Performance Assessment of Representative Unreinforced Masonry Buildings in Addis Ababa as per ES EN 1996 and ES EN 1998

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This is to certify that the thesis prepared by Adam Getachew, entitled*: Performance Assessment of Representative Unreinforced Masonry Buildings in Addis Ababa as per ES EN 1996 and ES EN 1998* and submitted in partial fulfillment of the requirements for the degree of Master of Sciences in Civil Engineering (Structures) complies with the regulations of the University and meets the accepted standards with respect to originality and quality.

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ABSTRACT

Unreinforced Masonry (URM) buildings constructed earlier than 1930 are the largest stoke of buildings made in those years in many parts of the world. Such buildings are also common in older parts of Addis Ababa, which includes; many private residential, buildings, governmental offices, schools, hospitals, spiritual places, and market centers; these buildings were constructed in the absence of mandatory earthquake design requirements and unquestionably recognized as the type of construction most vulnerable to earthquakes.

According to previous code EBCS 8 1995 seismic zoning, Addis Ababa was categorized under Peak Ground Acceleration of 0.05g. However, the revised Ethiopian Standard codes ES EN 1998:2015 specified a value 0.1g. This increased estimate indicates that Addis Ababa is no longer in low seismic zone and there is related posed seismic hazard.

The main goal of this study is to develop a simplified performance assessment procedure to URM buildings so that practitioners can be able to estimate quicker the status of similar buildings and ultimately to propose valuable intervention works to avert posed seismic hazard.

In particular simplified hand calculation approach which includes defining material properties from non destructive tests and visual data have been used to analyze sample building. The commercially available, finite element software tool, SAP2000, is used. So that to model a case-study-building. Results found both from the hand calculation and 3D modeling are compared which can be seen in respective parts of this thesis; and finally the performance level status of the sample building is evaluated.

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DECLARATION

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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NOTATION AND ABBRIVIATION

E	short term secant modulus of elasticity of masonry;
E	long-term long term modulus of elasticity of masonry;
f_b	normalized mean compressive strength of a masonry unit;
f_{cvk}	characteristic shear strength of concrete infill;
f_d	design compressive strength of masonry in the direction being considered;
f_k	characteristic compressive strength of masonry;
f_m	compressive strength of masonry mortar;
f_{vd}	design shear strength of masonry;
f_{vk}	characteristic shear strength of masonry;
f_{vko}	characteristic initial shear strength of masonry, under zero compressive stress;
f_{vlt}	limit to the value of f_{vk};
f_{xd}	design flexural strength appropriate to the plane of bending;
f_{xd1}	design flexural strength of masonry having the plane of failure parallel to the bed joints;
$f_{xd1,app}$	apparent design flexural strength of masonry having the plane of failure parallel to the bed joints;
f_{xd2}	design flexural strength of masonry having the plane of failure perpendicular to the bed joints;
$f_{xd2\ app}$	apparent design flexural strength of masonry having the plane of failure perpendicular to the bed joints;
f_{xk2}	characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints;
CIS	Corrugated Iron Sheet
PGA	Peak Ground Acceleration
URM	Unreinforced Masonry.

PART 1 INTRODUCTION

1.1 GENERAL

Masonry structures are one of the most ancient types of structure used as shelter for human kind. Mainly due to the abundant availability of stone or brick material and durability of masonry structures, its use in most part of the world as major building structures was a usual observable fact. Most masonry buildings constructed before year 1930 are not reinforced with steel and do not have reinforced concrete frames. Due to the absence of reinforced frame system in such buildings, they referred as Unreinforced Masonry (URM) buildings. Most URM buildings elsewhere in the world are considered to have a historical heritage value; ordinary URM buildings also exists currently giving normal service in many parts of the world including Addis Ababa city.

Existing URM Buildings are common in many older parts of Addis Ababa. There exist several but also diverse URM buildings constructed by experts from local but mainly from different European and Asian countries in late 19th century and in the beginning of 20th century. These buildings were obviously designed and constructed without considering seismic actions; thus are highly vulnerable to serious seismic hazard due to little or no resistance to lateral seismic actions.

URM buildings have always been seriously damaged even to lower seismic action occurred to them. These types of structures usually have insufficient strength to resist lateral earthquake forces; lacks the ability to adequately dissipate energy and exploit ductility. Due to high brittle properties of URM buildings, related mode of failure when laterally excited by seismic action is complete collapse, which resulted, in many parts of the world, with countless dramatic horror, in most past earthquake occurrences.

Other than the major source seismic hazard, which Natural earthquake mainly due to tectonic movement and volcanic eruption; there is compelling evidence which confirms that oil extraction is also linked with human induced Earthquakes. According to report compiled in [22], the seismicity rate in 2013 was 70 times greater than the back ground seismicity rate observed in Oklahoma prior to 2008; indicates that human activities related to oil and gas extraction, are beginning to play a significant role in triggering earthquakes in the surrounding areas near oil exploration taking place.

Addis Ababa is located in the middle of South Sudan where oil extraction is currently in progress since 1999; and rift valley, which are two potential sources of seismic action. Although the link between oil extraction in South Sudan and the increased forecast of earthquake in Addis Ababa is a topic for Earthquake engineers, the combined human induced and natural tectonic movements can be considered as primary source which induced seismicity exhibited in Addis Ababa.

The potential seismic hazard from URM buildings in Addis Ababa is possibly a great concern to structural engineers. In order to avoid or to minimize possible tragedy from seismic hazards first it is required to check past history of seismicity events in the area of concern. In this regard, according revised earthquake zoning stipulated in ES EN 1998 [9], Annex D, Addis Ababa is expecting a higher magnitude of seismic action as compared with previously understood EBCS 8, 1995 [18], which was Peak Ground Acceleration (PGA) of 0.05g to 0.1g. Thus, the increased seismic risk is evidently there. The next step could be to analyze these structures and find out the performance level under revised seismic action, which is the main task addressed in this thesis.

1.2 MOTIVATION

Among extremely devastating and terrifying natural hazards earthquake is perhaps the major and merciless phenomena. Due to its unpredictable character, swift but large catastrophe on human lives and huge economic loss at the events of such hazards at populated parts of the world there is no way to escape when it occurs other than to be prepared and resist or face the disaster.

The death tall acknowledged in 2015 Nepal [28] is more than 8,030, in 2011 Japan [14] more than 18,000, in 2010 Haiti [33] anywhere between 220000 and 316,000. In 2008 China [32] Sichuan more than 60,000, in 2005 Pakistan [31] Kashmir more than 80,000, and in 2004 off shore Indonesia including tsunami [30], close to 230,000 are perfect evidence for the ruthlessness of this natural hazard to humankind.

Given the double fold increase in seismic action forecast in the revised code ES EN 1998:2015, [9], and with the exiting URM buildings continued giving service; a defined performance assessment tool is urgent necessity due to the fact that the unpredictable nature of Earthquakes. It is also necessary to recommend relevant structural upgrading for URM buildings in Addis Ababa; and ultimately to protect the public from such horrific disaster before it is too late.

1.3 REVIEW OF UNREINFORCED MASONRY BUILDINGS IN ADDIS ABABA

This review of URM buildings in Addis Ababa will attempt to cover topics about the existing URM buildings how they were built, by whom, when and their status as regards to seismic action.

In Addis Ababa the construction of modern URM buildings has started somewhere in 1890 according to Abnet [1]. Agency for Governmental Housing of Addis Ababa City is currently carrying out building inventory. The agency currently has no defined information regarding the number and material used regarding URM buildings in the city.

A summarized database output from Culture and Tourism Addis Wubet, [2], had explained better information regarding URM buildings. Due to a relation with heritage values of these structures, it provided more details about these buildings such as construction time, picture, and material in some instances.

On a paper compiled by Abnet, [1], it is stated that how modern buildings construction started in Addis Ababa as follows “Right after Addis Ababa founded by Empress Taitu Betul in 1886 lots of traditional houses were built between the main palace near Entoto and Filwuha (Finfine area). The new settlement by the time was a mix mostly tents and temporary dwellings but later intermingled with stone buildings, grew rapidly in size and importance. The most important buildings among these buildings were built under the supervision of Swiss Advisor Alfred Ilg of King Menilik II. “

Selam, [25] in her thesis stated that the construction of Old URM buildings in Addis Ababa are influenced by foreign nationals like Indians, Arabs, Greek, Italian, German, French architect who have resided here from time to time for many different reasons. To conclude these historical buildings give the city unique character (especially residence of former dignitary) that no other countries have. Before or near 1920s, many of URM buildings were built in city of Addis Ababa. Among the existing historic buildings are three story buildings Muse Minas Kerbekian Residence 1915, Besmelian (Elias) Residence 1915, two story Bete Sayda or Yekatit hospital 1924, Yohanis Wade (Wole) / Trinity College 1910, and one story buildings Aderash Lij Iyassu Goethe Institute 1910, Bank of Abyssinia 1907.

Most exiting URM buildings in Addis Ababa are built from stones or brick walls, and timber suspended floor system. A dressed stone is laid on other stone with a mud or lime mortar between them then walls are formed while door and window openings made using with arch type lintels and solid stone beam lintels.

Roof trusses and purlins are made of timber structures covered with corrugated sheet iron. Most doors and windows are made of timber and some with steel. To transfer heavier loads from floor to walls timber beams are used. In order to increase the section capacity of such timber beams sometime they have used two or four logs of timber beams used to make one stronger beam. Timber beams allowed just to sit on the wall without any significant connection with URM wall structures.

Most of the first story wall thicknesses are 50 to70 cm, usually made of dressed stone or rubble stone masonry.

For two-story buildings, it will have stone masonry for 1st story and brick masonry for 2nd story. Although not much, there are also three story brick URM buildings. The most frequent type of masonry buildings especially in Arada Sub-city are those with two story dressed stone. The other important features of these buildings are how the masonry walls laid with a technique to controlling crack. As can be seen from Muse Minas Fig.4 residence building wall and other pictures from partially demolished walls it is evident that an attempt is made to control the diagonal crack by using timber lacing.

This method is recognized as timber lacing see Fig. 4. It is self evident that Timber Laced URM buildings perform better due to the elastic property of timber than without it.

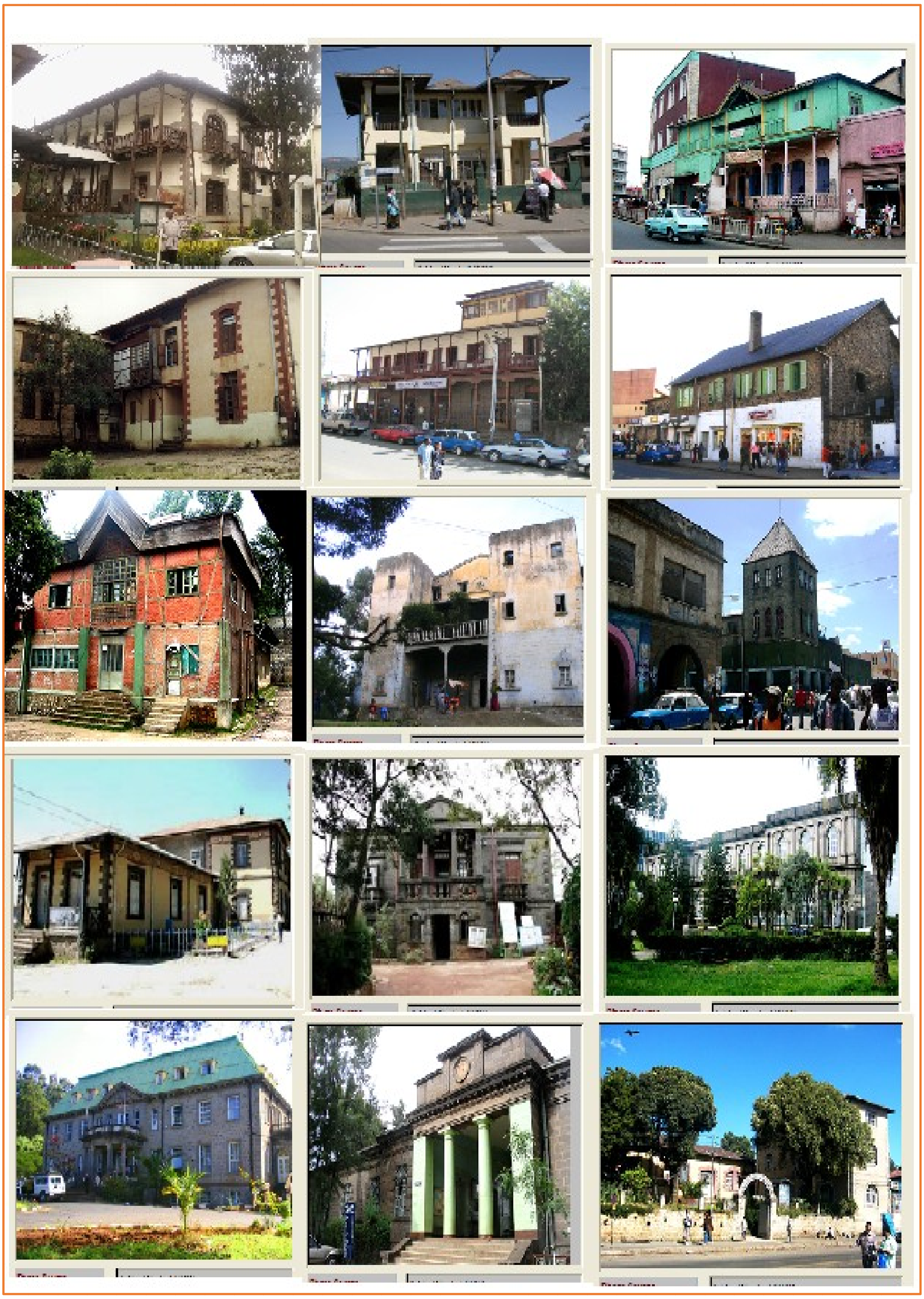

Figure 1: Existing URM buildings in Addis Ababa (Addis Ababa heritage database)

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Figure 2: Two story, Bete Sayda building in Yekatit 12 Hospital built in 1924 G.C

Figure 3: Loosely supported connection of timber beam with stone column

Figure 4: Timber laced building existing and demolished

1.4 BACKGROUND AND PROBLEM STATEMENT

Every day, regions of high seismicity experience many small earthquakes. However, structural damage does not usually occur until the magnitude approaches 5.0M in Richter Scale [29].

Since damage can mean anything from minor cracks to total collapse, categories of damage have been developed as shown in Table 1.

Table 1: Categories of Structural Damage

No.	Damage state	Functionality	Repairs required	Expected time to be ready for service
(1)	None (pre-yield)	No loss	None	None
(2)	Minor/slight	Slight loss	Inspect, adjust, patch	<3 days
(3)	Moderate	Some loss	Repair components	<3 weeks
(4)	Major/extensive	Considerable loss	Rebuild components	<3 months
(5)	Complete/collapse	Total loss	Rebuild structure	>3 months

These levels of damage give engineers a choice for the performance of their structure during earthquakes. Most engineered structures are designed only to prevent collapse. This is not only to save money, but also because as a structure becomes stronger it attracts larger forces.

Thus, most structures are designed to have sufficient ductility to survive an earthquake. This means that elements will yield and deform but they will be strong in shear and continue to support their load during and after the earthquake. As shown in Table 1, the time that is required to repair damaged structures is an important parameter that weighs heavily on the decision making process. When a structure must be quickly repaired or must remain in service, a different damage state should be chosen.

During large earthquakes the ground is jerked back and forth, causing damage to the element whose capacity is furthest below the earthquake demand. In Figure 5 it is shown that the cause may be the supporting soil, the foundation, weak flexural or shear elements, or secondary hazards such as surface faulting or failure of a nearby structure. Damage also frequently occurs due to the failure of connections, from large torsional moments, from tension and compression, buckling, pounding, etc.

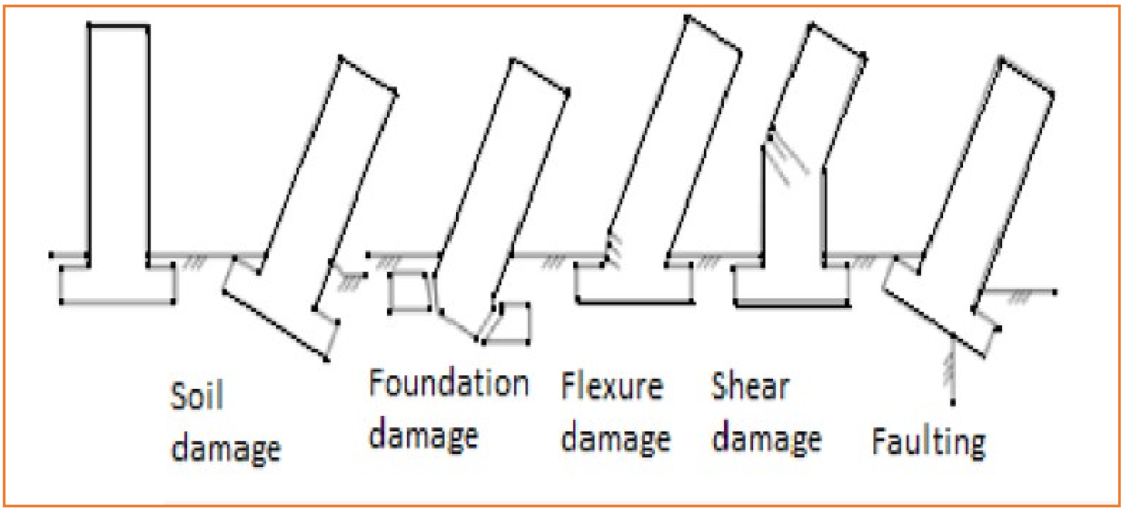

Figure 5: Common types of damage during large earthquakes.

A seismic data compiled by GSHAP which stands for Global Seismic Hazard Assessment Program and as cited by Asrat [6], a more detailed zoning is prepared which resulted in that PGA values much higher than zonings appeared in EBCS 8-1995 [18].

For instance one can compare the values for Addis Ababa which was 0.05g on EBCS 8-1995 [17], with a value estimated in GSHAP, cited by Asrat [6], which became ~0.16g this clearly indicates that the values have increased significantly which suggests that seismic hazard should be considered in detail especially for URM buildings in such moderately seismic areas.

As noted in the conclusion of Asrat [6], the most vulnerable structures are those with a fundamental period up to around 1 second that encompass most commonly constructed buildings. Since most URM buildings in Addis Ababa are from one to three story the effect of increased seismic loading is mainly a concern to the performance of these buildings.

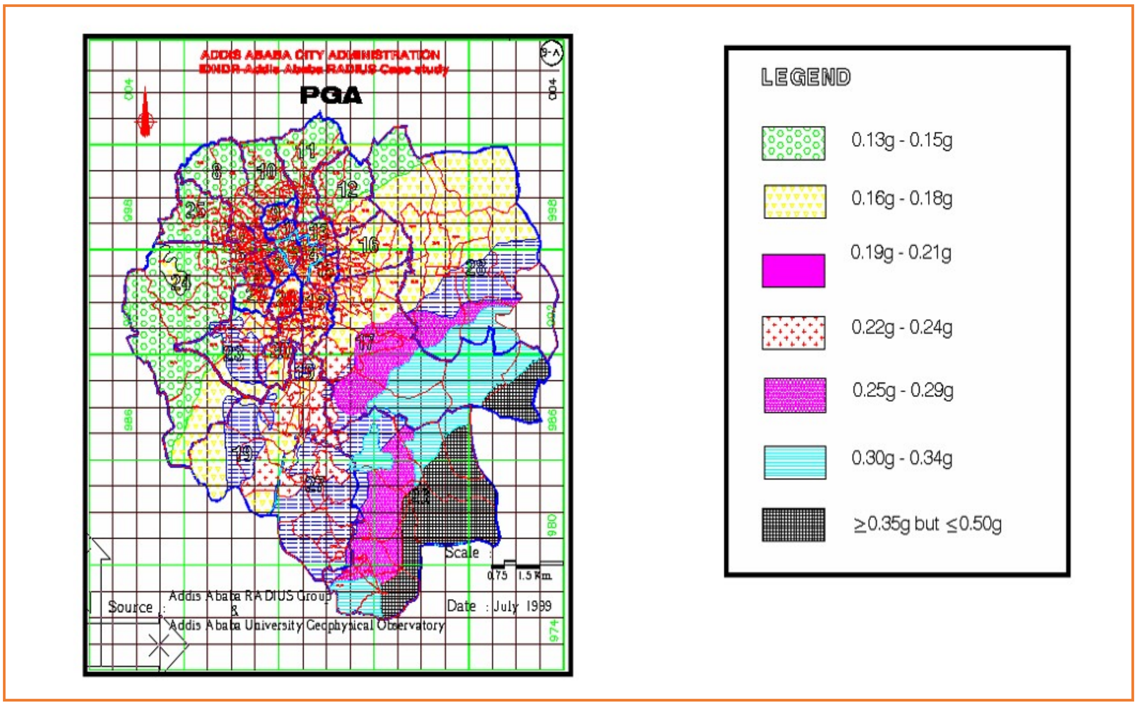

Figure 6: Seismic zoning of Addis Ababa as per RADIUS Project (1999).

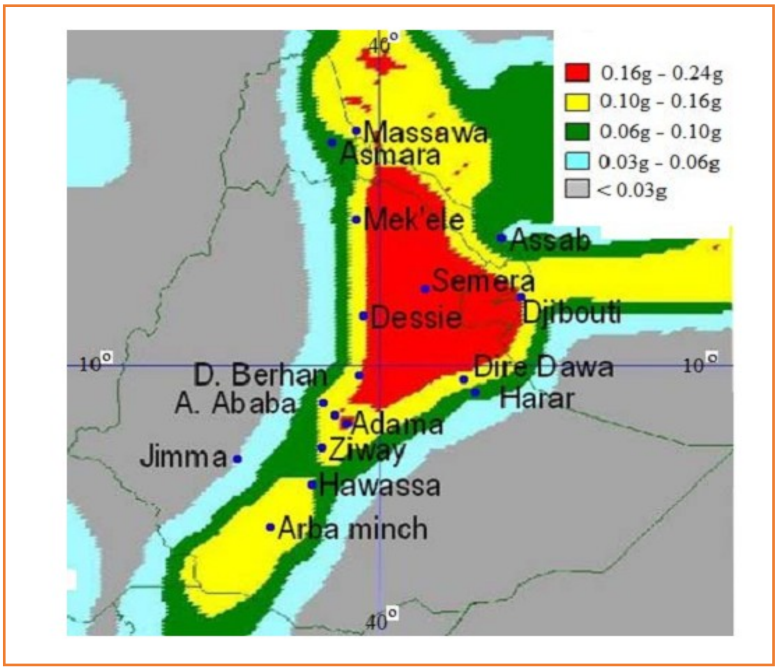

Figure 7: The seismic hazard map of Ethiopia based on the GSHAP data for a return period of 475 years

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According to EBCS 8-1995, pp110 [18], the allowed story above ground is one. Since, now the PGA of Addis Ababa is close to or greater than 0.1g it can be deducted that its zone to be zone IV of EBCS 8-1995 [18], zone coding. Taking in to consideration the increase in PGA of Addis Ababa, one can conclude the exiting URM buildings in Addis Ababa which have greater than one story are not safe.

In the revised code ES EN 1998 [9], limitation on the number of story for URM building is dependent on minimum sum of cross sections areas of horizontal shear walls in each direction, as percentage of the total floor area per storey which relay on further investigation on particular buildings. Thus it can be deducted that due to existence of several two and three story URM buildings in Addis Ababa both historical and non-historical buildings which are giving service currently urgent preliminary performance assessment needs to be carried out to according to the requirements stipulated in the revised ES EN 1998 [9].

It is evident that the revised PGA of Addis Ababa is much higher than understood before 20 years when EBCS 8-1995 published. With the available investigation more specifically according to the revised Earthquake zoning of ES EN-1998:2015 doubled (0.10g) PGA is expected.

1.5 OBJECTIVE

The main objectives of this study are two: The first is to investigate simple approaches and methods of performance assessment which are friendly for practitioners in design offices; and make sound judgment related to the attained performance level depending on the importance of the building under question. The second is based on the easiest approach to evaluate the performance level of existing URM buildings in Addis Ababa.

The first objective is accomplished by discussing selected works from literature and combining with the revised, Ethiopian Codes specifically ES EN 1998: [9], guide lines a defined procedure is prepared. Since evaluating existing structures is less emphasized in currently existing codes, guidelines from FEMA 356 [4] and Pantazoupoulou [27] are referred when needed.

The second objective is achieved by applying detailed analysis procedures outlined in literature review. Specifically, hand calculation approach and 3D Finite Element Analysis Software tool, SAP2000 v.18 analysis are applied.

Sample performance assessment procedures is applied on existing Two Story Building named Bete Sayda in Yekatit Asrahulet Hospital in Addis Ababa which is displayed in Fig. 1 to show the detailed process of assessment URM buildings including results.

1.6 MATERIAL AND METHODS

Since this study is analytical the materials used are the followings: literature reviews are covered on performance assessment methods for URM buildings. Suitable method to analyze URM buildings in Addis Ababa is investigated and sorted for application. Relevant parts of the latest Ethiopian Standards: such as ES EN 1998 [9], for seismic design and analysis are referred and arranged. .

Sample building has been selected which can show the basic behavior of URM building in Addis Ababa and various actual data on this building have been collected. While collecting data detailed information about the representative building was secured; mainly, detailed dimension; year of construction, information on how the building was built and observed material properties on the building have been collected.

Since the major concern of this thesis work is to present simple methods of analysis and decide the level of performance of any URM buildings quicker; homogenous material property of masonry wall as per ES EN 1996 [8], was used. Simple approaches have been investigated and presented with sample calculations.

Finite Element Method of analysis was carried out to approximated shape of the sample building. In order to simplify the complexity of analysis and to reduce the analysis time commercially available software tools specifically Finite Element Analysis Software tool, SAP2000 v.18 is used.

