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ABSTRACT

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Historical masonry structures are recognized as global wealth whose conservation does not only preserve cultural value but also contributes to human development, international cooperation and scientific pursuit. The seismic performance of these existing structures nowadays is becoming a current issue in seismic prone areas in Europe, majorly in Italy and also in Portugal. This thesis presents the limit analysis of masonry structures under the lateral exposures, that's the most intentionally given sudden earthquakes and also to the associated lateral forces. The current study here in is concerned firstly on the parametric analysis of masonry walls to the lateral loads. Secondly, the analyses of representative existing masonry walls under lateral exposures are done. A simple user friendly but very important software, named Block, is used for this study. It is based on the rigid block modelling concept. In the parametric part, the basic and immediate first step, it is studied the relation of seismic safety factor with respect to the masonry wall shapes, masonry unit sizes, type of masonry bond, compressive stress and also the effect of overloading in which they are applied for a significant set of masonry two dimensional walls and their associated collapse mechanisms to lateral loads. The software gives the safety factor for a certain masonry structure as a function of the gravitational load of the masonry and the overloads on the masonry that are in the direction of gravity. With this, it was possible to know the relations among safety factor and other parameters. The effects of different characteristics that exist among the masonry units and also the reasonable use of strengthening ties is also studied. In this specific study, the parametric seismic limit analysis is applied to the two dimensional masonry structures where they are likely to represent the real existing masonry structures. The importance of block analysis from micro to macro is seen under masonry wall basis selected to comply with the work limit. Developing macro models from the failure mechanisms obtained is used to compute the capacity curves with the kinematic analysis.

RESUMO

As estruturas históricas de alvenaria são reconhecidamente uma riqueza, cuja conservação preserva não só o seu valor como contribui igualmente para o desenvolvimento humano, cooperação internacional avanço científico. O desempenho sísmico destas estruturas tornou-se um assunto central nas zonas sísmicas Europeias, como por exemplo em Itália e Portugal.

Esta tese apresenta resultados de análise limite de estruturas de alvenaria sujeitas a acções horizontais de natureza sísmica. Este estudo apresenta inicialmente uma análise paramétrica de paredes de alvenaria a cargas horizontais. Seguidamente, faz-se a análise plana de paredes representativas de construções de alvenaria à acção sísmica. Para o efeito, usou-se um programa de cálculo automático, designado por Block. Este programa é baseado na teoria da análise limite, considerando a hipótese de blocos rígidos.

Estudou-se a relação entre o factor de carga sísmico e a forma da parede, o tamanho das unidades, o tipo de interligação, a resistência à compressão e o nível de compressão, bem como os mecanismos de colapso obtidos. O factor de carga sísmico depende do valor carga gravítica aplicada. Foi ainda considerado o efeito de tirantes metálicos na alteração da capacidade de carga e mecanismos de colapso.

A abordagem "micro" conseguida com esta metodologia permite posteriormente definir macro-blocos associados aos mecanismos de colapso mais prováveis. Esta definição permite obter de forma relativamente fácil as curvas de capacidade associadas à análise cinemática de estruturas de alvenaria.

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CHAPTER 1

INTRODUCTION AND OBJECTIVES

1.1 Aims

Historical masonry structures are the world's most widely but still less studied on their structural behaviours. Material mechanics and vast non linearity makes them very specific for study. Structural responses to excitations or natural forces are still under investigation. Many experiments are trying to come to unify and characterise all associated things under basic circumstances that consider good assumptions making close to the realistic of the masonry geometry and mechanics. Various kinds of associative formulations are developed in a reasonable manner so that homogenized or standardized method of analysis will be available.

Among different philosophies of structural analysis for historical constructions, limit analysis is one of them. Limit analysis is formerly developed and easy for assessing blocky structures. Under time basis, for validation and contribution to the study by limit analysis; it is aimed to reach in obtaining what kind of relation exists in the geometry effect on the seismic safety factor. Moreover, obtaining failure mechanism of representative masonry walls of historic structures can be analysed by using the block limit analysis. Models composed by micro models are used to give more precise results of simulations. Good results from micro models are then implicative to macro modelling, because of the fact that micro is derived from the macro model. Failure kinematic chains from failure modes by macro models are importantly used for the application in virtual work in the kinematic analysis. Hence, measure of capacity can be obtained from the non linear kinematic analysis.

1.2 Methodology

Seismic limit analysis is done by the help of Block software. This software is friendly programmed in such a way that it inputs the geometry of the masonry structure in AutoCAD drawings and the output is also in AutoCAD drawing. The results will be two things; the failure mechanism and the plot for the thrust lines that are along the joints in between the

masonry units. The non linear programme solving process takes time for models having higher number of units as the mesh is fined. It is done as much as possible up to the time allocated for this work. The main steps carried out within this dissertation are the following:

- Parametric seismic analysis of the most important parameters influencing the seismic response; building shape and height, presence of openings, compressive strength, increase in service load and also existence of strengthening ties, are the main issues detailed in this part.
- Transition from the preliminary analysis to block analysis of selected masonry walls. Standard masonry walls with typical configuration on openings are analysed. Macro models from failure mechanisms are developed and computation of capacity curves through non-linear kinematic analysis and safety verification.
- Plots of results and comparisons of the existing results of analysis and conclusions.

1.3 Objectives of the Thesis

Seismic analysis of masonry constructions is the main objective of this dissertation work. As a matter of specifying the work, it is focused on the in plane lateral seismic capacity of masonry wall models. Analysis of the models is by using user friendly computational tool which bases its applications on limit analysis. This software provides both the maximum seismic load that the structure can bear as well as the associated mechanism. Associating the failure mechanism to the possible types of strengthening, furthermore, the simulation of strengthening ties is also possible.

Generally, objectives of the task are:

- Preliminary Parametric analysis to masonry shear walls: this contains studying on the effect of masonry mechanical and geometrical characteristics to the capacity in the lateral resistance. Parametric analysis to walls with variations on their masonry unit size, arrangement, bond and compressive strength

- Obtaining failure mechanisms to selected typical masonry walls: for the block analysis in the second major objective, typical masonry walls are selected from experimental

prototypes done before. From the output of failure mechanisms, in turn can be used to develop macro blocks.

- Macro modelling from the results of block analysis: this is used as an input in non linear kinematic analysis for obtaining the capacity curve.

1.4 Thesis Organisation

Basically, seismic limit analysis of masonry structures is very wide. This might be because of, in the first place it depends on the type of structure going to be analysed and secondly on the extent of the structure whether it is small or medium or large buildings. Furthermore, depending on the analyst, it may be divided based on their ages of existence. Here, for better composition on the seismic limit analysis for masonry structures, this thesis is basically deeply studied in two major parts. The first major part is on parametric seismic analysis in which a set of walls are analysed to understand the effect of changes in some parameters considered. And the other one is analysis conducted on selected masonry walls.

Chapter 1 deals with the introduction of the thesis and its main objectives. This contains the introductive part what the contribution of this thesis is and the overview of the whole study.

The second section, **Chapter** 2, is parted into the state of the art and the revision of sources in the goal of the thesis as literature review. The main motivation to this work is stated in the first part of this chapter. It reasons out why need to study on this specific topic. Most important parts of the contributing books, journals, electronic papers and also some PhD thesis researches are discussed according to their link to this study. It presents works of other researchers that are relevant for forms of validation or confirmation of some facts and results.

One of the main and first major parts of this work is in **Chapter** 3, the preliminary study on the seismic parametric analysis. Works on different masonry walls modified from a standard reference wall in order to meet the criteria of parametric analysis are done. Masonry shear walls are analysed seismic limit capacity or safety factor using the Block software. This is done by realizing the walls to the conditions that currently existing historic masonry structures show. It is made to simulate the real conditions of shear walls and obtain their seismic capacities. Development of graphs for relations in between the safety factor and

various important parameters is plotted. The most important parameters are: wall aspect ratio, masonry unit aspect ratio, masonry unit size ratio, bond ratio, compressive stress ratio and overload ratio. Comparison to existing literatures and confirmations are also done.

The second major work of this thesis goes to **Chapter** 4, it is tried to analyse typical and representative masonry walls. The walls analysed are taken from the prototypes tested in the Universita degli di Pavia in January 1995. The prototype was a masonry building tested under cyclic loads and here it is analysed using the block software and compared with its tested result. Models of analysis for the test and for the analysis are made as close as approximate. Moreover, as equivalent to done in third chapter, it was analysed for some parametric variations and collapse mechanisms are obtained. Finally macro modelling of this collapse mechanisms obtained from block limit analysis is used to obtain the capacity curves.

Chapter 5 is the general conclusion of the thesis. The outcomes and the contributions to the existing current condition of study on the seismic limit analysis are summarized.

CHAPTER 2

STATE OF THE ART AND LITERATURE REVIEW

2.1 Masonry: Construction

The term masonry in its original significance means: a construction of dressed or fitted stones and mortar. It was thus properly limited to stone masonry in ancient times. But, nowadays, the term includes many different materials and types of construction. Natural stone as well as manufactured units of clay brick, concrete block, cast stone, structural clay tile, terra cotta, adobe, and glass block are all masonry materials. Brick, concrete block, and stone are the most popular and most widely used types of masonry. Among those types, the most ancient and common adapted in earlier times is the stone masonry. The term stone masonry is used to designate any work in which stones are fitted and cemented together so as to form a structure. It can be further subdivided into rubble masonry, squared stone masonry, and ashlars or cut-stone masonry.

Historical or traditional materials that may be of earth, stone, masonry or wood are characterized by very complex mechanical and strength phenomena still with a significant challenging in the practical modeling capabilities. Historic masonry buildings in seismically active regions are severely damaged by intensive earthquakes, since they certainly have not been explicitly designed by the original builders to withstand seismic effects, at least not in a scientific way from today's point of view. The assessment of their seismic safety is an important first step in planning the appropriate interventions for improving their pertinent resistance for their existence in the future times.

In particular, masonry is characterized by having:

- composite character (masonry unit & mortar),
- brittle type of response under tension (almost null tensile strength),
- frictional sliding type of response in shear (appearance once when the limited bond between units and mortar is lost),

- anisotropy in the whole make up (response depends with the orientation of loads and orientation of joints).

Assembly of units in different forms of bond are made to form up a wall or parts of the structure so that functions can be of load bearing or transferring loads to the load bearing parts as in the case of flying arches of gothic cathedrals. Walls of multiple leafs were also used in the assumption for load resistance. More can be found in the arch bridges by formation of different shapes and combinations. Roman arch bridges and their equivalent types of bridges in Portugal are the best to mention.

Some of the world's most significant architectural achievements, for instance, the Egyptian Pyramids, the Colosseum in Rome, India's Taj Mahal, the Great Wall of China and many others of later times have been built with masonry. Through civilization, architects and builders have chosen masonry for its beauty, versatility, and durability. Masonry is resistant to fire, earthquakes, and sound. Artistic and durable masonry structures can withstand the normal wear and tear of centuries.

2.2 Structural Analysis of Historical Constructions

When a structure is already standing, the role of analysis comes into mind. The evidence that it already provides of its strengths and weaknesses and modes of action becomes a useful starting point in order to assess the life of the building. The models of the structure updated from time to time must adequately represent not only the geometry but also all relevant details of its actual present situation where by it gives satisfaction or leads to the professional who is trying to assess its situation or getting its health. When the structure is an old one this can be difficult because it will have been built of materials like timber and masonry which are very difficult to characterize their mechanical characteristics and also they have more variable strengths and elasticity's (Lagomarsino, 2006). Through the time that the historical structure has gone and passed by the different damages and catastrophic exposures, these will have ideas on the basis for the start over analysis of that precious historical structure. Different modes of deformation are likely to predominate and to occur due the natural exposures and also human effects and behaviour may have changed somewhat over a long life.

(a)

(b)

Figure 2.1: Historical masonry structures; (a) Colloseum in Italy (Colosseum in Rome, Italy), (b) Mallorca cathedral in Spain (Mallorca Cathedral, Spain)

The strength features of masonry structures allowed in the past for geometrical rules sanctioned by the experience and the observation of past successful structures. Having this as primary, today's basis and also the theory for masonry structures stems its contents from ancient practices and early rational insights. Typical gothic styles of historic structures are very common in Spain; Mallorca cathedral is an example, shown in Figure 2.1 (b) (Roca, 2008).

In earlier times, the structural analysis of historical constructions was not as deeply studied as when you expect it being now at this time. Through time, professionals, in earlier times, started to discover how to preserve and pass it to times in the future. That made them to start on how to have the analysis for coming that possible to the extent that is achieved. In recent times, the quantitative analysis of historic structures has received fresh stimulus from rapid advances in the numerical processing capacity of computers and especially from the increased versatility of the tool known as the finite element method, FEM.

In summarising terms the aim of structural analysis in any of the methods is linked to characterize the response of the structure subjected to a variety of conditions and actions. This characterization involves results on stresses, deformation and displacements, reactions. In some types of analysis like the non linear analysis may provide further details as damages due cracking in tension or crushing in compression by determining the ultimate capacity of the structure for varying actions imposed.

Present Structural Theory, Analysis, and understanding

The theory and also the associated concepts concerned on the structural analysis and of structural behaviour can be traced back to very general ideas about the nature how these behaviours were studied. On the other hand they are our recognitions of the different ways in which materials respond to load and of the parts played by their different strength and stiffness in determining how loads are shared between elements of a structure when there is more than one way in which they could be carried.

Statically determinate behaviour was the only law of static equilibrium that was given emphasis in the second half of the nineteenth century. Much effort was devoted to studying the equilibrium of masonry arches and then that of various types of truss (Rowland, 1997).

Basic assumptions were made about the ways in which forces would split where there were alternative paths for them to follow, which will come to the point called thrust lines or thrust paths (Philippe, 2005). To avoid the need for such simplifications it is necessary to distribute the loads according to the relative stiffness's of the alternative paths, with more loads following the stiffer paths. Initially this was done solely on the basis of assumed linear elasticity's of the materials; later on the basis of assumed plastic yield stresses and the corresponding calculated load-bearing capacities for yielding over the whole cross section; still more recently on the basis of combined elastic and plastic behaviour during the approach to maximum load. Calculation based on linear elasticity gave the deflections and internal stresses to be expected under normal working conditions, this is based on the assumption that if certain initial stresses were ignored. Calculation based on full plasticity at critical sections gave the ultimate strength.

Generally, analyses on historic structures can be seen mainly in three types: linear analysis, non linear analysis and limit analysis. Linear analysis can be linear static analysis or modal dynamic analysis. Different types of non linear behaviour exist: mechanical which is connected to the non linearity of the material, geometrical which is related to the fact that the application point of the loads changes increasing the actions and thirdly by contact in between units (Romano, 2005). It is also possible to carry out non linear analyses with damage models very useful into the evaluation of the stiffness loss at global and local level. The third type, limit analysis can be of static analysis and kinematic analysis which are aimed on obtaining the collapse loads.

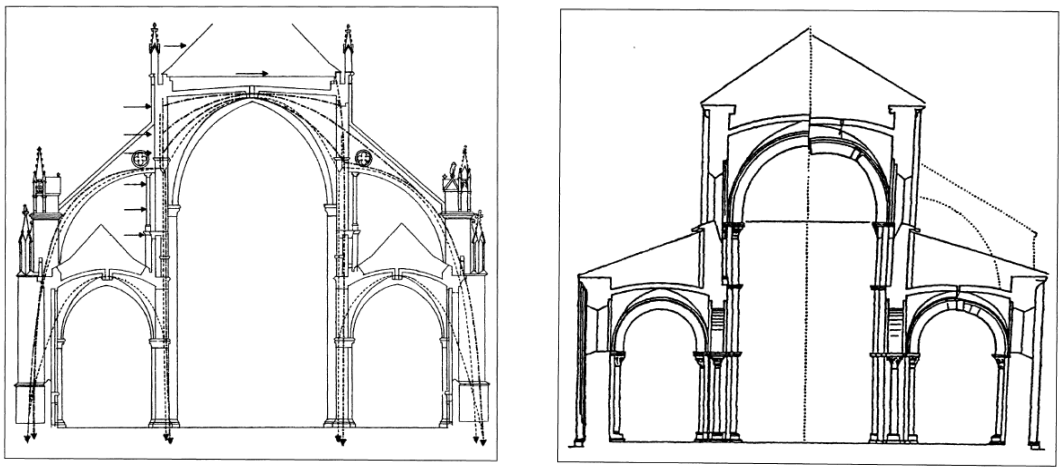


Figure 2.2: Graphical method of analysis in ancient times. (a) Thrust-line analysis of the nave of Strasbourg cathedral for self weight only, (b) Transverse section of a Romanesque church shown undeformed (on the left) and with typical deformation (on the right) (Rowland, 1997).

Nowadays analysis and simulations of historic structures are made to focus on solving the problems in characterising anisotropy mechanics of masonry. Basic tools aimed for this purpose can be linear Elastic analysis, Limit analysis or generalized matrix formulations (sectional analysis). More powerful, recent and sophisticated tools include Macro-modeling, Micro-modeling and Distinct element method (Roca, 2008).

Kept in those ideas concerning structural analysis of masonry structures (Roca, 2008), the main and most important roles of analysis in a heritage structure are:

- Its Contribution to the diagnosis of the structure.
- Relationship with history, inspection and monitoring.
- Importance in safety evaluation.
- Contribution to design and or validation of intervention.

2.3 Seismic Actions

Natural hazards, such as earthquakes, landslides and floods, are often more devastating, in terms of loss of life and environmental damage, and also have potential to precipitate technological hazards. It is apparent and visible that several types of natural hazards have the

potential to cause large numbers of fatalities. The hazards that cause the largest numbers of fatalities in one event are earthquakes.

The real need of time is to construct a structure, which is safe and economical against the lateral forces, originating from wind or earthquakes. The importance of the shear walls in the structural planning of tall buildings has long been recognized as an effective component in resisting these lateral forces (Ciampoli & Augusti, 2000). The analysis of shear wall with opening is a critical design component, especially in seismic and high wind zones. The value given to the lateral resistance in the most Earthquake prone areas is resorted to the provision of walls to resist this lateral loads..

In the southern part of Europe, there exists a severely dangerous exposure to the most value hitting earthquakes, even it is evident from the past to the recent powerful earthquakes, from the September 26, 1997 Umbria-Marche strike ,to the L'Aquila, 6.3 in moment magnitude that occurred in the central Italian region of Abruzzo on 6th April 2009, in Italy.

Depending on the seismicity zone of the location, seismic actions are accidental actions which, occur in the building's life-time. It is very important that the stability and safety of buildings located in earthquake-prone areas should be verified for seismic loads. These safety verifications are based on the results of data on the seismic activity of the location, and these data are able to lead for recommending the values of parameters to be used in the assessment of the expected and or probable seismic actions. Moreover to laying the basics for the development of methods for structural verification of newly designed buildings, safety verification is based on the inter analysis of earthquake damages and associated mechanisms of collapse, moreover subsequent experimental investigations in the seismic behaviour also provide a very important concepts in characterising the seismic behaviour and also is able to define the mechanisms that can be obtained from analysis softwares. As this verification will give the results of seismic damage and mechanisms of collapse, then these results will define the conditions where the structure needs for strengthening or future improvements. Hence, it will keep up the preservation information for the health of the structure regarding its safety.

Earthquake performance of Masonry structures

Stone masonry construction generally shows very poor seismic performance. Poor quality of mortar is the main reason for the low tensile strength of rubble stone masonry (Tianyi, 2006). Moreover, the detailed connection in between the walls, floors and floor to roof is another main issue for performance of walls under lateral loading. There is also a difficulty in multiple wythe walls how the leafs are connected. Timber floor and roof structures are usually not heavy and therefore do not induce large seismic forces. However, typical timber floor structures are made of timber joists that are not properly connected to structural walls. These structures are rather flexible and are not able to act as rigid diaphragms. Due to their large thickness, stone masonry walls are rather heavy and induce significant seismic forces.

As a binding point, the most important factors affecting the seismic performance of these buildings are:

- the strength of the masonry units and binding units (mortar),
- the quality of construction,
- the density, configuration and distribution of structural walls,
- connections: wall intersections, floor connections and roof-wall connections.

In addition to the above mentioned characters for better performance, the structural configuration of masonry buildings is highly considerable. It includes aspects like (a) overall shape and size of the building, and (b) distribution of mass and (horizontal) lateral load resisting elements across the building. Large, tall, long and unsymmetrical buildings perform poorly during earthquakes (Tomaževič, 1999). A strategy used in making them earthquake resistant is developing good box action between all the elements of the building, i.e., between roof, walls and foundation. Loosely connected roof or unduly slender walls are threats to good seismic behaviour. For example, a horizontal band introduced at the lintel level ties the walls together and helps to make them behave as a single unit.

Masonry structures, therefore, stand largely by the art of internal compressions force transmissions which carry their weights and all other loads to the ground. Cracks that occur on the masonry structures life influence the paths taken by these compressions forces but they do not usually imply to threaten stability. This concept is the very important point that most persons run to decide for the stability of an existing masonry structure because only they see

cracks, but that doesn't necessarily mean the structure is not stable. Collapse of the structure may be precipitated only by excessive deformations caused by the cracking contribution, but of which it is not the only cause, or by local compressive failures, due material compression capacity limits. Moreover, masonry structures capacity will be affected by, so with the extent of cracking. These effects are all the more important because self weight is usually the predominant load in masonry structures. Further changes will occur after completion of its construction through shrinkage and creep of the mortar and the alternate opening and closing of cracks under the influence of changes in environmental temperature and humidity which are generally associated to the material timely characteristics.

Walls of two exterior wythes and rubble infill in weak mud mortar with many air voids are typical in showing delamination and disintegration of the masonry and such kind of damage patterns are typical to such kinds of walls. Out of plane failure can occur when the connections between the exterior and interior walls are inadequate (Magenes, Masonry Building Design in Sesimic areas: Recent Experiences and Prospects from a European Standpoint, 2006). When the connections between the perpendicular walls are strong, the wall shear capacity can be exhausted, thus causing typical shear cracks to develop. The percentage of stone masonry buildings that collapsed or were damaged beyond repair in recent earthquakes depends on the general quality of construction and the earthquake intensity.

2.4 Methods in Seismic analysis

Seismic Analysis is a subset of structural analysis and is the calculation of the response of a building (or structure) to earthquakes. It is part of the process of structural design, earthquake engineering or structural assessment and retrofit in regions where earthquakes are prevalent .

As noted in subtopic under 2.2, in general there are three kinds of structural analysis. In the case of existing masonry buildings, it can be simulated under diverse analysis methods, depending on the model which best describes the structure and its seismic behaviour (Calvi & Cecchi, 2005). In the case of cultural heritage, the evaluation of the structural capacity and its seismic safety will be effectuated, on both a local level as well as on the whole, utilising the most adept analysis method. In particular the following may exist ((Romano, 2005) and (Calvi & Cecchi, 2005)):

- Static linear analysis: consists in the application of a force system distributed along the height of the building assuming a linear distribution of the displacements.
- Modal Dynamic analysis: is the identification and study of the dynamic properties of structures by applying vibration excitation. This kind of analysis is performed in the elastic range of the model.
- Non linear static analysis (so called pushover analysis): this analysis is the application into the structure of the vertical loads that may be of self weight and dead loads and a horizontal forces system particularly monotonously increasing until the reaching of the limit conditions defined for the material or in terms of other set failure criteria. It is a generalized relation between force and shifting.
- Non linear dynamic analysis (or time history analysis): is the most rigorous approach in which the non-linear properties of the structure are considered as part of a time domain analysis. It subjects the system to a ground motion record (seismic input). Hence, calculated responses are very sensitive to ground motions used as seismic input.

2.4.1. Limit Analysis: Concepts

Basics in Limit Analysis

Limit analysis is concerned with direct calculation of the maximum load sustainable by solids and structures. It is widely used in engineering practice, for example to directly determine the ultimate load sustainable by masonry arches, steel frames, concrete slabs, foundations, etc. However, practicing engineers often find that limit analysis can only be applied to geometrically simple problems (Gilbert, 2005).

Analysis can contain various criteria's depending on the method applied and also on the structure. It can be related to also the material. For instance, materials that can deform plastically under constant load may be modeled as being rigid-plastic. The method for determining the applied load that causes structural collapse is commonly referred to as 'limit analysis'. In order to mathematically model this type of material, the yield function, f, is defined in such a way that for $f < 0$ the material remains rigid, for $f = 0$, materials become plastic, and for $f > 0$ correspond to inadmissible stress states.

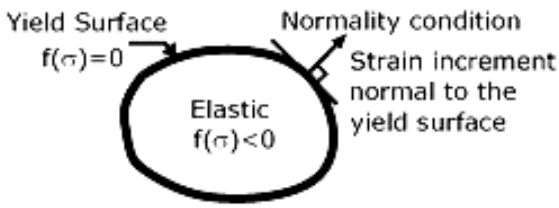


Figure 2.3: Normality condition and yield surface (Gilbert, 2005).

On the yield surface (that shows the boundary of yield in Figure 2.3), material becomes plastic and it is necessary to define the flow direction. The classical limit analysis theory assumed that the flow direction is normal to the yield surface. This hypothesis, also called normality condition, implies that this type of flow provides greatest resistance against deformation and the energy dissipation by this flow is the maximum. However, there is much evidence to show that this assumption does not always hold true in practice, especially in the case of a structure containing frictional interfaces.

The condition of convexity and the respect of the normality condition to the limit surface from the cracking strains, imply a tight connection between the theory developed by NRT (non-resisting tensile) materials and the classic theory of perfect plastic materials (Gilbert, 2005). The study of masonry structures through limit analysis investigates the very essential aspects of the behaviour at collapse and, at the same time, seems to match modern analysis techniques with geometrical static principles rising from traditional theories.

Theorems of Limit Analysis

- Safe (Lower bound) Theorem: The lower bound theorem of limit analysis states that if we can find a stress distribution that lies totally within the yield surface, the corresponding load is less than or equal to the collapse load. The collapse load of a structure is one from set of loads that transforms the structure into a mechanism in failure (Figure 2.4 (a)).

- Upper bound theorem: If a kinematically admissible mechanism can be found, for which the work developed by external forces is positive or zero, then the arch will collapse (Roca, 2008). This theorem also implies that if we have various geometrically

possible strain fields, and we use virtual work to identify the level of applied load causing the strain, that load will be greater or equal to the collapse load. Note that it may be found that several geometrically distinct strain fields can lead to the same collapse load (Figure 2.4 (b)).

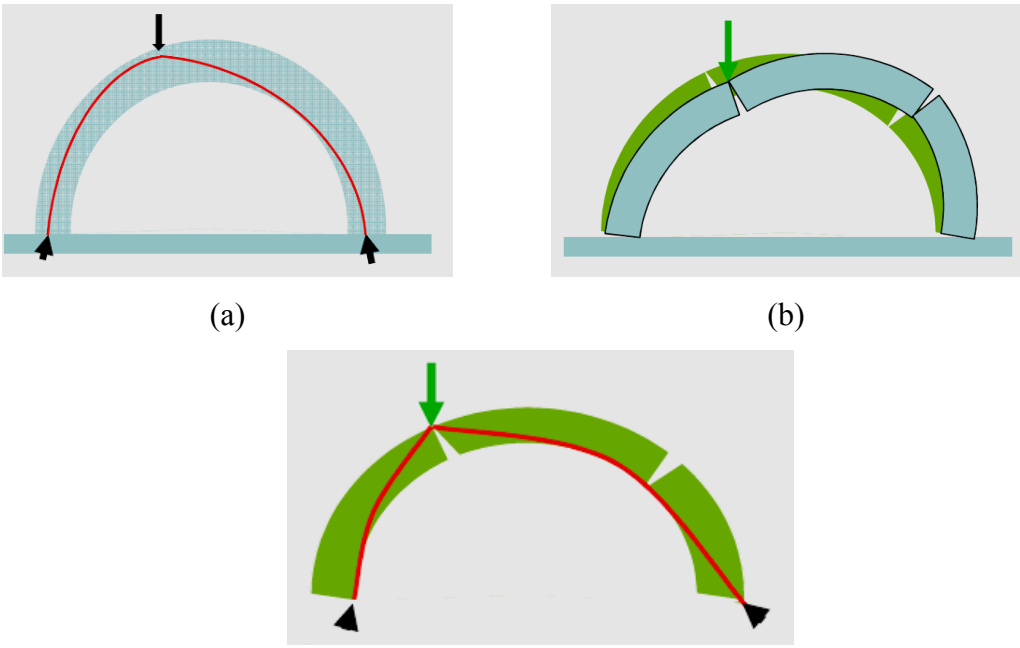


Figure 2.4: Theorems of limit analysis applied to an arch; (a) Safe (Lower bound) theorem, (b) Upper bound theorem, (c) Uniqueness theorem (Roca, 2008).

- The uniqueness theorem states that if a load satisfies the lower bound theorem as well as the upper bound theorem, the load is therefore equal to the collapse load, which is thus uniquely determined. When this occurs, the load is the true ultimate load, the mechanism is the true ultimate mechanism, and the thrust line is the only possible one (Figure 2.4 (c)).

Applicability of Limit Analysis in Masonry structures

Limit analysis to masonry structures bases back to the assumptions and clear hypothesis from Heyman (1966) (Orduña A., 2003). These hypotheses together with the basic plasticity theory are applied to masonry structures. Here, the plastic theory is implying that the failure is due to

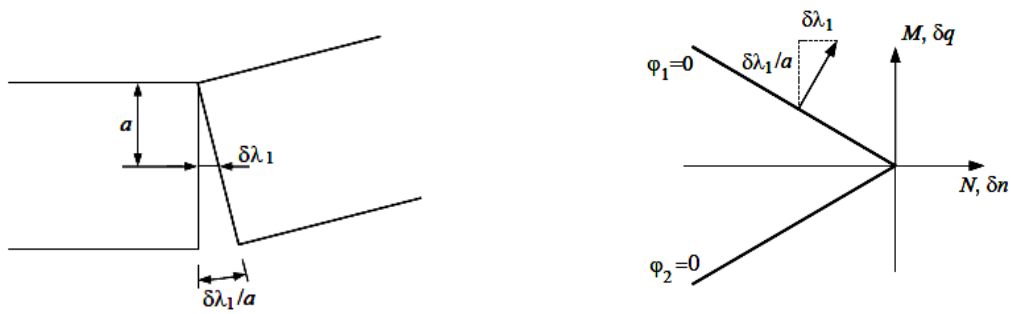
the formation of a mechanism or plastic hinges. The following three basic show the Heyman hypothesis (Heyman, 1995):

(1) Masonry has no tensile strength: The low tensile strength and the quasi-brittle tensile failure of masonry justify this condition of hypothesis. This is observable from the failure mechanics of masonry structures which have basically crack problems.

