendirilmiş depreme dayanıklı STD içeren binalar hakkında araştırma ve geliştirme çalışmaları yapılmıştır. Bu sayede hem daha ekonomik hem de sürdürülebilir ve çevre dostu inşaat teknikleri oluşturulması planlanmıştır. Killi toprakların bulunduğu Ege, Marmara, Karadeniz ve İç Anadolu bölgesi gibi deprem yoğunluğu olan alanlarda geliştirilecek düşük maliyetli ve depreme dayanıklı yapım

tekniğinin yaygınlaştırılması, halkımızın hem ekonomik, hem güvenli, hem de doğal malzemelerden sağlıklı barınma ihtiyacına yönelik önemli bir adım olmuştur.

Yapılan çalışmada, toprak sargılama ve UCS testleri yapılarak, çimento kullanmadan en iyi dayanıma sahip karışım bulunup, ¼ ölçeğinde STD numuneleri üretilmiştir. Güçlendirilmemiş baz model yarı durağan yatay yükleme testi yapılmıştır. Bu baz modelin deprem performansı ve davranışı araştırılmıştır. Duvar sıkıştırmasında optimum su muhtevası ile ilgili malzeme çalışmaları yapılmıştır. Daha sonra, birinci derece deprem bölgeleri veya diğer deprem bölgelerinde STD binalar için duvar içine farklı malzeme ve güçlendirme unsurları katılarak performansının iyileştirilmesi (çatlama sonrası sünek davranış açılarından) çalışmalar yapılmıştır.

Yapılan deneyler sonucunda en iyi dayanımı dışarıdan düşey ve çapraz elemanlarla güçlendirilmiş STD gösterirken, en ideal davranışı yarı sünek davranış sergileyen, düşey ve çapraz elemanlarla güçlendirmenin yanı sıra, sargı etriyesi kullanılan STD göstermiştir.

Anahtar Sözcükler: Sıkıştırılmış Toprak, Güçlendirilmiş Sıkıştırılmış Toprak, Sargı Etriyesi, Çelik Çubuk, Çimento Alternatifi

To My Family

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LIST OF ABBREVIATIONS

FEM	Finite Element Method
LVDT	Linear Variable Differential Transformers
NREW	Natural Rammed Earth Wall
RE	Rammed Earth
REW	Rammed Earth Wall
RREW	Reinforced Rammed Earth Wall
OMC	Optimum Moisture Content
SREW	Stabilized Rammed Earth Wall
UCS	Unconfined Compressive Strength
UrREW	Unreinforced Rammed Earth Wall

CHAPTER 1

INTRODUCTION

1.1 General

Building with earth is one of the oldest and quite widespread construction technique of all times. In early ages, earthen buildings were used because of the lack of technology; however, they were easy to build and sustainable. On the other hand, earthen walls provided good heat isolation. However, in the 21st century earthen walls were preferred due to its nature friendly properties. Carbon dioxide (CO_2) and green-house effect is becoming more significant on the construction industry together with the industrial revolution. Therefore, modern construction sector is more and more putting emphasis on reduction of greenhouse gasses during manufacturing and sustainable and “green structures” are getting more importance. The CO_2 production during cement manufacturing is significant and has adverse effects on the climate. Rammed Earth Walls (REW) production is mostly green and uses no cement, which is an air pollutant during its manufacturing. There are many benefits of REWs and sustainability is one of the most important of these (i.e., production of new REW and houses will significantly reduce air pollutants and can be replicated with minimum intervention to the nature). According to basic building material environmental classification by NIBE, forms of earthen materials have the lowest environmental cost [1].

Rammed earth (RE) has significantly lower embodied energy in comparison to its alternatives as concrete and brick masonry structures (Figure 1.1). Embodied energy is the total energy spent from the manufacturing of construction materials to the transportation of materials to site, and construction works in total. However, embodied energy does not include demolition and transportation of demolished material; therefore, it is the front-end energy spent during the life-cycle impact of a building. [33]. On the other hand, earth has excellent abilities to maintain stable interior air humidity level and has a thermal mass potential superior to that of most alternative building materials [1]. RE keeps the ideal humidity range for asthma sufferers and has also suitable storage conditions for example for books and artworks [2]. Because of its dense and porous properties, RE can be used as a good sound isolator. Especially in the concert halls and recording studies, REW is preferred. In addition, it has excellent sound reverberation characteristics [2].

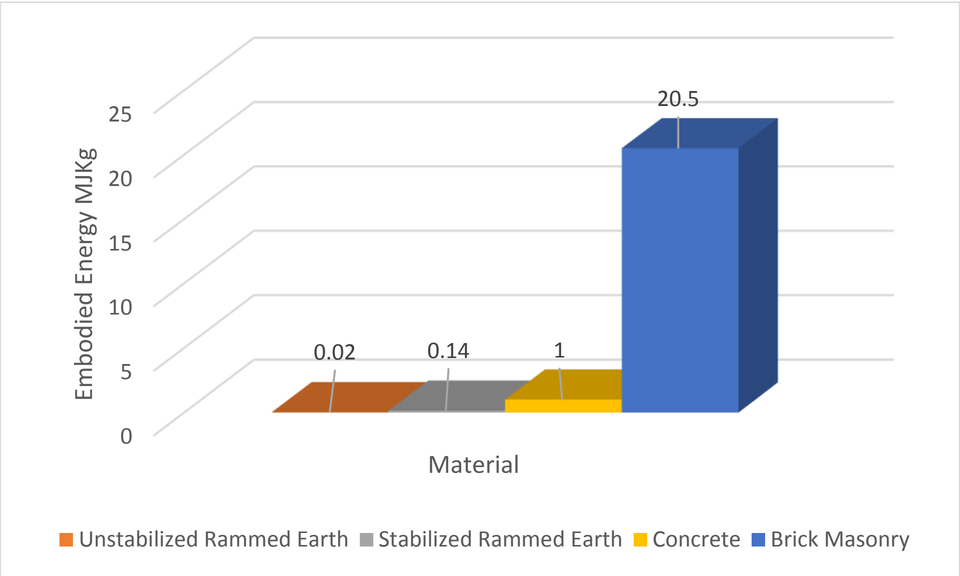


Figure 1.1 Embodied energies of materials

Ramming is a process with compaction of loam layers. Enough loam should be added to the formworks, which is generally 0.15m thick in common practice, and it should be rammed in layers until all walls are completed. The ramming of layers causes thin straight horizontal layers on the surface of the wall. These horizontal lines give an aesthetic view to the wall. A number of color pigments can be added as well as surface texturing may become a field of creative expression in Architecture [2] (Figure 1.2).

Figure 1.2 REW with colored layers (URL - 1)

Measurements of the radiation of beta and gamma rays show that loam has values no higher on average than concrete. Alpha rays emitted by the radioactive gas radon, which cannot penetrate to human body as they are absorbed by the skin; but can be inhaled by breathing, which may cause lung cancer. The exhalation rate of radon given by the OECD [3] for Germany, measured in m Becquerel/kg h [Table 1.1], shows that a clay brick from a clayey soil discharges very little radon [4].

Table 1.1 Exhalation rates of radon in different materials

Material	Exhalation rate of radon
Natural gypsum	25.2
Cement	57.6
Sand	54
Baked clay bricks	5
Lime-sand bricks	13.3
Porous concrete	18

RE walls are economical. The main materials are clay, silt and sand. In a comparison with other building materials, it is easy to obtain these material with a low cost. In ancient times many kind of binders were used in the mixture of the RE structures and also between the ramming layers. Although lime and cement are the most preferred binders, different kinds of organic and inorganic binders were also used. It was not standardized at past. Therefore, people were building RE structures according to formers receipts. However, today there are a lot of specification and code for RE structures. Besides, technology reduced the effort for crushing clay loams, mixing the materials and also ramming. Taking into account all of these, REWs become a low cost, low tech and economical nature friendly alternative for buildings. Nonetheless, RE is not as strong as concrete, not even the stabilized ones.

Constructing REW is not always easy and experience is required [16]. Design should be done by a geotechnical or structural engineer [12]. Enough workmanship is required for mixing and ramming process and engineers should inform workers on techniques.

REW is a low cost construction technique. According to some of the previous researches, cost of REW was defined between 100-400 $ per m^2 for 300 mm thick wall [12]. If clay is available in the construction area, the cost of material and transportation will reduce. Clay can also be found on the construction areas excavation zones. Sand can be obtained from building material distributors or can be found in nature and can be used after sieving process. Fly ash is a waste product of power plants. That is why, it can be obtained with from power plants by paying the cost of transportation only. Lime also can be obtained from construction markets. In this thesis, the cost of materials for 4 m x 3 m x 0.5 m REW was calculated as 88 $ (Table 1.2). For a 96 m2 house, including 3 rooms as shown in Appendix A, the cost for a REW is 12 $/m2, for a concrete slab it is 7.7 $/m^2, and reinforcements for the slab is 20 $/m^2.

Table 1.2 Cost of materials

Sand + Clay %	Clay %	Sand %	Fly Ash %	Lime %	Total
80	40	60	15	5	
Weights (tons)					
8.16	3.264	4.896	1.53	0.51	10.2
Cost per ton ($)					
43	0	43	0	45	88

Different mixings and additional reinforcements were used in previous studies to improve the strength of the RE and increase its earthquake resistance. The aim of this study is to increase the resistance of RREW's performance against seismic forces by using the most suitable mixing proportion with reinforcements.

1.2 Rammed earth from past to present

Earthen constructions have been known for over 9000 years. Adobe houses have been discovered in Russia between 8000 to 6000 BC. In 5000 BC, rammed earth wall foundations have been discovered in Assyria. Earth was used in private houses, religious buildings and defensive structures in ancient cultures. 2500 years ago in the Bam city of Iran, a citadel was constructed by using earth (Figure 1.3). One of the very famous man-made construction which can be visible from space, The Great Wall of China, was built solely with rammed earth technique 4000 years ago (Figure 1.4). Another ancient example is from North America. Between 300 and 900 AD, the core of the Sun Pyramid in Teotihuacan (Figure 1.5), Mexico, was built with rammed earth technique. In early times there was a lack of wood in dry climatic zones. That forced people to build mud bricks without support system or formwork during construction. People were living in underground houses or caves that were dug in the silty soil.

Figure 1.3 Citadel in Bam city of Iran (URL – 2)

Figure 1.4 Great Wall of China (URL – 3)

Figure 1.5 Sun Pyramid in Teotihuacan, Mexico (URL – 4)

In Bronze Age, infill timber-framed houses were discovered in Germany. Heuneburg Fort (Figure 1.6) near the Lake Constance, Germany, was built in the 6th century and, is known as the oldest example of mud brick walls in northern Europe. In the 12th century, almost all early mosques in Africa were built from earth (Figure 1.7). Between the 15th to the 19th centuryies, a rammed earth technique was discovered in France which is called *terre pise*. This technique was first used in the building of Lyon, France, and later also other European countries started to use this technique. Especially Germany conducted researches to improve and to see the most advantageous aspects of rammed earth construction method. They built the tallest house with solid earth walls in Europe, at Weilburg, Germany in 1828 which still stands (Figure 1.8).

Figure 1.6 Infill timber-framed house in Germany [16]

Figure 1.7 A mosque in Africa (URL – 5)

Figure 1.8 The tallest REW building in Germany (URL – 6)

1.3 Literature survey

In the reviewed literature, studies on earthquake resistant Reinforced Rammed Earth Walls (RREW) with “cement alternative materials” could not be found and has been selected as the subject of this thesis. However, there are quite many studies on soil based wall mixture proportioning, effects of confinement, and seismic behavior in general. The following studies, which are grouped by their topics, are necessary to mention because of their similarities to the current study made in this thesis.

1.3.1 Unstabilized rammed earth walls

Jaguin and Augarde [10] presented the laboratory test results of unstabilized rammed earth materials. The aim was to show suction is a source of strength in unstabilized rammed earth. Specimens were prepared at constant water content and were air-dried. According to the different water contents, the suction is measured prior to shearing. Similar to concrete, strength increases when water content reduces. Dry density was highest at 8% water content. After that peak, when water content increase, dry density decreases. Low water contents suction was seen to drop. However, the opposite was apparent in the higher water contents. For shearing, initial low suctions show an increase in suction during shear.

Maniatidis and Walker [13] made a research on structural capacity of rammed earth in compression. They made material tests, large scale prism tests and, full size column tests. Effects of the load eccentricity and slenderness were taken into account. A simple analytical model, using basic strut theory, was developed. A mixing with clay, sand, silt and gravel was prepared. Mixing contained 8-15% clay. Cylinders, 200 mm height and 100 mm diameters were tested. According to modified proctor results, water content was determined as 12.5%. Earth was rammed by using a pneumatic hammer. In different temperatures, specimens were left to dry for 4 weeks. The average dry density was 1850 kg/m3 and the average unconfined compressive strength was 2.46 MPa. Full scale columns, 300 mm wide x 300 mm thick and 1800/2400/3000 mm heights were prepared. Specimens with vertical loads applied both axially or eccentrically, were crushed occurred on one side but did not show visible cracks. For the 1800 mm heights columns, failure occurred suddenly. For 2400 mm height ones, which were subjected to axial loads and loads at 10% eccentricity, failure occurred at 450-600 mm from to top. The 3000 mm heights ones failed at 750-1000mm from the top.

Silva and Oliveira [27] did a research on modeling rammed earth under shear loading. 5 unstabilized rammed earth walls were tested under axial compression and 5 unstabilized rammed earth walls were tested under diagonal compression. Every wall was compacted in 6 layers, these each had a thickness of approximately 84 mm. The average dimensions of the walls were 499x505x117 mm^3. The bulk density was about 2190 kg/m^3 and the mix proportion was 11% clay, 25% silt and 64% gravel. Compression test were carried out under displacement control. LVDT's were used to measure the deformations. Non-linear stress-strain curves occurred. The diagonal compression tests were carried out according to ASTM E 519-10. A low strength cement mortar was used to regularize contact between specimens and the supports. In general, walls exhibited an early peak for shear stress, followed by later shear hardening. Failures occurred just after the cracks close to the early peak shear stress. Further cracks occurred diagonally. Cracks at the interfaces between the layers also occurred.

1.3.2 Stabilized rammed earth walls

Ciancio and Beckett [7] studied the optimum lime content for lime-stabilized rammed earth. Lime and soil can react in three general phenomena. Carbonation, cation exchange and puzzolanic reaction. According to these reactions, to maximize the unconfined compressive strength, optimum lime content was investigated. The samples were tested under environmental conditions and the optimum lime content was found to be 4%. Additionally, the compressive strength peak for porosity/lime ratio was between 3%-4% of the lime content. pH test results have shown that when, lime saturation of the pore water is more than 4% there is an absence of strength and stiffness gain.

Taghiloha [26] conducted research about using rammed earth mixings with recycled aggregate. He used artificial soil and recycled aggregate to find out how much difference exists between them and the natural ones. According to the unconfined compressive strength test results, all samples exceeded 2 MPa which is enough for rammed earth structures and quite close to the natural aggregate. The linear shrinkage of recycled aggregate samples was higher in comparison to natural aggregate. However, the results were quite acceptable for the recommendations.

Pflughoeft-Hassett and Dockter [24] compared the compressive strength of fly ash and bottom ash. They prepared different mixings with different percentages. After 7 days curing, %100 soil and %100 soil + %30 bottom ash has a quite low strength. Samples with fly ash have doubles the compressive strength in compression to only soil one. They test %80 soil and %20 fly ash. Although bottom ash decreased the compressive strength of the soil, they prepared another mixing with %56 soil + %24 bottom ash and %20 fly ash. That sample gave better compressive strength than previous ones. However, none of these samples could exceed the recommendations. Only the ones that have cement addition could reach the recommendations. One of them has soil-bottom ash-fly ash composition with an addition of %5 cement. When they doubled the cement percentage, the compressive strength also almost doubled. On the other hand, bottom ash increased the R-value of the rammed earth specimens while it did not change the compressive strength.

Nabouch and Bui [17] studied on seismic assessment of rammed earth walls using the pushover test. They used the static nonlinear pushover method for the seismic performance of rammed earth wall. Nonlinear shear force-displacement curves were obtained. They scaled down the walls by 1/2 and 1/3 to represent the real walls. Water content was approximately 12%. The walls were built on a concrete beam. Another concrete beam was placed at the top to enable applying a horizontal load. A thin lime mortal layer was added for bonding. Two displacement sensors in vertical and horizontal directions were used to check if any movement occurred of the bottom concrete during the test. Another horizontal displacement sensor was used to measure the displacement of the wall. Vertical loads were added to simulate dead and live loads at the top. Two vertical actuators were used to apply loads. A hydraulic actuator with a displacement control was used to carry out the pushover test. 40 kN horizontal load was applied to the walls. According to results, none of the walls had a brittle behavior. Quasi-diagonal cracks were observed. At the left-lower part a horizontal crack appeared. This crack appeared when the horizontal load reached the 85% of the maximum load. Weak points were the interfaces between layers. However, results shown that, there is an acceptable cohesion between the layers. Local uplifts were developed due to the tensile stresses.

Batur and Kaplan [2] studied on stresses under earthquake load of the masonry walls. According to a specific seismic zone, structure was modelled. The live loads, dead loads and earthquake loads were calculated and assigned to the model. After loading, the stresses that occurred on the wall was obtained and compared with the factor of safety. There are a few kinds of damage of masonry walls. One of them is settlement cracks which occurs at the edges of the structure where the wall connects to the window or to door openings. The second threat to masonry walls is earthquake. In masonry walls, the whole system works as a bearing element. In contrast with of the concrete structures, there is no need for a calculation for both of bearing systems and non-bearing systems. If the strength of the mortar is weaker than the strength of the brick, a crack occurs at the corners. Otherwise, inclined tension cracks occur. The other damage during the earthquake occurs by separating the wall from the flooring. The masonry walls may also have damage in vertical direction. The reason of this the poor binding between the top of the walls with rigid floorings, roof trusses and girders. Other common earthquake-caused damage types occur at the footing areas of the walls as a smash.

1.3.3 Reinforced rammed earth walls

Parreira [23] studied on seismic analysis of rammed earth buildings. They designed the structure in the program SAP2000 to perform a three-dimensional dynamic analysis. The thicknesses of the walls were 50 cm. Three-dimensional elements were used to model the walls and bar elements were used to model the concrete beam at the top of the wall. Walls were modelled as fixed to the ground and linked to the beam. The roof and its structure were not modelled. However, their effects were simulated as loads. The roof was assumed to be a wooden structure and its action was 0,7 kN/m2. All walls were subjected to the tension analysis. According to the analysis results, it was observed that using bond beams over all building walls with structural function is essential. These increases the walls’ stiffness to the vertical bending which guarantees that the whole building has a compatible movement.

The seismic analysis results have shown that reinforcements can solve the problem of the rammed earth walls’ weaknesses. The connection between bond beam and wall should as strong as possible. Anchor bolt can be used to avoid the sliding. Buttress technique was also used to strengthen the walls which has not enough strength according to the analysis results. Buttresses were designed in the same way as the walls and with the same length. The results have shown that the walls, which failed in the first analysis, have passed the requirements after the buttress application.

Yamin and Phillips [31] studied on seismic behavior and rehabilitation alternatives on adobe and rammed earth buildings. First, they built a wall with a 2m x 2m x 0.5m size to determine the behavior of main structure, which was constructed with traditional techniques to compare new ones. As a reinforcement, wire mesh and lime mortar were used in vertical and horizontal directions to simulate confinement beans and columns. The other way to reinforce is to use wooden elements in the wall. The elements were interconnected to each other through bolts. The boundary wooden elements were installed both in the horizontal and vertical directions. In the horizontal direction elements were placed near the wall base and near the upper floor slabs. In the vertical directions boundary elements were placed near the corners. All wood used for strengthening was nailed in every 15 cm to produce a rough contact surface with the wall for compatibility. A test was carried out with applied cyclic loading. Reinforced walls had greater resistance, a higher deformation capacity and much better energy dissipation in comparison with non-reinforced walls. The out of plane overturning test had an increase of nearly 100% in the bending strength with boundary wooden elements. For seismic test, the wall with wire mesh had more ductile behavior as compared to unreinforced wall. Boundary wooden element reinforced one had also more ductile behavior and a better resistance to the earthquake input. Higher number of cycles of greater intensity were needed to produce collapse.

Ruiz and Silva [25] studied on seismic performance of rammed earth town halls reinforced with confinement wooden elements. Before implementing the reinforcement with confining wood elements, diagonal tensile strength test was carried out. Two specimens had confining wood reinforcements and two had not. According to the test results, unlike the unreinforced walls, reinforced walls kept their unity. They installed horizontal and vertical wooden reinforcements to increase the flexural strength of the walls and to keep the structure unity. Timbers were placed like lintels but unlike lintels, they were continuous in the whole wall. Reinforced and unreinforced walls were tested on the shaking table. The results have shown, the use of confining wood elements made walls work together and increased the energy dissipation capacity of the structure. Additionally, the reinforced wall had better seismic performance, while the unreinforced model collapsed due to the tensile, flexural and shear stresses. The confining woods reduced the displacement of the walls up to 69%.

CHAPTER 2

TEST PROCEDURE AND PREPARATIONS

2.1 Natural Rammed Earth

“Natural Rammed Earth” is a terminology used to describe compressed earth structure that is made only from natural clay, silt, and sand. Organic matter may lead to shrinkage, bio deterioration and may increase susceptibility to insect attack in rammed earth structures. Besides, organic matter would interfere with action of stabilizers such as cement, lime, etc. That is why organic matter content should be avoided or minimized as much as possible. The particle distribution of the rammed earth should be well arranged. Particle size distribution testing by sieving and sedimentation testing has become acceptable practice for appraisal of soil for RE. However, influence of variation in grading on physical characteristics of RE, including both strength and durability, remains unclear. [11]. One of the good method in order to increase the mechanical strength and weathering resistance of soil is minimizing the void ratio. This increases the interaction surfaces of particles. Particles should be as spherical as possible to address this problem. Particle size distribution follows the Fuller Formula:

$$P = 100(d/D)^n \qquad 1.1$$

In this equation, P equals proportion of grains of a given diameter, d equals the diameter of grains for a given value of P, D equals the largest grain diameter and n equals the grading coefficient. When the grains are entirely spherical then n is equal to 0.5. However, in earth constructions a value of n between 0.2 – 0.25 is more appropriate depending on grain shape. Gravel provides inert skeleton and together with sand enhances weathering resistance. The very primary characteristic of the clay is swelling and shrinking. Clay swells when it gets wet, and shrinks when it dries [32]. Size proportion of the soil can be classified as gravel, sand, silt, and clay [26] (Figure 2.1). The British Standard's [12] grades limits as following;

Gravel = 60 mm to 2 mm

Sand = 2 mm to 0.06 mm

Silt = 0.06 mm to 0.002 mm

Clay < 0.002 mm

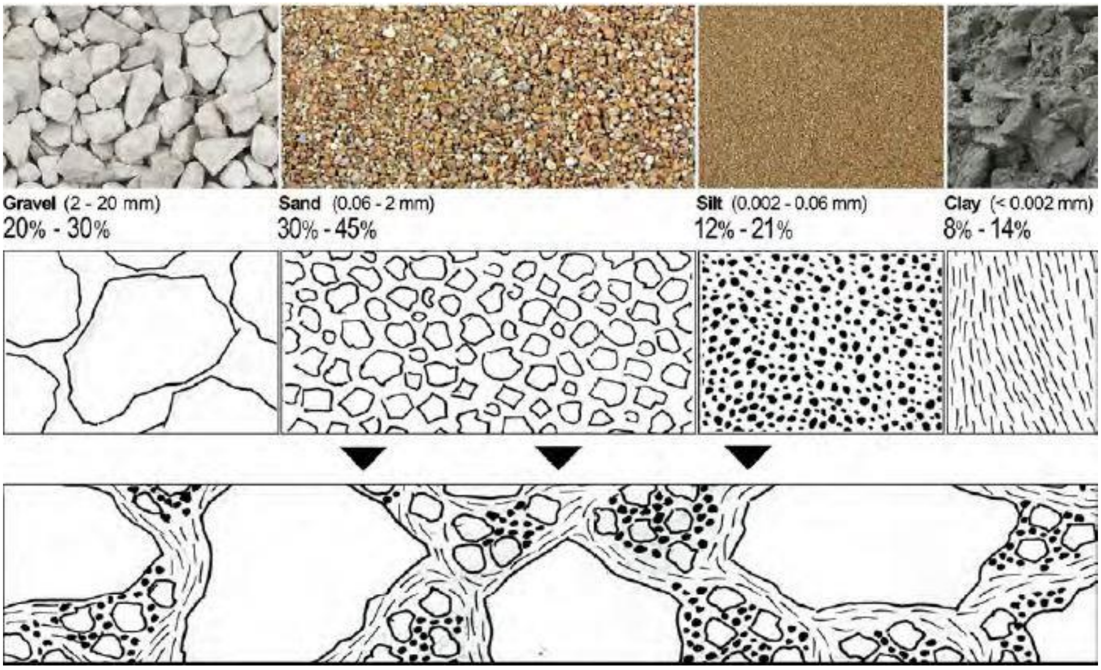


Figure 2.1 Size proportion of the soil [12]

The key to the success of RE is dependent on the mixing composition. The soil should have a high sand/gravel content. In addition to that, enough silt and clay act as a binder [11]. The compressive test results on cylindrical samples of RE have shown, when gravel size increases, compressive strength reduces [22]. Proposals tend to converge towards a 30% - 70% balance between clay/silt and sand proportions [3]. Nevertheless, there are various lower limits (Figure 2.2) and upper limits (Figure 2.3) for compositions of RE around the world.