During the analysis stage proper modeling of URM building was utilized which have account the material non-linearity by assigning equivalent homogenous material and shape non linearity by using approximate irregular openings for doors and windows.

It showed both the in and out of plane stresses and deflections at selected points in the model. Results in-terms of displacement at roof level have been presented with the corresponding performance level.

1.7 LIMITATION AND SCOPE OF THE STUDY

Although URM buildings exhibit vast range of material and/or shape non linearity even in one building; the modeling will be dependent on visual identification but generalized and simplified judgment have been made to decide the material property. Testing samples could be more ideal to decide the material property however due the budget limitation the scope of this study does not include material testing except few non-destructive tests such as Schmidt hammer, and Scratch index tests.

Moreover, the performance analysis methods used do not focus on URM buildings constructed in years later than 1935 due to reinforced concrete members presence in buildings constructed in recent years.

On the other hand ES EN 1996 [8] and ES EN 1998 [9], guide lines has been used for detailed analysis on the selected buildings. Some structural members may show deterioration due to age or weather conditions however these factors will not be emphasized.

While performing seismic analysis the soil type will be considered however detailed soil structure interaction analysis will not be entertained. Foundation soil failure will not be investigated and it is assumed that foundation soil is safe against the expected bearing capacity. Simplified, Linear Equivalent Static Seismic Analysis is used. Non linear analysis is not performed.

1.8 ORGANIZATION OF THE STUDY

The remaining major parts of this thesis are discussed here. The upcoming remaining parts are five. In Part 2 of this thesis a literature review will be covered. The part will be devoted in discussing the background of the methods on how to perform a performance assessment on URM buildings.

In the next part which will be Part 3, the analysis will be presented; here, detailed URM structure analysis will be discussed. Materials and methods of performance assessment of URM buildings will be utilized. The sample building selected for the research will be modeled and detailed analysis techniques will be displayed including a step by step procedures.

In Part 4 of the thesis results from the analysis will be displayed in different form. This will includes that figures, tables, analysis output graphs and model results.

In Part 5 of the thesis result discussion will be presented on the result attained.

In Part 6 conclusions on performance assessment and on level of performance for the case study building will be made and corresponding general recommendations on how to upgrade level of performance to acceptable range will be proposed. Finally, suggestions on future study area related to the thesis topic will be made.

PART 2 LITERATURE REVIEW

2.1 UNREINFORCED MASONRY BUILDING STRUCTURES

Unreinforced masonry can be defined generally as masonry that contains no reinforcing in it, FEMA 774 [5]. Masonry is made of earthen materials and includes the sub-types listed below.

- Brick: clay that is fired to a hard consistency.
- Hollow concrete block: "concrete masonry unit" in the terminology of building codes, commonly known as "cinder block."
- Hollow clay tile: similar to concrete block in shape, having hollow cells, but brick-colored.
- Stone: can be "dressed" or cut into rectangular blocks, or used in its natural shape.
- Adobe: mud poured into the form of walls or made into sun-dried bricks.

According to ES EN 1996, pp16, [8] types of masonry are termed as Load bearing wall, Single-leaf wall, Cavity wall, Double-leaf wall, Grouted cavity wall, Faced wall, Shell bedded wall, Veneer wall, Shear wall and Non-load-bearing wall.

Masonry materials are intrinsically strong when compressed under the usual gravity loads but are weak in resisting earthquake forces, which make materials to flex and also shear; 'shear' describes the tendency for a portion of the wall to slide in comparison with the rest.

When an earthquake shakes an unreinforced masonry building, it causes the building's walls to flex out-of-plane and to shear in-plane. Unreinforced masonry is weak in resisting both of those types of forces. Mortar is the "glue" that holds the masonry units together; however, when it eventually cracks, it does so in a brittle manner, similar to the way that the bricks crack. Generally speaking, older masonry constructions were built using much weaker mortar than current building codes require. Mortar also tends to deteriorate in strength over time more than the masonry units themselves do. Thus, earthquake engineers sometimes say that in old masonry buildings, “the mortar holds the bricks apart, not together.”

The most common kind of floor and roof in an unreinforced masonry building is wood frame, typically “two-by-ten” lumber such as 50mm × 250mm small beams (joists), which are usually sheathed with “one-by-six” boards (the use of plywood not being common until after World War II in building construction). The wood floor or roof diaphragm of a building is, unfortunately, very flexible when compared to the stiffer masonry walls.

This flexible wooden diaphragm can allow building walls to lean or bow excessively either inwardly or outwardly (out-of-plane). As the diaphragm bends sideways and vibrates back and forth, it dynamically pushes and pulls on the brick/stone walls, increasing their motions and damage.

Individual structural elements, such as a wall and the roof, only perform adequately in earthquakes when these elements are strongly connected. The typical connection of the wood beams or joists to the unreinforced masonry walls, however, is very weak. A common construction detail used over the decades was to rest the end of a beam in a pocket or niche in the stone/brick wall, with little or no steel hardware providing a strong, positive connection. When an unreinforced masonry building is shaken, the roof or floor framing can pull away from the walls. The walls need the roof to keep them from leaning too far and collapsing, while the roof needs the walls to support it, in order to keep from falling. Typical unreinforced masonry damage includes both the collapse of heavy masonry wall areas and the collapse of part or all of the roof or upper floors.

It is common that parapets, chimneys, and cornices or other decorative features on unreinforced masonry buildings are present in most cases. These elements do not play a structural role, but their failure can be very hazardous. The fact that unreinforced masonry buildings often have multiple seismic weaknesses is not surprising they were not designed to be seismically safe in the first place.

Thus, unreinforced masonry buildings not only have major weaknesses against them from an earthquake-engineering point of view but also they are highly vulnerable for many reasons such as [5]:

1. The walls are weak in resisting horizontal forces (and they lack ductility or toughness);

2. The walls are heavy (they have high mass, leading to high inertial forces);

3. Diaphragms are excessively flexible (insufficient lateral support for the walls);

4. Diaphragm-to-wall connections are either absent or weak;

5. Parapets and ornamentation are common (and made of masonry), and;

6. The buildings were not seismically designed by an engineer (because they were built prior to the time when seismic regulations pertaining to masonry began to be enforced in that particular region).

2.2 ASSESSING EXISTING STRUCTURES

The need to assess a structure may arise from different causes regarding safety hesitation for a particular structure [15]. The two main objectives of assessment of existing structures are to insure structural safety and serviceability, in a context of minimization of costs and time. Costs include inspection, evaluation, maintenance, and repair works.

For typical individual building assessment, possible solutions are in fact binary: yes or no; meanwhile the nonprofessional part of the population answer can be expressed in terms of risk qualifications (low/moderate/high risk). For individual building safety description a flowchart like the one proposed by Schneider [15] in Fig.8 can be used.

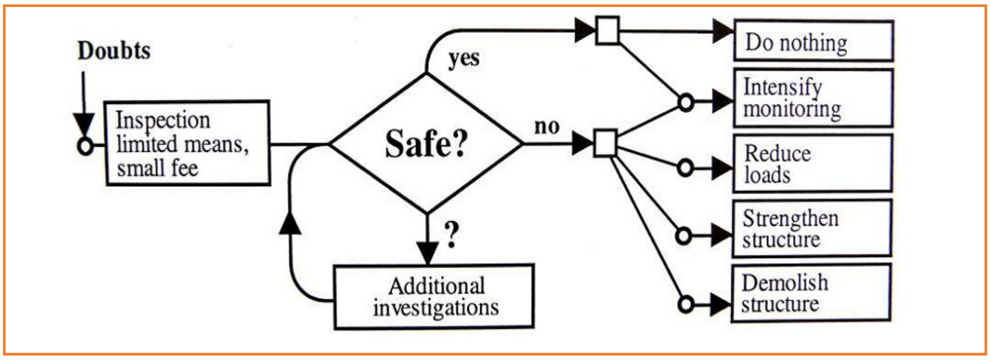


Figure 8: The assessment phase for an individual edification taken from [15]

If a building found to be unsafe, different options are possible according to Fig.8. Usually, for common building housing, it is not possible to intensify monitoring. Load reduction, from a seismic engineering point of view, can be accomplished by changing the use of the structure (e.g. from storage to office) or by changing heavy elements in the structure such as heavy roofs.

The decision neither to demolish nor strengthen of a structure is mainly taken on an economical or Cultural, Social and Historical (CSH) basis.

2.3 ETHIOPIAN BUILDING CODE STANDARD FOR URM BUILDINGS ASSESSMENT

On Technical Note compiled by Samuel [23] it is stated that historically the first draft Seismic code for Building in Ethiopia was published in year 1980 named (CP1-78) and latter it was revised in 1984 as (ESCP1-73) which was followed by the currently active in design offices code EBCS 8-1995 [17]. The latest revision which is ES EN 1998 [9] is however currently published but not fully capitalized in the design offices and university libraries.

ES EN 1996 [8] and ES EN 1998, [9] are devoted to Design of Masonry Structures and Design of Structures for Earthquake Resistance. These codes came to picture only after many years the URM buildings were built and hence the codes can be used only for performance assessment of existing URM structures.

Thus the author have used the revised codes for the analysis of performance assessment of URM buildings even though using the revised codes is preferred before construction, the older codes are used in the absence of some particular items due to no access to full package of the revised codes. There is obvious relationship between the revised codes and (Eurocode Parts) such that the former is dependent on the later. Thus, the author used substitute documents from Eurocode, similar other parts when needed.

Despite the fact that there is limited researches regarding the existing URM buildings in Addis Ababa or generally in Ethiopia; it is the author’s task in this thesis to try to apply the simplified structural analysis methods outlined in the revised code and further investigate from literatures such methods appropriate for the analysis of URM. The performance assessment analysis methods in this thesis are based on these codes.

In ES EN 1996, [8] the part devoted to URM is found in its Part six of the code document. However, the code does not cover the special requirements of seismic design.

Provisions related to such requirements are given in ES EN-1998 [9]. "Design of structures in seismic regions" and is consistent with ES EN-1996 [8].

2.4 SEISMIC ACTION, PEAK GROUND ACCELERATION AND LOAD COMBINATIONS

Seismic actions are inertial forces, which occur during the vibration of structures due to earthquakes. These forces demand certain amount of resistance in order not the structure to fail under predefined expected service. Thus designing such structure is the art of harmonizing the demand and capacity within the available economy while meeting certain requirements.

Table 2: Bedrock Acceleration Ratio α_o

Zone	5	4	3	2	1
$\alpha_o = a_g/g$	0.20	0.15	0.10	0.07	0.04

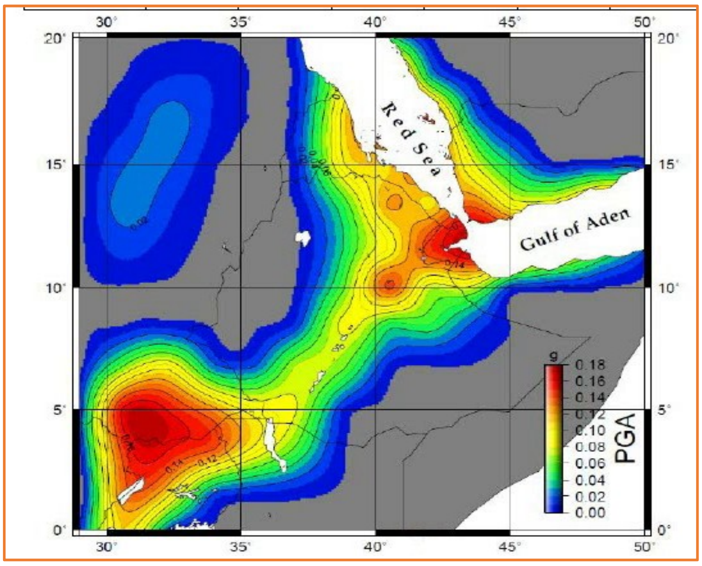


Figure 9: Seismic Hazard Map along the horn of Africa [9].

ES EN 1998 Annex D [9], divides the country into five zones as zone I, II, III, IV and V. Load combination is also specified in the revised ES EN 1998 [9] National Annex. Other seismic load parameters are calculated using the same National Annex. The load combinations used in Finite Element Analysis Software tool, SAP2000 v.18, modeling are shown in Appendix C.

2.5 MATERIAL PROPERTIES OF URM BUILDING ELEMENTS.

2.5.1 STONE COMPRESSIVE STRENGTH

The compressive strength of masonry units converted to the air dried compressive strength of an equivalent 100 mm wide x 100 mm high masonry unit as explained in the ES EN 1996 pp13, [8]. The normalized mean compressive strength of the units, in the direction of the applied action effect, in N/mm^2 is termed as f_b as used in Eq. (2-1)

However, due to the heritage value of the sample building selected for modeling only non-destructive test is applied. Schmidt hammer test method is used to get the representative compressive strength of the stone used for the structural wall system in the representative building. According to EN 1998-3; Annex C, pp82, [10], it is stated that Schmidt rebound hammer test is acceptable to evaluate surface hardness of exterior masonry walls. While applying Schmidt hammer test wide variations related to predicted compressive strength occurs due to the natural variation of material property of stone units used in masonry wall.

In spite of the variations of result expected from the test and due to lack of resource to apply other more accurate tests such as ultrasonic pulse or mechanical pulse, test result from Schmidt Hammer test used to estimate the existing compressive strength of stone unit.

The error in compressive strength reading from hammer test in a given region is predictable within acceptable range. It is hence possible to estimate the compressive strength of the selected URM building masonry unit by the Average Hammer number found from the Schmidt Hammer Test [13].

2.5.2 MUD MORTAR COMPRESSIVE STRENGTH

According to ES EN 1996, pp 26, [8], the property of mortar can be described as the compressive strength of masonry mortar, f_m, shall be determined in accordance with (EN 1015-11) as cited by [8].

The minimum compressive strength of mud mortar is not covered in ES EN 1996 [8]. As the sample building is historical building destructive test is not allowed. Estimation of mortar compressive strength using Schmidt Hammer Test found to be impossible for mud mortar due to the weak surface of this mud mortar which is part of the masonry to the sample representative URM building under investigation in this paper.

Hence, the other possible way to estimate the mud mortar *compressive* strength is by using Mortar Scratch Index method. This method also is not covered in ES EN-1996 [8], specifically for mortar performance checking. However, this method has gained acceptance and currently being used in Australian Standard for Masonry Structure. It is also well emphasized on how to use scratch test during performance assessment of URM buildings in New Zealand [19], the document with a table provided the mortar scratch index method to estimate mud mortar compressive strength. Hence, in this thesis, a non destructive test method to be used for mortar compressive strength is mortar scratch index method.

2.5.3 CHARACTERISTIC COMPRESSIVE STRENGTH OF MASONRY WALL

The Characteristic Compressive Strength of Masonry, f_k can be estimated using the formula in ES EN 1996 pp 29 [8], Characteristic compressive strength of masonry other than shell bedded masonry.

The characteristic compressive strength of masonry should be determined from either: (i) results of tests in accordance with (EN 1052-1) which tests may be carried out for the project or be available from tests previously carried out e. g. a database; the results of the tests should be expressed as a table, or in terms of Eq. (2-1)

$$\mathrm{f_k} = k f_b^{\alpha} f_m^{\beta} \tag{2-1}$$

where:

f_k is the characteristic compressive strength of the masonry, in N/mm2

K is a constant and where relevant, modified α, β are constants

f_b is the normalized mean compressive strength of the units, in the direction of the applied action effect, in N/mm^2

f_m is the compressive strength of the mortar, in N/mm^2

Limitations on the use of Eq. (2-1) should be given in terms of f_b, f_m, the coefficient of variation of the test results, and the Grouping of the units.

For masonry made with general purpose mortar where there is a mortar joint parallel to the face of the wall through all or any part of the length of the wall, the values of K can be obtained by multiplying the values given in (ES EN 1996 by 0.8).

The classification of mortar for the sample building falls in general purpose mortar and Group 1 masonry unit for Dimensioned Natural stone.

The constants α & β are referred from ES EN 1996, [8] to calculate f_k thus,

$$f_k = 0.8kf_b^{0.7}f_m^{0.3} \tag{2-2}$$

2.5.4 MODULUS OF ELASTICITY

The property of bodies of returning after unloading to their initial form is called elasticity. Using ES EN 1996 [8] for masonry structures the short term secant modulus of elasticity E shall be determined by tests.

In the absence of a value determined by tests, the short term secant modulus of elasticity of masonry E for use in structural analysis, may be taken to be K_E multiplied by f_k. The recommended value of K_E is 1000, where f_k is characteristic compressive strength of the masonry wall. Thus, the value E can be calculated as:

$$E = 1000f_k, \tag{2-3}$$

2.5.5 SHEAR MODULUS OF ELASTICITY

Shear modulus of Masonry can be found by using the formula in ES EN 1996 pp 42 [8];

$$G = 40\%E, \tag{2-4}$$

Note: However according to experiments by Tomazevic [16], made on different masonry walls the Eq. G =40%E is not realistic. It is shown that most masonry walls with various aspect ratios found to be much less than 40%. Results from the experiment showed on [16]; which says G = 0.1E should be used instead of (2-4) holding this for open discussion, here in this thesis Eq. (2-4) will be used for sample analysis works.

2.5.6 MATERIAL PROPERTIES OF FLEXIBLE TIMBER FLOOR DIAPHRAGMS

Due to flexible behavior of timber structure the structural contribution of timber truss structures will be ignored for the sample URM building assessment except for the dead load case. It is imperative to note that timber structures by which it means timber floors and timber trusses.

The loose connection at the tip where wall and timber truss meet there is no rigid connection. Due to pulling out of the timber away from wall where it is imbedded impacts become even worse due to the pounding impact in some cases by exaggerating the cyclic load during earthquake event up to four times stronger. However if proper embedment is secured between the timber diaphragm and the wall there is detailed separate method of assessment is can be applied.

A detailed investigation by New Zealand Society for Earthquake Engineering, (NZSEE) [19] classifies that if the lateral force resisting members drift value is compared with the diaphragm deflection. URM buildings shall be classified as flexible [11] unless the maximum lateral deformation of the diaphragm is less than half the average inter-storey drift of the vertical lateral-force-resisting elements of the associated storey. However, due to extended service age and little or no support connection in most URM buildings in Addis Ababa the timber floor or timber truss diaphragm contribution is neglected.

2.5.7 TIMBER DENSITY

Timber density shall be determined from testing in accordance with EBCS 1-1995 [17]. For the purpose of this thesis a unit weight table from EBCS 1 1995[17] is used to get the unit weight of timber. The timber works observed in the sample building for the analysis are made from Junipers Procera (Tid) 7.5 kN/m^3.

2.5.8 TIMBER EMBEDDED STRENGTH

In most of URM buildings rigid connection not usually observed thus in this thesis the sample building selected will not include timber embedded strength where it will be assumed flexible diaphragm.

2.5.9 POISON'S RATIO OF URM WALLS

An important constant for a given structural material is its Poisson's ratio. It is the ratio of lateral strain over axial strain with a minus sign to consider deformation during tensile stress. In order to obtain the elastic response of URM buildings the key elastic property Poisson's ratio ($\boldsymbol{\nu}$) must be available. The value of Poisson's ratio also connects the Shear rigidity modulus and Young's modulus of elasticity. No specific value is set in the codes both 1995 and 2015 version of codes. According to a number of literature the value for URM wall ($\boldsymbol{\nu}$) = 0.25, pp 60 [27].

2.5.10 CHARACTERISTIC SHEAR STRENGTH OF MASONRY

During seismic events, the masonry wall will be subjected lateral load due to their own weight and because of the gravity load shear resistance will develop in the wall elements. This resistance is vital for the stability of the wall. If the resistance is less that lateral disturbing force collapse of the structure will occur.

According to ES EN 1996 [8];

(1) The characteristic shear strength of masonry, f_{vk}, shall be determined from the results of tests on masonry.

NOTE: Test results may be obtained from tests carried out for the project, or be available from a database.

(2) The characteristic initial shear strength of masonry, f_{vko}, should be determined from tests in accordance with relevant Annex similar to (EN 1052-3 or EN 1052-4).

(3) The characteristic shear strength of masonry, f_{vk}, using general purpose mortar in accordance with (3.2.2(2), of ES EN 1996 [8]) or thin layer mortar in beds of thickness 0.5 mm to 3.0 mm, in accordance with pp 26 of ES EN 1996, [8] or light weight mortar in accordance with pp 26 of ES EN 1996 [8] with all joints satisfying the requirements of (pp 82 of ES EN 1996 [8])of so as to be considered as fulfilled, may be taken from.

$$f_{vk} = f_{vko} + 0.4\boldsymbol{\sigma}_d \tag{2-5}$$

but not greater than 0.065 f_b or f_{vlt} where:

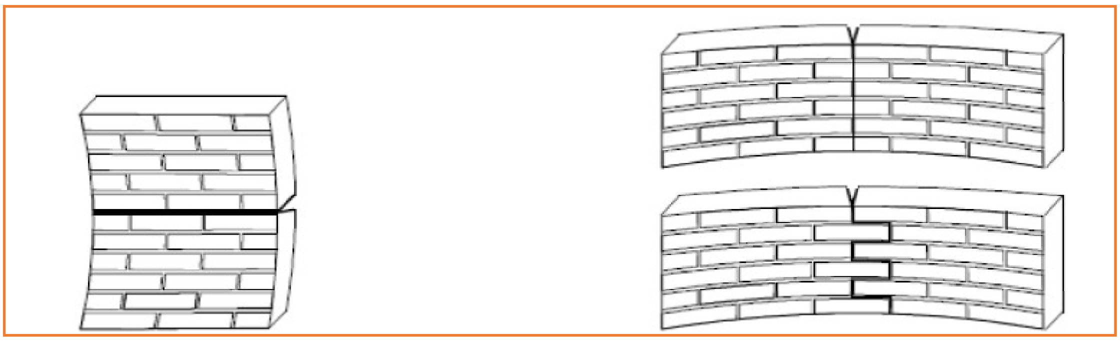
f_{vko}	is the characteristic initial shear strength, under zero compressive stress; when test is not possible recommended values can be read from table on pp 40 of ES EN 1996 [8].
f_{vlt}	is a limit to the value of f_{vk};
σ_d	is the design compressive stress perpendicular to the shear in the member at the level under consideration, using the appropriate load combination based on the average vertical stress over the compressed part of the wall that is providing shear resistance;
f_b	is the normalized compressive strength of the masonry units, as described in pp 26 of ES EN 1996 [8], for the direction of application of the load on the test specimens being perpendicular to the bed face.

As far as old URM buildings concerned usually may have historical value. Thus testing mortar may not be possible so as mentioned in the limitation the alternative option is to use Scratch Index test as discussed here below in (2.6.2).

2.5.11 CHARACTERISTIC FLEXURAL STRENGTH OF MASONRY

In relation to out-of plane bending, the following situations should be considered: flexural strength having a plane of failure parallel to the bedjoints, f_{xk1}; flexural strength having a plane of failure perpendicular to the bed-joints, f_{xk2} see Fig. 10.

The characteristic flexural strength of masonry, f_{xk1} and f_{xk2}, shall be determined from the results of tests on masonry.



a) plane of failure parallel to bedjoints, f_{xk1}

b) plane of failure perpendicular tobedjoints, f_{xk2}

Figure 10: Planes of failure of masonry in bending

Due to the heritag value of the URM building sample analyzed in thesis alternative methods will be used to determine the characteristic flexural strength of masonry based on Eq. (2-3). In such cases ES EN 1996 [8] provides convenient values to be judged during assessment or design analysis.

2.5.12 DEFORMATION PROPERTIES OF MASONRY

The stress-strain relationship of masonry in compression is non-linear and may be taken as linear, parabolic, parabolic rectangular see Fig. 11 or as rectangular, for the purposes of designing a masonry section.

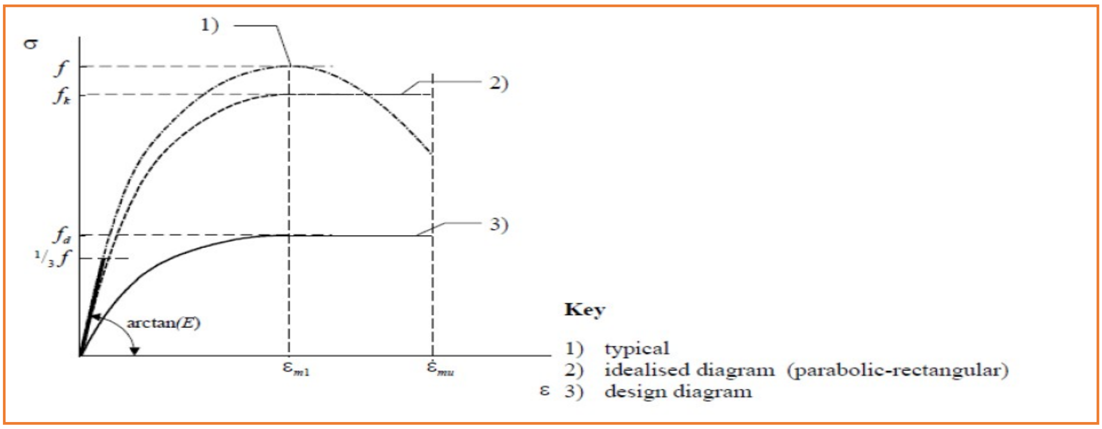


Figure 11: Stress-strain relationship for masonry in compression

The limiting compressive strain as verified in ES EN 1996 pp 78 [8], in the masonry is taken as -0.0035 for Group 1 units and -0.002 for Group 2, 3, and 4 where Autoclaved aerated concrete, manufactured stone and dimensioned natural stone units are considered to be Group 1. The geometrical requirements for grouping of clay, calcium silicate and aggregate concrete units for remaining groups are given in tabulated form in ES EN 1996 pp 24 [8].

The tensile strength of the masonry is ignored. However, to consider flexural capacity of the masonry it is assumed that in item 2.5.11 will consider bending flexural tension.