(2) Masonry has an infinite compressive strength: masonry, construction material, is a high compressive capacity material exhibits to have, indeed, to have a limited compressive strength. This is because, in most cases, collapse of masonry structures is not governed by compression failure, but is due to cracks opening and mechanisms formation. But this assumption is slightly un conservative, it is needed to have a limit and hence in this study it is limited to a finite compressive strength (Romano, 2005).

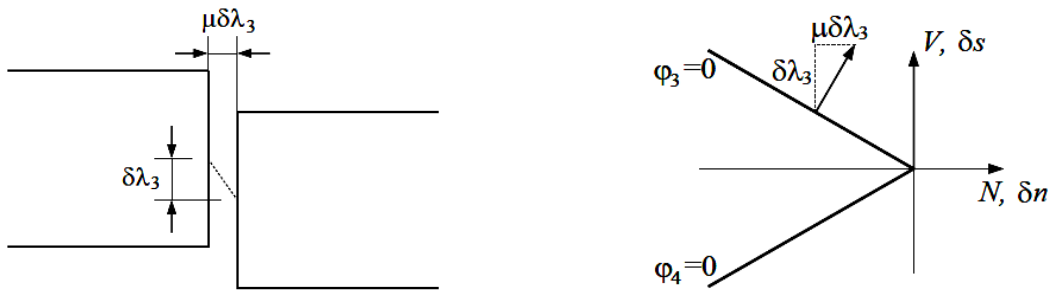
(3) Sliding types of failure in between units cannot occur: there is sometimes evidence of sliding in between unit directly on joints during failure. But generally, even if this failure occurs, the structure retains its shape remarkably in good condition hence it can be assumed that sliding is not pertinent failure.

Furthermore, for the structural assumption which is not related with the material masonry can be the limitations on displacements. The idea is implied on failure occurrence under small displacements. As presented in Orduna (2003), the variations in the external displacements depend linearly on the variations of the internal strains. it must be taken into account that if significant overall displacements are expected prior to failure the results might be inaccurate and generally unsafe (Orduña A., 2003).



(a) (b)

Figure 2.5: Hinging and the corresponding (a) yield function, (b) normality condition (Orduña A., 2003).



(a) (b)

Figure 2.6: Sliding and the corresponding (a) yield function, and (b) normality condition (Orduña A., 2003).

2.4.2. Seismic Limit Analysis

Throughout the centuries, earthquakes are the most representative causes for damage and losses to the cultural heritages that have been come from past times. Among the major issues in Europe and also in most part of the world of research is the seismic vulnerability of these heritage structures. Evaluation of seismic performance of masonry structures by the method of displacement based analysis was used in earlier decades. The procedures related to this method of seismic performance are based on a comparison between the displacement capacity of the structure, obtained as capacity curve, and the displacement demand of the predicted earthquake that is supposed to happen on the specific region the structure is located.

Limit analysis simplifies and gives enough relation into the load capacity and collapse mechanism with a reduced number of material parameters necessary to perform the analysis. Noted in different methods of seismic analysis in subtopic of 2.4, different computation strategies can be followed to perform the seismic analysis of masonry structures. Nowadays, there is an interest to completely define the characteristics of structure and material. Even though of this fact, in order to avoid highly sophisticated procedures associated to time-consuming nonlinear dynamic analysis, a simplified approach based limit analysis is frequently used. Moreover, application of strengthening tie (the most important and commonly used strengthening technique) is also possible in the analysis.

The Seismic Coefficients are dimensionless coefficients which represent the (maximum) earthquake acceleration as a fraction of the acceleration due to gravity. When a seismic coefficient is defined, an additional body force will be applied to micro element may be in the form of additional external load as in the case of putting transferred loads from floors to walls. Body force (due to gravity) is simply the self-weight of a micro element, in this case, the stone masonry units. Then, it is clear that:

Seismic Force = Seismic Coefficient * Body Force (due to gravity)
= Seismic Coefficient * (area of element * unit weight of material)

The Seismic body force is vectorially added to the (downward) body force which exists due to gravity, to obtain the total body force acting on the element (Romano, 2005).

Some Difficulties in Seismic Analysis

The difficulty of modelling the masonry depends on the following factors:

- masonry is a discrete material (blocks and mortar) in which the dimension of the single constituting element is large compared to the dimensions of the structural element;
- the geometry, origin and blocks placing can vary considerably;
- blocks are stiffer than mortar;
- the mortar thickness is limited (if compared to the block dimensions);
- Stiffness of the vertical joints is remarkably smaller than the one of the horizontal joints.

2.5 Affecting Parameters

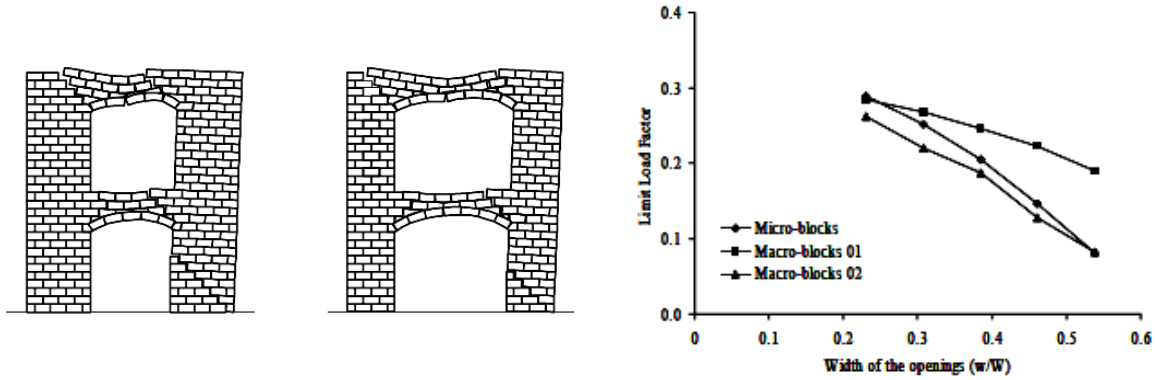
Earthquake effects are dependent on too many things starting from the material make up to the global response behaviour. Configurations of the systems inside the structure also create another set of depending topic for the seismic capacity. Unlike the earlier times where there was no any idea of how to configure a building for seismic resistance, nowadays at different levels of study there exist researches and their associated problems on how to understand the response and behaviour of masonry building to ground motions. Links to the parameters on the determinacy to the behaviour and actuation for ground motion response for a specific structure can be in many ways. It may be of mechanics, geometry or time behaviour of forces or responses of structure. As mentioned earlier, the configuration of systems is found to be basic and also important.

Many experimental and numerical researches starting from early decades are done concerning on the seismic behaviour of masonry building prototype. The tests have differences in their prototype in the geometry level and in material level. These two things are the main valued differences among the tests done in time. Hence, geometry is the parameter that's under investigation here in this thesis. Geometry implies for the variations in the models in respect to the masonry units size, masonry units arrangement, the wall aspect ratio of the model by itself, the load applicable to the structured that can vary with time, and also the compressive strength or stress of the masonry.Previous studies by rigid block limit analysis include by Livesley (1978), Giuffre (1993), Baggio and Trovalusci (1998) and recently by Orduna (2003), and also others.

Remarkable parametric analysis is done by Orduna (2003), which he applied to different walls. The walls taken were the same to those walls analysed by Baggio and Trovalusci (1998). Parametric studies by Orduna (2003) are linked with varying the wall aspect ratio and also the units in their arrangement and also bond, the masonry units size, the overload with respect to the horizontal load and effective compressive stress that is found as ratio to the standard compressive stress. Walls of different aspect ratio are analysed to each category of parameter. Hence it will form up to a multi-parametric analysis system. Orduna also compared the limit analysis results from those by simplified models by macro system and from De Buhan and De Felice results to each case.

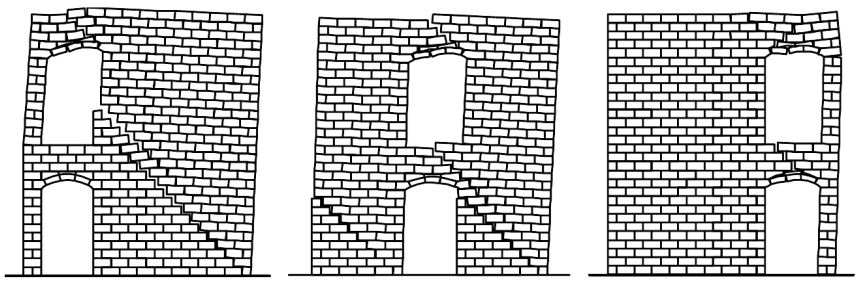
Shown in Figure 2.8 is a parametric study of opening position on the effect of seismic load factor. The behaviour of a two stories shear wall with two openings has been studied in relation to the position of the openings and to the width of them. The results indicate that the limit load factor decreases as the openings approach to the right side of the wall for lateral load from left to right (Orduna, Roeder, & Araiza, 2006).

Moreover in the related studies, the width of the openings normalized with the total wall width was also analysed. Obtained results are shown in Figure 2.7. This figure illustrates that the limit load factor decreases as the width of the openings increase.



(a) (b)

Figure 2.7: Parametric analysis to the effect of width of opening; (a) failure mechanisms for micro blocks, (b) Safety factor versus width of openings graph, (Orduna, Roeder, & Araiza, 2006)



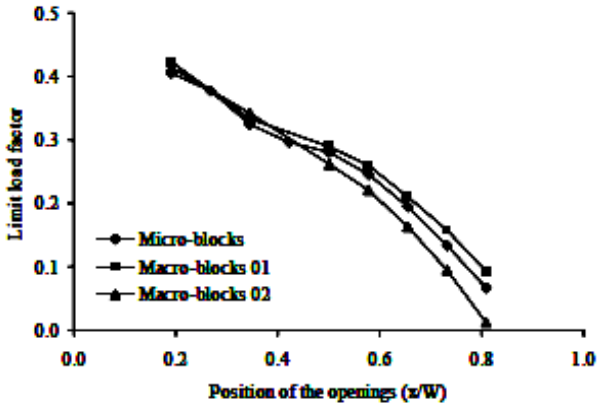


Figure 2.8: Parametric analysis to the effect of position of opening; (a) Basic geometry of macro models, (b) Failure mechanisms, (c) safety factor versus position of opening graph, (Orduna, Roeder, & Araiza, 2006)

As a matter of limiting the work vastness and also relevancy, the parameters relevant for effect in seismic analysis are those discussed above in inalienable association to the realistic conditions of existing masonry structures.

Failure mechanisms of Walls and Piers

Walls are one of the important structural functioning parts of historic building which basically used in load bearing purpose. It makes up and determines the response to loads, which mainly they carry lots of compressive pressures. Regarding walls in historic masonry buildings, there has been done experiments to verify the response to a horizontal and simultaneously an increasing compressive force. For instance, studies by Oliveira (2003) have shown realistic modes of failure and also arguments on numerical and experimental results are observed. The observed failure modes are clearly associated with the vertical load applied. For lower compressive stress levels, failure happened by rotation and sliding of part of the wall; whereas for higher vertical loads, cracking started to become noticeable (Oliveira, 2003).

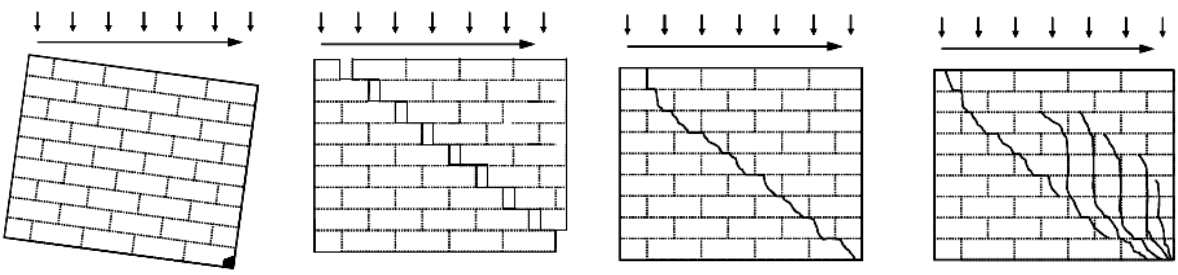
Experiments have shown almost similar characteristics or modes of failure. In Figure 2.9, it is presented the different failure mechanisms of the standard reference wall. Recalling to the general failure modes of masonry walls under lateral and vertical loads, there exist the following four major kinds of failure modes:

- *Rocking motion*: In this case, the failure is accompanied by the existence of relatively lower vertical loads as compared to the other failure modes. It is susceptible to overturning. This is to the cases when we will have low values of vertical loads and it may be associated to the compressive strength of the surface the wall rests on.

- *Frictional sliding*: this type of failure is mainly caused by the failure in frictional resistance. The joints, either vertical or horizontal, will be overcome by the lateral forces together with the effect of the increased vertical force.

- *Tensile unit cracking*: In this case, as shown in Figure 2.9, the failure is due to the masonry strength against lateral and also compression forces. The formation of the diagonal lines shows the effect due the lateral loads. But this type of failure due crack in the units is not allowed in the Block2D software, implying that the blocks are assumed to be rigid.

- *Compression failure*: this one simulates the high compressive exposure to the wall. It is expressed by having high vertical loads as compared to that of the combined horizontal load in comparison to the other types of failures.

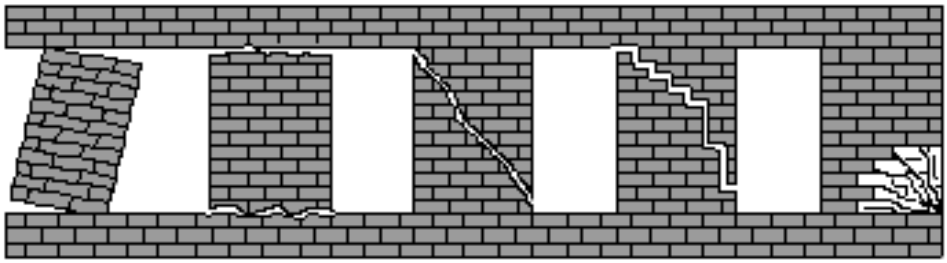


(a) (b) (c) (d)

Figure 2.9: Typical failure modes of masonry walls, subjected to in-plane seismic load, (a) Rocking motion, (b) Frictional sliding, (c) Tensile unit cracking, (d) Compression failure, (Lourenço, 2008)

Similar situations of failures are observed on piers found embedded in a masonry structure. Compressive stress, aspect ratio, boundary conditions, and relative strength between mortar joints and units determine the failure mechanisms of masonry piers. Many experiments have shown that masonry piers can have considerable deformability and ductility if certain failure

mechanisms prevail. They gave four typical crack patterns and failure modes for the masonry piers as shown in Figure 2.10.



(a) Rocking (b) Sliding (c) Diagonal tension (d) Toe crushing

Figure 2.10: Different crack patterns (or failure modes) on masonry piers (Tianyi, 2004).

Several post-earthquake investigations and experimental research have showed that the typical failure modes of a historical masonry building can be grouped into the following categories (Tianyi, Experimental Investigation and Numerical Simulation of an Unreinforced Masonry Structure with Flexible Diaphragms, 2004):

- Lack of anchorage and good connection in between masonry walls and diaphragms,
- Anchorage failure,
- Out of plane failures of masonry walls,
- In plane failures of walls (which is going to be studied in detail here),
- Combined in plane and out of plane failures, including cracks at the wall intersections,
- Floor Diaphragm related failures.

The most serious and hazardous type of failure modes from those is the out of plane failure of structural walls. The failure of the structural walls is to mean the failure of the whole structure gradually. However, this type of failure can be prevented by properly anchoring the masonry walls to the floor and roof system. In this case, the in-plane failure of walls is the dominating failure mode which is the main research focus of this dissertation.

2.6 Seismic safety of Historic structures

Masonry buildings, whether ordinary or monumental, still constitute one of the most vulnerable classes of structures. Despite the numerous studies that were carried out in recent decades, still almost countless open problems exist regarding assessment methods, strengthening strategies and techniques, availability of adequate design-assessment strengthening standards and codes of practice (Curti & Podestá, 2006).

Recent earthquakes have shown that historic buildings retrofitted to withstand earthquakes survive better than those that have not been upgraded. Even simple efforts, such as bracing parapets, tying buildings to foundations, and anchoring brick walls at the highest, or roof level, have been extremely effective. It has also been proven that well maintained buildings are found better than those in poor condition during and after an earthquake. Thus, maintenance and seismic retrofit are two critical components for the protection of historic buildings in areas of seismic activity. It makes no sense to retrofit a building, then leave the improvements, such as braced parapets or metal bolts with plates, to deteriorate due to lack of maintenance.

Damage to historic buildings after an earthquake can be as great as the initial damage from the earthquake itself. The ability to act quickly to shore up and stabilize a building and to begin its sensitive rehabilitation is imperative. Communities' without earthquake hazard reduction plans in place put their historic buildings as well as the proper safety and economic well being of their residents at risk.

CHAPTER 3

PARAMETRIC SEISMIC ANALYSIS

3.1 Introduction

For the description to the parametric study of masonry structures, there comes a starting point that will associate the geometry of the structures to the specific topic under this circumstance, is the seismic coefficient, or the safety factor. As from the previous studies and also from the observable sources about historical structures concerning their seismic limits, the geometric relations among the structural elements and also the connections, moreover the material characteristics are the main determinant things to this limit analysis. Hence, it is basic that having the relation in between whether the material effects or is the geometry of the structure that affects the response of the structure highly. In other words, essential differences on the material effects and or the structural configuration should be studied more deeply as from the fact that masonry structures are difficult to characterize. For this purpose, in years, there has been development of some analysis softwares depending on their way of modelling; for example, in the case of micro-modelling, the interface elements in the model represent explicitly the weak mortar joints and mortar-unit interface, while the blocks model the masonry pieces; and in the case of macro-modelling, the joints represent or model the cracks and the blocks model undamaged material.

In this study, as mentioned in before, the Block2D software is used, by which it is based on defining micro blocks to represent the specific masonry units. This is basically important to define the study results from the geometric effects to seismic limit analysis. From the structural aspect of view, the configuration of openings, the and also the sizes affect dominantly the seismic performance of the building. Masonry structures have different non-linear behaviours starting from the materials they are made up of. When we come to the old buildings with a historical and/or artistic value, defined as heritage buildings, the uncertainties and difficulties increase. This is because the original design documents are seldom available, the structural diagram is also difficult both to define and to model (this might be also because

of the possible alterations made on the original structure), the properties of the materials (both in the original and the present state) are unknown or difficult to evaluate. Hence, the need to study this is very important and very difficult, as mentioned.

3.2 Short Description of the Block Software

The Block2D software was developed taking in mind the seismic assessment of ancient masonry constructions (Orduña A., 2003). It is an analysis software that performs the necessary calculations for the limit analysis of rigid block assemblages and there are different versions for two dimensional, 2D and three dimensional, 3D models, respectively. It bases its assumptions on the realization of the systems in the masonry structures. It is a good representation of the structural modelling of rigid blocks interacting through frictional interfaces. Moreover, on conditions when there is a need to present or see the effect of some interventions like strengthening ties, this software can be applied to assess such on such cases also. As it is clear that the realization of strengthening ties in existing building might be on different situations, and in parallel one can check for such situations easily. The most important thing with this software is in the situations that, when you have different failure or collapse modes of a specific existing building, then this tool is very important in that it will provide you the most possible failure mode amongst the other likely ones from the set of abacus of failure modes.

Thrusts of compression for the specific failure mode to each joint in between blocks are also possible in the Block2D AutoCAD (Orduña, 2004).

3.2.1 Modelling and Interpretation

Modelling in Block2D

Modelling in Block2D is done using AutoCAD, but then there are three possible ways of computing the analysis. Once there found the model in the AutoCAD, then the pre- and post-processor visual basic application will extract the data of the block analysis. There after the output from the extraction of the drawing will be an input for the solution part and post-processing. The solution phase is done by the mathematical programming modelling environment GAMS. The limit analysis mathematical problem has been modelled in the

GAMS language and the data for a particular model can be read separately. The GAMS software produces an output file with the solution information that is later read by the post-processor in order to draw the failure mechanism and the joints thrusts in the model AutoCAD drawing.

Models in Block2D consist of the cohesionless Coulomb criterion for shear stresses and a no-tension and limited compressive stress criterion for normal stresses. The Coulomb friction model features a non-associated flow rule with zero dilatancy and the torsion failure mode is included in the three-dimensional model (Orduña, Block User's Manual, 2004).

During modeling, there are options for defining the characteristics discussed before, for instance, one can apply the loads that can be applied either laterally or vertically in addition to the existing gravity loads. In such cases, there is a need to apply the attribute edit options. One important thing should be noted that during lateral loading, there exist a possibility to calculate the seismic coefficient as a function of both the applied force and the gravitational force. Moreover there exist the following terms and their concepts;

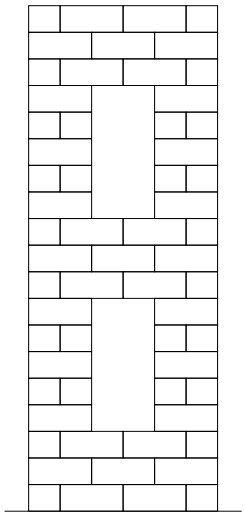
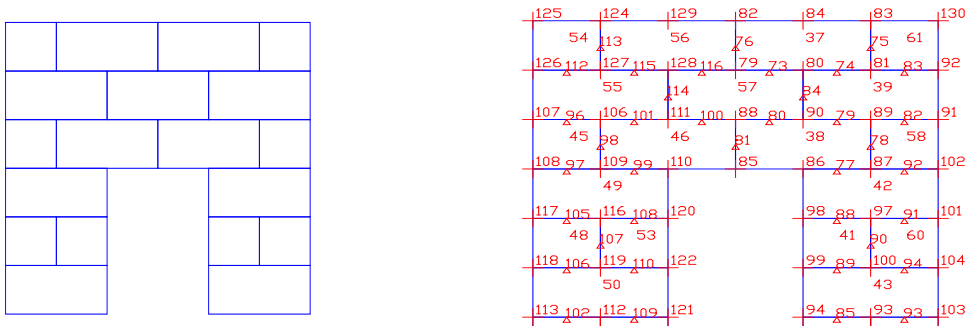


Figure 3.1: Typical Model developed using Block2D.

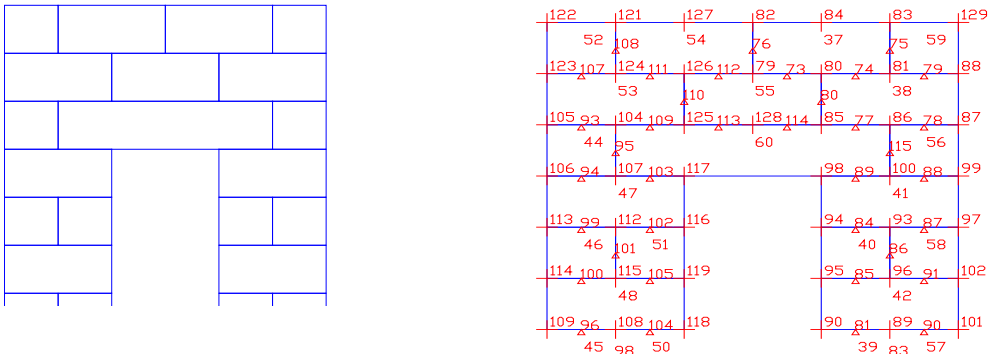
Inputting additional loads to the blocks is also possible. This is done by inserting the loads, which may be forces or moments, to the centroid of the block within the geometric tolerance.

Openings can be normal openings having joints of openings in the boundary of the opening, as second alternative it may be with lintels or arch form shapes. In any of the three cases, it is

possible to create the rigid blocks in the software. This makes it to associate the failure mechanism to the more realistic one. The change in the failure mechanism is also clear to happen when the openings are changed.



(a)



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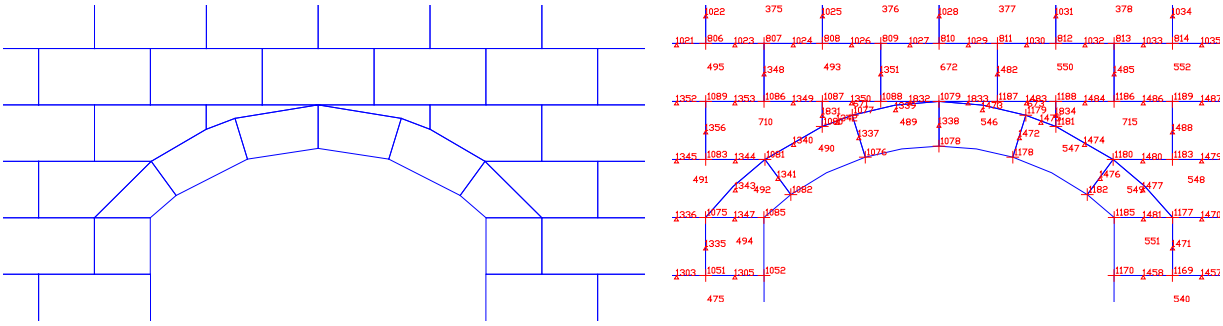


(c)

Figure 3.2: Typical representation for the connection between masonry rigid blocks in the case of (a) normal openings, (b) lintels and (c) arches

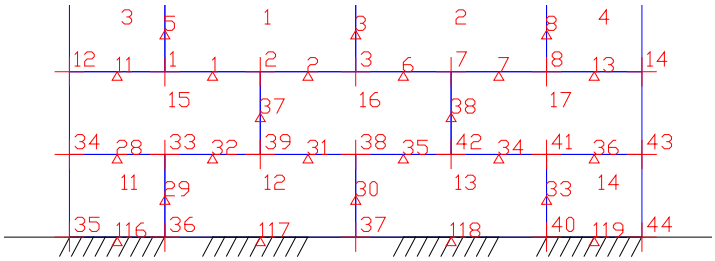


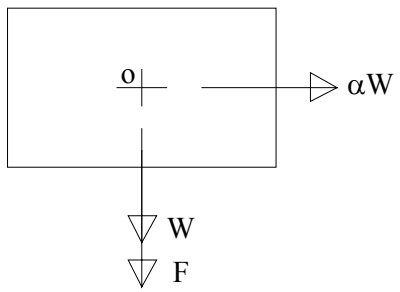
Figure 3.3: Details in the boundary condition, this shows the case for fixed support

The Block software was developed specifically for seismic assessment of structures. Concerning the gravity loads, by default the constant loading consists of the blocks self weights downwards applied, and the variable loading consists also of the blocks self weights but horizontally applied. In this way, the safety factor that results from the calculations is also interpreted as seismic coefficient. Nevertheless, it is possible to apply additional, constant and variable loading, and also to substitute the default seismic loading for user defined variable loading. More notes can be consulted from PhD thesis of Orduña, 2003.

3.2.2 Results and Applications

In the structural analysis of masonry structures, the most important challenging things are, as already mentioned, the non linearity in the behaviour of the masonry material and the way of modelling. The modelling part is the more important part, hence, from basic elementary properties of the interaction between the masonry assemblies, it is possible to have some formulations like applying the principle of non-interpenetration of units when the units are under high compression and also almost no tensile strength among units can also be taken as an assumptions in order to simplify the problems in the real modelling world. In this aspect Block2D has reduced the number of complicated properties to only in applying the number of characteristics and obtain the very important results of the structure, for instance, the mechanism of the structure and the value of the factor the multiplies the total gravitational load that the wall or facade can carry in the in-plane direction. Consider the block unit in Figure 3.4, the lateral force necessary to create a failure mechanism will be applied at the centroid of the block and moreover the lateral force will be computed in one of the two cases: it may include the additional forces or it may not include. The frictional force will be needed to keep the balance with lateral forces. Generally, the Block2D can be mostly applied to any walls having a medium structural configuration, in other words, it is more effective on studies

in intermediate walls. Similarly, the Block3D is also used. But in the later case, the structure 3D is taken whereby it consist of macro blocks.



Where

W = gravitational self weight,

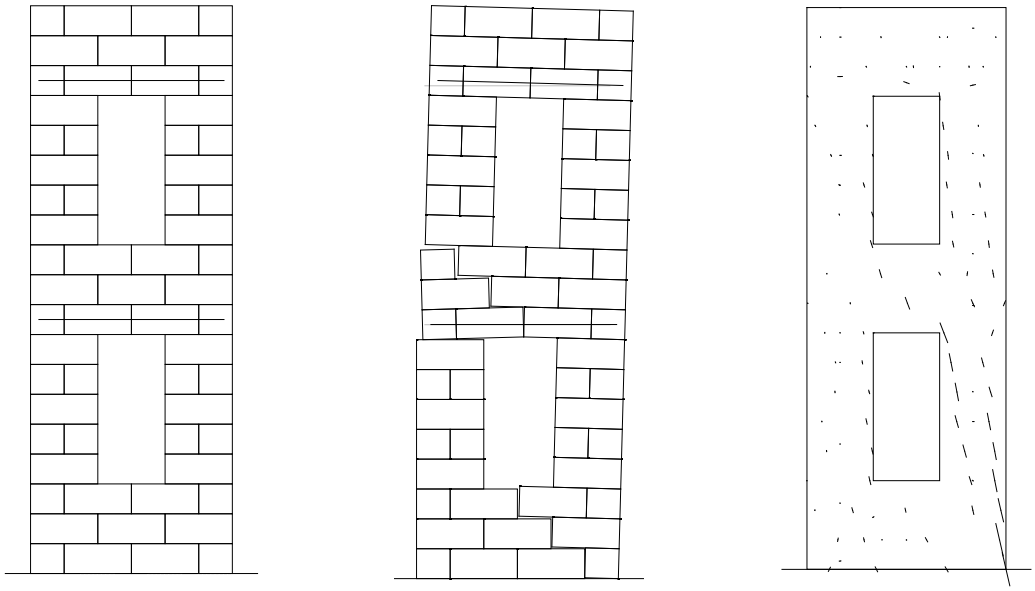
F = additional constant or variable load,

α = seismic coefficient.

