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Increasing the seismic shear strength of informal masonry houses with FRCM

An urban disaster risk prevention program in Peru

L’aumento della resistenza a taglio in case informali in muratura con FRCM

Un programma di prevenzione di rischio urbano in Perù

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INTRODUCTION

Masonry is one of the oldest techniques that have characterized the history of construction. Not for this reason, however, we can claim to know it fully. In fact, it is still a source of study for engineers and scientists, as it seeks in particular to standardize the calculation procedures as well as it was done for reinforced concrete and steel.

Masonry has developed in a systematic way with the advent of the great urban civilizations and marks the transition from construction techniques related to wood and straw to switch to a more mature period of more durable and solid buildings.

Initially bearing walls were made of dry-laid, simply by placing one on the other hewn stones, trying to fix them in place in the best as possible, to achieve a good stability and bearing capacity.

Cyclopean walls or megalithic walls were built in prehistoric times with large irregular blocks, which were matched and compensating the gaps with smaller stones and mortar clay, trying to fill the cavities on the surface.

Fig.I.1 – Ancient ruins of Machu Picchu, built in the classical Inca style, with polished dry-stone walls

Compared to the buildings of reinforced concrete or steel, masonry technique is thought, erroneously, as inadequate for the construction of buildings, this due to problems caused by seismic events. In recent years, following a careful examination of various wall structures, there have been found several reasons that would have determine the inadequacy to seismic phenomena of those structures, such as low quality of materials, poor implementation of construction, bad design of the structure, lack of careful design, expansion and modification of the building without a careful study.

It is however a fact that buildings having a structure of bearing masonry and realized in more recent times in compliance with construction standards, together with masonry of the past that hold out till present days, give witness of the robustness and validity of the masonry. In fact all those buildings built according to standards, in which neither minor elements have been neglected, have withstood for centuries even to unforeseen actions, despite still missing the advanced theoretical knowledge and powerful computing means available to our availability.

In this context we introduce the problems which the country of Peru is affected by several years. In Peru, the brickwork is the most widely used for housing construction. Ease of construction using hand without computers or sophisticated machine tools, the economic cost of materials and flexibility of the construction process that allows do this in stages, has allowed hundreds of thousands of families have build their houses houses progressively according to their means. Thus the vast majority of houses on the outskirts of the urban area were built informally without the participation of professionals or the application of building regulations.

Fig. I.2 – Informal house condition after a seismic event

The brick masonry alone resists very well the gravity loads but when it is located in seismic areas such as Peru, requires additional reinforcements to adequately resist the horizontal loads of an earthquake. But informal houses do not have the backup system or these are poorly constructed and not fully comply with its function, making them vulnerable to seismic risk and high forces in the event of large magnitude earthquakes occur.

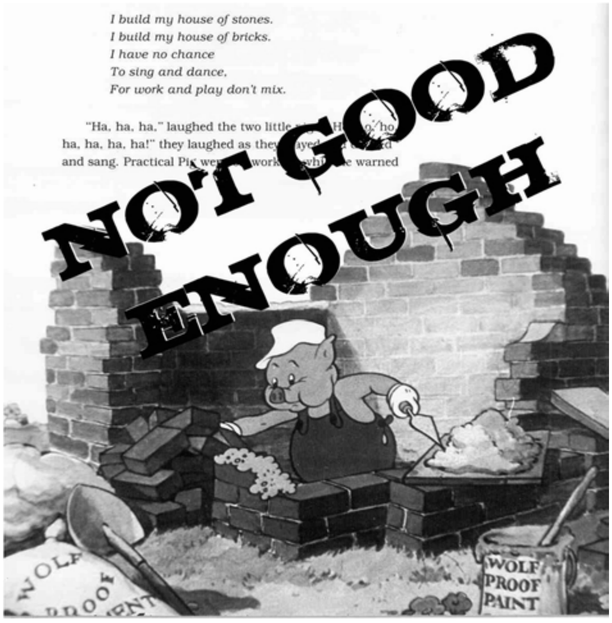


Fig. I.3 - Example of informal house construction

In this context, *Article 9 of Law N° 30191*, that establishes measures for the prevention, mitigation and adequate preparation for disaster response, provides a “Bonus for Protection of Vulnerable Housing Earthquake Risk” as part of the sectoral policy of the *Ministry of Housing, Construction and Sanitation* to reduce the vulnerability of the effects of the earthquake risk in favor of households in poverty, without charge to restitution by them, exclusively for interventions of structural reinforcement of houses located in soils vulnerable to seismic risk or that have been built in conditions of fragility.

The objectives of this project is to protect the lives of families living in poverty, reduce the material and personal damage caused by earthquakes, reinforcing a home environment.

In Peru the technique of seismic reinforcement most used is confining masonry walls with beams and columns of reinforced concrete, which have the function of maintaining the unity of the wall avoiding dislocations and maintaining its ability to withstand vertical loads after the wall has started to crack because of shear force in their plane.

So, thinking in a cheaper and more practical alternative to solve the problem of stability of houses subjected to earthquakes, it is proposed the polymer surface reinforcement. The surface reinforcement of masonry walls has been applied for several decades, being the most common the application of electro welded steel mesh embedded in mortar. In the last decade have been incorporated other materials of surface reinforcement as FRP, fibers reinforced polymer, applied with special resins, polymeric grid applied with a mortar, and other polymer materials embedded in resin or cement matrices. Polymer grids used as reinforcement for masonry walls appears in these days as a good alternative to the welded wire mesh. The use of polymer mesh has some advantages over the steel mesh, like been stable against chemical and biological agents, enhance the displacement capacity and is easy to install and adapt to any architectural shape. An additional advantage of this grid is that can be used in historical constructions since ancient plasters include lime and gypsum in the mortar and grids are not affected by its chemical components. The post elastic behaviour of reinforced walls subjected to in plane cyclic shear forces and out of plane bending is a key issue to investigate the change from a traditionally brittle seismic behaviour into an energy dissipation system that allows the masonry to successfully stand earthquake forces.

This study began more than a decade ago, initially with adobe and subsequently with bricks. Now studies are ongoing to ensure the effectiveness of the increase of strength of this material and to spread this method of seismic retrofit as an economical alternative than traditional solutions.

Chapter 1 – REINFORCED MASONRY

Brick masonry is the only construction material built up with the aid of gravity and during its all service remains dependent by gravity. The other traditional materials of construction like stone, wood and steel are fully independent by gravity. Indeed, gravity plays an initial role when the bricks as heavy units are successively one by one laid down on the soft mortar and when by so-called "sandwich effect" they are carefully arranged in precise directions of the horizontal planes. Then later, after the mortar was hardened, the bed layers of bricks are kept horizontal by the same gravity. Any deviation from those horizontal planes could be severely sanctioned by lateral deformations or even dislocations. The masonry remains the only construction material essentially devoted to gravity and always symmetrically balanced. Never and nowhere the masonry was primarily conceived and then deliberately used against lateral actions like the seismic ones. Problems occurs when is not only the gravity force that interact with masonry, but also other horizontal forces with relevant magnitude. In those case, the only solution is the aid of reinforcement.

1.1 – Masonry

Masonry is not at all a composite material, as sometimes is stated, because it does not fulfil the Principle of De Saint Venant regarding the geometric continuity of strains. Conceptually masonry could be best defined as an association of two different materials with complementary geometric, physical and mechanical properties. Indeed, the brick are elastic but brittle units while the mortars, as continuous binding materials, are plastic and ductile. Although in average the volume of mortar is less than 10% of masonry volume as a whole its vertical deformation under gravitational loads is almost 90% of masonry deformation.

1.1.1 - Masonry as the oldest construction material

At natural scale masonry is essentially non-homogeneous but due to the special arrangement of its horizontal layers it contains certain regularity. If virtually the bricks would be reduced a hundred time or more the masonry will appear to the same natural scale as a homogeneous material. However, even so reduced, masonry remains an artificial stone governed by gravity.

The consequence of masonry relation with gravity consist in its geometry. Masonry is first of all associated with the plumb wire. Then all masonry members and building are accordingly shaped. By shaping the buildings are balanced in the gravity field. Two intrinsic centres permanently control the equilibrium that of gravity C.G. and of rotation or stiffness C.R. Symmetry axes and planes are essential for masonry buildings. The primordial axis or so called axis mundi used during centuries for location and shaping a masonry building has physically also a gravity meaning.

Why this so simple construction material was lasting so long? It is still currently used like in ancient times for construction of buildings and structures. Really, one could speak about a secret of masonry? An answer to these two questions could be found by returning to the constitution of original masonry: elastic/porous brittle bricks and plastic ductile mortars. The elastic modulus of bricks is always higher than that of mortar. Together the two constitutive components form an essentially non-homogeneous construction material. As an artificial stone it is usually devoted to structural members submitted to vertical compressive actions.

When such members like columns or walls but of homogeneous materials are submitted to axial forces of compression then by virtue of Bernoulli's hypothesis the corresponding stresses are uniformly distributed on their cross sections. As long as the buckling is prevented the behaviour of structural members are changed to non-homogeneous ones then the uniform flow of compressive forces is disturbed and Bernoulli's hypothesis no longer subsists. Indeed around geometric imperfections and structural faults some stress concentrations occur. According to the Theory of dislocations for brittle materials when the level of stresses reach the strength of material then local damages like cracks or failures are generated. What is worse they are not only irreversible but generate new stress concentrations and the danger of collapse is extending.

In the case of masonry, the geometric imperfections and structural faults like the vertical joints between bricks cannot be avoided. Consequently, under the action of compressive forces around the corners of bricks stress concentrations occur. Then in the contact areas between the stressed bricks and bed layers in the lime mortar plastic deformations are slowly developing and a phenomenon of stress redistribution is spontaneously released. The danger of cracking some bricks or occurrence of any faults in masonry is automatically avoided. This mechanism of stress redistribution could repeat of countless times always when is necessary. It is a self-protection mechanism of masonry against long lasting actions and is usually called adaptation. The ductility of mortar is essential in producing the phenomenon of masonry adaption to the new actions. When forces of short time are acting up to a certain level the induced energy is stored by elastic bricks as potential energy. Then, by a slow relaxation it is gradually dissipated by the plastic deformations of mortar. This is why the idea of using lime ductile mortars for masonry with thoroughly burned bricks does not longer surprise. Te role of gravity to balance and control the whole process of adaptation is now clearly understood. Therefore masonry has no secrets but only qualities worth to be well known and thoroughly applied by professionals.

There are proofs that masonry was invented in a certain region scarce in natural stone of ancient Palestine. The original masonry consisted in bricks of burned clay and lime mortars. It was also used at the beginning for replacing adobe in construction of ziggurats and walls of defence. During the proto-historic age adobe was the basic material of construction. The core of 90m tower of the Etemenanki Ziggurat of Babylon c. 1100 BCE as well as the structural components of Urartu palaces and houses, the palace of Croesus, king of Lydia, and the monumental grave of Mausolus, king of Caria in Bodrum, were all built of adobe but externally protected with masonry of burnt clay bricks. Etemenanki, which means “the house where earth and heaven were set up”, also called Marduk Ziggurat, had a squared base of 91 m and was at times believed to be the biblical Tower of Babel according to an early form of city’s name. Ishtar Gate and throne room wall in glazed brick from Babylon, c. 575 BCE, are still preserved at Staatliche Museen in Berlin.

The binding propriety of lime seems to be discovered since the Neolithic age during the life in limestone caves. Neolithic communities knew to prepare mortar by ixing lime with sand and water. Lime mortars with volcanic tuff were found to have been used to the masonry walls constructed 2300 years ago in Bali Island, south of Java. It is reported that the Babylonians had currently used limekilns. Lime mortar was also used in Anatolia much before the Hellenistic age and also to the El-Tajin pyramid in Mexico. Therefore lime mortars played an important role in the development of ancient civilizations. Later, during Roman Empire, brick masonry based on lime mortars was extensively used, and some vestiges are nowadays still well preserved.

There are also three biblical information of particular interest. First comes from Genesis 11.3: And they said to one another, “Come, let’s make bricks, and burn them thoroughly.” And they had brick for stone, and bitumen for mortar. Therefore at that date, probably around 478 BCE, there had already existed an experience in stone and brick masonry with lime mortar. They did not use any longer adobe for Babel Tower or at least for its structural parts. It was important to burns thoroughly the bricks in order to achieve by their elastic proprieties high compressive strengths. The conquest of heights by human being in order to escape of gravity influence involves facing huge vertical compressive forces. Fortunately, the site of Babel Tower is located in a non-seismic area and no lateral actions should have to be added. But the fascinating news refers to bitumen, a native oxygenated hydrocarbon imported at that time from the Iranian plateau and widely used in the

Mesopotamian plain. Why bitumen was better for masonry than lime? Pure bitumen is essentially a viscous material; it freely flows under the gravity action. By mixing bitumen with sand rubble one obtains asphalt, a binding material without water like burnt clay bricks. In addition the asphalt used as mortar is able of plastic deformations what means nothing else but ductility. It seems that for masonry of perfect elastic bricks equally perfect plastic mortar was preferred. There are not proofs how bitumen was practically used for brick masonry but the concept is worth of high attention.

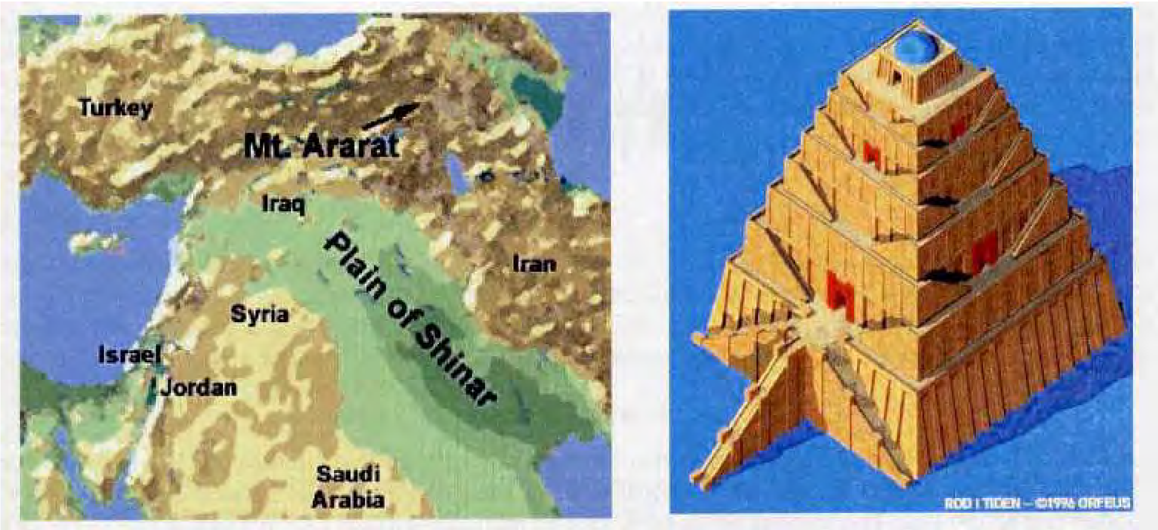


Fig. 1.1 - Plane of Shinar and the reconstructed model of Babel Tower

The second information comes from *Exodus 5, 7-9: You shall no longer give people straw to make brick as before. Let them go and gather straw for themselves. And you shall lay on them the quota of bricks that they made before. You shall not reduce it. For they are idle; therefore they cry out, saying, "Let us go and sacrifice to our God." Let more work be laid on men, that they may labour in it, and let them not regard false words.* Two ideas should be here consider. First is the knowledge of reinforcing adobe with straw. This simple but efficient method is still currently used in some Balkan countries and also in south of France. Later, by burning the units of clay with inserted straw, the porous bricks have been obtained. They were so appreciated during centuries due to their two qualities, elasticity and thermal isolation. Secondly, the reference to the quota of bricks is the recognition of their ergonomic role. Indeed, during manufacturing the bricks are handled in the gravity field. It depends by their individual dimension and weights how ong the process of production could last without breaking off for recovering the manpower. It should be mentioned that although a lot of labour have been then available the bricks never were cored any of the three reason: better burning, thermos isolation or ergonomics.

The third information from *Amos 7, 7-8: He showed me: Behold, the Lord was standing beside a wall built with a plumb line, with a plumb line in His hand. And the Lord said to me, "Amos, what do you see?" And I said, "A plumb line".* This statement explicitly recognises the close relation of masonry with gravity. With the aid of that simple device structural members of masonry have been shaped in the gravity field. That means geometry of gravity equilibrium based on axes and vertical planes of symmetry. These rules are also applied with the same rigor to masonry units, i.e. to bricks and their bed layers. The masonry as integer and the bricks as its parts are geometrically subordinated to the same vertical and constant flow of gravity. The high level of knowledge of that époque is impressive.

1.1.2 - Construction techniques of new masonry

Masonry is the building of structures from individual units laid in and bound together by mortar. Common materials of masonry construction are brick, building stone such as marble, granite, travertine, and limestone, cast stone, concrete block, glass block, and cob. Masonry is generally a highly durable form of construction. However, the materials used, the quality of the mortar and workmanship, and the pattern in which the units are assembled can significantly affect the durability of the overall masonry construction.

Fig. 1.2 - Type of binder of mortar

Fig. 1.3 - Type of units of masonry

Construction techniques of modern masonry can be subdivided in 3 types:

- **Unreinforced masonry**

  This type of technique is the easiest techniques used since the Ziggurat temples at Eridu, which have withstood not only earthquakes but also wars and invasions. From Roman aqueducts and public buildings to the Great Wall of China, from the domes of Islamic architecture to the early railway arch bridges, from the first 19th century American tall buildings to the 20th century nuclear power plants, bricks have been used as structural material in all applications of building and civil engineering.

  The most commonplace use of bricks worldwide throughout time is in residential dwellings. The shape and size of bricks can vary considerably, and similarly the mortars used depend on local material availability, but the basic form of construction for houses has minor geographical variations and has changed relatively little over time.

Fig. 1.4 - One and four story unreinforced brick masonry

In more recent times, seismic codes place substantial constraints on unreinforced brick masonry construction in earthquake-prone areas, limiting the allowed number of stories, the minimum thickness of walls, and the number and position of openings. As a result, construction of load-bearing unreinforced brick masonry structures has dwindled in these countries, and alternative forms of construction such as confined masonry or reinforced masonry, considered less vulnerable, have been developed instead.

Brickwork is an assembly of brick units bonded together with mortar. While brick size can vary considerably depending on the quality of the clay and the manufacturing tradition, the basic firing technology is common worldwide. Variations in kiln typology are very limited. The major factors influencing the strength of the bricks are the purity of the clay and the firing temperature. Mortars are subject to greater variation, but the basic materials used in mortar mixes are sand, water, and one or more of the bonding agents, mud, clay, or cement, depending on local availability. The proportion of bonding agent/s to sand determines the compressive and bonding strength of the mortar. In earthquake-prone areas, the development of an effective level of bonding between mortar and bricks is essential to resist shear-cracking. Bricks might be frogged or specially shaped to create mechanical interlocking and improve bonding. Brick construction is relatively simple and cheap. In certain cases bricklaying may require highly skilled labour; however, this type of construction is usually performed by small to very small building contractors or as self-built construction.

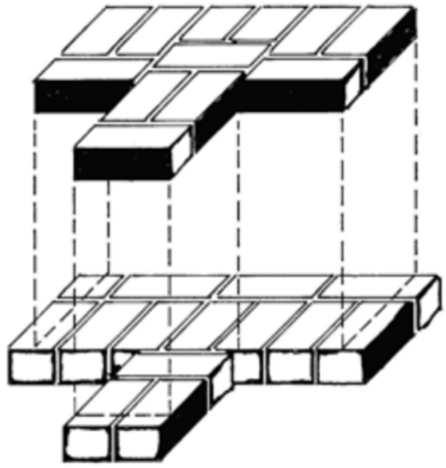


Fig. 1.5 - Details of bonding arrangement for masonry units at wall junction

From an architectural point of view, brick construction is rather flexible, allowing substantial freedom in the layout of internal spaces and the distribution of openings, making it quite adaptable to different climatic conditions.

From an environmental and structural point of view, masonry performance depends on the performance of mortar and brick units, and their composite behaviour.

Modern building codes provide guidelines for the preferred combinations of mortar mixes and brick units in order to optimize both the strength and the environmental performance of the wall assemblages made of these components.