2.6 OTHER NECESSARY PARAMETERS FOR URM BUILDING ANALYSIS

2.6.1 INITIAL SHEAR STRENGTH OF MASONRY (COHESION AND COEFFICIENT OF FRICTION)

The initial shear strength of masonry (f_{vko}): In cases where destructive sampling is impossible other codes or literatures may provide a reasonable approximation method. Here it is provided the mortar cohesion value for optional usage. However, the f_{vko} value is also provided in table from of ES EN 1996 pp 40, [8], as discussed in (2-5). Masonry bed-joint shear strength under zero axial compression (Cohesion), c, can be calculated using the average mortar compressive strength in accordance with Eq. (2-6) instead of directly taking the predefined value. The bed-joint coefficient of friction, μ_f, can be taken in accordance with [19].

$$f_{vko} = C = 0.045\, f_j' \tag{2-6}$$

$$\mu_f, = 0.65 \tag{2-7}$$

Where: f_j' = average mortar compressive strength, μ_f, = bed coefficient of friction

2.6.2 MORTAR PROPERTIES

The average mortar compressive strength shall be determined using the scratch index test in accordance with the procedure outlined here below. Table 3, describes the mortar scratch test and the corresponding scratch index with corresponding mortar compressive strength.

Table 3: Mortar Scratch Test Description

Mortar Description	Mortar Scratch Index	Hardness	Average mortar compressive strength, f'_j (Mpa)	Lower characteristic mortar compressive strength, f'_{jl} (Mpa)
Easily scratches using fingernail	1.5	Soft	1.4	1.0
Scratches using fingernail	2.0	Medium	5.5	3.2
Scratches using aluminum	2.5	Hard	7.4	5.8

2.6.3 SHEAR STRENGTH OF TIER LACED TYPE OF MASONRY

In the case of tier-laced type of masonry, shear strength is enhanced by the contribution of the horizontal tiers (timber or other prismatic material such as reinforced concrete) intersecting the diagonal plane of failure.

$$f_{vk}^{tot} = f_{vk} + f_{vl} \tag{2-8}$$

$$f_{vl} = f_{vko} P_{tier} \sum_{i=1}^{n} L_{bi} \tag{2-9}$$

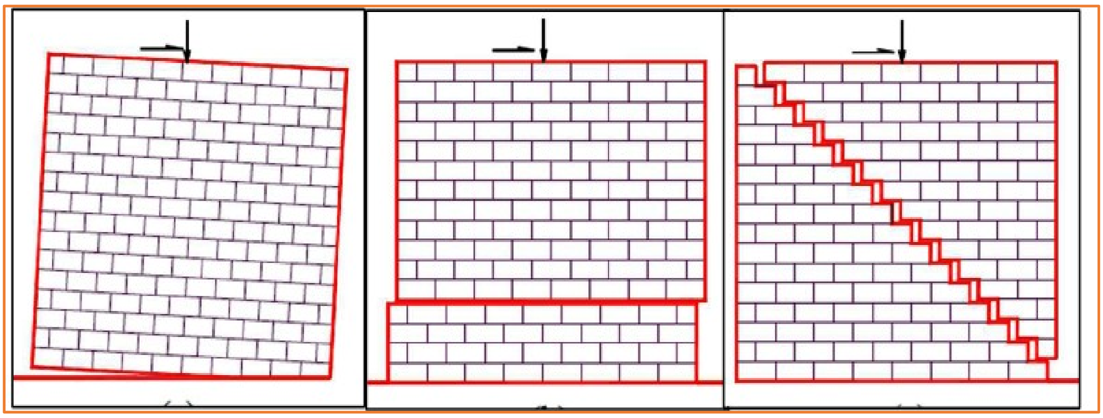
Where: f_{vk} , f_{vko} are defined earlier in (2-5)

f_{vl} = Shear strength contribution from lacing members

P_{tier} = P_{tiers} the contact perimeter of the tier's cross section (=2b for 2-sided contact of timber tiers with a cross sectional width =b

2.7 FAILURE MODES OF URM BUILDINGS

Failure modes in URM buildings are mainly two types which are I) Out of plane failure; and II) In plane failure. Other failure modes relatively less evasive are III) Diaphragm failure, IV) Connection Failure, V) Failure due to opening of wall, VI) Pounding, and VII) Non-structural component failures [15]. Since most URM buildings possess relatively large cross section with compressive strength, they automatically satisfy serviceability requirements. However, if not designed for lateral loads especially those from accidental seismic loads URM structure may show one or a combination failure modes mentioned.



a) Rocking failure b) sliding failure c) Diagonal tension failure

Figure 12: In plane failure mode of URM wall due lateral and vertical load.

Specifically, URM buildings resist lateral loads by in-plane action where the major stability sources are walls parallel to the direction of the seismic action. Hence, the in-plane failure modes are prior interest to carry out performance assessment. The brittle nature of URM walls gave rise that for a very little deformation the resulting failure is high where parts of the URM wall plane manifest bold failure modes. The most probable failure modes due to both vertical and lateral loads in the in-plane case are a) Rocking failure, b) Sliding (Shear Slip failure) and C) Diagonal tension failure [16].

2.7.1 ROCKING FAILURE (TENSION CONTROLLED)

In URM walls the lateral resistance steams from overbearing load, when the axial load is low and the aspect ratio is (slenderness) is high this failure occur. The unique character of Rocking is the wall will exhibit a rigid body rotation about or near the toe of the wall accompanied by long horizontal cracks. When the wall laterally loaded parallel to its plane an overturning moment will occur at the base of the wall as shown in Fig. 12 (a). As the crack propagates, the bearing area will reduce leading to localized compression rupture at the toe of the wall and eventually rocking failure will occur [26].

2.7.2 SLIDING (SHEAR-SLIP) FAILURE

Sliding failure also called Shear –Slip failure occurs when the masonry wall slides horizontally from bed joint. Walls and Piers with low slenderness; low cohesion of mortar especially if axial load is low; are prone to this type of failure. The critical load for this failure to occur depends on the friction coefficient between the face of masonry unit and mortar with overbearing axial load normal to mortar joint. Fig. 12 (b) shows the possible crack line associated with sliding failure [26].

2.7.3 DIAGONAL CRACKING (TENSION) FAILURE

In cases where the Rocking and Sliding failure are taken care by the plane action safely the wall will act as fixed cantilever beam. When the wall is subjected to a combination of lateral shear and vertical axial compressive stresses, the resultant stress result will act diagonally leading to diagonal cracks. Here the major responsible part the masonry is the mortar where its tensile or cohesive strength should be within the realm of the tensile strength demand. Fig. 12 (c) shows the possible crack during diagonal tension failure mode [26].

2.8 PERFORMANCE ASSESSMENT ANALYSIS METHODS FOR URM STRUCTURES

It is stated in [27] that the term analysis in the context of URM is used to refer two complementary attributes of the procedures used in assessment of the building response

A. Methods of discretization and assembly of the structural model used in order to represent the actual building in the framework of a calculation algorithm.
B. Methods used to satisfy the governing Eq.s and subsequently solving for the internal stress state generated in the structure in response to applied external disturbance (here, earthquake effects).

There are several alternative options classified under the two groups in (A) different methods of idealization are concerned with accurately representing in a computational environment the physical world namely the spatial, the geometric and material complexity of the structure. A critical decision is the selection of the typical elementary building component (a virtual “building block”) that is used to assemble the computer model of the structure [27].

Establishing the governing Eq.s and obtaining a solution for the response under the applied excitation is pursued using different alternative solution strategies; thus, (B) in the preceding classification of analysis methods encompasses a variety of solution algorithms used in order to calculate the dynamic response from the field Eq.s of motion, namely:

$$M.\ddot{U} + C\dot{U} + \mathrm{K.U} = -M.r.\ddot{U}_{\mathrm{g}} \quad (2\text{-}10)$$

Eq. (2-10), which is a coupled system of Eq.s, where the vector U represents the nodal relative displacements along the degrees of freedom of the structure, $\ddot{U}$ and $\dot{U}$ are the time derivatives (acceleration and velocity vectors), r is the influence vector (i.e. the vector containing the displacements occurring in the degrees of freedom of the structure when a unit displacement occurs at the point of input of the ground motion (usually at the base), M , C and K the mass, damping and stiffness matrices of the structure and $\ddot{u}_g$ the ground acceleration (a function of time) [27].

2.8.1 ANALYSIS OF URM IN THE CONTEXT OF STRUCTURAL MODELING

This part of analysis refers to the discretization of the structure into elements, and the proper selection of the element type, connectivity and material properties. Implementation is critical in that it determines the kind of mechanical function that may be supported by the components of the analytical model through the degrees of freedom by which the individual elements are equipped, such as frame elements, 3D solids, shell elements, plane stress elements [27]. Alternative approaches that qualify under this classification of methods are:

A.1 Equivalent Frame Models, Strut and tie Models

A.2 Finite Element Analysis

A.3 Discrete Element Analysis

A.4 Simple Models Linear Static Procedure

A.1: Modeling walls with plate element performs well in linear analysis but it is difficult to model nonlinear element properties with the plate modeling. Hence, URM buildings can be modeled with Equivalent Frame (line) element for the non-linear analysis.

There is a possibility of modeling complex 3D models of URM structures, obtained by assembling walls and floors, however mainly referring to their in-plane strength and stiffness contributions. This particular issue of floor inclusion is the main disadvantage of the method when applied to structures with flexible diaphragms, where the out-of-plane component is dominating the overall response.

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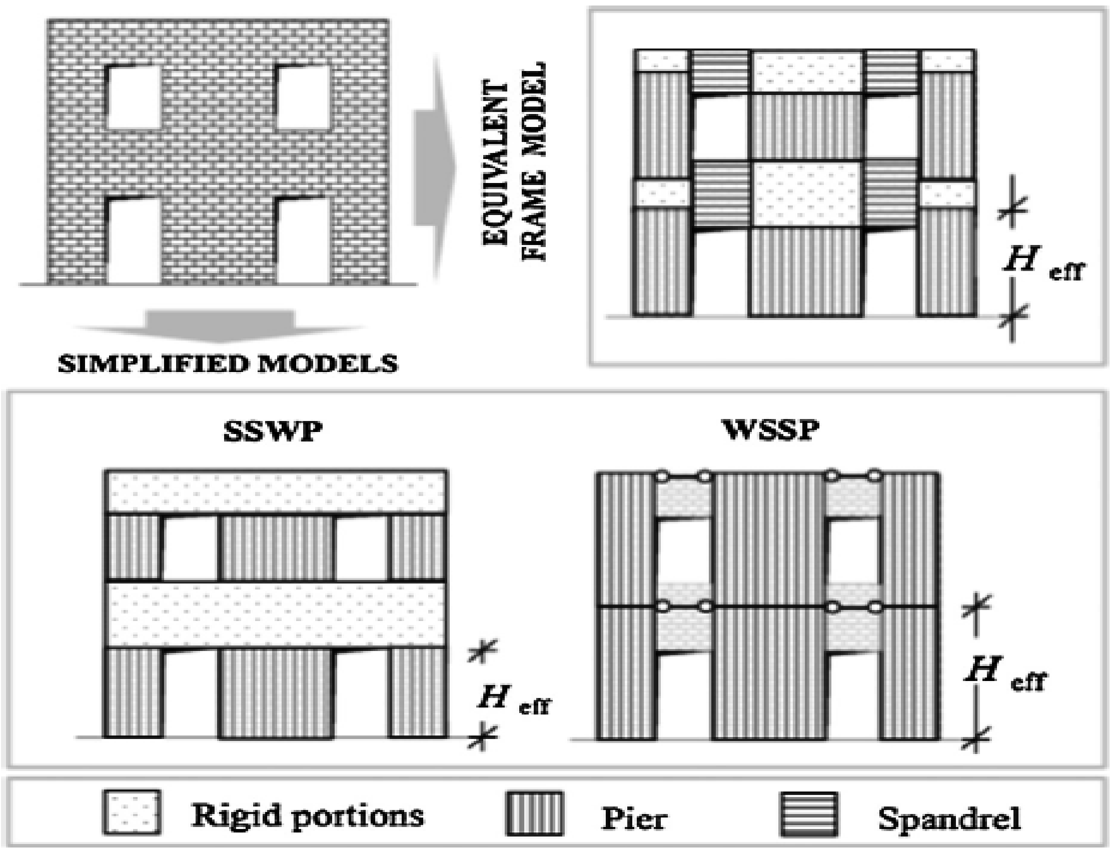


Figure 13: URM wall idealization according to simplified and equivalent frame models [26]

A.2: The Finite Element Analysis is obviously a general procedure that enables the user through the element library to represent in the computer model the geometric, spatial and complexity of the structure with significant accuracy. Masonry is by construction anisotropic (the response being biased by the orientation of the joints), exhibiting quite a complex mechanical behavior owing to the composite nature of the material (mortar and blocks) [27].

Behavior in tension is rather brittle to the extent that the assumption of tension cutoff may be reasonable in constitutive characterization of the material. Compressive strength is dependable and considered high, although it is actually one quarter the value of normal concrete. Frictional response is governed by shear. Describing this complex mechanical behavior usually requires very sophisticated constitutive formulations, equipped with elastic and plastic response, failure surfaces and flow potential function (non associated plasticity). For this reason, masonry in most finite element formulations is treated as homogenized masonry is taken as in Eq. (2-1)

Clearly nonlinear Finite Element solutions are still hampered today by the convergence and algorithmic stability problems instigated by the tension cutoff after cracking in URM, (where the tensile strength is insignificant), combined with the absence of the stabilizing participation of reinforcement which is available in the case of concrete structures. Thus, once failure is detected, the process of several revisions of the stiffness matrix slows down considerably the rate of convergence of the solution, to the extent that dynamic nonlinear time history analysis becomes quickly non-feasible. So in these cases conducting nonlinear pushover analysis in simple masonry elements is the limit of current nonlinear F.E. technologies [27].

Modeling approaches at material level when considering detailed complex analysis two major modeling techniques can be considered:

I- Simplified micro-modeling (masonry as a two-phases material)

Expanded units are represented by continuum elements whereas the behavior of the mortar joints and unit–mortar interfaces is lumped in discontinuum elements Fig.14 (c). According to these procedures, which are intermediate approaches, the properties of the mortar and the unit/mortar interface (masonry as a two-phase material) are lumped into a common element, while expanded elements are used to represent the brick units. This approach leads to the reduction in computational intensiveness, and yields a model, which is applicable to a wider range of structural systems [27].

II- Detailed micro-modeling (masonry as a three-phases material)

Units and mortar in the joints are represented by continuum elements whereas the unit–mortar interface is represented by discontinuum elements Fig.14 (d). While this leads to accurate results, the level of refinement means that any analysis will be computationally intensive, and so will limit its application to small laboratory specimens and structural details, [21] have proposed simplified micro- modeling procedures to overcome the problem.

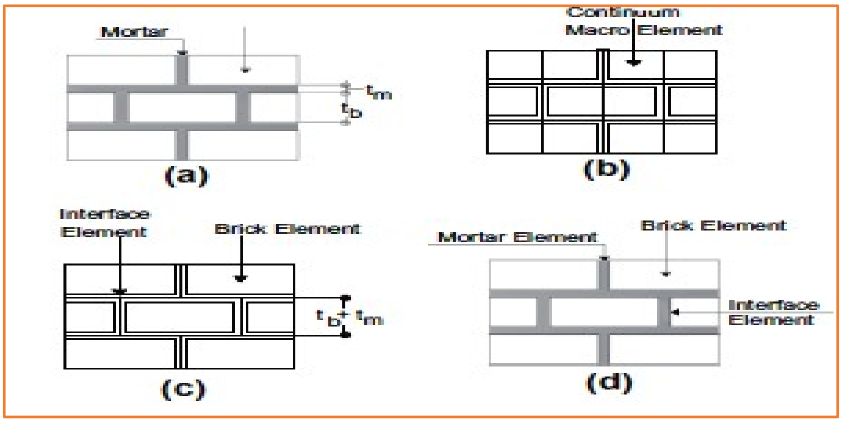


Figure 14: Masonry modeling strategies: (a) masonry sample; (b) macro-modeling; (c) simplified micro-modeling and (d) detailed micro-modeling. [20],

A.3: Discrete Element Analysis Modeling (DEM) and Analysis: This idealization technique appears to hold significant promise for nonlinear simulation of brittle structures, as it does not present the types of convergence instabilities associated with the occurrence of a post peak descending branch after tensile fracture [27].

Here the structure is subdivided to discrete volumes with a polygonal surface, and either rigid or elastic constitutive properties. Inelasticity occurs at the points of contact between discrete volumes, which are modeled through one dimensional (axial or shear) springs, with predetermined force – displacement relationships. In this context masonry blocks are treated as separate units. DEM methods may use ‘compliant contacts’ or ‘unilateral contacts’.

A.4: Simple Models: In this type of analysis, a variety of simplifications have been tried with the objective to exploit the capabilities of commercially available, easy to use software, or in order to achieve a hands on, mechanistic model that may evade numerical simulation altogether. The wide range of possibilities cannot be treated with fairness in the summary presented herein; some key points are highlighted for the benefit of appreciation of the practical issues involved [27].

Three Dimensional Elastic Finite Element Analysis: A favored approach among engineers that have access to the use of common engineering software (such as SAP 2000) is to develop a finite element model of the URM building, using shells as a basic unit for discretization and elastic material properties [27].

Analyses of this type can be used to identify the force path through the structure, and those regions of high deformation demand (regions where sharp strain gradients identify strain and stress concentrations). : Of course, such models may not be taken literally to quantify URM behavior under seismic loads, but they may prove very useful in highlighting the regions of anticipated damage, but also in demonstrating in a qualitative sense, the effectiveness of retrofit measures in moderating the intensity of stress localization throughout the structure (i.e. results are to be used as relative indicators of the parametric dependencies of the response and not as quantifiersof the response).

Some marginal nonlinearity may be introduced in this class of models without risking overall convergence failure: this refers to uniaxial nonlinear springs or gap elements that are necessary in order to account for imperfect contact between components, particularly in the case of dissimilar materials. Such cases include but are not limited to:

- Unilateral contact of masonry walls with the surrounding soil when the structure has a basement
- Unilateral contact of timber diaphragm beam penetrating into a wall
- Partial Bonding of timber laces embedded in masonry.
- Contact between parts of the structure that are not integrally connected.

Properties for such springs may be derived from basic mechanics of the problem idealized and the tributary area of the contact joint that is represented by the springs/gap elements [27].

Mechanistic Models: Simple hand calculation tools for rapid assessment of masonry have been developed ranging from simple strut and tie equilibrium models, to distributed mass/stiffness single degree of freedom idealizations of the URM building.

2.8.2 ANALYSIS IN THE CONTEXT OF RESPONSE CALCULATION (SOLUTION OF THE EQ.S OF MOTION)

It was stated earlier that there are two general attributes implied under the term "Analysis" of URM Buildings; Section 2.8 was dedicated to issues of idealization of the structural system and properties, which defines the property matrices *M*, *K*, and *C* in Eq. (2-10), but also the nature and topology of the degrees of freedom that enter the response vector, U. The present section is concerned with alternative procedures and solution strategies that may be used in order to solve the Eq.s of Motion (Eq. 2.10). Thus, (B) in the preceding classification of analysis methods encompasses a variety of solution algorithms used in order to calculate the dynamic response from the field Eq.s of motion, namely: U

B.1: Nonlinear Step – by – Step Time History Analysis

B.2: Nonlinear Static Analysis (Pushover)

B.3. Linear Elastic Modal Analysis, using either Modal Spectral Response, or Modal Time History Response with mode superposition

B.4. Static Analysis

The solution algorithms above are ordered with decreasing degree of complexity and computational demand. In all cases there is a significant degree of interaction with the choice of mechanical model – the more complex the idealization strategy chosen in the context of methods (A), the more restrictions will be imposed on the choice of a solution strategy under (B).

Linear Elastic Modal Analysis may be used with both spectral and time history definitions of the seismic hazard, but is nevertheless restricted to the use of linear-elastic idealizations of the structure, as the underlying premise of this method is the principle of superposition. Of the two alternatives of linear elastic analysis, clearly the option of time history analysis of modal response and real time superposition of modal contributions is the only accurate option. Despite this, most commercial popular software that could be used for analysis of URM structures only offer the possibility of combination of the Modal Maxima either through the Complete Quadratic Combination (CQC) or the Square - Root - of the - Sum- of the - Squares (SRSS) options [27].

These approaches combine maximum responses of different modal contributions which, in time history records, are not concurrent. The danger from these approaches is that overly conservative estimates may be obtained from such an analysis, in light of the fact that several modes need be accounted for before 70% of the total URM building mass may be said to be engaged in elastic F.E. modal analysis since in the absence of diaphragms several hundreds of subordinate modes are excited in such computer models. These modes are occasionally spurious and bear no relevance to the actual behavior. Overly conservative estimates of response emanating from SRSS may lead to excessively invasive (nonreversible) interventions in URM buildings during rehabilitation, and as such they should be always treated with particular caution [27].

2.9 PERFORMANCE EVALUATION GUIDE: MODELS FOR GLOBAL ASSESSMENT OF ELEMENTS UNDER NORMAL FORCE AND BENDING

A detailed evaluation guide is presented in EN 1998-3, 2005, pp84 to 86, for Existing URM buildings.

In setting up the model for the analysis, the stiffness of the walls should be evaluated taking into account both flexural and shear flexibility, using cracked stiffness. In the absence of more accurate evaluations, both contributions to stiffness may be taken as one-half of their respective un-cracked values.

Based on this guide, if a linear analysis is performed the criterion for global assessment is defined in terms of the base shear in the horizontal direction of the seismic action. The capacity may be taken equal to the sum of shear force capacities of the individual walls, as this is controlled by flexure by shear in the horizontal direction of the seismic action. The demand is the estimate of the maximum base shear in that direction from the linear analysis [10].

2.9.1 LS OF SIGNIFICANT DAMAGE (SD)

(1) The capacity of an unreinforced masonry wall is controlled by f1exure, if the value of its shear force capacity given in 2.9.1 (3) is less than the value given in 2.10.1(3).

(2) The capacity of an unreinforced masonry wall controlled by flexure may be

expressed in terms of drift and taken equal to 0.008Ho/D for primary seismic walls and to 0.012Ho/D for secondary ones, where:

D is the in-plane horizontal dimension of the wall (depth)

Ho is the distance between the section where the flexural capacity is attained and the contra-flexure point.

(3) The shear force capacity of an unreinforced masonry wal1 controlled by flexure under an axial load N, may be taken equal to:

$$V_f = DN\frac{DN}{2H_o}(1-1.15\upsilon_d) \qquad (2\text{-}11)$$

Where, D and Ho are as defined in (2),

υ_d= N/D(f_d) is the normalized axial load (with. $f_d = f_m/CF_m$), where f_m is the mean compressive strength as obtained from in-situ tests and from the additional sources of information, and CF_m is the confidence factor for masonry given in table from EN 1998-3, 2005, pp19, [10] the appropriate knowledge level m), t is the wall thickness.

2.9.2 LS OF NEAR COLLAPSE (NC)

(1) .2.9.1(1) and 2.9.1 (3) apply.

(2) The capacity of a masonry wall controlled by flexure may be expressed in terms

of drift and taken equal to 4/3 of the values in 2.9.1(2).

2.9.3 LS OF DAMAGE LIMITATION (DL)

(1) 2.9.1 (1) applies.

(2) The capacity of URM wall controlled by flexure may be taken as the shear force capacity given in 2.9.1 (3)

2.10 PERFORMANCE EVALUATION GUIDE: ELEMENTS UNDER SHEAR FORCE

2.10.1 LS OF SIGNIFICANT DAMAGE (SD)

(1) The capacity of an URM wall is controlled by shear, if the value of its shear force capacity given in .2.10.1 (3) is less than or equal to the value given in 2.9.1 (3)

(2) The capacity of URM wall controlled by shear may be expressed in terms of drift and taken equal to 0.004 for primary seismic walls and to 0.006 for secondary ones.

(3) The shear force capacity of an URM wall controlled by shear under an axial load N, may be taken equal to:

$$V_f = f_{vd}\, D't \qquad (2\text{-}12)$$

where:

D' is the depth of the compressed area of the wall,

t is the wall thickness, and

f_{vd} is the masonry shear strength accounting for the presence of vertical load:

$=f_{vmo} + 0{,}4\ N/(D't) \leq 0{,}065 f_m$ where , f_{vmo} is the mean shear strength in the absence of vertical load and f_m the mean compressive strength, both as obtained from in-situ tests and from the additional sources of information, and divided by the confidence factors, as defined in the table from EN 1998-3, 2005, pp19, [10] accounting for level of knowledge attained. In primary seismic walls, both these material strengths are further divided by the partial factor for masonry in accordance with ES EN 1998 pp 174 [9].

2.10.1 LS OF NEAR COLLAPSE (NC)

(l) 2.10.1 (1) and 2.10.1 (3)) apply.

(2) The capacity of an unreinforced masonry wall controlled by shear may be expressed in terms of drift and taken as 4/3 of the values in 2.10.1 (2)

2.10.3 LS OF DAMAGE LIMITATION CDL)

(l) 2.10.1 (1) applies.

(2) The capacity of an unreinforced masonry wall controlled by shear may be taken as the shear force capacity given in. 2.10.1 (3).

2.11 ALTERNATIVE PERFORMANCE EVALUATION

Different assessment criteria which are based on comparing strength demand and capacity may limit the magnitude of seismic action which can be supported within elastic response. If the out of plane deflection is restrained by rigid floor system or beam belt at roof levels the in-plane shear strength may exhaust fully and diagonal shear failure may occur earlier than that of out of plane failure.