According to the upper and lower limits on the charts above, the minimum percentage of clay and silt should be between 20% - 25% while the maximum between 30% - 35%. Likewise, the minimum percentage of sand should be between 50% - 55% while the maximum is between 70% - 75% [12]. In this study, an average value of upper and lower limits was used. Percentage of sand was chosen as an average value of 50% and 75%. It was rounded to 60%. Similarly, the percentage of clay and silt was chosen 40%.

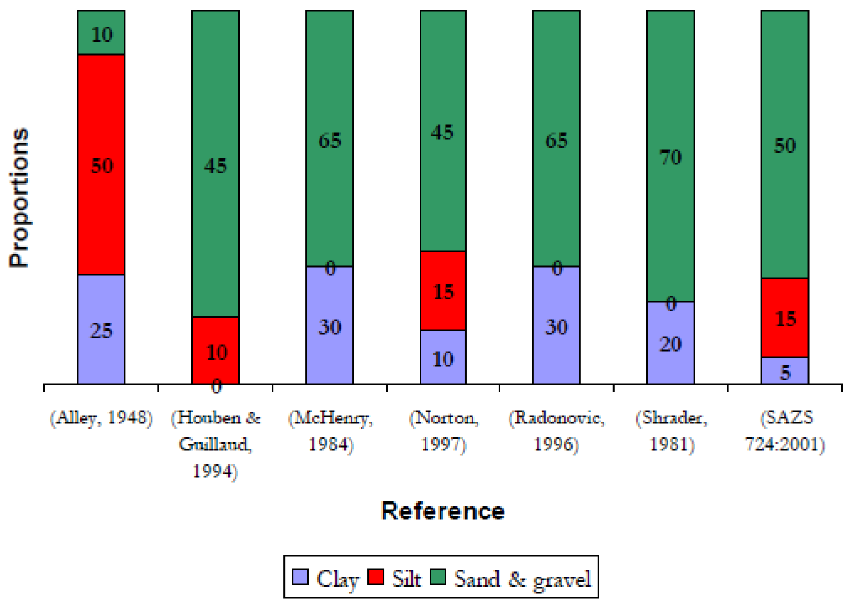


Figure 2.2 Lower limits of the mixing composition [12]

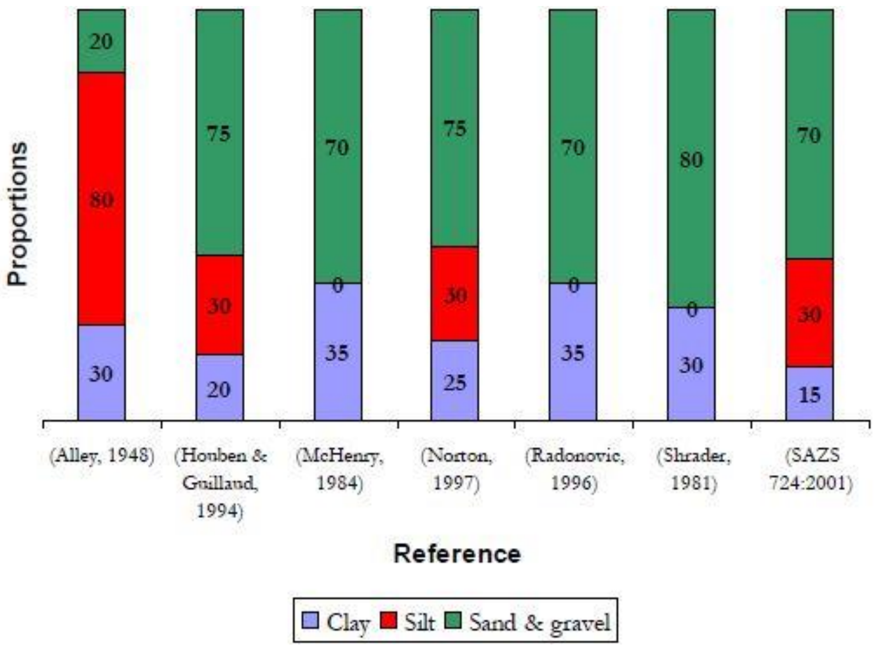


Figure 2.3 Upper limits of the mixing composition [12]

2.2 Binder

A widely used stabilizer is cement because of its strong compressive strength performance. However, cement seems to reduce the sustainability of RE and increases both its cost and environmental impact. [7]. One of the aims of this study is to build a REW without using any cement addition. One of the best stabilizer for soil is lime even when it is slaked or unslaked. Lime reacts with the clay to form a binder [30]. Unslaked lime is harmful for skin. That is why, in this study slaked lime was used. An experimental study to identify optimum lime content was followed to decide the percentage of the lime content of mixture. Even though in this experiment a mixture of kaolin clay powder, silica flour, sand and gravel was used, it proved to be very useful a good chance for estimating the lime content for mixture. UCS test results have shown that compressive strength increases with increasing lime content up to an optimum value around 4% (Figure 2.4) [7]. With reference to this information and chart, optimum lime content was chosen as 5%.

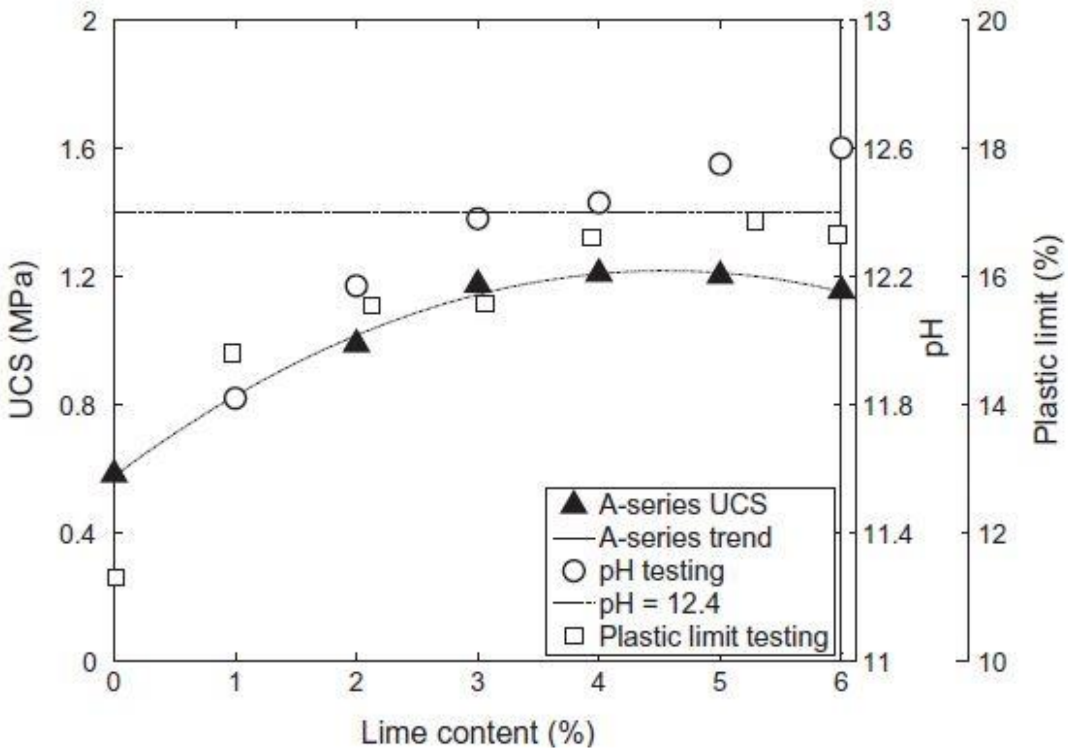


Figure 2.4 Optimum lime content [7]

Fly ash represents a critical supplementary cementitious material as a replacement of ordinary Portland cement to reduce carbon dioxide footprint of concretes [21]. In addition to lime, fly ash was also used as a stabilizer. The main constituents of fly ashes are SiO_2, Al_2O_3, Fe_2O_3 and CaO and their amounts change depending on the type of fly ash [28]. Free lime content and reactive silicate ratio should be higher to support carbonation in RE. According to the Turkish Classification of Fly Ashes [28], type C fly ash was decided to be used because of its suitability (Table 2.1). Class C fly ash with the highest ratio of free lime and reactive silicate is produced in Soma Thermal Power Plant [28]. Thus, fly ash was provided from there.

Table 2.1 SOMA fly ash chemical properties [28]

Oxide (%)	Fly Ash	TS EN 450 (V)	TS EN 450 (W)	TR EN 197-1	TS 639	ASTM C 618 (F)	ASTM C 618 (C)
SiO_2	44.04						
Al_2O_3	22.07						
Fe_2O_3	4.37						
S+A+F	70.48				> 70.00	> 70.00	> 50.00
CaO	20.95						
MgO	1.66				< 5.00		
SO_3	2.55	< 3.00			< 5.00	< 5.00	< 5.00
K_2O	1.22						
Na_2O	0.34						
KK	2.09	< 5.00	< 5.00	< 5.00	< 10.00	< 6.00	< 6.00
Cl	0.009	< 0.10					
Free CaO	7.66	< 1.00					
Reac. SiO_2	31.5	> 25.00	> 25.00	> 25.00			
Reac. Cao	17.71		< 10.00	> 10.00			

In the experimental research [24], different percentages of base soil with the different percentages of additions were compared. The highest strength results after 7 days curing without cement addition was 80% soil + 20% fly ash and 90% soil + %10% fly ash among only soil + fly ash mixtures (Table 2.2) [24]. There were also different strengths of soil as mentioned in literature review part. Compaction level, impact force, clay type, sand & clay percentage, moisture content, and curing conditions might affect compressive strength. In this study, lime and fly ash was used together. The lime and fly ash were used together which made a cement almost as good as Portland cement. The mixing proportion of lime and fly ash are about 2 to 4 times as more fly ash than lime [30]. In this thesis with this information, 15% fly ash was used to 5% lime. Additionally, this was also proved by the different mixing proportions of samples on the unconfined compressive strength test.

Table 2.2 The highest strengths of mixtures after 7 days of curing [24]

Mix Design		7-day strength (MPa)
1	100% soil	0.35
4	70% soil and 30% bottom ash	0.34
8	80% soil and 20% fly ash	0.73
11	80% Base 1 (70% soil and 30% bottom ash) and 20% fly ash	0.84
13	80% Base 1, 10% fly ash, and 10% cement	5.43
14	90% Base 1 and 10% fly ash	0.68
16	90% Base 1, 5% fly ash, and 5% cement	3.18

2.3 Mix properties

Although previous studies have shown and suggested the most appropriate mixing proportions for clay, lime and fly ash admixtures, there is no information regarding to mixing them all together. Clay and sand mixture was named as soil. Soil mixture percentage and addition mixture percentage was calculated separately. That is why, 4 test mixings were prepared and subjected to an unconfined compressive strength test (Figure 2.5 (a) 100% soil, (b) 90% soil + 10% lime, (c) 80% soil + 15% fly ash + 5% lime, (d) 80% soil + 10% fly ash + 5% lime + 5% cement). These are;

1) 100% Soil (40% Clay + 60% Sand)
2) 90% Soil (40% Clay + 60% Sand) + 10% Lime
3) 80% Soil (40% Clay + 60% Sand) + 15% Fly Ash + 5% Lime
4) 80% Soil (40% Clay + 60% Sand) + 10% Fly Ash + 5% Lime + 5% Cement

(a) (b)

(c) (d)

Figure 2.5 Mixture preparations a) 1st mixture, b) 2nd mixture, c) 3rd mixture, and d) 4th mixture

Density is a pretty important parameter for REWs. Even a small difference in density can produce a significant difference in strength. For this reason, having a value as high as possible for dry density is considered important since density is related to strength and durability [5].

In addition to dry density, optimum moisture content also plays a key role for wall strength. By achieving OMC, the compaction can be applied more efficiently. In general, when the moisture content is less then OMC, it is more difficult to compact the soil. In the opposite case, soil is not as dense under a given effort because the water interferes with the close packing of the soil particles [32].

After soil and admixture proportions were decided, a standard proctor test was performed to define the optimum moisture content for compaction. After the empty mold was weighted, it was filled by the mixing and rammed by using a hammer in three stage. Same soil layer thickness was used for compaction and similar compaction impact energy was applied; therefore, similar compaction ratio was obtained for all of the samples. As a practical rule of thumb, all layers were compacted to half of their initial thickness reducing their volume by half. Firstly, the mixture was added to the mold until it reached the half of its volume. Later it was compacted until it reached a quarter of its volume. This procedure was repeated until the mold was filled its completely.

Samples were prepared by using 50 mm diameter x 100 mm height forms and compacted in three stage similar to the proctor test. Ordinary oils for frames when casting concrete samples may penetrate into the specimens. Therefore, Vaseline was used inside of the molds Because of the compaction effect and fraction area, it was not possible to take samples out from the molds manually. A hydraulic machine was used to remove the samples from the molds. Samples were cured for 7, 28, and 90 days (Figure 2.6).

Figure 2.6 Samples after curing

2.4 Reinforcement

Rammed Earth Walls (REW) can be categorized as Natural REW (NREW), where only clay, silt, and sand are used for compaction while Stabilized REW (SREW) has one or more of admixtures such as lime, cement, fly ash, other puzzolanic materials, animal blood, egg, straw, hair, monofilament-synthetic-steel fibers, etc. Reinforced REW (RREW) is composed of NREW or SREW that has internal or external rebars or timber elements.

SREW stabilized walls using fly ash and lime have been used in this study; and then strengthened using steel external and internal rebars. A final test was conducted on a wall that had improved strength using confinement hoops at the extreme compression zone.

Reinforcement elements were used for increasing the performance of REW during an earthquake. During an earthquake, REW tends to tumble after the separation caused by cracks. Vertical steel reinforcement bars were used to avoid rocking-overturning and strengthening the corner of the wall (Figure 2.7). However, REW are still weak on the diagonal direction and a diagonal crack opening would lead to a shear failure although vertical bars exist. In addition to the vertical steel reinforcement bars, diagonal steel reinforcement bars were also used to strengthen walls in the diagonal direction (Figure 2.8). Later on, compressive stress concentrations were expected and observed at the bottom wall corners. Confinement hoop elements were used to reinforce these areas creating confinement zones (Figure 2.9). Lateral reinforcement layers were added inside the third wall specimen at two layers (1/3rd and 2/3rd) in addition to implanted horizontal hoops at corners and exterior vertical and diagonal reinforcement to improve the diagonal cracking strength and ductility of the wall.

Figure 2.7 REW with vertical reinforcement

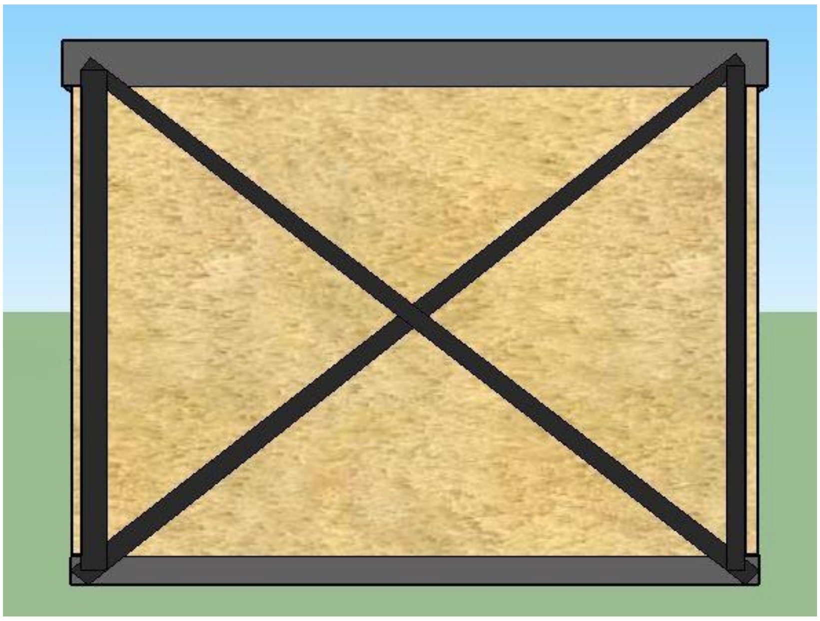


Figure 2.8 REW with vertical + diagonal reinforcement

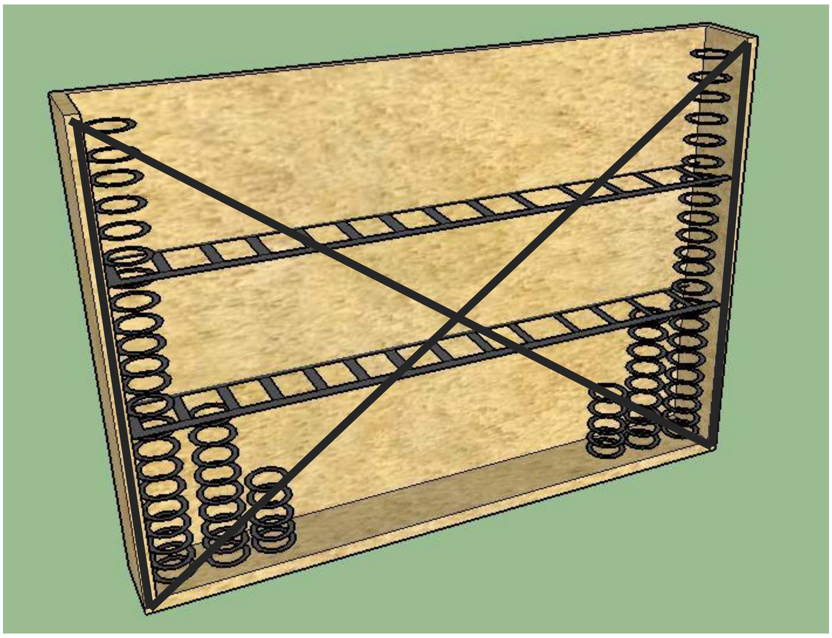


Figure 2.9 REW with confinement

2.4.1 Confinement

Confining wood elements as a reinforcement, makes the walls work together, provides better seismic performance, and reduces the displacement during the earthquake [25], [31]. One of the very common damage types of REW during earthquake occurs at the corners [2]. Considering corner failure mechanism and observation of the first wall test, confinement elements were used at wall bottom corners. These confinement elements were designed as steel hoop members to form a column-like section inside the wall. Wooden confinement reinforcement was considered as an alternative to steel hoop column. Two specimens of hoop and timber strengthened 110mmx110mmx250mm RE column were prepared to see their behavior under uniaxial loading and decide which one of the wooden or steel reinforcement to be used as a confinement element is better.

2.4.1.1 Wooden confinement column

Wood is not an isotropic material. It is strongest when loaded to induce stress parallel to grain, either in tension or compression [1]. That is why, timber was prepared to be loaded parallel to its grains. A square shaped confinement cage element was prepared. Tension test was applied to see the material strength. These are, i) only nails, ii) nails + bench clamp, iii) only screw, iv) screws in 3 directions, v) screws in 3 directions + bench clamp, vi) glue + screw, and vii) nail + glue + bench clamp. Wooden confinement elements were held from one direction and pulled using the crane. Results have shown that the strongest is screws + glue system. Timber members of 20 x 20 x 110 mm pieces were prepared and holes were opened on their both sides. Timber members were placed one on top of another in alternating directions and connected to each other by threaded rods and nuts (Figure 2.10). Glue was used on the touching areas of wooden pieces to increase the friction area and connection strength. A formwork for column was prepared and wooden confinement element was placed inside it (Figure 2.11). The formwork was filled with the REW design mixture and rammed (Figure 2.12). A smaller hammer was used to ram because the area inside of confinement elements was smaller and not easy to ram. In addition to that, mixture should reach at the sides of the confinement element holes and nicely compacted. After ramming, 110 x 110 x 250 mm column was ready for testing (Figure 2.13).

Figure 2.10 Wooden confinement column

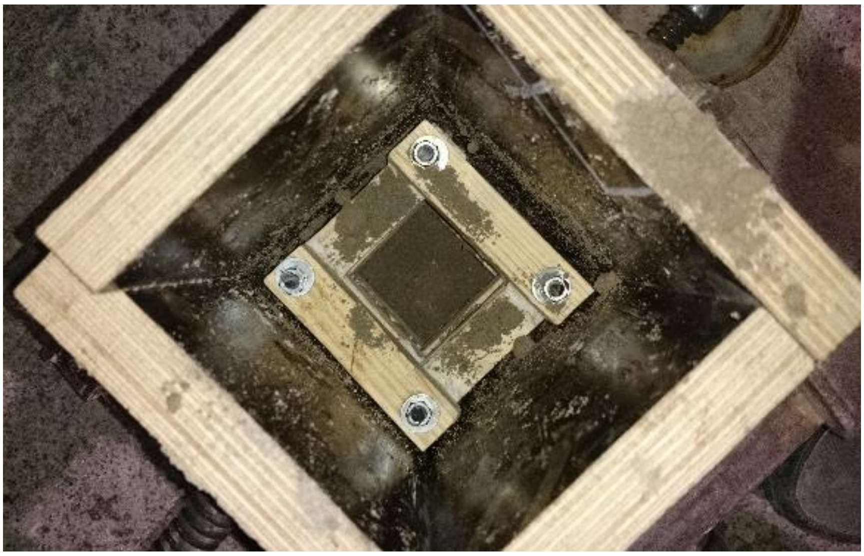


Figure 2.11 Formwork for wooden reinforced column

Figure 2.12 Ramming process (top of the specimen and formwork)

Figure 2.13 Wooden confinement column final view

2.4.1.2 Steel confinement column

Similar to the wooden confinement elements formwork, another formwork was prepared. Ø4 mm steel bars were used to prepare 100 mm diameter confinement steel hoops. First, the steel bars were bended to give it a hoop shape. Then, they were welded at the ends. Steel hoops were placed in the column formwork in every 20 mm to increase confinement effect (Figure 2.14). Because there were no ramming space limiting components like the of wooden confinement elements, it was easier to ram. After ramming, 110 x 110 x 250 mm column was prepared (Figure 2.15).

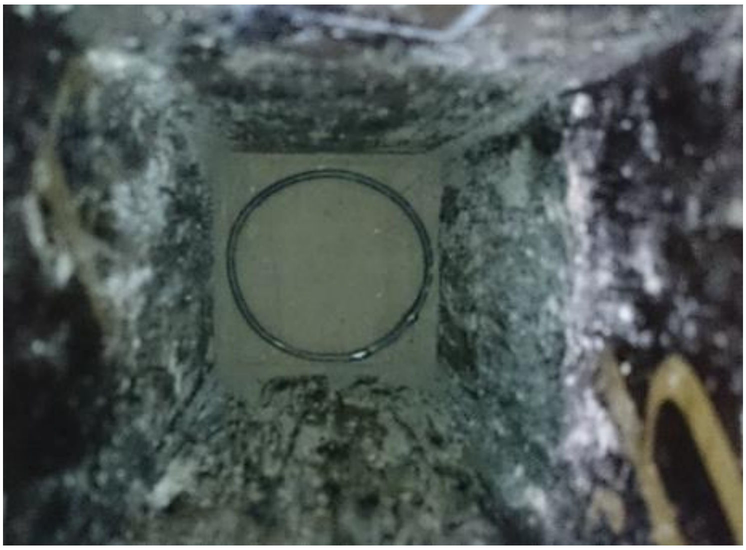


Figure 2.14 Ramming process

Figure 2.15 Steel hoop confined RE column

2.5 Stabilized unreinforced REW

Stabilized REW (SREW) has one or more of admixtures such as lime, cement, fly ash, other puzzolanic materials, animal blood, egg, straw, hair, monofilament-synthetic-steel fibers, etc. Reinforced REW (RREW) is composed of NREW or SREW that has internal or external rebars or timber elements.

2.5.1 Mixing

A cement mixer was used in this study to evenly mix RE materials, although hand mixing is also possible with more workmanship and labor. First, the dry components were mixed well before water was added. Mixing of all materials should be continued until the optimum amount of water should be added and soil should be mixed well [26]. When the samples were prepared for the Unconfined Compressive Strength (UCS) test, smaller amount of mixing material can be manually mixed in a tray. However, the amount of REW mixture was plenty for a larger sample or for a building, therefore a cement mixer was used. All the dry materials were added one by one and mixed thoroughly. After lime and clay addition, lime and fly ash were added. These materials have a fine texture. During the mixing, these fine elements may get out of the mixer as dust and may disturb the workers. Additionally, the loss of material may lead to a change in mixing proportions. Therefore, while the mixture is mixing, the opening of the mixture mixer was closed by using a plastic sheet.

After the dry components mixed well, water was added carefully and slowly. When the water is added, it may cause flocculation. Dry mixture has a tendency to stick together when water is added. That is why, water should be added as slow as possible. Otherwise, if the water is added fast, it will cause balls in the mixture (Figure 2.16). However, they can be crushed by using a hammer or even by hand.

The disadvantage of this is that it may cause extra work and will not allow all parts to have the same approximate amount of water.