The structural performance of brick masonry buildings depends on the following four types of connections within masonry elements:

1. Integrity and shear resistance of brick masonry walls is influenced by the extent and quality of bond between mortar and bricks. It is essential for the brickwork to be properly constructed to allow for the best possible level of bonding to develop. It is also important to ensure repointing of bed and head joints at regular time intervals so as to ensure the maximum possible surface of contact.
2. The second level of connection is among the wythes of brick walls. Modern masonry construction standards require regularly spaced ties between the wythes of a cavity wall to ensure monolithic behaviour and redistribution between the wall wythes. In historic masonry construction it is common for the walls to be either one- or two-brick-wide solid brick, or to consist of two external wythes with a cavity filled with rubble (to improve the thermal capacity of the wall). The connection between the

two wythes was ensured by headers, bricks placed through the wall at regular intervals.

3. The third level of connection is among the walls at the corners and junctions and depends on the specific fabric of corner returns. Such connections ensure 3D behaviour of the masonry box-like structure and the redistribution of lateral forces among walls.
4. The forth level of connection is between the walls and the horizontal structures (floors and roof); this connection highly influences the seismic performance of the building.

Main types of unreinforced masonry related to horizontal joints (EC6 classification) are:

a) Ordinary horizontal bed joint
b) Shell bedded masonry
c) Thin layer masonry

Main types of unreinforced masonry related to vertical joints (EC6-EC8 classification) are:

a) Filled vertical joint
d) Unfilled vertical joint
e) Tongue and groove unfilled vertical joint

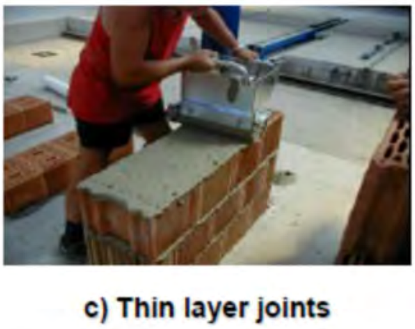


Fig. 1.6 - Details of unreinforced related to horizontal joints

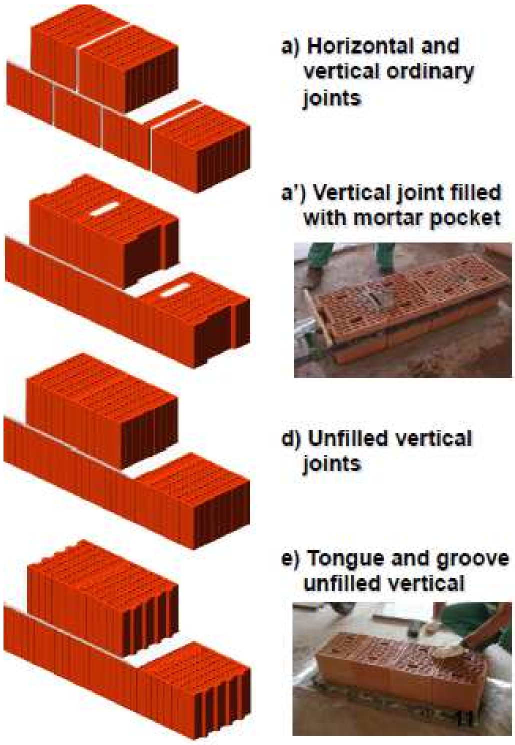


Fig. 1.7 - Types of unreinforced masonry related to vertical joints

In proportion to its widespread presence worldwide, there are many examples of brick masonry performance in past earthquakes. The extent of damage depends on the seismic hazard and the earthquake intensity at a particular site.

Common damage patterns found in World Housing Encyclopedia reports include the following:

- Collapse of chimneys and plaster cracks (MMI intensity VII)
- Shear cracks in the walls, mainly starting from corners of openings (MMI intensity VIII)
- Partial or complete out-of-plane wall collapse due to lack of wall-to-wall anchorage and wall-to-roof anchorage. In extreme cases this is accompanied by partial or total collapse of floor and roof structures (MMI intensity VIII-IX)
- Total collapse of walls and entire buildings in some cases (MMI intensity X), for example, 2001 Bhuj (India) earthquake

Evidence from recent earthquakes has confirmed that the overall performance of brick masonry buildings is dependent on the type of roof system: buildings with lightweight roofs suffered relatively less damage while buildings with reinforced concrete roofs suffered much greater damage. This performance was observed after the 2001 Bhuj (India) earthquake (M7.7), where brick buildings in the epicentral area (MMI intensity X) were surveyed (IIT Powai 2001). This is in line with the evidence collected after the 1997 Umbria-Marche (Italy) earthquake (MMI VIII), where many buildings with heavy reinforced concrete roofs suffered substantial structural damage and partial collapse.

Fig. 1.8- Shear cracks in an unreinforced brick masonry building from the 1993 Killari earthquake

Fig. 1.9 - Collapse of brick masonry buildings in the 1988 Spitak earthquake

- **Reinforced masonry**

  The reinforcement of masonry is not a new concept. In the 18th Century external iron straps were commonly used in stonework. It was not until 1825 that the first use of reinforced brickwork was recorded. Sir Marc Brunel used the technique in the construction of two caissons, one either side of the River Thames for the Wapping—Rotherhithe Tunnel. The diameter of each caisson was 50 ft. (about 15 m) and they were 42 ft. (about 14 m) and 70 ft. (21 m) deep respectively. The walls consisted of two leaves of 9 in. (about 23 cm) brickwork reinforced horizontally by iron hoops 9 in. wide and ½ in. (1.25 cm) thick and vertically by 1 in. (2.5 cm) diameter wrought iron bars. Brunel was impressed by the structural performance of reinforced masonry and during the period 1836—1838 he carried out experiments on reinforced brickwork beams and cantilevers. The most important of these tests was the "Nine Elms" beam which had a clear span on 21 ft. 4 in. (about 6.5 m), which is shown in Figure 1.10. Tensile failure of the reinforcement occurred at a load of approximately 30 ton f. It is interesting to note that this work predates the development of both Portland cement and reinforced concrete. There were few other significant uses of reinforced masonry in the 19th Century, with the exception of a 100 ft. (about 30 m) diameter 35 ft. (about 11 m) high reservoir built in Georgetown, USA, in 1853. This was used until 1897 and was eventually demolished in 1932.

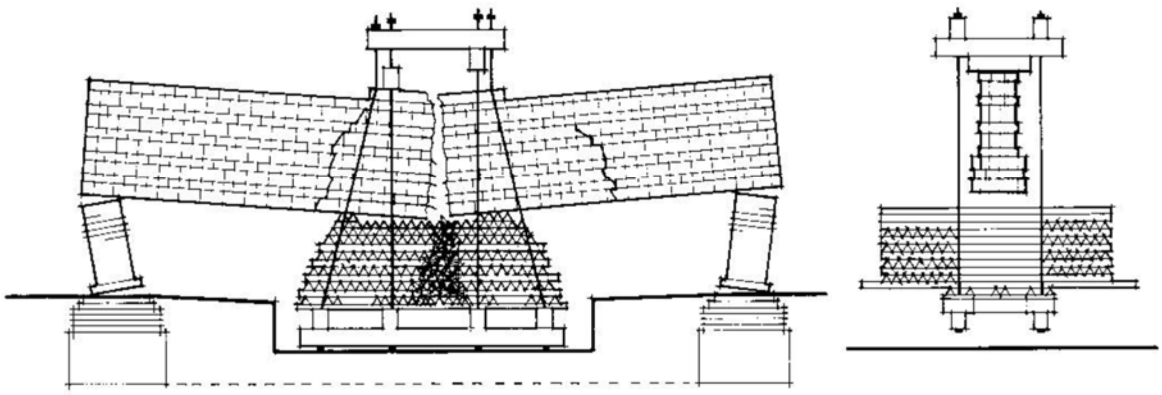


Fig. 1.10 - Nine Elm beam test in 1838

Reinforced masonry is any type of brick, concrete or other type of masonry that is strengthened or fortified with the use of other building materials to increase resistance to deterioration due to weight bearing or other forms of stress. The actual design of reinforced masonry structures will vary, with some designs calling for the inclusion of steel rods in the construction, or filling hollow masonry units such as cement blocks with additional concrete. With any type of masonry reinforcement, the goal is to create masonry that is capable of withstanding additional exposure to the elements and other factors that could weaken the overall structure and cause it to fail.

One of the most common examples of reinforced masonry involves exterior walls that are created using concrete blocks or clay bricks. Along with the blocks or bricks, steel rods are worked into the structure, often using some type of vertical framework that aids in allowing the walls to bear up under its own weight, and the weight of the connecting walls and floors within the building. When concrete blocks are used, it is not unusual for the rods to be woven through the openings of the hollow blocks, then fill in the cavities with the use of additional concrete. The end result is a wall that is sturdy and capable of withstanding a great deal of stress for a number of decades.

Main types of reinforced masonry, according to Eurocode 6 classification, in relation to arrangement of reinforcement are:

a) Concrete units with pre-formed cavities that will be filled afterward
b) Vertically and horizontally reinforced wall
c) Cavity wall with concrete infill
d) Units for horizontal reinforcement in bed joints

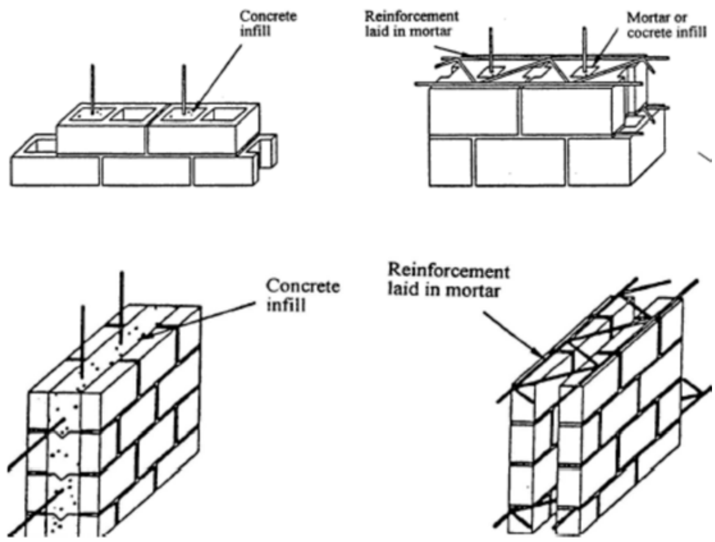


Fig. 1.11 - Types of reinforced masonry

Along with providing additional strength to the overall structure, reinforced masonry also provides the benefit of blocking out noise with greater efficiency than some other construction options. This can be especially important for business offices and similar operations that require a minimum of distraction from the outside world. By using the reinforced masonry for the exterior of the building, it is possible to lower the costs of soundproofing the rooms or chambers within the building, focusing more on preventing the transmission of sound from one room to another and less on minimizing the intrusion of noise from the outside.

- **Confined masonry**

Confined masonry construction consists of unreinforced masonry walls confined with reinforced concrete (RC) tie-columns and RC tie beams. This type of construction is used both in urban and rural areas, either for single-family residential construction or for multifamily construction up to four or five stories in height.

Fig. 1.12 - Confined masonry housing

Confined masonry housing construction is practiced in several countries that are located in regions of high seismic risk. The use of this type of construction in various countries in South America is very extensive and in most countries, it has been practiced in the last 30 to 35 years.

The tie-columns and tie beams provide confinement in the plane of the walls and also reduce out-of-plane bending effects in the walls. The walls are made of different masonry units, ranging from hollow clay or hollow concrete blocks to solid masonry units of either clay or concrete.

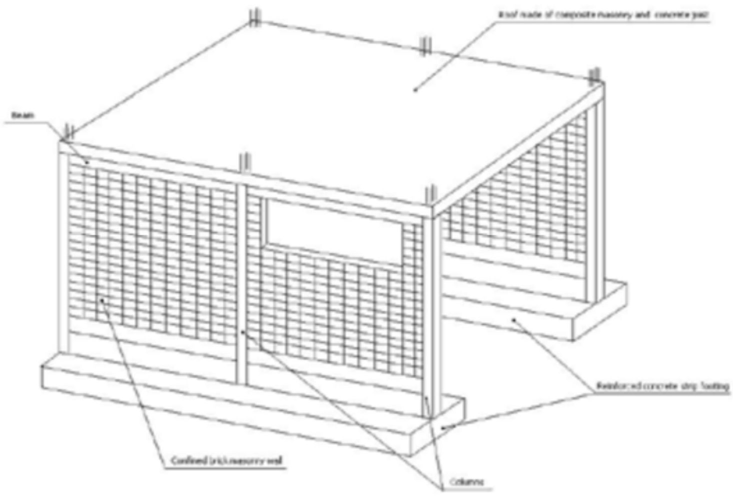


Fig. 1.13 - Key structural components in confined masonry construction

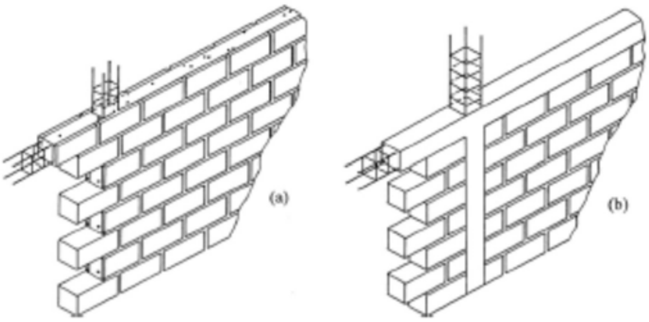


Fig. 1.14 - Difference of reinforced masonry confining elements (a) and masonry provided with reinforced concrete (b)

A very important feature of confined masonry is that tie-columns are cast-in-place after the masonry wall construction has been completed. Alternatively, tie-columns are constructed using hollow masonry blocks to allow for placement of vertical reinforcement and cement-based grout. Figure 1.15 shows an example of the use of both types of tie-columns in residential construction. It can be observed that cast-in-place RC tie-columns are constructed at the first level, whereas hollow-block tie-columns are constructed at the second level. Both tie-columns and tie beams have ties that provide confinement to the longitudinal reinforcement and help prevent the buckling of reinforcement. Typically, tie-columns consist of a rectangular section, with cross-sectional dimensions corresponding to the wall thickness, usually within the range of 150 to 200 mm.

Fig. 1.15 - Confined masonry housing during construction

The floor system in confined masonry construction typically consists of composite masonry and concrete joists or cast-in-place concrete slabs. In some cases, timber roofs are used in combination with tie beams as shown in Figure 1.15.

If properly constructed, confined masonry construction is expected to show satisfactory performance in earthquakes. The adverse behaviour observed in past earthquakes involved houses that were built without tie-columns and/or tie beams, with inadequate roof-to-wall connection, or with poor-quality materials and construction. Major earthquakes that have affected confined masonry construction include the 1985 Llolleo, Chile, earthquake (M 7.8) and the 1990 Manjil, Iran, earthquake (M 7.6). Confined masonry buildings suffered light damage in the 1985 Lloleo earthquake and collapse was not reported. In most of the damaged buildings, tie-columns were missing and the following characteristic damage patterns were observed:

- Shear cracks in walls that propagate into the tie-columns; most cracks passed through mortar joints. Also, crushing of masonry units has been observed in the middle portion of the walls subjected to maximum stresses.
- Horizontal cracks at the joints between masonry walls and reinforced concrete floors or foundations.
- Cracks in window piers and walls due to out-of-plane action in inadequately confined walls.
- Crushing of concrete at the joints between vertical tie-columns and horizontal tie beams when the reinforcement was not properly anchored.

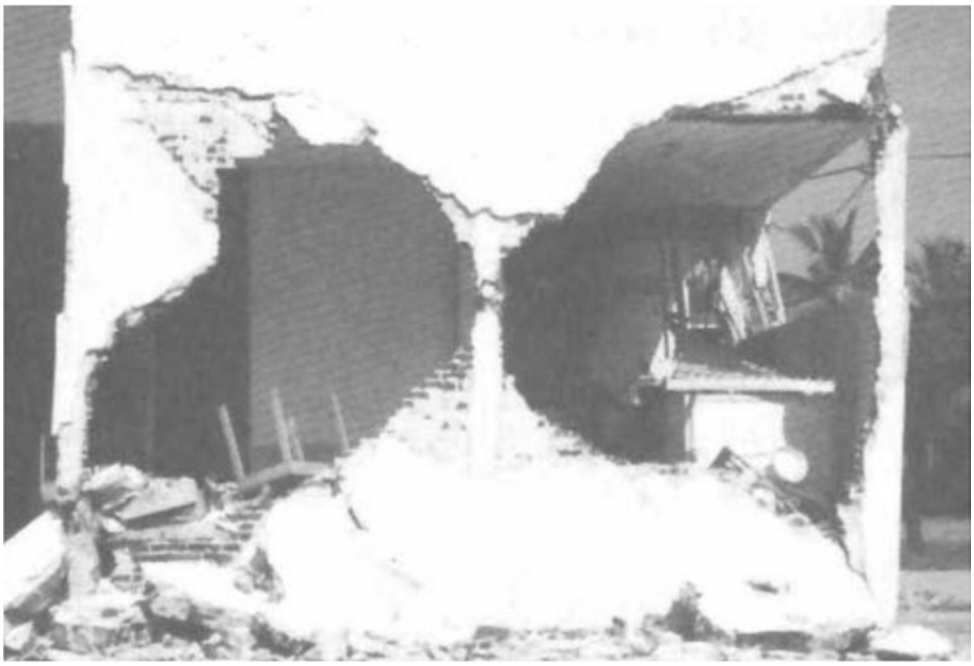


Fig. 1.16 - Damage to confined masonry construction in the 1985 Llolleo, Chile Earthquake

Seismic features that were found to improve the performance of confined masonry in past earthquakes include a minimum wall density, as well as a symmetric and regular wall layout, both in plan and elevation, as prescribed by NTE- 070 and Eurocode 6.

Generally, the main characteristic of masonry is the good compressive strength, but in the other hand it has a poor tensile strength, for interface between unit and mortar may be 1/30 of compressive strength. It means that the main problems are with the resistance to horizontal actions, like wind and earthquake, but the walls have a good resistance to actions acting in their plane (in-plane), much higher than resistance orthogonal to their plane (out of plane).

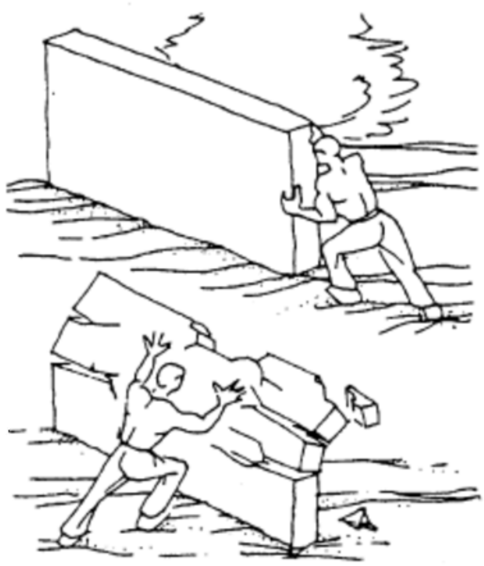


Fig. 1.17 - Representation of masonry behaviour walls

The major hypothesis for the application of seismic assessment procedures based on the in-plane response of the walls, when using performance-based approaches for seismic safety, is the “box” behaviour of masonry structure.

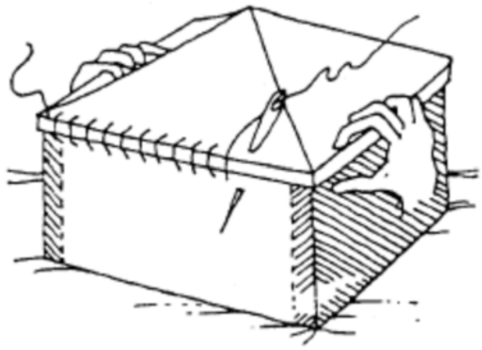


Fig. 1.18 - Representation of the masonry “box” behaviour

To ensure the “box” behaviour, walls and horizontal elements must be suitably connected together. All the walls must be connected to the level of the floors by means of reinforced concrete ring-beams and suitably joined together along the vertical intersections. Appropriate straps should also be provided to the level of the floors, which serve to connect the parallel walls of the “box” walls.

The load-bearing walls are considered to be resistant to horizontal actions when they have a length of not less than 0.3 times the interstory height, and must have a minimum thickness, indicated in each country’s Code.

The good structural design and proper implementation of the structural details provide an appropriate structural behaviour.