α W = the lateral force.

Figure 3.4: Typical representation of a block and its derived forces

The recent type of strengthening on existing historical structures may be of different types like applying strengthening ties. Usually, ties are applied on the walls that are parallel and he aim is to prevent out of plane failure. But, there is a possibility to have the ties close to the walls that are parallel to the ties. Hence, there will happen a possibility for the ties to withstand also the tensile forces coming from the walls parallel to it, in addition to the tensile force from the out of plane behaviour of the normal walls to the ties which they are supposed to carry on. This recent very important idea is put into analysis and it is possible in Block2D. At the same time, this is most recent and very important part of assessment on standing historic structures exposed in earthquake prone areas mostly in Italy. From common experiences there in existing ones, the ties are applied on the floor levels at the bottom. This is possible to this analysis tool by modelling the ties having specific material characteristics, tensile strength. Hence, in this thesis, it is proposed to apply the strengthening ties on the 2D models that are detailed in Chapter 4. Below shows the typical results forms from a simple model.



(a) (b) (c)

Figure 3.5: Typical results from the Block2D: (a) 2D model with ties, (b) failure mechanism, (c) thrust lines of joints

3.3 Parameters under study

The evaluation and even the definition of adequate simplified block models, here in this study, are used to assess basically the in-plane load-bearing capacity of masonry walls, in terms of both strength and displacement that play a fundamental role in the seismic verification of masonry buildings. The strength can be explained by the parameters stated before as the effective compressive strength of the masonry assembly even the friction coefficient expresses also the strength against sliding, and the loads acting on the wall that appear in the form of gravity loads and transferred loads from the floors contained by the walls periphery are the main playing parameters in the aspect of load variation. In this thesis, a critical review and confirmation of the most failure modes of piers that can be of rocking, crushing, bed joint sliding or diagonal cracking are proposed.

In the real existing masonry structures, a close look on the basic differences beyond to that of the structural content, it is easy to see the differences on the masonry units, their arrangement and their regularity, their size or even their strength quality. Hence, once one comes to think of the safety of these structures, there needs to relate the safety against those parameters.

Based on these circumstances, the most important parameters that are considered in this study are:

Wall aspect ratio (Wr): this parameter describes the ratio of the wall height to the width of the wall.

Unit aspect ratio (Ur): this ratio is the value representing the height to the width of the masonry unit.

Unit size or Scale effect (Sr): this is a parameter for the size variation of masonry units for the same unit aspect ratio. The variation of the masonry unit height with respect to the wall height is expressed by this parameter. It, in other words, expresses the magnifications of a masonry unit. This will be very helpful when one wants to realize the variation in the units of masonry how affects the in-plane lateral capacity.

Overload (Or): this is another parameter that represents the ratio of the load applied to the wall as the result of transferred load or may be directly applied load. It realizes the load variations that occur to the wall during some modifications on the structure as a whole during the life of the structure.

Bond ratio (Br): is the bond in between the masonry units, it depends on the unit size and also on the bond type.

Effective compressive stress ($f_{ceff,r}$): Masonry units have their own compressive strength and equivalently the masonry joints also have their own compressive load carrying capacity, when they are working together, they will have another effective compressive strength. This is the net compressive strength obtained from the combination of the masonry unit and the joint mortar compressive strength.

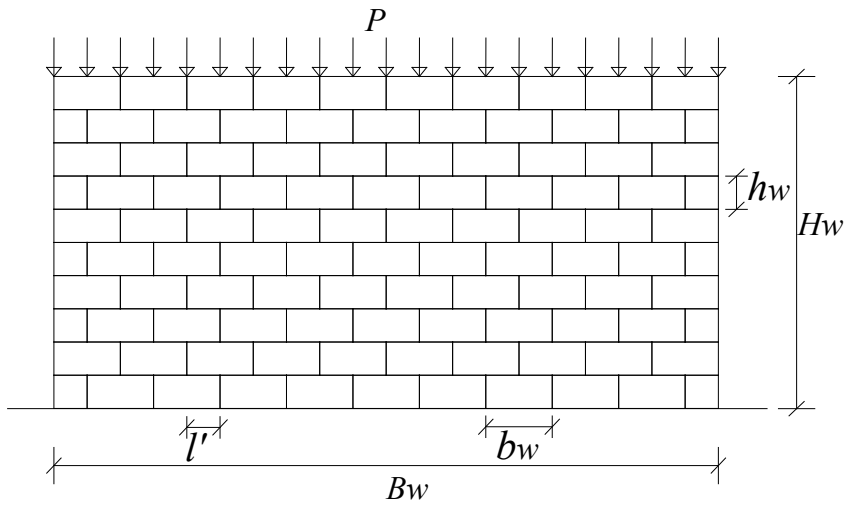


Figure 3.6: The standard shear wall and its parameters

The variables used in this study are the following; as shown in Figure 3.6 above:

Hw and **Bw** are the wall height and length, respectively;

$\mathbf{h_w}$ and $\mathbf{b_w}$ are the masonry unit height and length, respectively;

l′ is the overlap of units in consecutive courses, this measure represents the bond;

P is the vertical uniformly distributed load applied at the top of the wall.

Hence, relating these assigned variables, the parameters studied are:

$\frac{Hw}{Bw}$..... Wall aspect ratio (Wr)

$\frac{hw}{bw}$..... Masonry unit aspect ratio (Ur)

$\frac{Hw}{hw}$..... Unit size or scale effect (Sr)

$\frac{b_w - l'}{b_w}$..... Bond ratio (Br)

The overall summary of these parameters is shown in Table 3.1 below.

Table 3.1: Variables and definitions for the shear wall parametric analysis

Designation to the parameter	***Parameter***	***Definition***
Wr	Wall aspect ratio	Hw / Bw
Ur	Unit aspect ratio	hw / bw
Sr	Unit Size ratio or scale effect	Hw / hw
Br	Bond ratio	$(bw - l') / bw$
Or	Overload ratio	Ratio to the considered standard load, P
$f_{ceff,r}$	Effective compressive stress ratio	Ratio to the considered standard compressive stress, f_{ceff}

Moreover, the validation of the results to the previous studies of Agustín Orduña, to his PhD thesis in 2003, is also placed and compared with the current results accordingly.

3.4 Parametric Analysis

It is fact that, it is not so possible to logically state the condition of safety by simply considering the dimension variations in between different walls by simple observations. There is no any basis and characterizing precise definition for conclusions coming from simplified guesses. There comes into question that, for instance when an existing structure is changed by some means like floor failure or may be application of strengthening structures or it may be exposed to another additional floor load, it is true that the response to seismic loads will be changed.

In order to formalize the parametric analysis, there needs to have a standard or reference wall with other standard unit dimensions so that the variations from this reference can be aligned to conclude about the seismic factor due that variation. And it is this very important thing that will lead to more general findings on characterizing safety factor. Hence there needs to study this limit analysis concepts for masonry, as it is not even studied before except for the most

recent some studies by Hodge (1981), Nielsen (1999), Kamenjarzh (1996) and Orduña (2003).

3.5 Masonry wall Parameters

Masonry walls of different type exist in varying properties of material. Some of them are shown in Figure 3.7. Standardized walls of different properties with the combination of the variation in some other parameters like the wall aspect ratio, the unit aspect ratio, the bond ratio, the effective compressive stress, and the overload. The important and the main need of this thesis is to observe the in-plane seismic assessment by playing with this combination to the idea in compatible ways coming to the real existing masonry walls. This comes up to the precise conclusions.

Figure 3.7: Different types of masonry walls

Depending on the assumptions, it is possible to obtain specific seismic coefficients. The general information's for the standard wall are listed below. In other words, it informs the common assumptions for the each kind of parametric analysis.

- Wall aspect ratios considered are: Wr= $\frac{Hw}{Bw}$ = 3/10 (≈0.33), 1/2, 1.0, 2.0 and 5.0.

- Unit Aspect ratios: Ur=$\frac{hw}{bw}$, since we can have lots of same unit aspect ratio, a unit size ratio of 1/3, 1/2, 3/4 and 1.0 were used. The size of the standard masonry unit is 0.6m length with 0.3m height and the wall is of thickness 0.5m.

- Unit Size ratio or scale effect: In changing one parameter, there need to make constant for the other variables so that the net effect due that parameter is found clearly. Hence, as equivalent idea to this, the size of masonry units used here in the wall aspect analysis is 10 (i.e. $\frac{Hw}{hw}$=10), as can be seen also in Figure 3.6. This is another important consideration on the determinacy to safety factor, it is discussed more deeply on the subtopic under 3.5.5. The values taken generally for the parametric studies are: Sr= $\frac{Hw}{hw}$ = 3, 5, 10, and 20.

- Effective stress ratio is taken as a reference to the ratio with the standard value taken, that's equal to 8MPa, and the values assumed for the trend to the effect due increases in effective stress are multiples of this value, i.e. 8MPa. Values are 0.0125, 0.125, 0.5, 1.0, 1.25, 2.5 and infinite.

- On the loading condition, this is important that the increase may go to the cases where there will happen that, for instance when the floor is changed by a concrete floor or when there exist a strengthening means that will have a gravitational load as an external loading is taken into consideration. But for values more high than this value are unrealistic, hence values like multiples of the floor load are normally considered to see how seismic values change. In the case when needed to see the trend for loading changes, it is done up to some very high values of loading. But that was just to know what type of trend can be obtained, in other words, more general or deductible idea can be generated. For typical timber floor loads, a loading of 2.2kN/m^2 is considered. And the transferred load from the floor, considering a typical span of 5m, will be 5.5kN/m. Finally, the concentrated load that will be on the blocks at the top is found to be 5.5kN/m for 0.6m unit length, which is equal to 3.3kN.

In the presented results from Orduña 2003, the wall aspect ratio was considered in such cases that there is no overload (Orduña A., 2003). But, here in order to realize exact situations for walls that are incorporated in a historic masonry structure, it is considered to have included the loading condition as described above. Moreover, it is clear that walls are loaded in a structure as they are the main load bearing components of the structure. And also, moreover, the comparison for the unloaded case is also seen for the sake of confirmation to the results in verifying the effect of loading on a wall that is exposed to lateral loads. These parts are discussed as per the respective parameter.

- Regarding the bond ratio:

Bond ratio refers to the ratio of the the common bond to the total length of unit, it is formulated in Table 3.1. The situation of bond in reality is generally very different. Bond can have the two extreme types called stack bond and regular or common bond. The intermediate bond types are discussed out in the sub topic of 3.5.5 under bond ratio parametric analysis.

- *Stack bond* is given by the bond ratio of Br=1.0 (i.e. 50% bond), where by dimension of masonry unit $b_w = l'$ which implies to the bond type as shown in Figure 3.8.

- *Regular* or *common bond* is given by Br=0.5 (i.e. 50% bond), where by the $b_w = 2l'$, this is the most common type of bond existing in masonry units. And it is also the standard or reference bond for this parametric analysis.

(a) (b)

Figure 3.8: Bond types (a) regular or common bond with bond ratio of 0.5, (b) stack bond with bond ratio of 1.0

3.5.1 Wall Aspect Ratio

Among those parameters describing wall assemblies, the wall aspect ratio, is the basic one, where by the major difference in existing masonry structures there happens that they have difference on this parameter in addition to the material and other minor parameters.

In this part of parametric study for seismic safety, a masonry wall of dimensions Bw (describing width of the wall) and Hw (describing height of the wall) is generally taken as the basic reference called as standard wall. The safety factors are derived of the ratio Hw/Bw, hereafter termed as wall aspect ratio. In the wall aspect case, the constant type of unit aspect taken was Ur=0.5. Keeping the same type of masonry assembly or masonry units, it is possible to develop relations among this parameter and the seismic safety factor, α. How the variation of the wall size affects the lateral load factor is understood by varying the width of the wall not instead of varying the height of the wall. It seems more realistic that varying the width is more representative to the real situations which we can see now on existing historic structures. Using the Block software, there results the realistic failure modes that come as an input for the kinematic mechanism analysis for the lateral load capacity. Moreover, it will give the one and only one failure mechanism amongst the possible failure modes that are derived by thinking failures of the possible combinations of the masonry assembly.

The wall aspect ratio studied by Orduña (2003) is also studied here for the validity to the study. In this parametric part of analysis, according to his analysis the wall presented is not simulated for the loads that can be transferred from floors. But, here in this thesis, for the wall aspect effects on the seismic limit is acted under an application exposition of the wall to a defined realized vertical distributed loads in such a way that they that act on the centroids of each block that represent the top of the wall.

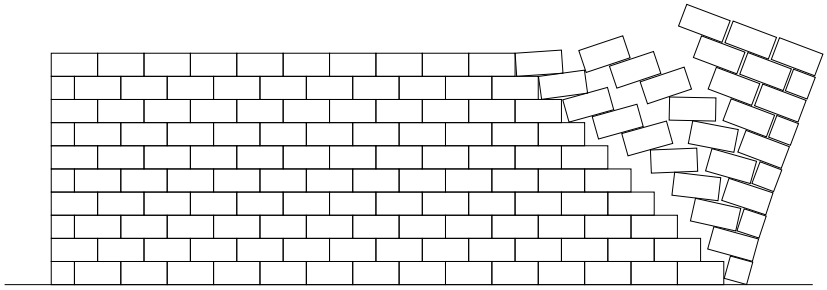
Results

The wall aspect is analysed in two different ways, as mentioned above, it is conducted for the case when load is associated to represent transferred loads from floor, and for the case when the wall is simply standing. It is, of course, necessary to simulate the loads. And the application of the loads is in such a way that the block units at the top are assumed to carry the distributed load in the form of concentrated loads applied at their centroids just to come

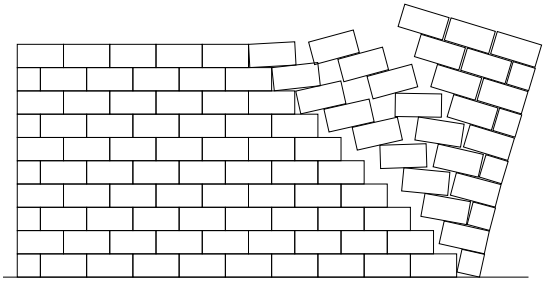
up with uniform distributions. It is visible that, the safety coefficient will increase, because it is increasing the gravitational load and in turn the seismic factor is dependant of the gravity loads. The associated thing here is, to how much is the increase?, and for which types of walls is the increase very high or low?, these things are finalized under the paragraphs following.

The effect of the rigidity on the blocks keeps the blocks to respond mainly by the crack formation on the units, the Block software is based on that idea. And as it can be seen on the results of mechanism in the Figure 3.9 and Figure 3.10, the masonry units are kept rigid but the rotation of these units' leads for the formation of cracks due the failure in the very limited tension capacity of the masonry assembly.

From the results under the case when there is no transferred external load, it is the failure of the far end units that persists for walls that are categorised as long walls. From Figure 3.9, the failure mechanism of the wall with Wr of 0.33 is the same to that for the case of the standard wall, Wr=0.5 (in the figure, the blue lines show the original position of the blocks and the black line) The seismic coefficient is found same for the walls with Wr value less than the standard. Even, the failure mechanism results can show these things easily, in that, the region of the walls where the failure occurs is equivalently same as the walls becomes very long or as Wr goes to low values. There is more than enough effective weight to resist the lateral load.



(a)



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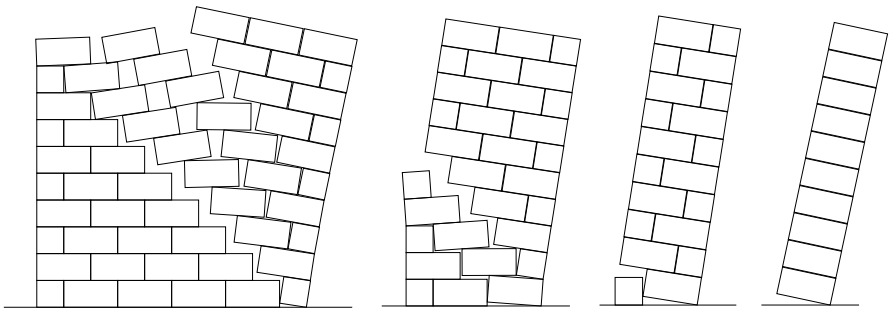


(c) (d) (e) (f)

Figure 3.9: Failure mechanisms for the variation in wall aspect ratio without external load; (a) for Wr=1/3 (seismic coefficient, α=0.493), b) for Wr=1/2, the standard or reference wall (α=0.493), c) for Wr=1.0 (α=0.493), d) for Wr=2.0 (α=0.408), e) for Wr=10/3 (α=0.282), f) for Wr=5 (α=0.198)

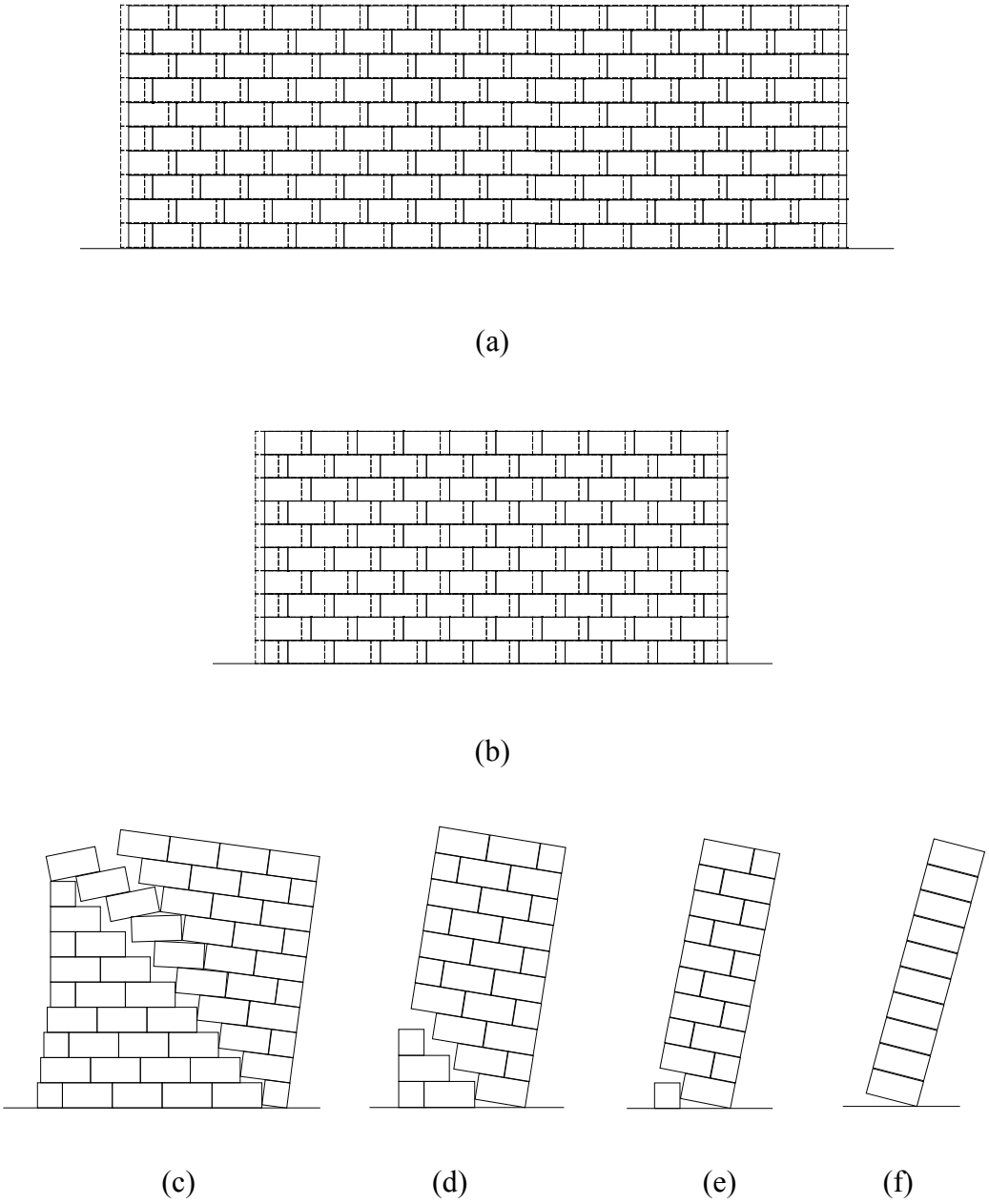


Figure 3.10: Failure mechanisms for the variation in wall aspect ratio with inclusion of external transferred load; (a) for Wr=1/3 (seismic coefficient, α=0.688), b) for Wr=1/2, the standard or reference wall (α=0.688), c) for Wr=1.0 (α=0.685), d) for Wr=2.0 (α=0.490), e) for Wr=10/3 (α=0.325), f) for Wr=5 (α=0.227)

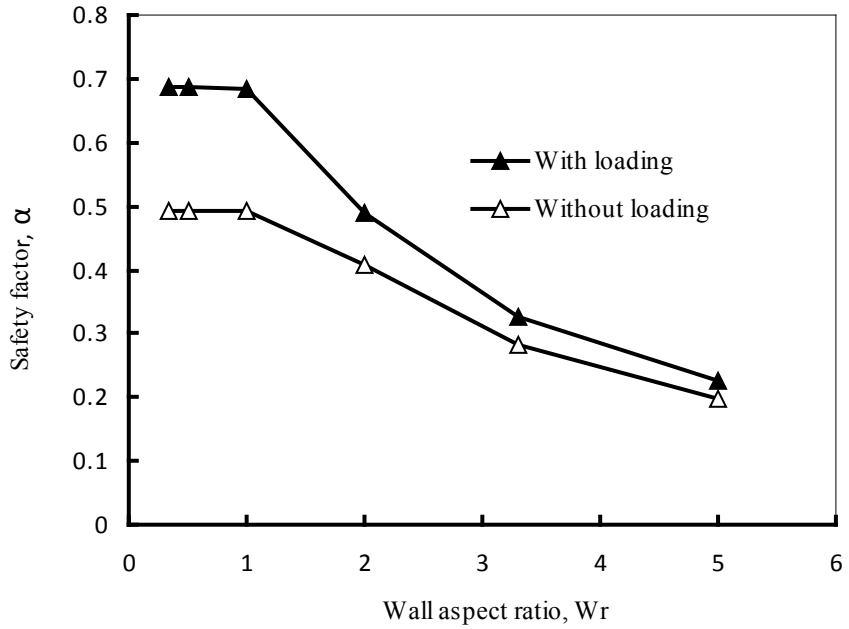


Figure 3.11: Wall aspect ratio effect on seismic safety factor

The application of vertical loads will increase the multiplier value, as it was pointed out from the definition of the seismic coefficient; it is the value dependent on the acting vertical load by the function that it will increase whenever the load is increasing.

From the graph of the Figure 3.11, the case for the loading condition presented that walls of aspect ratio lower than 1.0 are characterised by having same safety factor values, but this factor will change significantly when the walls are decreasing in their width. This is from the fact that walls will tend to experience overturning failure modes when they have lower dimensions in their width. In addition to that, the value of seismic coefficient will change in very rapid range for walls more or less in the range of 1 to 4, then for higher values its decrease is not significant. More or less same results pattern for the seismic coefficients of another shear wall study are presented by Orduña (2003). It is placed as Figure 3.12

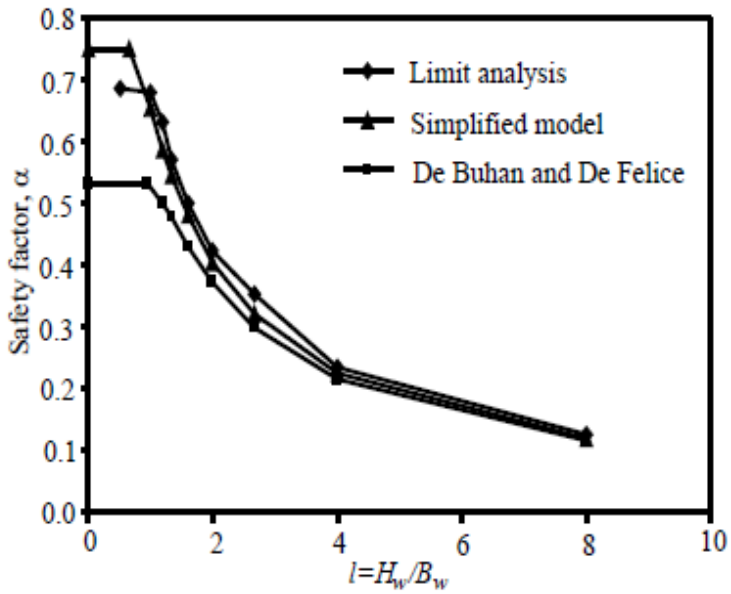


Figure 3.12: Wall aspect ratio effect on seismic coefficient, (Orduña A., 2003).

Figure 3.12 contains the safety factors obtained by the two simplified models (by Giuffr`e 1991) and those calculated by limit analysis for the reference walls with variable aspect ratios. For long walls, the simplified model overestimates the safety factor with respect to the limit analysis approach because the assumption for the crack slope implies that a larger part of the wall overturns compared with the limit analysis mechanism (Orduña, 2003). It is observed, for slender walls both simplified models agrees well with the limit analysis results.

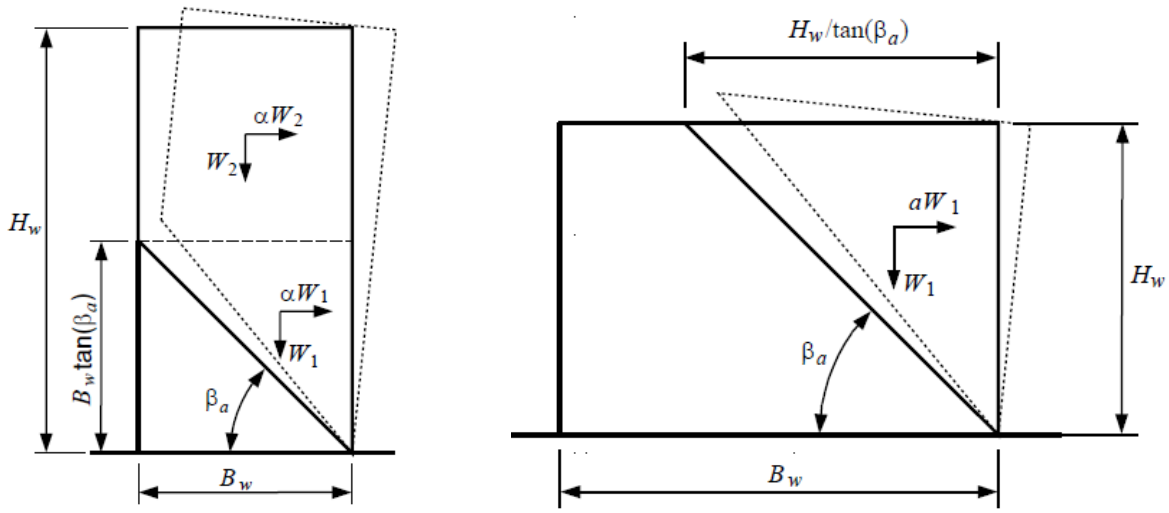


Figure 3.13: failure mechanisms in the simplified models: (a) slender walls (b) long walls, (Orduña A., 2003)

As a set of validation to the results, the simplified model analysis by Giuffr'e agrees very well to the cases when the walls are very narrow in width, but there is a difficulty in positioning the crack in the case of long walls failure. In long walls, for instance in Wr values of 1/3 and 1/2, the crack position is almost the same in the case of limit analysis, but in the simplified method of analysis, it is somewhat difficult where to put the crack. Due to that reason, the results by the simplification of the walls to macro models had resulted for high correlated values with the limit analysis mostly in the case of slender walls (walls of small width with respect to the height.)

Effect of Loading

Getting back to Figure 3.11, the curve for the case with loading shows a trend which is generally a decrease in safety value for a decrease in the width of the wall, kept that the height of the wall is constant. Also same trend follows for the walls for the unloaded case. But here, there is some variation that can be seen on the gaps between the two curves. This variation shows for the significance or power of the load to the effect on the seismic factors. This concept is shown in a self explaining Table 3.2.