The behavior factor, q, is decided according to the seismic resisting members property and using recommended values from a table on ES EN 1998-1 pp170, [9]. With proper selection of q value by which the displacement ductility demand of the structure, and eventually the actual inelastic displacement attained at the top of the structure (at the monitored point), which enables assessment of the extent and intensity of the anticipated damage. The procedure by which to perform these steps is as follows:

First, it can be assumed that the apparent base shear strength at yielding (i.e., at the occurrence of visible cracking in the structure, whether this occurs in the out-of-plane or in the in-plane walls first), is F_y, occurring at a displacement at the point of reference, μ_y. Beyond that point the resistance curve of the structure is assumed to follow a perfectly plastic branch up until the displacement associated with collapse, μ_u, Thus,

$$q = \frac{F_o}{F_y} = \frac{C_1 \bullet C_m \bullet S_e(T)(\frac{W}{g})}{F_y} \qquad (2\text{-}13)$$

If the structure had not yielded, then its elastic displacement demand would be *Δ=(Le/m)·Sd(T) = Γ·Sd(T) where Le* the excitation factor and m the effective mass of the equivalent SDOF system representing the structure and $\Gamma = L_e/m$.

$$\mu = \frac{q^2+1}{2}\text{, where as } \Delta_{inel} = \frac{\mu}{q}\,\frac{\Gamma.S_d(T)}{F_y} = \frac{q^2+1}{2q}\,\frac{\Gamma.S_d(T)}{F_y} \qquad (2\text{-}14)$$

For low period structures such as the common URM stone masonry systems, the equal energy assumption may be used to relate the displacement of the inelastic system, Δ_{inel}, with those of the elastic structure, and therefore, the ductility demand is estimated from Eq. (2-14).

2.12 RELATIVE DRIFT CALCULATION

Relative drift is the angle forming between the final and original chord of any deformed member and is calculated from the ratio of relative displacement divided by the distance. As can be seen in Fig. 15, for continuous systems such as the URM structure with flexible diaphragms, drift ratio is a meaningful index both height wise and in the building plan [27].

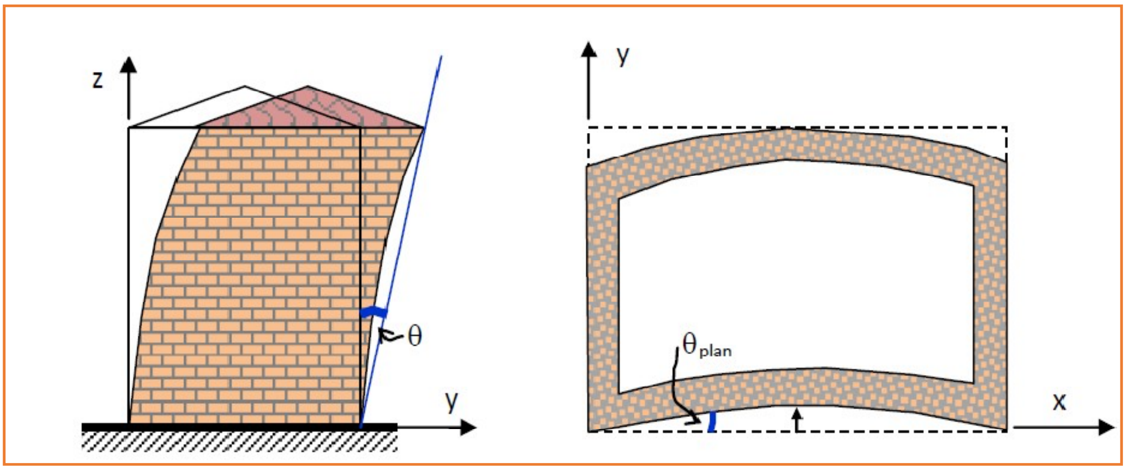


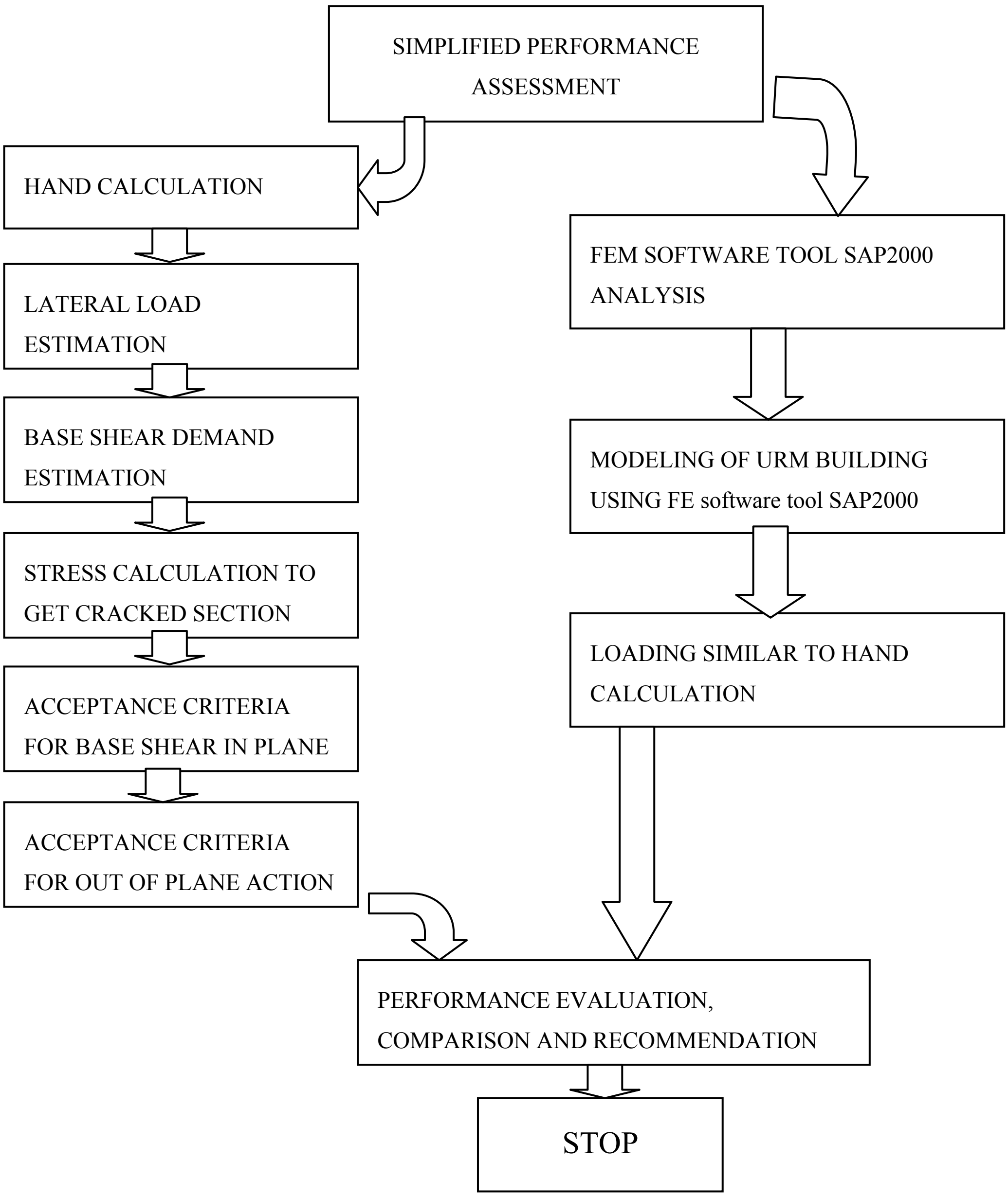
Figure 15 : Definition of Relative Drift Ratios in height and in plan, for assessing the intensity of demand and supply as well as the associated damage level: θ, and θ plan from [27]

PART 3 ANALYSIS

3.1 SIMPLIFIED PERFORMANCE ASSESSMENT PROCEDURE OF EXISTING URM BUILDINGS

Based on the guide lines in literature review in Part 2 the following procedure can be used to carry out the simplified performance assessment on URM buildings in question.

<u>Schematic Representation Of Simplified Performance Assessment Procedures On URM Buildings After Getting Primary Data From The URM Building In Question</u>.



3.1.1 NECESSITY OF SIMPLE AND RAPID APPROXIMATE ASSESSMENT METHOD

The highly brittle nature of URM walls makes their analysis very complex. Due to material non homogeneity each masonry unit with the mortar surrounding it will become the element under investigation. So this is equivalent to saying that in-order to analyze one wall the number of elements needs to be modeled should be equal to the number of masonry units and connecting mortar layer interface. Although this is the ideal approach, the time and effort required to complete assessing one simple URM building may not be acceptable depending on the importance the building under investigation.

A simple and rapid approximate method will eliminate to the lowest possible level both time and energy expenses to solve the given problem. It will also provide relative simpler modeling approach. The main objective of this thesis being to present simplified URM assessment this topic will be demonstrated in detail.

3.1.2 COMPULSORY OUTPUTS EXPECTED FROM RAPID ASSESSMENT

Rapid assessment must be able to provide the following compulsory outputs in order to meet its task of assessing.

I- It has to produce lateral deflection of the structural systems

II- It has to produce the stress distribution in all critical structural members.

These data will provide the basic answers about the level of performance of the structure, which is the main concern of this thesis. They will also give insight to clear decision making on how to proceed with the retrofitting method, cost time and other alternatives such as service limitation or demolition in worst-case scenario.

Sometimes it is must to apply detailed micro finite element analysis if the importance of the structure in question is high but the output from such detailed analysis may show unacceptable errors. In such cases, the output from the Rapid Assessment can be used as a rough check.

3.1.3 SIMPLIFIED MODELING OF URM BUILDING STRUCTURES

Analysis procedures presented under this topic are concerned mainly with buildings that have fundamental natural translational period T, in both principal direction of the floor plan that falls below the value of 2Tc. Where Tc is the characteristic period value at the plateau of the design acceleration spectrum ES EN 1998 [9] and a building which may be considered regular in plan and in height so that torsional effects may be considered negligible, continuous walls along the height, adjacent floors on opposite sides of a single wall are at the same height.

In order to obtain the seismic demand from the spectrum an estimation of the fundamental translational period of the building is required. This can be achieved by directly estimating from the acceleration and displacement related with T_c in the design spectrum or alternatively simple empirical approximation from ES EN-1998 [9], can be used.

$$T = C_t.H^{3/4}(sec) \text{ but } T \geq 0.05H^{3/4} (sec); \tag{3-1}$$

C_t value is 0.05 for other structures

For masonry structure value of C_t is given by

$$C_t = 0.075 / \sqrt{A_c} \tag{3-2}$$

$$A_c = \Sigma \left[A_i \text{ x } (0.2 + (l_{wi}/H)^2)\right] \tag{3-3}$$

A_c is the total effective area of the shear walls in the first storey of the building, in m^2;

A_i is the effective cross-sectional area of the shear wall i in the first storey of the building, in m^2;

H is the height of the building, in m, from the foundation or from the top of a rigid basement.

L_{wi} is the length of the shear wall i in the first-storey in the direction parallel to the applied forces, in m, with the restriction that l_{wi}/H should not exceed 0.9.

3.1.4 SIMPLIFIED LINEAR BASE SHEAR DEMAND ESTIMATION OF URM BUILDINGS

Most URM building walls built earlier than year 1935 are large in thickness. This robust morphology gives reserved capacity which can be considered as a source of lateral load resistance other than carrying the minimal axial load from tributary floors and own weight which implicitly satisfies serviceability limit state requirements.

According to ES EN 1996, pp 59, [8] the serviceability requirement is automatically satisfied if lateral resistance requirement is satisfied. Thus it is reasonable to primarily evaluate the ultimate limit state for seismic performance assessment.

The seismic base shear force F_b, for each horizontal direction in which the building is analyzed shall be determined using the following expression from ES EN 1998 (4.5) for rigid diaphragm with lumped mass systems:

$$F_b = S_d(T_i) \bullet m \bullet \lambda \tag{3-4}$$

Where

$S_d (T_1)$	is the ordinate of the design spectrum pp 26 of ES EN 1998 [9], at period T_1;
T_1	is the fundamental period of vibration of the building for lateral motion in the direction considered as expressed in (3-1);
m	is the total mass of the building, above the foundation or above the top of a rigid basement,
λ	is the correction factor, the value of which is equal to: A = 0,85 if $T_1 \leq 2T_c$ and the building has more than two storeys, or λ = 1,0 otherwise.

For the horizontal components of the seismic action, the design spectrum, $S_d(T)$ is defined by the following expressions;

$$0 \leq T \leq T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3}\right)\right] \tag{3-5}$$

$$T_B \leq T \leq T_C : S_d(T) = a_g \cdot S \cdot \eta \cdot \frac{2.5}{q} \tag{3-6}$$

$$T_B \leq T \leq T_C : S_d(T) = \begin{cases} = a_g \cdot S \frac{2.5}{q} \cdot \left[\frac{T_C}{T}\right] \\ \geq \beta \cdot a_g \end{cases} \quad (3\text{-}7)$$

$$T_D \leq T : \; S_d(T) = \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C T_D}{T^2}\right] \\ \geq \beta \cdot a_g \end{cases} \quad (3\text{-}8)$$

Where a_g, *S, T_B, T_C and T_D* are as defined in pp 21 to 22 of ES EN 1998, [9]

a_g is the design ground acceleration on type A ground ($a_g = \gamma_1 \cdot a_{gR}$);

T_B is the lower limit of the period of the constant spectral acceleration branch;

T_C is the upper limit of the period of the constant spectral acceleration branch;

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor;

η is the damping correction factor with a reference value of η= 1 for 5% viscous damping.

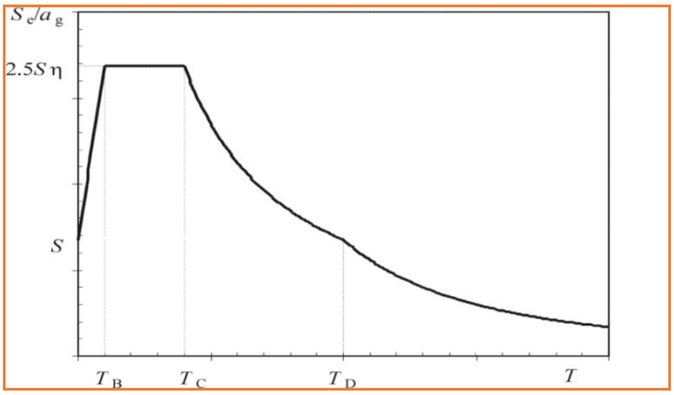


Figure 16: Shape of the elastic response spectrum

Table 4: Values of the parameters describing the recommended Type 1 elastic response spectra

Ground	S	T_B (S)	T_C (S)	T_D(S)
A	1.0	0.05	0.25	1.2
B	1.35	0.05	0.25	1.2
C	1.5	0.10	0.25	1.2
D	1.8	0.10	0.30	1.2
E	1.6	0.05	0.25	1.2

Note: Type 1 table in ES EN 1998 is similar to Type 2 EN 1998:2004.

Using simplified analysis method the URM building is treated as a cantilever box structure. It is also advantageous to neglect the random position of openings while considering the overall response. This lends simpler idealization similar to hollow tubular beam with constant wall thickness t same as the actual thickness of the perimeter walls of the building; but later amplification factors has been applied to account for openings.

The lateral distributed force *F(z)* along the height of the wall from the weight of the wall which is inertial force will be resisted by the base shear F_o, its magnitude can be estimated by the Eq.,

$$F_o = C_1 \cdot C_m \cdot S_e(T) \cdot (W/g)\ (kN) \tag{3-9}$$

$$C_m = \frac{L_e^2}{m}\ ;\ L_e = \int_0^L \phi(x,y,z) \bullet \bar{m}(dx, dy, dz) + \sum_i m_i \phi_i \tag{3-10}$$

$$m = \int_0^L \phi(x,y,z)^2 \bullet \bar{m}(dx, dy, dz) + \sum_i m_i\, \phi_i^2\ (tn) \tag{3-11}$$

Where $S_e(T)$ is the spectral acceleration in m/s^2,

C_1 is the inelastic amplification (≥1), will be 1 for this paper.

C_m is the mass participation coefficient;

$\phi(x,y,z)$ is the fundamental shape of vibration

$\bar{m}$ is the distributed mass of the structure (weight per unit volume of masonry)

m_i is for lumped masses wherever they exist (e.g. floor, roof etc.)

Note: Eq. (3-9) is applicable to distributed mass system along the cantilever wall without diaphragm action whereas (3-4) for lumped mass system with rigid diaphragm action.

For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by the following expressions:

$$0 \le T \le T_B: \quad S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2.5 - 1)\right] \tag{3-12}$$

$$T_B \le T \le T_C: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \tag{3-13}$$

$$T_C \le T \le T_D: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \left[\frac{T_C}{T}\right] \tag{3-14}$$

$$T_D \le T \le 4s: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \left[\frac{T_C T_D}{T^2}\right] \tag{3-15}$$

Generally, for lumped systems, C_m is around 1 for distributed mass systems, 0.8 for higher structures; for distributed mass systems, C_m in the fundamental mode is far less, around 0.6. For example, in the case of a tower with uniformly distributed mass, excited at the base, the mass participation coefficient is 0.58.

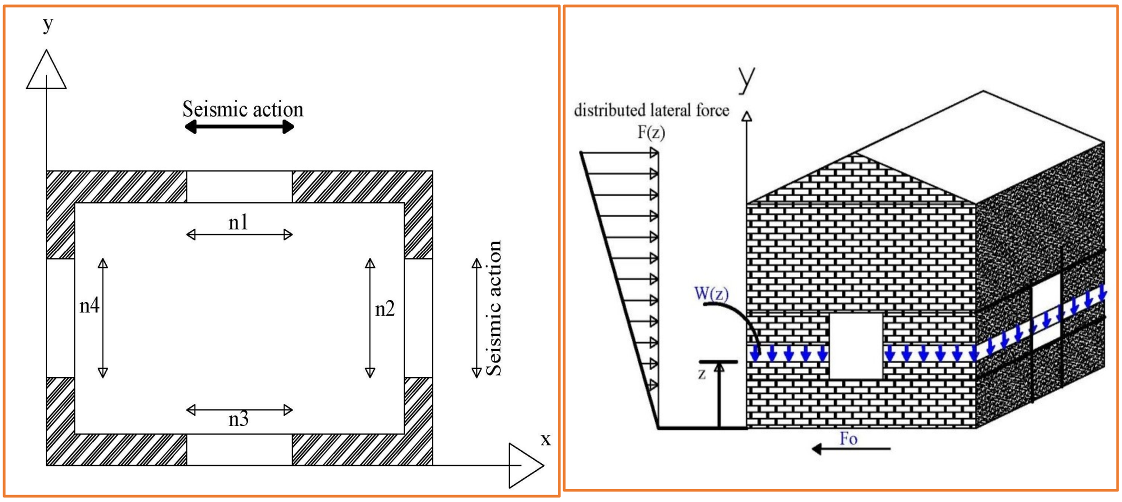


Figure 17: Seismic loading on a traditional URM structure

The distributed lateral forces acting at a distance z from the base, $F(z)$, result in internal shear and flexural moment resultants at any building cross section. A linear distribution is assumed. Thus, at distance z from the base, the lateral force $F(z)$ is related to the total base shear, F_o, from:

$f(z) = f(z|H)\cdot(z/H)$, with $f(z|H)=f(H)=2F_o/H$. From equilibrium requirements it follows:

$$F(z) = (H-z)\,(f(z) + f(H))/2 \text{ (kN)} \quad (3\text{-}16)$$

$$M(z) = f(z)\frac{(H-z)^2}{6} + f(H)\frac{(H-z)^3}{3} \quad \text{(kN-m)} \quad (3\text{-}17)$$

The most critical section is at the ground level (z=0); from Eq.s (3-16) and (3-17) yield the base shear value, F_o and overturning moment, M_o, of the building.

For the idealized tubular cantilever, the plane section assumption is applied to determine normal and shear stresses through the wall thickness of the building. During seismic action the two side walls parallel to seismic action will be subjected stresses $\boldsymbol{\sigma}_{P1}$ and $\boldsymbol{\sigma}_{P2}$ on the cross sectional area A_w (m^2) of the walls in the plan, and with the flexural-section modulus $\Omega_w = [I/(l_a/2)]$ of the building cross section in plan, normal stresses are calculated at the extremes of the building plan, as schematically presented in Fig. 18.

$$\boldsymbol{\sigma}_{P1} = -\frac{W}{A_w} + \frac{M_o}{\Omega_w} \;\&\; \boldsymbol{\sigma}_{P2} = -\frac{W}{A_w} - \frac{M_o}{\Omega_w} \quad \text{(MPa)} \quad (3\text{-}18)$$

Where W(z=0) is the overbearing self-weight, M_o = Flexural moment at onsidered level.

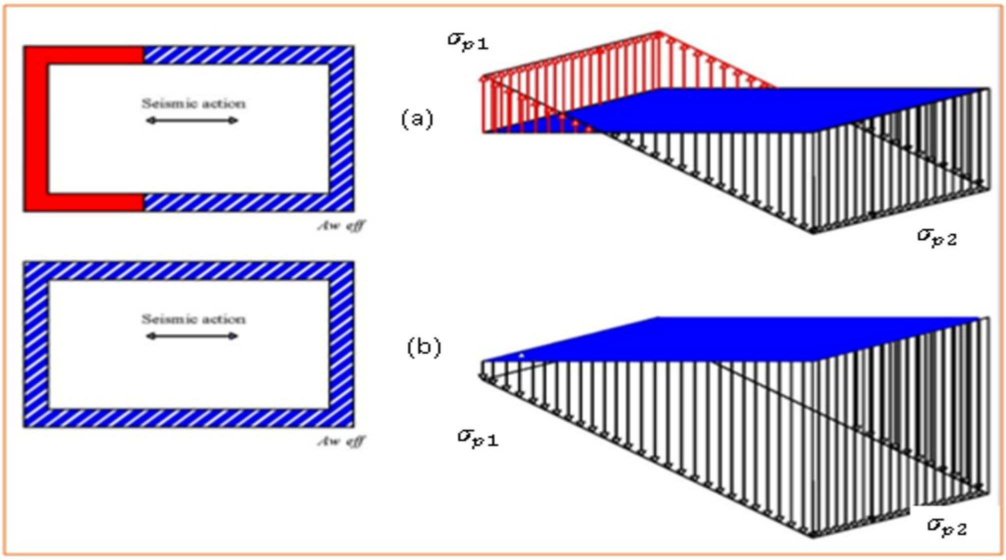


Figure 18: Schematic representation of plane stress for the plan of structure

where $\sigma_{p1}$and σ_{p2} are stresses distributed over plan area of wall section where red solid line shows tension zone and blue hatched lines effective compression zones. (a) Normal tension induced due to insufficient overbearing pressure from axial compressive load so partial effective area. (b) Fully effective area due to higher overbearing pressure larger than internal resistance.

From Eq. (3-9)

$$F_o = C_1 \bullet C_m \bullet S_e(T) \bullet (W/g)\ (kN) \tag{3-19}$$

$$= H \times (\ f(H)/2) = (\ 2\ C_1 \bullet C_m \bullet S_e(T) \bullet (W/g)\ )/H\ (kN)$$

$$M_o = f(H)\ (H^2)/3 = (2/3)\ C_1 \bullet C_m \bullet S_e(T) \bullet (W/g) \bullet H\ \ (kN\text{-}m) \tag{3-20}$$

The term *W(z)* is the total overbearing gravity load made up of the roof weight (W_r) and own weight of wall from top of wall to bottom level. During considering weight of roof and flexible diaphragm floor the load will be transferred to walls according to their tributary distribution.

Depending on the level of rigidity of the load distribution can be decided as per the recommendation in (FEMA-451B) for flexible and as two-way concrete slab using coefficient method if rigid diaphragm exist. The weight of the wall may be calculated multiplying the unit weight of masonry by the cross-sectional area of the wall without allowing the openings to get simplified calculation effort.

By assuming the seismic action, parallel to one critical side of the perimeter wall with rectangular shape a stress distribution will be produced. For large axial loads due to overbearing pressure from walls the neutral axis will be located outside the building which increases the shear resistance.

If the equilibrium condition provides the location of axial load resultant within the building but outside the kern then the tensile stresses will occur, which means the effective shear resistance contribution from each walls will be reduced leading to instability. Such situation will raise important condition which is a criterion limiting the magnitude of the earthquake acceleration, a_g, that may be resisted by the building without local failure will be related the magnitude of the nominal tensile strength of masonry f_{tm}, as follows [27]:

$$\boldsymbol{\sigma}_{P1} = f_{tm}, = W\left(-\frac{1}{A_w} + \frac{C1\bullet Cm\bullet Se(T)\bullet}{g\Omega_w}\frac{2}{3}H\right) = W\left(-\frac{1}{A_w} + \frac{C1\bullet Cm\bullet a_g\bullet 2.5\eta Se(T)\bullet}{g\Omega_w}\frac{2}{3}H\right) \quad (3\text{-}21)$$

$$\boldsymbol{\sigma}_{P2} = -W\left(\frac{1}{A_w} + \frac{C1\bullet Cm\bullet Se(T)\bullet}{g\Omega_w}\frac{2}{3}H\right) = -W\left(\frac{1}{A_w} + \frac{C1\bullet Cm\bullet a_g\bullet 2.5\eta Se(T)\bullet}{g\Omega_w}\frac{2}{3}H\right) \quad (3\text{-}22)$$

From Eq. (3-21) the limit for the level of ground earthquake acceleration will be determined, which demarcate the tension zone at the base of structure is defined by:

$$a_g \leq 0.6\left(\frac{g\Omega_w}{C1.\,Cm.\eta\; s_e(T)H}\right)\left[\frac{f_{tm}}{W} + \frac{1}{A_w}\right] \text{m.s}^{-2} \quad (3\text{-}23)$$

Another alternative assessment test is definition of the eccentricity of the vertical load (structural weight) at any critical level of the structure, z. Setting the tensile stress $\boldsymbol{\sigma}_{P1} = 0$ at the most critical section at z=0, where eccentricity $e = M_o/W$. The limiting eccentricity e_{lim} is calculated by considering arbitrary rectangular shaped building in plan with dimension of l_x and l_y with wall thickness t, subjected to ground shaking in the x-direction. It is required that $e \leq e_{lim}$:

$$-\frac{W}{A_w} + \frac{M_o}{\Omega_w} = 0 \Rightarrow e_{lim} = \frac{M_o}{W} = \frac{\Omega_w}{A_w} \approx \frac{1+3\left(\frac{l_y}{l_x} - 2\frac{t}{l_x}\right)\left(1-\frac{t}{l_x}\right)^2}{6\left(1-\frac{t}{l_x}\right)\left(1+\frac{l_y}{l_x} - 2\frac{t}{l_x}\right)} l_x \text{ (m)} \quad (3\text{-}24)$$

$$\text{Where } e = a_g \bullet \frac{5}{3} \bullet C_1 \bullet C_m \bullet S \bullet \eta\left(\frac{H}{g}\right) (m) \quad (3\text{-}25)$$

For usual wall thickness values and plan aspect ratio values (ℓ_y/ℓ_x), Eq. (3-24) yields the values shown for easy reference in Table 6, where the limiting eccentricity is given as a fraction of the respective dimension of the structure in the direction of seismic action. With these values for e_{lim}/ℓ_x, the acceleration limit beyond which a part of the building will suffer from direct tension is obtained by re-placing in Eq. (3-25):

$$\frac{e_{lim}}{l_x} = \frac{a_m}{g}\cdot\frac{5}{3}.C_1.\,C_m.\,S.\eta\left(\frac{H}{g}\right)(m) \Rightarrow \frac{a_m}{g} = \frac{0.6}{\eta c_1 c_m S}\frac{e_{lim}}{l_x}\frac{l_x}{H} \quad (3\text{-}26)$$

Where $\frac{e_{lim}}{l_x}$ can be found from Table 6 and $\frac{l_x}{H}$ from measuring the building dimension. Note that the peak tolerable ground acceleration reduces with increasing sleanderness of the structure (H/ℓ_x ratio).