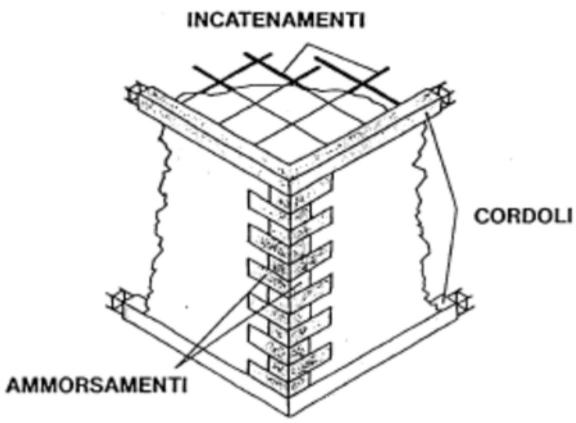


Fig. 1.19 - Example of a good connection between walls and horizontal elements

Masonry has been continually used for building, employing very different materials and bond patterns, but the main distinction between masonry constructions probably is the presence or absence of rigid floor diaphragms well connected to the walls. According to this aspect, masonry construction can be divided in two categories: structures with and without "box" behaviour, which present a very dissimilar response when subjected to seismic actions. For the case without "box" behaviour, the walls of the building behave independently and out of phase, with combined in-plane and out-of-plane deformations, and presenting mainly out-of-plane damage. On the other hand, when with "box" behaviour the building acts as jointly assemblage of walls and roof, with mainly in-plane response of the walls.

The concept of box behaviour was a key assumption of methods developed for the seismic analysis of masonry structures.

1.1.3 - Problems of masonry components

1.1.3.1 - Brittleness of cement mortars

After the Industrial Revolution in 18th century, cement started to be used in mortars. It partially or totally replaced lime in order to increase the bearing capacity of masonry in the new buildings. For the same purpose ceramic bricks produced since then industrially replaced the original light and porous bricks produced before manually. The success immediately was shown and masonry started to be used even for erecting tall buildings.

However, after a while in some masonry walls, due to unequal settlements, vertical cracks occurred. The masonry based on cement mortars, even when some lime was added, proved to be too brittle. Even reinforcing mortar with steel bars could not solve the conflict between masonry brittleness and the demands for increasing its bearing capacity. Masonry based on cement mortar is no longer ductile like the original one with lime mortar. Therefore, the self-protection of masonry against heavy loads by adaptation cannot be expected, and any other structural measures are either unsafe or extremely expensive.

Then, later studies discovered the incompatibility between the bricks produces by burning the clay to eliminate its water, and cement mortar hardened by its added water. The consequences of this antagonism between masonry constituents appears in its reduced capacity of thermal isolation and lack of comfort. The strong tradition of the original masonry is diminishing and alternative solutions were successively promoted on market. For example mortar can be reinforced with fibers of different materials (glass fiber, carbon fiber, geo-nets polypropylene, geo-nets polyester), increasing the ductility of masonry and increasing the energy dissipation, improving seismic behaviour of masonry buildings.

Fig. 1.20 – Brittleness of mortar

1.1.3.2 - Brittleness of cored bricks

One of the alternative solutions for masonry as mentioned above is based on cored bricks. These ceramic units were also produced after the Industrial Revolution. They are easily handled and have high productivity. A masonry building with such bricks can be erected in several hours instead of days or even weeks as before. But it does not last as long as the masonry with solid bricks.

Cored bricks are thin walled units extremely brittle. They are produced by burning the clay to so high temperatures that its vitrification point is reached. This is why when knocked these bricks utter metallic sound. Sometimes, due to non-uniform burning, they are produced with initial thermal cracks. Once included in masonry and loaded they act from the very beginning as stress concentrators on the reduced areas of contact between bricks units and mortar layers. They have no capacity to store the elastic energy of deformation. Under the smallest overload the cored bricks are either cracked or crushed. Palestinians and later Romans built much in masonry but they never perforated the bricks although plenty of labour was then available.

Cement mortar also similarly brittle has its responsibility in damaging the masonry with cored bricks. There is a practice accepted by all codes of filling the holes with mortar. Then the natural question is why the holes have been however provided in bricks? In what consist in fact the so much praised thermal isolation of masonry with cored bricks if cement mortar was used as filling material of holes? There is something preposterous is all these techniques.

According to another official approach, the masonry should be reinforced with steel bars inserted through the holes of cored bricks. The question is how and why to reinforce with steel of high strength a weak material like masonry with cored bricks? In fact by reinforcing the masonry with steel bars a parallel structural member of RC is created and of course its additional cost is included.

Finally, the damaged cored bricks in either plain or reinforced masonry cannot be repair any longer but should be replaced at the corresponding cost. It is not the case of masonry with solid bricks. The old saying that masonry building are lasting one thousand years is no valid for cored bricks. The main cause of damages in masonry without capacity of adaptation comes from unequal settlements. Nowadays, there where terrorist attacks could happen, living in masonry buildings with cored bricks became a danger. Under the action of strong explosions they have no capacity to store the included energy by successive elastic deformations and suddenly could fell down and collapse.

Fig.1.21 – Brittleness of cored bricks

1.1.4 - Stress concentration around structural faults

When a structural member of homogeneous material is submitted by a concentrated force P to axial compression then to normal stresses σ, developed according to Bernoulli's law

$$\sigma = \frac{P}{A}$$

where A is the area of cross section, are uniformly distributed. The magnitude of normal stresses is inversely proportional with the area A and therefore the stresses vary according to Bernoulli's hyperbola what means for large areas the stresses are small while for small areas the stresses remain uniformly distribute but become very high.

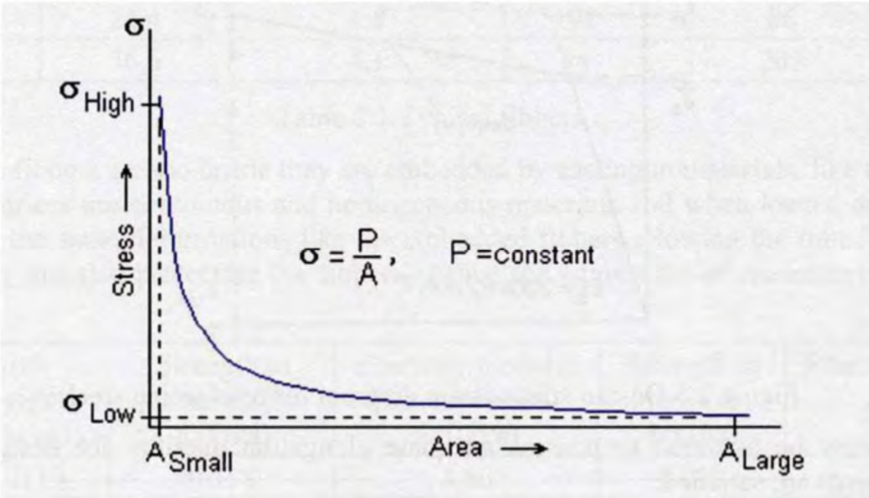


Fig. 1.22 - Bernoulli's hyperbola

When in the above considered member a structural fault is met the stresses are no longer uniformly distributed and around it they strongly concentrate. Under the peak of stresses, the strains are easily reaching their limit of elasticity. Further the material in the structural member is either cracking or plastically deforming. The process of stress transfer is energetically governed: the potential and dissipated energies are balancing the induced energy.

In the case of masonry the structural faults consist in he vertical joints between bricks. This is the real reason they are alternated from one layer to another by the so-called joint bond system. By the regular décalage of vertical joints the distribution of peak stresses is kept to constant and rather uniform levels. When is masonry the traditional solid bricks are used the levels reached by maximum stresses remain under acceptable values. On the contrary, ceramic cored bricks with their thin walls increase the values of maximum stresses from simple to double. Cement mortars make the things worse. Being brittle the mortars in bed layers easily crack, and the redistributed stresses are concentrating around any existing structural faults. After cracking the stresses are redistributing and concentrate around any other existing structural faults, cracks including.

Equilibrium and stability of cracks are extensively studied by Theory of dislocations using the vectors of Burgers. For the particular case of a symmetric crack of 2L length developed in a continuous and isotropic medium with an elasticity E under a uniformly distributed tension p_0, it was found that

$$2L = \frac{4\alpha E}{\pi(1-\mu^2){p_0}^2}$$

where μ is the coefficient of Poisson, α the force of superficial tension in N/m

$$\alpha = \frac{Eh^3}{24\,(1-\mu^2)}k^2$$

h the thickness of bent layer and k its curvature. For the extreme values of the tension p_0 the relation contains the geometric instability of the crack. It is due to the assumption of perfect elasticity and brittleness of the medium. In reality the plastic deformations developing at the ends of crack limits its extension.

1.2 - Reinforcing materials

There are various methods of retrofitting URM structures in different categories, and some of them are under research and being experimented. Application of these methods to URM structures is expected to increase strength and ductility of the structure. However, sometimes the cost of retrofitting is not reasonable, or advanced technology is needed and therefore isn't suitable for developing countries (that need to retrofit buildings), especially in rural regions. The most suitable methods for retrofitting of URM brick walls are introduced below.

1.2.1 - Metallic bars and nets

The reinforcements are usually, though not necessarily, in bars and nets. According to EC6 the reinforcing steel used for masonry may be carbon steel or austenitic stainless steel. Some standard masonry units, like blocks and bricks, are made with voids to accommodate rebar, which is then secured in place with grout. This combination is known as reinforced masonry.

This material is defined by strength, ductility and durability. The symbols used to describe the strength of the bar are f_{tk}, tensile strength, and f_{yk}, yield strength.

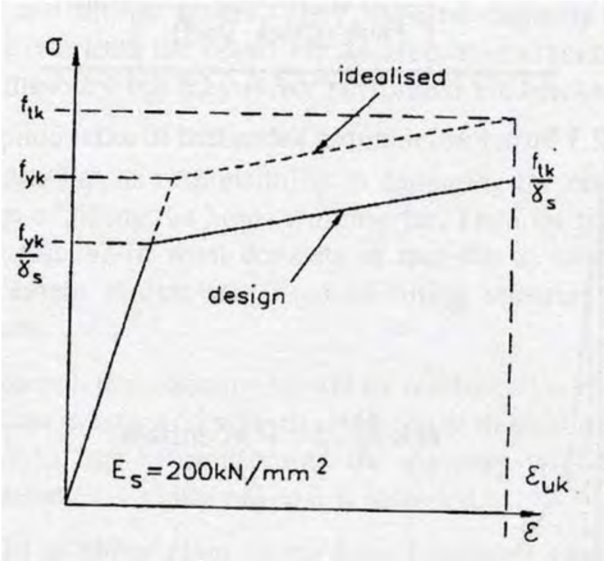


Fig. 1.23 - Design stress-strain diagram fro reinforcing steel

Reinforced steel may be assumed to possess adequate elongation ductility for design purposes if:

$\varepsilon_{uk} > 5\%$ and $f_{tk}/f_{yk} > 1.08$ (for high ductility)

$\varepsilon_{uk} > 2.5\%$ and $f_{tk}/f_{yk} > 1.05$ (for normal ductility)

where ε_{uk} is the characteristic value of the unit elongation at maximum tensile stress. The mean value of the modulus of elasticity of reinforcing steel may be assumed to be 200 GPa.

Reinforcing steel shall be sufficiently durable resist local exposure conditions for the intended life of the building. Where carbon steel requires protection to provide adequate durability, it should be galvanized such that the zinc coating is not less than that of required to provide the necessary durability or the steel should be given an equivalent protection such as by fusion bonded epoxy powder.

There are five exposure classes to environmental conditions:

I. Class 1, dry environment
II. Class 2, humid environment
III. Class 3, humid environment with members exposed to frost
IV. Class 4, seawater environment
V. Class 5, aggressive chemical environment

Reinforcing steel shall be adequately protected against corrosion by two measures: the first is minimum thickness of concrete cover from 20 mm for Class1 to 40 mm for Class 5, the second is the cement content not less than 260 kg/m^3 for Class 1 and 300 kg/m^3 for Class 5.

Essential for the mechanism of stress transfer from cement mortar or concrete to reinforcing steel is the clamping effect. It is obtained by the shrinkage of cement gel and cement stone around the circular bars of reinforcing steel and is lasting due to the same coefficient of thermal expansion for both steel and concrete. This is why steel bars should be well and continuously covered by cement mortar or concrete. Lime or other additional material are not allowed in the mixture not only because they are exposing the reinforcing steel to corrosion but mainly due to diminishing the clamping effect.

Eurocode 6 provides rules for the minimum area of reinforcement, size of reinforcement, anchorage, lapping and spacing of reinforcement. They are not much different by those used for reinforced concrete, in fact masonry is not directly reinforced by steel bars but only partially and indirectly. Reinforcing steel is used only for cement mortar in bed joints, concrete infills or plasters, and for that reason such reinforced masonry appears like a more non homogeneous reinforced concrete.

1.2.2 - Carbon and glass fibbers

Fibre-reinforced polymers are is a composite material made of a polymer matrix reinforced with fibbers. The fibres are usually glass, carbon, aramid or basalt. The polymer is usually an epoxy, vinylester or polyester thermosetting plastic.

This composite material is product of advanced technologies with outstandingly high strength in tension. Glass fibbers are several thousands time stronger than ordinary glass because in fibber the molecules are perfectly aligned and there are not structural faults. Geometrical and mechanical proprieties of fibbers are extremely attractive for practical applications.

Fibber diameter	Unit weight γ (kN/m³)	Strength to tension f (GPa)	Ratio f/γ (km)	Elasticity modulus E (GPa)	Ratio E/γ (Mm)
Carbon 5-10 μ	13.8	1.7	123	190	14
Glass 5-15 μ	24.4	4.8	197	86	3.5
Steel 12.7 μ	76.6	4.1	54	207	2.7

Tab. 1.1- Typically fibbers

Matrices are continuous and homogeneous materials and when loaded due to the clamping effect they develop the same deformations like the embedded fibbers allowing the transfer of stresses from and to fibbers. They are also protecting the fibbers against the aggression of environmental factors.

Matrix material	Unit weight γ (kN/m³)	Strength to compression f_c (GPa)	Elasticity modulus E_c (GPa)	Strength to tension f_t (GPa)	Elasticity modulus to tension E_t (GPa)
Epoxy	0.0119	0.158	3.86	0.029	3.38
Polyester	0.0125	0.140	-	0.055	3.50

Tab. 1.2- Usual matrices

Fibbers are created by the advanced technologies of the last several decades. Materials based on the combination between fibbers and matrices, obeying the Principle of De Saint Venant of geometric continuity of strain, are well known as composite material. They are able to reach remarkable mechanical performances, much higher than those of any existing non-composite material, ad in most cases at very convenient cost.

However, the idea of composing materials with different mechanical properties was often used during the long history of mankind. Masonry for instance is not a composite but instead an association of two materials with similar mechanical properties: brittle ceramic bricks and brittle cement mortars. The genuine composite of special interest is the reinforced concrete, which was patented in 1867 by Joseph Monier, and from then remained the basic material of modern civilization. Particularly, reinforced concrete is used for reinforcing masonry and there are special code provisions supporting the concept.

Many attempts of using fibbers in different composites for reinforcing masonry have been carried out so far in Europe, Japan and Usa. Unfortunately, the main problem of masonry, namely that of brittleness, remained unsolved. In the ultimate limit state the brittle masonry is crushed while the brittle fibbers are torn and the failure mechanism suddenly occur without any warning what is against safety principles. It is preposterous to fight against brittleness with essentially brittle materials.

1.2.3 - Polymer grids

Polymer grids are made from sheets of heavy-gauge polymer. In a patented manufacturing process, these sheets are perforated and stretched under controlled temperature and rates of strain. During the lengthwise stretching operation the punched holes become slots and long-chain molecules in the polymer align, increasing tensile strength. Stretching the elongated product in the transverse direction produces polymer grids with rectangular openings. The monoaxial grid such produced has well determined dimensions and strength, about the same strength as mild steel. It can be further used to provide feedstock for subsequent transverse stretching into a biaxial grid with squared apertures.

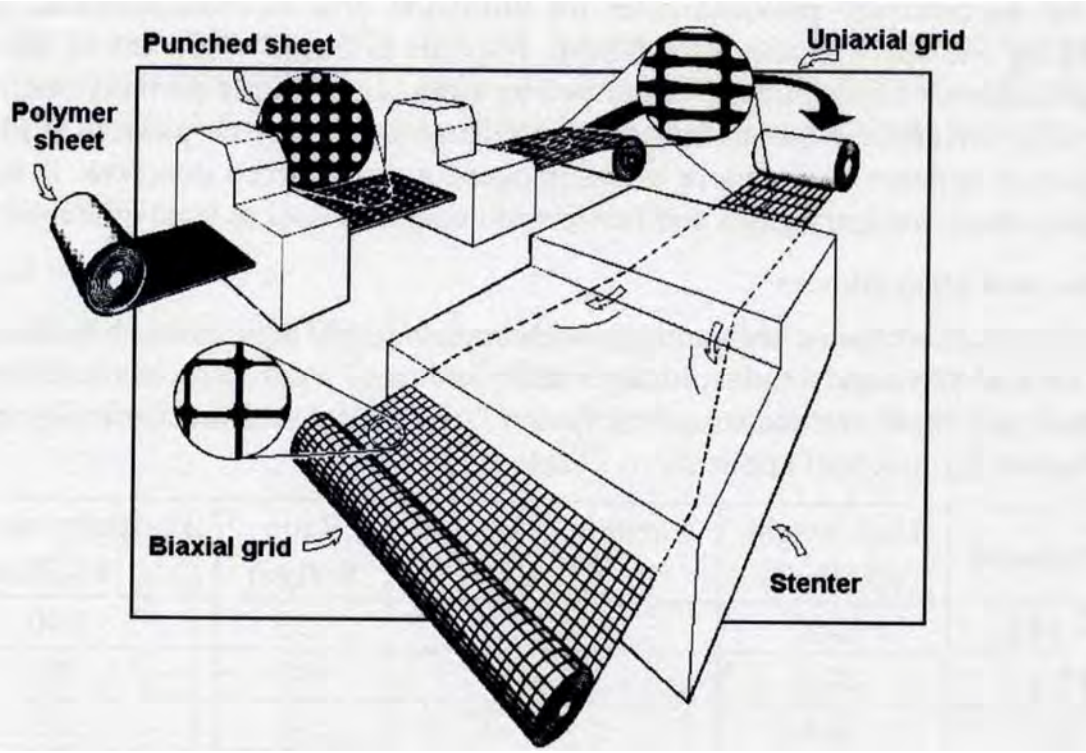


Fig. 1.24 - Tensar process

1.2.3.1 - Grid shaping

The process of stretching with an axial force N produces into tension direction according to the law of equilibrium

$$N = \int_A \sigma dA$$

Technologically it is assumed that normal stress σ is uniformly distributed on the area of cross section A. In these conditions by integration from equation the well-known Bernoulli’s formula

$$\sigma = \frac{N}{A} < f$$

is obtained, where f is the strength to tension in Pa. Mathematically the three factors in that formula could assume any values. However, if one of them is fixed then between the other two certain recurrence relations are establishing. For example, if A is constant then σ and N are proportional; if σ is constant then N and A are proportional; finally, if N is constant then σ and A are inversely proportional, namely in the cross sections with large areas the normal stresses of tension are reduced and inversely.