Table 3.2: Effect of loading on the seismic factor under the variation in the wall aspect ratio

Wall aspect ratio, Wr	***without loading***	***with loading***	***The increase in value of a***	***% Increase in seismic coefficient***
1/3	0.493	0.688	0.195	*39.5*
1/2	0.493	0.688	0.195	*39.5*
1	0.493	0.685	0.192	*38.9*
2	0.408	0.490	0.082	*20.3*
3.3	0.282	0.325	0.043	*15.4*
5	0.198	0.227	0.028	*14.5*

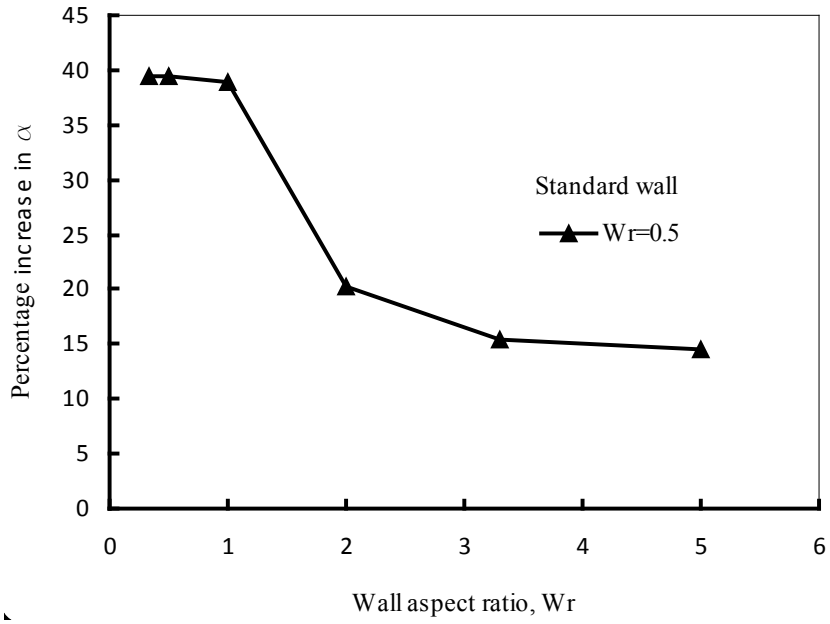


Figure 3.14 Effect of loading on the seismic coefficient under wall aspect variation

Hence, it is clear that walls of large width get increased the safety factors by almost 40%. In the cases for very common walls like in ratios between 1and 2.5 or 3, the increase in safety factor due to loading is lower in value and in average of 20%. The trend of the effect due loading is presented graphically in Figure 3.14.

3.5.2 Unit Aspect Ratio

The masonry units of the wall are simulated to vary in size along their length direction. As was shown from the standard masonry wall taken, see Figure 3.6, the masonry wall has Hw/hw=10 which equivalents to the units or rows of blocks. Then, unit aspect effect on seismic limit is basically determined as the result of the variation in the width of the units keeping the same number of rows of blocks. As same approach to the wall aspect ratio study, this condition was also studied for the considered wall aspects of 0.5, 1.0 and 2.0. Moreover, the effect of loading on the standard wall is also considered. The standard wall was simulated for the cases of loading and unloading due external applied vertical loads that may represent. Hence, there exist a number of possible unit aspect ratios. Here by, the followings types of arrangements on the units are considered.

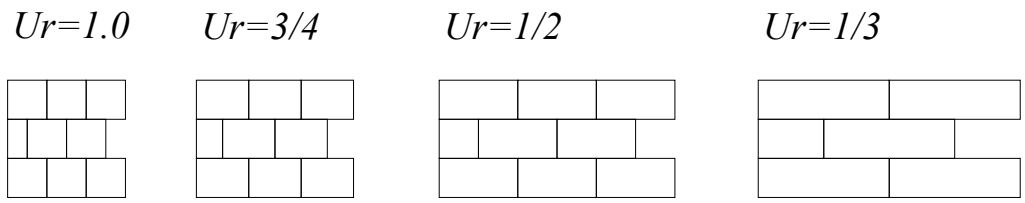


Figure 3.15: The types of unit aspects considered for the parametric analysis

Depending on the specific type of masonry unit arrangement, it is possible to obtain specific seismic coefficients. And the graphs of multiple sets of model results will lead to a formal way of understanding the behaviour of the effect of that parameter. Here, as similar to the case in the wall aspect ratios, the following things are assumed or in other words kept constant for parametrizing the unit aspect ratio to the seismic safety factor, α:

- Unit aspect ratios considered are: Ur= $\frac{hw}{bw}$= 1/3, 1/2, 3/4, and 1.0. It is unrealistic for the cases of the ratio 2.0, which means that the units are rubble masonry. Moreover, these dimensions are not observable in historical masonry structures. As described in the previous parameter, it is used same type of masonry units and the size of the standard masonry unit is 0.6m length with 0.3m height and the wall is of thickness 0.5m. In addition,

Results

Walls of ratios greater than or equal to the wall aspect ratios of 0.5 are generalized by having sliding type of failure for the same unit aspect ratio of 1/2. The masonry units' aspect has also a determinant role on the seismic coefficient. Walls of aspect ratio greater than the standard wall are experienced by having same failure mode the so called sliding failure. This type of failure is that having the horizontal movement of the masonry units along the direction of the earthquake. Mostly, the loaded cases will result for this type of failure. Whereas the unloading types of models results for a diagonal shear failure types, this types are known by having diagonal cracks. The models satisfying the stated requirements stated under the assumptions above have led to good results on the inspection of the seismic factors what trend they have when treated under the parametric variations.

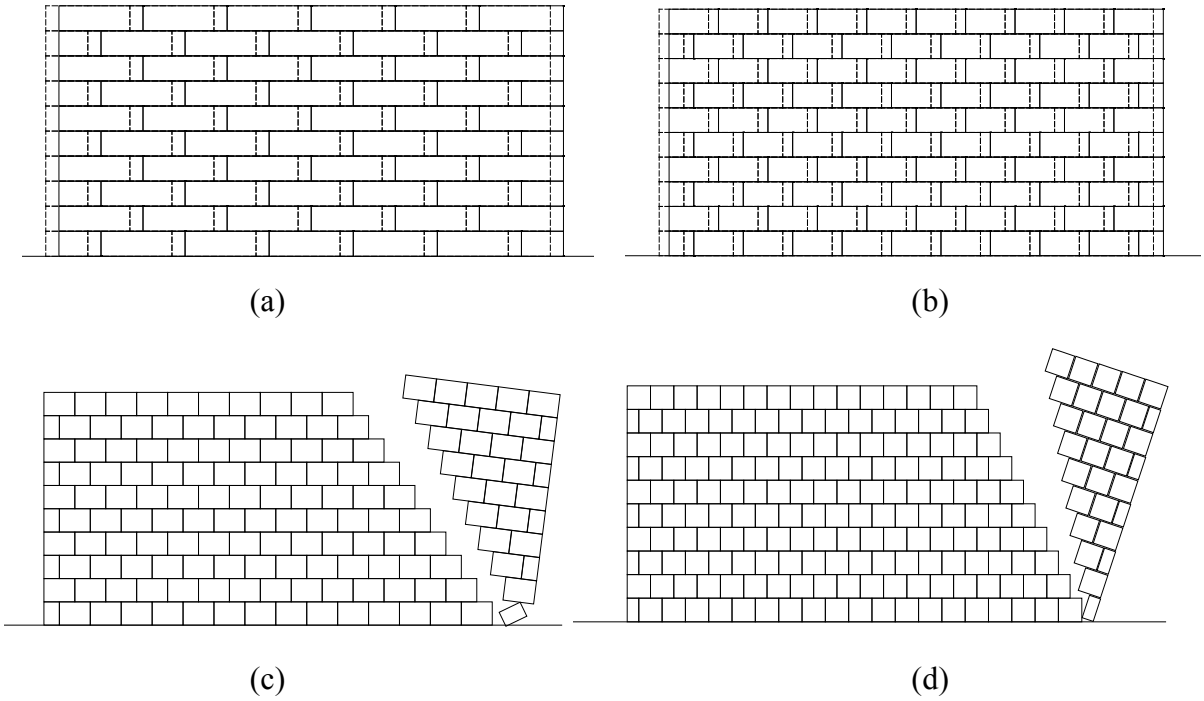


Figure 3.16: Failure mechanisms of the standard wall due to the variation in the masonry units aspect ratio (with external applied loads); (a) for hw/bw=1/3, α=0.688, (b) for hw/bw=1/2, α=0.688, (c) for hw/bw=3/4, α=0.463, (d) for hw/bw=1.0, α=0.369

There come an important idea that, in this study by the Block2D software, is consisting of the cases where there is no crack formation along the masonry units, in other words, the interpenetration of masonry units is not allowed. A stepped type of crack along the units is the major type of failure for the walls having Wr values of greater than 0.5. The associated type of failure is the diagonal shear failure and the units' bond length also determines the inclination of the line of crack. As given the mechanisms from Figure 3.16 (c) and (d), the angle of crack direction will be steeper as when the unit aspect ratio becomes high. It is to mean that when the height of the masonry units is higher as compared to the width, then the failure modes will give crack directions which are steeper. This will indirectly imply the fact that when the masonry units are very long, then the slope will decrease and even goes to the maximum case in the sliding failure.

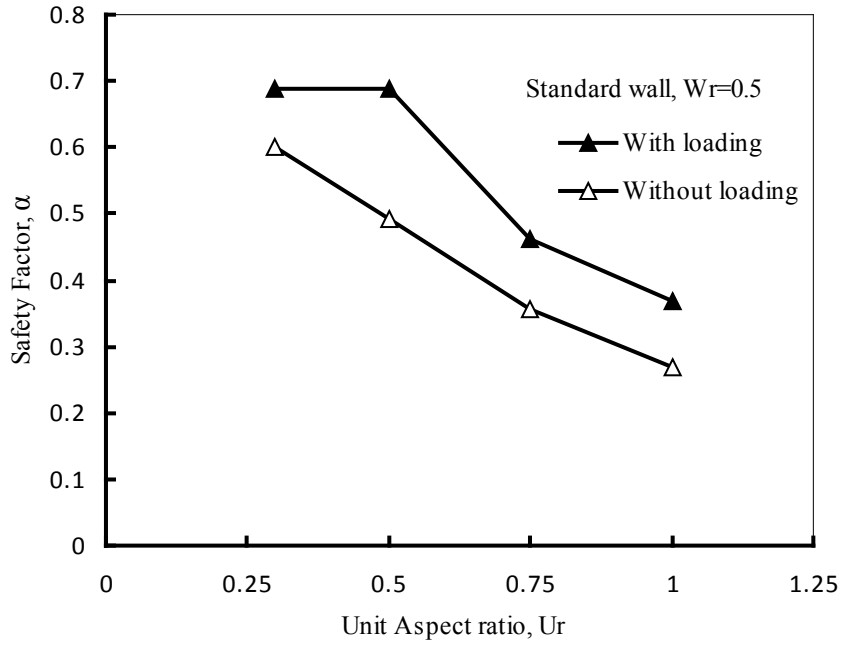


Figure 3.17: Unit aspect ratio effect on the seismic coefficient for the standard wall

In the case where there is no loading for the wall, it is observed that the failure occurs due sliding. It is shown in Figure 3.18. As it was discussed in the previous parameter study, the very important thing there is whenever the failure is approaching to the cases where sliding occurs, then it is the value of the friction coefficient that equals to the seismic factor. And to overcome the compressive force, it is very important that the load increases the friction in between the horizontal joints of the masonry assembly. This is obtained for the unit aspect ratio of 1 to 3, meaning long masonry units. It is also real that long walls are very common for sliding not instead for overturning keeping other things constant even though it depends on the friction coefficient. Moreover, it is the friction on the horizontal joints that will keep them tight; otherwise when the lateral loads are approaching to the collapse, the wall will show a slide failure. The higher the compressive force is the more the friction in between the units at the horizontal joints. Hence, the force that is acting on the wall as external is also determinant in changing the failure mode of the wall. This concept is seen under the graph presented Figure 3.17.

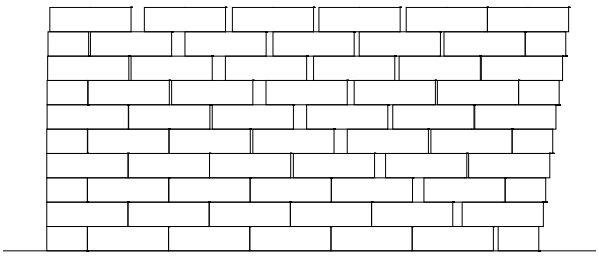


Figure 3.18: Sliding failure for the standard wall without any applied vertical overloads; α=0.6

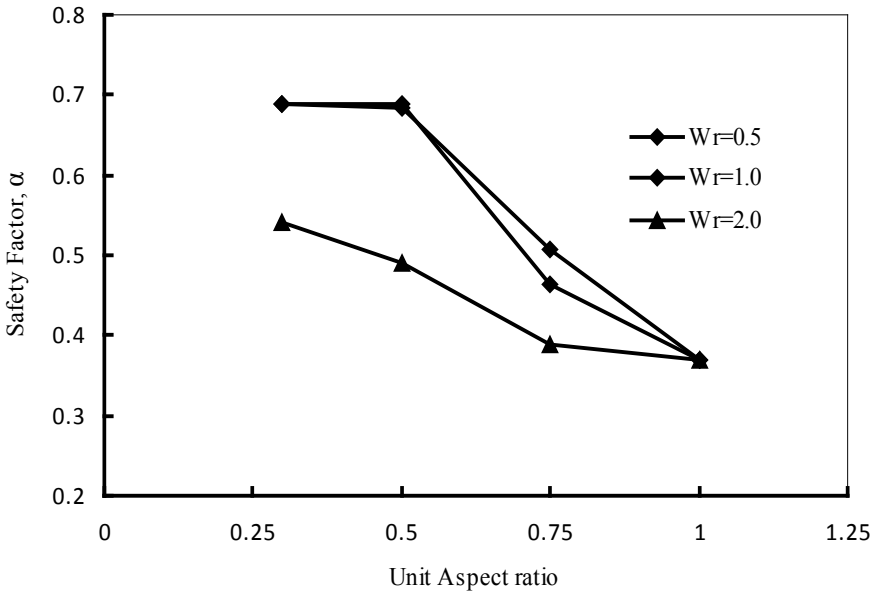


Figure 3.19: Unit aspect ratio effect on the seismic coefficient for the considered wall aspect ratios.

Hence, the walls with wall aspect ratios of 0.5, 1.0 and 2.0 are simulated for the variation in their masonry unit aspects variation. The results were put in the graph presented in Figure 3.19. The results obtained by the walls Wr=1/2 and 1.0 are very close values. From the fact long walls will have same failure modes and also very close values of seismic coefficient. There shows the top plateau graph is indicating that idea. Moreover, the approximate linear relationship of the wall Wr=2.0 shows the decrease in the seismic coefficient is at an approximate uniform rate as the decrease in the width of the masonry units.

These results are in good argument with Orduña results (Orduña A., 2003). Below graph is the result obtained by Orduña. In the graph, the results due to the wall Wr=0.5 is more close to the results from the wall Wr=1.0, and hence they represent one line trend. But, it is general that the safety factors decrease whenever the unit aspect ratio is decreasing.

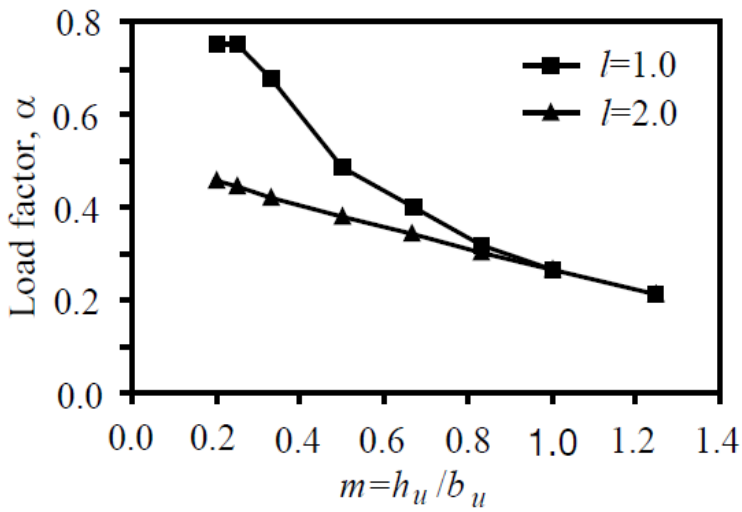


Figure 3.20: Unit aspect ratio effect; safety factor vs. unit aspect ratio graph, (Orduña A., 2003)

The general trend observed is that the simplified model here proposed agrees well with the limit analysis results, while the De Buhan and De Felice model underestimates the safety factor for small unit aspect ratios and overestimates the safety factor for large aspect ratios (Orduña A., 2003).

Effect of loading

In masonry structures the values of compressive forces against the lateral in-plane capacity is the most important idea that should be taken into consideration in seismic assessment. It may be due the cases when there exist majorly a variation in the loadings due some reasonable modifications or additional external applications. In such situations, there needs to assess the force factor against lateral forces, that is earthquakes. Below Table 3.3 show the compared values of seismic coefficients in the standard wall. Known the fact that, as compressive loads increase, the seismic factor also increases, in that case the percentage of increases for each type of wall is more or less the same. This is done for the standard wall as shown in Table 3.3.

Table 3.3: Effect of loading on the seismic coefficient under variation in the unit aspect ratio for the standard wall

Unit aspect ratio, Ur	***Seismic coefficient, α***		***The increase in the seismic coefficient***	***% increase in the seismic coefficient***
	With loading	***Without loading***		
1/3	0.688	0.600	0.088	*14.7*
1/2	0.688	0.493	0.195	*39.5*
3/4	0.463	0.356	0.107	*30.0*
1.0	0.369	0.267	0.102	*37.9*

3.5.3 Unit Size Ratio or Scale Effect

In masonry structures, it is common to see the units of masonry are not usually the same depending on the historical construction tradition of the country. In this sub-part, the masonry units of the wall are simulated to vary in size. It is named as the size ratio or scale effect, because the change is only in the magnification of the units. The other things are kept constant. As done before, walls of same Wr are selected and analysed by the Bolck2D software. Moreover, the effect of loading on the standard wall is also considered. The standard wall was simulated for the cases of loading and unloading due external applied vertical loads that may represent the real conditions. In parallel to this, the possible size ratios are considered. Below lists are the assumptions and also taken parameters for this analysis.

- Unit size ratios considered are: Sr= $\frac{Hw}{hw}$ = 3, 5, 10, and 20. It is unrealistic for the cases of the ratio less than 3.0, it means that there will be large stones and this is in fact unrealistic. Moreover, these dimensions are not common in historical masonry structures. The size of the stones is referred to masonry units which functions of the standard size. Hence, the size of the magnification is taken by multiplying this standard value by factors of 0.5, 1.0, 2.0 and 4.0.

Results

Among the various cases, masonry structures have differences on their size of units. As a consideration, the difference may not be of beyond half the unit size commonly considered. Based on this concept of argument, it is possible to detect their effective changes on seismic coefficient of various wall types, in this case to the walls selected above in the considerations. The failure modes resulting from each wall for each cases of size ratio are very important in characterizing the macro failure mechanism that can be taken as useful input for kinematic analysis.

For the failure modes, loadings over the walls are the main changing parameters in addition to the specific parameter considered as changing. This is verified on the standard wall. The failure mechanism results are shown in Figure 3.21 and Figure 3.23. Generally, the failure modes under the transferred loads cases are experienced by having sliding failure. This is due the friction in between the horizontal joints is not capable of resisting the lateral forces. Seismic coefficient also increases due to the additional compressive force. The trend by the increase in the seismic coefficient is the important output for the generalization on the relation.

The standard wall, the wall with Wr=0.5, under the Figure 3.21 shows the case without loading and it can be said that as the masonry unite sizes increase, the failure will occur in a similar slope of crack as step wise manner passing by the vertical and horizontal joints as in Figure 3.21 (c). Then as the units become small, the failure will have cracks in a distributed manner as multiple of cracks parallel to each other as can be seen on Figure 3.21 (b). Even when the units come to the realistic smallest units, the failure will have cracks parallel to each other and many in number, Figure 3.21 (a). The number of parallel cracks increases as the size of units decreases. There also, it is possible to conclude that for the standard wall without loading as the unit size decreases, the seismic coefficient will not be changed. On the reverse case, when it increases, the change in the seismic coefficient is generally decreasing and the decreasing rate is not high. Comparisons can be easily seen on the graph in Figure 3.22. In the Figure 3.22 (a), the results indicated by the lines under the top curves is showing similar pattern to the present result for the unloaded case in Figure 3.22 (b).

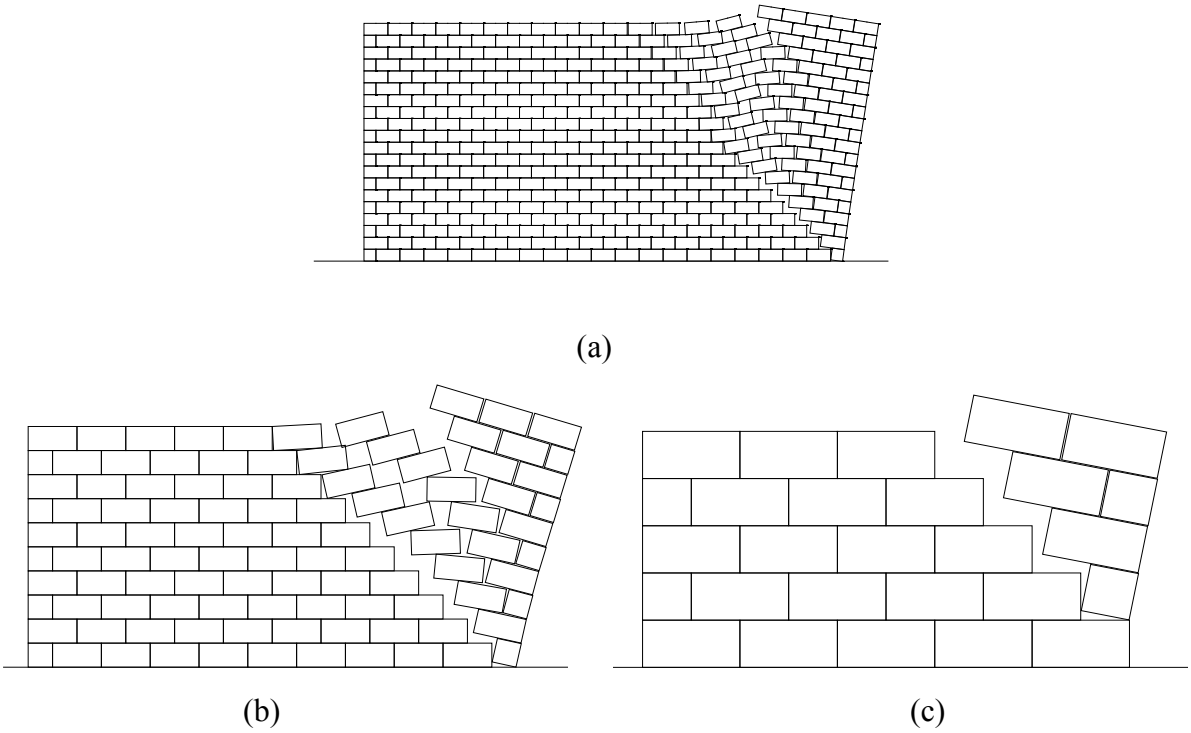


Figure 3.21: Failure mechanisms of the standard wall due to the variation in the masonry units size (without external applied loads); (a) for Hw/hw=20, α=0.429, (b) for Hw/hw=10, α=0.493, (c) for Hw/hw=5, α=0.596

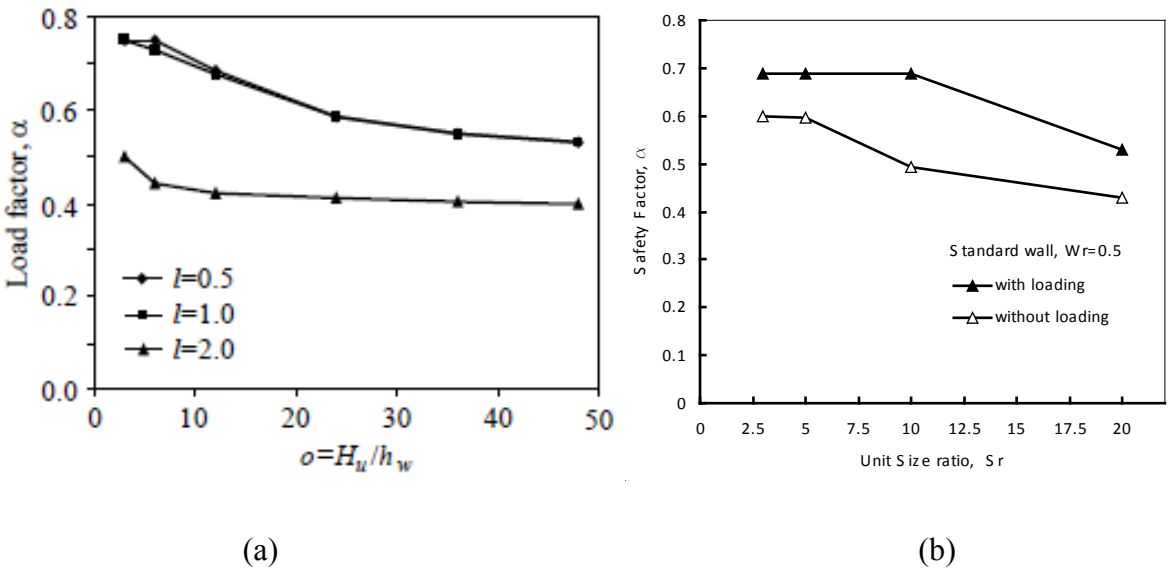


Figure 3.22: Size ratio effect on the seismic coefficient, (a) (Orduña A., 2003), without loading (b) present results for the standard wall

For the same wall in the loaded cases, it is found that the seismic coefficient is not changing for the size ratios below 10 (i.e. Hw/hw=10). It means that the loading has significant effect for the seismic coefficient increase. And their failure modes are characterized by shear sliding failure, Figure 3.23 (b) and Figure 3.23 (c). As smaller units are used, the failure will be changing to diagonal shear failure, Figure 3.23 (a).

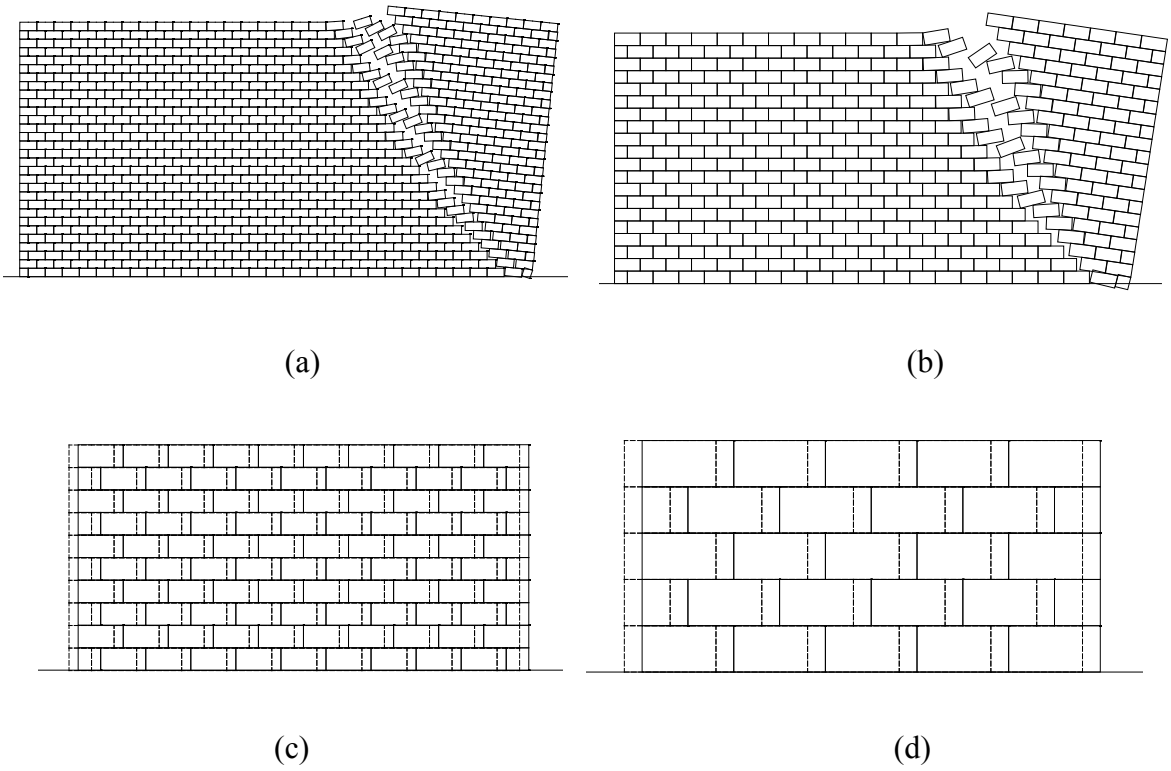


Figure 3.23: Failure mechanisms of the standard wall due to the variation in the masonry units size (with external applied loads); (a) for Hw/hw=30, α=0.517, (b) for Hw/hw=20, α=0.530, (c) for Hw/hw=10, α=0.688, (d) for Hw/hw=5, α=0.688

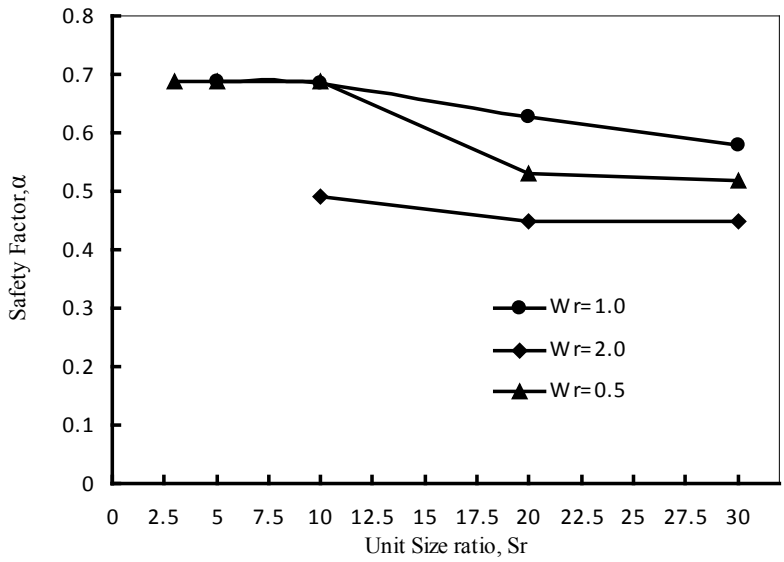
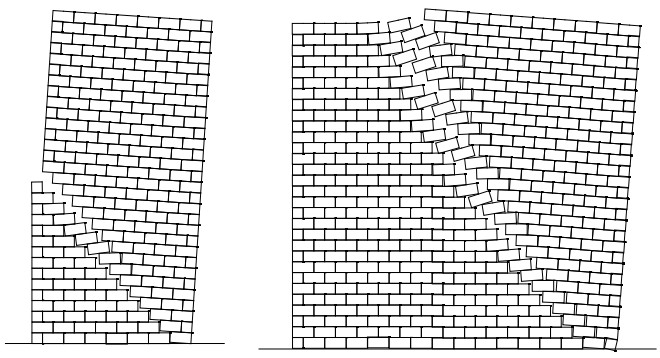


Figure 3.24: Effect of Size ratio on seismic coefficient of the considered walls.