Table 5: Calculated limits for e_{lim}/l_x

$\frac{l_y}{l_x}$ \ $\frac{t}{l_x}$	0.05	0.1	0.15	0.2
0.5	0.261	0.246	0.234	0.226
1	0.317	0.303	0.290	0.280
2	0.372	0.355	0.340	0.326

3.1.5 ACCEPTANCE CRITERIA: BASE SHEAR STRENGTH (DEMAND AND CAPACITY)

As indicated in the formula indicated in (2-1), masonry is treated as homogenized material with average mechanical properties both from masonry unit and mortar. The estimated shear demand is compared with the masonry wall shear strength item 2.5.10 of this thesis. Shear strength is developed through frictional and cohesion mechanisms in the presence of shear locking, clamping action of overbearing compression (if it exists). As depicted in Fig. 18 (a) the effective wall area which can resist the shear stress acted is located in the compressed zone. Note that the neutral axis where the tension zone ends and compression begins is dependent on the height of the building and the lateral force acting on it. Thus the average shear stress demand within the $A_{w.eff}$ wall area is obtained from:

$$\tau_{avg} = \varepsilon \bullet \frac{V_o}{A_{w,eff}} \tag{3-27}$$

Or, more generally,

$$\tau_{avg,amp}\,(z) = \varepsilon(z) \bullet \frac{V(z)}{A_{w,eff}(z)} \quad \text{amplified shear demand at any height} \tag{3-28}$$

In Eq. (3-26) coefficient ε(z) is an amplification factor, accounting for the reduction in the actual, instead of taking the gross shear area, along $A_{w,eff}$, the areas will be considered where there is a presence of openings; thus, ε is the ratio of $A_{w,eff}$ to the minimum effective wall area, $A_{w,eff}^{min}$ in the critical storey. The amplified shear demand, $\tau_{avg,amp}$, should be less than shear resistance V_{Rd} else failure will occur by diagonal cracking of the adjacent piers.

Acceptance criterion is that shear demand should not exceed the shear strength of unreinforced masonry (MPa), v_{Rd}, or f_{vk} Eq. (2-5) which is estimated according with a Mohr-Coulomb type frictional law.

3.1.6 ACCEPTANCE CRITERIA FOR OUT-OF-PLANE ACTION

In the absence of rigid diaphragm in URM structures out of plane action can lead to failure. Hence acceptance criterions as in [27] were formulated to estimate the influences of normal pressure which bends the walls out of plane. The magnitude of this normal lateral pressure *is* $p(z) = (z/H)S_e(T_1)tW/g(kN/m^2)$ acting throughout the wall surface from the base to the roof level where W is self weight per unit volume of the walls (in kN/m^3).

Acceptance criteria for this problem are related to the moments M_y *and* M_z (the subscript corresponds to the orientation of the strip considered) which are compared with the cracking moments of the wall; the peak ground acceleration tolerated prior to cracking may be estimated by setting the peak moment equal to the cracking strength. ES EN 1996 provides the approximate values using the same concept, higher values of peak ground acceleration that may lead to cracking over prescribed fractions of the total wall area may be determined in the process of assessment. The cracking moment is estimated from:

$$M_{z,cr} = (f_{xk1} + w.(H - z))\frac{bt^2}{6} \qquad (3\text{-}29)$$

$$M_{y,cr} = f_{xk2}.\frac{bt^2}{6} \qquad (3\text{-}30)$$

Where f_{xk1} and f_{xk2} are the flexural strength values for failure plane parallel and normal to bed joints, respectively as stated in ES EN 1996 pp34, [8], where b is strip width usually 1m, and t is the thickness of the wall.

At any level, the moments in the z-strips should be amplified locally by the ratio L_y/ℓ_y, where ℓ_y is the dimension of the plan minus the breadth of the openings. If the strength of the orthogonal walls suffices to support a lower pressure, $p'(z)<p(z)$ than what would be estimated for the applied design value of a_g, then it follows that the assessed base

shear strength of the building should be scaled down to the reduced ground acceleration thus estimated.

3.2 RAPID NUMERICAL ANALYSIS A CASE STUDY FOR SAMPLE URM BUILDING

3.2.1 INTRODUCTION OF THE SAMPLE BUILDING

The sample was selected based on purposeful sampling method. The sampling is selected based on the following criteria.

- I- The plan layout should be approximately rectangular.
- II- The lateral force resisting members should not have RC columns, Steel frames or other elastic members which alter the strength of the URM wall system. Timber lacing is allowed but the analysis should consider the strength contribution from Timber lacing.
- III- Other properties masonry unit properties for URM such as blocks and mortar should be as per ES EN 1996 [8].

The plan and elevation of the sample building is located in Fig.19 and Fig.20 with details of dimensions wall thicknesses, width, length and height of wall parts. The sample URM building selected in this thesis is historical building, which dictated the only test method used is non-destructive test methods.

The sample building is located in Arada Sub-city of Addis Ababa 47^{o} 36’ 18” E 99o 94’ 80” N constructed in 1924. serving as major part of Yekatit Asrahulet Hospital and currently the building is actively giving service.

The walls are made of Trachytic Dressed Natural stone masonry with adobe (mud) mortar. The building consist two stories with a total height of 8 meters where each story being 4 meters. The first story walls thickness is 66cm and the second floor 55cm. A timber suspended floor with timber truss with CIS is used as roof system is made of Thid (Junipers Procera).

The building is oriented at its longest part 21.77m stretched from East to West and 5.4m width toward North South direction accompanied by 1.95m hanging verandah shade both North and South direction.

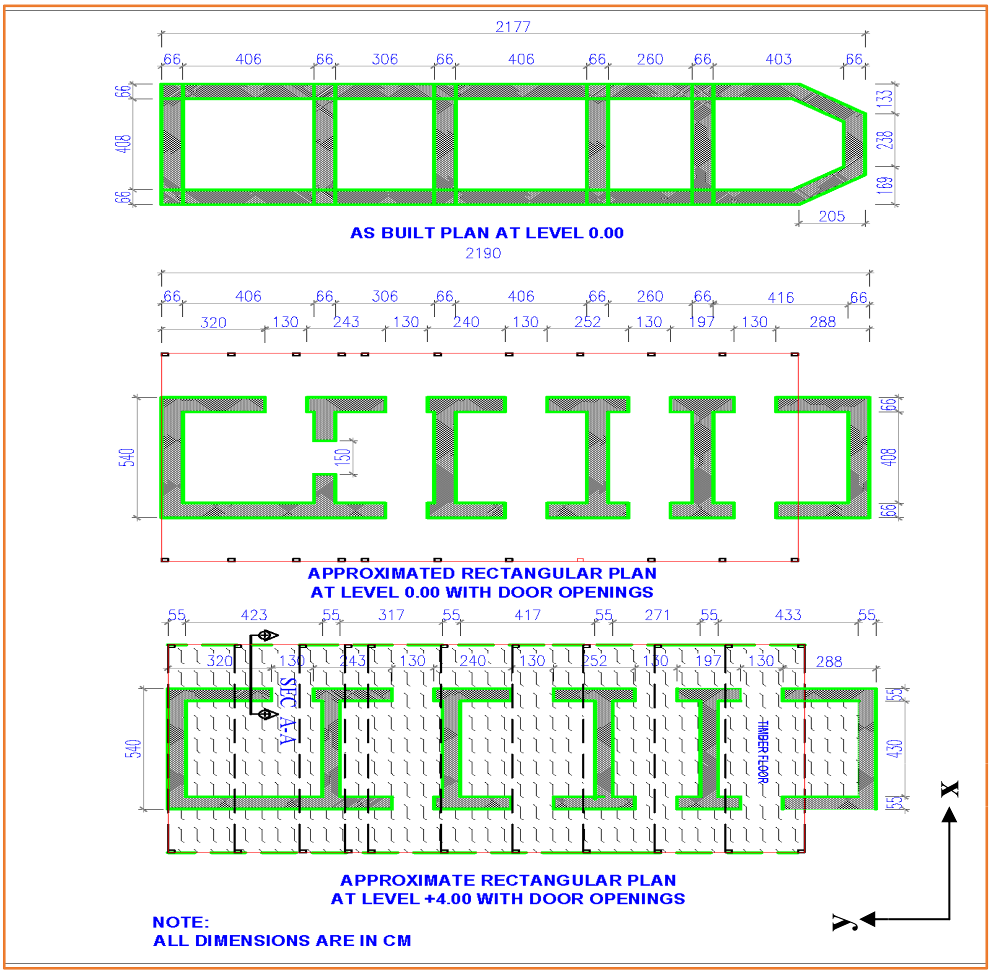


Figure 19: Plan of the sample building with relevant dimensions

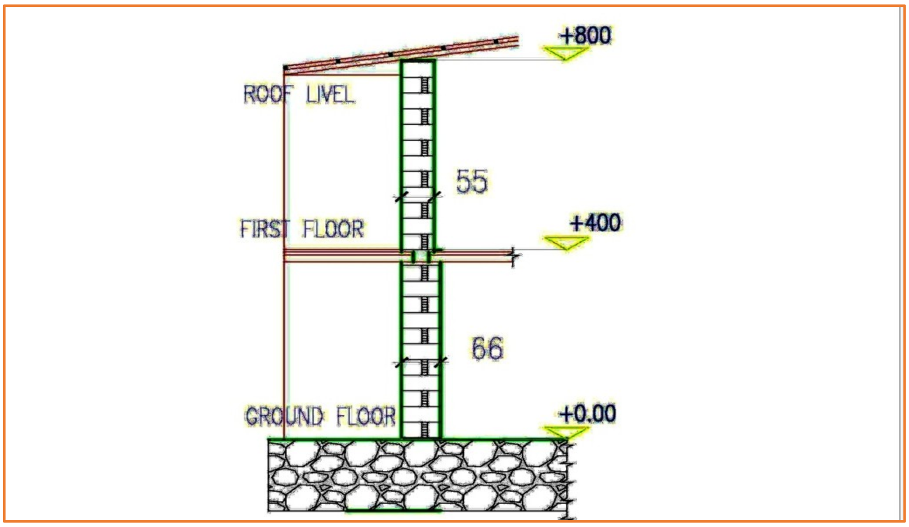


Figure 20: Elevation section of the sample building with relevant dimensions

3.2.2 NUMERICAL ANALYSIS FOR THE SAMPLE BUILDING

a) Serviceability requirement:

In this part of the analysis the diaphragm effect of the timber floor and truss due to flexible nature of the connection of timber structure. Thus it is assumed that the wall to act as cantilever tube wall.

The serviceability requirement: by dividing the value of the effective height, h_{ef}, by the

$$t_{ef} = \rho_t t \tag{3-33}$$

where: t_{ef} is the effective thickness;

ρ_t is a coefficient obtained from, pp 69 of ES EN 1996, [8]; = 1;

t is the thickness of the wall; Since: thickness of wall is 55cm for story 2 walls, 66cm for story 1 walls, and the ratio of pier and actual wall thickness is 1, $\rho_t = 1$,

hence taking the smallest t; t_{ef} = 1x55 = 55cm

The slenderness ratio: from h_{ef}/t_{ef}:

where $h_{ef} = \rho_n h$ and ρ_n=1 from Eq.s in (pp 67 of ES EN 1996)

taking the clear height to box shaped assumed cantilever wall as 8 meter

h_{ef} = 8x1 = 8m, thus h_{ef}/t_{ef}

= 8/0.55 = 14.55 << 27

Serviceability requirement of the sample building under vertical compression load is satisfied

b) Material properties:

Masonry Unit Compressive Strength(f_b): The sample building is said built from stone masonry thus the compressive strength of units (f_b) should be found laboratory tests but due the historical value of the structure destructive tests is not allowed thus Schmidt hammer test is used to evaluate f_b:

From Appendix B-1; it is found that f_b= 41.3MPa

Characteristic Mortar Compressive Strength (f_m):

Another non destructive test used is Scratch Index [19] which resulted from scratch test using finger nail and the test showed to be soft level of hardness. Hence, due to the adobe mud type of mortar the lower characteristic mortar compressive reading prediction is selected;

Thus from Table 4, f_m = 1.0MPa.

Note: This value is much less than specified in ES EN 1998 [9] where it should be 5MPa as minimum requirement.

Characteristic Compressive Strength of The Masonry(f_k):

Using Eq. (2-2) and reading from ES EN 1996 for the value of (k = 0.45) for Natural stone and Group 1, thus:

$$f_k = 0.8 k f_b^{0.7} f_m^{0.3} = (0.8x0.45)x41.30^{(0.7)}x1^{(0.3)} = 4.86931MPa. \quad (3\text{-}34)$$

Modulus of Elasticity of Masonry Wall (E): Using Eq. (2-3)

$$E = 1000f_k = 1000x4.86931MPa = 4.86931GPa \quad (3\text{-}35)$$

Shear Modulus of Masonry (G): from Eq. (2-4)

$$G = 0.4E = 0.4x4.86931GPa = 1.947724GPa \quad (3\text{-}36)$$

Poisson's ratio ($\boldsymbol{\nu}$): from Part 2.5.9 = 0.25 (3-37)

The Characteristic Shear Strength of Masonry (F_{vk}): is dependent on vertical axial compressive pressure and from Eq. (2-5) and using the recommended f_{vko} initial shear strength of masonry value from pp 37 of ES EN 1996 $f_{vko} = 0.1MPa$.

Combination of the seismic action from pp 28 of ES EN 1998 [9]:

$$\sum G_{k,j} \text{"} + \text{"} \sum \psi_{E,i} \, Q_{k,i} \quad (3\text{-}38)$$

Where: $G_{k,j}$ are the dead load; for hospital Category A

$Q_{k,i}$ = 2.0 kN/m^2 the variable load; from ES EN 1998 4.2.4 Eq.(4.2)

φ =1 for roof, (EN 1990.2002)

ψ_2= 0.3 for category A.

Thus, Eq. 3-38 may be written as:

$$\sum G_{k,j} + 0.6kN/m^2 \qquad (3\text{-}39)$$

Consider one stone masonry wall having average thickness for story (1 and 2),

(55+66)/2 = 60.5cm with a total height of 8m wall assuming 25% adobe (mud) mortar and 75% trachytic stone,

Masonry unit weight is:

= (0.75x26+0.25x19) =24.5kN/m^3.

Maximum tributary loading from timber floor and timber roof is calculated as follows:

From duopitch roof loading summary Appendix D loading

G_k = 0.64kN/m^2

From suspended floor section detail and using unit weight data from EBCS 1 1995, [17]:

2mm pvc tile	(16kN/m^3),
5cm lime mortar bedding	(19 kN/m^3),
5cm timber sheathed layer	(7.5 kN/m^3),

taking unit weight of 8x7(cmxcm) joist c/c 43cm,

3cm gypsum ceiling	(17 kN/m^3),

= [0.002x16 + 0.05x19 + 0.05x7.5 + (1/0.43)x0.08x0.07x 7.5 + 0.03x17]

= 1.88 kN/m^2

Assuming the seismic action is parallel to x- axis to ward axis 3 wall where tributary area width is 4.22m which makes the average shear resisting from axial overburden is calculated as follows.

Now consider tributary loading of wall on axis 3, it is assumed that flexible diaphragms will transfer the loads 50% to both sides only to walls parallel to the seismic excitation action.

Thus from midpoint of spans axis 2-3/3-4 the length transverse to the seismic action is 4.22m; length parallel to the axis 3 wall including verandah extended 1.95m away from wall end is {(2x1.95)+5.4} = 9.3m; uniformly distributing loads from roof and suspended timber floor then,

Permanent Load G_{ki} *on axis 3:*

From 1st floor: = {(4.22x9.3 – 0.66x12.52)x1.88}/5.4 = 10.79 kN/m (3-39a)

From roof: = {(4.22x9.3 – 0.66x12.52)x0.64}/5.4 = 3.67 kN/m (3-39b)

Load from wall: = {(0.605x8.0)x24.5} = 118.58 kN/m (3-39c)

$\sum G_{k,j}$ = 10.79 + 3.67 + 118.58 = 133.04 kN/m (3-39d)

Variable load as considered in (3-39) where wind load is neglected, tributary variable load becomes from floor:

= {(4.22x9.3 – 0.66x12.52)x0.6}/5.4 = 3.44 kN/m (3-39e)

Substituting values in Eq. 3-39 factored axial load is: = 133.04 + 3.44 =136.48 kN/m.

σ_d = Overburden pressure at ground level: = $136.48 \times 10^3/(1000 \times 660)$

= 0.207MPa.

Now, the characteristic shear strength of masonry for Axis 3 walls, f_{vk}, from Eq. (2-5) becomes:

$f_{vk} = f_{vko} + 0.4\sigma_d$ = 0.1MPa + 0.4x0.207 = 0.183MPa. (3-40a)

Checking the limit: 0.065xf_b = 0.065x41.3MPa = 2.6845MPa >> f_{vk} Ok!

Using the same procedure used for axis 3, the average shear strength of the masonry wall along axis A is calculated as follows.

Permanent Load G_{ki} *on axis A:*

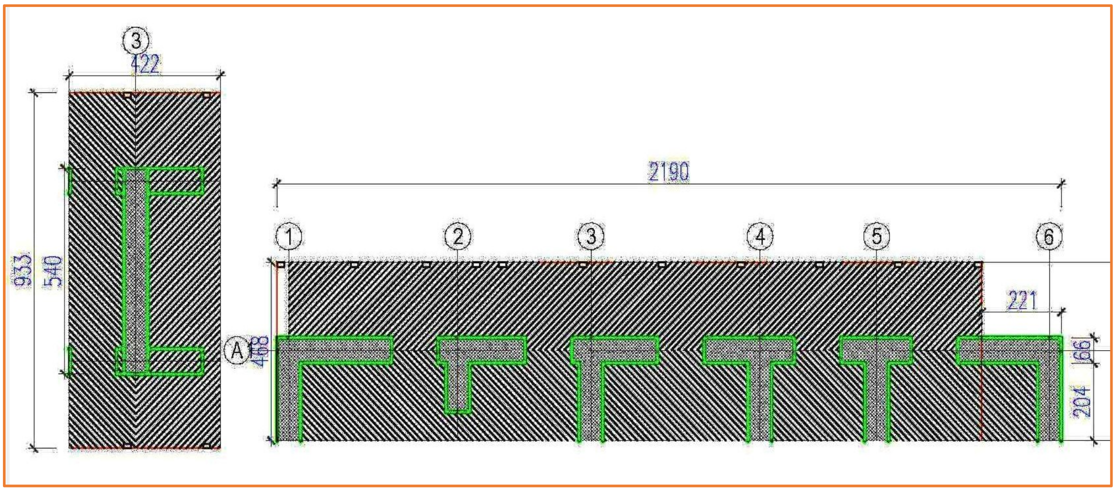


Figure 21: Tributary area plan for axis 3 and axis A direction

1st floor: = {(4.68x21.90 – 0.66x(34.01)-(2.16x2.21))x1.88}/21.90 = 6.46kN/m

roof: = {(4.68x21.90 – 0.66x(34.01)-(2.16x2.21))x0.64}/21.90 = 2.2 kN/m

Load from wall: = {(0.605x8.0)x24.5} = 118.58 kN/m

$\sum G_{k,j}$ = 6.46 + 2.2 + 118.58 = 127.24 kN/m

Variable load as considered in (3-39) where wind load is neglected, tributary variable load from floor:

= {(4.68x21.90 – 0.66x(34.01)-(2.16x2.21))x0.6}/21.90 = 2.06 kN/m

Substituting values in Eq. 3-39 factored axial load is:

= 127.24 + 2.06 =129.30 kN/m.

σ_d = Overburden pressure at ground level:

= 129.30x10^3/(1000x660) = 0.196MPa.

Now, the characteristic shear strength of masonry for axis A direction walls, f_{vk}, from Eq. (2-5) becomes:

$$f_{vk} = f_{vko} + 0.4\sigma_d = 0.1\text{MPa} + 0.4\text{x}0.196 = 0.178\text{MPa}. \quad (3\text{-}40b)$$

Checking the limit: $0.065xf_b = 0.065\text{x}41.3\text{MPa} = 2.6845\text{MPa} >> f_{vk}$ Ok!

c) Demand calculation using elastic response spectrum from ES EN 1998 [9]

Checking $a_{g,urm}$: upper value of the design ground acceleration at the site for use of unreinforced masonry

The ground type is classed as dense or medium-dense sand, gravel or stiff clay, which falls on Ground Type C from Appendix A-2 and A-3. Since Addis Ababa is expecting a magnitude greater than 5.5 M which is pertaining with type 1 response spectra.

Thus the soil factor for Type 1, with ground type C, S = 1.5 which gives $a_g.S = 0.1g\text{x}1.5 = 0.15g$; the value $a_{g,urm} = 0.2g$ threshold is not exceeded thus analysis may proceed to check other parameters also satisfied..

Estimation of seismic hazard: Using Type 1 response spectra, for 5% the acceleration the top of the structure can be estimated from S_e values from the corresponding period in Eq. (3-12) to (3-15) where these Eq.s are dependent on the time period T of the structure system, since the sample structure is 8m in height, T is approximately estimated from the Eq. (3-1) with values in (3-2).

$$T = C_t.H^{3/4}(sec), \textit{ for masonry structures } C_t.= 0.05$$

$$T = 0.05\text{x}8^{3/4} = 0.24sec; \quad (3\text{-}41)$$

Since S=1.5, and $\eta = 1$, using

Horizontal elastic response spectrum, Table 5, the values $T_B(s)$ and $T_C(s)$ are 0.1 and 0.25sec respectively, ($T_B \leq T \leq T_C$), it indicates that the period of the building is in short period range where the shape of the elastic response is controlled by acceleration $S_e(T)$ shall be used to estimate the remaining seismic values.

From Eq. 3-13 $S_e(T) = a_g.S.\eta.2.5 = 0.1g\text{x}1.5\text{x}.1\text{x}.2.5 = 0.375g$ (3-42)

Target displacement $S_{De}(T)$ of the building can be estimated using the Eq. from ES EN 1998 Eq.(3.7)

$$S_{De}(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 = 0.375gx\left[\frac{0.24}{2\pi}\right]^2 = 0.0054m \quad (3\text{-}43)$$

The base-shear force has been estimated from the Eq. (3-4), as per the comments made in EN-1998-3, [10]. The method shall be applied as described as the elastic spectrum $S_e(T_d)$ instead of the design spectrum $S_d(T_1)$. Thus:

$$F_b = S_e(T_i) \bullet m \bullet \lambda \quad (3\text{-}44)$$

Weight calculated from the dimension of sketch taking (3-40) inputs,

Load from 1st floor: ((9.3x21.90)-(68.28x0.66))x1.88 = 298.18kN

Load from roof: ((9.3x21.90)-(68.28x0.66))x0.64 = 101.51kN

Load from wall: = ((2x21.90) + 6x(5.4 – (2x0.66))) x118.58 = 8096.64kN

$W = 298.18 + 101.51 + 8096.64 = 8496.33kN$

values $\lambda = 1$, $S_e(T) = 0.375g$ substituted in (3-44)

$$F_b = 0.375g\text{x}8496.33/g = 3186.12kN \quad (3\text{-}44a)$$

M_o = Overturning seismic moment is (2/3) x (3186.12) x (8)

= 16992.64kN.m (3-44b)

Corresponding eccentricity e = M/W = 16992.64kN.m/8496.33kN = 2m;

Checking the kern, consider flexible diaphragm will not transfer loads to orthogonal direction, thus effective lateral resistance is only from the walls parallel to seismic action which is toward axis 3 direction, with the dimension of 66cm thick by 5.4m length on 6 axes:

limiting kern to be (e= 2m) > ((L_{Short}/6) = 5.4/6 = 0.9m) Not Ok!

The compression zone is: 3L' = 3x(L_{Short} /2 – e) = 3x0.7 =2.1m

Now considering the seismic action is toward axis A on the longer direction, using the dimensions stated for short direction, again assuming the walls parallel to seismic action are only effective due to flexible diaphragm floor assumption:

limiting kern to be (e= 2m) $\leq$ ($(L_{Long}/6) = 21.9/6 = 3.65m$) Ok!

The walls are fully under compression.