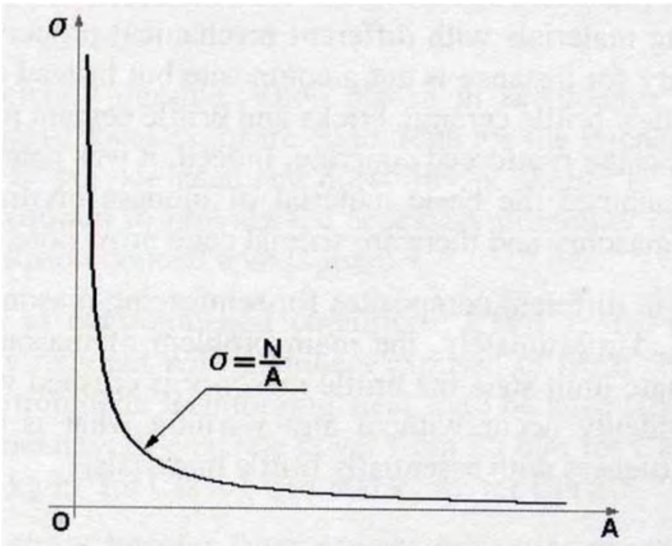


Fig. 1.25 - Bernoulli’s equilateral hyperbola

Stretching of the punched sheet or monoaxial grid o a force N constant is technologically the most convenient method. But the greatest advantage of Tensar process consists in the macromolecular structure of linear polymer chosen as stock. Indeed, by rolling the raw material of polyethylene or polypropylene between two steel cylinders, the sheet was strongly compacted and reached an advanced density. However, in the plane of sheet the long, the rolling process has not yet oriented sticks like macromolecules. The directions of cuneiform molecules of polymers remained randomly like in the original mixture. Only by stretching the macromolecules started to be aligned and oriented in the direction of tension. The intermolecular spaces have been drastically reduced and so the density further increased. In the same time the strength to tension f of cross sections became variable along the stretched bars. The strength f is strongly increasing by cross section reduction according to relation Af= constant which expresses the law of bars with constant bearing capacity to axial tension. This is an ideal mechanical performance and was obtained by the most economic solution. If is taken into consideration that stretched bars have thickness t constant and their cross sections are rectangular then A variable means width b variable and result $p=bf$= constant thus defining the constancy of tension flow in N/m. This remarkable property was not yet identified to any other metallic or synthetic material used as reinforcement. This is why the producer Tensar defines the grids by their tension flows p called Quality Control Strengths. For instance *Tensar SS20* means $p=20$ kN/m.

On the other hand, the polymer grids have been shaped according to the same law of tension flow. It is a kind of natural or self-shaping process. Indeed, by successive congruence or mirroring of Bernoulli's hyperbola against the two orthogonal axes σ-A the shape of an integrated joint is obtained. Then, by further mirroring of such shaped joints according to certain orthogonal directions conveniently chosen, operation that in mathematics is called isomorphism, the mono or biaxial grids of any length and width are generated. These grids appear in their planes as systems of bars with equal bearing capacity to axial tension, but also with some stiffness to bending and shear, able to collect and redistribute stresses through their integrated joints.

1.2.3.2 - Grid geometry

Polymer grids consist of flexible ribs and rigid joints. The ribs are of constant thickness but have a variable geometry in their plane. They narrower at the middle and towards joints are becoming wider. The ribs are connected with joints by continuous curves. The joints are integrated with the ribs but are up to four times thicker. Monoaxial grids have symmetry axes in two directions, parallel with the ribs and perpendicular on them. Biaxial grids have the same structure but with two more symmetry axes, those parallel with diagonals of squared apertures.

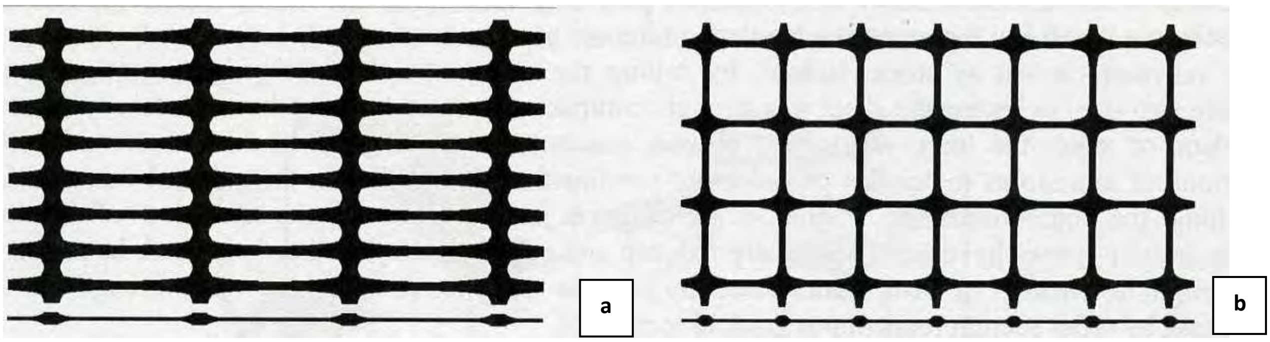


Fig. 1.26 - Geometry of monoaxial grids (a) and biaxial grids (b)

In their planes, both types of polymer grids are distinguished by two remarkable geometric qualities. Firstly, they appear as multiple connected surfaces. All apertures are limited only by continuous and closed curves. Geometric continuity of aperture outlines guarantees the fluency of tension flow *p*. By avoiding geometric and physic discontinuities exclude any phenomena of local stress concentration or deviation, known in mathematics as catastrophic. Secondly, all grid apertures are small, of only several centimetres, and rigorously equal between them. By this outstanding geometric quality, the grids are able to uniformly distribute, balance and keep under some level the stresses concentrated around the integrated joints. It is a practical way to fulfil Bernoulli’s assumption of uniformly stress distribution and reduction the frequency of their local concentrations. When however the values of stresses are increasing over some limits they are redistributed to the neighbour joint by a self-protection mechanism called adaptation. Mechanically, it is a practical way of transferring the vector actions of forces into tensor effects of stresses.

1.2.3.3 - Numerical validation

With the aid of computing program SAP2000 a biaxial grid was modelled in two alternatives, with integrated joints and with free ribs. Both models were first fixed on the left vertical side and submitted to forces in horizontal direction according to different laws of distribution and then fixed again on two sides and submitted to the same forces but in two directions. The resulting elastic deformations have been reported to the initial orthogonal shapes and they emphasise the essential different behaviour of the two opposite types of grids currently used as reinforcement.

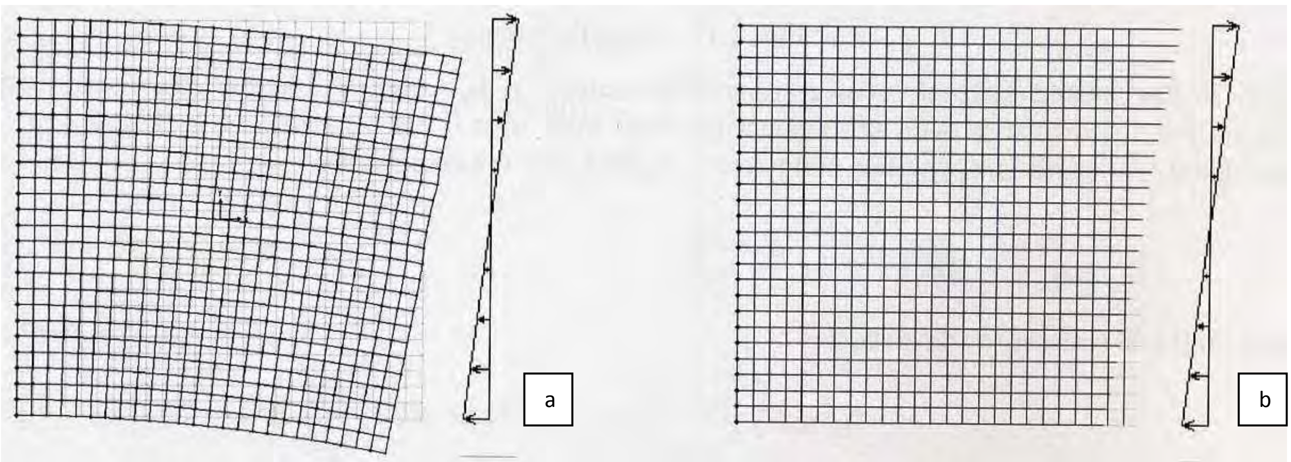


Fig. 1.27 - Grid models with integrated joints (a) and free ribs (b) submitted to moniaxial actions

Quantitatively, the cumulative deformations in the model with free ribs are much larger than in the model with integrated joints. The lack of cooperation between ribs through joints in the two directions also qualitatively appears in the deformed shapes of the two models. Such numerical analyses are useful not only for designers but also for producers of grids in order to optimise the geometric and mechanical characteristics of grids with the specific practical requirements.

1.2.3.4 - Stress transfer

Generally, by the word *reinforcing* the consolidation or strengthening of a weaker material with a stronger one is understood. Based on this approach during the time successively have been developed reinforced concrete, pre-stresses concrete, reinforced soil and recently a large number of composite materials. Both reinforcements and reinforcing techniques differ from one material to another. For instance, concrete is reinforced with metallic reinforcement produced by ductile steel. The reinforcing is passive, and the mechanism of stress transfer from concrete to reinforcement is based on the *clamping effect*. It produces continuously along the lateral surface of circular bars, exclusively by shear stresses τ. On the other hand the same concrete is actively pre-stressed by post-tensioning the metallic reinforcement that is also by steel but brittle, no longer ductile. Transfer mechanism of stresses from concrete to reinforcement is bases in this case on anchoring the cable at their ends. It discontinuously produces only through limited and reduced surfaces. Finally, the approach of composite materials is based on De Saint Venant's principle of geometric continuity of strains and there are some well-defined technologies to fulfil that principle.

Polymer grids with integrated joints of Tensar type are used for reinforcing both granular soils and lime mortars. Therefore the matrices are either discontinuous in the case soils or continuous in the case of mortars. In both cases the reinforcing is passive and consist in anchoring of polymer grids. Transfer mechanism of stresses from matrices to grids produces discontinuously only through integrated joints. These specially shaped joints transfer the normal stresses σ, locally concentrated around them, into sectional stresses. They are mainly stretching forces and provide the connections with the neighbour joints.

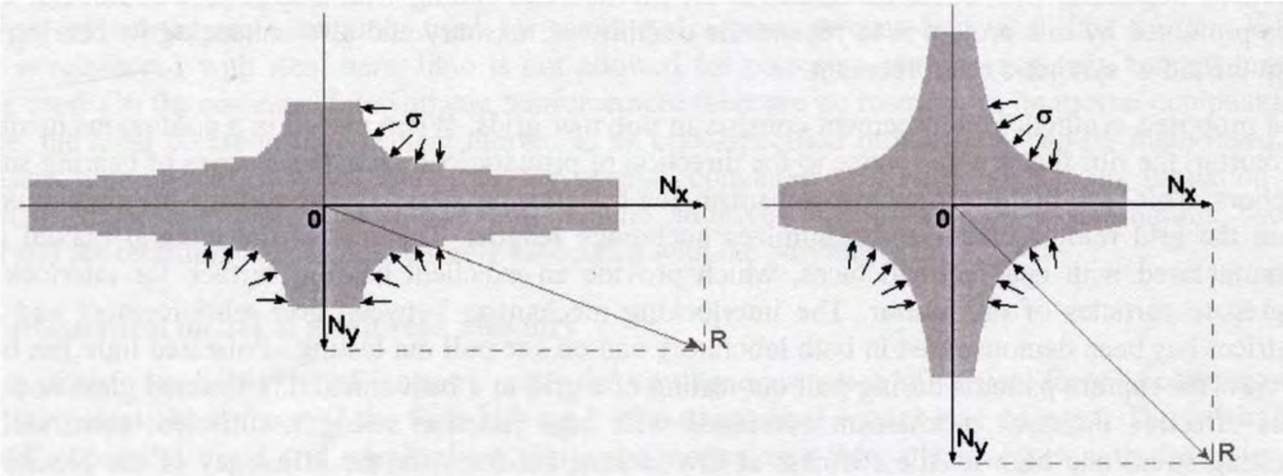


Fig. 1.28 - Mechanism transfer of stresses around the integrated joints

Due to their slenderness the ribs never take over compressions. The rigid joints always redistribute the stresses to be transmitted only by stretching forces. However, the ribs by their variable shapes are stiff enough in their plane to be able for taking over shear forces and transmit them by bending towards joints.

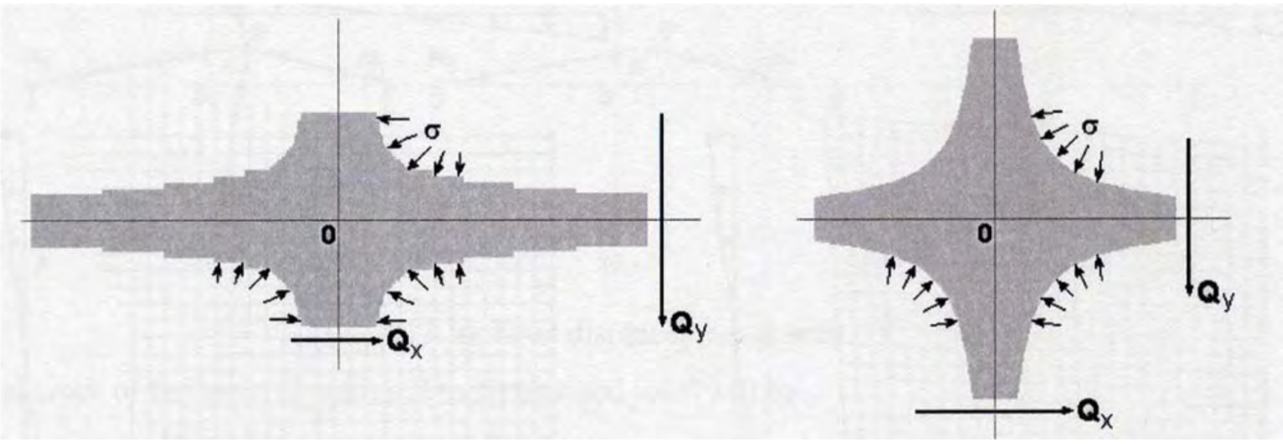


Fig. 1.29 - Shear forces around the integrated joints

As a consequence of the above-mentioned mechanisms of stress transfer the polymer grids with integrated joints have the outstanding ability to confine the matrix material interlocked in their apertures. In the plane of grids the stress transfer from matrices to the four joints of each aperture produces by the so-called *vault effect*. In the case of reinforced soils it is a bi-directional phenomenon that enhances somewhat the bearing capacity of reinforced structure. Vault effect is strongly amplified by reinforcing mortars in bed joints because due to brick pressure the phenomenon becomes three directional.

In all loading cases the reinforcement of polymer grids is deformed. The developing strains in both joints and grids, which are always elastic and plastic, simultaneously occur. The elastic stresses and strains form together the potential energy of deformation that is transformed in mechanical work when the unloading is produced. On the other hand the plastic strains together with the corresponding stresses dissipate by heat the rest of induced energy. By this energetic mechanism, based on the ductility of polymer grids, the reinforced material, either soil or masonry, is protected against the local concentrations of stresses. It is a spontaneous and self-adjustable mechanism typical only for grids with integrated joints. Indeed, if grid ribs would be free then the sectional stresses are no longer composing as shown in FIG while the deformations in each direction become much larger. In addition, if the reinforcement would be perfectly elastic, without ductility qualities like in the case of fibbers, then large amounts of stored energy become extremely dangerous by causing sudden failures and even dislocations. Since all works of reinforcing any matrix material have a hidden character, they should be carefully watched on construction sites and checked on physical and mathematical models.

1.2.3.5 - Field application

The grids have been used in new construction and repair applications that include formed concrete and shotcrete. To repair an existing seawall, workers attached the grid to mushroom headed nylon fasteners with a stainless steel core embedded in the wall. This held the grid away from the wall and ensured that it was enveloped by dry-process shotcrete applied later. Other repair applications include sewer and tunnel linings and pile jacketing of pier supports subjected to impact loading.

When used in pavements or floors, the grids are frequently vibrated into place using an attachment that fits a vibrating screed. Workers place concrete to full depth between edge forms. Then they roll out the grid across the top of the concrete. Moving the vibrating screed across the surface embeds the grid while compacting the concrete.

Fig. 1.30 - Polymer grid used for seawall repair is attached to nylon coated fasteners embedded in the wall

Chapter 2 - SEISMIC BEHAVIOUR OF CLAY BRICK MASONRY REINFORCED WITH POLYMER GRID

In the last century, more than 60% of human accidents during earthquakes have mainly due to structural damage and failure of single or unreinforced masonry. Several methods have been proposed to improved the strength, ductility and energy dissipation capacity of simple masonry, ranging from adding new elements of reinforced concrete to the implementation of seismic isolation. However, in developing countries, the strengthening of brick masonry structures is limited by economy and the feasibility of implementation.

Polymer grids used as reinforcement for masonry walls appears in these days as a good alternative to the welded wire mesh, that has been commonly proposed and used for retrofitting masonry walls around the world. The use of polymer mesh has some advantages over the steel mesh, like been stable against chemical and biological agents having a ductile behaviour in tension and been easy to install and adapt to any architectural shape. An additional advantage of this grid is that can be used in historical constructions since ancient plasters include lime and gypsum in the mortar and grids are not affected by its chemical components. The post elastic behaviour of reinforced walls subjected to in plane cyclic shear forces and out of plane bending is a key issue to investigate the change from a traditionally brittle seismic behaviour into an energy dissipation system that allows the masonry to successfully stand earthquake forces.

This study began more than a decade ago, initially with adobe and subsequently with bricks. Now studies are ongoing to ensure the effectiveness of the increase of strength of this material and to spread this method of seismic retrofit as an economical alternative than traditional solutions.

Research is based on various square panels, walls of single brick masonry and polymer mesh reinforced walls, to be tested to in plane shear force, made in the Structures Laboratory of the *Catholic University of Peru* and papers and information of experiments in abroad.

2.1 - Characteristics of tested components

To investigate the performance of brick masonry walls reinforced with polymer grid in the post elastic range, panels of brick masonry subjected to in plane cyclic shear force were tested in the Seismic Laboratory of *Catholic University of Perù*.

2.1.1 - Material properties

Masonry properties were obtained from simple component tests. Brick samples were subjected to dimensional variation, absorption, density and axial compression tests. The average density was 1.83 gr/cm3 and the average compressive strength of full bricks was 5.49 MPa.

Fig. 2.1 – Brick units (a) and ready to be tested (b)

The mortar for the layers was a mix of cement, lime and coarse sand 1:1:7 in volume proportion. This mortar had an average strength of 4.21 MPa. The mortar for the plaster was a mix of cement, lime and coarse sand with a volume proportion of 1:1:5. This mortar had an average compressive strength of 7.12 MPa.

Fig. 2.2 – Flexure test of mortar sample

Compressive strength of masonry was measured in piles of five brick units; joint thickness and mortar quality were similar to testing panel. A total of five piles were tested and the average compressive strength obtained was 3.68 MPa.

Fig. 2.3 - Compression test of masonry piles

Five wallets of dimensions 440 x 440 x 220 mm were subjected to diagonal tension test (ASTM 1981) in order to obtain the ultimate shear strength of masonry. Typical failure cut the mortar and the brick units what means a good bonding between mortar and bricks. Average compressive strength obtained was 0.35 MPa.

Fig. 2.4 – Diagonal compression test on wallets

The tension resistance of the grid was estimated by testing samples of the grid as shown in FIG. Tests were performed in the two orthogonal directions obtaining a tension resistance of 47 kN/m in the longitudinal and 34 kN/m in the transversal direction.