It is not real to make masonry units of size larger than the allowable width in the case of the wall aspect, Wr=2.0. This is implied in Figure 3.24, the curve starts at the size ratio of Hw/hw=10.



(a) (b)

Figure 3.25: Failure mechanisms for two different wall aspect ratios for the same size ratio, (a) Wr=2.0, α=0.447, (b) Wr=1.0, α=0.578

Generally, the failure modes of walls for same size ratio but for different walls is of two types: for long walls, the failure mode is an inclined crack formed at the far end of the earthquake direction and it will not be changed for an increase in the wall width, and for

slender walls, cracks will be formed at the base. Above Figure 3.25 describes these conditions.

3.5.4 Overload Ratio

Masonry walls are the main load bearing components of the structure. The more the compression loads exist on the structure is to mean, the more the exposure to seismic risk. Walls of same under the previous study are considered here. It was also made to overload values of high value for the aim of developing a trend to see how the effect goes on.

In addition to the standard parameters listed for the standard wall, the following type of loading is taken for this part of analysis:

- The distributed load is the same as to the previous ones. It is 5.5kN/m. For the standard wall, the load will come to 3.3kN per block unit applied at the centroid. The load increment is done by a multiple of the primary load, which is 3.3kN. The values include 3.3kN, 6.6kN, 9.9kN and 13.2kN. But the idea here is that in reality the loads will not vary that much significant, but in order to plot and look the trend of the load increment over the seismic factor it is done for enough increment to see the changes.

Results

This part is not as that much important as to the other parameters but in order to confirm how loads affect the seismic capacity of an existing structure. The load increment will increase the seismic factor. This is easily proved that the seismic factor is the factor multiplying the vertical forces acting over the structure to the expected forces acting horizontally.

Load increments are applied to each of the walls considered, Wr=0.5, 1.0 and 2.0. For each step of the increment, there exists a specific seismic factor and these values developed a graph which is shown in Figure 3.26. It is generally clear that the increase in the vertical loads will increase the safety factor and the associated failure modes are sliding types until the crashing failure of the stones under high load cases. This relation can be observed in the rising curve. Load resistance for masonry under compression is to a certain limit and equivalently it is seen the down curve for this concept. But it is so observable that the seismic safety goes to higher values for walls of high width to height ratio. Related to the same idea, generally as walls

become lower in width, the lower is the safety factor for the same application of loads. Real situations will goes to a certain limit of loads and for such cases the difference in the values of α is more or less the same. Moreover, it is not necessarily a need to have higher extreme loads, otherwise crushing failure of the units will occur.

Loads in the standard reference wall are applied in such a way that they increase in a magnitude of equal value to each analysis. The increase in magnitude is 3.3kN for a single masonry block unit at top level. Analysis to the standard wall gave the result as shown in Figure 3.26. There it is possible to see that the curve goes up and after a certain limit value it starts to decrease. This is from the fact that the wall can resist to vertical loads up to a certain limited maximum capacity. The failure on the compression capacity will lead to such situations. But generally, since the load will not exist or it is unrealistic to reach such value, then it can be stated that as the load increases, the seismic factor will increase.

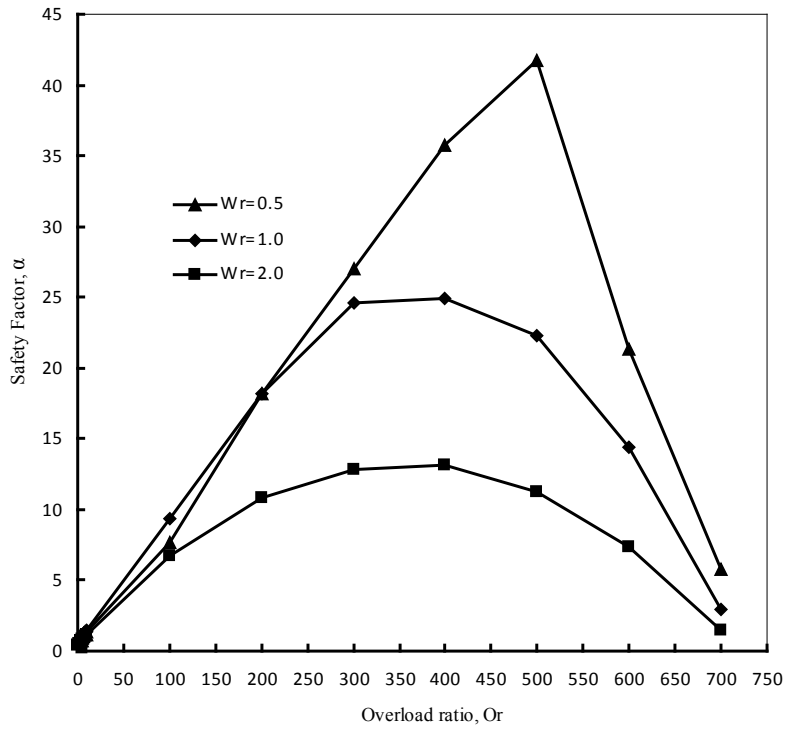


Figure 3.26: Effect of Overload on seismic coefficient for the three walls.

3.5.5 Bond Ratio

The bond between the mortar and the masonry units is one of the most important properties of masonry construction, particularly when it is non load bearing such as in low-rise buildings. Most masonry walls are at times subject to forces such as raking from wind, minor settlement of foundations and, in some instances, more-severe forces resulting from earthquakes. It is the development of tensile strength (i.e. the ability to resist elongation) that is crucial, and that makes bond strength a priority in masonry construction.

Bond is the other most important affecting parameter for seismic capacity. It is used to represent the relationship of the seismic factor with the bond in between the masonry units. There exist different walls with some patterns of arrangement by the units to each other. This might range from very complex cases that have irregular pattern to the cases having relatively good pattern, some of them are shown in Figure 3.7. It may be difficult to put the real situation of the interlock in the units. But in Block software, this is very easy and it can be applied to any of the arrangements that really exist on masonry structures. In addition to keeping the other parameters similar to the standard wall, below are the considered bond ratios for this parametric analysis:

- The bond ratio, Br: the overall set of bond types consists of 0.5, 0.625, 0.75, 0.875, and 1.0. They equivalently indicate the ratio resulting from $\frac{b_w - l'}{b_w}$; it can be stated in percentage. Generally, each type of bond ratio is applied to each wall and the results are also compared.

Results

There is a method applied here to show the possible better way of understanding how the change in bond affects lateral seismic coefficient. It is true that the earthquake direction determines the value. For symmetric cases, for instance in the case of 50% bond, it doesn't matter which direction is considered. But for the rest types of bond, it matters that the wall should be analysed for both cases of directions for the same type of bond. In other words, a considered wall with one type of bond is analysed for the two directions except for the 50%

bond type. This is fact that it should be known or verified because the earthquake is going to happen in the two directions.

The consideration of the direction of bond arrangement is shown below. The graph will be developed according to the direction of bond running arrangement.

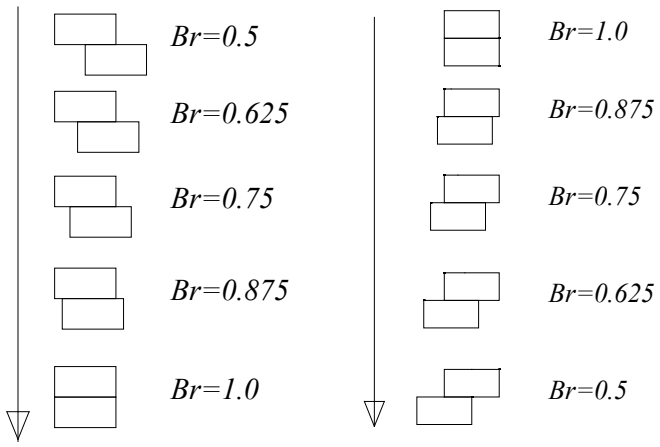


Figure 3.27: Arrangement of bond

The failure mechanism of the walls is show from the case with 50% bond to the case coming back to 50%. It is clear that up to the bond ratio value of 1.0, as seen from the left arrow of Figure 3.27, the earthquake direction is taken the same direction and then for the coming arrangement of bonds in the right arrow of Figure 3.27 the direction will be in the reverse. Moreover each of the failure mechanisms is plotted with the same scale factor.

In the Figure 3.28, the standard wall bond ratio versus seismic safety factor is shown. There also it is shown for the unloaded case. One important point can be concluded from here that when the direction of the earthquake is reversed and analysed, for each bond ratio except 0.5 it is clear that the seismic values are not equal.

Cracks become nearly vertical as when the bond runs from 50% to 100% bond. This is also clear from the fact that the angle of the vertical joints is also directing to this occurrence. The failure mechanisms are presented in (d) (e)

Figure 3.29. The standard wall has shown the sliding failure mode. But the rests have the same failure pattern with observable differences on the angle of crack. The mechanisms of the

cases for the unloaded walls are more or less the same except with the variation in the value of seismic coefficient.

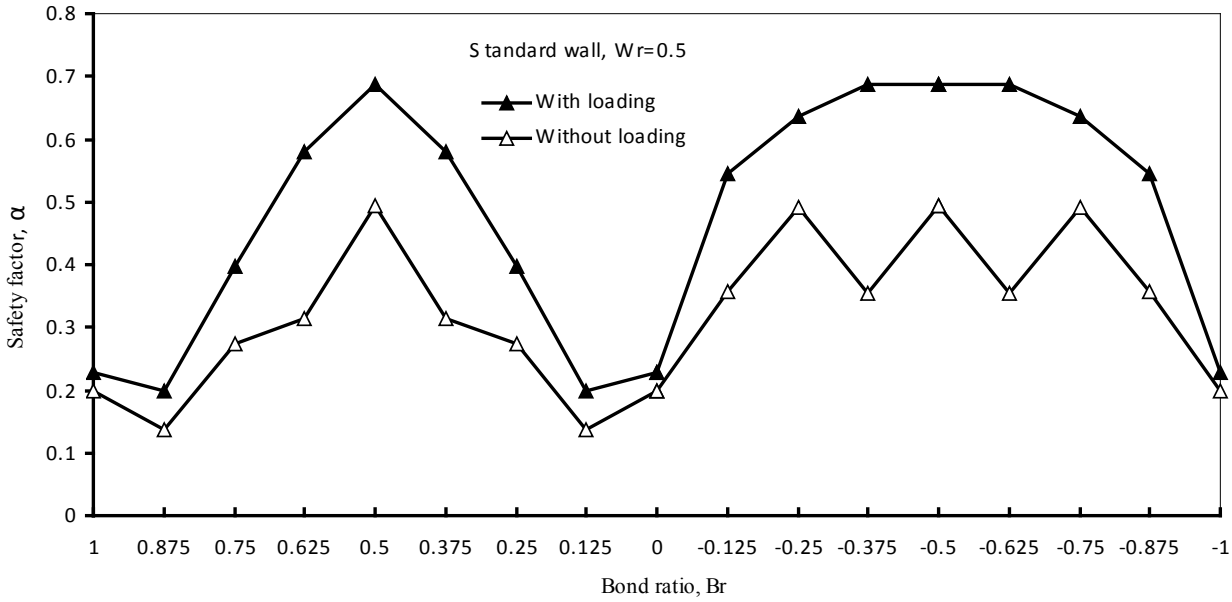


Figure 3.28: Effect of bond ratio on seismic coefficient for the reference standard wall.

Similar concept of bond effect was observed for the walls under the consideration for the whole parametric analysis. The important common behaviour of the trend in the safety factor is the decrease when the bond ratio increases. For the same bond ratios taken under the standard wall, failure mechanisms are obtained to have the same pattern or behaviour in that the crack formation goes vertical as when the bond runs to the stack form of bond, Br=1.0.

Generally, as plotted under the graph of Figure 3.30, they show a decrease in the safety factor and also the relation in between them is not specific. For instance, the wall of Wr=1.0, passes to behaviour that it will immediately show the drop in the safety factor as when bond is increasing. But for Wr=2.0, it is very slow decrease and even the values of safety factor are very low values. Hence, the decrease in safety factor is the generalizing character in those walls. The way of presentation in these other walls is not necessary to show the full case of bond, because it is valid that the safety factor for the other half part of bond is higher than the same bond type but with reverse direction of earthquake application. Moreover, the need for the higher values of safety factor is not more important than the lower value for the same bond type. Hence, the lower cases are shown in the graph of Figure 3.29 (d) and (e).

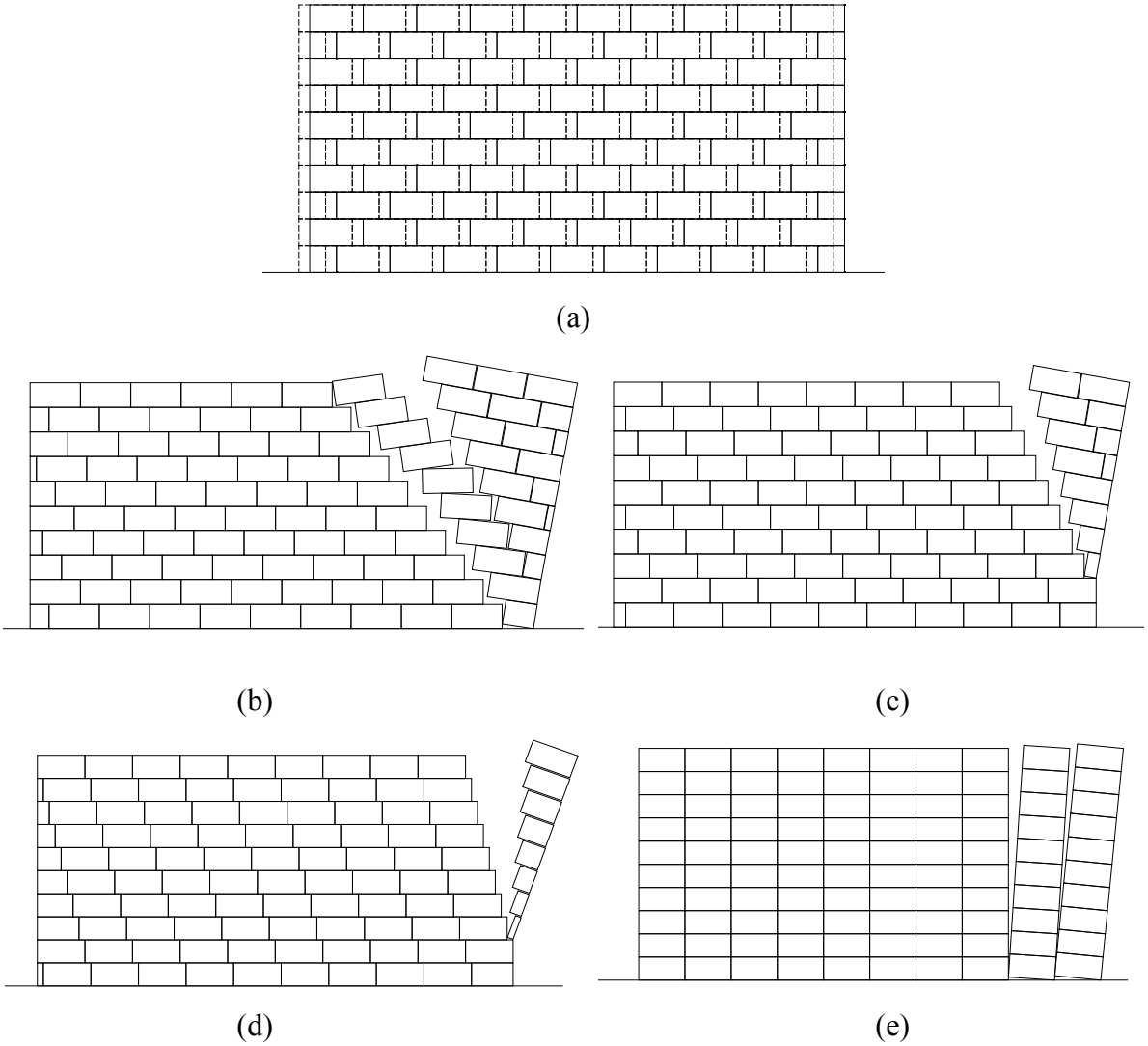


Figure 3.29: Failure mechanisms of the standard wall due to the variation in the bond between masonry units (with external applied loads); $Br = \frac{b_w - l'}{b_w}$, (a) for Br=0.5, α=0.688, (b) for Br=0.625, α=0.579, (c) for Br=0.75, α=0.397, (d) for Br=0.875, α=0.199, (e) for Br=1.0 (Stack bond), α=0.227

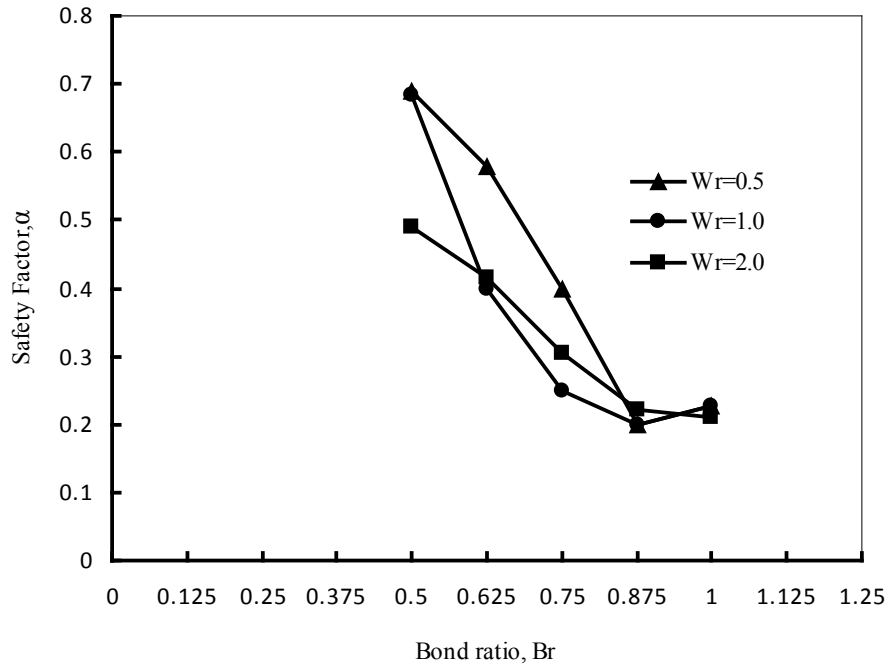


Figure 3.30: Effect of overload on seismic coefficient of the considered wall aspect ratios

3.5.6 Compressive Stress Ratio

The compressive strength of the various types of masonry has been established on the basis of large numbers of tests on walls and prisms. It depends on the strength of the unit and on the mortar strength both of which are determined by standardized tests. In Europe, masonry strength is defined as characteristic strength, in statistical terms a value which would be achieved by 95% specimens tested. This parameter depends on the normalized compressive strength of the unit, the mean compressive strength of the mortar and other factors that depend on the type of masonry.

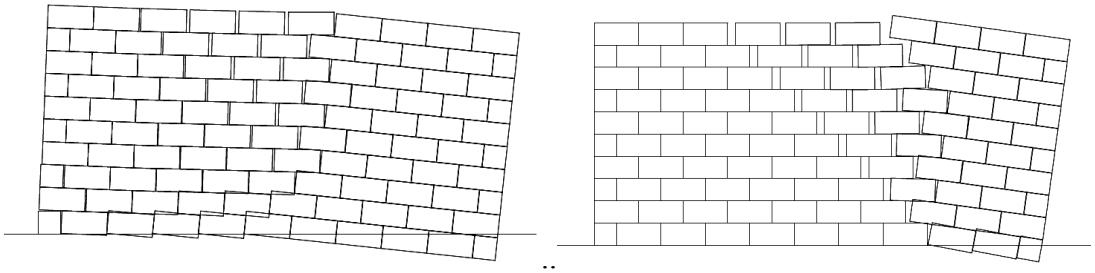
Masonry walls which can carry any significant vertical load will have considerably higher resistance to lateral loading than those whose resistance relies on the flexural strength resistance of masonry (Tomaževič, 1999). For this reason, the lateral strength of load bearing walls seldom has to be investigated in relation to wind forces but it may be necessary to verify that a particular wall is capable of remaining in place following a gas explosion or other accidental occurrences like earthquakes in which here it is being studied.

Following the same procedure of analysis, the following geometry, load and material assumptions are taken for the limit analysis related to the compressive stress:

- Effective stress is taken as a multiple of the reference value, 8MPa. This increment was taken analysis after analysis, meaning the values representative to the real situations were firstly taken and the successive other value are multiples of these values to overcome the trend behind with the seismic limit. Values are 0.0125, 0.125, 0.5, 1.0, 1.25, 2.5 and infinite. The last case is defining to have a very large value of compressive stress

Results

In this study, primarily, the standard wall is analysed by the set of different cases to same applied methodology as above. The loaded and unloaded wall cases are also analysed.



(a) (b)

Figure 3.31: Typical failure mechanism for lower values of compressive stress, for 1MPa (a) with loading, (b) without loading

It is shown the results of the standard wall analysis under the graph in Figure 3.32. The compressive stress increase is not significantly changing the seismic coefficient. For lower values, as shown on the left margins of the curve under Figure 3.32, seismic response of the wall is majorly an indication.

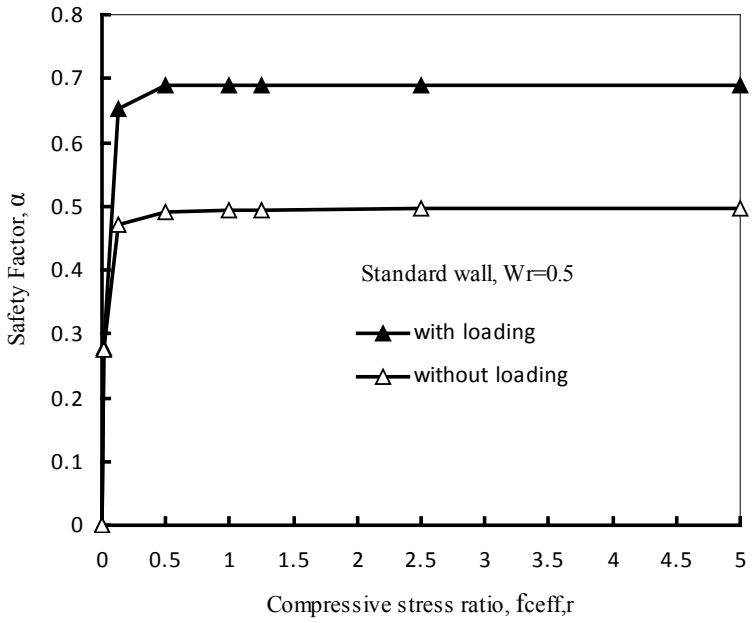


Figure 3.32: Effect of compressive stress on seismic coefficient for the standard wall.

Values run in a geometric change, the values in the range of the importance that are very representative compressive stresses in historic structures. This includes values from 6.5MPa to 15MPa. In the same graph, it means the ratio values of 0.8 to 2.0. But, as the values of compressive stress are higher, the wall is not more affected by the change and moreover this behaviour is also the same for each wall. It means that it has already achieved the capacity or the resistance for that required amount of external load.

From previous study results, walls of same aspect ratio were analysed by Orduña. According to his results (Orduña A., 2003), it is obtained a good approximate result. But there exists that the walls are not loaded to external vertical forces, they just represent a standing wall. Hence, comparisons are made with models which have no loads, as done in graph Figure 3.34.Also the interpretation of the ratio of compressive stress is in terms of the amount of effective stress relative to that produced by the wall self weight.

These mechanisms illustrate that for small effective stresses, the wall needs a wide contact surface in order to transfer the vertical load to the support (Orduña & Lourenço, Cap Model for Limit Analysis and Strengthening of Masonry Structures, 2003). While for increasing effective stresses, the contact area reduces proportionately. The reduction in area indirectly implies that the wall is having good lateral capacity.

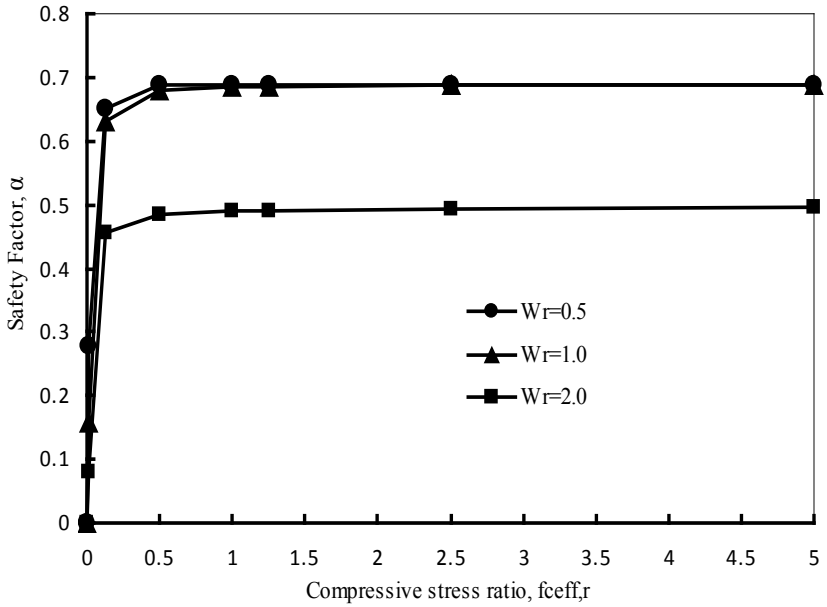


Figure 3.33: The behaviour in the seismic coefficient due to the variation in the effective compressive stress for the walls of Wr=0.5, 1.0 and 2.0.

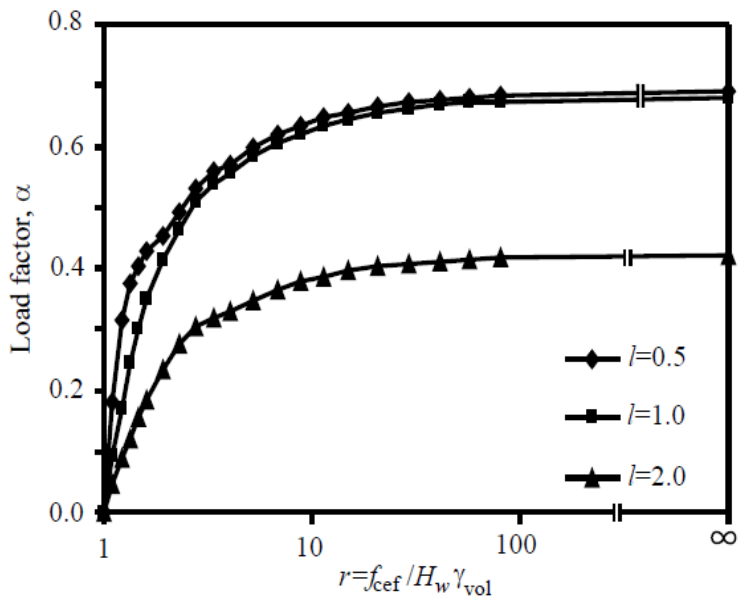


Figure 3.34: The behaviour in the seismic coefficient due to the variation in the effective compressive stress for the walls of Wr=0.5, 1.0 and 2.0.

3.6 Conclusions

Parametric analysis of a set of walls is done for arriving to some conclusions. In this chapter, it is done the basic part of characterizing the seismic capacity or in other words seismic safety factor of different walls in different aspects of basis. Effects of wall aspect, unit aspect, unit size, bond types, effective compressive stress and the overload are also observed by applying to different walls. The wall aspect ratio effect for longer walls is found to be similar and the failure mode is also found to be the same because of the fact that the wall end will only be exposed to failure. Hence, only that effective part of failure will be kept the same. The change in the unit aspect ratio has also considerable effect on seismic limit. The longer the unit aspect, the more the prevailing is sliding failure. And when the units are coming to rubble masonry type, the failure will be due crack formation which is characterised by an angle of crack steeper and steeper. Size effect is very important thing to have idea in the failure modes. The more fine the units, the decrease in the seismic coefficient. For an attainment of masonry units of large size, it is found that the wall is not going to have any change in the seismic safety factor. But for the cases in having small unit sizes, there will occur a decrease in value of safety factor. Bond effects are also investigated. It is considered to analyse the bond running from 50% (regular bond) to 100% bond (stack bond). The curve of safety factor is plotted and implied how safety varies for same bond but with the variation in seismic load direction application. Even the most important point, the overload over the wall is simulated for an increase and checked for how the safety responds. It is found that the loads increase safety factor. Load application or load increase is up to the capacity by the masonry units. As the last part of the parametric analysis, the compressive stress that compensates the vertical load to the contact surface is also found as determinant. As when it varies with increase, the safety factor reaches to its stability in value. It will be constant for very large values of compressive stress. The compressive stress is taken as a multiple of the reference value. Moreover, in each case of parametric analysis, the walls were compared for the cases with and without external applied vertical loads. The cases for the standard wall in optimization of the real conditions are done. These helped for developing the relation in between the behaviour of the common failure mechanism with the safety factor associated to that specific parameter.