Figure 2.16 Floc balls in the mixture

The optimum moisture content, which was decided by a proctor compaction test, was slowly reached in the mixture. In addition to the proctor compaction test results, a ball dropping test was also suggested as a practical alternative to approximately find the optimum moisture content. According to this technique, approximately 40 mm diameter of moist soil balls should be compacted by hand. Then, the soil ball should be dropped onto a hard flat surface from a height of about 1.5 m or from shoulder height. When the ball breaks into many pieces, this means the soil is too dry. When the ball remains in one piece or shows small cracks but is still together, this means the soil is too wet. When the soil breaks into only a few pieces, this means the soil is very close to its optimum moisture content (Figure 2.17). [12], [16], [30].

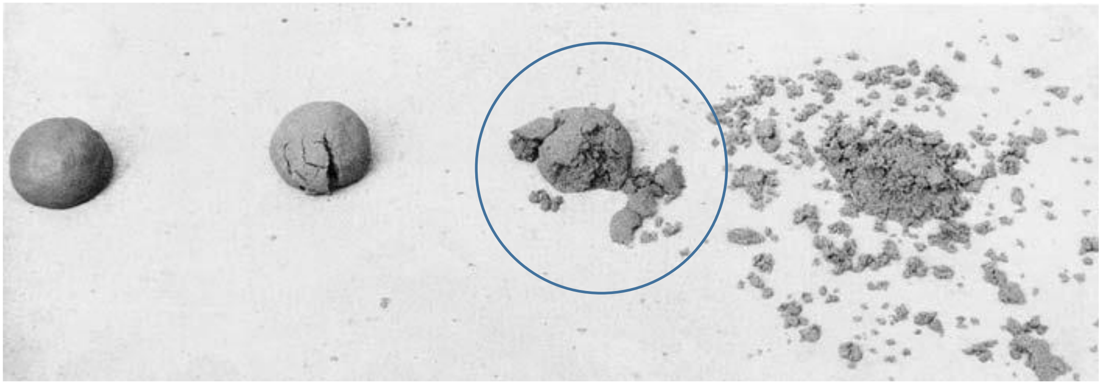


Figure 2.17 Drop test [16]

First, water was added into the mixture by using a plastic bottle. However, splashing the water inside to the mixer caused flocculation. Therefore, a few holes were opened to the cap of the plastic bottle and water was spilled into the mixture slowly. This technique remarkably reduced flocculation (Figure 2.18).

Figure 2.18 Well mixed mixture

2.5.2 Formwork

Timber or steel formwork is necessary to hold the wall mixing material in an organized shape during compaction REW construction. Different than the formwork in concrete casting practice, the formwork in REW construction can be partial since compacted material can be used in smaller portions of the wall and the compacted material is capable of carrying relatively large forces just after compaction is completed. Therefore, it is a common practice to use a total depth of each formwork lift varying between 600 mm and 900mm [34] sliding formwork (Figure 2.19) although single piece formwork for the whole wall is also used at a higher cost but for a more even look (Figure 2.20).

Figure 2.19 An example of sliding formwork (URL – 7)

Figure 2.20 Bigger formwork system (URL – 8)

It was experienced in the lab experiments that the formwork has a tendency to swell because of the lateral pressure of compacted soil. It is a common practice to use snap ties in formwork to prevent swelling or opening the formwork (Figure 2.21). In addition to the snap ties, horizontal beams may also be used. Alternatively, these beams may be clamped from above to prevent holes and the cost of snap ties in REW construction system. In traditional RE structures around the world, generally two timber shutters made out of softwood planks with 20 – 30 mm thick formworks are used [19]. Basic elements of modern formwork comprise sheeting materials (steel, aluminium, timber etc.), stiffening elements (soldiers, walers etc.), ties and bolts (Figure 2.22). Using the traditional formworks may cause different and distinctive finish to a wall [12].

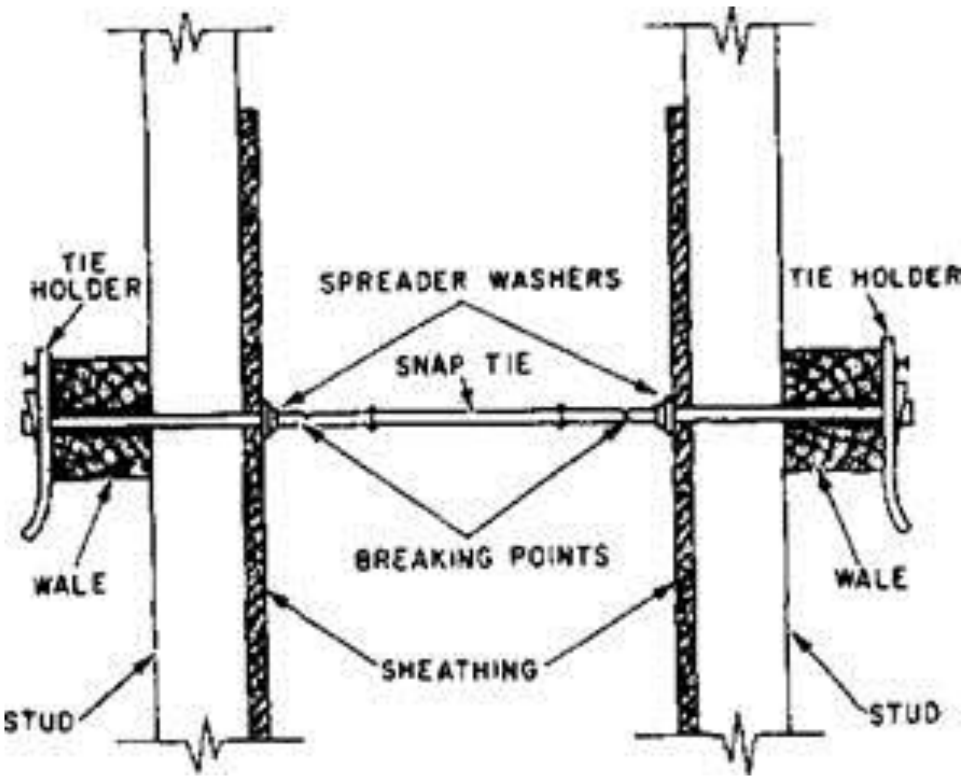


Figure 2.21 Snap tie connection (URL – 9)

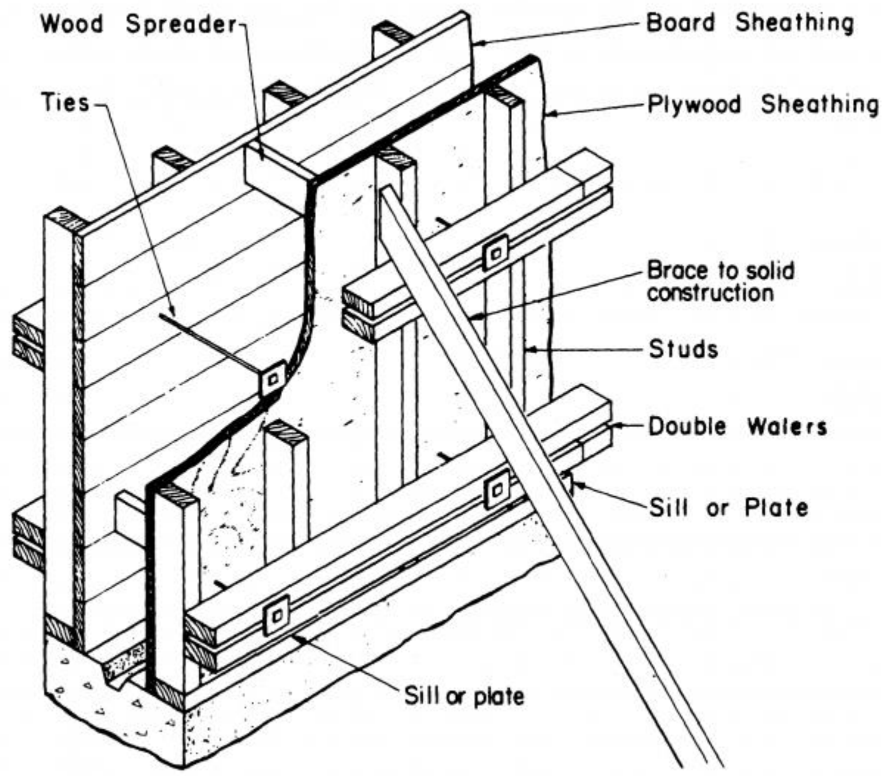


Figure 2.22 Example of sheathing and stiffening elements (URL – 10)

A greater height of the walls means more difficulties to set and align the forms. That is why, two main types have been developed. These are a small unit formwork and an integral formwork. For small walls, a horizontally sliding crawler formwork has been developed by CSIRO Building Construction and Engineering [15]. This technique lets formworks move in a horizontal direction without being dismantled. The other type for small walls is vertical sliding forms, which has been developed by Building Research Institute [16]. Shutters slide within a steel or timber frame. This type of formwork can significantly accelerate the construction process [9].

For large sections of walls as high as the entire height of the building Integral Formwork has been developed. There are three types of this system, which are i)

Australian Forming System (Figure 2.23), ii) California Forming System (Figure 2.24) , and iii) Continuous-wall System (Figure 2.25).

Figure 2.23 Australian forming system [12]

Figure 2.24 California forming system (URL – 11)

Figure 2.25 Continuous-wall system (URL – 12)

In this study, plywood was used to make formworks to decrease the labor and increase the accessibility. The inside of the formworks was covered by a plastic sheet and to both sides of the formwork vaseline was applied to improve the slipperiness. It resulted in successful slippery / non-sticking surfaces, led formworks open and separate from the walls easily. However, compacted soil wrinkled the plastic sheet at some points because of the compaction force in the direction of the compaction. This caused approximately 2mm wide and 5 mm deep dent lines that look like cracks on the faces of REW. That is why, formworks without plastic sheet were used for the following tests. Scratches and holes inside faces of formworks reduces the slipperiness. Therefore, it was considered to use a clean surface formwork especially using it without plastic sheet. Vaseline was applied to the all inner surface of formworks (Figure 2.26). Additional plywood pieces were placed at the narrow side of the walls, which were screwed and secured to support the major surface formworks. Nails and bench clamps were used on the transverse directions to support formwork because, nails alone are not strong enough to hold the sides together (Figure 2.27). Square cold formed steel frames were used to avoid the enlargement at the middle of the formworks. Two of the square steel frames were placed to the 250 mm and 450 mm high from the bottom horizontally (Figure 2.28). They were held together by using bench clamps. The formwork was designed to cast a REW of 0.65m high, 0.9m long, and 0.11m thick.

Figure 2.26 Vaseline on the formwork

Figure 2.27 Preparation of formwork

Figure 2.28 Formworks with supports

2.5.3 Ramming

Some of the most important factors are the rammers head's material, weight, shape and area for manual compaction [14]. Suggestions of various authors are summarized [16]. In this study, a manual hammer close to Norton's description [19] was used. A moist mixture is poured into a formwork in layers of 150 mm thickness, and then compacted by ramming with RE techniques each layer should be 80 – 100 mm thick after compaction [26]. The compaction force is not defined in any codes and varies depending on the user, the compressed earth thickness, the soil mixture type, and the mixture moisture content. If the formworks have the same length as the wall, it will be more difficult to ram the top layer. It is recommended to use a collar to avoid the problem of ramming the top layer [6]. In this study, a 150 mm thick mixture was poured into the formworks and it was compacted until it reached half of its length. Formworks were prepared 150 mm longer than the walls to avoid the less compaction problem of the top layer (Figure 2.30). Similarly, ramming the bottom layer is difficult as well since the equipment to impact the lower layers need to be long and any snap ties might block the ramming process (Figure 2.29, step 2).

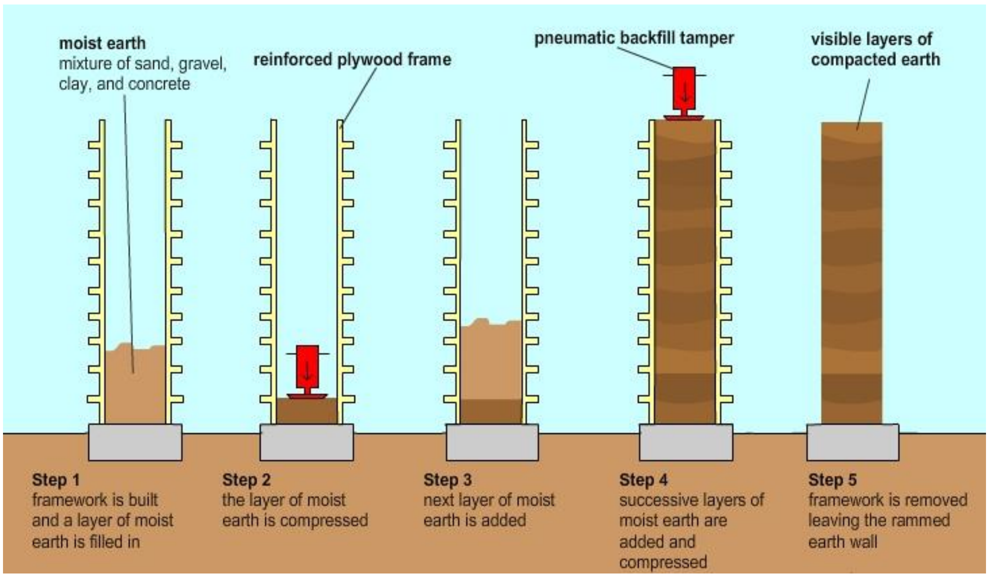


Figure 2.29 Ramming process (URL – 13)

Figure 2.30 Top layer of ramming

2.5.4 Removal of forms

Removal of forms is an important part of the work. Inattentively removals may ruin all work. In this study, the constructed wall represents 4:1 scale of the real wall. That is why, an entire formwork was preferred hence it is recommended for larger walls. However, this type of formwork caused removal problems. Small wall formworks can be removed on each layer. Opposing to sliding formwork types, the formwork should be removed after the whole wall was constructed. Removing formworks in out-of-plane direction may damage the wall. Some parts may stick to the formwork and may come off together with the formwork. That is why, formworks should be slid slowly in plane direction.

After the ramming process, the bench clamps were removed first. After bench the clamps and the horizontal steel support elements were removed, it was observed that the formworks were already a little separated from the wall because of the pressure (Figure 2.31). Nails were removed by using an adze to slide formworks nicely. Afterwards, the front formwork was removed first. Thereafter the side formworks were removed which were used to hold the wall in horizontal direction. Finally, the back formwork was removed (Figure 2.32).

Figure 2.31 Removal of forms

Figure 2.32 REW after formworks were removed

2.5.5 Test setup

A U channel profile was used to build the wall on. Small steel pieces were welded inside to the profile to make it work as a serration (Figure 2.33). The U profile was fixed to the ground by using stud bolts. A square steel profile was placed at the top of the wall for a horizontal area loading on the top surface. Also steel bar pieces were welded as a serration.

Figure 2.33 Serrations of the bottom profile

A cement mortar was prepared which was applied between the wall and the square profile in order to make these two work together. Equivalent to the roof load, two weights of 75 kg were added.

They were placed at the top of the beam and centered. 1 load cell and 4 LVDT's were installed. The channel 1 LVDT was for measuring horizontal displacement of the wall at the top. The channel 2 LVDT was for measuring horizontal displacement of the wall at the bottom. The channel 3 LVDT was for measuring vertical displacement of the wall at the left corner. The channel 4 LVDT was for measuring vertical displacement of the wall at the right cornet. And The channel 5 load cell was for measuring the force (Figure 2.34). The channel 2 LVDT was placed to measure the horizontal sliding of the system. That is why it was subtracted from channel 1 to have the real displacement at the top.

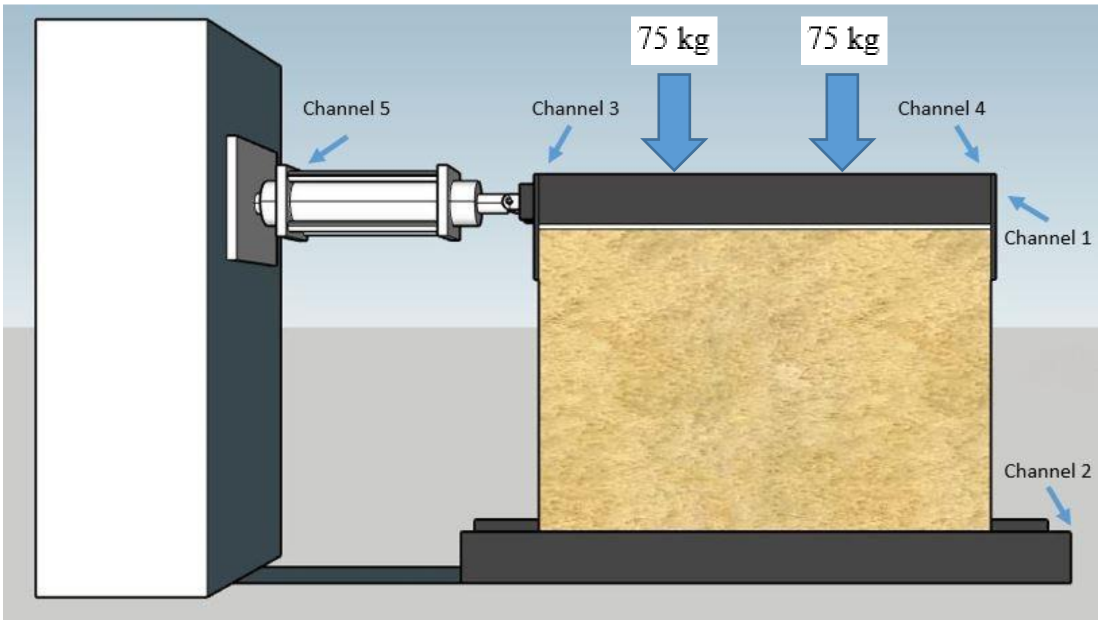


Figure 2.34 Instrumentation installation

2.6 Stabilized and outside (vertical) steel RREW

After the stabilized unreinforced REW was tested, the test continued with reinforcement additions. Ø12 steel bars were welded vertically to both the end sides of the wall (Figure 2.35). The reason for using vertical reinforcements at these points is to prevent the wall from tumbling down after it cracked and separated from the ground. In addition to that, vertical reinforcement strengthens the bottom corners where the most common RE damages occur during an earthquake. Calculations have shown that, vertical reinforcements increase strength 15 times.

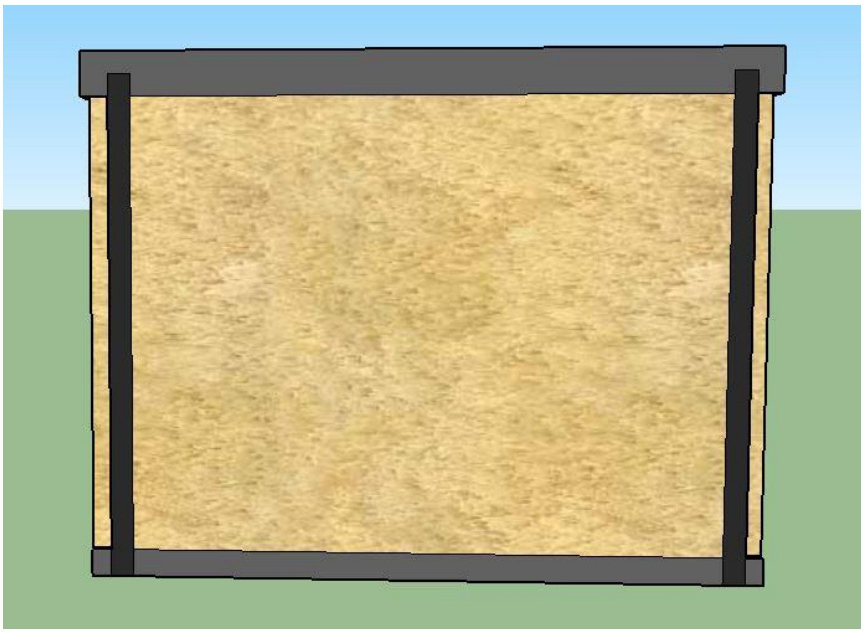


Figure 2.35 REW with vertical reinforcement

2.7 Stabilized and vertical (outside) + diagonal (outside) steel RREW

After the stabilized and only vertical reinforced REW was tested, the test continued with additional reinforcements. Ø14 steel bars were welded to both sides of the wall diagonally (Figure 2.36). The reason of using diagonal reinforcements at these points is to prevent REW from another common damage type during an earthquake. These reinforcements support the wall diagonally and prevent it from breaking in this direction. Calculations have shown that, diagonal reinforcements together with vertical reinforcements increase strength 45 times.

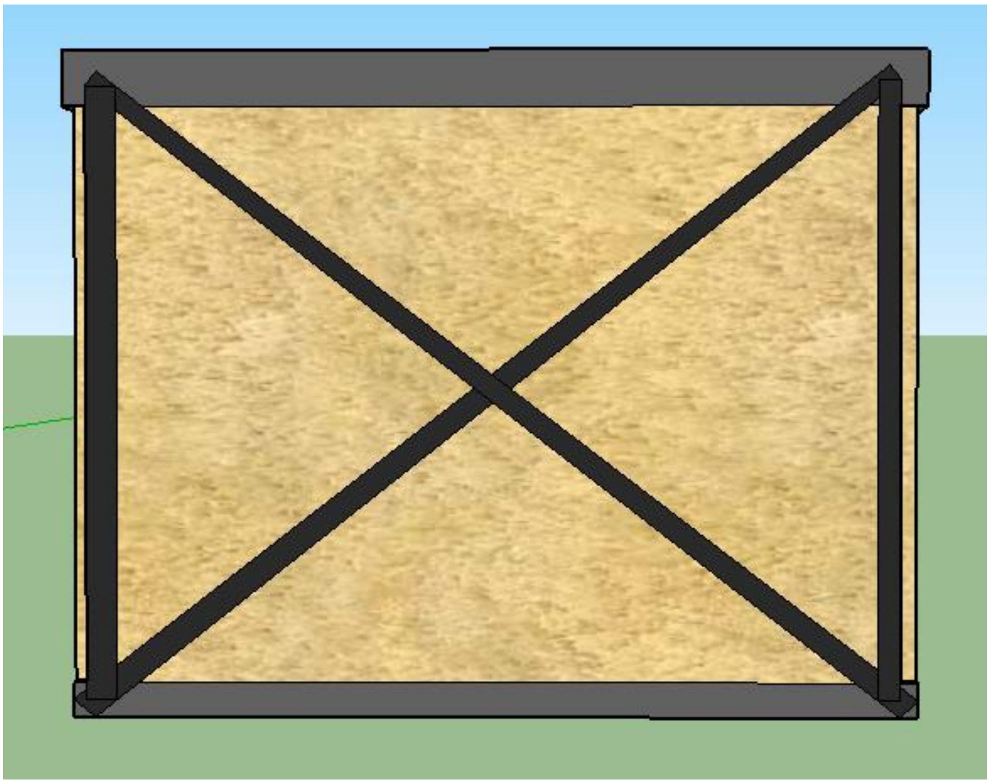


Figure 2.36 REW with vertical + diagonal reinforcement

2.8 Stabilized and vertical (inside) + diagonal (outside) steel RREW

For this test, almost the same procedure, that was used for the previous wall, was followed. Moreover, vertical reinforcement elements were placed inside the wall. 4 Ø12 steel bars were welded to the U channel profile (Figure 2.37). At each side 2 reinforcement bars were placed. Then, the same ramming process was followed same as the previous wall (Figure 2.38). However, this time it was harder to compact nearby the bars. That is why, a smaller hammer (Figure 2.39) was used to compact this area. Still it was not as good as the other areas. During the ramming, because of the compaction pressure, bars started to change position. They moved to the side of the wall and also got close to each other. Bars can be pulled to reorient then but this may cause some cracks or gap between the bars and the RE. The better solution for this is using an apparatus for bars to keep them in their original position.

Figure 2.37 Inside vertical reinforcements

Figure 2.38 Ramming

Figure 2.39 Second hammer

After fastening up the reinforcement bars, the ramming process was continued. When the ramming was done, the formworks were removed carefully. However, after removing the formworks, there were cracks and there was loss at the sides of the REW, especially at the behind side of the vertical reinforcement bars (Figure 2.40). Even though a small hammer was used to compact this area, it was not good enough. Ø14 steel bars were also prepared to use after the first failure of the wall as a diagonal reinforcement.

Figure 2.40 Cracks

2.9 Stabilized and confined + vertical + diagonal (outside) steel RREW

The ramming process was conducted similarly to the previous walls. Furthermore, steel confinement hoops were placed inside the sides of the REW. The reason for using these confinement elements were to reinforce the side bottoms of the REW and build an area which can work as a column inside the REW. The design has been made by using concrete column confinement calculations of [29]. Confinement steel hoops were placed inside the wall in every 20 mm height (Figure 2.41). After pouring the mixture up to 20 mm height of, another hoop was placed and this continued since reaching to ordinary ramming layer height. After reaching this height, all layers were compacted at one time and then the same procedure was followed for the other layers (Figure 2.42). 3 confinement steel hoops were placed collateral for 3 layers. After that, 2 confinement steel hoops were placed collateral for 3 layers too.