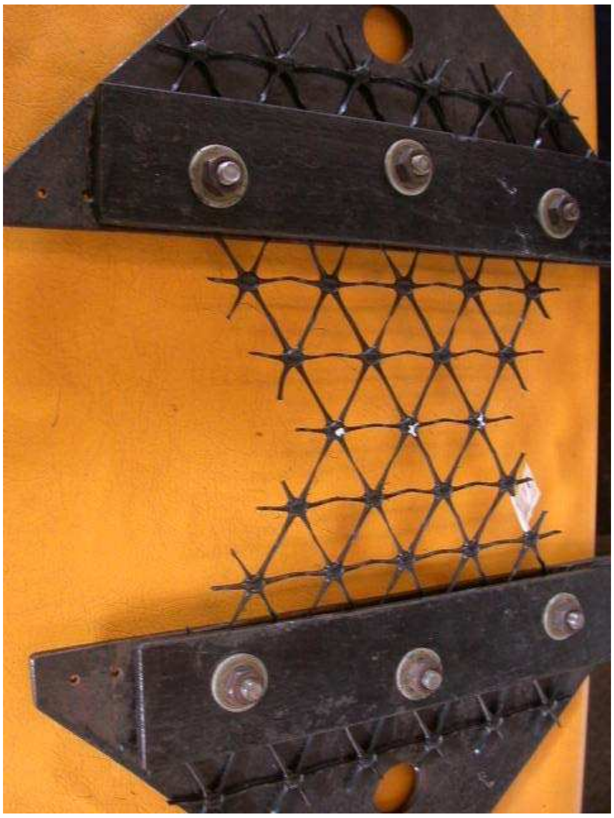


Fig. 2.5 – Tension test on polymer grid

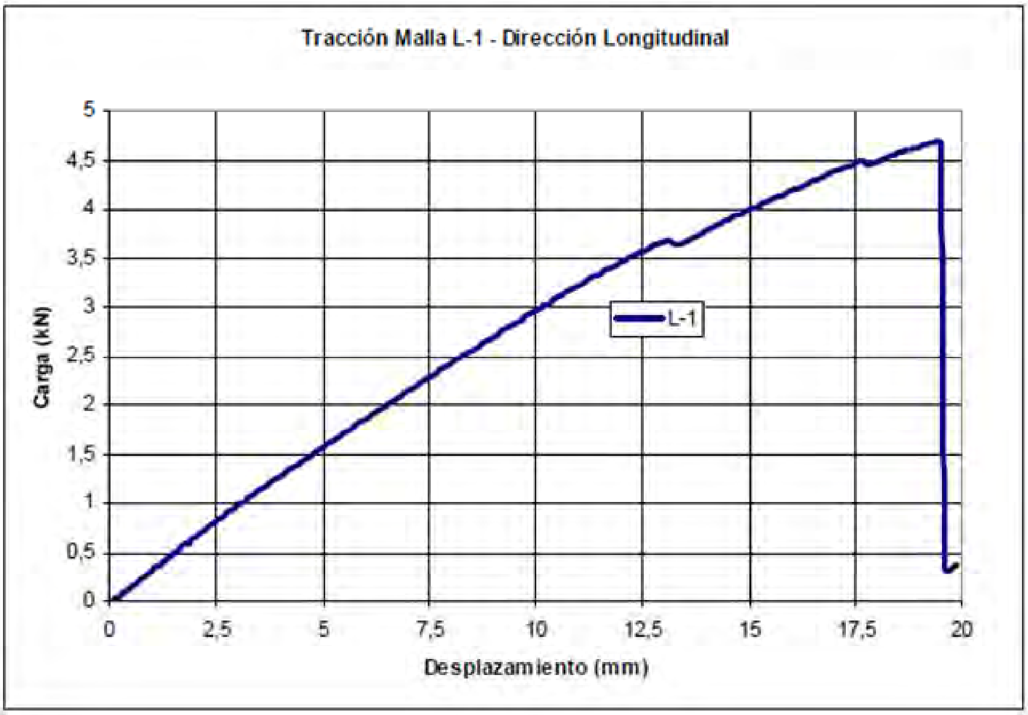


Fig. 2.6 - Load displacement curve for grid in longitudinal direction

2.1.2 - Specimens description

Twelve square panels of 1.2 x 1.2 m, as shown in Figure 2.7, were tested to in plane cyclic shear force, where four walls were tested without plaster, four with sand cement plaster and the last four with the grid reinforcement embedded in the plaster on both sides.

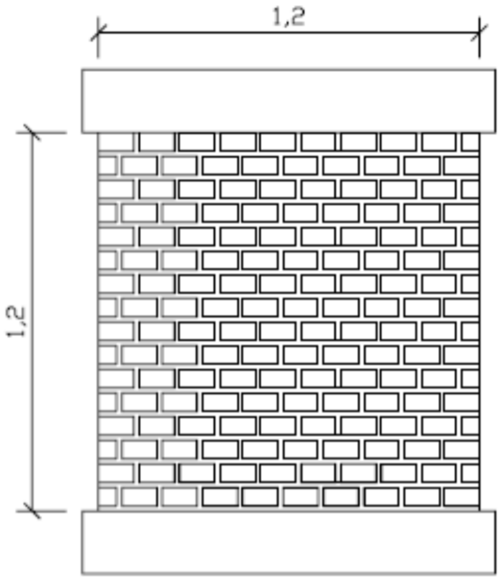


Fig. 2.7 – Panel for shear - compression test

Wall thickness in both cases was 220 mm for the non plastered walls and 260 mm when plastered on both sides. Solid bricks 110x 220x70 mm from the current industrial production were used to build panels, laying the bricks with a mortar of 1:1:7 (cement: lime: sand): The mortar for the plaster was 1:1:5, stronger than the mortar used for laying the bricks, in an attempt to simulate real retrofitting conditions. Concrete beams 220 x 200 mm at the bottom and top were built to transmit the vertical and horizontal loads in the square panels and to transmit vertical load and to work as horizontal support in the case of the panels subjected to bending. Table 1 shows the identification of plain panels, non reinforced plastered panels and the reinforced panels. Four of each type was tested to cyclic shear force and constant compression load. As seen in Table 2, the flexural panels, can be divided in two main categories: the non reinforced panels, and the reinforced panels; the non reinforced panels had two subcategories, with and without plaster; and the reinforced panels had three sub- categories, with vertical load, without vertical load and with the reinforcement overlapped at mid span. A single panel with reinforcement on the compression side was also tested. The panels were subjected to different maximum horizontal displacements based on the judgement about its observed stability.

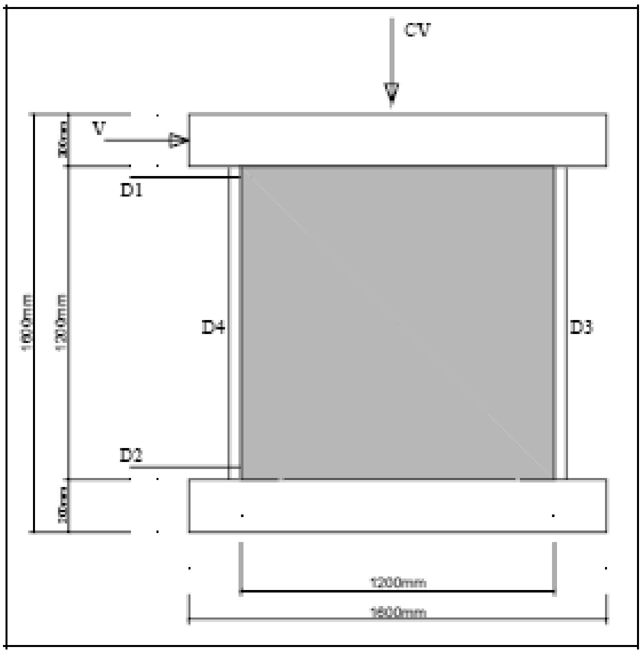


Fig. 2.8 – Shear – Compression panel

Panel Id.	Plaster	Polymer grid	Vertical stress (MPa)
SC-1 to SC-4	-	-	0.75
SC-5 to SC-8	Both faces	-	0.75
SC-9 to SC-12	Both faces	Both faces	0.75

Tab. 2.1 – Identification of the shear – compression panels

2.1.3 - Construction procedure

Every panel was built on a 0.22 x 0.2 m reinforced concrete beam; the bricks were cleaned with a brush and submerged in water for approximately 1.5 minutes before laying. The amount of water in the mortar was such as to allow an adequate workability during the construction of the wall.

Fig. 2.9 – Panels construction process

Horizontal and vertical joints were 15 mm for all panels. A top reinforced concrete beam 0.22 x 0.20 m was placed to transmit vertical and horizontal loads to the panels.

Fig. 2.10 – Upper beam on shear – compression panels

The polymer grid used as reinforcement was anchored to the panels using 50 mm steel anchors in pre drilled holes spaced 400 mm horizontal and vertically. In two of the bending panels, the grid was placed with 150 mm overlapping at the mid span with no anchors placed in the overlapping area. Each wall of the panel was watering before applying the 2 cm mortar plaster.

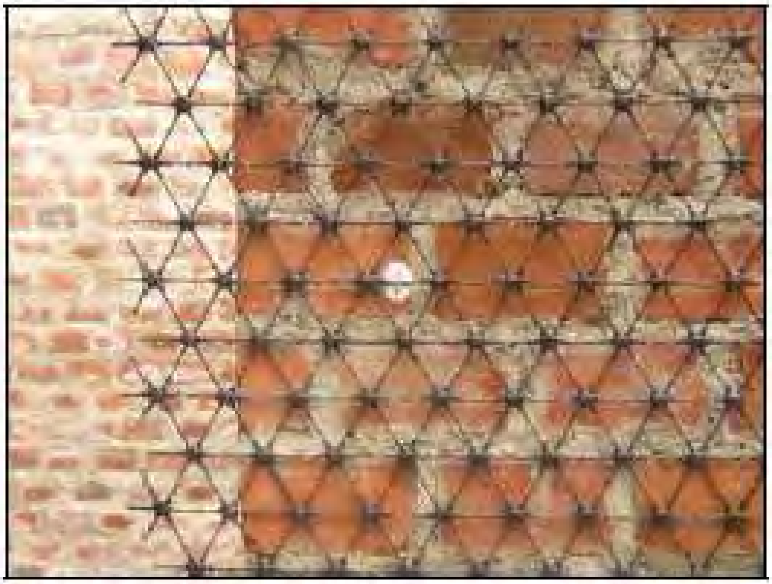


Fig. 2.11 – Polymer grid on panel

2.2 - Shear-compression test

2.2.1 - Instrumentation

The horizontal force was applied at the top beam with a 500 kN MTS hydraulic actuator. The vertical load was applied with one manual pump in the case of the non reinforced panels and with two symmetrical pumps for the grid reinforced panels. Six displacement transducers (D1 to D6) were used in this test. Figure 2.12 shows the distribution of the LVDT’s: horizontal displacement D1 was used to control the actuator, D2 was used to monitor the sliding at the base, D3 and D4 for the vertical displacement at both ends and D5 and D6 to monitor diagonal cracks in the panel.

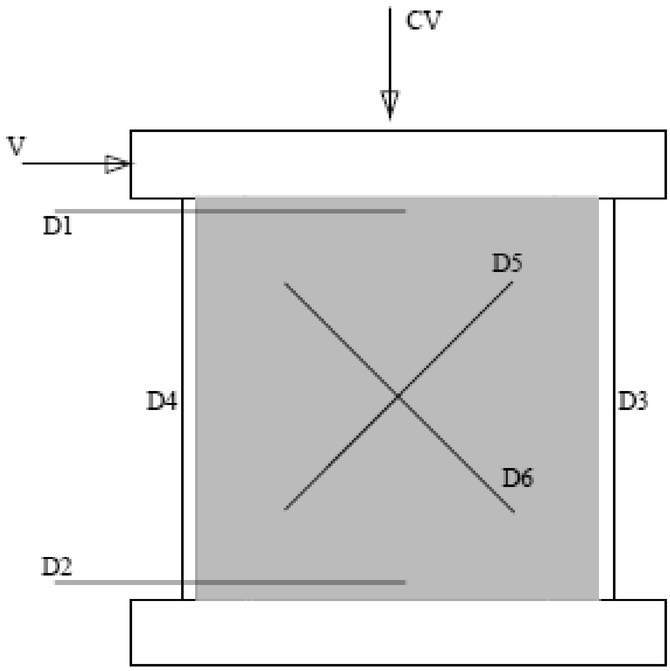


Fig. 2.12 – LVDT’s distribution in shear – compression panels

The test setup for panel without grid foresaw the application of the vertical load in a single position located at the middle of each panel. Meanwhile the test setup for reinforced panel was modified and the vertical load was applied at two points close to the top panel ends, in such a way that the vertical load applied was approximately 255 kN.

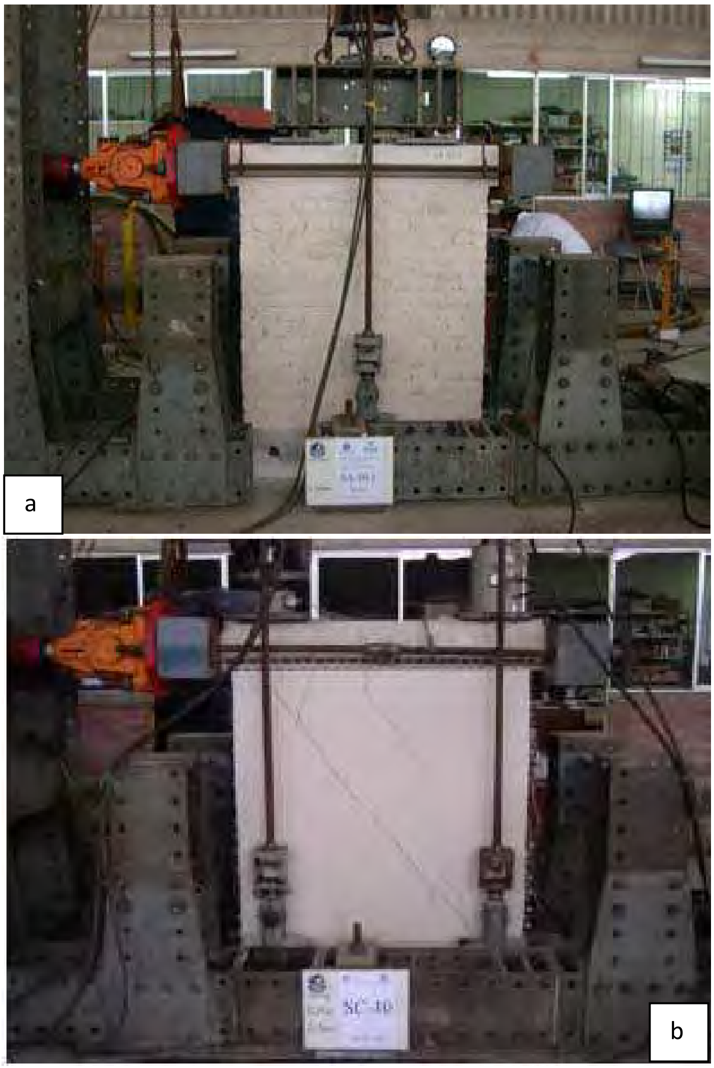


Fig. 2.13 – Experimental set-up for unreinforced panels (a) and for grid reinforced panels (b)

2.2.2 - Experimental results

Description and visual observations made during testing and interpretation of instrumental data are presented in this section.

2.2.2.1 - Masonry panels without plaster and without grid

A vertical load of 225 kN has been applied in a single position located at the middle of each plain masonry panel, thus leading to a compression stress of 0,97 MPa. At 2 mm of horizontal displacement thus corresponding to a horizontal load of approximately 70 kN, horizontal small cracks appeared near to the panels' bases and in correspondence to both ends of the panels. When reaching 4mm of horizontal displacement, with a horizontal force of the order of 90 kN, vertical small cracks (compression cracks) appeared at both ends of the panels; the above mentioned horizontal small cracks extended and enlarged, leading to the appearance of the first diagonal cracks. The diagonal small cracks enlarged at 7 mm of horizontal displacement, corresponding to a maximum horizontal load of approximately 98 kN, and the panels tried to rotate on their bases. Complete diagonal cracks then appeared at 10 mm of horizontal displacement and the force-displacement curve began to decrease.