CHAPTER 4

NUMERICAL ANALYSIS OF MASONRY WALLS

4.1 Introduction

Aassessment of the seismic behaviour of ancient masonry structures lacks scientific background while for nowadays structures there exists where the seismic vulnerability can be assumed and or supposed by means of existing codes and analysis methodologies. Reliable numerical simulation of masonry structures will be the main determining and basic input for the characterisation of the seismic response. The, numerical modelling of the seismic behaviour of masonry structures is a very complex problem due to the presence in the unspecific constitutive material characteristics. Moreover, physically and geometrically, the structural material is highly characterised by non-linear behaviour when exposed to a high ground motion. In any of the analysis methods for masonry structures, meet or account on the mechanical behaviour for its discontinuities is a must to be included. The blocky nature in failure mechanisms and being composed of two completely unlikely different materials; the mortar joint and the masonry unit, makes it to exhibit a heterogeneous nature.

Most of the existing historical monumental structures, at least in Europe, are made of masonry, using either stone or brick blocks. These unreinforced blocky masonry structures cannot be considered a continuum, but rather an assemblage of compact stone or brick elements linked by means of mortar or dry joints (Azevedo & Sincraian, 2000).

Research on the seismic behaviour of masonry structures is nowadays almost entirely dedicated to existing buildings and to the issues related to assessment and reduction of their seismic vulnerability (Magenes, Masonry Building Design in Sesimic areas: Recent Experiences and Prospects from a European Standpoint, 2006). In most cases, masonry shear walls are the main reaction system of old buildings. This chapter is focused on the study under real model structures experiment held in Universitá degli di Pavia (Michele, Giovanni, Guido, & Alberto, 1992). The prototype in the experiment looks like as shown in Figure 4.1 (a) and it plan view in (a). The experiment was held in the purpose of prevention of damages

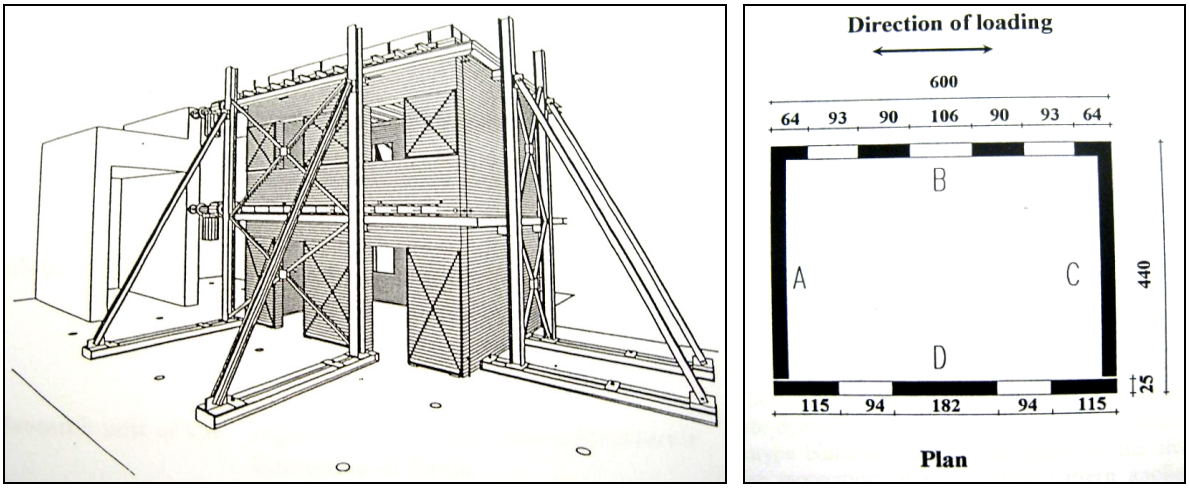
to buildings that exist still. Through the experimental evaluation of the seismic behaviour of structures, it is possible to verify the task of the investigation on the prototype used in representing the buildings in present. It was designed to provide support for the complete building assessment process including the following key phases (Michele, Giovanni, Guido, & Alberto, 1992):

- Survey and observation;
- Interpretation of non destructive testing as compared to the results obtained from detailed destructive laboratory testing;
- Analytical modelling of parts or components and of whole building as compared to experimental destructive tests;
- Design of strengthening techniques;
- Analytical modelling of strengthened structures as compared to experiments.

In this complex experiment, the prototype was done as much as possible to represent the typical old construction in Italy. In addition to the real experimental full scale test, several analytical predictions of the response were prepared before the tests by different research groups.

Along with the previous Chapter 3, the preliminary parts of parametric analysis are made on various sound ways of parameters typical to a historic structure applying on a standard wall functioned in load bearing. Variations on the diverse walls derived from the standard wall are made for the development in the relations in those parameters like wall aspect ratio, unit aspect ratio, unit size or scale effect, bond effect, additional loads effect and even the compressive stress of the masonry. Trends among these parameters to the seismic safety factor were developed. Hence, the effects of some important parameters over the prototypes taken from this experimental and numerical investigation are made. This experimental and numerical investigation on a brick masonry building prototype was done in the aim of evaluating the seismic behaviour of structures. For this specific study, it is taken to have the two facades from the prototype. There are two models of different configuration on the openings. For these two models, generally standard walls or facades are taken as the same model of approximately similar material and geometry characteristics. From the standard

walls, it is possible to modify the model in such a way that the modification is important in the evaluation and assessing the variation in the seismic safety factor. The important things here are the variation in the wall aspect effect, the overload effect and lastly the strengthening ties effect which is related to the failure mechanism developed from the results in the standard case. The wall aspect effect was considered by adding another typical pier of the wall. Effect of storey also is investigated on wall 2.



(a) (b)

Figure 4.1: (a) General view of the Prototype, (b) Plan view of the prototype (Michele, Giovanni, Guido, & Alberto, 1992)

In each part of the analysis, it is discussed and presented the failure mechanisms of the specific case and the thrust lines (compressive principal stresses).

4.2 Model Description

In structural analysis, modelling of masonry structures is obvious that it is very complex and it is difficult to simulate the non linear behaviour of materials, in this case masonry. In developing models of masonry in limit analysis, typically it needs very simple mechanical characteristics: the effective compressive strength of masonry, the geometry and the unit weight of the composed masonry. This makes limit analysis very simple and very important analysis method as compared to the other tools of analysis.

The building tested was made of two-wythe unreinforced solid brick walls held to each other in english bond, with a total thickness of 250mm. The plan and elevation views provided with dimensions of the tested building are as shown in Figure 4.1. In the same figure in part (b), referred walls D and B are those walls here called as masonry wall 1 and masonry wall 2, respectively. Material characteristics in the model here in this specific study and the test prototype used are described below.

Material Properties of Masonry wall Models

The full scale experiment done in the University of Pavia, as mentioned before, is made to precisely represent the typical urban classical residential buildings of unreinforced masonry. Mean compressive strength of cubes taken from the samples of the solid fired clay bricks gave a value of 16MPa. The corresponding joints, a mix of lime and sand, measured to a compressive test capacity of 2 to 3MPa (Michele, Giovanni, Guido, & Alberto, 1992). As a result, the compressive strength of masonry composition measured to a value of 6.2MPa. Moreover, to the later cases in this chapter, it is applied strengthening ties for the support in the in-plane strength of the walls. The model properties are ade to comply with the block modelling. Hence, some features of the experimental characteristics are modified to more realistic cases. For instance, the transferred load in the case of the experiment consists of additional vertical dead load of 530kN, which means a distributed load of 10kN/m^2 (equivalent to transferred line load of 22kN/m) but this is a very high load as compared to the structure and its functionality, due to this considerations, it is taken 6.67kN/m transferred line load. Complete list of properties used in the model for the rigid block limit analysis is tabulated below in Table 4.1:

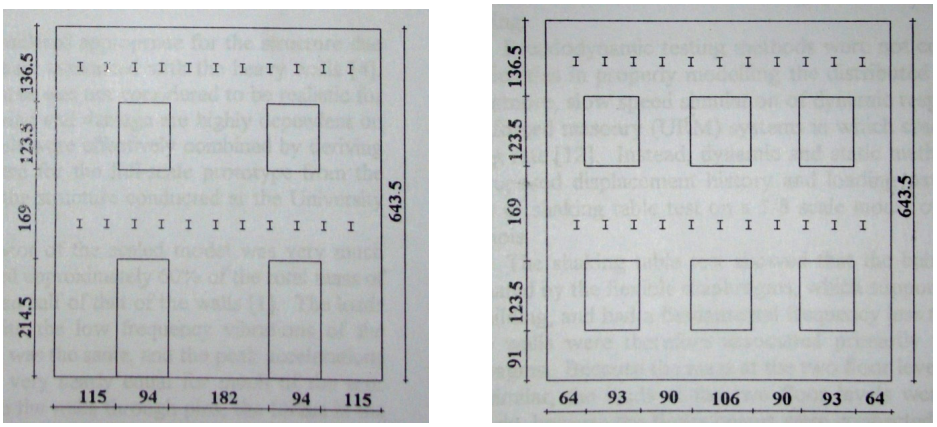
Geometry of Masonry wall Models

During modelling in the block software, there are some values of approximations. Since, we have constant dimensions of masonry units, the model was made as much as possible very precise to the experimental prototype dimensions. The size of masonry blocks was selected according to two major criteria's: it should represent typical or common type of units and secondly the failure mechanisms will give very important and precise type of failure modes, it shows clearly the crack formations and compression thrust lines. Moreover, as matter of better model approximation, arches are used in the openings. The original dimensions of the

prototype in the experiment are shown in Figure 4.2, and the block model dimensions are shown in Figure 4.3. Note that the dimension units by the experimental prototype are in centimetres.

Table 4.1: Masonry wall model characteristics (Michele, Giovanni, Guido, & Alberto, 1992)

Characteristic	***Value***
Masonry	
- Effective compressive stress	6.2MPa.
- Unit weight of masonry	17kN/m^3.
- Size of masonry units (blocks)	Rectangular units of 30cm width by 15 cm height
- Friction coefficient, μ	0.6
- Bond ratio among masonry blocks	0.5
- Thickness of masonry walls	250mm
Strengthening ties	
- Yield stress under tension, for ties	235MPa.
- Cross section of ties	diameter of 25mm.
- Volumetric weight of ties	77kN/m^3.
Loading condition	
- Uniformly distributed load transferred from floors	6.67kN/m



(a) (b)

Figure 4.2: Geometry of Experimental prototype (a) wall 1, (b) wall 2, (Michele, Giovanni, Guido, & Alberto, 1992)

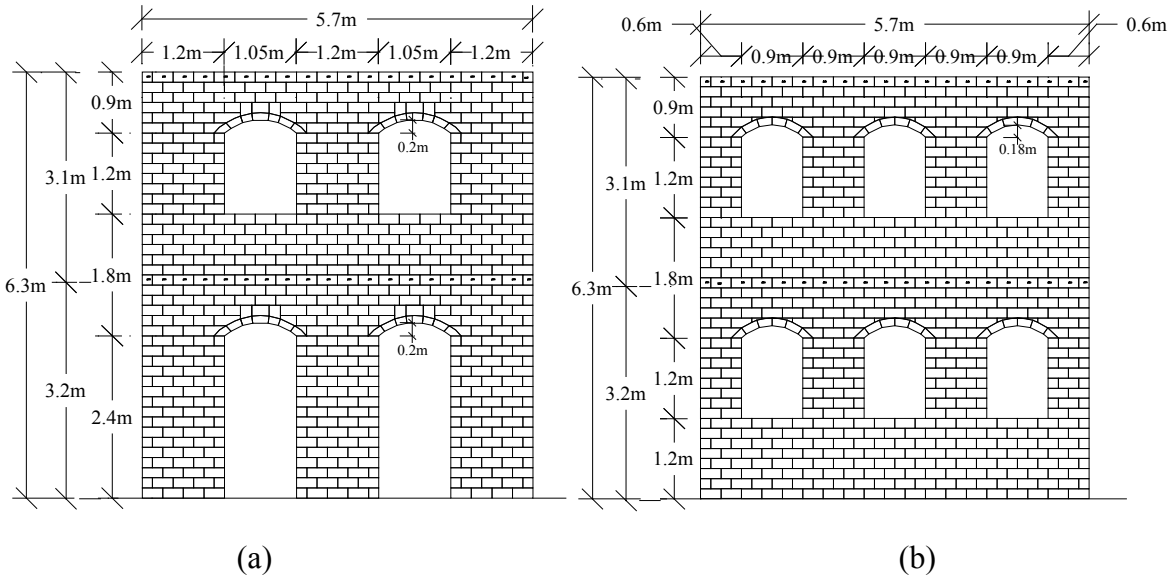


Figure 4.3: Geometry of Approximate Block models; (a) wall 1, (b) wall 2.

4.3 Parametric Analysis

The analysis with respect to the different parameters discussed in the previous chapter will lead to preliminary conclusions based up on the developed relations existing among the associated interactions of parameters versus the seismic coefficient. For instance, the study made on the masonry units aspect ratio or even size ratio will lead to selecting the best preferable units for the masonry building wall that can withstand seismic actions. When translations of those concepts and developed relations are transferred to a bigger masonry shear walls or facades, there needs a careful formation of conclusions.

To be able to visualize the parametric effects, variations on the walls are exercised in terms of the wall aspect, meaning that the walls are varied in their height by increasing the number of storey and alternatively by increasing the piers of the structural façade which means indirectly the decrease in the wall aspect ratio; approximation on the increment of the floor loads in the life of the masonry walls are also applied as an overload. Once the failure modes and mechanisms are obtained from each one of the cases, strengthening techniques like using ties are simulated to help in the tensile capacity of the wall in the earthquake direction. In most of the cases, strengthening ties are used for the purpose of preventing the out of plane behaviour of walls. They are used in such a way holding the two parallel walls of a masonry building. For instance, in Italy where it is very prone to earthquake, the use of ties in that purpose is

very common in historic structures in addition to other methods of strengthening. Similar ideas can be analysed for the in-plane tensile strength of walls. This is also tried in one of the failure modes and is discussed in here under sub topic of 4.3.1.4.

Standard Masonry wall Models

Better understanding on the effect for some parametric variations is done by having standard masonry walls. These are the same walls used to that of the experimental one. Both are characterised by the masonry properties given under Table 4.1 above and also their geometries are as shown in Figure 4.3 (a) and (b). Material characteristics are same to all the models analysed but as parametric analysis, the changes are applied on the floor load, the dimensions of the wall, the number of storey, the number of piers and on application of ties. These things need some basic reference or standard wall. Moreover, it will be taken as a reference for comparisons on the failure modes and also on the seismic coefficient, α.

4.3.1 Analysis of Standard Masonry Walls

4.3.1.1 Standard Masonry wall 1

Seismic forces to the prototype of the test were applied as concentrated forces at the floor levels in the longitudinal walls. They are shown in Figure 4.4 and they are labelled from 1 to 4. In the block analysis, the lateral forces are applied at the geometric centroid of each block and its direction is either from left to right or from right to left (because the model is symmetric). Other features on the prototype include: existence of partial disconnections between intersecting walls, floors are made flexible and there was no use of tie-beams. Hence, there will be a complement on the results of block analysis and experimental results.

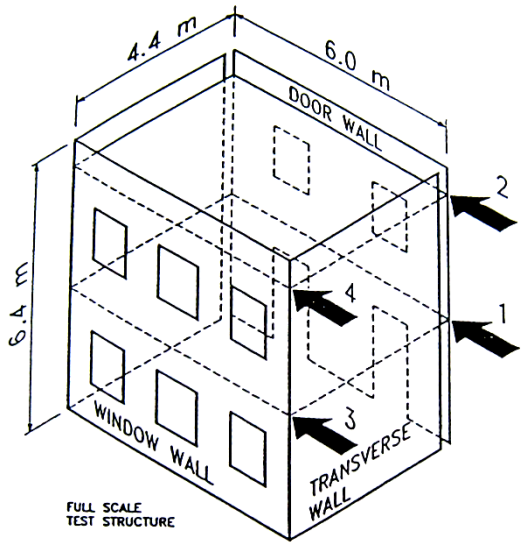


Figure 4.4: Seismic force application in the test structure

Results of Analysis for the Standard Masonry wall 1

Presentation of the results includes the block wall model, the failure mechanism, the formation of cracks and the graph for the direction and distribution of thrust lines. From the very basic definition of the limit analysis with rigid blocks, seismic coefficient equals to multiplying factor for the gravitational self weight that is applied as horizontal load at each block centroid where by it is considered as a single block Figure 3.4. The load factor can be monotonically increased from zero up to a limit for the structure to remain safe. The experiment simulated this by applying concentrated forces at floor levels. In both cases, there resulted similar failure modes. Note that the direction of the earthquake simulated here is from left to right. Comparisons on this are supported with self explaining important figures.

In Figure 4.5, the block model is shown in (a), and the failure mechanism (b) is drawn to a certain scale factor just to show how failure is associated within the block units.

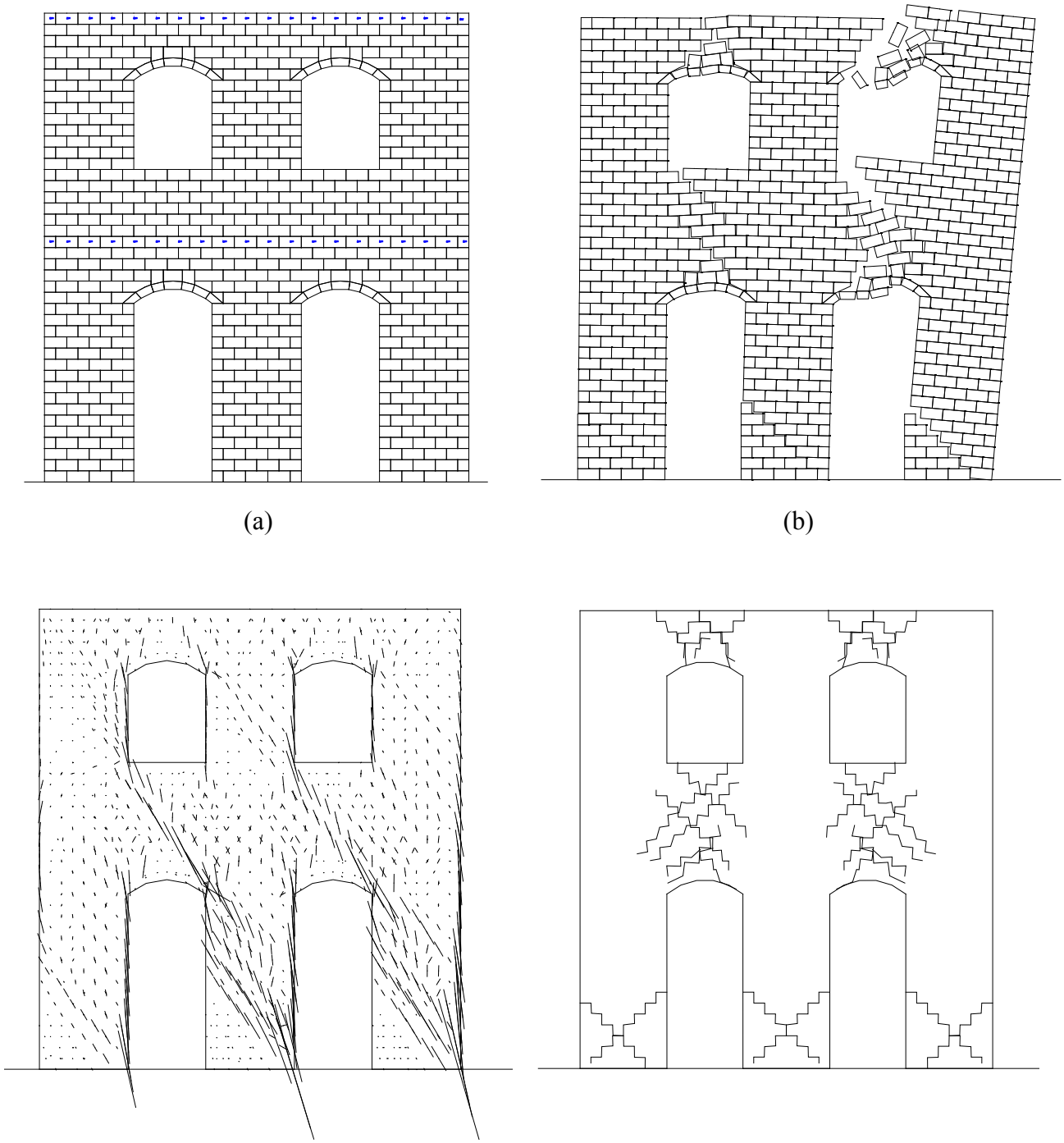


Figure 4.5: Standard Masonry wall 1; seismic coefficient, α=0.505, (a) Block model, (b) Failure mechanism, (c) Thrust lines, (d) Crack pattern

Observations on the cracks pattern can make one to categorize them as light or intense cracks. Intense cracks are basically found following the spandrels at each floor and also on the top right spandrel. Spandrel located in the second bay of the wall is the major affected one and in addition to the parallel cracks formation, also distributed cracks are seen. The trend of the cracks is same to along the spandrel. Concerning the damage to piers, the first pier experienced crack at the base, in Figure 4.6. Actually at the base of each piers it is possible to see cracks, intense cracks are found at the base of the pier at the right end. In these cases, it is reasonable and true that this situation gets back to the main assumptions of rigid block limit analysis; models of rigid block masonry are incorporated in having no interpenetration to each other. The dilatancy is allowed and seen to occur following the patterns of the bond in between the units. Moreover, it is true that masonry is assumed to have null tensile strength; this makes one to be on the safe side during analysis but the real modelling is not confirmed. Real situations are observed in walls under in plane loads.

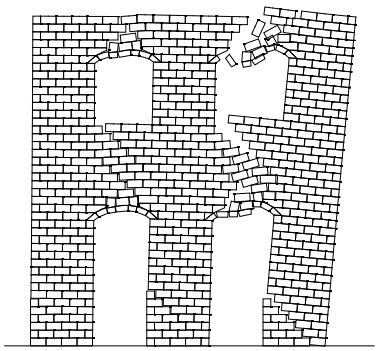


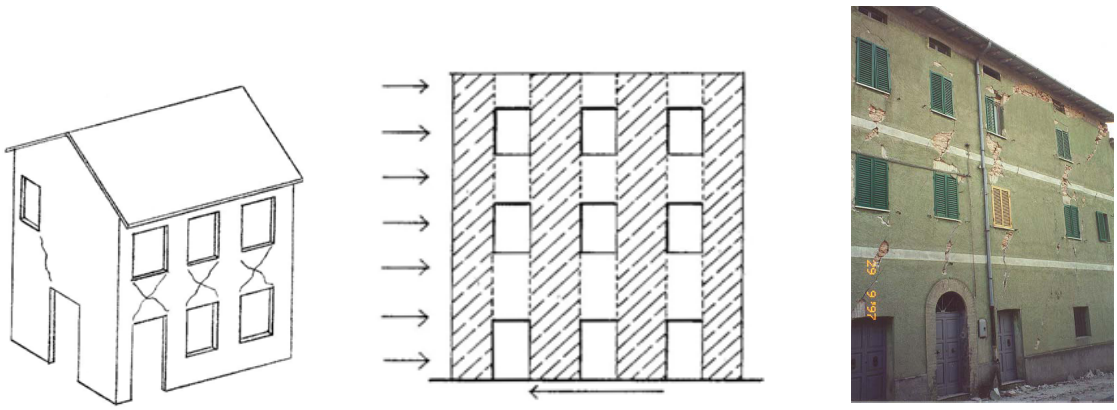
Figure 4.6: Pattern of cracks during failure for the Standard masonry wall 1

As a consequence to the failure modes shown in the Figure 4.6, there appear to happen due the following facts of failure modes (these failure modes are more related to the situations in these particular cases under study):

(1) Shear of the floor Bands (Spandrels)

Typical damage occurrence shows significant and visible crossed or diffused forms of lesions in the area over the lintels of the openings or in other terms in the spandrels. The mechanism, moreover, relates that failure of the floor bands or spandrels is cased due to the less shear capacity for bending in the plane of the wall. Conservation to the seismic force in the

structural masonry wall continues by dividing the seismic force between the masonry walls and hence, the spandrels take part on this division. This will create a system of resisting as a form of independent cantilever beams (Figure 4.7 (b)), in this case called as piers which are fixed at the base. The wall comes to a mechanism by which it will develop a form of resistance by the floor band-masonry wall (Casapulla & D'Ayala, 2006). This may be due to the weakness in lintels, arches in this case, indirectly coming from the weakness in the material make up. Or, it may occur due to inadequate height and depth of the masonry spandrel. The later idea is very important point in the determination of the failure modes of various masonry walls.



(a) (b) (c)

Figure 4.7: In plane mechanisms; Shear of the floor Bands (spandrels), (Franchetti, 2008)

(2) Shear of the Masonry walls (Piers)

This is another type of failure mode characterised typically by the failure in the masonry piers that are under high stress in its in-plane. From the sample picture of Figure 4.8, it is common to see failure on the middle wall parts that make up the masonry in between the openings. It is characterised by having crossed cracks in the piers that are in the middle. In the edge panels, there occurs an inclined form of cracks. Generally, such kind of damages can be resulted from the presence of many openings; moreover the irregularity of the opening has also an impact on failure mode determination (Shariq, Abbas, Irtaza, & Qamaruddin, 2008). Presence of discontinuities in the masonry will have a structural cause of failure.

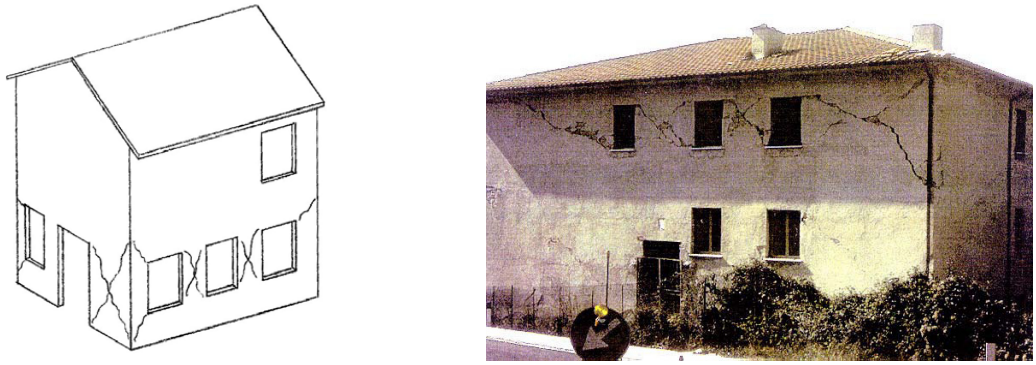


Figure 4.8: In plane mechanisms; Shear in masonry walls (Piers), (Franchetti, 2008)

Relating to these two basic and common types of failures in masonry walls mainly under in-plane actions, the block model analysis result has similar failure mode described in the first case (1) above. Getting back to the trends in the crack development in Figure 4.6, as earthquakes create the loads in both the directions as cyclic loading, the immediate and first damage prone part is on the top spandrels located at the far end of the wall. The distribution of the cracks then appears at the middle spandrels located to the right. Due to symmetry in the building geometry, cracks will exhibit symmetry. Finally, distributed set of cracks formed on the spandrels on the middle spandrels and opening of the top spandrels. The bases of the piers are also exposed to cracks at later stages in relative to the other formed cracks. Piers are the undamaged parts in relative to the spandrels. This implies that resistance to the lateral loads comes from the piers.

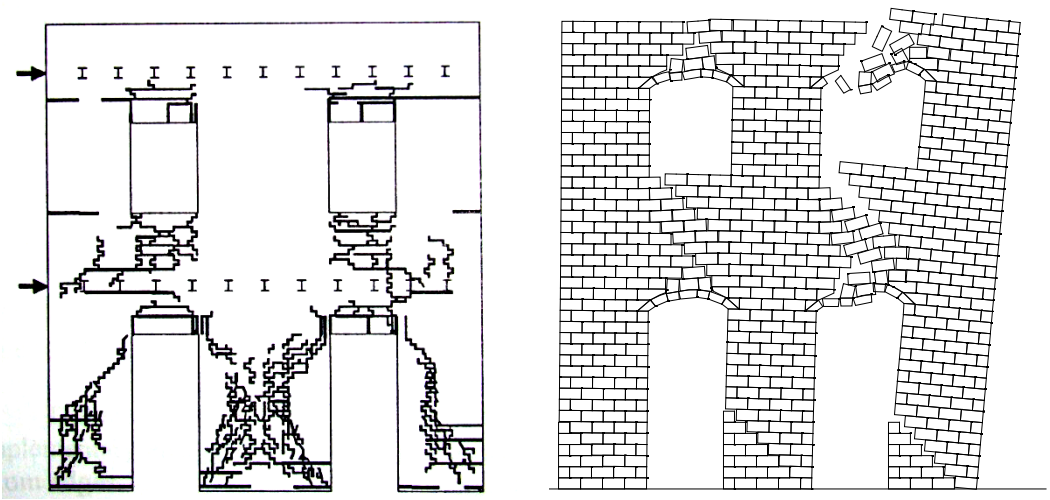
Contribution of thrust lines to interpretation of cracks is very relevant. Looking to the Figure 4.5 (c), the concentration of the thrust lines are associated to the regions where cracks appear. This implies to the situation in which the cracks are formed by the tensile forces, and indirectly thrust lines show this fact. In addition, the piers to the right position are under high compression just on the areas where concentration of thrust lines is shown and indirectly on the reverse case, the pier to the left is under tension at its base where there is low concentration of thrust lines.

One important idea can be concluded from these two types of failure types: the second case discussed is the most important and more susceptible to the masonry building in such a way that the storey fails and then total failure of the structure follows. But in the first type of failure (Figure 4.8), failure of the spandrels is not as much dangerous to that of the piers

failure; the structure can resist as a swinging mode by the piers existing. The structure response can come to the one as shown in Figure 4.7 part (b).

Comparisons of the experimental and Block analysis results

Even though there are some modifications on the models of the experiment and the block model, it is obtained almost the same conditions of failure modes. Comparison of failure modes can be made on the picture in Figure 4.9. Concerning the floor loads, the experimental values were very high values which are difficult to realize, hence in this analysis it is made to a reasonable value meeting to the real conditions. Beyond this difference, the cyclic experimental result at the final cycles brought basically same failure modes except in the extent of damage. The experimental loadings are too high as described, and the lateral loads are applied at the floor levels, this will make a little difference on the damage extent. More information's to each cyclic loading results can be consulted from the report book Experimental and Numerical Investigation on a brick masonry Building Prototype (Michele, Giovanni, Guido, & Alberto, 1992).