Therefore, the critical direction for flexural moment is toward axis 3 (short direction). Calculating Compressive and Tensile Stresses according to Eq. (3-18): Here due to location of eccentricity the area which may contribute to overbearing stress, is reduced.

Thus, its value from walls parallel to the direction of seismic action assumed toward short direction:

$A_{w,tot,Short}$ =6x(5.4)x0.66= 21.383.06m^2,

and I_{short} = area moment of inertia for the parallel walls only:

$$I_{\text{plan,short}} = \frac{(6t)(D)^3}{12} = \frac{(6*0.66)(5.4)^3}{12} = 51.96m^4. \quad (3\text{-}44c)$$

Ω_w = I/y_c = where y_c is the distance from the geometric center = 2.7m,

Ω_w = 51.96/2.7 = 19.24m^3

$$\boldsymbol{\sigma}_{P1} = -\frac{8496.33x10^3}{51.96} + \frac{16992.64x10^3}{19.24} = 0.49MPa \text{ (Tension)}$$

$$\boldsymbol{\sigma}_{P2} = -\frac{8496.33x10^3}{45.06} - \frac{16992.64x10^3}{21.38} = -1.28MPa \text{ (Compression)} \quad (3\text{-}44d)$$

Since tension strength is negligible in URM crack will be assumed to develop on the tension side:

The maximum compressive stress demand is 1.28MPa

This is much smaller than 4.87MPa. So it indicates that the building is safe against local compression failure at the compressed side for seismic action on the short direction.

From the triangular shape of the stress and equilibrium condition the location of the neutral axis formed by compression and tension which is 3.3m from axis B (tension) the remaining section 2.1m is under compression.

It can be seen that from the Fig 22 and Eq. (3-44d) that the stress distribution and plan where red hatch shows:

σ_{p1} = 0.49MPa tension stress and blue hatch σ_{p2} = 1.28MPa compression stress zone.

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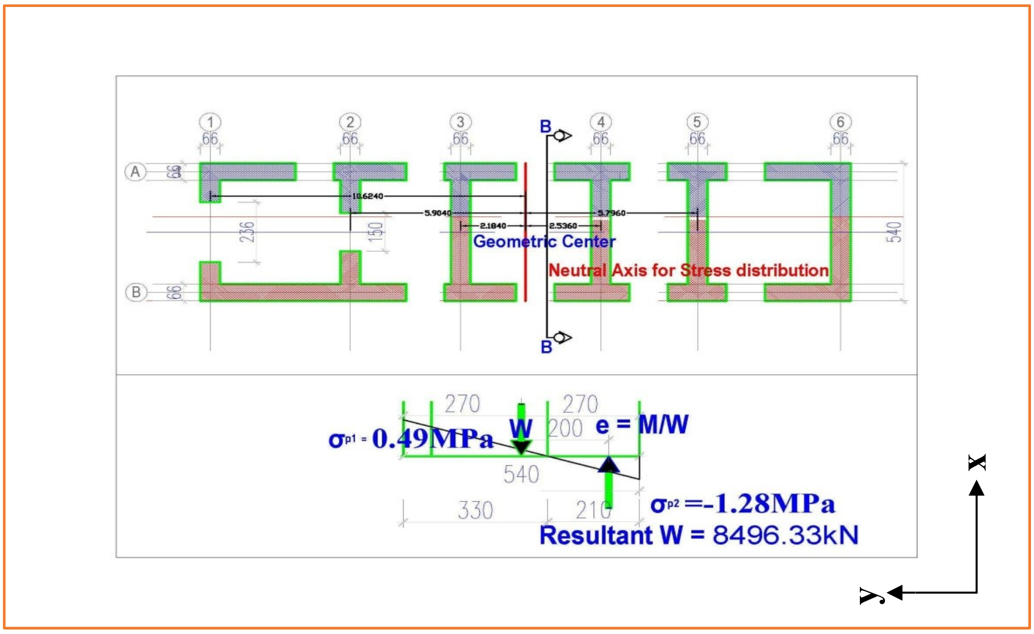

Figure 22: Stress distribution plan and section for seismic action on axis 3 direction

It is also shown that the location of eccentricity and corresponding couple axial resultants from overbearing pressure and reaction from the supporting foundation system. Next, the stress distribution along the long direction is evidently under compression only due to the eccentricity falling within the middle one third of the section.

However due to the presence of many opening for doors and windows, the effective shear resistance along this direction shall be checked. Consequently, the area moment of inertia (I_{long}) is calculated assuming walls only parallel to long direction contribute to lateral force resistant due to flexible floor assumption.

$A_{w,tot,\ Long}$ $=2$x(21.9)x0.605 = 26.5m^2,

and I_{Longt} = area moment of inertia for the parallel walls only:

$$I_{\text{plan,Long}} = \frac{(2t)(D)^3}{12} = \frac{(2x0.66)(21.9)^3}{12} = 1059.1\text{m}^4. \qquad (3\text{-}44c)$$

Ω_w = I/y$_c$ = where y$_c$ is the distance from the geometric center = 10.95m,

Ω_w = 1059.1/10.95 = 96.72m^3

$$\boldsymbol{\sigma}_{P1} = -\frac{8496.33\text{x}10^3}{26.5} + \frac{16992.64\text{x}10^3}{96.72} = -0.15\text{MPa (Compression)} \qquad (3\text{-}44e)$$

$$\boldsymbol{\sigma}_{P2} = -\frac{8496.33\text{x}10^3}{26.5} - \frac{16992.64\text{x}10^3}{96.72} = -0.50\text{MPa (Compression)} \qquad (3\text{-}44f)$$

Ω_w = I/y$_c$ = where y$_c$ is the distance from the geometric center = 10.95m,

Since the walls in all both directions are under compression where the maximum stress is 0.5Mpa which much less than the estimated capacity 4.87Mpa, no crushing will occur and no tension will occur which gives the full shear wall to contribute for shear resistance. Assuming the shear resistance will be contributed by walls on Axis A and B only which area parallel to the assumed seismic action, the shear capacity in this direction will be calculated in (d)

d) Acceptance Criteria: Base shear (In plane),

Having at hand the base shear value in (3-44a), for seismic excitation parallel to axis 3 direction, assuming that shear resistance is provided only by shear walls on parallel to axis 3 direction, the average shear stress demand on area toward this direction is:

Using Eq. (3-27) Base Shear acceptance criteria,

$\tau_{avg} = \varepsilon_{x}\ F_o/A_{w,eff,}$,where $\varepsilon = A_{w,eff}/A_{w,eff}{}^{min}$ since the walls with opening are those on axis 1, & 2, with open length (2.36 +1.5) = 3.86m,

then ε=(6x5.4)/((6x5.4)-3.86) = 1.135 thus from (3.44a) using F_o = 3186.12KN, $A_{w,eff,}$ = 6x0.66x2.1 = 8.32m^2. Thus amplified demand will be:

$$\tau_{avg} = 1.135\text{x}3186.12\text{x}1000/8.32 = 0.435\text{MPa} \qquad \textit{(3-44g)}$$

Checking against the characteristic shear strength of masonry on short direction from *Eq. (3-40a)* $f_{vk} = 0.183MPa$, it indicates that the shear capacity is exceeded 2.38 times.

Checking the Base shear criteria in the long direction axis A and B: The area toward long direction is:

$A_{w,eff}$ = 2x(21.90x0.66) = $28.91m^2$, $A_{w,eff}^{min}$ = 2x(21.9-(9x1.3))x0.66 =13.46m^2, the average shear stress demand on area toward this direction is: Using Eq. (3-27) Base Shear acceptance criteria:,

$\tau_{avg} = \varepsilon_{\cdot} F_o/A_{w,eff}$, ,where $\varepsilon = A_{w,eff}/A_{w,eff}^{\ min}$ since there are 9 openings for doors where $\varepsilon = (28.91)/(13.464) = 2.147$; thus from (3.44a) using $F_o = 3186.12KN$, $A_{w,eff}$, = $28.91m^2$. Thus amplified demand will be:

$$\tau_{avg} = 2.147\text{x}3186.12\text{x}1000/28.91 = 0.237MPa \qquad (3\text{-}44h)$$

This values greater than (0.178MPa Eq.(3.40b)). This indicates the shear capacity is highly exceeded on the longer direction too.

e) Additional Acceptance criteria: Out of Plane Action,

As described in 3.1.6, the acceptance criteria to out of plane action are related to the moments M_y and M_z where these values are moments parallel to the bed joint and perpendicular to bed joint and the subscripts indicate the strips considered. Using Eq. (3-29) and (3-30), the critical cracking moments are estimated as follows. The characteristic flexural strength of masonry, f_{xk1} and f_{xk2}, shall be determined from tests. Where test data are not available values of the characteristic flexural strength of masonry made with general purpose mortar which is mortar with $f_m < 5N/mm^2$, values may be taken from the tables in ES EN 1996 [8]. Hence for

Dimensioned natural stone $f_{xk1} = 0.05N/mm^2$, and $f_{xk2} = 0.2N/mm^2$; where f_{xk1}, and f_{xk2} are as described in item (3.1.6) .

The critical section for z-strip is located at z = 0, at the bed joint. Hence, taking w = 24.5kN/m^3, the height H =8m, b= 1m width of strip thus the critical moments at any wall section are estimated as:

$M_{z,cr} = (f_{xk1} + w.(H - z))\frac{bt^2}{6}$

$= (0.05 + 24.5\text{x}1000\text{x}10^{-9}(8000))\text{x}\frac{1000\text{x}605^2}{6}$ = 17.63kN.m *(3-44i)*

$M_{y,cr} = f_{xk2}.\frac{bt^2}{6}$ = (0.2)x$\frac{1000x605^2}{6}$ = 12.2kN.m *(3-44j)*

Taking 1m width strip from the z(vertical) direction and y(horizontal) direction at critical sections moment calculation is as follows:

p(z=H) = (z/H) Se(T) t w/g = (H/H=1)(0.375g)(0.605)(24.5/g) = 5.56kN/m^2.

Here the assumption is that the walls perpendicular to the excitation are cantilever walls with flexible timber floor and timber truss structure; ignoring the timber floor and trusses. It is also considered that the loads from the roof and floor will be transferred to the walls parallel to the excitation with the understanding that the seismic load resistance is only to be taken care by those parallel walls thus the out of plane deflection from due to the seismic loading perpendicular to the walls.

This can be considered at any wall section on axis A or B between the walls will be carrying only the self-weight excited by seismic action. With this triangular shaped stress maximum at top p(H) and zero at bottom the resulting moment for the strip vertical is;

The M_z*(vertical strip), = (0.5)* x *(5.56)* x *(2/3)* x 8^2

= 118.58kN.m *(3-44k)*

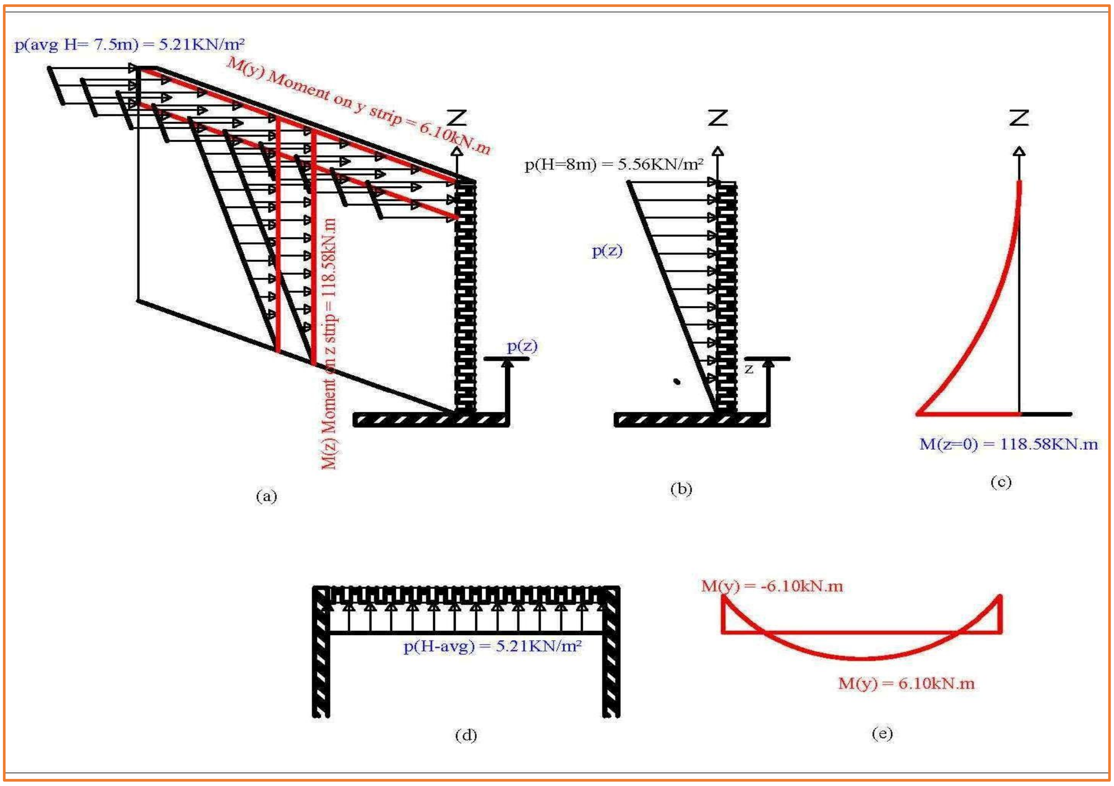

Figure 23: a) 3D representation of z-strip and y-strip, b) Stress distribution on z-strip, c) Bending moment on z-strip, d) Stress distribution on y-strip, e) Bending moment on y-strip.

This indicates that the flexural capacity less than the estimated demand. The ratio Demand Capacity becomes 118.58/17.63 = 6.3 which is far from the behavior factor 1.5 which leads to local failure is sure to occur.

The M_y(horizontal strip), the critical span along axis A and B may be the span between 5 and 6 taken as 4.16m which is the largest clear span. The degree of fixity at the ends of the strips may be reflected through a value λ: where λ=0 implies no restraint to rotation, λ=0.7 corresponds to fixed supports, λ<0.5 refers to partial restraint against rotation [27]. Stress on y strip is critical at z = 8m, taking 1m width strip the trapezoidal shaped stress at 7m is: (7.5/8)x5.56 = 5.21kN/m^2, working out the average stress on this strip for simply supported 4.16m clear span.

$$M_y = (p_{avg}\, l_y^2/8) = (5.21 \text{ x } 4.16^2/8) = 11.27kN.m < (M_{y,cr} = 12.20kN.m) \quad (3\text{-}44l)$$

This can be reduced subjected to a partial restraint factor λ=0.5 then resulting 0.5x12.20 = 6.10kN.m both at support and mid-span.

This indicates that the horizontal strip maximum moment is not ruling the failure mode. As can be seen from the Fig. 23, sketch of bending moment on z-strip and y-strip, the flexural bending moment estimated using equilibrium Eq. is calculated for the sample building walls.

3.3 MATHEMATICAL MODELING OF REPRESENTATIVE URM BUILDING.

3.3.1 MODELING OF SAMPLE BUILDING USING FINITE ELEMENT SOFTWARE TOOL.

Dimension and material property of the sample building is similar as in Part 3.2.1. Finite Element Analysis Software tool, SAP2000 v.18 [24], program has been used for modeling the considered building typologies. The 3D modeling started from the hypothesis of how the walls respond seismic reaction within the simplified assumption in mind that the only lateral force resisting members are assumed to be, those aligned parallel to the excitation force which should act by in-plane action. The orthogonal walls will be responsible only to carry own weight while acting out of plane. The modeling comprises thick shell element to account both in plane deflection of walls parallel to seismic action and out of plane deformation of walls, where the main lateral force resistance is fully allocated to walls parallel to each seismic action.

3.3.2 MODEL DEFINITION

The sample URM building model is created using the software tool SAP2000 v.18 [24], which is commercially available and user friendly tool. Although masonry is known for its material non linearity, the material is assumed homogenous as per empirical formulae provided in ES EN 1996 [8]. The constitutive model is a macro model with given elastic material property given in the same code. Detailed material property definitions are as in Part 3.2.2. It is assumed that the flexural elastic modulus is half the gross section modulus. The shear transfer coefficients are taken to be 0.1.

3.3.3 MODEL VERIFICATION

Prior to using certain software tool for structural analysis it is good practice to verify the model. The most compelling verification option could be testing the equivalent specimen which is modeled for verification. Although this is possible for small structures, it is not affordable to full scale modeling. Here, the scope of the study does not include testing of URM structure; hence it is opted to take base shear values from hand calculation made for unit width URM wall which is discussed in Part 3.2.2 for the similar material properties to represent the actual structural behavior. Then similar replica in-terms of material property and size modeled on SAP2000 v.18 [24], with similar loading cases.

Since performance level of certain building is highly dependent on drift values evaluated at expected stress demand level the model verification is directed to compare drift values obtain from the approaches namely Hand calculation and SAP2000 v.18 [24] modeling as follows.

The lateral displacement of the unit width URM wall which comprises similar material properties as in Part 3.2.2 is calculated using equation from mechanics of material for uniform rectangular cantilever wall acted up on a uniformly distributed with triangular varying seismic load under the assumption of equivalent static loading which is derived from response spectrum analysis.

From simplified URM building idealization the maximum deflections at different critical sections can be estimated using Eq. from mechanics of material Eq. (*3-45*). For the walls parallel to short (x-direction) and long (y-direction) a vertical cantilever wall separate deflection calculations will be tabulated as follows.

In plane deflection of walls will be calculated using the mechanical properties defined in section 3.1 and 3.2 of this thesis. The seismic action will be distributed according to triangular distribution thus the maximum intensity will be twice of the shared base shear per wall. Here the base shear is assumed to be shared according to the tributary area as calculated in Appendix E-1 and E-2 by each parallel wall to the assumed excitation.

$$\Delta_{IP,OP} = (w_o/EIL) \times \left((L^3\ Z^2/6) - (LZ^4/12) + (Z^5/120) \right) \quad (3\text{-}45)$$

Out of Plane deflection of walls: $\Delta_{OP,Total} = \Delta_{IP} + \Delta_{0P}$

After finding total deflection will be multiplied by the amplification factor introduced in Eq. (3-27), since the walls on axis 1 and 2 have openings, the average amplification factor will be

$$\varepsilon = (5.4/(5.4 - (2.63+1.5)/2) = 1.556 \tag{3-46}$$

Seismic direction: Short (x-direction)

$E_{eff} = 0.5E = 2.4\text{x}10^6\ (kN/m^2),$

Note: The E_{eff} value will be further reduced to account for openings for doors and windows in respective x and y direction, using Eq. (3-46) and

$I_{IP} = tL_w^3/12 = 7.94\ (m^4),$

$w_{o,IP} = 2x(axis\ 2, F_b)/8 = 156.89\ (kN/m)$ *...from Appendix E-1*

$I_{OP} = 1t_w^3/12 = 0.018 (m^4),\ \ w_{o,OP} = 5.56 (kN/m)$

In column (5) of Table 7, the in-plane deflection is calculated for 5.4m URM wall free cantilever assumption and found to be 2.36 mm at crest of roof level which is 8m. Whereas, the out-of-plane deflection for 1m width free cantilever wall resulted 46.46mm as shown in column (8), at the same roof level which shows about 94% of the total displacement is due to out of plane only. This entails that the portion of in-plane is not significant. On the other hand, during SAP2000 v.18 modeling similar deflections are obtained while loaded independently using similar material properties and dimensions.

Table 6: Deflection from Simplified Hand Calculation for Short (x-direction)

Seismic direction: Short (x-direction)									
1	2	3	4	5	6	7	8	9	10
Z Height (m)	*E_{eff} = 0.5E (kN/m^2)*	*$I_{IP} = tL_w3/12$ (m^4)*	*Base shear Intensity w_o = 2*($F_b$on axis 2)/8 (kN/m)*	*Δ_{IP} =Eq. (3-45) (mm)*	*$I_{OP} = 1t_w3/12$ (m^4)*	*Own weigh distribution (kN/m)*	*Δ_{OP} =Eq. (3-45) (mm)*	*Δ_{Total} = (5) + (8) (mm)*	*Amplified Deflection due to openings = (ε=1.135)Δ_{Total}*
0.0	2.E+06	7.94	156.89	0.00	0.018	5.56	0.00	0.0	0.00
0.5	2.4E+06	7.94	156.89	0.02	0.018	5.56	0.33	0.4	0.40
1.0	2.4E+06	7.94	156.89	0.09	0.018	5.56	1.31	1.4	1.58
1.5	2.4E+06	7.94	156.89	0.19	0.018	5.56	2.92	3.1	3.53
2.0	2.4E+06	7.94	156.89	0.34	0.018	5.56	5.12	5.5	6.19
2.5	2.4E+06	7.94	156.89	0.52	0.018	5.56	7.86	8.4	9.51
3.0	2.4E+06	7.94	156.89	0.73	0.018	5.56	11.08	11.8	13.40
3.5	2.4E+06	7.94	156.89	0.96	0.018	5.56	14.69	15.7	17.77
4.0	2.4E+06	7.94	156.89	1.22	0.018	5.56	18.61	19.8	22.51
4.5	2.4E+06	7.94	156.89	1.49	0.018	5.56	22.74	24.2	27.50
5.0	2.4E+06	7.94	156.89	1.77	0.018	5.56	26.96	28.7	32.60
5.5	2.4E+06	7.94	156.89	2.04	0.018	5.56	31.14	33.2	37.67
6.0	2.4E+06	7.94	156.89	2.31	0.018	5.56	35.16	37.5	42.52
6.5	2.4E+06	7.94	156.89	2.55	0.018	5.56	38.86	41.4	47.00
7.0	2.4E+06	7.94	156.89	2.76	0.018	5.56	42.09	44.8	50.90
7.5	2.4E+06	7.94	156.89	2.93	0.018	5.56	44.68	47.6	54.04
8.0	2.4E+06	7.94	156.89	**3.05**	0.018	5.56	**46.46**	49.5	56.20

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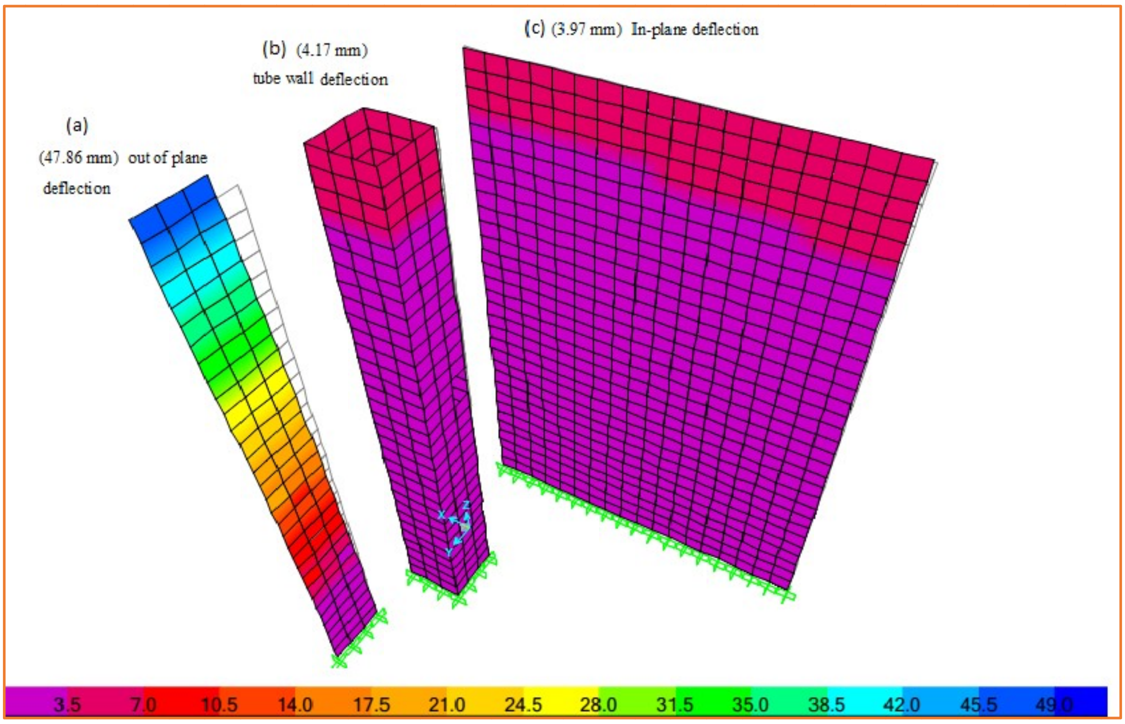


Figure 24: SAP2000 v18 output for deflection of a) loaded for out-of-plane response: 1m width, b) similar loading as in (a) but tube structure 1m x 1m center to center measured c) loaded for in-plane response 5.5m length and all heights are 8m high.

As can be seen from the hand calculation analysis as in Table 7, the in-plane and out-of plane maximum displacement at roof level which is at 8m above ground, found to be 3.05mm and 46.46mm respectively. On the other hand, the output from SAP2000 v.18 [24], shown in Fig. 24, indicates that the in-plane and out of plane found to be 3.97 mm and 47.86 mm respectively. This indicates that the discrepancy of out-of-plane displacement is in the order of 4%. From the observed results it can be verified that the SAP2000 v.18 [24], software tool can be used to model analyze URM individual walls to estimate out-of-plane displacements.

However, care should be taken during the connecting four sided walls. Due to the box nature of connected walls the second moment of area of the connected walls is far larger than individual walls as shown in Fig 24 (b). This will significantly increase the calculated capacity of four side connected walls as opposed to the governing assumption which this thesis is based that walls parallel to the seismic excitation are responsible for carrying the lateral load and walls orthogonal to the seismic direction are carrying own weight only. The reason for this condition is explained earlier which is due to the absence of rigid diaphragm at floor and roof level.

Therefore, it is required to accommodate a method of modeling which gives reasonable out-of-plane displacement while modeling full scale URM building modeling. This is achieved by careful observation of the out-of-plane displacement of 1m width wall and 1m x 1m tube structure modeled in Fig. 24 (a) and (b).