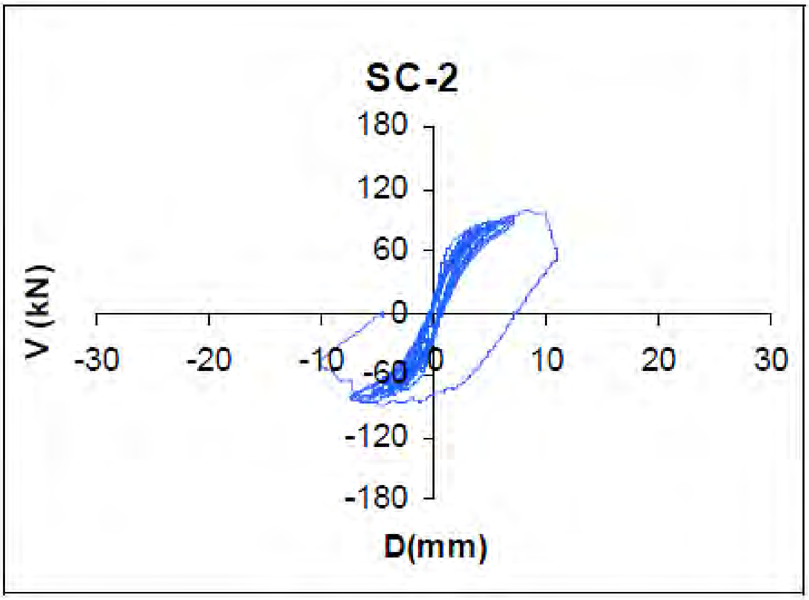


Fig. 2.14 – Force – Displacement curve of a non reinforced panel

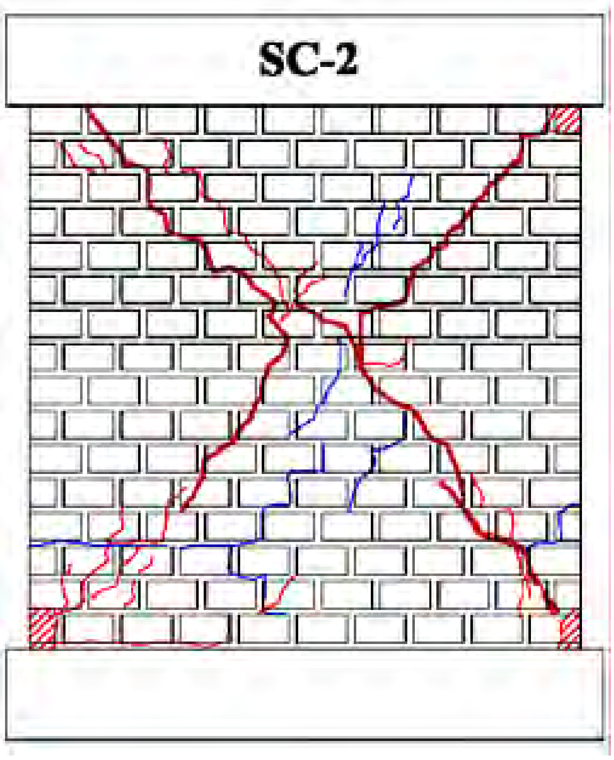


Fig. 2.15 – Crack pattern of panel SC-2

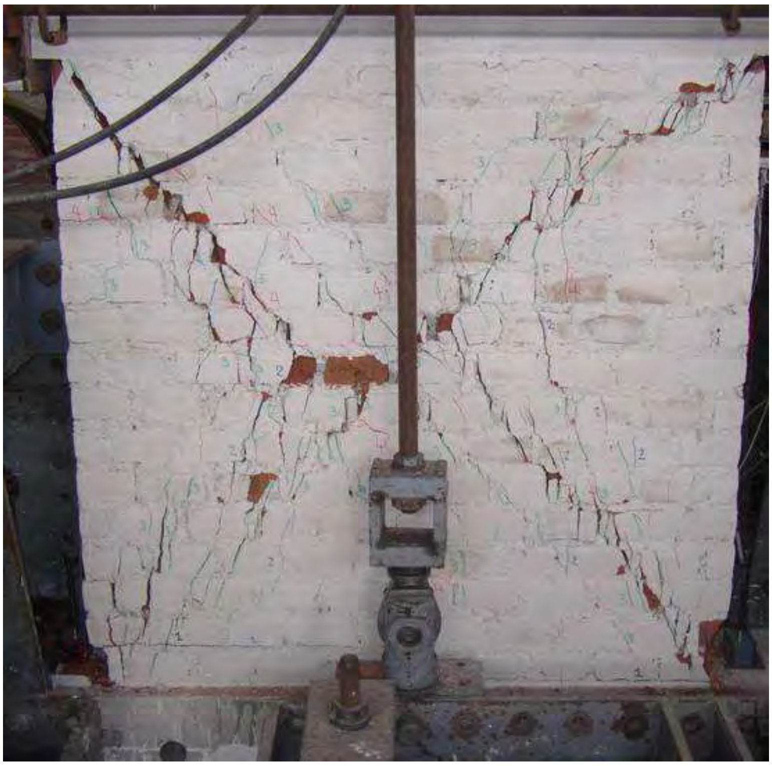


Fig. 2.16 – Cracks formed during the shear – compression test

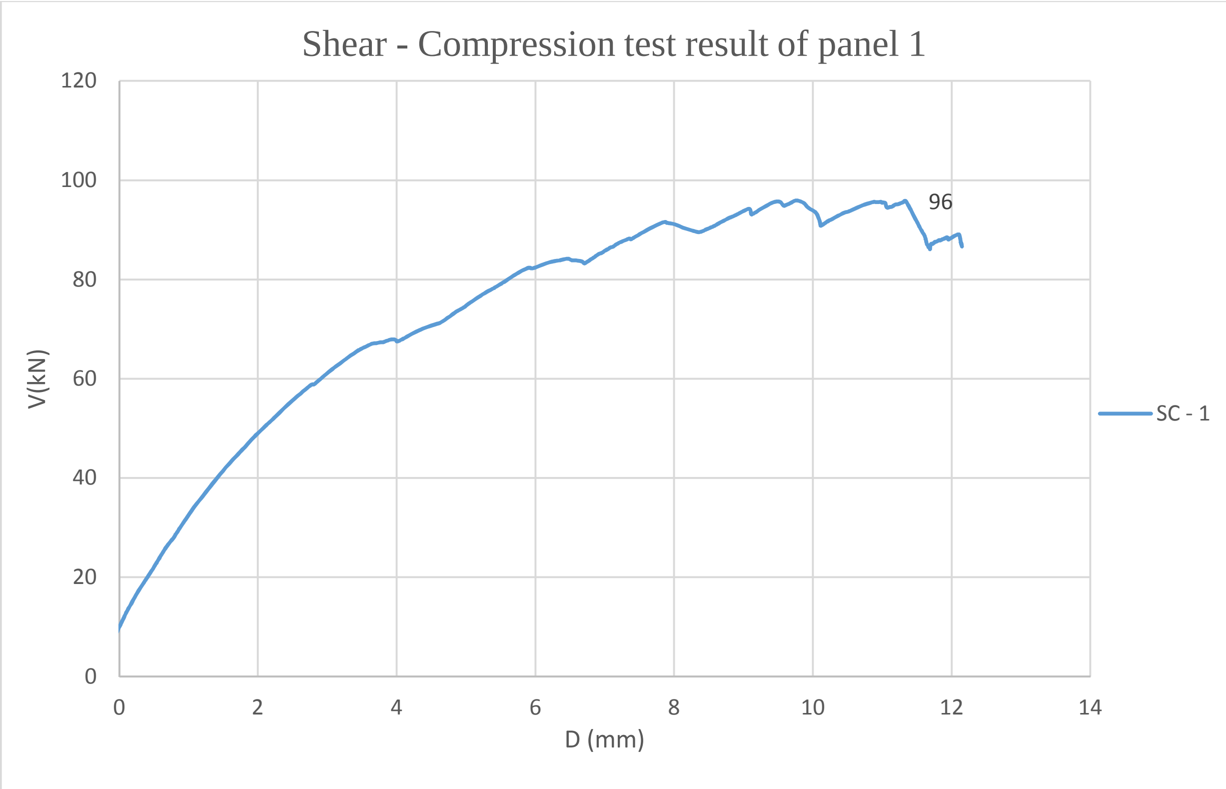


Fig. 2.17 –Force- Displacement curve of panel 1

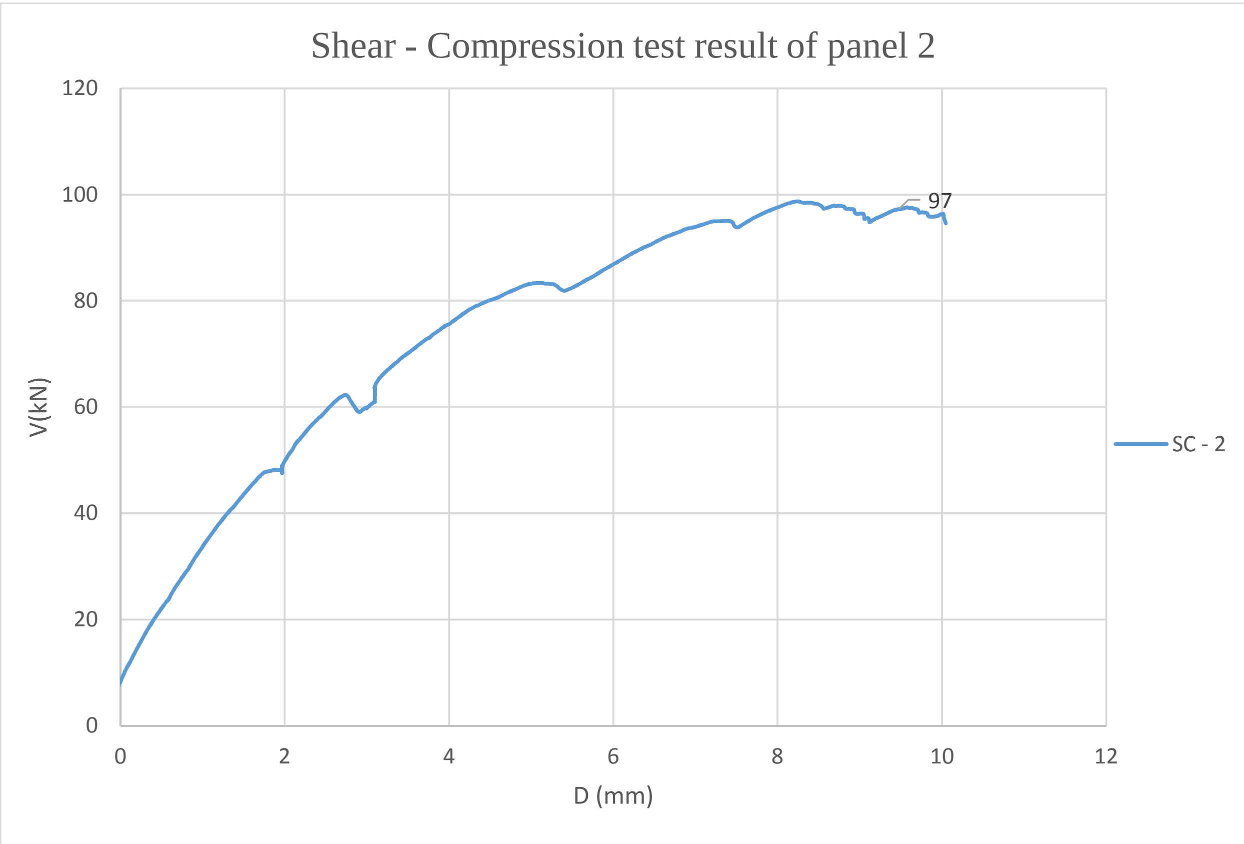


Fig. 2.18 –Force- Displacement curve of panel 2

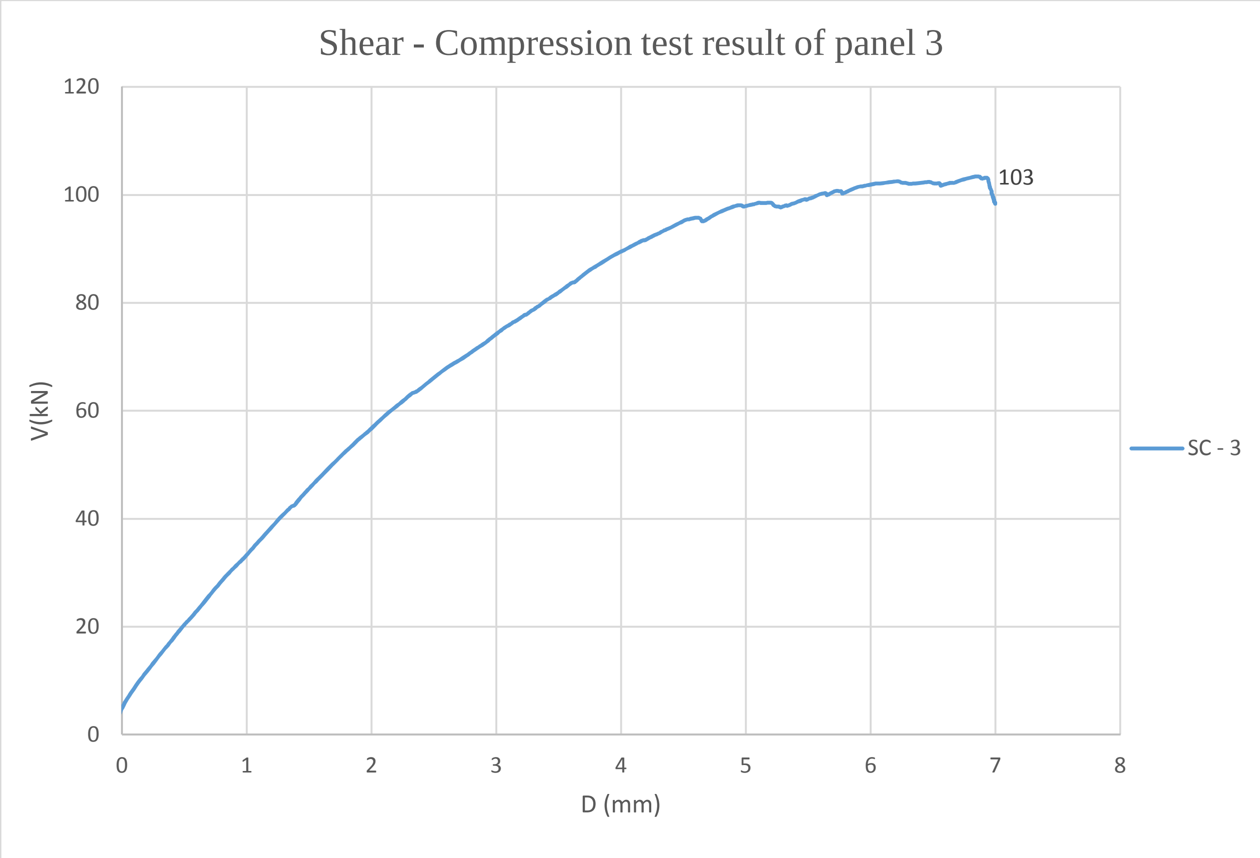


Fig. 2.19 –Force- Displacement curve of panel 3

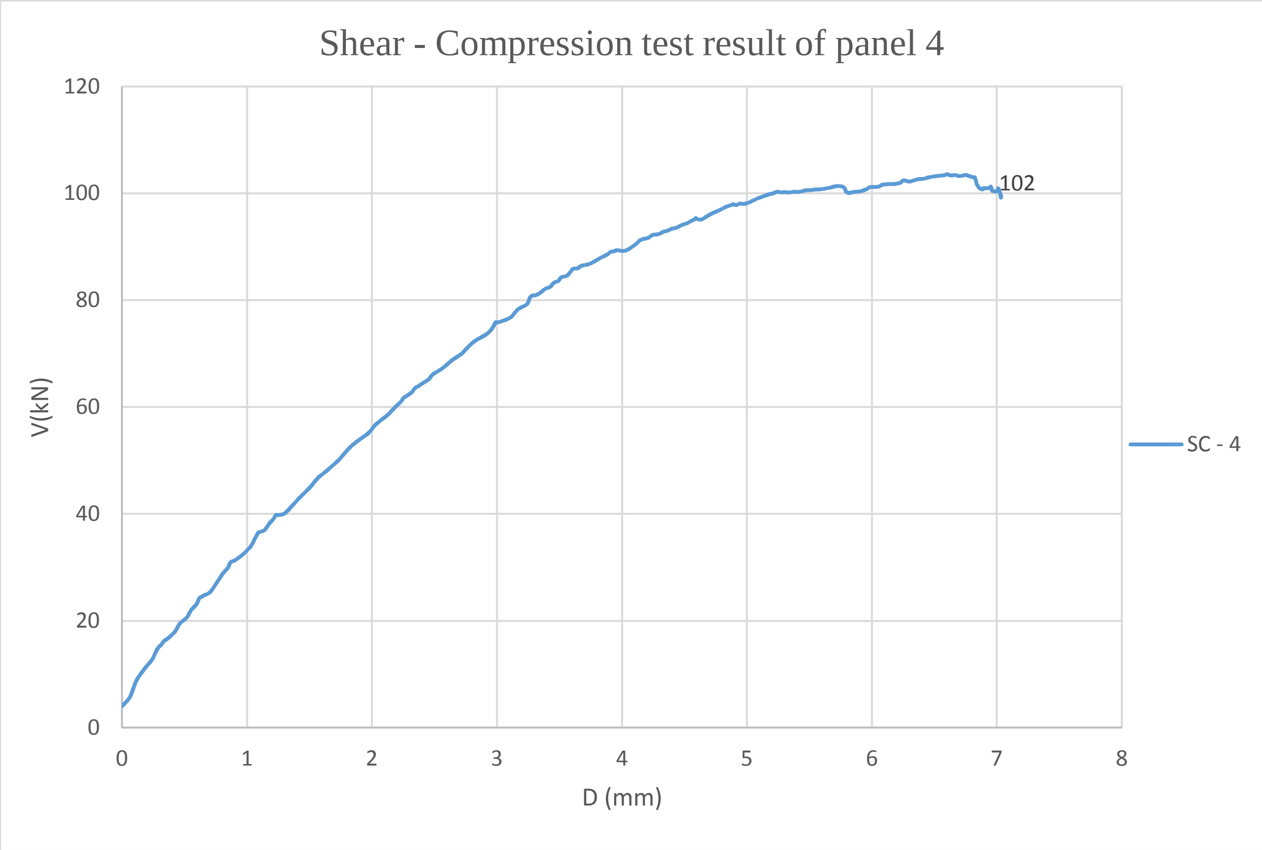


Fig. 2.20 –Force- Displacement curve of panel 4

2.2.2.2 - Masonry panels with plaster on both sides and without grid

A vertical load of 225 kN has been applied in a single position located at the middle of each plastered panels. The first horizontal small cracks near to the base of the panel took place at 2 mm of horizontal cyclic displacement at both sides, associated to a force of 100 kN. When reaching 4 mm of horizontal cyclic displacement, more horizontal small cracks appeared at the base of the panel. The associate horizontal load in this stage was approximately 105 kN. The panels experienced a certain rotation at their bases at 7 mm of horizontal cyclic displacement, while the horizontal lower small cracks began to enlarge. Vertical small cracks by compression appeared at both ends of the panels at 10 mm of horizontal cyclic displacement, being the maximum horizontal load approximately 120 kN. Diagonal small cracks appeared at 15 mm of horizontal displacement and grew when reaching 20 mm; the plaster began to detach from the surface, especially at the bottom outer side. The failure took place suddenly with the opening of a wide diagonal crack.