One confinement steel hoop was placed for the rest, reaching the top of the wall. Reinforcement elements were placed more frequent at the bottom corners to increase confinement effect at these areas.

Lateral reinforcement binders were prepared (Figure 2.43). These were placed inside the REW at 200 mm and 450 mm height from the bottom. The reason for using lateral reinforcement binders was to reinforce wall against diagonal stresses and avoid diagonally brittle cracks. Ø12 and Ø14 steel bars were prepared to use after the first failure as a reinforcement they were placed respectively vertical and diagonal.

Figure 2.41 Hoops’ positions

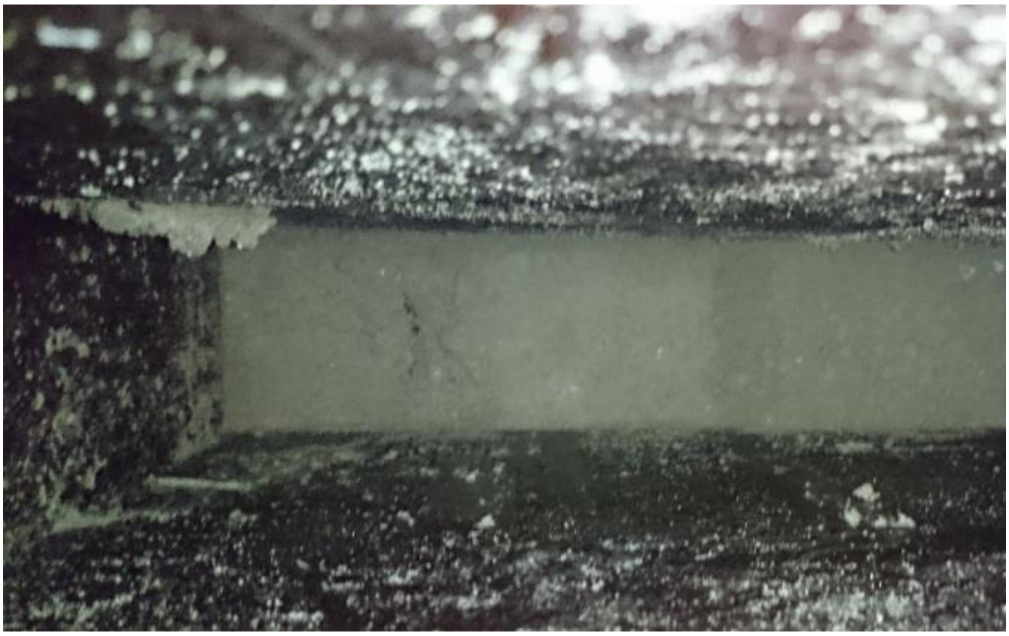


Figure 2.42 Ramming

Figure 2.43 Horizontal reinforcement with tie bars

Although the formworks are as carefully removed as possible, there were damages at the edges of wall. These might occur because of a lack of compaction at these areas. This problem increased for RREWs. There were cracks and damages at the side faces of the wall because of a lack of compaction force behind the vertically placed reinforcement elements. Because there were no vertically placed reinforcement elements in the third wall, the side faces were okay. However, similar to the others, this wall also had shrinkage cracks.

CHAPTER 3

TEST RESULTS

3.1 Results

3.1.1 Sieve analysis

Sieve analysis test was carried out by using by using sieves with mesh numbers as following; #4, #10, #30, #50, #70, #100 and #200. These sieves' openings in mm are; 4.75, 2, 0.595, 0.297, 0.21, 0.149 and 0.074 respectively. A hydrometer analysis was carried out for particles finer than 0.074 mm and results were added to the graph. Results are shown in Figure 3.1.

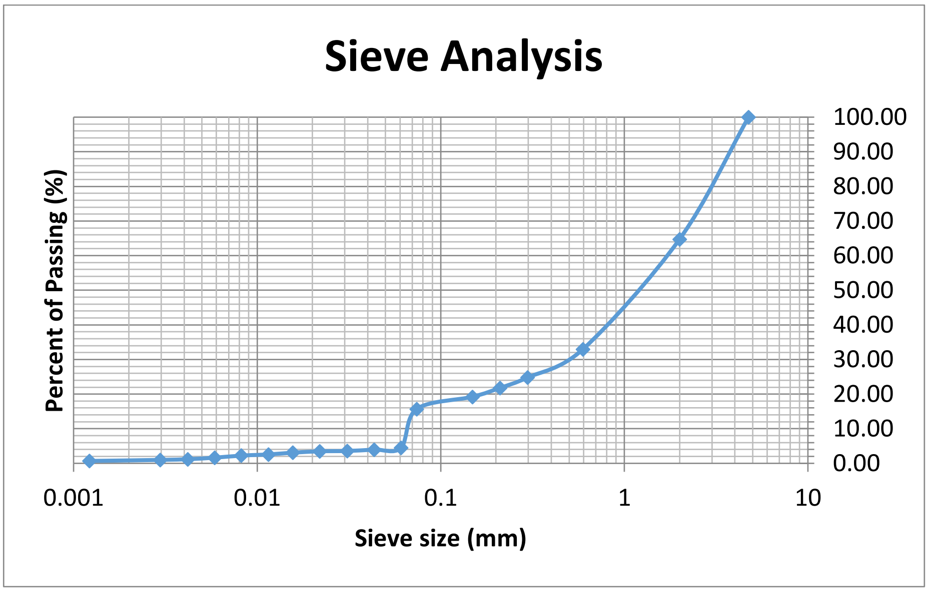


Figure 3.1 Sieve analysis results

3.1.2 Shrinkage limit test and Atterberg limit test

A shrinkage limit test and an Atterberg limit test were carried out due to shrinkage cracks on the REWs after casting. The Atterberg limit test was carried out and moisture content was determined after 25 drops of the hammer. The linear shrinkage test was carried out on the third mixture which has 80% soil + 15% fly ash + 5% lime (Figure 3.2). The initial length of the specimen and the length of the dried specimen were measured. According to the test results, the liquid limit was around 30% and the shrinkage limit was around 3.5%.

Figure 3.2 Linear shrinkage test sample after drying

3.1.3 Standard proctor test

A standard proctor test was performed on all mixtures and the optimum moisture contents were found. Although the mixtures were different from each other, there was no significant difference between the OMC values. Results are shown respectively in the following graphs for the 1st mixture with 100% soil (Figure 3.3), the 2nd mixture with 90% soil + 10% lime (Figure 3.4), the 3rd mixture with 80% soil + 15% fly ash + 5% lime (Figure 3.5), and the 4th mixture 80% soil + 10% fly ash + 5% lime + 5% cement (Figure 3.6).

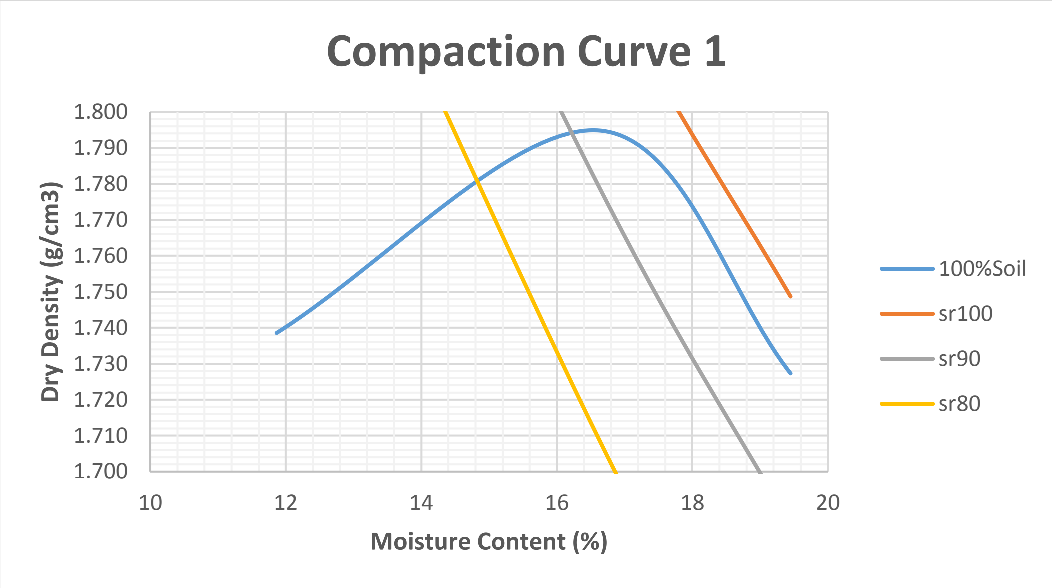


Figure 3.3 Compaction curve of 1st mixture

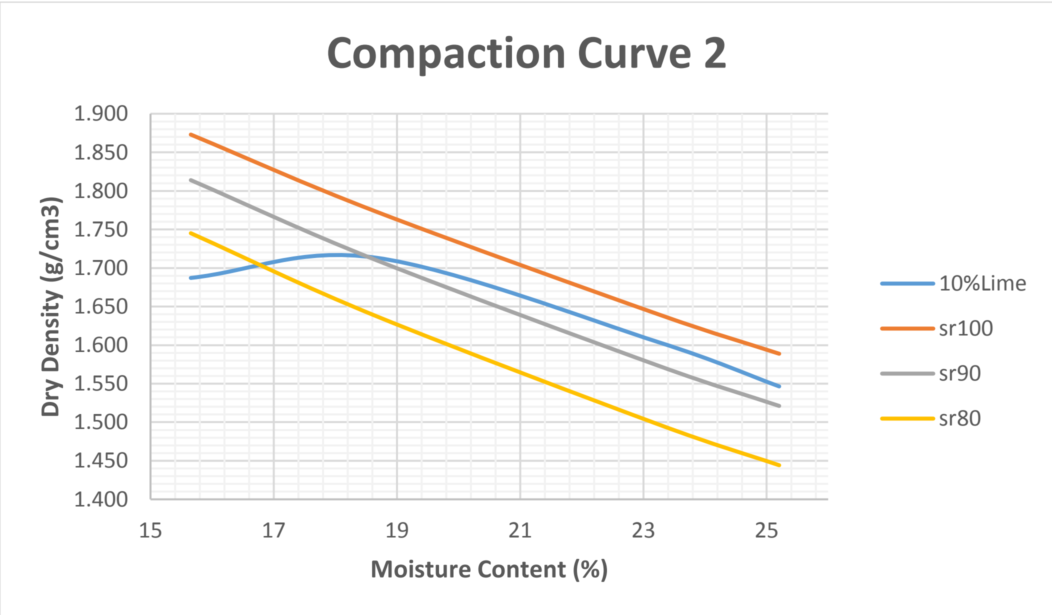


Figure 3.4 Compaction curve of 2nd mixture

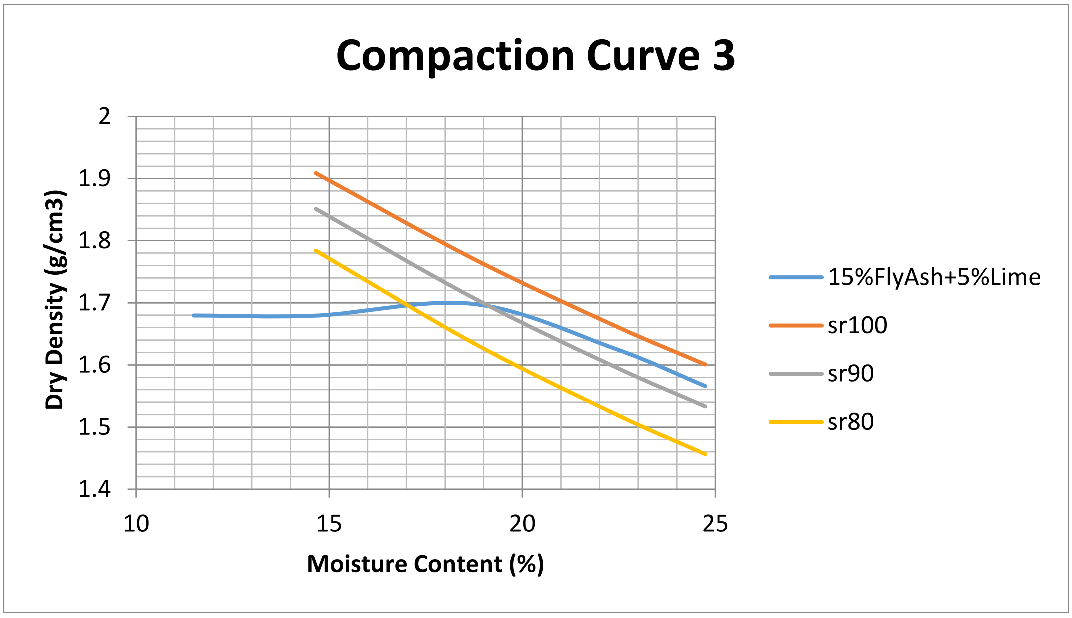


Figure 3.5 Compaction curve of 3rd mixture

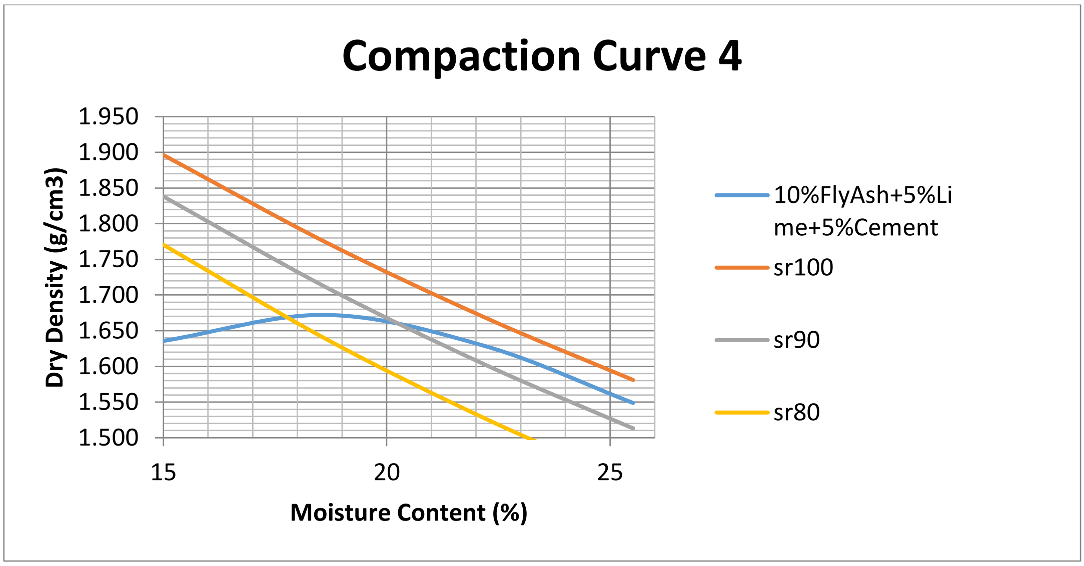


Figure 3.6 Compaction curve of 4th mixture

3.1.4 Unconfined compressive strength test

Ultimate compressive strength of all RE soil shall be a minimum of 300 p.s.i. (2,07 MPa) [18]. Samples for each mixture were prepared and the UCS test was performed (Table 3.1). 5 samples were prepared for each mixture. The average of these 5 samples was calculated and listed for each mixture. According to the results, the 1st and the 2nd mixtures (100% soil and 90% soil + 10% lime) had lower strength than the other mixtures. The 1st mixture had the lowest compressive strength value. In addition to that, because of shrinkage in this sample there were visible shrinkage cracks and a shortening in length (Figure 3.7). The 2nd mixture was better than the 1st one. Even though the 2nd mixture had better strength, it was not as good as the rest. The 3rd and the 4th mixtures (80% soil + 15% fly ash + 5% lime and 80% soil + 10% fly ash + 5% cement) had the highest compressive strength values. Also their results were quite close to each other. All UCS results (Figure 3.8) were compared and it was decided to use the 3rd mixture (Figure 3.9). In addition to that, three samples for each REW test were tested to understand their compressive strength values. Compressive strength values for section 2.7, 2.8 and 2.9 REWs are respectively 4.1 MPa, 3.8 MPa, and 4 MPa.

Table 3.1 UCS test results in MPa

Mixtures	0 Days Results	0 Days Average	7 Days Results	7 Days Average	28 Days Results	28 Days Average	90 Days Results	90 Days Average
100% Soil	0.2		0.96		1.02		1.11	
	0.19		1.12		1.04		0.94	
	0.24	0.204	1.03	1.008	0.98	1.032	1	1.052
	0.19		1.06		1.18		1.09	
	0.2		0.87		0.94		1.12	
90% Soil + 10% Lime	0.17		2.26		1.17		1.13	
	0.15		1.72		1.12		1.16	
	0.13	0.15	1.82	1.75	1.45	1.234	1.25	1.224
	0.19		1.55		1.16		1.21	
	0.11		1.4		1.27		1.37	
80% Soil + 15% Fly Ash + 5% Lime	0.54		3.92		2.32		2.3	
	0.58		4.35		4.18		4.25	
	0.55	0.56	2.91	3.84	2.1	3	2.23	3.004
	0.55		4.27		2.25		3.24	
	0.58		3.73		4.15		3	
80% Soil + 10% Fly Ash + 5% Lime + 5% Cement	0.38		4.6		2.36		3.15	
	0.3		4.09		3.18		3.18	
	0.38	0.352	3.38	4.12	2.3	3.204	2.73	3.202
	0.34		3.78		4.08		4.12	
	0.36		4.75		4.1		2.83	

Figure 3.7 UCS result of sample

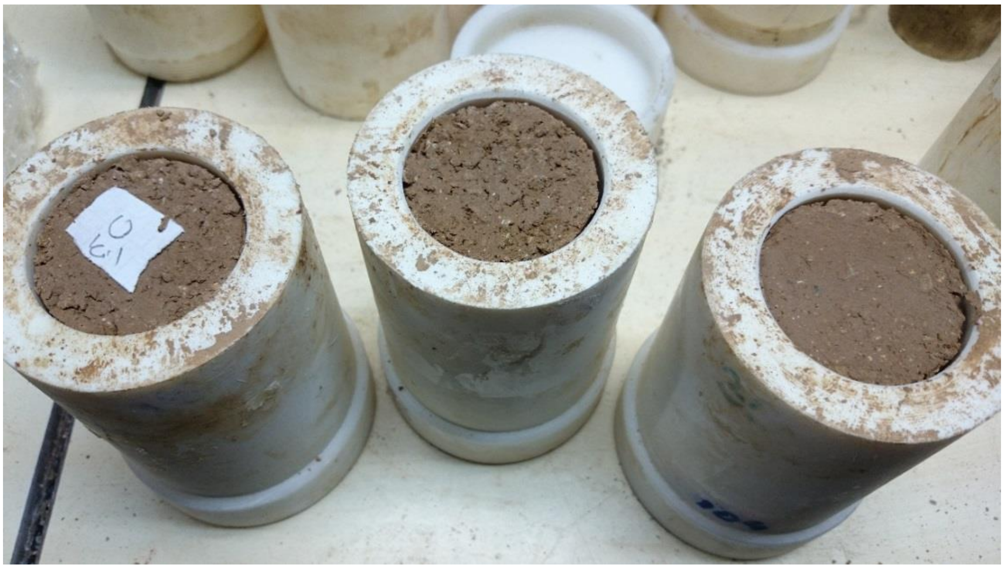


Figure 3.8 Shrinkage

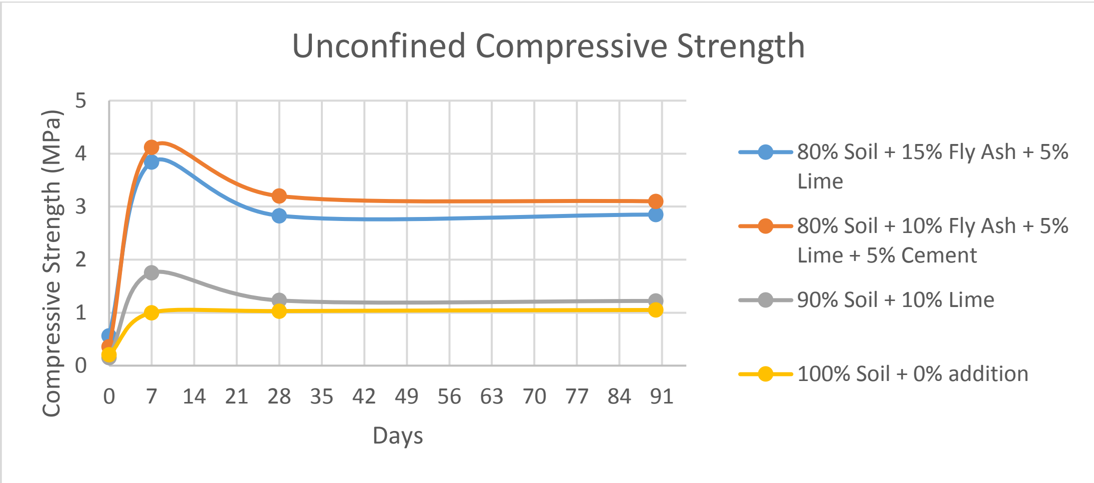


Figure 3.9 Graph of UCS results

3.1.5 Reinforcements

The reinforcement elements used for vertical and diagonal reinforcements were tested. A 12 mm diameter steel bar was tested to determine its material properties. According to the results, yields stress was 275 MPa (S275 steel). Results have shown that the reinforcement bars were S275 (Figure 3.10).

Figure 3.10 Reinforcement bar tensile strength result

3.1.6 Confinement column tests

A MTS machine was used to do the compressive test (Figure 3.11). After the first part of the wooden sticks failed, the system started to have higher cycles. This continued at each layer of the wooden sticks. (Figure 3.12), (Figure 3.13). The test was stopped when the displacement reached 100 mm (Figure 3.14), i.e. 50% of its initial length. Red line at 4 MPa represent the compressive strength of mixture. Wooden sticks were placed and connected to each other's test results of "section 2.4.1.1.". Test vi from the "section 2.4.1.1.", had performed the best result and a single wooden confinement element failed around 2.45 kN. The wooden confinement column failed around 40 kN. The mixing's itself compressive strength value is 4 MPa (40 kN for 100 mm x 100 mm sample).

Figure 3.11 Wooden confinement column test

Figure 3.12 Wooden confinement column crashing fail of 1st level

Figure 3.13 Wooden confinement column crashing fail of 2nd level

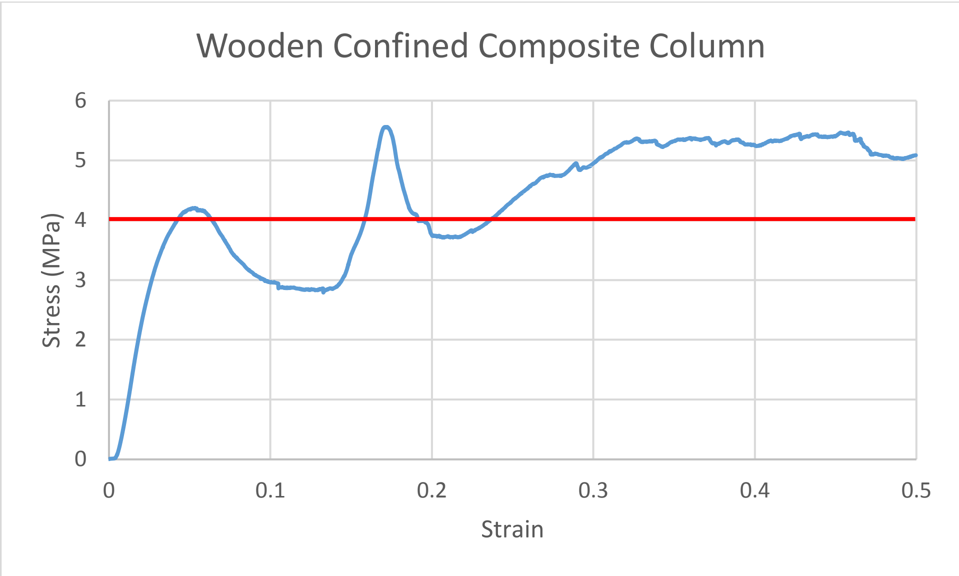


Figure 3.14 Wooden confined column results

Timber confinement elements were prepared and tested without any soil to understand the effect of the timber on the previous test of timber confinement column. Timber confinement elements were prepared by following the same procedure of the timber confinement column test (Figure 3.15). However, this time soil was not added to the column. Only timber confinement element was tested (Figure 3.16) (Figure 3.17). The result of the timber confinement element test was calculated by considering the vertically continuous timber area (Figure 3.18). Previous test specimen of timber confined column failed around 42 kN. Only timber confinement element test was also tested and a failure capacity of around 49 kN was obtained, which is surprisingly larger than the confined column. The reason of the difference might be caused by the stresses developing due to the outward expanding infill soil pressure exerted on the timber confinement frame. Possible damages to the timber during the compaction, effect of the wetness of soil on timber pieces, or unnoticeable cracks during drilling holes to the timber pieces might have also had important effects on the reduced timber confined soil column strength.

However, the overall plastic behavior and the way column sustains load on it is drastically reduced due to slender nature of the timber frame. The rammed soil column with timber cage has sustained the load more successfully; the plastic load level on the timber confined rammed earth column was around 55 kN, while timber cage has dropped down to 19 kN, which is about 34% of the earth column.