Considering Eq. (3-45), the displacement at roof level is inversely proportional to the EI value, these values are assumed to be fairly constant along the height of the wall. Thus reducing the tube section property by the ratio displacement calculated in Fig. 24. The loading magnitude and direction in each case are similar. The out-of-plane displacements observed are 47.89 mm and 4.17 mm for plane wall and tube wall respectively. The resulting ratio is thus 47.89/4.17 = 11.48 times. Dividing the section property of tube shaped wall results similar out-of-plane displacement while the in-plane displacement increased in the same ratio. Since, more than 93% of the information to decide the performance level of URM building comes from the out-of-plane displacement the reduction of section property or stiffness of the wall element will give reasonable out-of-plane displacement information when applied on actual URM building.

3.3.4 MODELING USING SAP2000 V.18 SOFTWARE TOOL.

The geometric shape of the sample building is created on grid making to match the overall structural shape of the walls. URM wall is best represented by thick shell element thus full height cantilever tube like plane structures created to account both in-plane and the out of plane deflections.

Due to large uncertainty of the connection between the timber structure and walls, and moreover due high flexibility bowing effects of these timber floor and roof structure no timber stiffness is considered in 3D modeling. The wall section property data is similar to hand calculation made Part 3.2. Lateral loading is based on the tributary area where the base shear is shared among the walls parallel to excitation direction, and on the orthogonal direction loads only due to own weight of the wall is applied.

Load combination defined according the rules in ES EN 1991 [7] is displayed in Appendix C-1. Meshing is created by considering the masonry units height about 25cm high and 35cm width in most cases. Creating openings for door and windows is made with close approximation of the shape of openings. Nodes missing other nodes during meshing checking refined meshing created to connect all nodes to their respective vicinity nodes.

Due to the absence of rigid diaphragms both on 1st floor level and roof level accidental eccentricity is not fully considered according to code stipulation where (e_a = 5%) of width of structure parallel to seismic action. In order to include eccentricity in the load combination 5% of each parallel walls width multiplied by the distributed base shear component. Thus, the calculated moment is applied at 2/3rd height of the wall as concentrated moment alternatively, as per the load combination Eq.s shown in appendix C-1.

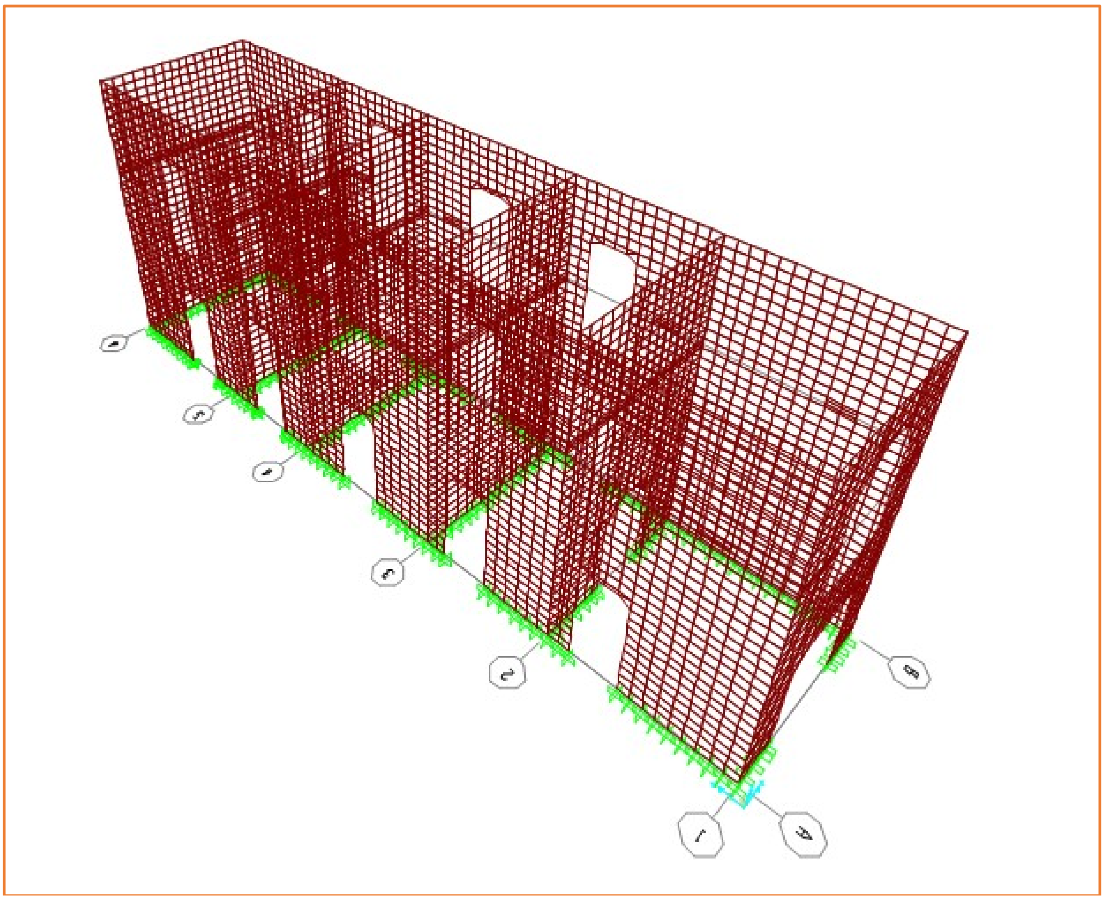


Figure 25: SAP2000 v18, three dimensional modeling.

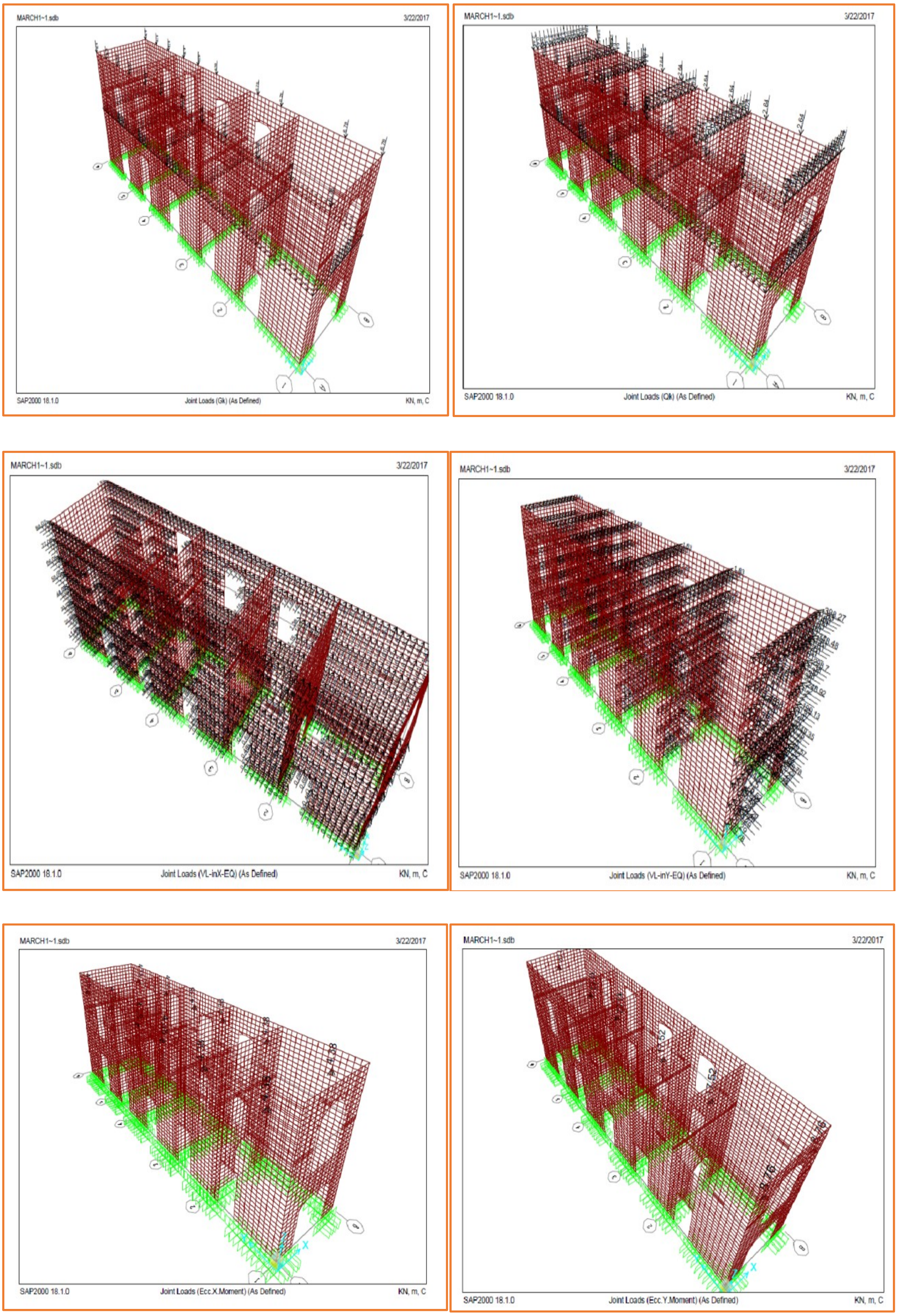


Figure 26: Loading pictorial view in 3D from SAP2000 as modeled

PART 4. RESULT

4.1 RESULT FROM VISUAL INSPECTION, CODE SUGGESTIONS AND HAND CALCULATION

4.1.1 RESULT FROM VISUAL INSPECTION WITH CODE STIPULATION

According the inspection made the mortar used to connect the masonry units is mud (adobe) type Fig. 27. The compressive strength estimated by scratch index method was found to 1MPa which much smaller than 5Mpa. It is also observed that the suspended floor at 1st floor level timber with no sufficient connection which means no concrete beam or no steel connector used.

(a) Timber verandah (b) Mud mortar

Figure 27: Visual inspection data collected from the sample case study building.

(c) Floor level without concrete beam (d) Typical floor timber connection

In Fig. 27 (a) it can be seen that the mortar used is mud (abobe) mortar its strength is definitely smaller than cement mortar.

Although the sample building was not designed for currently recognized seismic load it is important to list specific rules which are violated from the rapid analysis and visual or test data collections to relate the effects of missed rules and governing failure modes. Accordingly the critical items violated are listed in Table 8.

Table 7: List of specific rules violated from ES EN 1998

Violated Specific rules For Masonry Buildings ES EN 8	
Description	**ES EN -1998**
Minimum mortar strength parallel to bed face $f_{m,min}$ is 5MPa	9.2.3 (1)
Connection between the floors and walls shall be provided	9.5.1 (2)
Effective diaphragm action shall be satisfied	9.5.1 (3)
Horizontal concrete beams or , alternatively, steel ties should be placed in the plane of the wall at every floor level and in any case with a vertical spacing not more than 4m. These beams or ties should form continuous bounding elements physically connected to each other	9.5.2 (1)

4.1.2 RESULT FROM HAND CALCULATION

Relative drift ratio from Table 9 here below:

Maximum amplified lateral displacement at roof level 8m above ground = 56.20mm

Relative drift ratio in height = (56.20/8000) x 100% = 0.70%

Table 8: Drift Estimation from Simplified Hand Calculation.

1	2	3	4	5	6	7	8	9	10	11
Z Height (m)	$E_{eff} = 0.5E$ (kN/m^2)	$I_{IP} = tL_w3/12$ (m^4)	Base shear Intensity $w_o = 2*(F_b/6)/8$ (kN/m)	Δ_{IP} =Eq. (3-45) (mm)	$I_{OP} = 1t_w3/12$ (m^4)	Own weigh distribution (kN/m)	Δ_{OP} =Eq. (3-45) (mm)	$\Delta_{Total} = (5) + (8)$ (mm)	Amplified Deflection due to openings = (ε=1.135)Δ_{Total} (mm)	**Relative Drift ration (%)**
0.0	2.4E+06	7.94	156.89	**0.00**	0.018	5.56	0.00	0.0	0.00	**0.00**
0.5	2.4E+06	7.94	156.89	**0.02**	0.018	5.56	0.33	0.4	0.40	**0.00**
1.0	2.4E+06	7.94	156.89	**0.09**	0.018	5.56	1.31	1.4	1.58	**0.02**
1.5	2.4E+06	7.94	156.89	**0.19**	0.018	5.56	2.92	3.1	3.53	**0.04**
2.0	2.4E+06	7.94	156.89	**0.34**	0.018	5.56	5.12	5.5	6.19	**0.08**
2.5	2.4E+06	7.94	156.89	**0.52**	0.018	5.56	7.86	8.4	9.51	**0.12**
3.0	2.4E+06	7.94	156.89	**0.73**	0.018	5.56	11.08	11.8	13.40	**0.17**
3.5	2.4E+06	7.94	156.89	**0.96**	0.018	5.56	14.69	15.7	17.77	**0.22**
4.0	2.43E+06	7.94	156.89	**1.22**	0.018	5.56	18.61	19.8	22.51	**0.28**
4.5	2.4E+06	7.94	156.89	**1.49**	0.018	5.56	22.74	24.2	27.50	**0.34**
5.0	2.4E+06	7.94	156.89	**1.77**	0.018	5.56	26.96	28.7	32.60	**0.41**
5.5	2.4E+06	7.94	156.89	**2.04**	0.018	5.56	31.14	33.2	37.67	**0.47**
6.0	2.4E+06	7.94	156.89	**2.31**	0.018	5.56	35.16	37.5	42.52	**0.53**
6.5	2.4E+06	7.94	156.89	**2.55**	0.018	5.56	38.86	41.4	47.00	**0.59**
7.0	2.4E+06	7.94	156.89	**2.76**	0.018	5.56	42.09	44.8	50.90	**0.64**
7.5	2.4E+06	7.94	156.89	**2.93**	0.018	5.56	44.68	47.6	54.04	**0.68**
8.0	2.4E+06	7.94	156.89	**3.05**	0.018	5.56	46.46	49.5	56.20	**0.70**

From the simplified Rapid Performance assessment (a) to (e), it can be inferred that the building considered as a sample is evaluated in its short direction and long direction separately both for in-plane shear resistance and out of plane flexural resistance aspect. The summary of the analysis output is tabulated as follows.

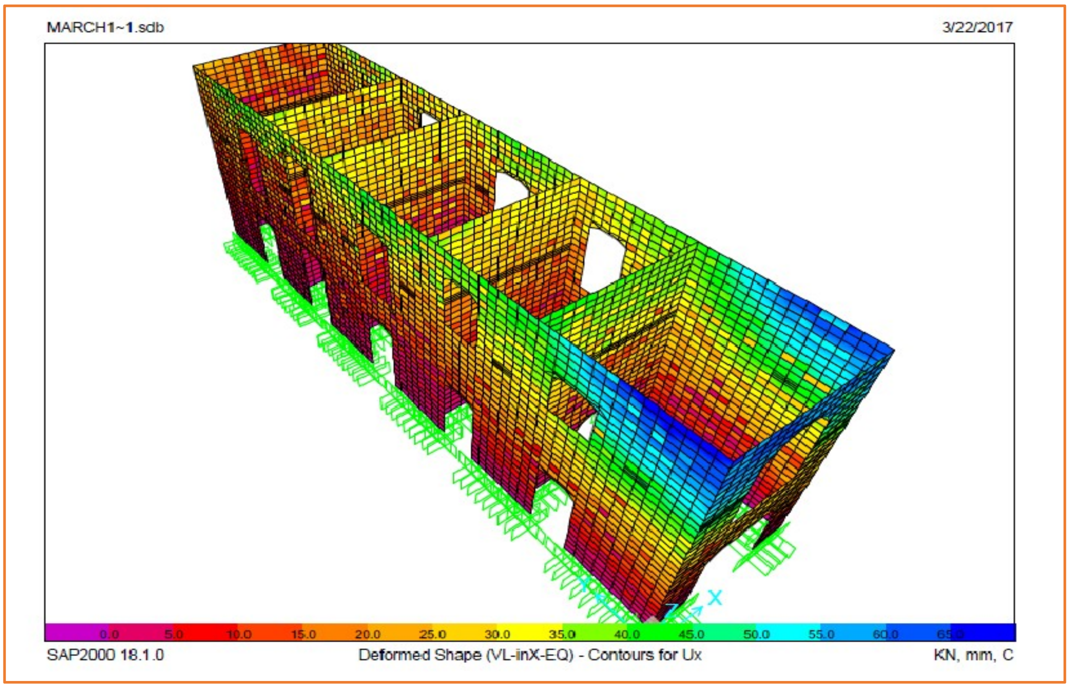
Table 9: Summary of simplified analysis

Analysis used	Eq. No.	Demand Magnitude	Capacity Magnitude	Demand Capacity
Calculating combined stress for short direction	(3-44d)	1.28MPa	4.87Mpa	26,3%
Calculating combined stress for long direction	(3-44f)	0.50Mpa	4.87Mpa	10.27%
Base shear on short direction	*(3-44g)*	0.435MPa	0.183MPa	237.7%
Base shear on long direction	*(3-44h)*	0.237MPa	0.178MPa	133.17%
Out of plane flexure (M_z) vertical strip	*(3-44k)*	*118.58kN.m*	17.63kN.m	672.60%
Out of plane flexure (M_y) horizontal strip	*(3-44l)*	*11.27kN.m*	*12.20kN.m*	92.38%

4.2 RESULT FROM SAP2000 V.18 SOFTWARE TOOL ANALYSIS

As described in Part 3.3.4 linear analysis are performed. Since the governing failure mode observed from hand calculation is out-of-plane flexural bending these load cases were given more emphasis. However, other possible load cases are also included in the analysis, namely: Dead, Live, Eccentricity in x-direction, and y-direction. This is accomplished by

Results for the individual lateral seismic loading based on appendix E-1 and E-2, both in x-direction (short) and y-direction (longer) as dimensioned in Fig. 19.



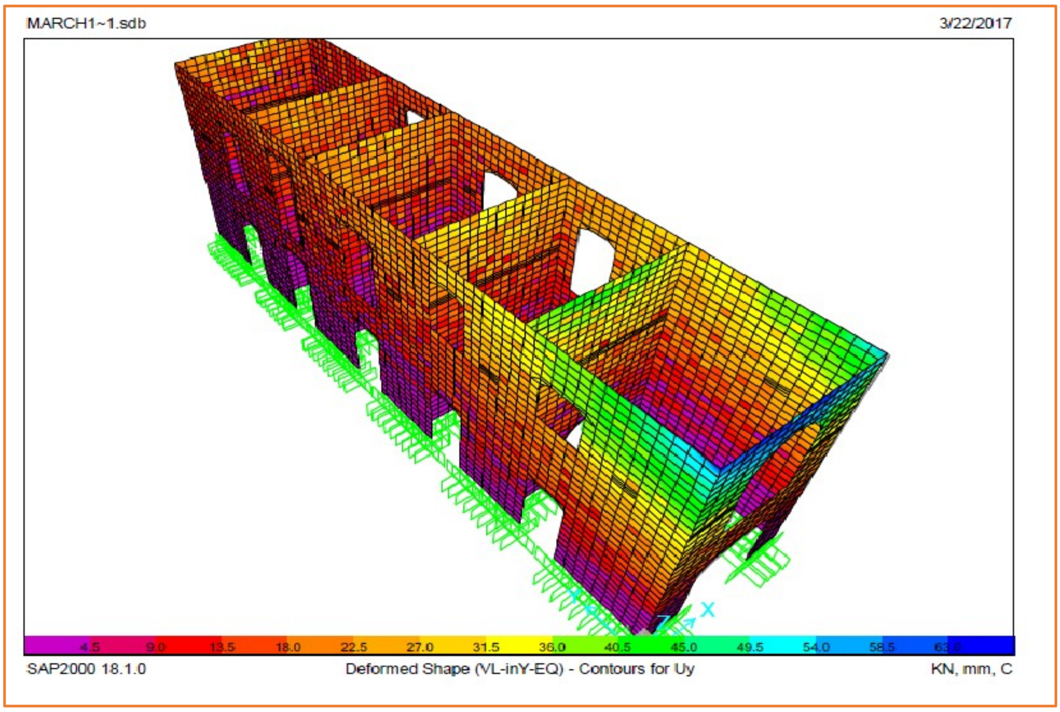


Figure 28 : SAP2000 v.18 output which shows 3D model with a contour for Ux and Uy which is exited by Lateral Loading in x and y direction respectively.

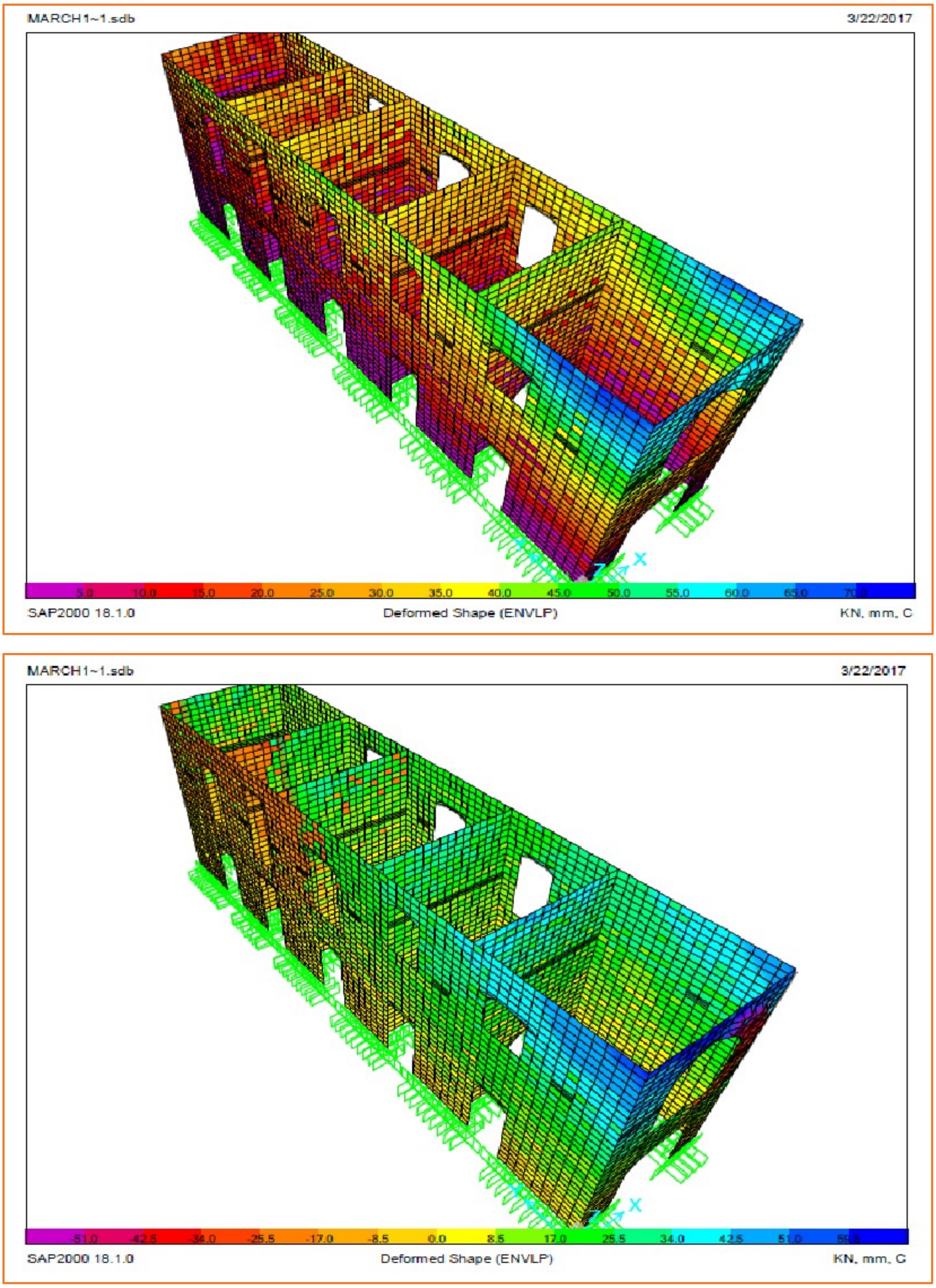


Figure 29 : SAP2000 v.18 output which shows 3D model with a contour for Ux and Uy which is exited by Envelope loading according Appendix C-1.

PART 5. DISCUSSION OF THE RESULT

5.1 RESULTS FROM THE ANALYSIS

Although Masonry wall is un-isotropic, material the main advantage of transforming to equivalent homogenous materials comes in to play when trying to analyze complex 3D elastic finite element analysis. Displacement in elastic materials is proportional to stored energy as long as it is in the elastic zone.

Lateral drift is also gives vital information for structure undergone deformation. Hence at the critical section between Axis (1 & 2) the out of plane deflections of wall on axis A will be compared using data from Table 9 and Table 11.

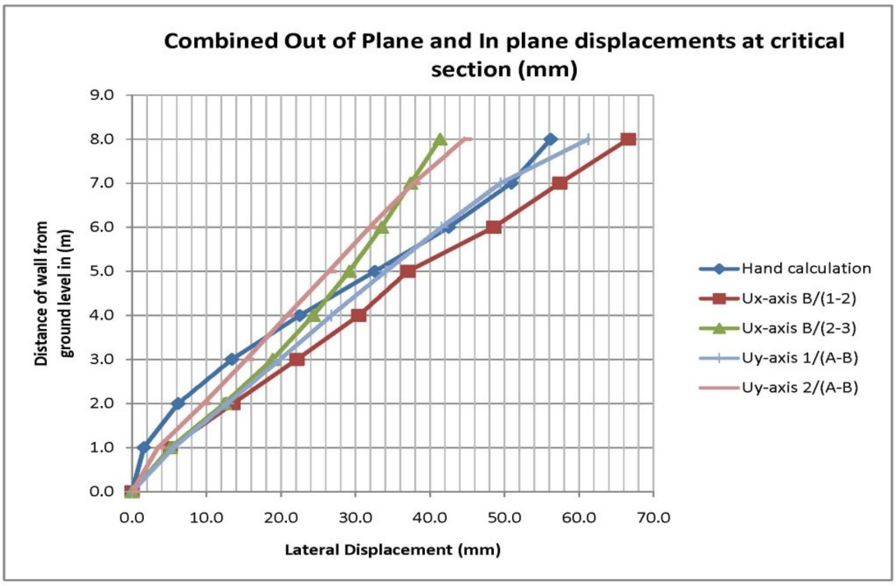


Figure 30: Comparison of Hand calculation displacement at critical section with SAP2000 output values for Envelope load Case.