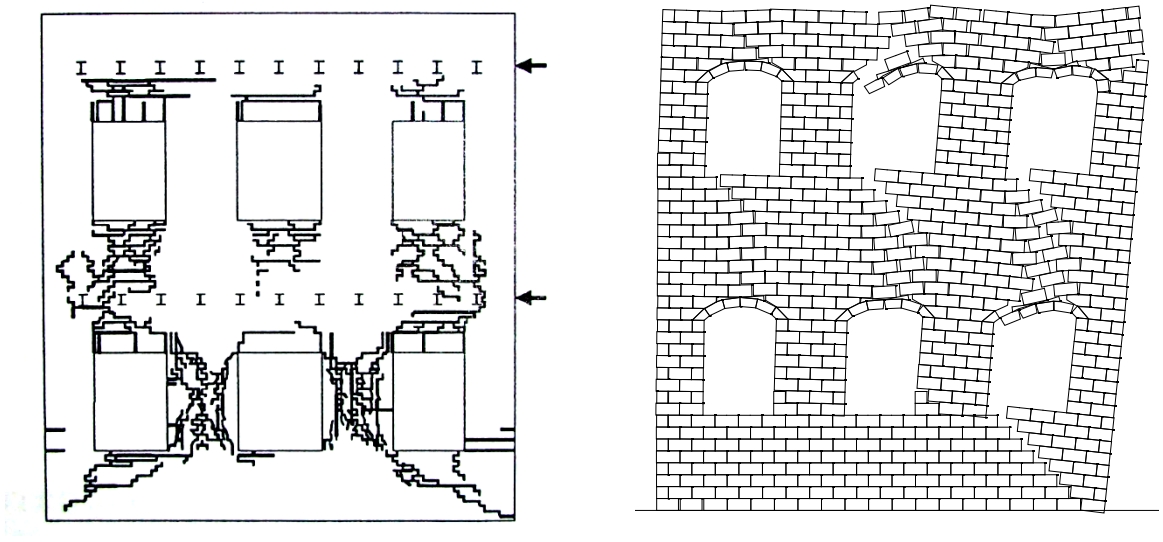
(a) (b)

Figure 4.9: Results comparison for masonry wall 1; (a) Experimental test result (Magenes, Michele, Giovanni, & Alberto, 1995), (b) Block analysis result,

4.3.1.2 Standard Masonry wall 2

The second wall, shown in Figure 4.3 (d), is analysed for its seismic capacity under lateral loads. This wall is typical or representative to walls having window openings in a regular manner. It is carrying the same value of transferred floor load, 6.67kN/m. The model is assembled of the same masonry units as to the masonry wall 1. Moreover, it contains totally four piers, but in the first cases there were only three piers. Hence, in these analyses, it is possible to see the differences on failure mechanisms and also on the changes in failure mechanism due similar changes made in the first analysis. Of course, comparisons to these models are not possible on the cases of values of seismic safety factor.

Model analysis results are having similar failure experiences to the masonry wall 1. It is characterised by spandrel failure as described in detail in the wall 1. Comparison on the failure modes can be seen below in Figure 4.10. Detail description of the failure mechanism is as described in (1) and (2) of 4.3.1.1.



(a) (b)

Figure 4.10: Results comparison for masonry wall 2; (a) Experimental test result (Magenes, Michele, Giovanni, & Alberto, 1995), (b) Block analysis result,

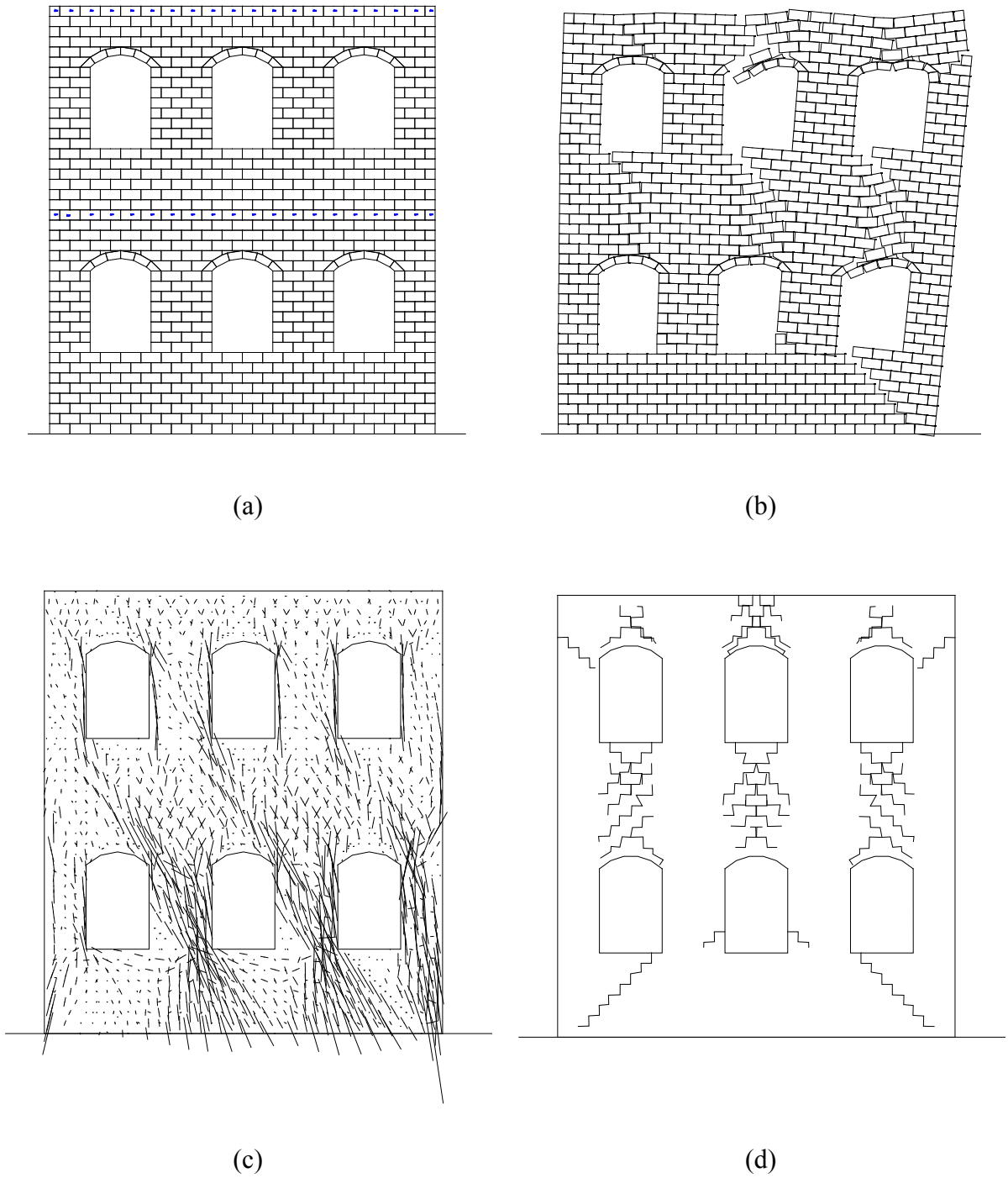


Figure 4.11: Standard Masonry wall 2; seismic coefficient, α=0.446, (a) Block model, (b) Failure mechanism, (c) Thrust lines, (d) Crack pattern

4.3.2 Variation Analysis

4.3.1.1 Effect of Overload

Consideration of systems under variations is the important part to predict the relations of characteristics in interest. One of those question of interest is what will happen to the seismic limit capacity when loads are changed over the system by means of when the floors are modified or may be additional floor loads are increased. The change on service of the structure may also change the load condition. These and other like conditions, as mentioned out in the main purposes of this analysis put the masonry walls into a system which will have a change in the capacity of response.

Loads are generally main parts of the structure involving in the lateral capacity. Recalling back to Chapter 3, analysis on the response of the standard reference wall to the overload had shown that the seismic limit will increase directly by the increase in the load increase Figure 3.26. This is observed also on the structural walls analysed here. The increase in loads is assumed to happen twice the existing load. For more value of ranges, it is not an interest and also it is not necessary, even the level of compressive strength will lead to crushing. But basically the failure mechanisms of both cases, the standard one and the overload case, are the same.

Seismic limit capacity increase is checked for the overload. From the result in Figure 4.12, the seismic safety factor is found as 0.660. As compared to the standard wall, it showed an increase of almost 31%, which is a significant value.

$$\text{Percentage increase in safety factor} = \frac{0.660 - 0.505}{0.505} \approx 31\%$$

As a general idea in terms of structural safety, the loads are increasing the safety against the lateral loads. This is also proved in the walls studied under Chapter 3.

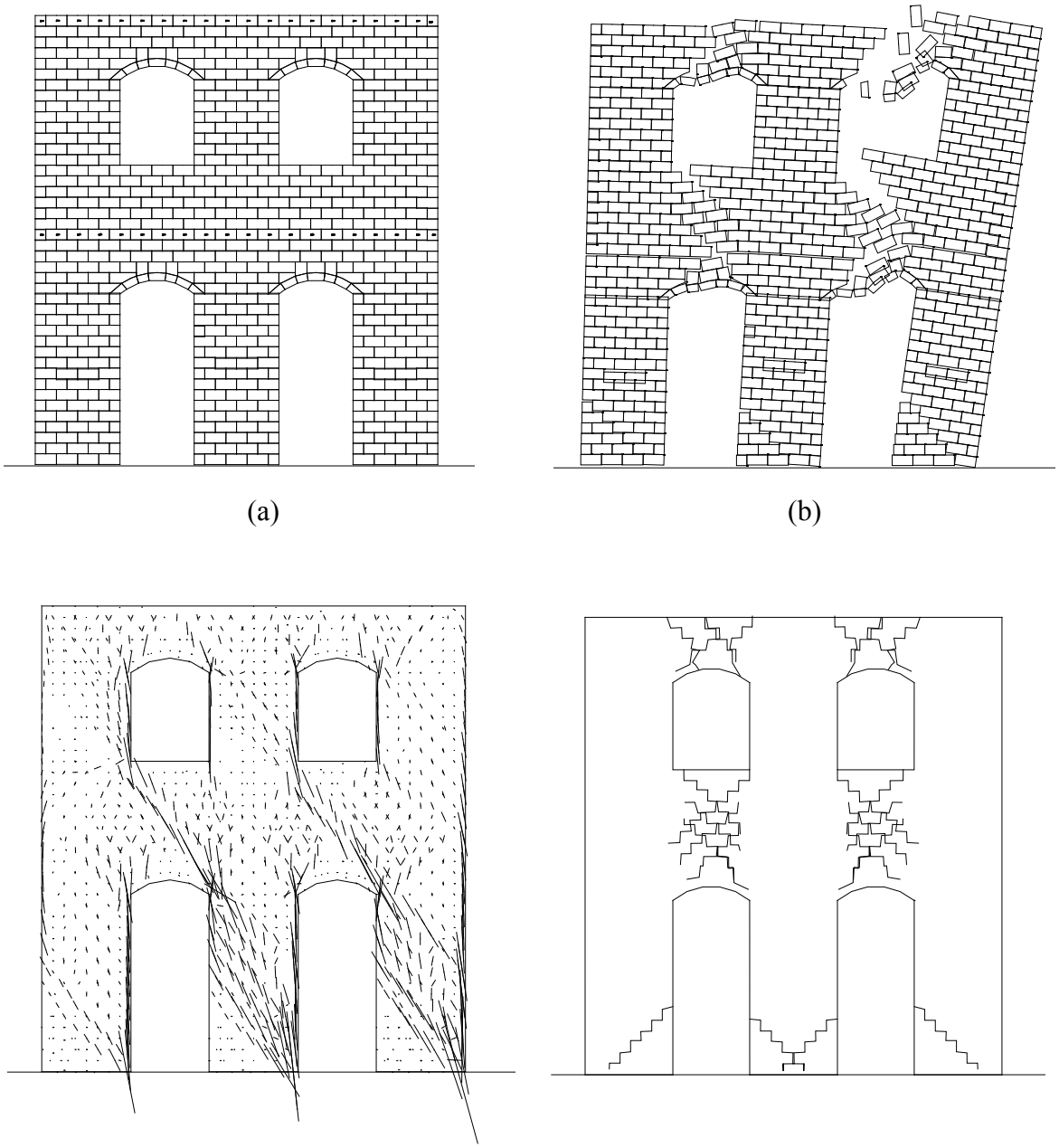


Figure 4.12: Standard Masonry wall 1 under Overload condition; Seismic coefficient, α=0.660, (a) Block model, (b) Failure mechanism, (c) Cracks formed, (d) Thrust lines

4.3.1.2 Effect of Piers

Width variations are real in masonry structures in addition to the storey variation. Understanding the effect of wall aspect ratio, in Chapter 3, is done by varying the width of the wall. This concept is also applied for the standard masonry wall 1 and 2.

A decrease in the width of the wall, keeping the height the same, will bring it to simulate like a cantilever or in other words, slender walls. In association to this, the wall will be very susceptible to lateral loads as when it is decreasing in width and also the safety factor decreases; this is discussed under the sub topic of 3.5.1. Hence, with same idea, what will happen to a structural masonry wall? This will need a series of model analysis for a complete set of conclusion. But generally, the increase in the piers will have a significant increase in safety factor, this is true when the reference wall in the standard analysis case is already in very low safety level (significantly damaged) otherwise if it is significantly safe, then the increase in the piers will not have a considerable change in the safety factor.

Based on the block analysis results, the piers effect is found as determinant for this specific masonry wall. One thing here should be clear that, it doesn't necessarily imply an increase in number of piers is an increase in number of piers. It may be unimportant in increasing the safety factor to seismic forces as in cases of long walls which are already attained their lateral safety. In this particular study, the increase in number of piers is not changing the safety factor. Safety factor is obtained is 0.497, for wall 1, which is very similar to the standard one, 0.505 and 0.578 for masonry wall 2 which is relatively an increase in value. Moreover, the failure mechanisms are obtained as having similar characters. The crack patterns follow the spandrels; intensive damage is seen on the spandrel which is found at far end in relative to the direction of lateral forces. Results of analysis are shown in Figure 4.13 and Figure 4.14 below.

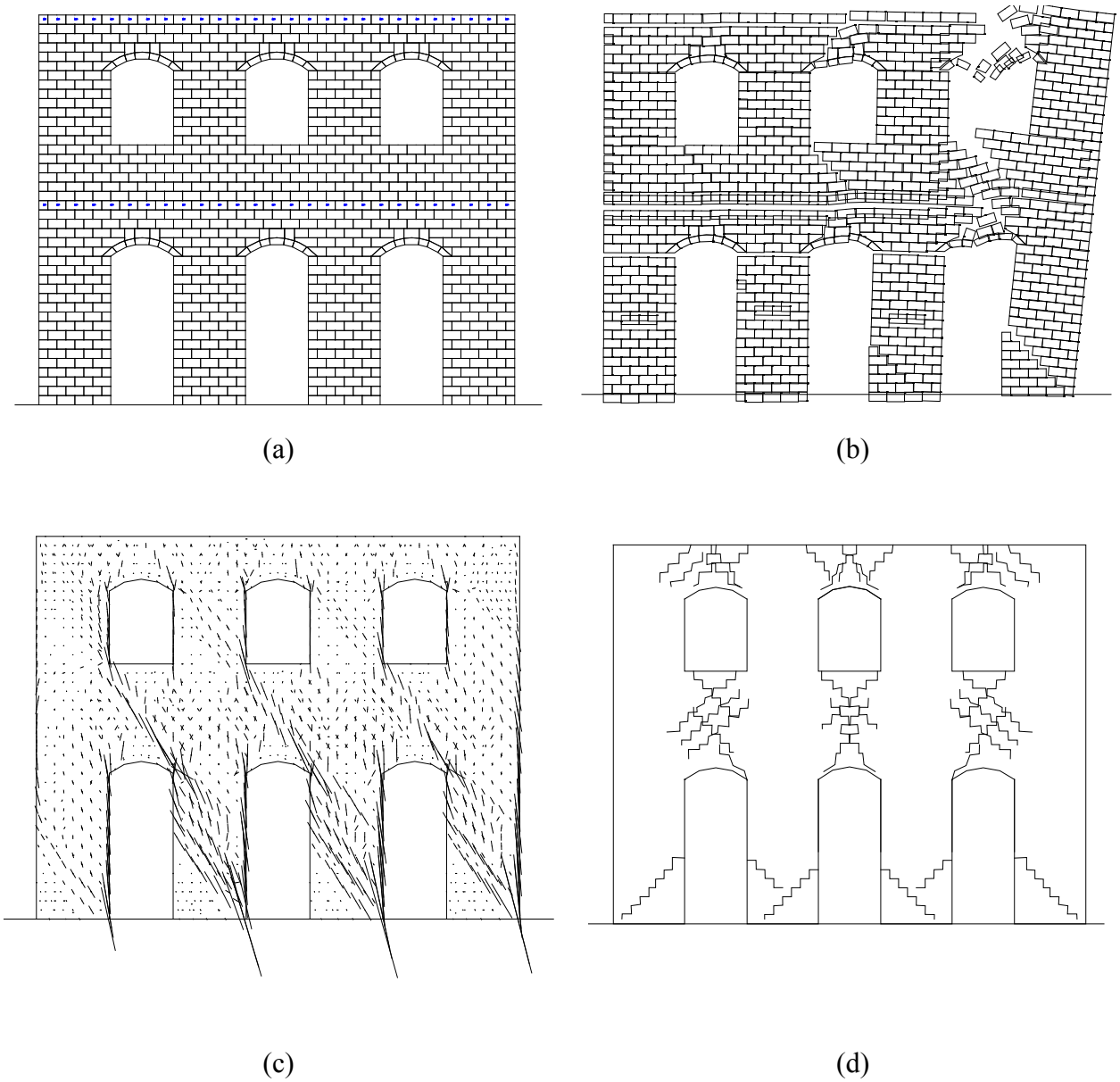


Figure 4.13: Addition of a Pier to the Standard Masonry wall 1; Seismic coefficient, α=0.497, (a) Block model, (b) Failure mechanism, (c) Thrust lines, (d) Crack pattern

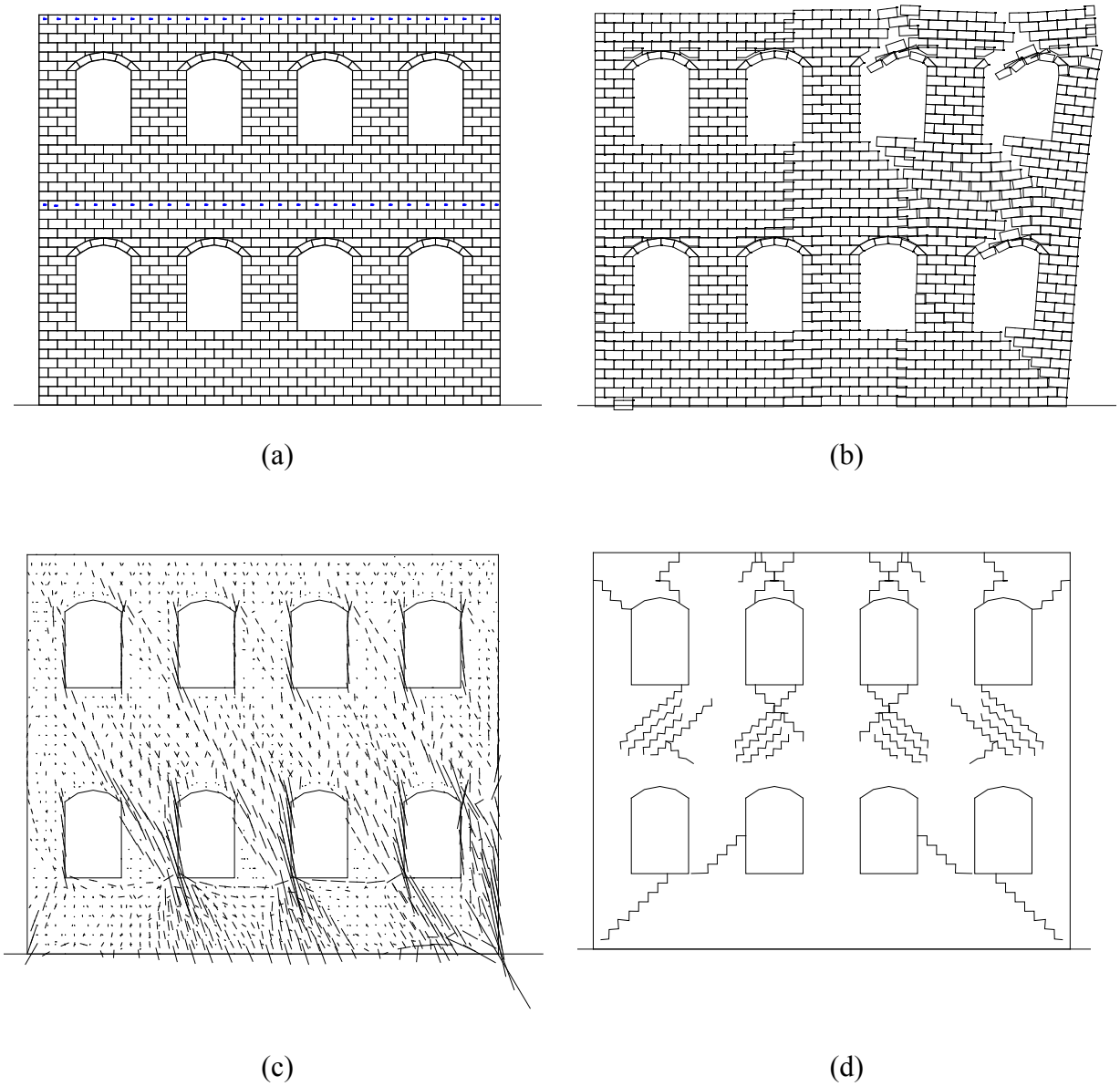


Figure 4.14: Addition of a Pier to the Standard Masonry wall 2; Seismic coefficient, α=0.578, (a) Block model, (b) Failure mechanism, (c) Thrust lines, (d) Crack pattern

4.3.1.3 Effect of Additional Storey

Storeys of masonry buildings are indirectly concerns on the slenderness of the building. In other terms, they describe about the degree of slenderness. The higher the number of stories is the slender the structure and the higher the susceptibility to lateral force damages. Historical masonry structures are common to have storeys in the range of 2 to 4.

In this part of analysis, the other indirect idea applied in the case of piers will be assessed. This part will be intended to understand the rise effect instead of the width effect. This is done by keeping constant the width of the wall.

In real situations of earlier times, storeys were constructed after once the building is finished. This kind of modification is not that much common but they were done for some space requirement reasons and at the same time in order to keep its functionality the safety of the structure should be verified. Hence, in such situations there will happen structural effect. It is so clear that the stability of these masonry walls of high storeys will be a direct question to our mind. This idea is verified by the block analysis for lateral load capacity.

Realistic failure modes are observed from the analysis result. Moreover, the safety factor decrease indicates the more dangerous situation which is an increase in the storey number. It is observed that the safety factor decreased by almost 18%.

$$\text{Percentage decrease in safety factor} = \frac{0.505 - 0.416}{0.505} \approx 18\%$$

Cantilever forms of failure are very common to masonry structures which are high rise. It is also parameterised for a masonry wall in the previous Chapter 3. Typical failure mode can be of Figure 3.9 (f). The wall aspect in terms of the increase in the height of the masonry wall will lead to very low lateral capacity situations where by the cantilever or overturning failure governs. The piers are stiffer than the spandrels and this led to crack development in the spandrels block units. Overturning of the whole system seems to happen at cracks developed at the base of the second storey.

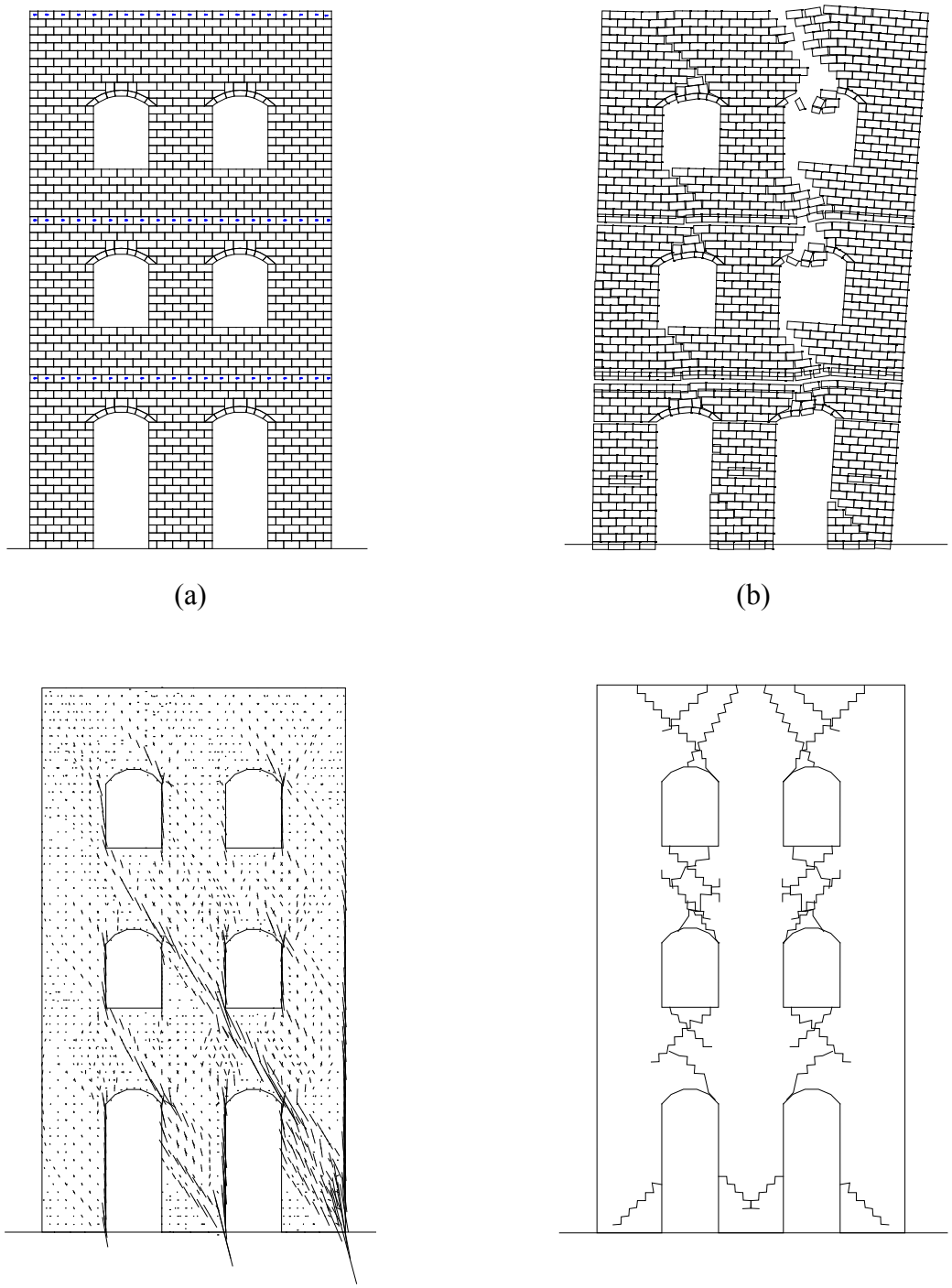


Figure 4.15: Addition of a Storey to the Standard Masonry wall 1; Seismic coefficient, α=0.416, (a) Block model, (b) Failure mechanism, (c) Thrust lines, (d) Crack pattern

4.3.1.4 Effect of strengthening ties

The most common form of failure of masonry buildings is the separation of walls at the corners. This directly contributes to the collapse of a building by freeing the walls which then disintegrate. The most important single element to restrain the progression of damage is any device which ties one wall more firmly to another. Having this on one hand, the use of ties can be used with same analytical reasoning for the resistance in the in-plane mechanism. Applying ties for the support in the in-plane failure mechanism is a new idea and it will be examined in this thesis as a preliminary part of investigation. The application of ties continues from the identification in the failure mechanism. Once the failure mechanism is identified, it is possible to know where there is a need of strengthening positions; usually it is under the floor levels. Furthermore, application of ties in the out of plane mechanisms is more experienced in Italy.

For this wall, as can be seen in the characteristics under Table 4.1, the ties of 25mm diameter are supposed to be used. Getting back to the failure mechanism given under Figure 4.5 and the one with ties in Figure 4.17, it is possible to compare the changes in the failure mechanism. It is clear to see that blocks found in the spandrels are fitted to prevent the crack formation by the tensile behaviour of the tie. Directions of principal stresses in the blocks close to the tie, in Figure 4.17 (c); indicate the forces that try to close this crack formations which were more visible in the un strengthened wall failure. The tie will develop forces in the reverse direction, this is the main importance of the ties to investigate in the in plane strength.

Moreover, recommendations on the dimensions for ties and also for the tensile strength can be checked through various analysis of same model. The analysis modifications to improve in the lateral capacity can be achieved in one of the two cases below:

- *Strength*: the change in the strength of the ties increases the strength generally. But variations or increases on the strength of the ties will be up to a certain limit, the increase in furthermore is not important. This is because the strength needed is to a specific value but more than that it is uneconomic and structurally unimportant.

- *Area of ties*: ties can be selected from commercial products. Structurally, it is not a question whichever is used, but for different ties of same specific tensile strength, it is

dependent on the area of ties. For each failure mode, there exists a minimum area of ties basically for establishing equilibrium.