Figure 3.15 Wooden confinement element

Figure 3.16 Wooden confinement element's deformation

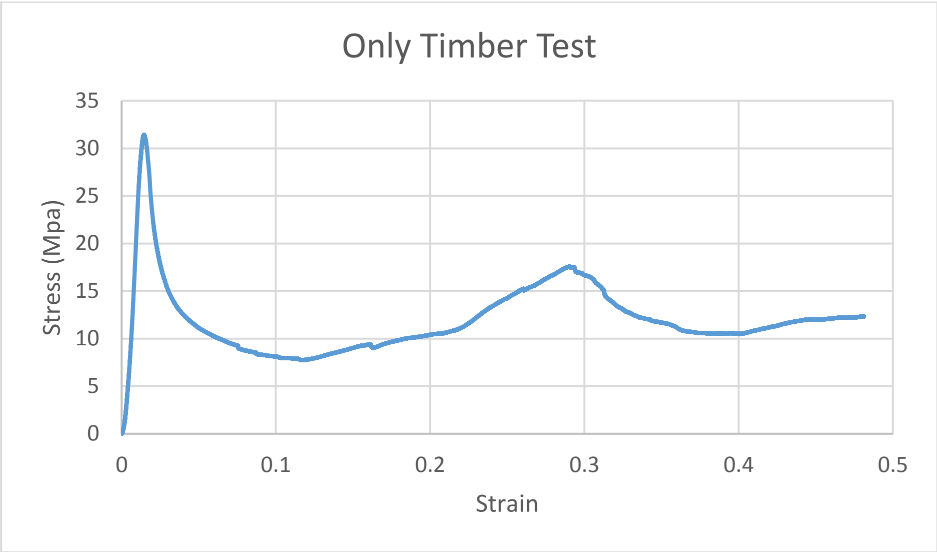


Figure 3.17 Wooden confinement element test results

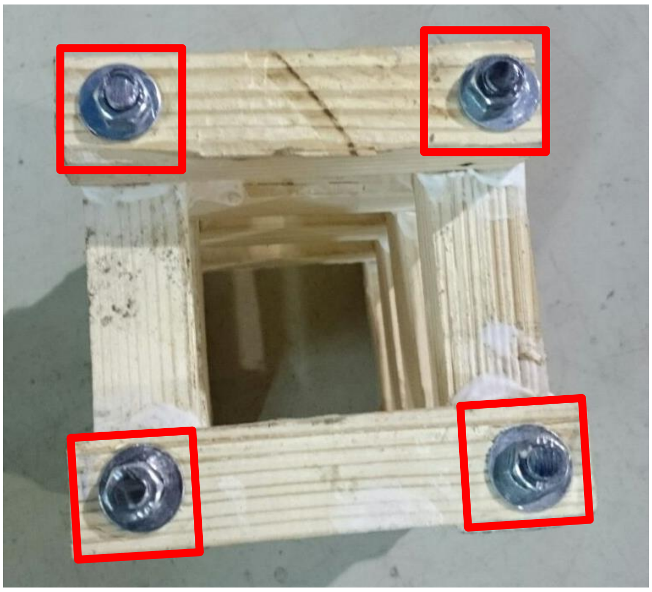


Figure 3.18 Continious compressive areas

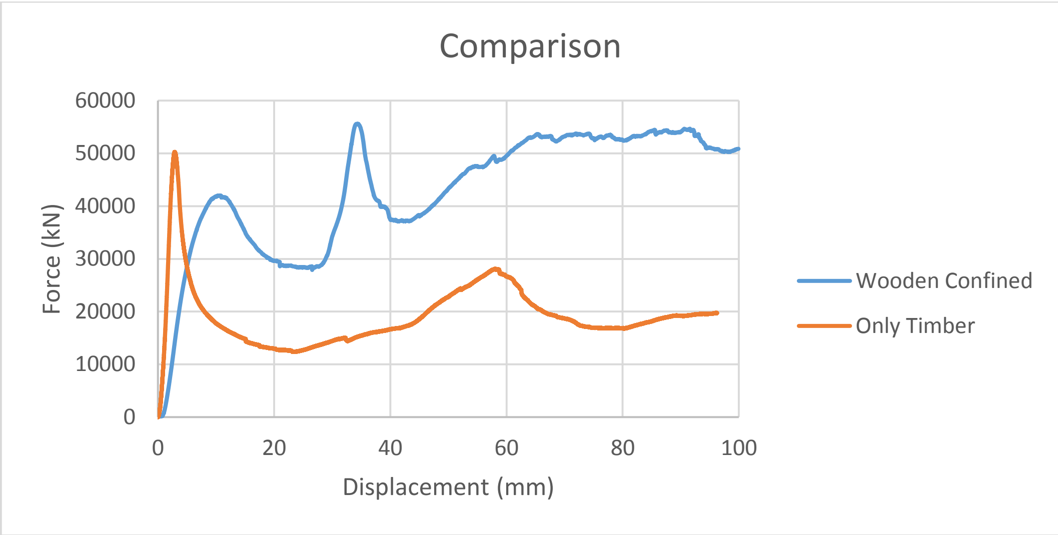


Figure 3.19 Comparison of results

Comparison of wooden confined and only timber test results have shown (Figure 3.19). Confinement effect is prematurely lost because of shear failure of bolted connection. High humidity of timber confinement during testing may have also adversely affected results. If wood confinement could be applied without drilling holes as a continuous element, then the confined timber RE column would have been expected to achieve higher strength.

A hoop steel confinement column test was performed with the same machine (Figure 3.21). Unlike the wooden confinement column, there were not significant cycles on the force-displacement diagram. However, it didn't break apart and protected its entirety (Figure 3.22), (Figure 3.23). Red line at 4 MPa represents the compressive strength of mixture. The test was stopped when displacement reached 50 mm (Figure 3.24), i.e. 25% of its initial length.

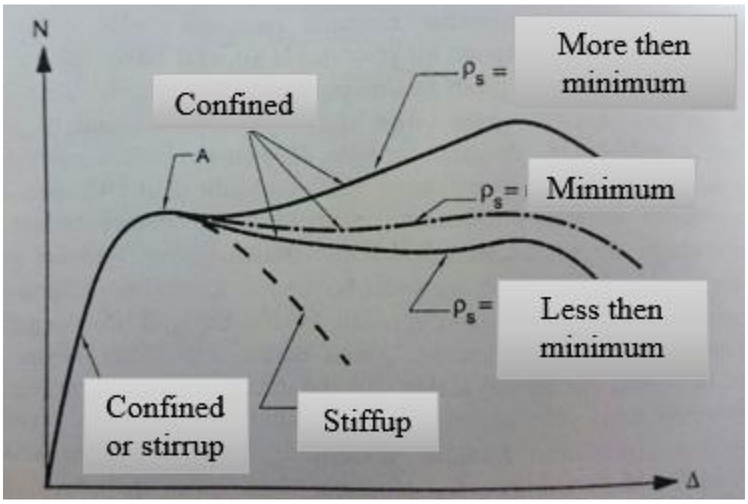


Figure 3.20 Reinforced and confined column behavior graph [29]

According to Ersoy's book [29] (Figure 3.20), confinement hoops does not strengthen column at the first peak. Strength gain can be obtained according to the hoops ratio. In this test, hoops already effect the strengthening on the first peak because of the weak material properties of infill soil; increased to 6 MPa from the original strength of around 4 MPa. However, strength gain did not reach to the calculation results as shown in section 4.2.2; which was expected to be 4 times the confinement stress plus the original strength found to be 16 MPa (6 MPa < 16 MPa). The big difference between the strength of soil and the strength of steel hoops as well as the lack of vertical reinforcement cause the steel hoop column to failure much earlier than the steel hoops reaching their yield strength.

Figure 3.21 Steel hoop confinement column test

Figure 3.22 Steel hoop confinement column crushing

Figure 3.23 Final view of steel hoop confinement column

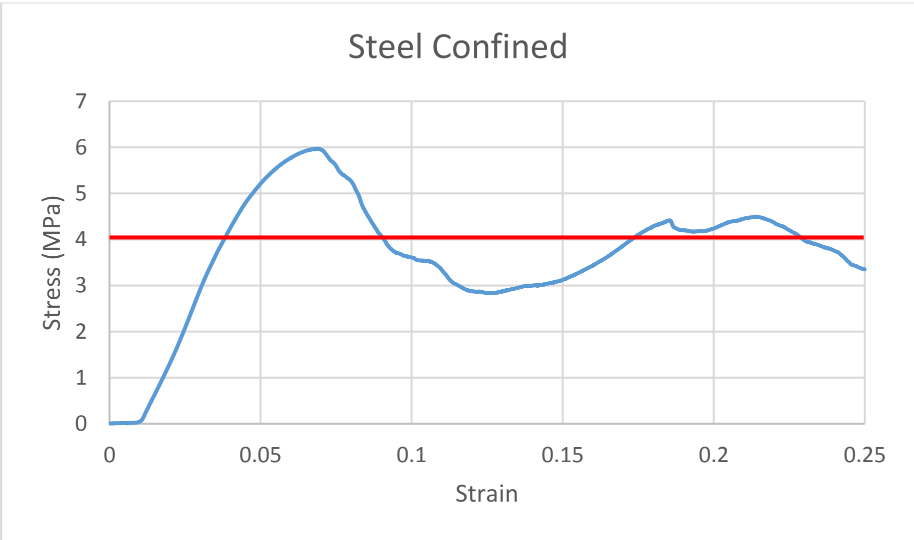


Figure 3.24 Steel hoop confined column test results

In this study, the first failure peak was considered. Since the steel hoop column had higher first peak than the timber confinement column, the column with steel hoops confinement elements was preferred and used inside the REW test specimens.

3.1.7 Stabilized unreinforced rammed earth wall

The first REW test was performed as a stabilized unreinforced sample. After the 28 days of curing, there were already cracks at the corners because of shrinkage (Figure 3.25). The REW tended to shrink, however the serrations at the bottom didn't let this happen. That caused cracks at these points. Subsequently test setup was prepared, two 75 kg weights (150 kg in total) were added as dead loads to the top and measurement instruments were placed. The dead load and measurement instruments were applied in the same was as in the other tests. In this test first wall was tested without any reinforcement. Failure was a horizontal crack close to the bottom line (Figure 3.26). Failure occurred when the force reached 3.15 kN (Figure 3.27).

Figure 3.25 1st Cracks

Figure 3.26 2nd cracks

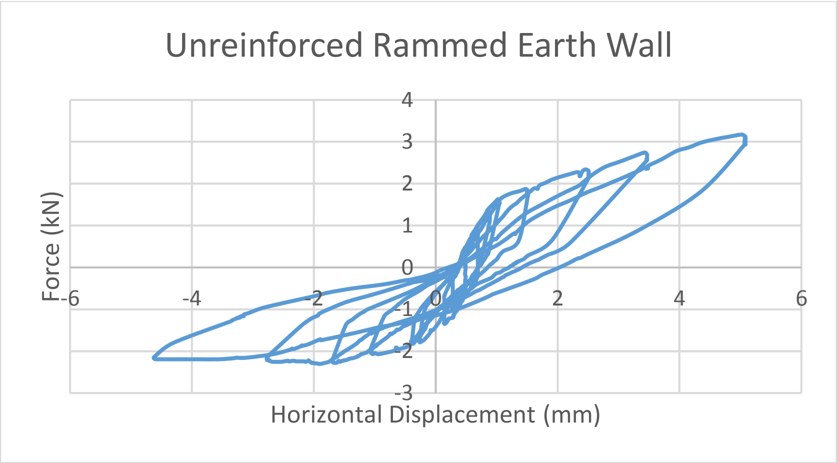


Figure 3.27 Unreinforced REW results

3.1.7.1 Stabilized and vertical (outside) RREW

After the first failure vertical reinforcements were welded. Reinforce elements were added to both faces but only at one side of the faces. That is why, only impulse was carried out. Stabilized vertical reinforced REW failed and tried to slide on previous cracks around 48 kN (Figure 3.28).

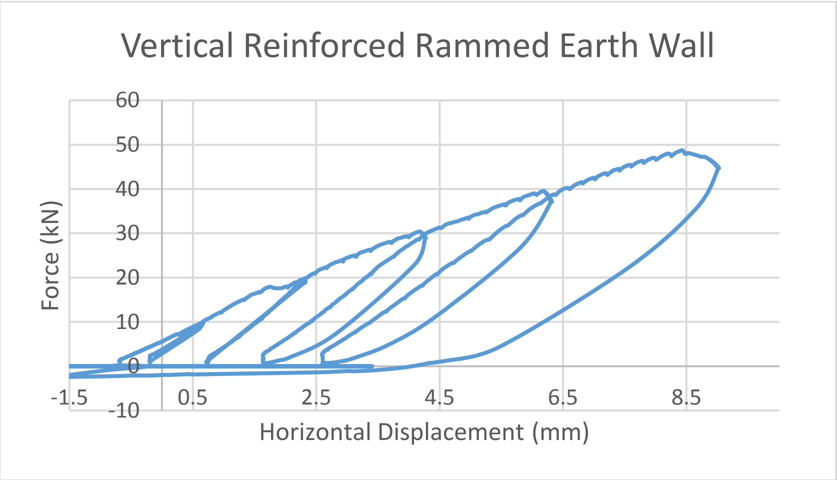


Figure 3.28 Vertical RREW results

3.1.7.2 Stabilized and vertical + diagonal (outside) RREW

After this failure of vertical reinforced REW, diagonal reinforcement elements were welded in addition to the current system. During the loading, first cracks occurred and expanded at the joint (Figure 3.29). Subsequently, this crack expanded and a small smash started at this point (Figure 3.30). Finally, just after steel reinforcements failed, the wall failed with a great diagonal crack (Figure 3.31). The wall consisting of two reinforcements failed around 140 kN (Figure 3.32).

Figure 3.29 Cracks at corners

Figure 3.30 Expansion of the cracks at joint

Figure 3.31 Failure

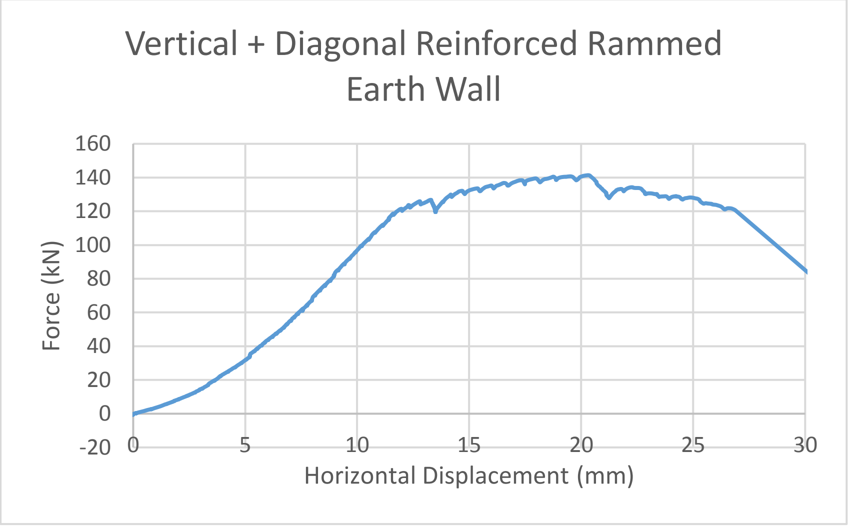


Figure 3.32 Test 1.3 - results of vertical + diagonal RREW

3.1.8 Stabilized vertical (inside) + diagonal (outside) RREW

First the REW was built by placing vertical reinforcement bars inside. However, because of the poor compaction between the bars and the formworks at the sides, there were visible cracks after removing the formworks (Figure 3.33). In addition to that, the following three days, horizontal cracks occurred at the layers (Figure 3.34). Because of the shrinkage, the wall tended to shrink horizontally and vertically. The bonding between the bars and the compacted soil was strong. Therefore, during the shrinkage, horizontal cracks occurred.

Figure 3.33 Cracks at the sides

Figure 3.34 Shrinkage cracks

First test was performed. During the test, deflection of vertical reinforcements was observed and when vertical reinforcements buckled, they harmed the sides of the wall (Figure 3.35). This was noted as a disadvantage of using reinforcements inside the wall. They might cause loss of the earthen material at the sides (Figure 3.36). The wall failed when force the reached 30.3 kN (Figure 3.37). This was more soon than expected. The cracks might have occurred because of the shrinkage.

Figure 3.35 1st failure of RREW

Figure 3.36 Expansion of the cracks at side

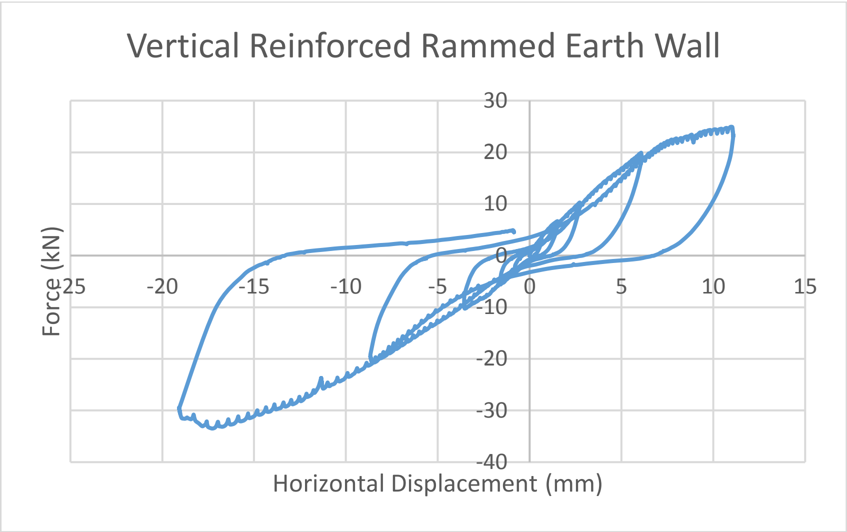


Figure 3.37 Test 2.1 - results of embedded vertical RREW

Later diagonal reinforcement elements were welded (Figure 3.38). The test was performed again. During the test, first the vertical cracks at the sides which occurred due to the vertical reinforcements were expanded and caused more earth loss (Figure 3.39). The wall failed when the force reached around 120 kN (Figure 3.40). Failure occurred after the diagonal reinforcement elements yielded (Figure 3.41). Even though it didn’t show a ductile behavior, its failure was not as brittle as the first rammed earth wall test at section 3.1.5.2.

Figure 3.38 Addition of diagonal reinforcements

Figure 3.39 Expansion of the cracks at side

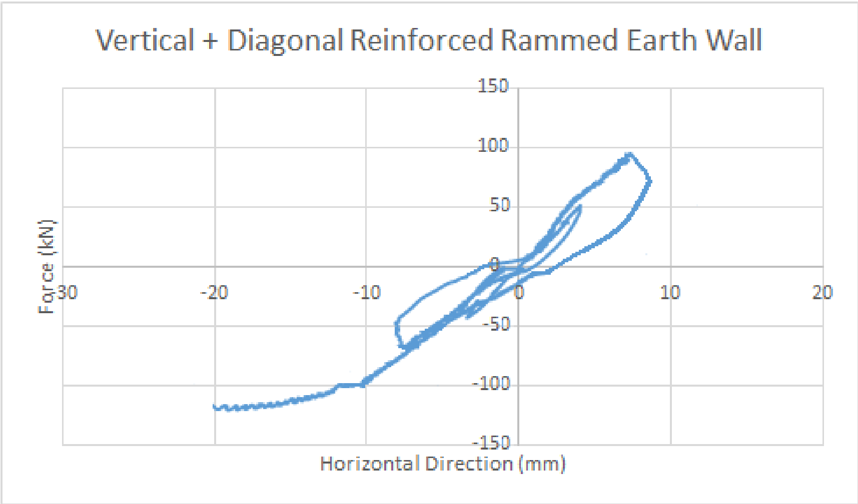


Figure 3.40 Test 2.2 - vertical + diagonal RREW results

Figure 3.41 Failure

3.1.9 Stabilized steel confinement vertical + diagonal (outside) RREW

During the curing, shrinkage cracks occurred. The most visible ones were at the lateral reinforcement binders' locations (Figure 3.42). This might occur because of unwell-placed binders during the compaction. While compacting one side of the binders, the other side dislocated and during the compaction of the rest of it, by the compaction force located it back in its place again. This might have caused the very visible cracks at this location.

The test was performed and failure occurred as an expansion of existing cracks due to the shrinkage (Figure 3.43). When the force reached 2,75 kN wall failed (Figure 3.44).

Figure 3.42 Cracks because of shrinkage

Figure 3.43 Shrinkage cracks after curing

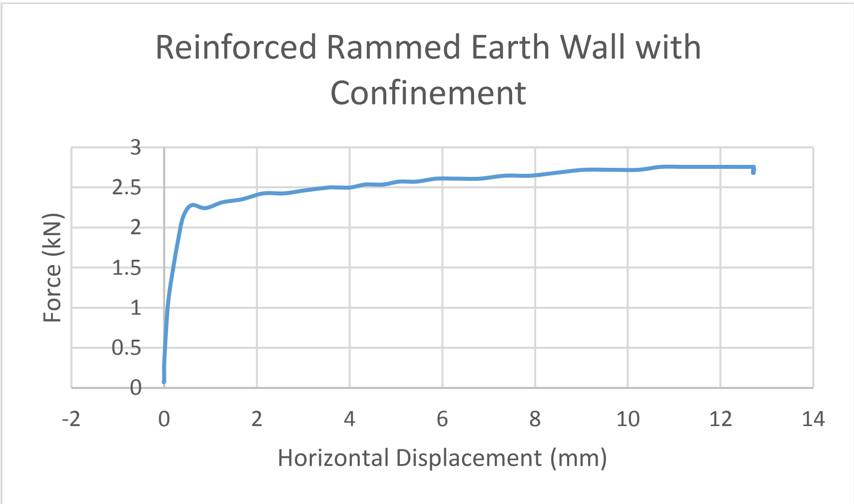


Figure 3.44 Test 3.1 - results of confinement reinforced REW

After the first failure without any outside reinforcements, vertical reinforcement bars and diagonal reinforcement bars were welded and the test was performed again (Figure 3.45). Similar to the previous ones, first cracks occurred as an expansion of existing cracks due to the shrinkage and previous test (Figure 3.46). During the test it was also observed that especially horizontal reinforcement bars had a significant deflection. If these reinforcement bars would have been placed inside the wall, this might have harmed the wall from the inside and might have caused very early failure. After a while, cracks and loss of earth were observed at the sides of the wall. Subsequently confinement hoop elements were visible (Figure 3.47). When the force reached 130 kN the wall failed (Figure 3.48). Failure started with expansion of cracks and continued with crushing (Figure 3.49).

Figure 3.45 Addition of vertical + diagonal reinforcements

Figure 3.46 Expansion of cracks

Figure 3.47 Confinements

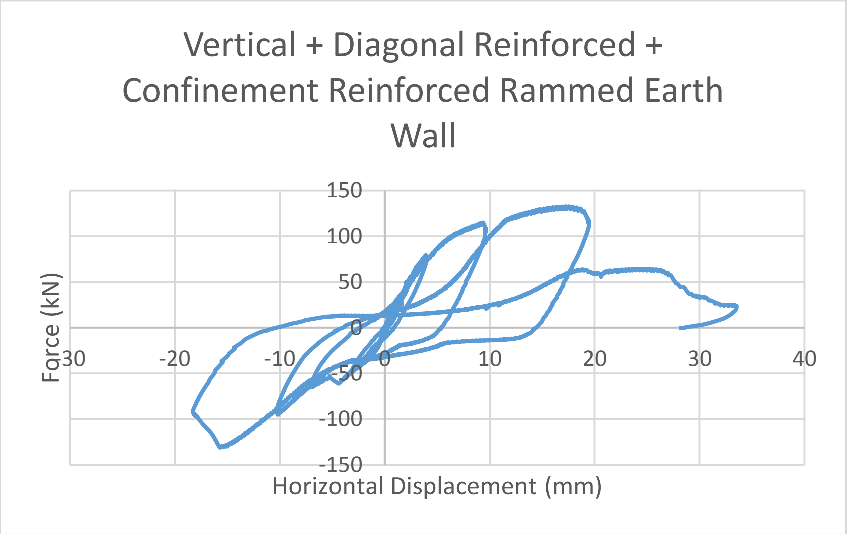


Figure 3.48 Test 3.2 - results of vertical + diagonal RREW with confinement

Figure 3.49 Failure

3.1.10 Summary of test results

Three REW test samples were prepared. The first wall sample was tested and when the force reached 3.15 kN, the REW failed with a horizontal crack at the bottom in overturning mechanism. Vertical reinforcements were added to the sides of the UrREW to strengthen the wall against overturning. The test was carried out again and failure occurred around 48 kN with a diagonal crack, meaning that the wall was 15 times more strong. Diagonal reinforcements were also added to the existing system to strengthen the wall against stresses. The RREW failed around 140 kN which means that this wall was 45 times more strong than the UrREW. However, failure was brittle. Even though the RREW's strength was satisfactory, it should present a ductile behavior for safety.