It can be seen from Fig. 30 that the graph made from hand calculation is in good agreement with that of the Finite Element Analysis Software tool, SAP2000 v.18 output.

Table 10: SAP2000 output and hand calculation comparison table with relative drift ratio.

		SAP2000 output at crest of each span in (mm)										
Height of wall above ground in (mm)	Hand calculation at for 1m width in (mm)	Ux-axis B/(1-2)	Ux-axis B/(2-3)	Ux-axis B/(3-4)	Ux-axis B/(4-5)	Ux-axis B/(5-6)	Uy-axis 1/(A-B)	Uy-axis 2/(A-B)	Uy-axis 3/(A-B)	Uy-axis 4/(A-B)	Uy-axis 5/(A-B)	Uy-axis 6/(A-B)
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.0	1.6	5.2	5.2	2.3	2.8	2.1	5.6	3.5	2.6	2.0	1.7	2.7
2.0	6.2	13.6	12.4	6.8	7.1	6.4	12.8	9.6	7.0	5.5	4.7	7.5
3.0	13.4	22.1	18.9	12.3	11.7	11.8	19.9	15.4	11.8	9.5	8.2	13.4
4.0	22.5	30.5	24.3	18.1	16.5	17.4	26.7	20.8	16.5	13.6	11.8	15.8
5.0	32.6	37.0	29.1	24.5	22.0	24.6	34.0	26.4	21.1	17.7	15.6	18.4
6.0	42.5	48.5	33.5	30.1	26.5	30.1	41.5	31.9	25.5	21.8	19.3	21.3
7.0	50.9	57.4	37.4	35.2	30.0	34.2	49.5	37.8	30.3	26.1	23.4	25.1
8.0	56.2	66.6	41.3	42.1	34.7	40.1	61.2	44.6	36.1	31.6	28.3	30.3
Drift ratio (%)	**0.70**	**0.83**	**0.52**	**0.53**	**0.43**	**0.50**	**0.77**	**0.56**	**0.45**	**0.40**	**0.35**	**0.38**

Hand calculation result is generally higher for most walls except for walls on axis B/(1-2) and axis 1/(A-B). These walls are observed to be highly deformed. The main reason behind this large deformation is that both walls have large door and windows opening than other walls which is four sides of this span walls have large openings. On the other hand walls with smaller openings showed relatively smaller roof displacement.

5.2 PERFORMANCE EVALUATION

5.2.1 PERFORMANCE EVALUATION BASED ON PART 2.9

2.9.1(1):**SD**: drift and taken equal to $0.008H_o/D = (0.008x8/0.66) = 0.097$

Due to existence of the term contraflexure, this part of the guide-line is applicable only for walls with rigid diaphragm system where there is a restrain at every floor otherwise it will lead to overestimation of the capacity. As can be seen from 2.9.1(1), the drift limit is dependent on story height i.e., if there is no rigid floor the term H_o will be higher resulting exaggerated drift limit. Moreover the method depicted in this part assumes that all criterion are satisfied stated in EN 1998-3. Therefore, the next alternative is applying Part 2.11,

5.2.2. PERFORMANCE EVALUATION BASED ON PART 2.11

A performance level is a limit stage on the capacity curve that is used to quantify the damage. There are different approaches to damage limit states classification for masonry.

Researchers like Calvi [11], as cited by Huseyin [10] , have introduced inter-storey drift ratios with three limit states, proposed three damage limit states for masonry structures as follows. In most cases masonry buildings do not show a clear elastic limit, since cracking starts at early stages and tends to extend progressively. It is therefore reasonable to condense the two first limit states into a single one.

LS1 and LS2 - Minor structural damage and/or moderate non-structural damage; the building can be utilized after the earthquake, without any need for significant strengthening and repair to structural elements. The suggested drift limit is **0.1%.**

LS3 - Significant structural damage and extensive non-structural damage. The building cannot be used after the earthquake without significant repair. Still, repair and strengthening is feasible. The suggested drift limit is **0.3%.**

LS4 - Collapse; repairing the building is neither possible nor economically reasonable. The structure will have to be demolished after the earthquake. Beyond this LS global collapse with danger for human life has to be expected. The suggested drift limit is **0.5%.**

According to Calvi [11], as cited Huseyin [10], criteria the sample building critical section is currently at performance level of LS4 based on less conservative result from SAP2000 of Table 11. All walls analyzed found to be worst than LS3 which is 0.3%.

PART 6. CONCLUSION AND RECOMMENDATIONS

6.1 CONCLUSION

The theoretical bases of Simplified Performance Assessment for Representative Existing URM building are explained with particular emphasis on In plane and Out of Plane stresses on URM wall.

Creation of 3D modeling based on Equivalent Static Analysis of Seismic Load is done. Sample building was analyzed using both Simplified Hand Calculation and Simplified Finite Element Analysis Software tool, SAP2000 v.18 , modeling.

The result from the analysis indicates that URM buildings similar to the one considered in the sample analysis in this thesis should be considered unsafe and urgent upgrading intervention shall be executed timely to protect the users life.

6.2 RECOMMENDATIONS

In order to improve the URM buildings' capacity the most effective intervention can be to introduce a rigid floor system by retrofitting the available system and by introducing strong beam belt around the roof level.

Changing the type of service such as from hospital dormitory to warehouse can reduce the hazard level from loss of human life to material loss. If possible reducing non-structural masses can be effective method however mass reduction should not affect the in plane shear resistance component of overburden pressure.

Walls with large opening can be closed to increase both in-plane and out-of-plane stiffness if applicable.

Further studies on related topic may include the followings; but not limited to:

- There is no substitute like performing testing thus proto type of local URM buildings which includes the overall material behavior of the building can be investigated which is potential study area is leading to vulnerability assessment of various URM building types in Addis Ababa or other seismic zones of the country provided that there is sufficient budget.

- Although time consuming, micro modeling of URM buildings on variety kinds of buildings which require different approach. Such cases are brick buildings, combination of Stone and Brick buildings, URM buildings with rigid floor systems are some of the uncovered topic in related area.
- After evaluation of a given building, there are different approaches to perform retrofitting; however, since cost is another constraint, optimization of retrofitting a given building can be another area of research.

REFERENCE

[1] Abnet Gezahegn Berhe, 2010. Conservation and Management of Historical Buildings Advanced International Training Program 2009-2010 Lund University SIDA

[2] Addis Wubet, 2009. Addis Ababa Heritage Database.

[3] Ahmed Y. Elghazouli, 2009. Seismic Design of Buildings to Eurocode 8

[4] American Society of Civil Engineers and Federal Emergency Management Agency, FEMA 356, 2000 'FEMA 356' Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Reston, VA.

[5] American Society of Civil Engineers and Federal Emergency Management Agency, FEMA 774, 2009. Unreinforced Masonry Buildings and Earthquakes Developing Successful Risk Reduction Programs.

[6] Asrat Worku, Phd. 2011. Recent Developments in the Definition of Design Earthquake Ground Motions Calling for a Revision of the Current Ethiopian Seismic Code - EBCS 8:1995,Journal of EEA, Vol. 28 2011, ISSN: 0514-6216. pp(1-15).

[7] Construction Minister, 2015, Ethiopian Standards Based on Euro Norms, Actions on structures - Part 1-1: General actions -Densities, self-weight, Imposed loads for buildings. Addis Ababa, Ethiopia

[8] Construction Minister, 2015, Ethiopian Standards Based on Euro Norms, Design of Masonry Structures Part 1-1: Common Rules for Reinforced and Unreinforced Masonry Structures. Addis Ababa, Ethiopia

[9] Construction Minister, 2015, Ethiopian Standards Based on Euro Norms, Design of Structures for Earthquake Resistance - Part 1: General Rules - Seismic Actions and Rules for Buildings. Addis Ababa, Ethiopia

[10] European Committee for Standardization, 2005, Eurocode 8: Design of structures for earthquake resistance -Part 3: Assessment and retrofitting of buildings, Ref. No. EN 1998-3:2005: E, B·1050 Brussels

[11] Gian Michele Calvi, (1999) A Displacement-Based Approach for Vulnerability Evaluation of Classes of Buildings, Journal of Earthquake Engineering, 3:3, 411-438, Doi: 10.1080/13632469909350353

[12] Huseyin Bilgin, Ergys Huta, 2016, Earthquake Performance Assessment of a Low-rise URM Building, 3rd International Balkans Conference on Challenges of Civil Engineering, 3-BCCCE, 19-21 May 2016, Epoka University, Tirana, Albania

[13] Jen-Chei Liu, Mou-Lin Sue and Chang-Huan Kou, 2009, Estimating the Strength of Concrete Using Surface Rebound Value and Design Parameters of Concrete Material, Tamkang Journal of Science and Engineering. Vol 12 No.1. pp. 1 7 (2009), Department of Civil Engineering and Engineering Informatics, Chung-Hua University, Hsin Chu, Taiwan 300, R.O.C

[14] Joe Blakey, 2011, Case Study Japan Earthquake and Tsunami, Wikipedia

[15] Jorge Muñoz Barrantes, 2012. PhD Dessertation on a title Seismic Risk Assessment of Unreinforced Masonry Buildings at a Territorial Scale.

[16] Miha Tomazevic, 2006. Earthquake-Resistant Design of Masonry Buildings, Series on Innovation in Structures and Construction — Vol. 1, Imperial College Press, 57 Shelton Street Covent Garden London WC2H 9HE

[17] Ministry of Works and Urban Development, 1995, Ethiopian Building Code Standard 1, Basis of Design and Actions on Structures, Addis Ababa, Ethiopia

[18] Ministry of Works and Urban Development, 1995, Ethiopian Building Code Standard 8, Design of Structures for Earthquake Resistance, Addis Ababa, Ethiopia

[19] NZSEE, New Zealand Society for Earthquake Engineering, Draft, 2011. Assessment and Improvement of the Structural Performance of Buildings in Earthquakes

[20] Panagiotis G. Asteris et al. 2014, Engineering Structures 62-63. Seismic vulnerability assessment of historical masonry structural systems.

[21] Paulo B, 2014. Lourenco, Analysis Methods for Unreinforced Heritage Masonry

[22] Richard D. Andrews, Interim Director and State Geologist; Dr. Austin Holland, State Seismologist, Oklahoma Seismicity, April 21, 2015, Okalhoma Geoloical Survey, Statement on Oklahoma Seismicity.

[23] Samuel Kinde Kassegne, Samson Engeda, Asnake Kebede and Eyob Tessema PhD, PE, 2011. Notes and Proposed Guidelines on Updated Seismic codes in Ethiopia. Implication for Large-Scale Infrastructures, Journal of EEA, Vol. 28 2011, ISSN: 0514-6216. pp(90-110).

[24] SAP2000 Ultimate, Version 18.1.0, Build 1227, 2016, Structural Analysis Program, Software tool, a product of Computer and Structures, Inc.

[25] Selam Tewoldebirhan July 2011, Thesis, The Status of Historic Buildings in Addis Ababa and Their Contribution to The Urban Development, Addis Ababa University, Department of Urban Development and Management

[26] Sergio Lagomarsino, 2013, An equivalent frame model for the nonlinear seismic analysis of masonry buildings, Engineering Structres 56 (2013) 1787–1799

[27] Stavroula. J. Pantazopoulou, Professor of Structural Engineering, 2013. State of the Art Reports for the Analysis Methods for Unreinforced Masonry Heritage Structures and Monuments.

[28] Sujan Malla, Dr. Eng., 2015, Structural Engineer, Zurich, LESSONS OF 2015 NEPAL EARTHQUAKE DISASTER, A Short Report on Effects of 7.8 Mw Earthquake of 25 April 2015 and Its Aftershocks. Switzerland

[29] Wai Fah Chen, 2006, Earthquake engineering for structural design, by Taylor & Francis Group, LLC

[30] Wikipedia, 2004, en.wikipedia.org, 2004 Indian Ocean Earthquake and Tsunami - Wikipedia.

[31] Wikipedia, 2005, en.wikipedia.org, 2005 Kashmir Earthquake - Wikipedia.

[32] Wikipedia, 2008, en.wikipedia.org, 2008 Sichuan Earthquake - Wikipedia.

[33] Wikipedia, 2010, en.wikipedia.org, Causalities of the 2010 Haiti earthquake - Wikipedia.

APPENDIX A-1

CONSTRUCTION DESIGN SCo.

GEOTECHNICAL BUSNIESS UNIT

☒ 40036 ☎ 0114-420800, 0114-423366, 0114-420616, FAX (251-1) 420153, 400621

PROJECT N° T7-076/2003

Project :- G+7 Building

Client :- Yekatit 12 Memorial Hospital

Location :- Addis Ababa

NOTIFICATION OF TEST RESULTS

The Geotechnical Business Unit of the Construction Design Share Company has carried out sub surface investigation work for G+7 Building of Yekatit 12 Memorial Hospital.

This report is composed of field sub soil investigation works, Laboratory test results & foundation recommendation.

The Federal Democratic Republic of Ethiopia
Construction Design

Addis Ababa
April, 2011

APPENDIX A-2

Company Name	Form No
CONSTRUCTION DESIGN SCo.	OF/CDSCO/10

Title	Issue No	Page No
Borehole Log Sheet	1	Page 1 of

PROJECT G+7 BUILDING
BORING TYPE ROTARY CORING
LOCATION ADDIS ABABA
GROUND WATER LEVEL 4.20m
CLIENT YEKATIT 12 HOSPITAL
BH ELEVATION
DATE STARTED 18/07/2003
INCLINATION VERTICAL
DATE COMPLETED 21/07/2003

BH 2

DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. (N-VALUE)	DEPTH (m)	PROFILE	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
0 – 4		116			0.00, 0.40, 1.00, 1.50, 2.00, 2.40, 2.70, 3.40, 3.90, 4.00		100, 100, 100, 100, 100, 100, 100, 100, 100, 100		CLAY with concrete and gravel Fill	
4 – 7.30		100	5.50	13	4.50, 5.50, 6.00, 7.30		100, 100, 100, 100		Medium dense to dense, yellowish grey to light grey sandy SILT	
7.30 – 12.00		55			9.00, 10.80, 12.00		100, 100, 98, 70	58, 95, 70	Strong, grey, slightly weathered Basalt	
12.00 – 15.00		55			13.20, 13.60, 15.00		100, 100, 100		Very dense, yellowish grey sandy SILT/silty SAND	

(Nc) CONE PENETRATION TEST
N BLOWS/30cm
RS ROCK SAMPLE
W WATER SAMPLE
TCR TOTAL CORE RECOVERY
RQD ROCK QUALITY DESIGNATION
DISTURBED SOIL SAMPLE
UNDISTURBED SOIL SAMPLE
SPT STANDARD PENETRATION
END OF DRILLING

CREW Ibrahim Adem
DRAWN BY Bezunesh W/Tsadi
SUPERVISOR Tewodros Mako SIG
LOGGED BY Tewodros Mako SIG
APPROVED BY Getachew Teferi SIG

The Federal Democratic Republic of E... Construction Design Share Company

APPENDIX A-3

Company Name		Form No
CONSTRUCTION DESIGN SCo. Architects, Engineers & Town Planners	**CONSTRUCTION DESIGN SCo.**	**OF/CDSCO/1(**
Title	**Borehole Log Sheet**	Issue No **1** / Page No **Page 1 of**

PROJECT G+7 BUILDING BORING TYPE ROTARY CORING

LOCATION ADDIS ABABA GROUND WATER LEVEL 2.50m

CLIENT YEKATIT 12 HOSPITAL BH ELEVATION

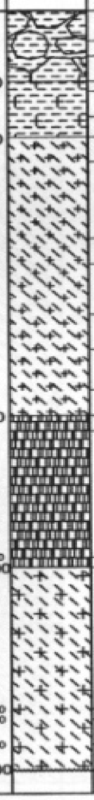
DATE STARTED 14/07/2003 INCLINATION VERTICAL

DATE COMPLETED 17/07/2003

BH 1

DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. /N-VALUE/	DEPTH (m)	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMAR
0–2.50		100			0.00, 0.60, 0.90, 1.40, 1.60	100, 100, 100, 100, 100		CLAY with gravel	Fill mate
			2.50	13	2.10, 2.50	100, 100		Firm, dark brown silty CLAY	
			5.50	58	3.00, 4.50, 5.00, 6.00, 6.70, 7.50, 8.00	100, 100, 100, 100, 100, 100, 100	100	Medium dense to dense, yellowish grey to light grey sandy SILT	
		75			8.30, 10.80, 11.00	90, 100, 100	90	Strong, grey, slightly weathered Basalt	
–15		55		61	13.80, 14.00, 14.50, 15.00	100, 100, 100, 100		Very dense, yellowish grey sandy SILT/silty SAND	

PROFILE



[Nc] CONE PENETRATION TEST
N BLOWS/30cm
RS ROCK SAMPLE
W WATER SAMPLE
TCR TOTAL CORE RECOVERY
RQD ROCK QUALITY DESIGNATION
DISTURBED SOIL SAMPLE
UNDISTURBED SOIL SAMPLE
SPT STANDARD PENETRATION

CREW Ibrahim Adem DRAWN BY Bezunesh W/Ts
SUPERVISOR Tewodros Mako SIG
LOGGED BY Tewodros Mako SIG
APPROVED BY Getachew Teferi SIG for

The Federal Democratic Republic ... Construction Design Share Company

APPENDIX B-1

AAiT
Addis Ababa Institute of Technology
Addis Ababa University
School of Civil & Environmental Engineering

Date: July 13, 2016

CONSTRUCTION MATERIALS TESTING LABORATORY

Client-
Project-
Test Required:- Concrete Hammer Test

SUMMARY OF TEST RESULTS

pt 1 [α =0^0]

Trial No.	Hammer Rebound	Equivalent Cube Strength [MPa]
1	38.0	37.0
2	38.0	37.0
3	40.0	40.0
4	40.0	40.0
5	40.0	40.0
6	42.0	44.0
7	42.0	44.0
8	44.0	48.0
9	44.0	48.0
10	46.0	52.0
	Average	**43.0**

SH-1 [α =0^0]

Trial No.	Hammer Rebound	Equivalent Cube Strength [MPa]
1	40	40.0
2	40	40.0
3	40	40.0
4	42	44.0
5	42	44.0
6	42	44.0
7	42	44.0
8	42	44.0
9	42	44.0
10	44	48.0
	Average	**43.2**

Coulmn2 [α =0^0]

Trial No.	Hammer Rebound	Equivalent Cube Strength [MPa]
1	46.0	52.0
2	46.0	52.0
3	46.0	52.0
4	48.0	56.0
5	48.0	56.0
6	48.0	56.0
7	48.0	56.0
8	48.0	56.0
9	48.0	56.0
10	48.0	56.0
	Average	**54.8**

GFC 1 [α =0^0]

Trial No.	Hammer Rebound	Equivalent Cube Strength [MPa]
1	28	24.0
2	28	24.0
3	30	24.0
4	30	24.0
5	30	24.0
6	30	24.0
7	30	24.0
8	30	24.0
9	30	24.0
10	30	24.0
	Average	**24.0**

Tested by:
Ato Fikru Bedada
Ato Demessew Melaku

Verified by:
Dr. Esayas G.Youhannes

Using the values from 4-point test results in the summary table and taking the average of averaged resulting average Hammer Number the compressive strength of the masonry unit (stone) is calculated as follows.

Thus $f_b = (43.0+43.2+54.8+24)/4 =$ $\boldsymbol{41.3MPa}$

APPENDIX C-1

No.	Load Combination
1	1.35xGk +1.5Qk
2	Gk + 0.3x Qk + Ex + 0.3xEy + MEx + 0.3MEy
3	Gk + 0.3xQk + Ex + 0.3xEy + MEx - 0.3MEy
4	Gk + 0.3xQk + Ex + 0.3xEy - MEx + 0.3MEy
5	Gk + 0.3xQk + Ex + 0.3xEy - MEx - 0.3MEy
6	Gk + 0.3xQk + Ex - 0.3xEy + MEx + 0.3MEy
7	Gk + 0.3xQk + Ex - 0.3xEy + MEx - 0.3MEy
8	Gk + 0.3xQk + Ex - 0.3xEy - MEx + 0.3MEy
9	Gk + 0.3xQk + Ex - 0.3xEy - MEx - 0.3MEy
10	Gk + 0.3xQk - Ex + 0.3xEy + MEx + 0.3MEy
11	Gk + 0.3xQk- Ex + 0.3xEy + MEx - 0.3MEy
12	Gk + 0.3xQk- Ex + 0.3xEy - MEx + 0.3MEy
13	Gk + 0.3xQk- Ex + 0.3xEy - MEx - 0.3MEy
14	Gk + 0.3xQk- Ex - 0.3xEy + MEx + 0.3MEy
15	Gk + 0.3xQk- Ex - 0.3xEy + MEx - 0.3MEy
16	Gk + 0.3xQk - Ex - 0.3xEy - MEx + 0.3MEy
17	Gk + 0.3xQk- Ex - 0.3xEy - MEx - 0.3MEy
18	Gk + 0.3xQk + Ey + 0.3xEx + MEy + 0.3xMEx
19	Gk + 0.3xQk + Ey + 0.3xEx + MEy - 0.3xMEx
20	Gk + 0.3xQk + Ey + 0.3xEx - MEy + 0.3xMEx
21	Gk + 0.3xQk + Ey + 0.3xEx - MEy - 0.3xMEx
22	Gk + 0.3xQk + Ey - 0.3xEx + MEy + 0.3xMEx
23	Gk + 0.3xQk + Ey - 0.3xEx + MEy - 0.3xMEx
24	Gk + 0.3xQk + Ey - 0.3xEx + MEy + 0.3xMEx
25	Gk + 0.3xQk + Ey - 0.3xEx + MEy - 0.3xMEx
26	Gk + 0.3xQk - Ey + 0.3xEx + MEy + 0.3xMEx
27	Gk + 0.3xQk- Ey + 0.3xEx + MEy - 0.3xMEx
28	Gk + 0.3xQk - Ey + 0.3xEx + MEy + 0.3xMEx
29	Gk + 0.3xQk- Ey + 0.3xEx + MEy - 0.3xMEx
30	Gk + 0.3xQk - Ey - 0.3xEx + MEy + 0.3xMEx
31	Gk + 0.3xQk- Ey - 0.3xEx + MEy - 0.3xMEx
32	Gk + 0.3xQk- Ey - 0.3xEx + MEy + 0.3xMEx
33	Gk + 0.3xQk - Ey - 0.3xEx + MEy - 0.3xMEx

APPENDIX D-1

Tributary loading on roof assuming doupitch

Summary of Loads				
	Left	**Right**		
Max Truss Spacing	1.96	1.96		
Max Purlin Spacing	0.9	0.9		
Dead Load	0.64	kN/m^2		
Live Load	0.25	kN/m^2		
	Load on Purlin		**Load on joint**	
Dead Load on Purlin	0.58	kN/m	1.13	kN
Live Load on Purlin	0.23	kN/m	0.44	kN
	Load transferred to masonry			
Dead load from truss to wall	6x1.13	each gable side	6.78	kN
Live load from truss to wall	6x0.44	each gable side	2.64	kN

APPENDIX E-1

X- direction base shear loading (Short direction)						
Fb =	3186.12 kN		From Eq. (3-44a)			
Distributing according to tributary area on parallel walls each axis						
axis 1	2.37		2.37	0.110037	**350.59**	**kN**
axis 2	2.37	1.88	4.25	0.196963	**627.55**	**kN**
axis 3	1.88	2.37	4.24	0.196569	**626.29**	**kN**
axis 4	2.37	1.64	4.00	0.185443	**590.84**	**kN**
axis 5	1.64	2.39	4.02	0.18637	**593.80**	**kN**
axis 6	2.39		2.39	0.11057	**352.29**	**kN**
Triangular Base shear distribution on each wall						
	axis 1	axis 2	axis 3	axis 4	axis 5	axis 6
level in (m)	87.65	156.89	156.57	147.71	148.45	88.07
8	87.65	156.89	156.57	147.71	148.45	88.07
7	76.69	137.28	137.00	129.25	129.89	77.06
6	65.74	117.67	117.43	110.78	111.34	66.05
5	54.78	98.05	97.86	92.32	92.78	55.05
4	43.82	78.44	78.29	73.86	74.22	44.04
3	32.87	58.83	58.72	55.39	55.67	33.03
2	21.91	39.22	39.14	36.93	37.11	22.02
1	10.96	19.61	19.57	18.46	18.56	11.01
0	0.00	0.00	0.00	0.00	0.00	0.00

APPENDIX E-2

Y- direction base shear loading (Long direction)						
Fb =	3186.12 kN	From Eq. (3-44a)				
Distributing according to tributary area						
axis A	1593.06	**1593.06**	**kN**			
axis B	1593.06	**1593.06**	**kN**			
Triangular Base shear distribution on each wall						
	axis A	axis B	**kN**			
level in (m)	398.27	398.27	**kN**			
8	398.27	398.27	**kN**			
7	348.48	348.48	**kN**			
6	298.70	298.70	**kN**			
5	248.92	248.92	**kN**			
4	199.13	199.13	**kN**			
3	149.35	149.35	**kN**			
2	99.57	99.57	**kN**			
1	49.78	49.78	**kN**			
0	0.00	0.00	**kN**			

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