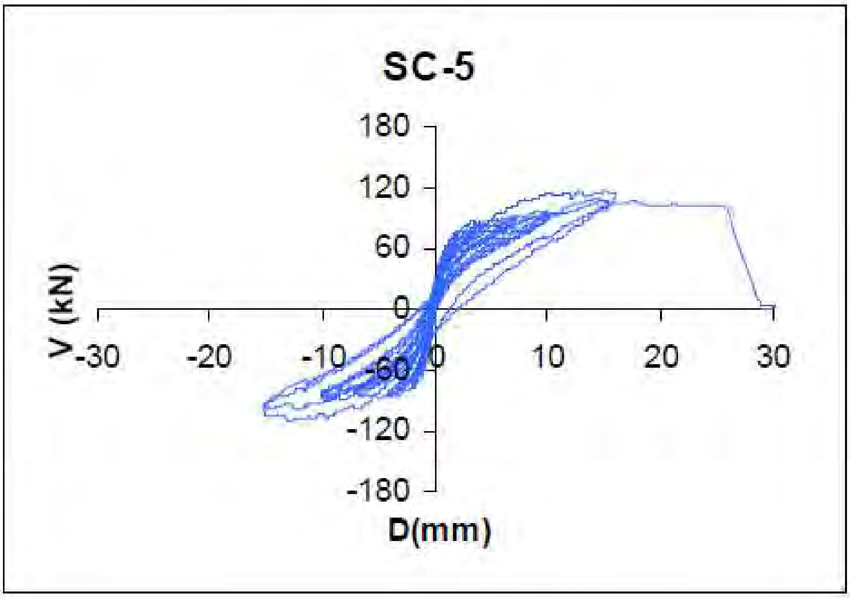


Fig. 2.21 – Force – Displacement curve of a plastered panel

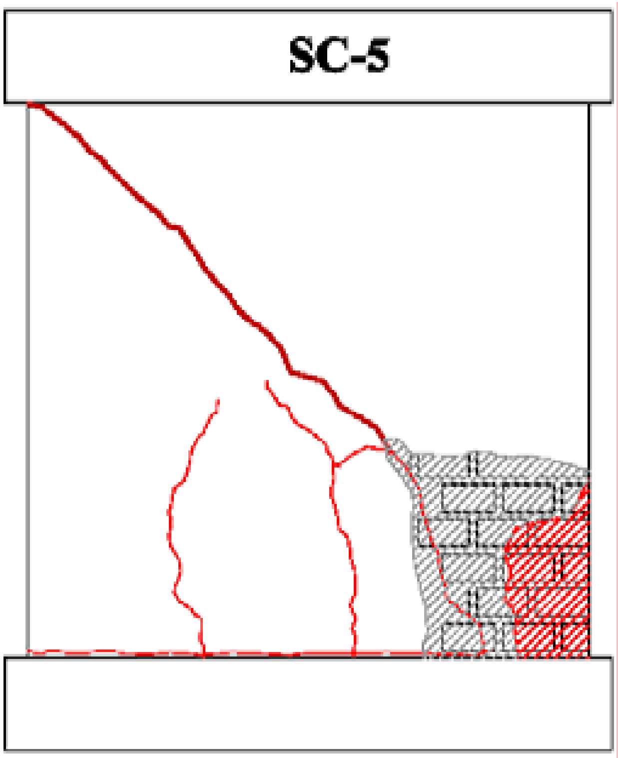


Fig. 2.22 – Crack pattern of panel SC-5

Fig. 2.23 – Diagonal crack formed during the shear – compression test

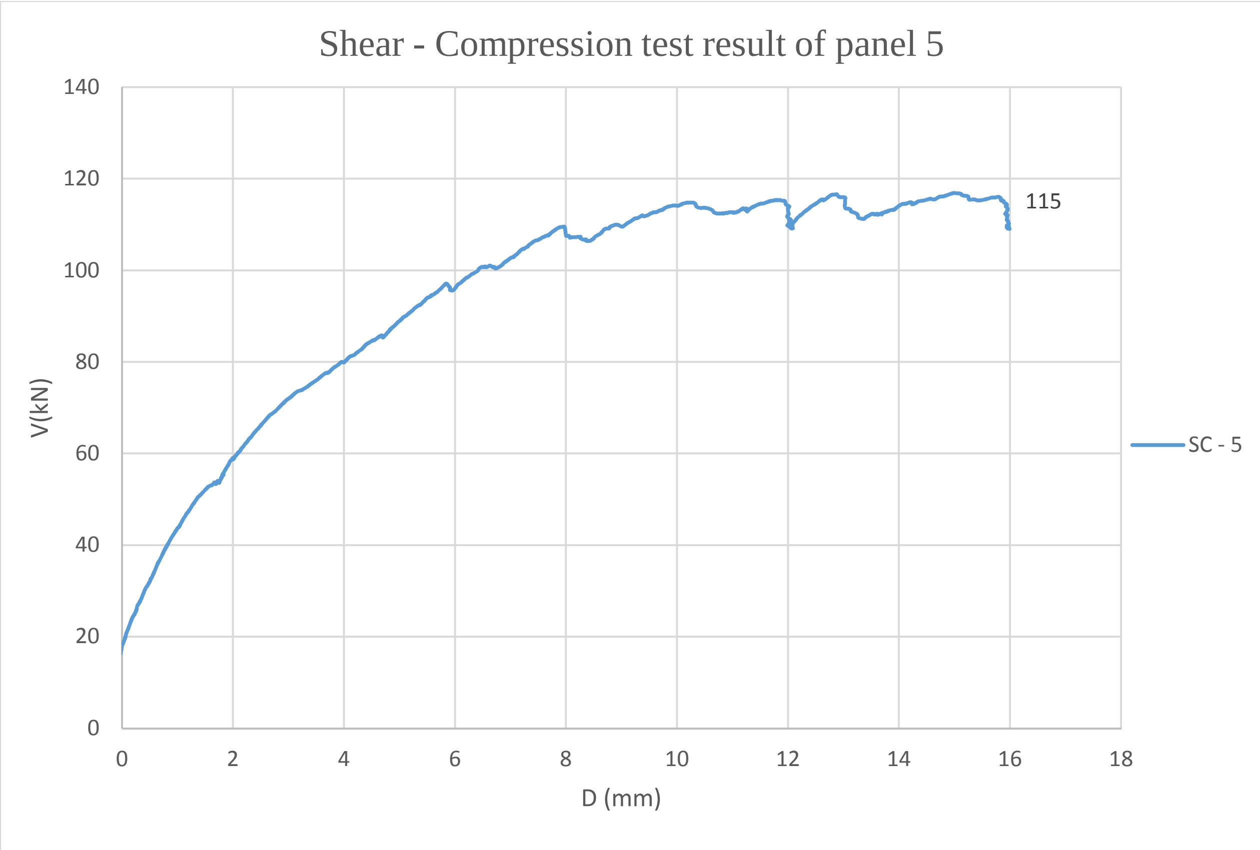


Fig. 2.24 –Force- Displacement curve of panel 5

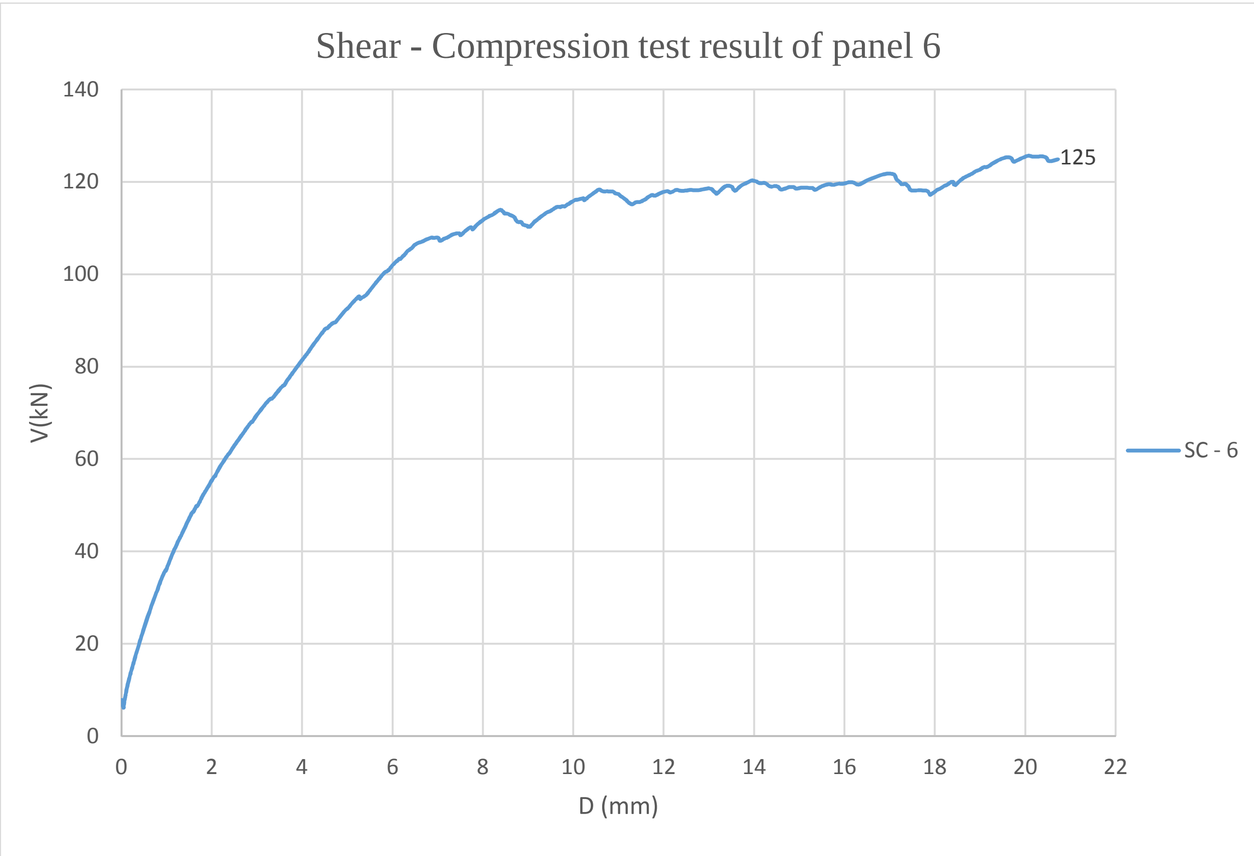


Fig. 2.25 –Force- Displacement curve of panel 6

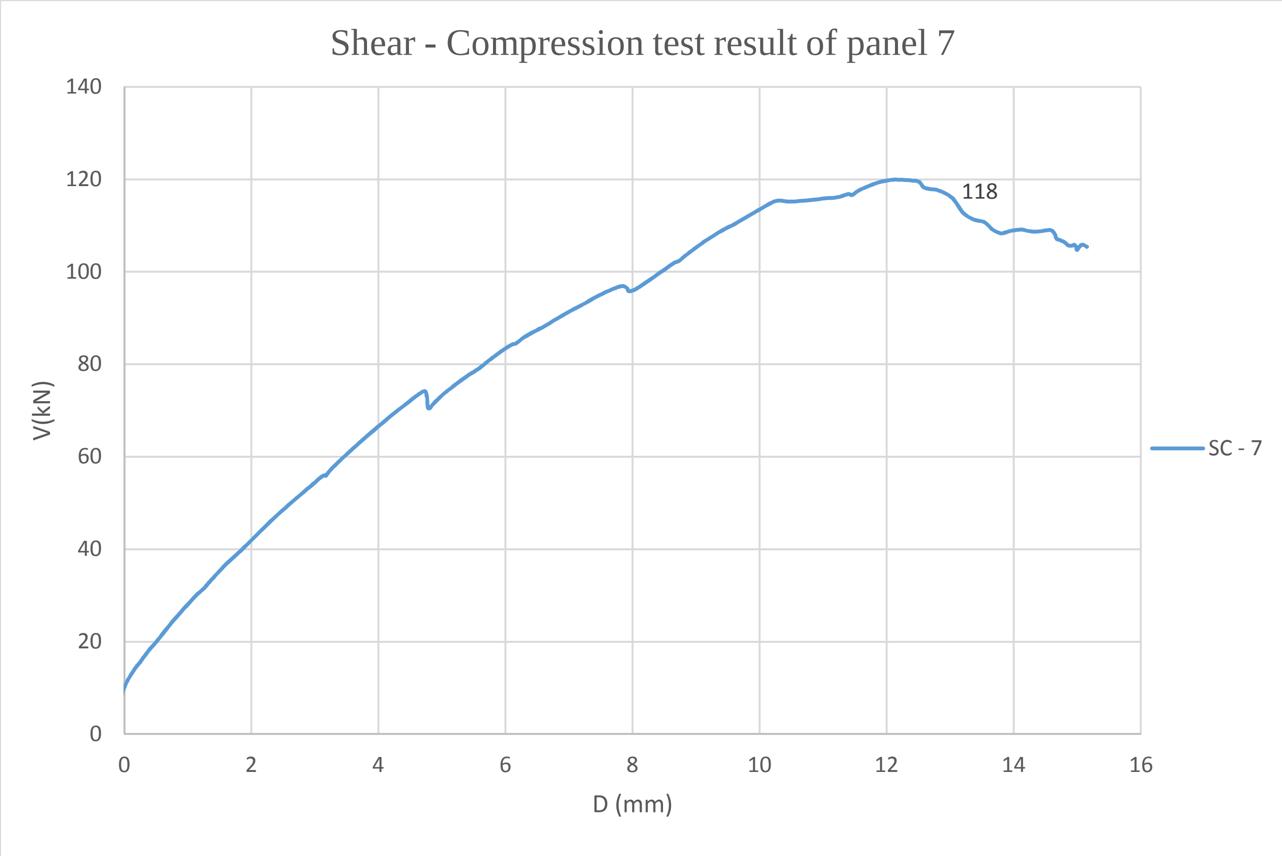


Fig. 2.26 –Force- Displacement curve of panel 7

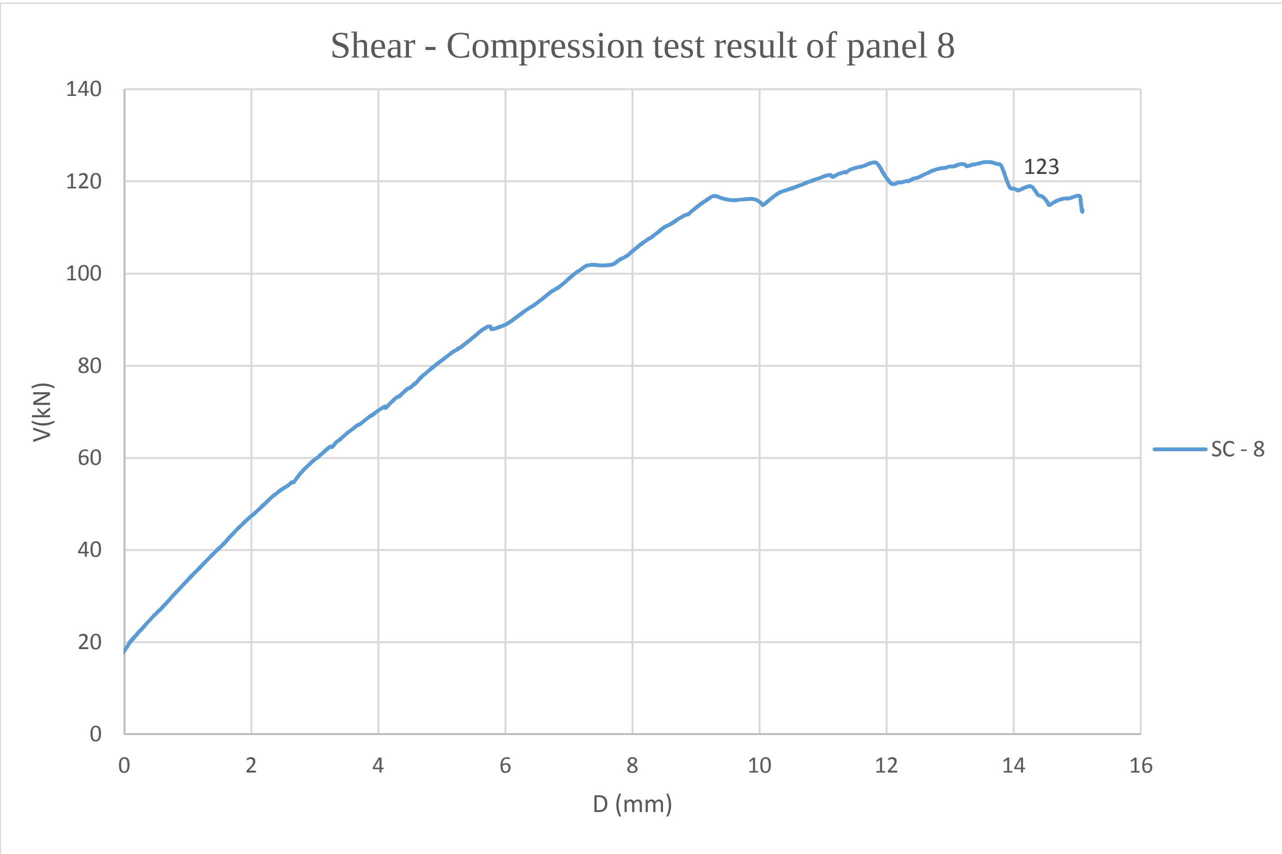


Fig. 2.27 –Force- Displacement curve of panel 8

2.2.2.3 - Masonry panels with plaster on both sides and reinforced with grid

For this test, the setup was modified and the vertical load was applied at two points close to the top panel ends, in such a way that the vertical load applied was approximately 255 kN. This modification was needed to better control the rotation that took place at the base of the panels. Panel SC-9 wall was tested with the first test setup; however, in this case the specimen failure by diagonal cracks did not take place due to the rotation at its base.

For specimens SC-10, SC-11 and SC-12, horizontal small cracks near the base of the panel continued appearing for low horizontal cyclic displacements, but in a one more controlled form than in the previous cases. For 8 mm of horizontal displacement, the maximum load reached was slighter bigger than the one of the previous tests. The small cracks pattern was scattered, diagonal small cracks appeared in a sequential way and followed the configuration of the reinforcement grid. For the maximum horizontal displacement, that was 20 mm, the plaster of both bottom corners detached from the walls due to the high concentrated compression forces.