A second REW was prepared with the same procedure of the first REW with a difference of reinforcement placement. In the second REW, vertical reinforcements were placed inside of the wall to protect the reinforcement bars from the corrosion. However, the end sides of the REW could not compacted well because of the small space between formwork and reinforcement. Lack of compaction on these areas caused early cracks and loss of earthen material. This already decreased the prediction of strength of the second REW vis-à-vis first RREW results. When the test was carried out without any external reinforcements (inner vertical reinforcement only), the REW failed around 30.3 kN; which is almost 9.6 times stronger than the UrREW but weaker than the first RREW test, which was 45 kN. The loss of earthen material and early cracks due to lack of compaction caused that loss of strength (9.6 < 15). Diagonal reinforcements were added to the system from outside and the test was carried out once again. The RREW failed around 120 kN, which was 38 times stronger then the UrREW but smaller than 1st test ultimate capacity (120 kN < 140 kN). However, the second RREW also failed brittle. The reduction of the ultimate strength as compared to the first wall was thought to be due to the loss of earthen material around the internal reinforcement. Additionally, cracking that occurred prior to testing was caused by shrinkage.

Confinement steel hoops and two horizontal reinforcement grid were placed in the third REW specimen to achieve better ductile behavior. Confinement hoops were placed to the bottom corners by 20 mm vertical spacing and the horizontal reinforcement grids were placed at 200 mm and at 400 mm above the bottom. These two layers of horizontal reinforcement were planned to reduce diagonal cracking and increase ductility. Vertical and diagonal reinforcement bars were also externally added to the system. When the test was carried out, the RREW failed around 130 kN, which was 43 times stronger then the UrREW. Even though the third test strength was less than the first one (130 kN< 140 kN), failure was ductile and due to its ductile behavior, the third RREW was chosen as the best option to provide the highest level of safety during an earthquake.

CHAPTER 4

STRUCTURAL ANALYSES AND DESIGN PROCEDURE

4.1 Analytical software analysis

In the scope of this thesis, seismic performance of rammed earth walls with Structural analysis of the REWs was a tricky and tedious task. SAP2000 has been choose as an analytical software since it is capable of performing nonlinear analysis. Modeling in SAP2000 was performed using nonlinear shell elements and frame members with hinges having nonlinear capabilities (Figure 4.1). The analytical models created in SAP2000 have yielded meaningful results similar to hand calculations (Appendix A) and test results. However, successful convergence to results were not always possible and the reason behind unsuccessful analyses without a convergence using SAP2000 remained unclear. The comparison of structural analyses and test results are listed below under each heading.

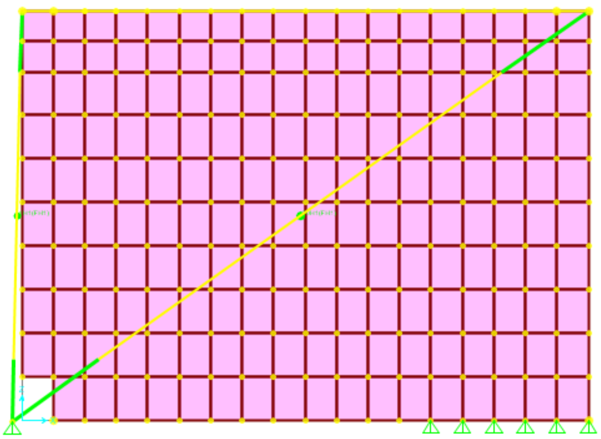


Figure 4.1 Modelling the system in SAP2000

The Finite Element Model (FEM) of the first wall, which was reinforced from outside by vertical and diagonal reinforcements, had frame elements that are not attached to the shell elements that represent the REW. On the contrary, FEM's vertical frame elements of the second wall, which had embedded vertical reinforcement and exterior diagonal rebars, were attached to the shell members while diagonal members were not attached. The third wall's FEM had exterior frame members that are not attached to the shell members but additional vertical frame members with confinement properties that are attached to the shell members. Each one of the analytical models were nonlinearly analyzed using push-over nonlinear analysis with large deformations and P-delta effects (Figure 4.2), (Figure 4.3), (Figure 4.4).

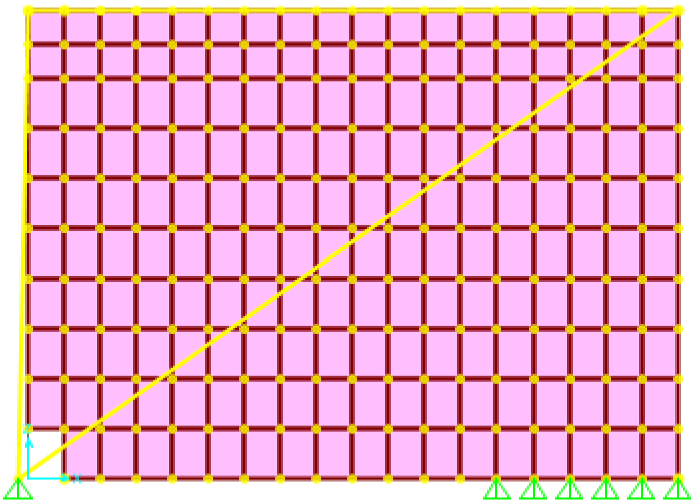


Figure 4.2 1st RREW test model

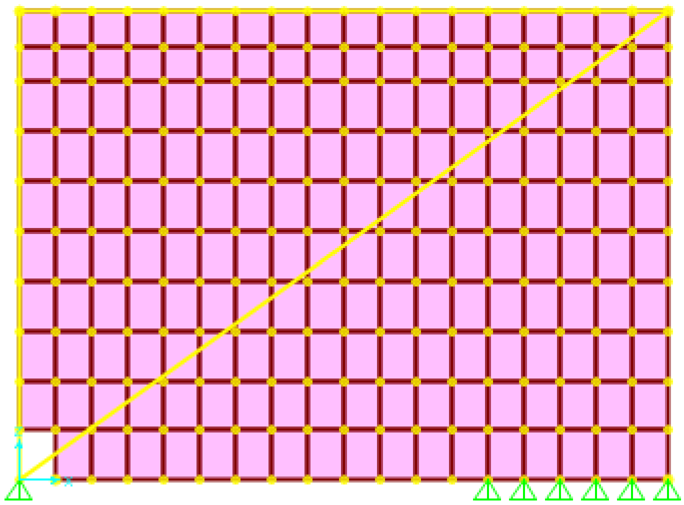


Figure 4.3 2nd RREW test model

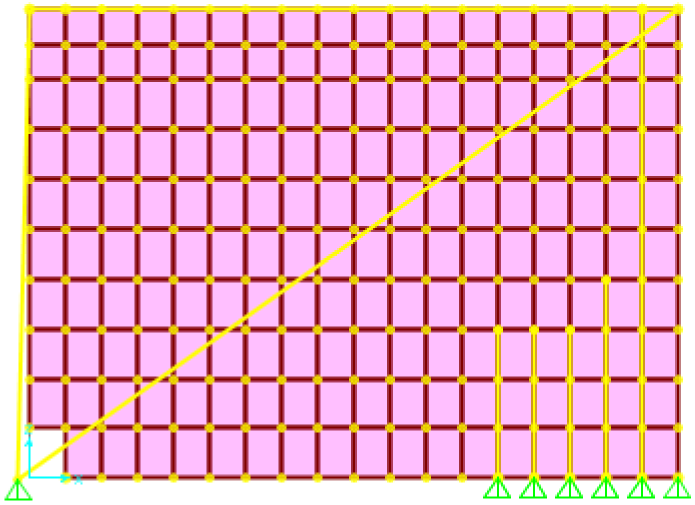


Figure 4.4 3rd RREW test model

Material data definition in the analytical model was the first step. The material properties listed (Table 4.1) was defined in SAP2000 model. 50 mm diameter x 100 mm height samples were prepared from the same mixtures of the walls', in the day of construction the wall. These samples were left to dry for 28 days and then unconfined compressive strength test were carried out. The average value of the samples was used in each wall design in SAP200. That is why, in SAP2000, 4.1 MPa, 3.8 MPa, and 4 MPa used as compressive strength. Tensile strength was assumed 0.4 MPa. Additional nonlinear properties for each material have been defined using the nonlinear hinge properties for the frame elements and confinement hoops. Shell member nonlinearities were defined using the shell section layer definition screen of shell section data menu.

Table 4.1 Material properties

	Yield or crushing strength (MPa)	Poisson's ratio	Elasticity (MPa)	Density (kg/m3)
Steel Rebars (Frame member)	275	0.30	210000	7850
Earth wall (Shell members)	4	0.13	1600	1700

Reinforcement material with S275 steel was already available and defined by selecting it on SAP2000 (Figure 4.6) and hinge properties were defined (Figure 4.7). However, the soil mixture properties should be added by manually. That is why, elasticity modulus, poisson ratio, density and strength values were defined to the nonlinear material data part of SAP2000 (Figure 4.5). Earthen material wall was created by using layered nonlinear shell elements (Figure 4.8) (Figure 4.9). Earthen material's behavior was considered elastoplastic (Figure 4.10). Third RREW's confinement areas were defined as confined column (Figure 4.11) (Figure 4.12).

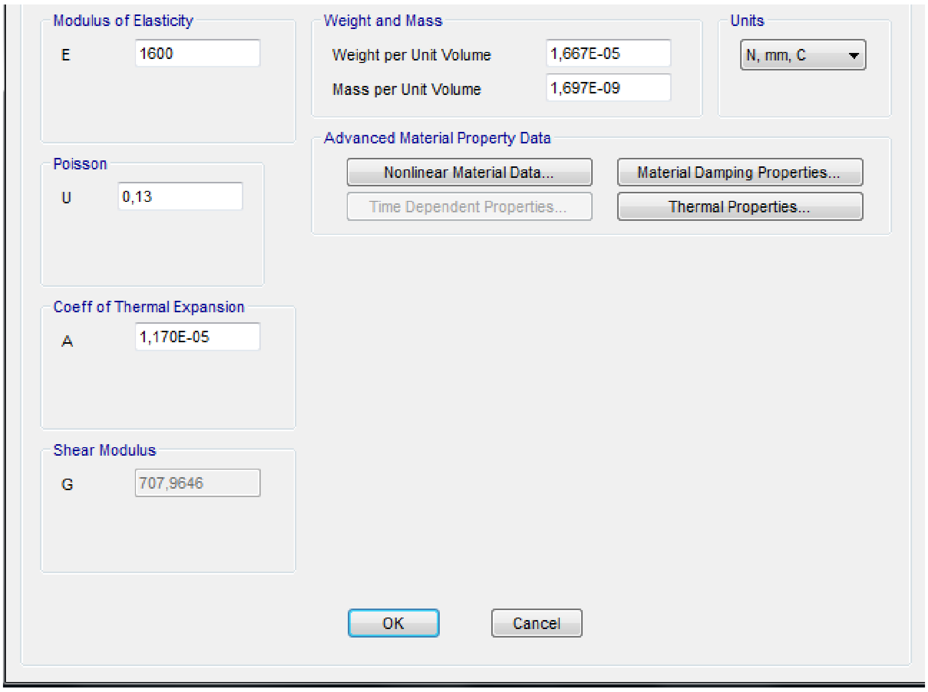


Figure 4.5 Material properties of shell members

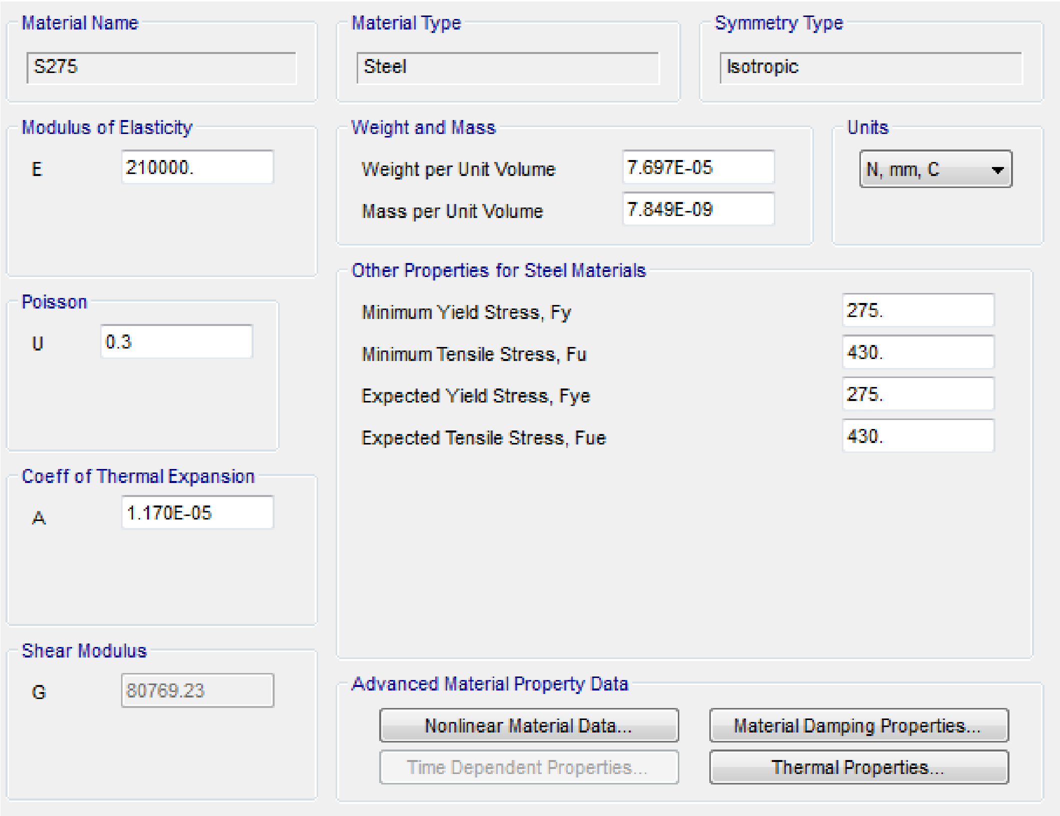


Figure 4.6 Material properties of steel

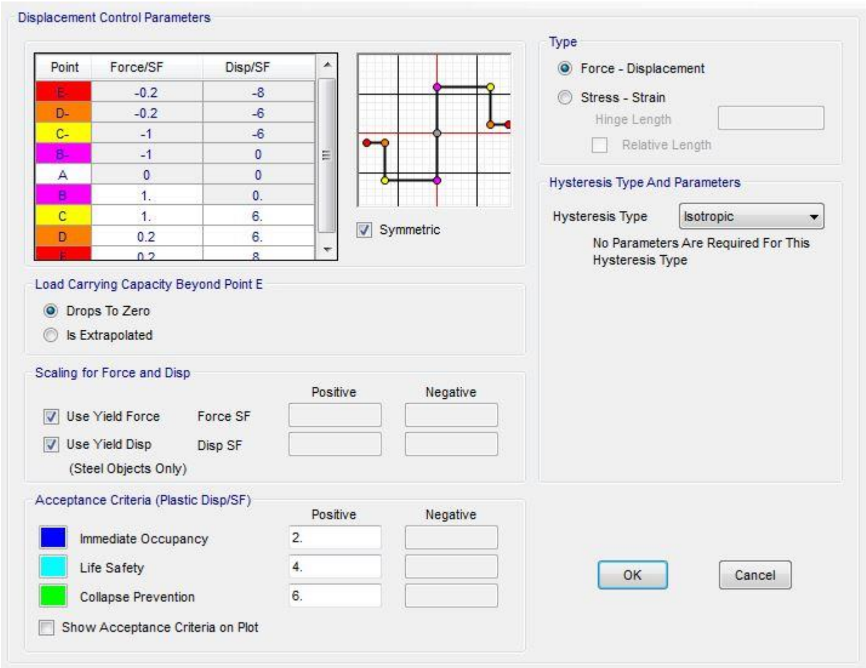


Figure 4.7 Hinge properties

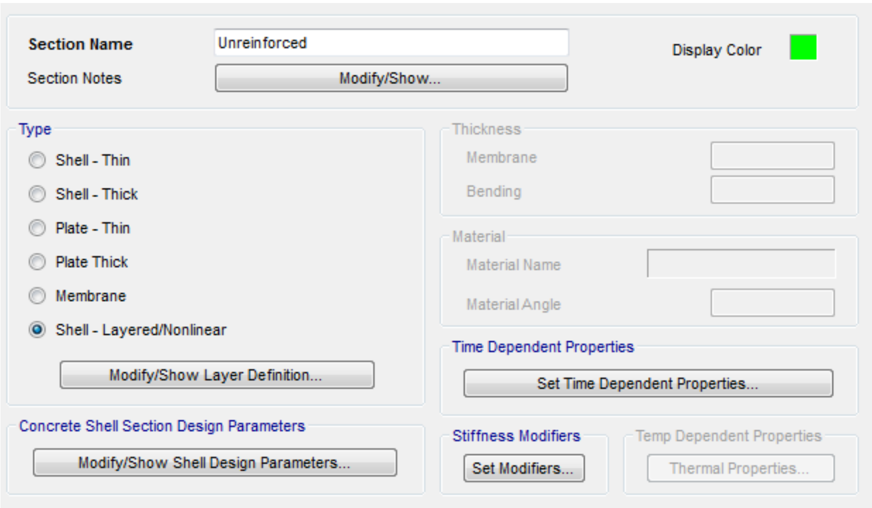


Figure 4.8 Shell's nonlinear properties

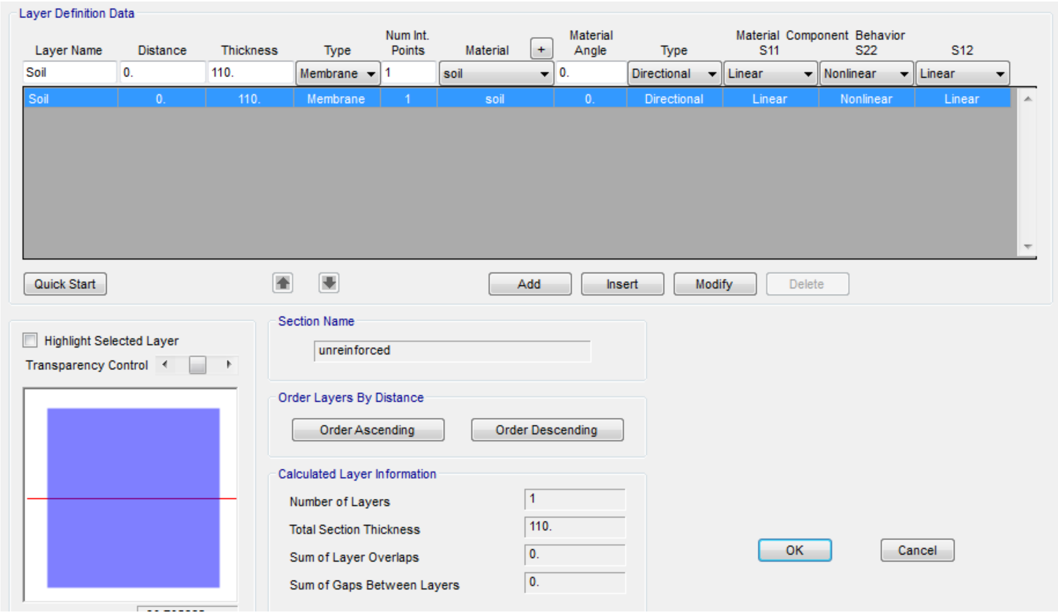


Figure 4.9 Shell's nonlinear properties

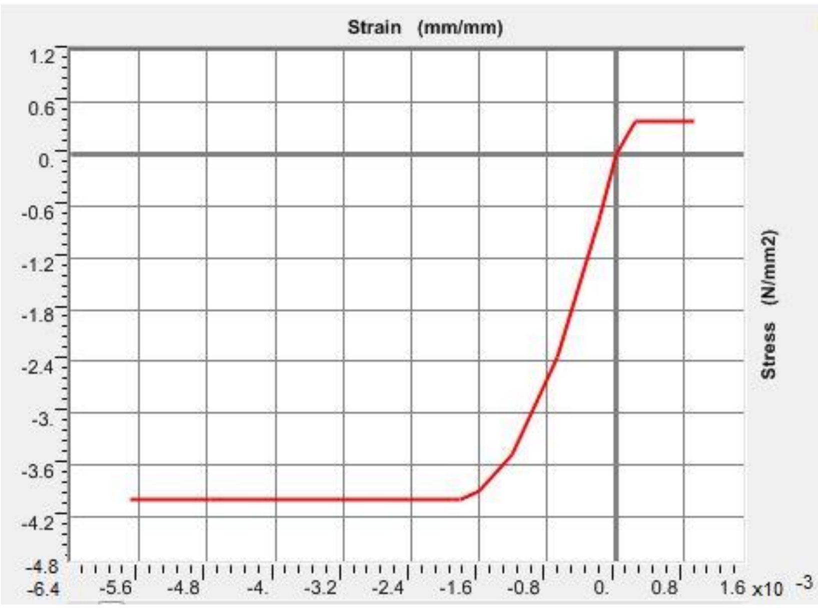


Figure 4.10 Earthen material properties

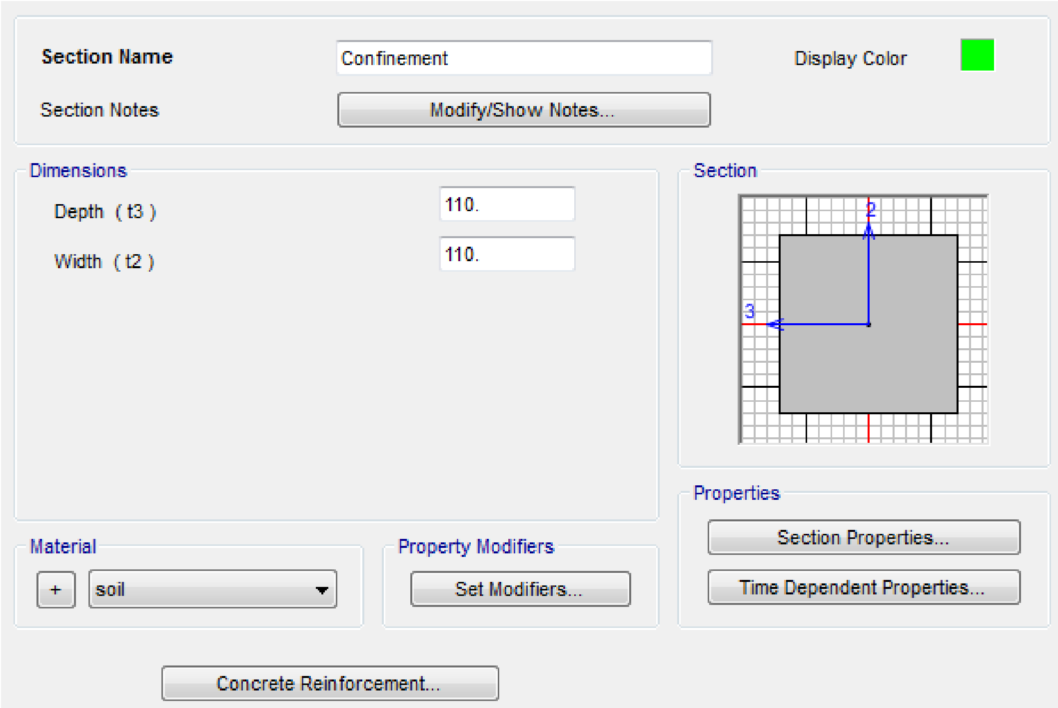


Figure 4.11 Confinement properties

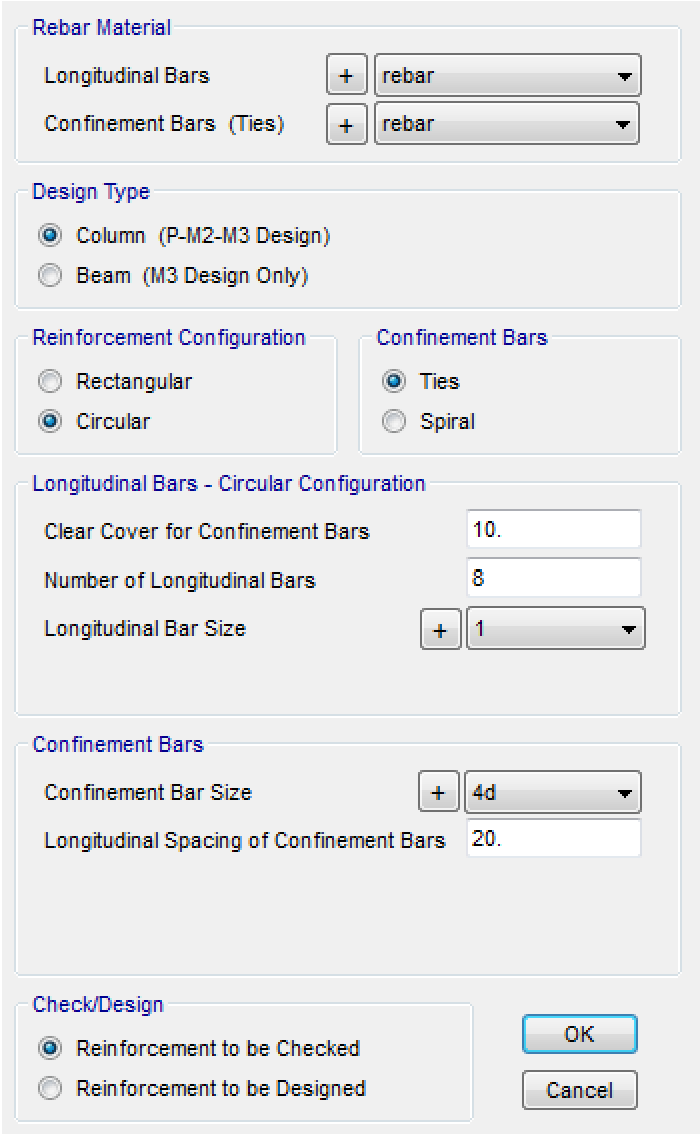


Figure 4.12 Confinement properties

Geometric properties were simply modeled using a proper meshing size (Figure 4.13).