Relation to those two ways with respect to the change in the seismic safety factor is applied to the simple masonry shear wall indicated in Chapter three of Figure 3.5. As it shows in Figure 4.16, the safety factor will increase for an increase in tensile strength up to a certain limit of requirement. But then the structure is under a condition for not further need on support for additional strength of steel for its in-plane strength. The wall failure mechanism cannot be changed for further increase in the strength of the ties. So the necessary part of this graph is to the peak of the increase. This is basically important because for big wall models under investigation for seismic safety factor, there is no idea for selection of the ties before analysis. Hence, the need at least numbers of analysis will guide one to select for the appropriate specification for the ties whether the area or strength even for the same types of model walls by simply approximating the models. Similar implying ideas are found in the second case, steel area.

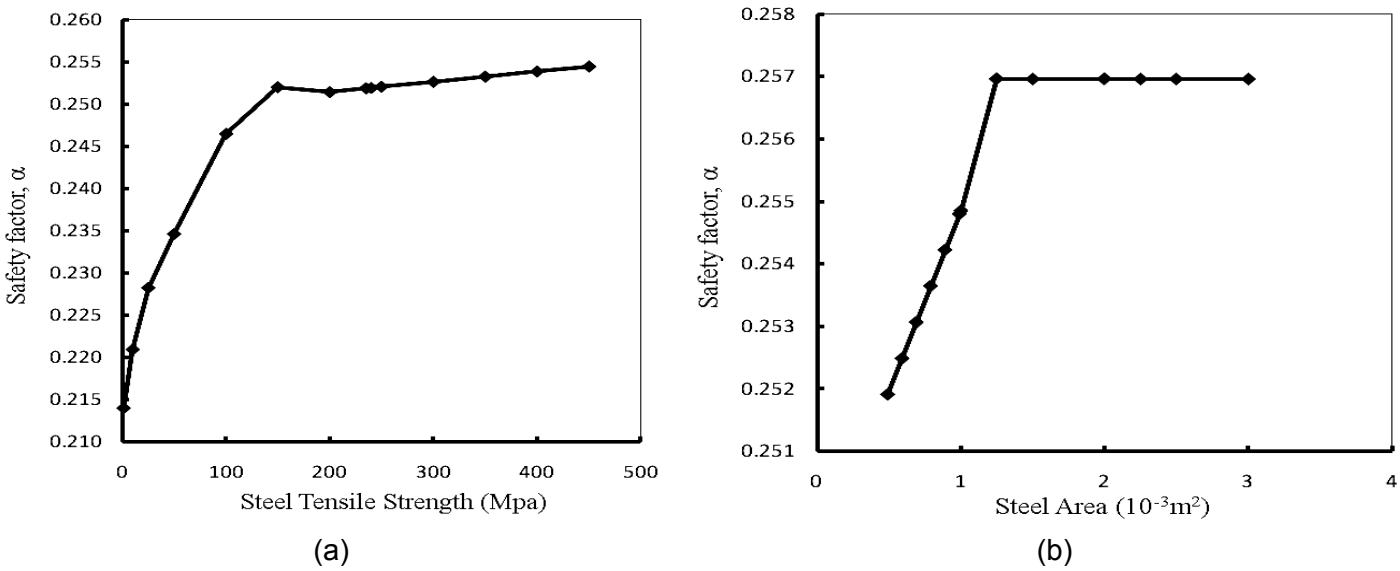


Figure 4.16: Effect on seismic safety factor; (a) Tie tensile strength, and (b)cross sectional area

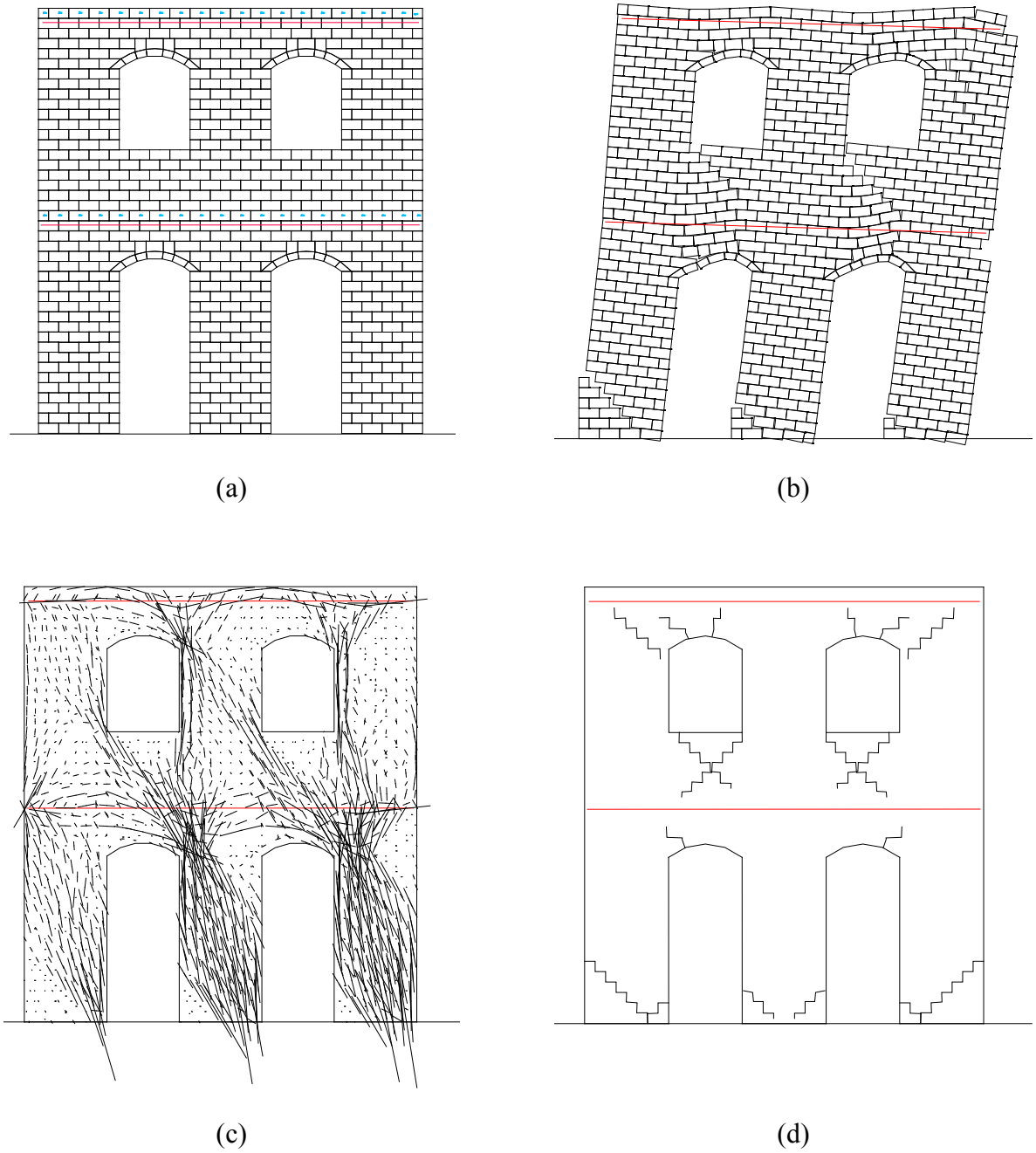


Figure 4.17: Strengthening ties used in the Standard Masonry wall 1; Seismic coefficient, α=0.478, (a) Block model, (b) Failure mechanism, (c) Thrust lines, (d) Cracks pattern

From the analysis on the failure mechanism from Figure 4.17 (b) gave to a local failure mechanism. Local failures are not such important or are not such sufferings to the structure. Block analysis terminates when the failure occurs at the blocks found just below the tie in the first floor. Due to this, the safety factor reduced to a value very close to the standard one. In

fact this is not the point for the comparison. A local failure on single unit is not a big deal for the safety of the structure. The idea is to modify again the model so that the local failures will not occur and the safety factor will get increased. Observing on the failure mechanism will guide on how the ties keep the system as possible as rigid as single response.

Table 4.2: Summary of seismic safety factor results for masonry wall 1

Parameter (considered variation)	***Seismic safety factor,*** α
Standard model (reference)	0.505
Effect of overload	0.660
Effect of additional piers	0.497
Effect of additional stories	0.416
Effect of strengthening ties	0.478

4.4 Propositions for Macro Modelling

The predictive modelling of the behaviour of masonry structures, particularly in the non-linear range, remains a challenge, due predominantly to their semi-discrete and composite nature. An adequate computational model must include the fundamental mechanisms that characterise the composite action. Micro modelling studies are necessary to give a better understanding of the local behaviour of masonry structures (Zuccaro & Rauci, 2008). Macro models are applicable when the structure is composed of solid walls with sufficient large dimensions where the stress across or along a macro-length will be essentially uniform. They describe the response of masonry with acceptable accuracy. However, they fail to simulate failure modes involving separation or sliding between different parts. Some FEM macro-models afford the analysis of large structural members (for instance in analysis of the typical bay of a church). Using FEM macro-modelling to analyze entire large structures is still challenging the capacity of modern computers.

In the levels of micro modelling, the different components of masonry; block units, joining mortar, interfaces, are modeled separately and in addition to that some specific constitutive equations are equivalently utilized for each type of component. In particular, interfaces are

described so that they can represent frictional sliding or complete separation between units as a realization for cracks. A possible simplification consists of using the blocks to model the combined response of units and mortar. In that case, the model, only consisting of blocks and interfaces, allows significant reduction of computational effort. Due to their computational needs, micro-models can only be used to analyze simple members. Moreover, it is clear that masonry is described as a homogeneous material characterized by a set of average (or homogenized) properties. A criteria or technique is needed to derive the homogenized properties from those of the individual components (units –stone blocks, bricks-, mortar and the unit-mortar interface). Homogenization is the most rewarding approach for the transition from micro to macro modelling. Some of the concepts on it can be consulted from the paper by Lourenço, Milani and Tralli (Milani, Tralli, & Lourenço, 2006).

In relation to this, when the seismic assessment of masonry structures is composed or attempted of by macro modelling, then the different cases from the multiple types of failures are under treatment for each case of failure. In other words, for any masonry structure assembly, it is possible to identify many possible failure modes composed by large blocks (referred as macro). The development of macro blocks from among the many possible mechanisms of failure is of interested in searching the crack positions. Cracks of the masonry structure indicate the very important fact of masonry as it is characterised by rigid behaviour high under compression and less in tension. Hence, the tensile incapability will give cracks very informative for the macro-model development. As mentioned in the previous chapter concerning Block software, sub topic 3.2, it gives the one and only one failure mode for a specific masonry structure. This implies that, it is very important and specific for one who wanted to develop macro-block failure mechanism for that structure. In this sub topic it is developed the macro-model for both the standard walls under study. Moreover, there is a much related importance of macro block models; they are used for the seismic safety verification by the non-linear kinematic analysis (OPCM, 2005).

Macro-model for Masonry wall 1

Predictions of failure mechanisms from the abacus of different failure modes for a specific masonry structure are solved and or selected by the one and only one mode of failure given by Block software. This is very important use of Block software; it selects the possible one

among the computations to be conducted for a set of modes of failure. The macro block models for the masonry wall model 1 are developed from the failure mechanisms presented in Figure 4.5. The cracks are drawn as separation lines for the initialization of the kinematics as in the direction provided by the motion direction. The important blocks associated in the mechanism are numbered as 1, 2 and 3 (see Figure 4.18). Macro blocks in the failure mechanism will have the corresponding inertial forces and induced lateral forces obtained from the seismic limit capacity coefficient analysis by the Block software (the lateral force at the centroid will be equal to the product of the seismic coefficient and the gravitational loads).

The predictive modelling of the behaviour of masonry structures, particularly in the non-linear range, remains a challenge, due predominantly to their semi-discrete and composite nature. An adequate computational model must include the fundamental mechanisms that characterise the composite action (OPCM, 2005).

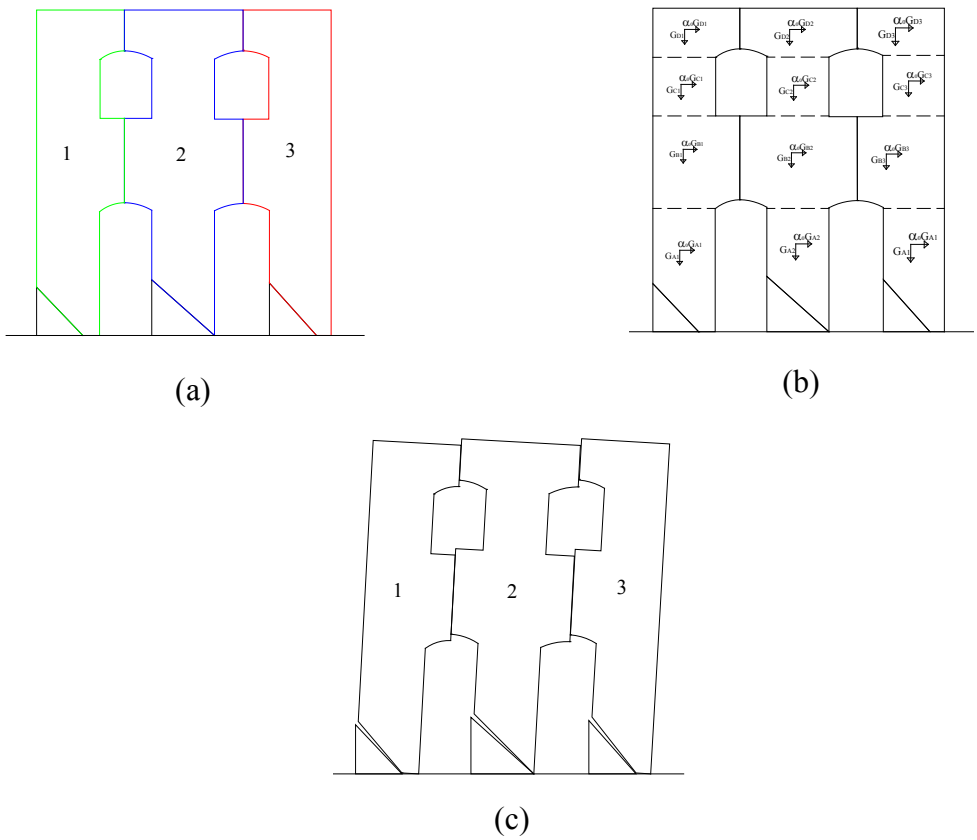


Figure 4.18: Macro block modelling for masonry wall 1, (a) macro blocks, (b) lateral forces at macro blocks, (c) failure mechanism

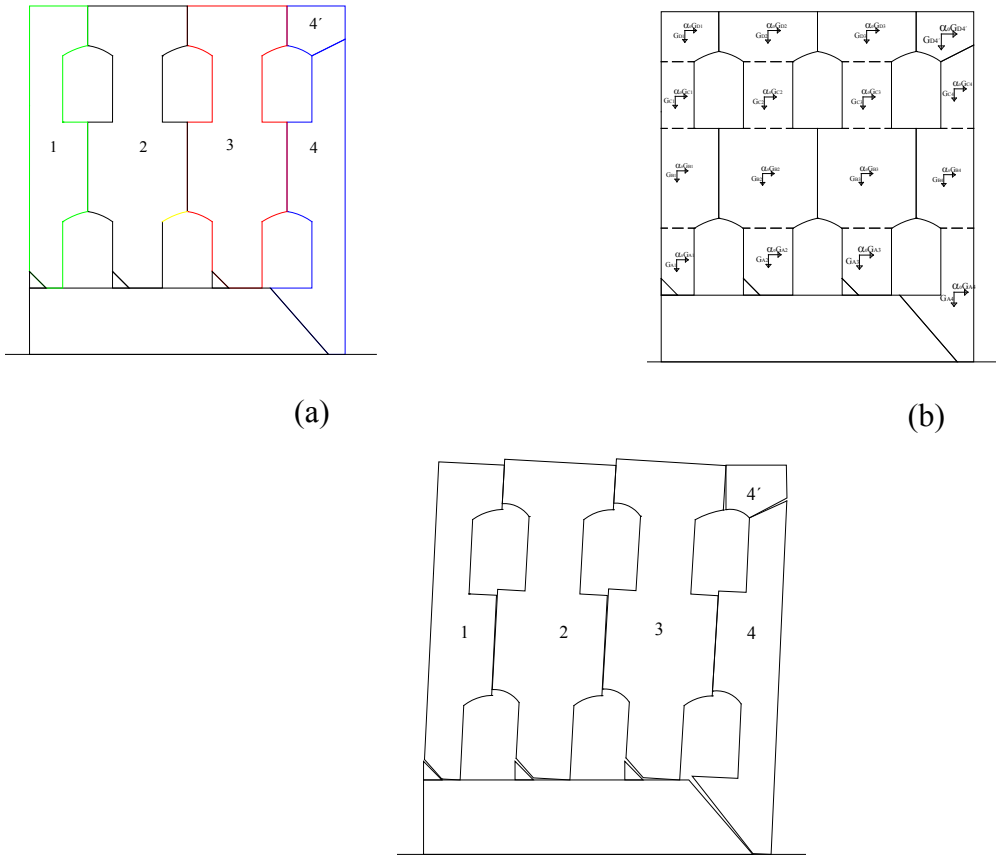


Figure 4.19: Macro block modelling for masonry wall 2, (a) macro blocks, (b) inertial forces at macro blocks, (c) failure mechanism

Capacity curves

In the kinematic analysis of masonry structures, it can be applied rotation to a specific system and results of mechanism with respect to the movement of a certain control point on the rigid block (in this cases macro blocks) can be plotted. This results obtained defines the capacity curve of that system. The system will be, in fact, moving until there appears a total incapability in stability where by the effect of the derived lateral force (seismic force) is the only force on the stricture rigid center. At that point, the gravitational loads will become zero because it will not create any moment force at the rigid centroid.

The application of the kinematic method brings to the determination of the capacity curve of the analyzed element. There are two key points here in kinematic approach to be noted:

1. The evaluation of the horizontal action that activates (starts) the kinematism. This is associated to the value of α_0. That indirectly means the values obtained from the block limit analysis and is the output safety factor from block software.
2. The determination of the process of the horizontal action that the structure is progressively able to stand with the evolution of the mechanism, until the annulment of the horizontal force itself; in other words, as long as the structure is not able anymore of stand horizontal actions (Franchetti, 2008).

The seismic coefficient or α multiplier is introduced, ratio between the horizontal forces and a quantity depending of the corresponding weights of the present rigid macro block masses.

For the application of this analysis method it is hypothesized to have the following general things which are basically similar to the limit analysis of masonry assumptions:

(1) No tension resistance of the masonry;
(2) Absence of slipping between blocks;
(3) Infinite compression resistance of the masonry.

However, approximate simulations for the realistic cases are also necessary and additional assumptions that should be taken into consideration may be (Lagomarsino, Resemini, & Giovinazzi, 2006):

- slipping between blocks, considering the presence of friction;
- connections, even of limited resistance, between masonry walls;
- presence of metallic chains;
- limited compression resistance of the masonry,
- presence of walls with disconnected faces.

Procedures for determining the Capacity Curve

Displacement capacity of the structure up to collapse can be presented by the relation of the two variables: the displacement and the seismic multiplier, α. by means of the mechanism considered from the block limit analysis from block software, the initial values of coefficient or the initializing factor is obtained. Then, the horizontal load multiplier α can be estimated,

not only based on the initial configuration, but also on varied configurations of the kinematic chain representative of the evolution of the mechanism and described by the displacement d_k of a control point selected on the system (see Figure 4.20). Series of kinematic chains for with respective rotation can be found as presented in Figure 4.21, for instance for the masonry wall model 1. This analysis has to be performed until the configuration corresponding to a multiplier α=0 with the respective displacement $d_{k,0}$, is reached (Lagomarsino, Resemini, & Giovinazzi, 2006).

Basics for the kinematic analysis is by applying virtual principle in which work done by applied force is balanced by the work done by the system response, displacement generally the mechanism by itself. Computation for each kinematic chain can be done by expressing the geometry of a generic configuration as a function of the finite rotation $\Theta_{k,0}$, and applying the virtual work principle. For every configuration of the kinematic mechanism of the rigid blocks in Figure 4.21, the value of the multiplier α can be obtained by equation in 4.1 below with reference to the varied geometry. The analysis can be performed either graphically, by identifying the geometry of the system in the different configurations until collapse or analytically, by considering a sequence of virtual finite rotations and progressively updating the system geometry (OPCM, 2005).

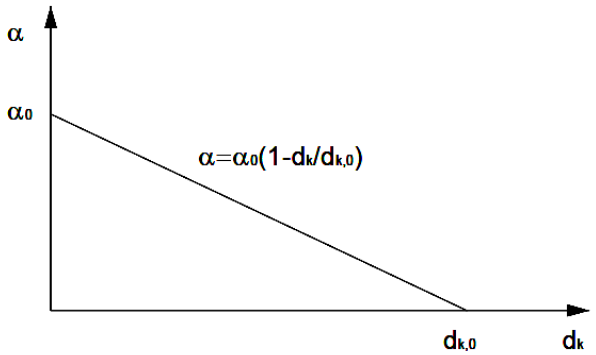
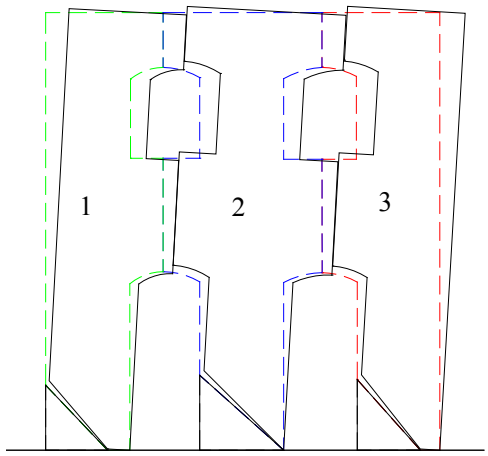
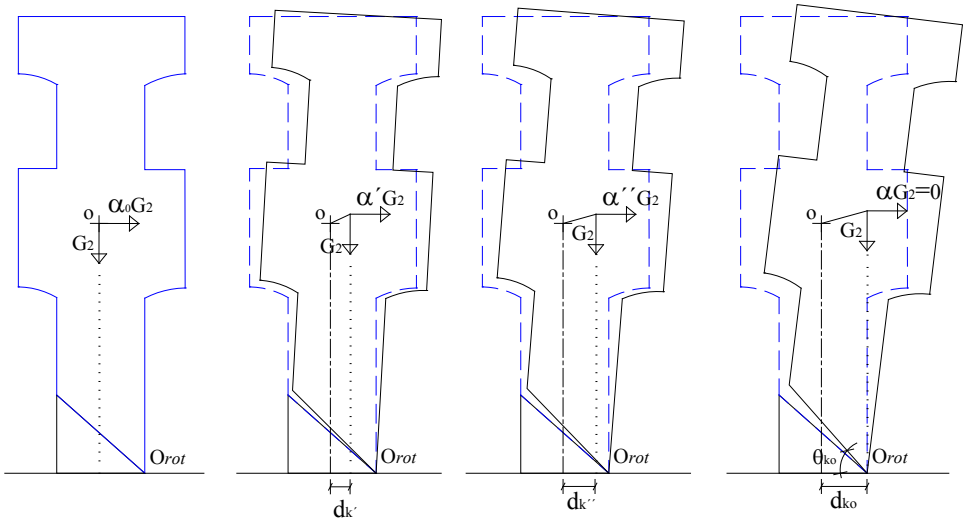


Figure 4.20: Generalised Linear capacity curve (Franchetti, 2008)



(a)



(b)

Figure 4.21: Kinematic chain of the second pier, (a) failure mechanism of the macro models, (b) rotation kinematics of second macro block

From the generalized multiplier **α - d_k** relationship shown in Figure 4.20;

- **α_0** is the multiplier that activates the mechanism (damage limitation state) obtained from the block limit analysis by block software (safety factor).
- **d_k** displacement of a suitable control point k of the system (for example the center of the mass)
- **$d_{k,0}$** refers to the displacement of a selected control point k on the system, for which the multiplier of the derived lateral loads is equivalent to zero (α =0), in this stage the system is already going to fall.

If the various forces acting (weight, external and internal actions) are constant as the mechanism evolves, the curve that is obtained is almost linear and the displacement $d_{k,0}$ corresponding to the $\alpha = 0$ condition can be derived from the simplified expression:

$$\alpha = \alpha_o (1 - d_k/d_{k,o}). \qquad 4.1$$

There is another important thing that may appear in cases when progressive variation in external forces with the evolution of the mechanism is considered (for example the elongation of a tie-rod), the curve can be assumed to be piecewise linear and the displacements can be estimated at significant points (for example: tie-rod yielding, tie-rod failure, etc.). These are some of the special consideration that can be useful in the analysis as they may change the values but the relation still keeps linearity.

4.5 Conclusions

In this chapter, models of masonry walls representing the ancient Italy buildings are analysed by Block limit analysis. In association to the previous chapter, the parametric part of work here is by varying the number of piers, increasing the load to a supposed service load, increasing the number of stories, and introducing in plane strengthening ties, as a new foundation for this work, are the preferred analyses chosen. Standard walls of block models are used as a reference for the variation in relation to describe the change. Both walls of analysis are found to have similar results from the previous studies. Majorly, the standard walls are found to have failures in their spandrels. This led to the system to have a response in form of cantilevers. The changes are then applied to both models. A decrease in the width of the wall, keeping the height the same, will bring it to simulate like a cantilever or in other words, slender walls. Pier effect to the reference wall safety factor is dependent on basically on the situation how the safety level is. It is found that the increase in the piers will have a significant increase in safety factor. This is not necessarily true. It will be leading to that conclusion when the reference wall in the standard analysis case is already in very low safety level. But, if the safety level is under high significant level, then the increase in the piers will not have a considerable change in the safety factor. Storey increase is the indirect meaning of the decrease in the safety factor due the width decrease in the wall aspect ratio terms. It makes the walls to be under high non safe stage due the fact that it will lead high rotating moments due lateral loads. Overload is observed to change in historic structures as a reasonable means for modifications. Hence, here in these models, it is found to have an increase in safety level, as is also obtained in chapter 3. Strengthening ties are introduced to the walls in positions of floor levels for purpose of in plane strengthening. It is found as important changing parameter, even though in this block limit analysis software there happened due local failure. Macro models are developed from the failure modes to help in the non linear kinematic analysis. Due time limit, it is not possible to have all the modified analysis results and their non linear kinematic analysis; this is the future continual work for this thesis.

CHAPTER 5

SUMMARY

Limit analysis depicts realistically the collapse and capacity of masonry structures. It constitutes a very reliable and powerful tool and should normally be used to assess masonry structures in combination with other possible tools. Limit analysis can be easily applied to arches and skeletal structures. Applications of limit analysis base up their concepts on the loads that case failure of the structure. Theories on the limit analysis are used possibly by reduced material input parameters and geometry simplifications. They are expressive in its analysis and makes problems in association to that come very simple and easy. Effects of parameters creating significant changes on the seismic coefficient are found. Wall aspect, unit aspect, size ratio of units, compressive stress, overload and bond types are the main parameters considered.

The wall aspect effect is one of those parameters. It is majorly important determinant parameter in case of walls with lower width values in relative to the reference wall. The change in safety factor is important at that point and it is found to change the safety factor by an amount of relatively 20%. Unit aspect ratio is also another issue on the determinacy. It is found that walls of ratios greater than or equal to the wall aspect ratios of 0.5 are generalized by having sliding type of failure for the same unit aspect ratio of 1/2. Regarding the size effect of units, the failure modes of walls for same size ratio but for different walls is found of two types: for long walls, the failure mode is an inclined crack formed at the far end of the earthquake direction and it will not be changed for an increase in the wall width, and for slender walls, cracks will be formed at the base. As units of masonry become small, the failure will have cracks in a distributed manner as multiple of cracks parallel to each other. The load effect is the other criteria considered for analysis. It is fact that walls will resist to vertical loads up to a certain limited maximum capacity. This is limited by the crushing capacity of stone masonry. It is the major thing to increase the safety factor. Vertical loads increase increases the lateral capacity as much the masonry strength can bear. Bond is another directionally determining parameter. Different bond types are analysed successively for

obtaining the clear variation in the seismic coefficient value, α. One important point concluded is that when the direction of the earthquake is reversed and analysed, for each bond ratio except for symmetric one, it is clear that the seismic values are not equal. It is seen that bond types give the direction for which one is better than the other reverse direction in lateral resistance. Since, earthquakes occur in both directions, the value with the lower will be determinant. Finally, compressive stress is analysed by taking as a multiple of the reference standard value of the models. As when it varies with increase, the safety factor reaches to its stability in value. It will be constant for very large values of compressive stress.

Masonry wall models used for reinvestigations in the seismic capacity of typical urban ancient buildings in Italy were analysed. Various results of numerical and experimental results are done on this area. Block limit analysis is done for similar models. Comparisons to the walls pier effect, overload, storey and strengthening ties is found. In pier effect analysis, generally, the increase in the piers has a significant increase in safety factor, this is true when the reference wall in the standard analysis case is already in very low safety level, and otherwise, if it is significantly safe, then the increase in the piers will not have a considerable change in the safety factor. Loads are generally main parts of the structure involving in the lateral capacity. Generally in terms of structural safety, the loads are increasing the safety against the lateral loads. This is also proved in the walls studied under Chapter 3. The other important aspect of study is storey effect. The safety factor decrease indicated the more dangerous situation which is an increase in the storey number. Ties are the frequently and most widely used strengthening techniques. Even though they are used for out of plane failure prevention, here they are applied to in plane strengthening. The local failure is the governing results for analysis and it gave more close values to the standard one. But the failure observed is nothing to do with the structure. The programme block limit analysis terminated its analysis by the local failure. The analysis took too much time and modifications to the results are the recommended future works for proceeding. Macro blocks are derived from the micro failure modes obtained and used for obtaining the capacity curve.

Hence, limit analysis combined with the macro-block approach obtained as a result from micro block failure mechanism are feasible tools for the estimation of seismic performance of masonry walls under in plane strength, thus allowing not using very demanding finite element procedures that need vast material knowledge, constitutive models and time.

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