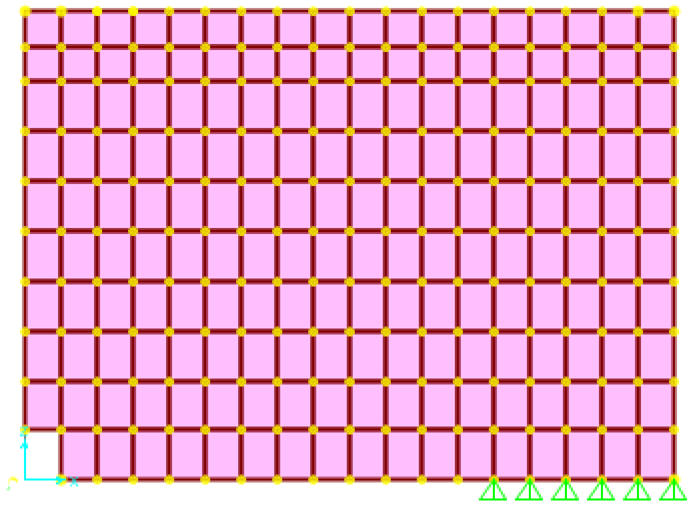


Figure 4.13 Meshing

In all of the FEMs, a horizontal steel box section was modeled at the top of the wall similar to the one in the laboratory tests. All joints of the frame were attached to the wall's shell members to gradually transfer the shear force to the wall.

First wall with vertical and diagonal reinforcement from outside created by adding frame elements: vertical and diagonal reinforcement were added to system. The aim of using these reinforcements were to take tension stress and they would buckle under compression; therefore, their compressive capacity was defined as zero. Hinges were defined and assigned to the frame elements to define their nonlinear properties. Reinforcement elements were outside of the wall and were not touching the wall from either faces. That is why, shell points at the meshing locations were disconnected from the vertical reinforcement. The same procedure was repeated for the diagonal frame element modeling as well. The push-over analyses for each test was carried out using the nonlinear FEM in SAP2000 and the corresponding analyses results are provided under each figure below (Figure 4.14), (Figure 4.15),

(Figure 4.16). The comparison of UrREW and RREW analytical model ultimate strengths and corresponding experimental results (Table 4.2) and quite similar results were obtained.

Figure 4.14 Pushover curve of the first wall's final test

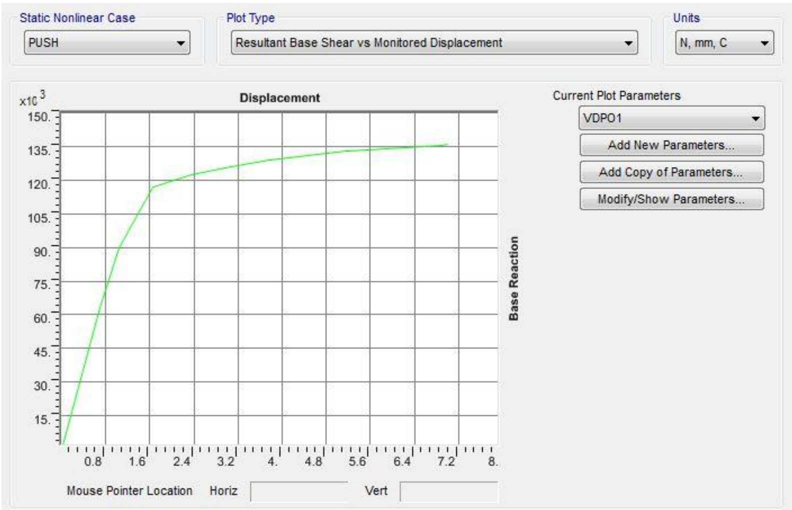


Figure 4.15 Pushover curve of the second wall's final test

Figure 4.16 Pushover curve of the third wall's final test

Design of the second wall was quite similar to the first one. The only difference from the first one was the vertical reinforcement elements were placed inside the wall instead of outside. Because of that, the vertical reinforcing frame element was attached to all shell nodes on its way and then restrained at the end using pin support. The second wall’s diagonal member was added by following the previous wall’s procedure and not connected to the shell member nodes.

In addition to the vertical and diagonal reinforcements, the third wall also had confinement steel hoops at the corners. Shell area elements, vertical and diagonal steel reinforcement elements were designed same as the first wall’s design. A new reinforced frame section was designed to represent confinement area. Material of this section was defined same as the soil properties and longitudinal bars in these columns were defined as four very thin sections with 1mm diameter. However, the confinement reinforcement was defined as circular ties with 20 mm spacing and 4mm rebars. After section defining, these confinement elements were assigned to the corner of the system and connected from the points to the shell to let the system work together. Nonlinear hinges were also assigned to these elements.

The ultimate strength in analytical results obtained from SAP2000 were matching with the hand calculations and also lab wall loading test results. However, the displacement values obtained in the SAP2000 analysis were different than the test deflections. Further investigation of deformation demand calculated by hand matched the analysis results but not the test deflections. The conclusion was drawn that the experimental wall setup was not perfectly fixed to the ground and the vertical and diagonal rebars had some deviation from being perfectly straight. Flexibility at the welded connections and some possible gaps are thought to have played a role in the relatively large horizontal deflections of the test walls when compared against the computer modeling and hand calculation results. This however, is an important result since the actual wall deformation capabilities will be much higher than those calculated by hand or predicted by computer analysis. An actual drift ratio up to 3% of the height is a very promising result for a brittle REW alleviated using vertical and diagonal rebars and confinement hoops.

Table 4.2 Comparison of the results

	Experiment (kN)	Analysis (SAP2000) (kN)	Hand Calculations (Appendix A) (kN)
RREW Test 1.3	140	142	154
RREW Test 2.2	120	135	154
RREW Test 3.2	130	138	154

4.2 Design procedure

A practical design procedure to be used in actual structures is prepared. The design procedure includes determination of reinforcement bars' (that will be placed outside of the REW) cross sectional areas, which are named as A1 for the vertical and A2 for the diagonal members. Furthermore, the single segment wall with length of 'L' and height of 'H' is used to calculate compression strut direction and areas by considering wall thickness 't'. Relevant generic FE modeling is carried out to determine the nonlinear behavior by removing support restraints that register tensile reaction. Iterative process carried out to determine a practical wall length that is under compression is listed (Figure 4.18 a) 1st iteration, b) 2nd iteration, c) 3rd iteration, d) 4th iteration) (Figure 4.19) and shown in the figure below as L/3 (Figure 4.17).

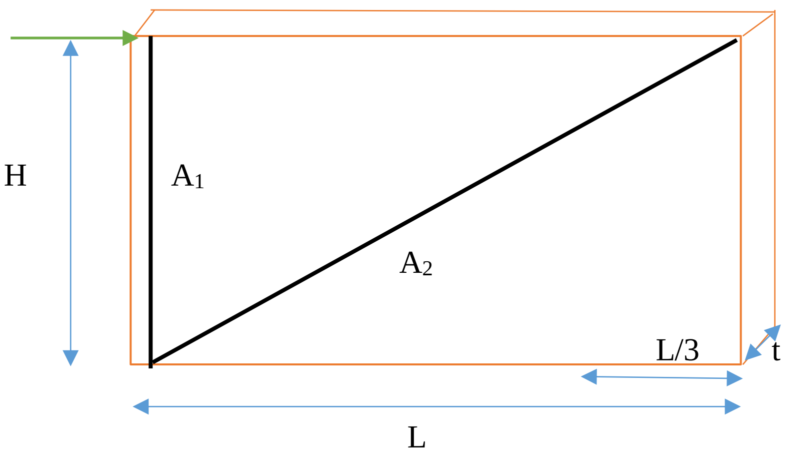
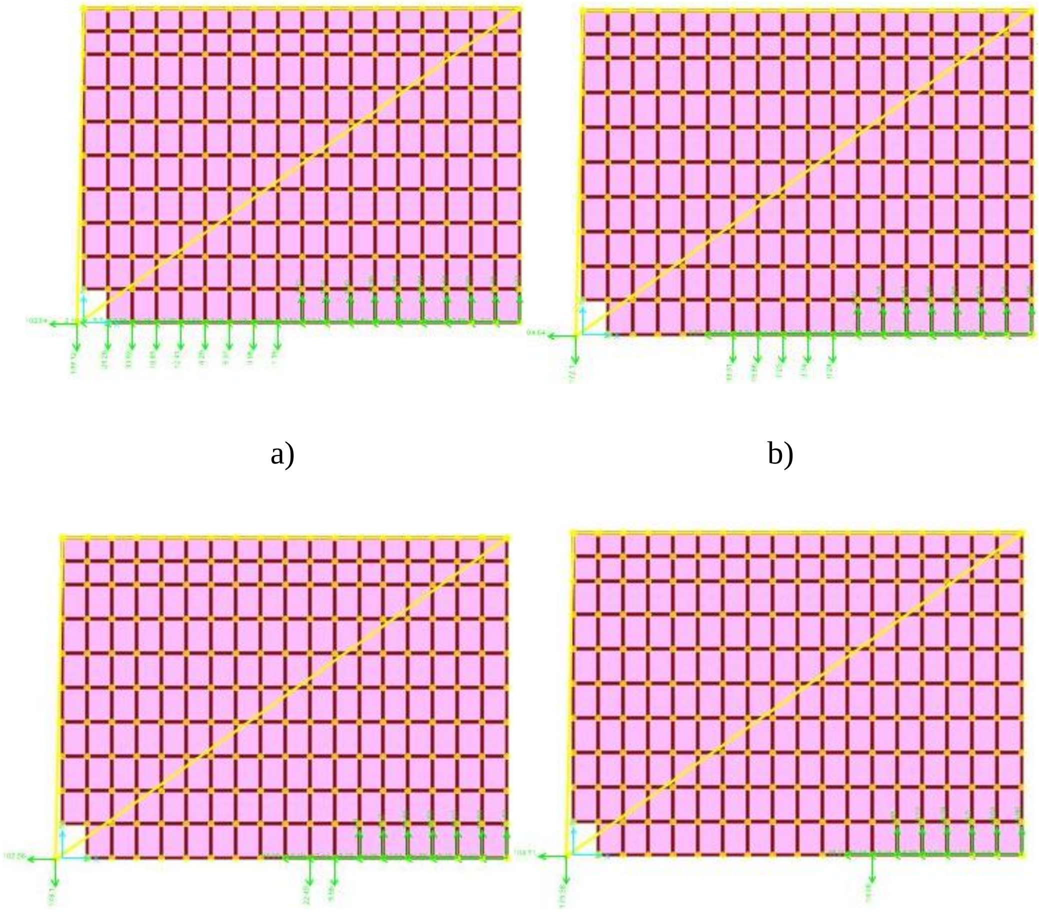


Figure 4.17 Sizes of REW



a) b)

c) d)

Figure 4.18 Iterations of REW a) 1st iteration, b) 2nd iteration, c) 3rd iteration, d) 4th iteration

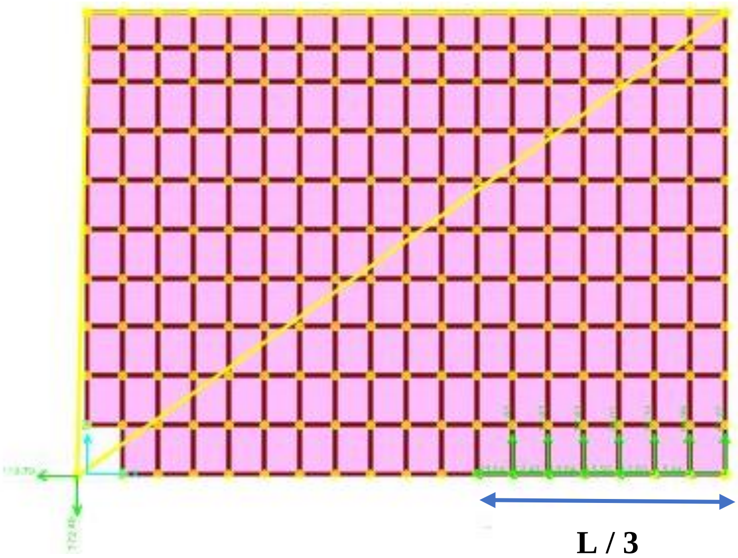


Figure 4.19 Final iteration of REW

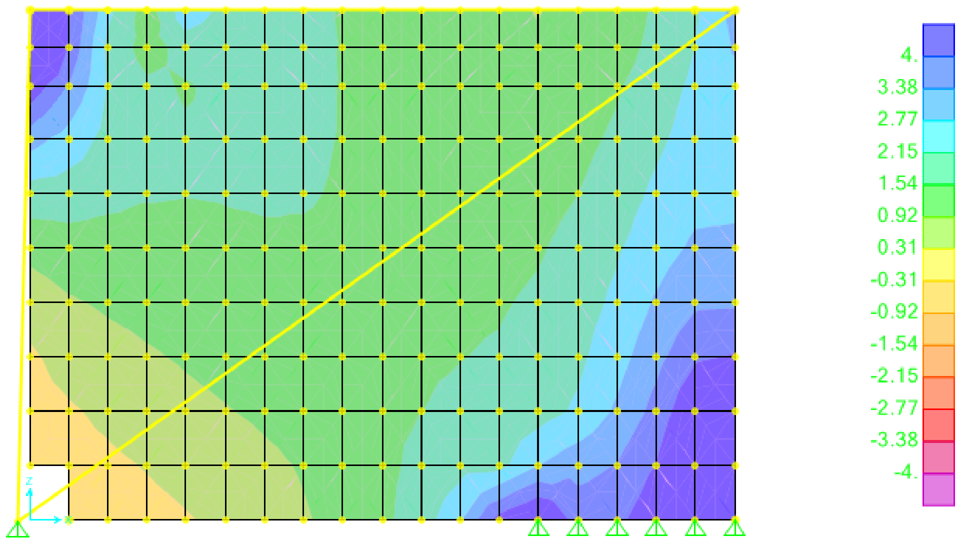


Figure 4.20 SAP2000 stress von-Mises results

The assumed diagonal and vertical compression struts were qualitatively observed on SAP2000 analysis results (Figure 4.20). Compression struts' thickness at the bottom corners were assumed L/3 of the wall length (L). Furthermore, the horizontal components coming from the diagonal compression strut (generated by the vertical rebars) and diagonal rebar are assumed to be approximately equal to each other. The wall is assumed to deform close to a rigid diaphragm in order to calculate the strains in the rebars. All rebars should have areas smaller than the "balanced case" where the compression at the lower end corner of the wall reaches to crushing strength when both of the rebars are yielded. For this reason, the rebar areas are limited to 80% of the rebar areas for "balanced case".

A further analysis of the principal tensile and compressive stresses as well as maximum shear stresses at the lower end corner of RREW are discussed using Mohr's circle approach in Appendix B. The wall strength calculations in the design procedure excluded the positive contribution of the roof and wall weight staying on the safe side.

4.2.1 Reinforcement area selection and wall capacity calculation

In this section, REW capacity was calculated while equalizing REW's strength to reinforcement's strength. REW's compression capacity was calculated by decreasing the compressive strength value by 2 times (safety factor is 2).

σ_1 = Decreased compressive strength of REW

σ_w = Compressive strength of REW

F_1 = Horizontal component of vertical reinforcement bar's strength

F_2 = Horizontal component of diagonal reinforcement bar's strength

C = Capacity of REW

L = Length of REW

H = Height of REW

A1 = Cross-sectional area of reinforcement bars

$\sigma_1 = \sigma_w / 2$ 4.1

$C = t \times L/3 \times \sigma_1$ 4.2

F_1 and F_2 were equalized to determine the diameter difference of the reinforcement bars.

$F_1 = A_1 \times \sigma_a \times (L / H)$ 4.3

$F_2 = A_1 \times \sigma_a \times (L / (H^2 + L^2)^{1/2})$ 4.4

When equations were equalized for a wall with 4 m length (L) and 3 m height (H);

$A_1/A_2 = 3/5$ 4.5

F_1 and F_2 values together were equalized to C which would result the minimum area of reinforcements.

$F_1 + F_2 = C$ 4.6

$((5/3) \times A_1 \times \sigma_a) + (A_1 \times (5/3) \times \sigma_a \times (3/5) = t \times L/3 \times \sigma_1$ 4.7

$A_1 = (\sigma_1 / \sigma_{a)} \times t \times L \times 0.2$ 4.8

According to the equation, A_1 was chosen Ø12 and A_2 was chosen Ø14.

A generalized capacity equation can be driven by using the equation above.

Capacity = $(A_1 \times \sigma_a \times (L / H)) + (A_1 \times (H / (H^2 + L^2)^{1/2}) \times \sigma_a \times (L / ((H^2 + L^2)^{1/2})$

$= (A_1 \times \sigma_a \times (L / H)) + (A_1 \times \sigma_a \times ((H \times L) / (H^2 + L^2))$

$= A_1 \times \sigma_a \times ((L / H) + ((H \times L) / (H^2 + L^2))$ 4.9

4.2.2 Confinement effects

Confinement elements were used to strengthen the bottom corners of the REW where REW has the most stress during the earthquake. Reinforced columns failure behavior is brittle, while confined columns failure behavior is ductile. The aim of using confinement elements is for increasing ductility [29]. Confinement effect to the wall capacity was calculated from Equation 4.1. Besides, confinement elements' size effects also calculated (Figure 4.21) in Equation 4.11.

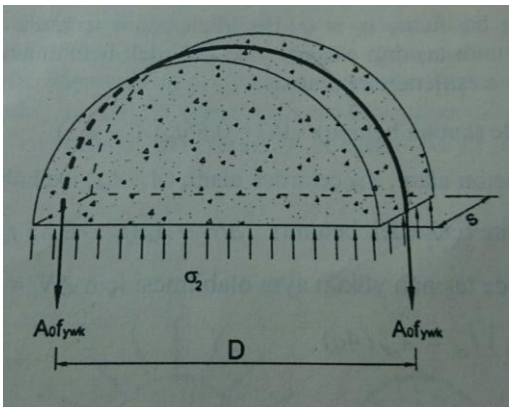


Figure 4.21 Free body diagram

A_0 = Area of confinement element

F_{ywk} = Yield stress of confinement element

D = Diameter of confinement element

σ_2 = Stress inside of confinement element

f_{st} = Total strength together with confinement effect

f_s = Strength of soil

S = Spacing between confinement elements

$\sigma_2 = ((2 \text{ x } A_0) / (D \text{ x } S)) \text{ x } f_{ywk}$ 4.10

A_0 = 12.56 mm^2

D = 90 mm

S = 20 mm

F_{ywk} = 220

σ_2 = 3.1 MPa

$f_{st} = f_s + (4 \text{ x } \sigma_2)$ 4.11

= 4 + 12

= 16.3 MPa

Even though the total strength of the wall during the earthquake was seem like strengthen 4 times in design procedure, in real test REW did not show strength gaining. Soil strength is a lot less than confinement bar's strength. That is why, soil crushed and slipped between the confinement bars before confinement bars reach to their yield strength value. As also mentioned in Ersoy's book in the first peak, there is no strength gain on the column due to confinement elements [29]. However, confinement elements caused REW to fail ductile, which is desirable effect.

CHAPTER 5

DISCUSSION OF RESULTS

In the scope of this thesis, seismic performance of rammed earth walls with reinforcements were studied. In Chapter 1, a brief introduction and literature survey was given. In Chapter 2, a detailed explanation of test procedure and preparations were mentioned. REW sample preparations, REW walls preparations, mixing, ramming and formwork works were described. In Chapter 3, results of the tests was shown. After the results of proctor tests and UCS tests have been given, the wall tests were considered.

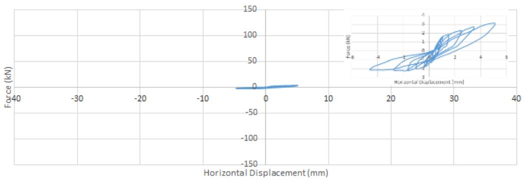
First REW test specimen without any reinforcement failed around 3.15 kN. Later, vertical reinforcements were welded and test was performed again. Wall failed around 48 kN. Vertical reinforcements strengthened wall approximately 15 times. In addition to vertical reinforcements, diagonal reinforcements were welded and wall failed around 140 kN. Vertical and diagonal reinforcements together strengthened the original UrREW wall with a significant improvement around 45 times. Nevertheless, brittle failure occurred even with the horizontal and vertical rebars. The wall formations kept on increasing after both of the rebars are yielded; when the ultimate deformation capacity of the REW is reached, it failed in a brittle manner.

In the second test, vertical reinforcements were placed inside of the wall to protect reinforcement bars from the environmental conditions. However, this caused significant horizontal shrinkage cracks around the embedded internal vertical rebars. Bonding between wall and soil was strong and when wall tended to shrink, this bonding did not allow the wall to deform and cracks were formed at the weakest horizontal points between layers. These cracks expanded during the curing as material lost its water content. Cracks caused overall strength reduction of the wall as expected. When the second test with only embedded vertical rebars was performed, wall failed around 30.3 kN, which is smaller than the 48 kN capacity measured in the first test. Later on, external diagonal reinforcements were welded and test was repeated. During the test, cracks were expanded especially at the edge of wall starting at the corners due to the vertical reinforcements' buckling and bulging movements. Another disadvantage of using reinforcements inside of the wall was when they bended they harmed wall from inside. Wall with inside vertical and outside diagonal reinforcements failed around 120 kN (38 times the UrREW strength), which is smaller than 140 kN capacity of the first wall. Failure occurred as the existing cracks that were occurred during the only vertical rebar testing progressed significantly and expanded to the point of failure. Even though the second wall failure was not as brittle as the first test failure, it was still not as ductile as intended.

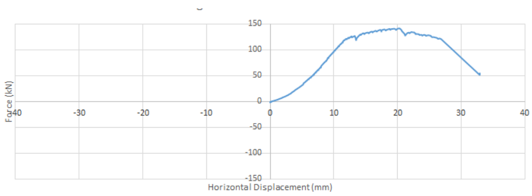
As a remedy, confinement hoop elements were placed inside of the third wall to alleviate brittle compressive failure of REW at ultimate loading stage. The aim of using these confinement elements was to strengthen earth wall at the corners and create a more ductile failure. Horizontal reinforcement elements were also placed at 1/3rd and 2/3rd of the height to keep both sides of the major diagonal crack together. Confinement steel hoops did not cause any shrinkage cracks. However, both embedded lateral and embedded vertical reinforcement elements have seen to cause shrinkage cracks. Therefore, both vertical and horizontal rebars were kept outside and horizontal reinforcement was used together with the confinement steel hoops. Timber members were not used since timber cage based earthen column tests were shown to improve compressive capacity increment lesser than the steel hoops. When the third test was performed, wall failed around 130 kN (41 times the UrREW strength) still less than the first test's 140 kN capacity; however, the failure was not brittle. After the failure, it was still carrying load assuming that it did not collapse.

UrREW, 1st RREW final test, 2nd RREW final test, and 3rd RREW final test results with the same scale for x and y coordinates are shown to compare (Figure 5.1).

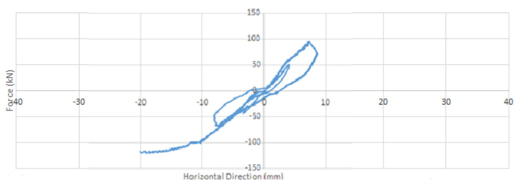
Envelope curves of all tests are shown below (Figure 5.2 (a) UrREW pushover test result, (b) 1st RREW pushover test result, (c) 2nd RREW pushover test result, (d) 3rd RREW pushover test result). UrREW, 1st RREW final test, 2nd RREW final test, and 3rd RREW final test results were also shown in one chart together to compare better (Figure 5.3).



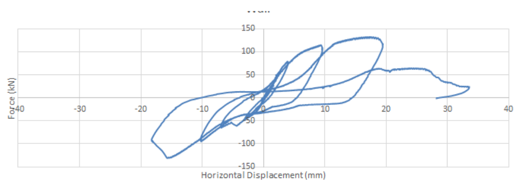
(a)



(b)



(c)



(d)

Figure 5.1 Same scale test results a) UrREW, b) 1st RREW, c) 2rd RREW, d) 3rd RREW

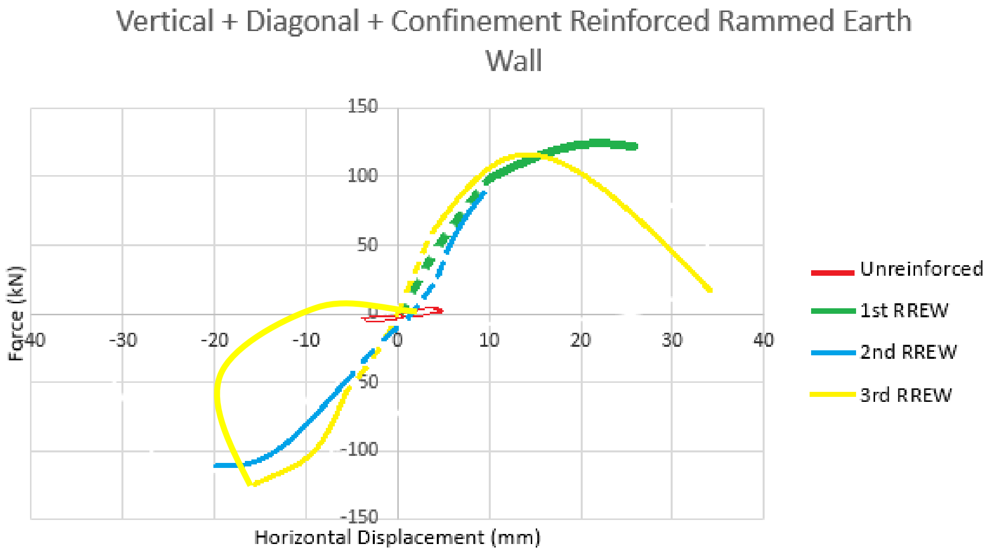


Figure 5.2 Envelope curves of results (idealized)

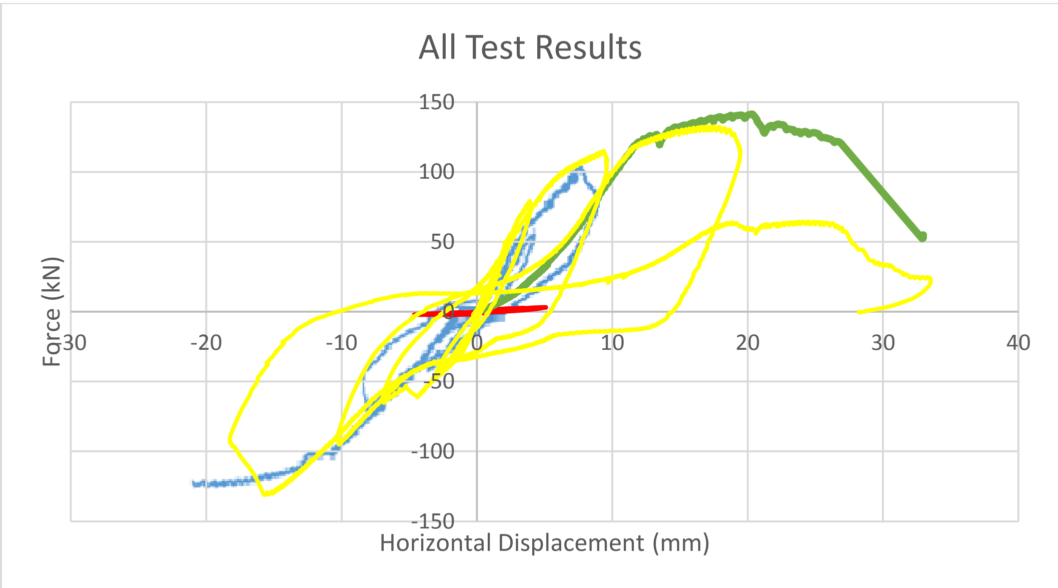


Figure 5.3 All test results together

CHAPTER 6

CONCLUSIONS

The Rammed Earth Walls (REW) are known for their low cost and nature friendly properties that have superior heat and sound insulation capabilities. However, due to their large mass and low tensile strength, REWs may not be suitable for earthquake prone regions. In this study, load bearing capacity of REWs against lateral earthquake forces have been improved using vertical and diagonal rebars while their inherent brittle behavior was also alleviated using tension members and confinement hoops at the compression zone.

This thesis involved different stages to approach the problem of making REW stronger and more ductile. The material tests were targeted using sustainable and renewable-recycled material but excluding cement usage. Steel bars were used for their ductile and large tensile force capability while lime and fly ash were used to improve the strength of REW. Additional steel hoops gave RE material larger strength and more ductile properties. Usage of timber was also investigated but kept out of scope since steel gave superior results.

The initial studies on material included unconfined compression strength tests using mix compositions and optimum moisture content studies for best compaction. Three different strengthening scenarios were tested using three different lab walls of 0.65mx0.9mx0.11m size. Each test specimen was tested more than once for different levels of reinforcement. Additional work was performed to simulate the behavior of tested walls analytically using nonlinear analysis in SAP2000. The ultimate strength results had good agreement with the tests except for the deformation capability. The deformation capacity of the tests was much larger than the calculated values; therefore, additional hand calculations were made. Since the hand calculations and analysis results agree with each other, it was concluded that the test had unforeseen deformation characteristics such as flexible connections and initial deviation from theoretically straight geometry giving extra deformation capabilities. The test setup might have moved on the ground but two LVDTs were used at the bottom and top of the wall to measure actual wall deformations. When the vertical displacement measurement using LVDTs are compared against the analytical values, it was observed that the test walls had a slight rocking motion which would generate a rigid body rotation, which would create no lateral movement at the base but relatively large horizontal movement at the top. It is also likely that such rocking effect would occur in real houses and the test results were found to be quite useful for that aspect as well.

All of the test, analyses, and hand calculation results indicate that the walls have been strengthened more than 40 times their initial strength, which is enough to support first-degree earthquake zone forces (i.e. A_o=0.4) without major failure and with a large factor of safety (FS) larger than 4 (Appendix A).

The vertical and diagonal rebars placed on the outer sides of REW not only provide strength and ductility in the in-plane direction but they are also expected to have useful effects on the overturning of walls in their out-of-plane directions. Although this effect has not been tested within the scope of this thesis, additional out-of-plane bending capacity increase has been considered as an additional reserved capacity.

The rebars were planned to be placed inside the REWs to protect them from external effects; however, excessive shrinkage caused a general problem of cracking. Therefore, the rebars were kept outside with better performance. Nevertheless, the protection of rebars may be necessary and can be achieved by plastering over rebars using similar mixture of the walls. Therefore, it would protect rebars from excessive heat variations (elongation during summer and contraction during winter) as well as humidity changes and splashes during rain and snow. Exterior surfaces can be protected from water by using thin tiles that are placed over the surface using thin clay, lime, and fly ash plaster especially at the ground level. As a common good practice, proper distance for roof eaves overhang is necessary to protect earthen walls from water.

The corners of REW has utmost importance as many masonry structures start to fail at the building corners. The horizontal reinforcement placed inside the REWs at 1/3 and 2/3 height should be continuous and extended to the neighboring perpendicular walls to maintain integrity of wall corners.

REW should be protected from frost-thawing. Besides, heavily rain also can be dangerous for the unity of REW. In addition to REW, reinforcement bars should also be protected from corrosion. That is why, isolation and covering materials can be used. Decoration stones can be placed outer surface of the walls to protect REW from weather conditions. Also, decoration stones can provide improved esthetics to earthen structure.

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APPENDIX A

Hand calculations;

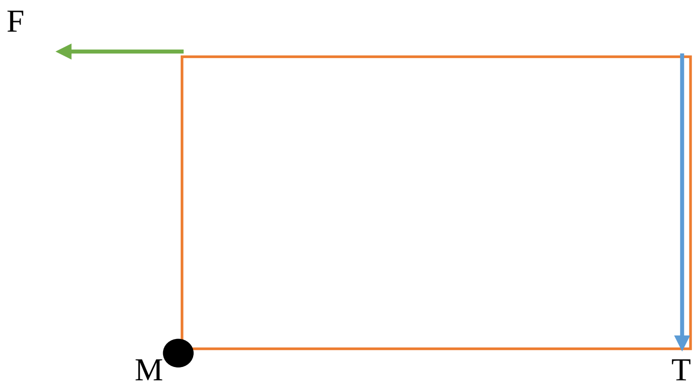


Figure A - 1 Moment equation by considering vertical reinforcement

The first very common failure occurs at the bottom horizontally. After the crack, wall tends to turn over. A vertical reinforcement bar welded to the side to strengthen the wall. In this calculation, a simple moment calculation was followed. The maximum force was calculated according to the yield stress of the vertical reinforcement bar and moment equation was driven to point M (Figure A - 1).

The diameter of reinforcement: 12 mm

The yield stress of reinforcement: 275 MPa

Area of reinforcement: $\pi \times 6^2 = 113.09\ mm^2$

Force = 113.09 x 275 = 31.1 kN

T (Both side) = 31.1 x 2 = 62.2 kN

Moment equation around point M = F x 650 = T x 900 A.1

F = (62.2 x 900) / 650 = 86.12 kN A.2

According to this equation, with vertical reinforcement wall fail when force at the top corner reaches 86.12 k

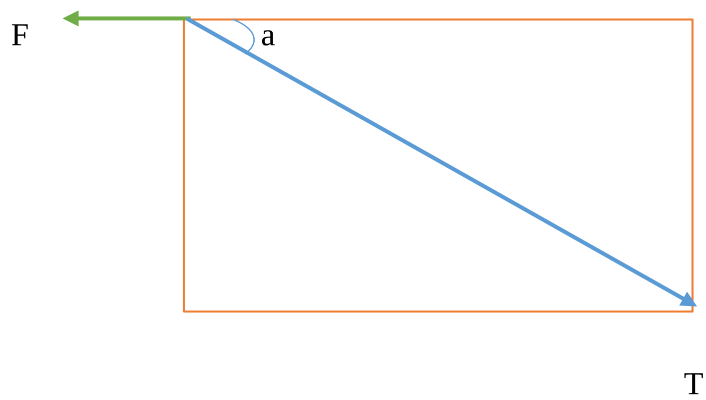


Figure A - 2 Calculation by considering diagonal reinforcement's components

The other very common failure occurs diagonally. A diagonal reinforcement was welded to strengthen the wall in diagonal direction. The maximum force was calculated according to the yield stress of the diagonal reinforcement bar. Calculated force's horizontal component calculated by multiplying Cos (37) (Figure A - 2).

The diameter of reinforcement: 14 mm

The yield stress of reinforcement: 275 MPa

Angle (a) = 37°

Area of reinforcement: π x 7^2 = 153.93 mm^2

Force = 153.93 x 275 = 42.33 kN

Horizontal T = Cos 37 x 42.33 = 34.24

T (Both side) = 34.24 x 2 = 68.49 kN

According to this equation, with vertical reinforcement wall fail when force at the top corner reaches 68.49 kN

Vertical and diagonal reinforcements together have;

$$86.12 + 68.49 = 154.60 \text{ kN} \qquad \text{A.3}$$

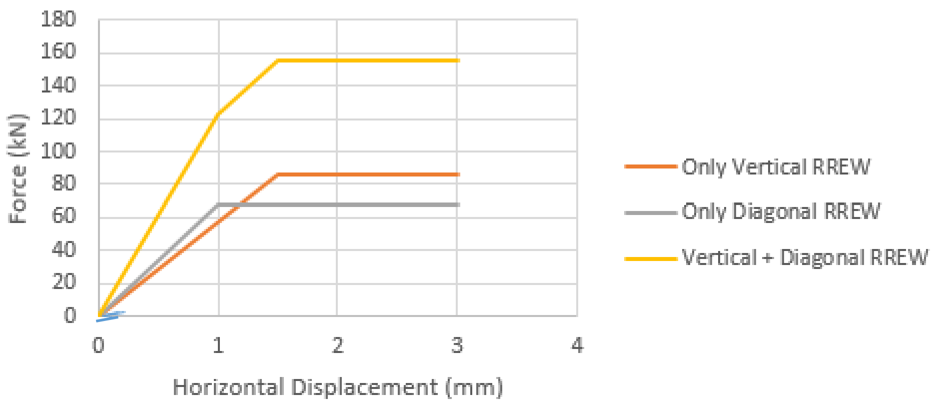
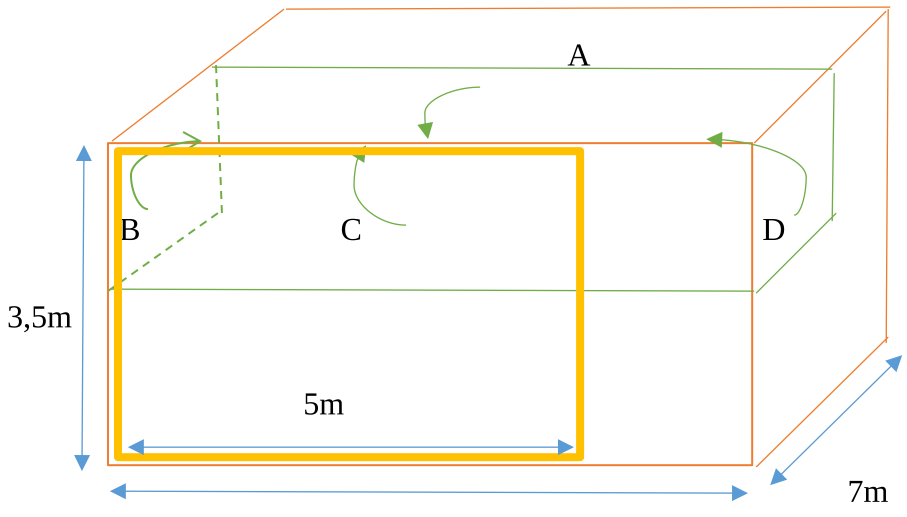


Figure A - 3 Wall failure predictions according to reinforcements

Seismic analysis for seismic zone 1;



7 m

Figure A - 4 Seismic analysis design

Earthquake load calculation was calculated according to the Turkish earthquake code "Deprem Bölgelerinde Yapılacak Binalar Hakkındaki Yönetmelik" (DBYBHY'07)

ST (characteristic spectrum period) =2.5

Ao (ground acceleration coefficient in zone 1) = 0.4

I (building importance coefficient) =1

R (load bearing system behavior coefficient) =1

A, B, C, and D are the areas of the wall's which was indicated. B and D are the ¼ areas of the wall at sides. A is the ½ area of the rood. C is the ½ area of the wall. These areas' weights were calculated and applied to the top of the wall. The total weight was used in the equation of Turkish earthquake code. Later on, capacity of the wall was calculated by using the calculation in section 4.2. These results were compared and safety factor was calculated.

Thickness of wall (t_w) = 0.5m

Height of wall (H) = 3.5m

Length of wall (L) = 7m

Length of REW (L_w) = 5m

Thickness of roof (t_f) = 0.18m

Density of wall (d_w) = 20 kN/m^3

Dead load of roof (d_l) = 25 kN/m^3

Total Weight = Weight of area of B + Weight of area of D + Weight of area of A

Total Weight = (t_w x (H / 2) x (L / 2) x d_w) + (t_w x (H / 2) x (L / 2) x d_w) + (L x L x t_f x d_l) / 2 A.4

= ((0.5 x (3.5 / 2) x 7 x 2) / 2) + ((0.5 x (3.5 / 2) x 7 x 2) / 2) + ((7 x 7 x 0.18 x 2.5) / 2) A.5

= 232 kN

Horizontal force = Total weight x ST x A_o x I x R A.6

Horizontal force = 23.2 x 2.5 x 1 x 0.4 / 1= 232 kN A.7

Capacity was calculated for 5m x 3.5m x 0.5m wall from using the calculation at section 4.2:

A1= (2/420) x (5000) x (500) x 0.2 = 2380 mm2 A.8

Cw (capacity of wall) = 2380 x 420 x (5 / 3.5 + (5 x 3 / (5 x 5 + (3.5 x 3.5))))

= 2380 x 420 x 1.83

= 1830 kN A.9

Wall's self-weight and roof dead load also cause a moment effect to opposite direction:

R_w (weight of roof) = (L x L) / 2 x t_f x d_l A.10

R_w = (7 x 7 x 0.18 x 25) / 2

R_w = 110 kN

W_w (weight of wall) /2 = t_w x L_w x H x d_w A.11

W_w /2 = (0.5 x 5 x 3.5 x 17) / 2

$W_w / 2 = 150$ kN

C (total capacity) $= C_w + ((R_w + W_w) \times ((L_w / 2) / H)$ A.12

$= 1830 + ((110 + 150) \times ((5 / 2) / 3.5)$

$= 2015$ kN

FS (factor of safety) = 8.68 A.13

A three-roomed house was designed as an earthquake resistant house, which is built with REW technique. Walls are 4m x 3m x 0.5m. Walls and roof slab were assumed rigid and in-plane behavior was considered such as RC slab (or CLT wooden slab). Calculation was made by equations below. All building's weight is the weight of all REWs and the roof/slab. The contribution of walls in their out-of-plane direction is also taken as horizontal inertial forces on the walls in their own strong axis. The house is analyzed in x and y directions.

V_{rew} = Volume of one REW

d_{rew} = Density of REW

W_{rew} = Weight of REW

W_{trew} = Total weight of REW

H = Height of REW

L = Length of REW

t = Thickness of REW

$V_{rew} = H \times L \times t$ A.14

$= 4m \times 3m \times 0.5m$

$= 6\ m^3$

$d_{rew} = 17\ kN/m^3$

$W_{rew} = V_{rew} \times d_{rew}$ A.15

$= 6 \times 17$

$= 102\ kN$

There is 14 RREWs in the building;

$W_{trew} = 14 \times W_{rew}$ A.16

$= 14 \times 102 = 1428\ kN$

Horizontal force = Total weight x ST x A_o x I x R A.17

Horizontal force = 142.8 x 2.5 x 1 x 0.4 / 1 = 1428 kN A.18

Whole building's weight is 1428 kN. However, only the RREWs on the direction of earthquake will resist to the horizontal force. There is 8 RREWs in the direction of earthquake. Per RREW has 178.5 kN horizontal load due to earthquake. Capacity of one RREW is 1464 kN. There is 8 RREW for to resist earthquake force. The capacity of house in total is 11712 kN. Which has 8.2 factor of safety.

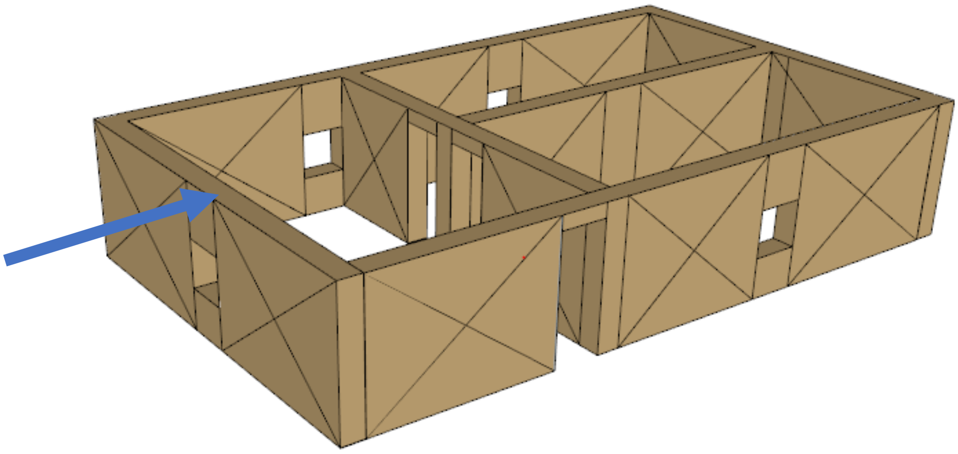


Figure A - 5 Earthquake force parallel to house

Whole building's weight is 1428 kN. However, only the RREWs on the direction of earthquake will resist to the horizontal force. There is 8 RREWs in the direction of earthquake. Per RREW has 178.5 kN horizontal load due to earthquake. Capacity of one RREW is 1464 kN. There is 8 RREW for to resist earthquake force. The capacity of house in total is 8784 kN. Which has 6.15 factor of safety.

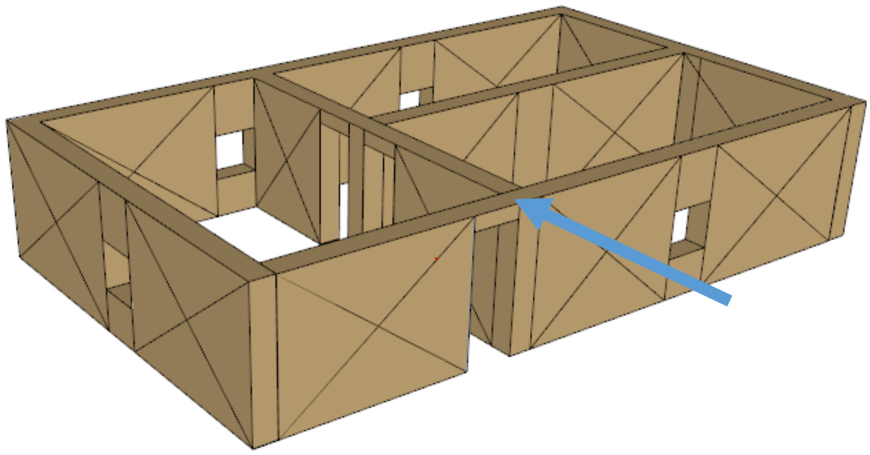


Figure A - 6 Earthquake force perpendicular to house

APPENDIX B

Principal tensile and compressive stresses as well as maximum shear stresses at the lower end corner of RREW are calculated using Mohr's circle approach. The wall strength calculations in the design procedure excluded the positive contribution of the roof and wall weight staying on the safe side.

Stresses due to horizontal load have shown in figure (Figure B - 1). Diagonal and vertical reinforcement were shown by orange arrows, horizontal load was shown by blue arrow, and diagonal and vertical reinforcement bars were shown by black lines

Figure B - 1 Compression struts

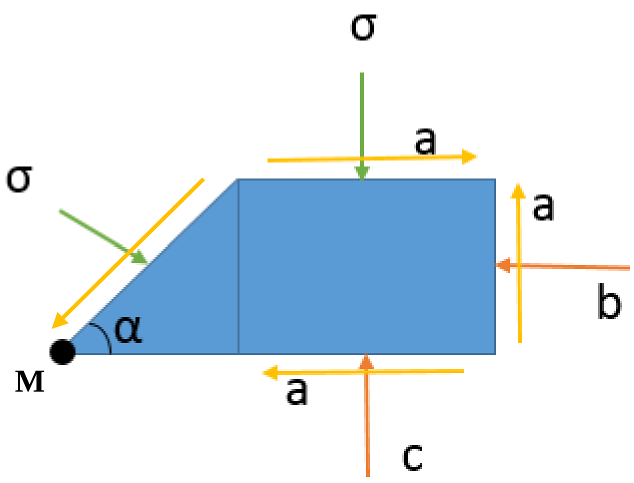


Figure B - 2 Stresses

Vertical and diagonal acting stresses shown in figure (Figure B - 2) as σ. σ was assumed as 4 MPa. REW's sizes were considered 4 m x 3 m x 0.5 m. That is why, angle α was calculated as 36.86º (Arc tan (3/4)). Three equations were driven to for three unknown variables. Shear force on the inclined face was ignored. Horizontal forces equation, vertical forces equation and moment at point M were calculated in Excel by using solver analysis. According to results,

a = 0.95 MPa

b = 4 MPa

c = 6.62 MPa

Mohr circle was plotted by using a, b, and c values.

Maximum principle stress = -3.692 MPa

Minimum principle stress = -6.928 MPa

Maximum shear stress = 1.618 MPa

Minimum shear stress = -1.618 MPa

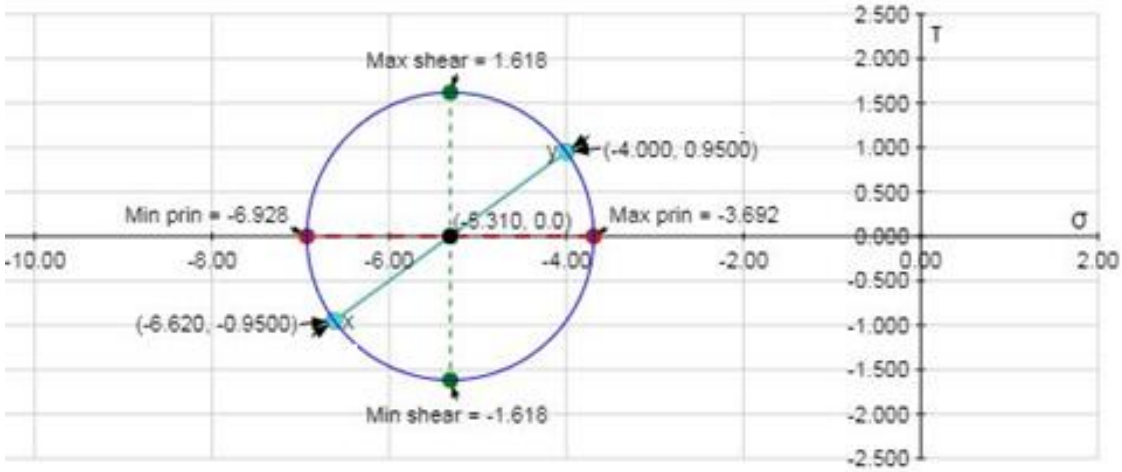


Figure 2.3 Mohr circle results