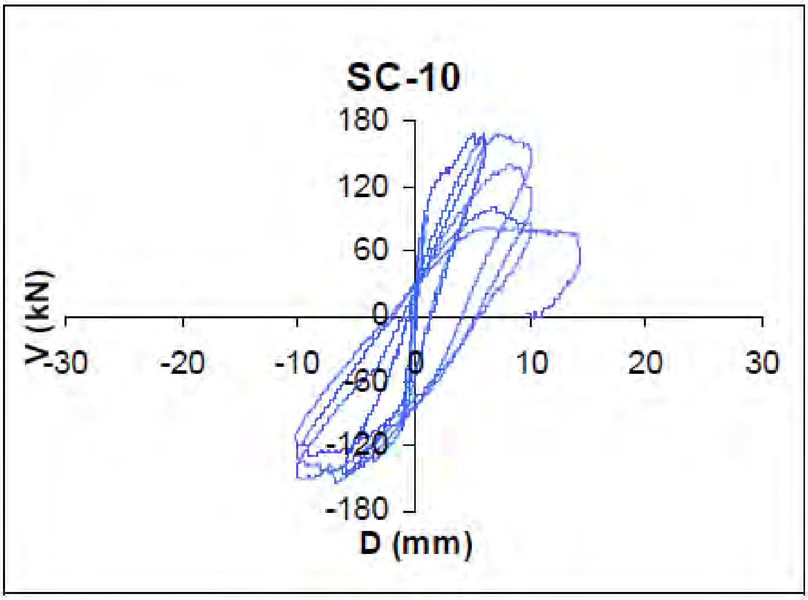


Fig. 2.28 – Force – Displacement curve of a plastered and reinforced panel

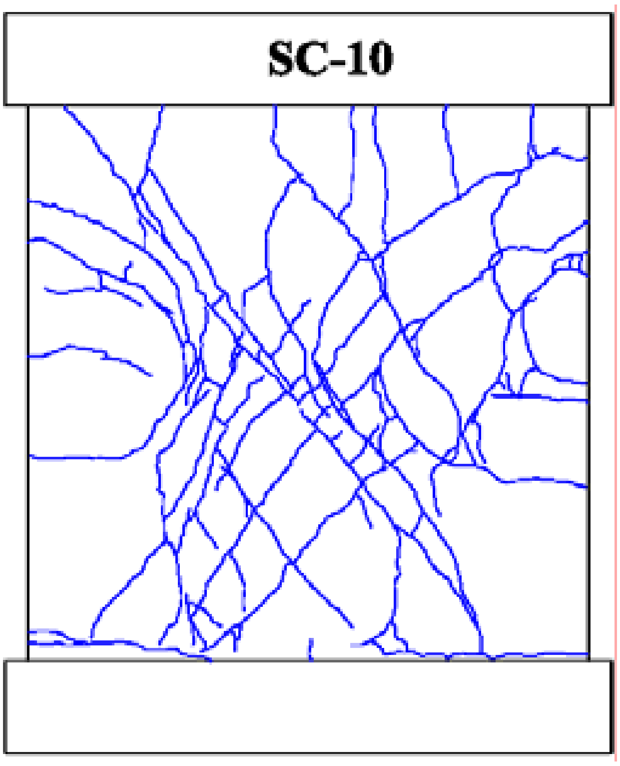


Fig. 2.29 – Crack pattern of panel SC-10

Fig. 2.30 – Cracks formed during the shear – compression test

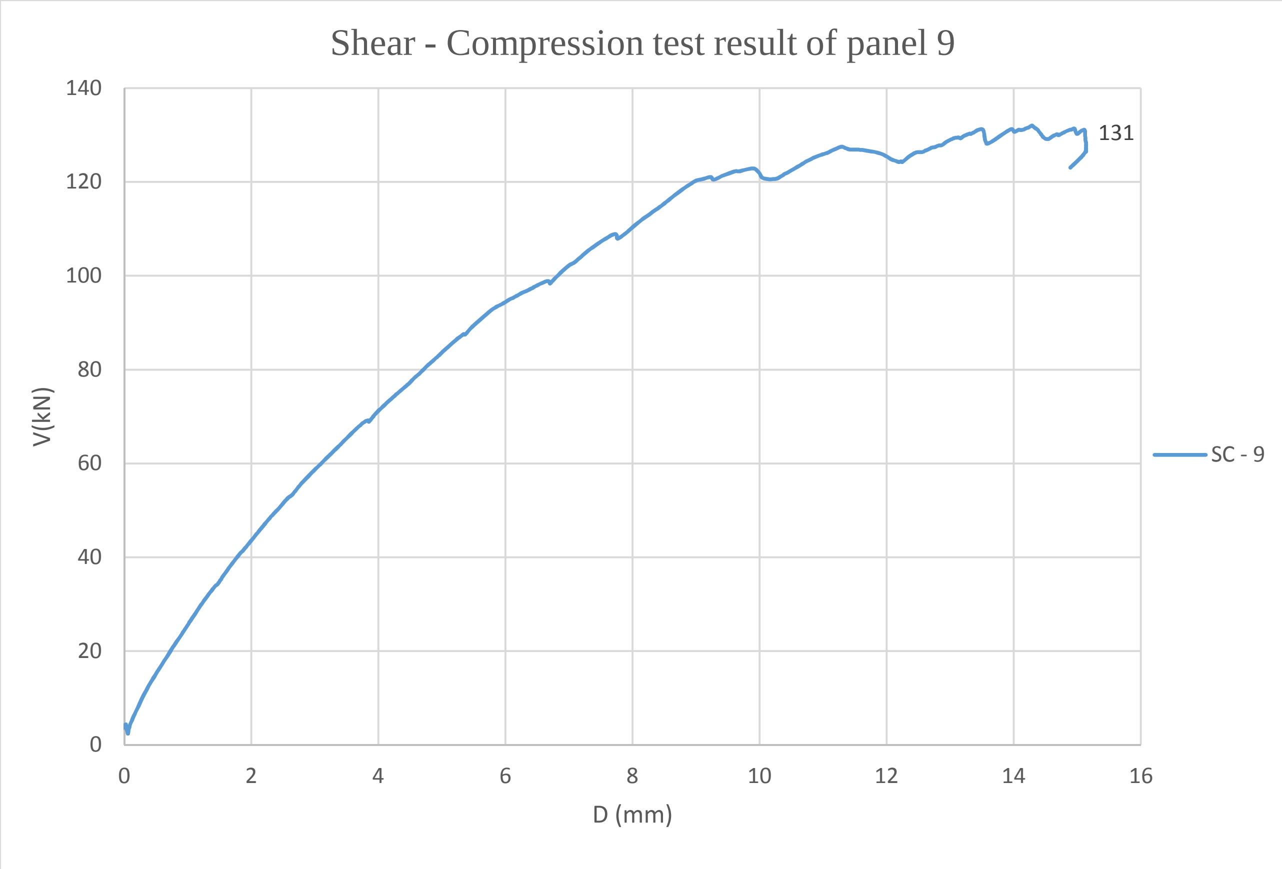


Fig. 2.31 –Force- Displacement curve of panel 9

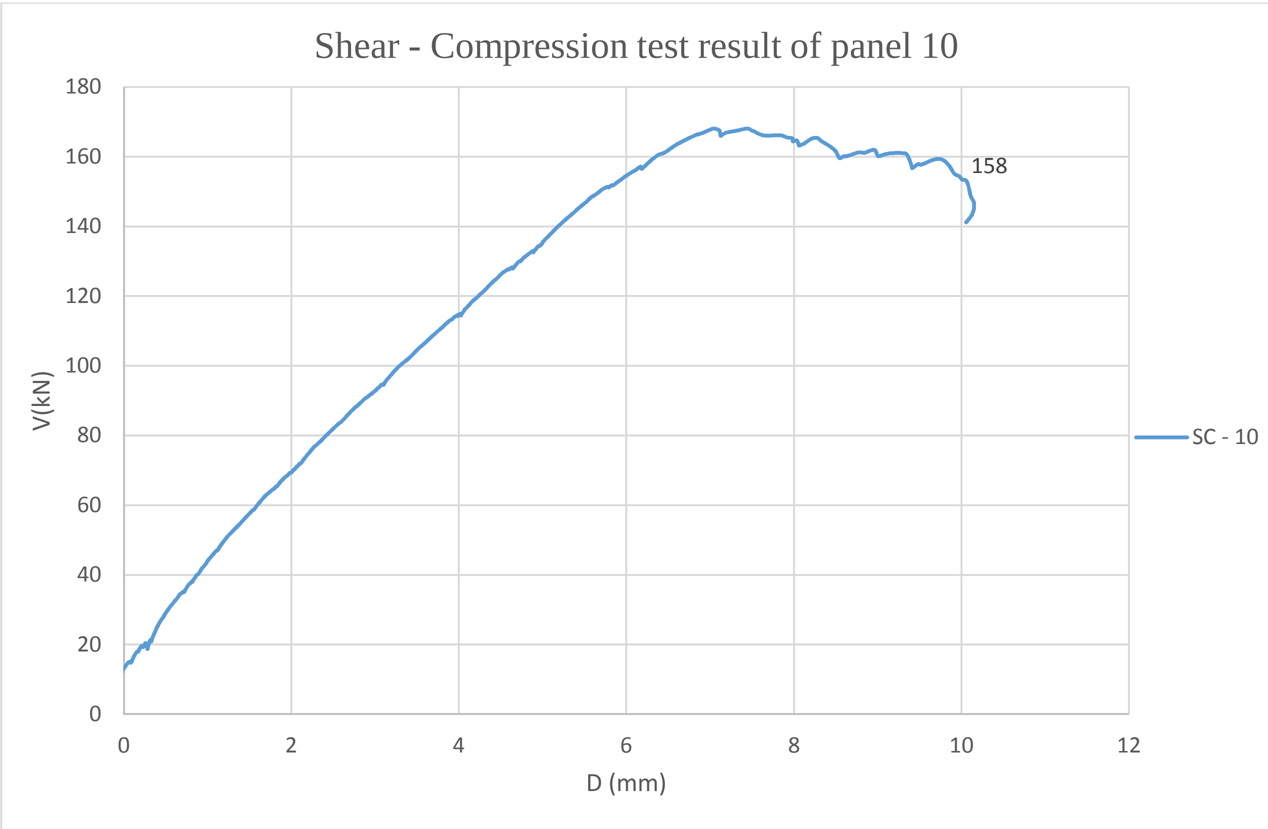


Fig. 2.32 –Force- Displacement curve of panel 10

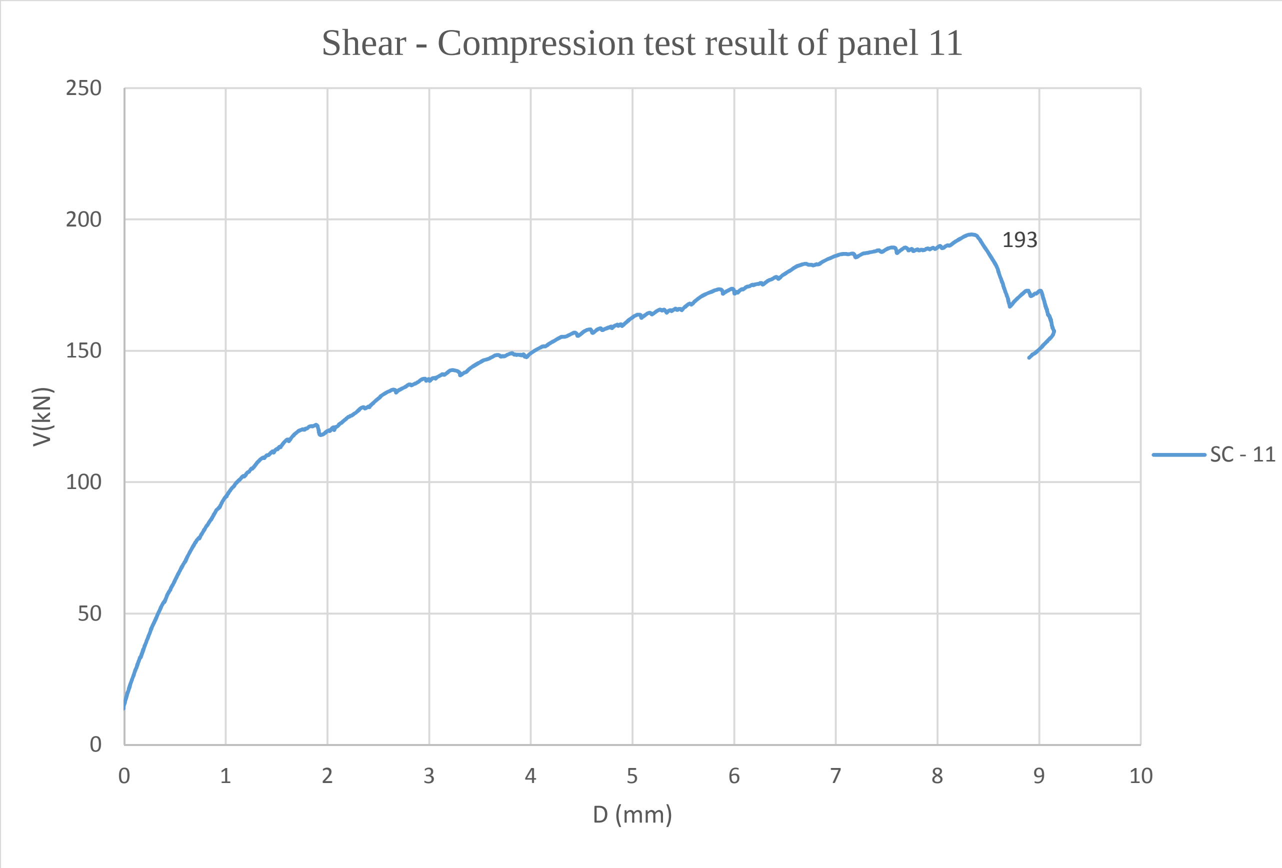


Fig. 2.33 –Force- Displacement curve of panel 11

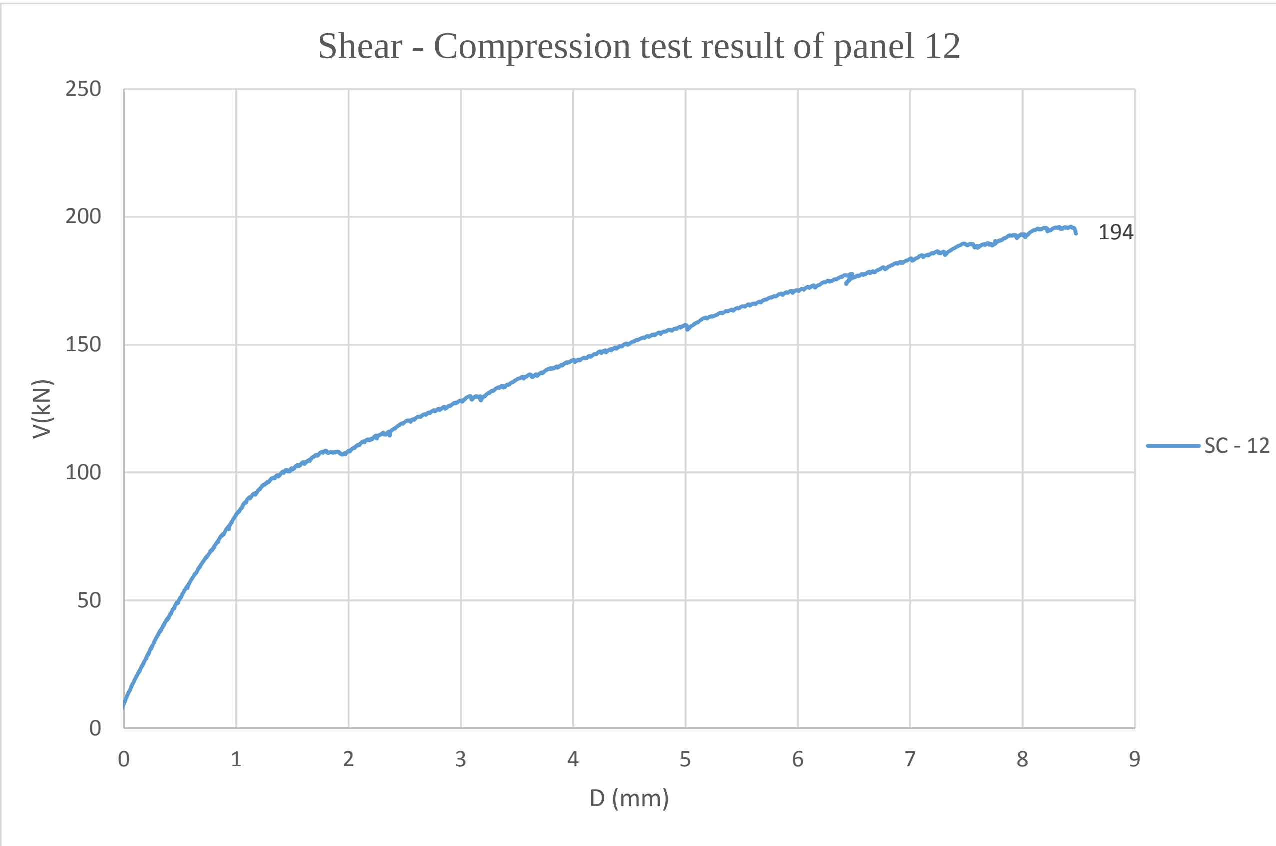


Fig. 2.34 –Force- Displacement curve of panel 12

2.2.3 - Analytical interpretation of results

In order to estimate a value of the increasing strength of shear force, a model based on equilibrium conditions has been assumed.

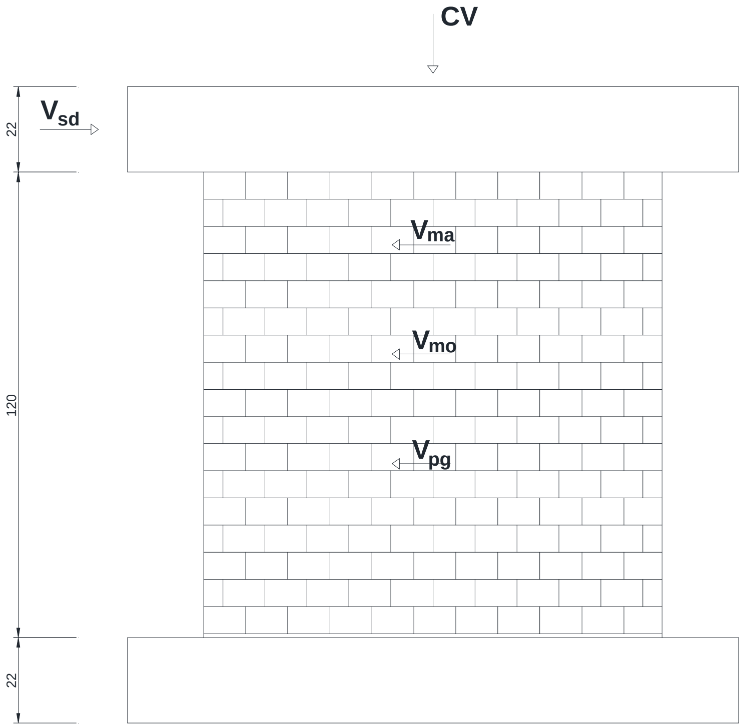


Fig. 2.35 – Structural model for the equilibrium force analysis

For simple equilibrium, we have:

$$Vmax = Vma + Vmo + Vpg$$

Where V_{ma} is the contribution of shear strength of unreinforced masonry to the nominal shear strength, V_{mo} the contribution of plaster in those panels which has been applied, V_{pg} the contribution of polymer grid composite material; V_{max} is the nominal shear strength to which the panels are subjected.

The same solution has been proposed by the *A*merican *C*oncrete *I*nstitute in *Guide to Design and Construction of Externally Bonded Fabric-Reinforced Cementitious Matrix (FRCM) System for repair and Strengthening Concrete and Masonry Structures* in which introduces the same formulation just written differently:

$$\Phi v Vn = \Phi v(Vm + Vf)$$

where V_n is the nominal shear strength and V_m and V_f are the contribution of (unreinforced or reinforced) masonry and FRCM composite material to the nominal shear strength.

This formulation, however, ignores the contribution shear strength given by plaster. Indeed we have seen from these experiments that this contribution relevant influence on the total shear strength V_{max}. In fact, we can see in graph. 2.36 the progressive increase of axial stiffness in tested panels unreinforced, plastered and plastered and reinforced with polymer grid.

2.3 - Critical remark

As the increase of strength is more relevant in the case in which studies the Ultimate Limit State, we discuss the underlying diagram.

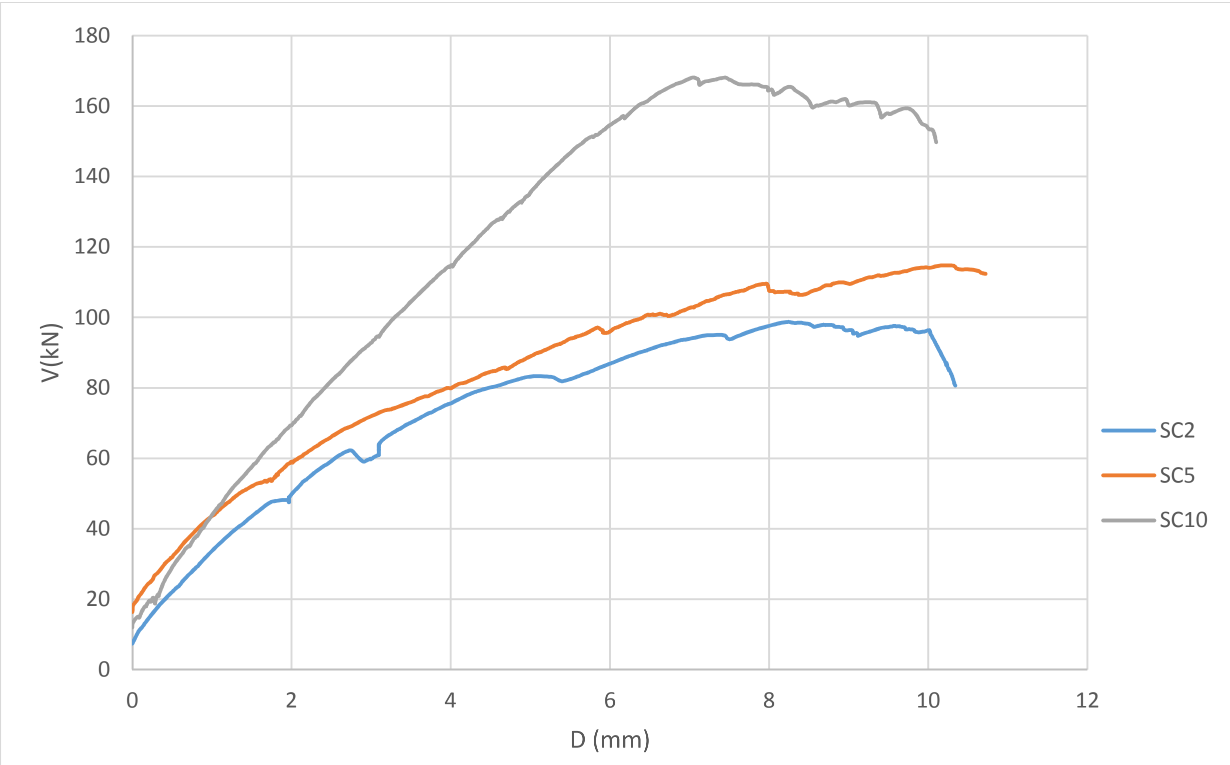


Fig. 2.36 – Force – Displacement curve of the three types of tested panels

From the diagram we note that up to a displacement about 3 mm the plastered wall starts to crack, changing its axial stiffness and entering in plastic range, while in contrast to the polymer FRCM wall remains in the elastic range. This contribution is given by the confinement given by the polymer grid that allows to reach a value higher before coming to cracking.

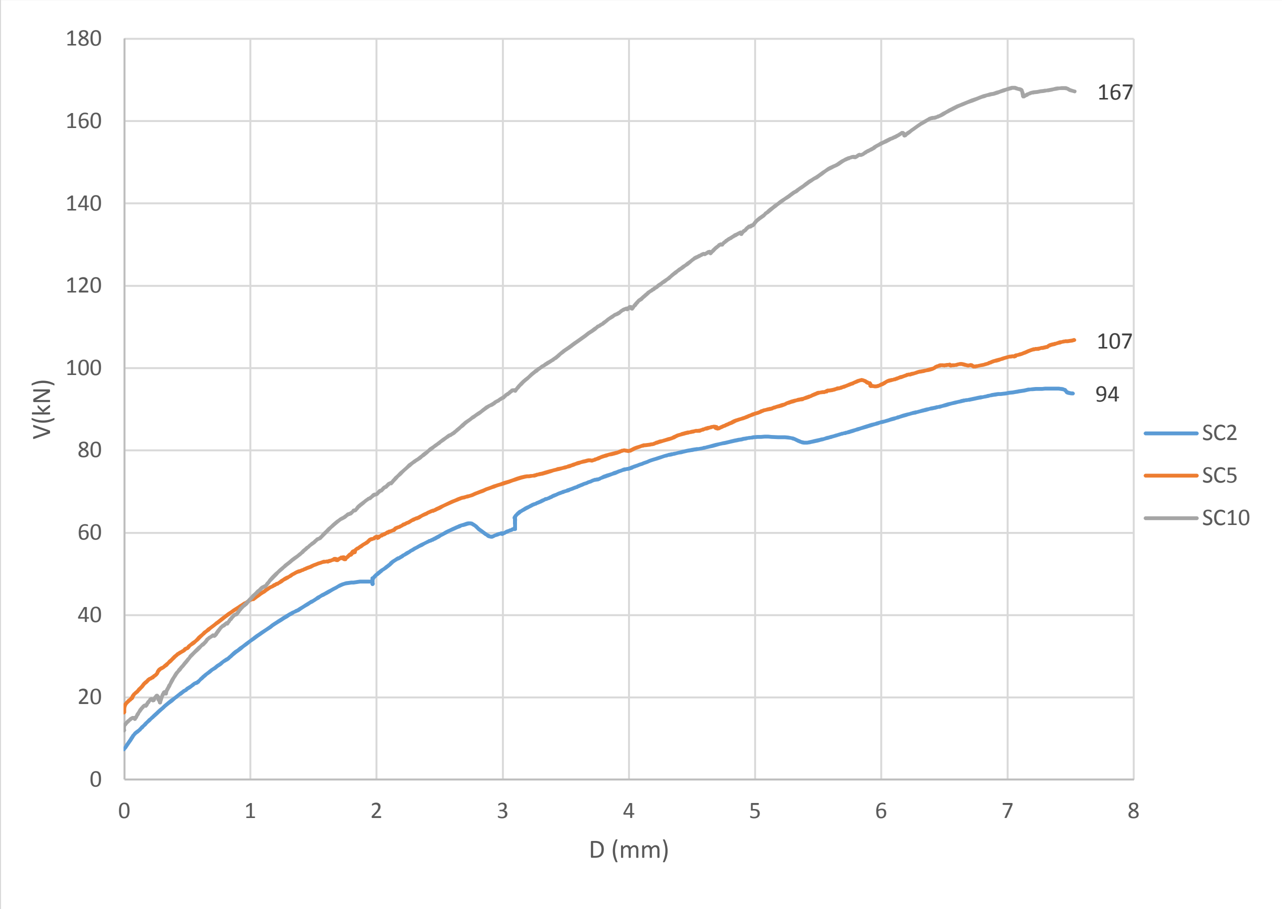


Fig. 2.37 - Force – Displacement curve of the three types of tested panels in elastic range

Observing well the graph, it is noted that in correspondence with the deformation of 7,5 mm the reinforced wall exits from the elastic range, since the fibers of the polymer grid reach their maximum tension, thereby nullifying the effect of the confinement and leading to cracking obliged the mortar. Then the wall enters in the plastic range and deforms itself about a 30% more than elastic deformation until it reaches the break point, thus having an elasto-plastic behaviour almost perfect.

This comparison can also be seen in the table 2.2, where it is immediately evident the amount of additional resistance that the polymer grid provides to the plastered wall, approximately 60,39 kN. While between the plastered wall and single wall we the increase is about 12,56 kN. This increase is given not only by the effective resistance of the mortar but also from the fact that increases of 4 cm the total thickness of the wall (2 cm each side) finding itself so to have a sectional area greater than the first. In fact is noted that the flexural rigidity does not vary much, which implies that both materials (bricks and mortar) work together as unique material in order to have a behaviour of a wall with 24 cm of total thickness.

Specimen	D	V_{rd}	Difference	%
SC-2	7,07	94,25	/	/
SC-5	7,53	106,81	12,56	13,33
SC-10	7,53	167,21	60,39	56,54

Tab. 2.2 – Difference of shear strength between the three types of specimens

It was also made a comparison between the shear strength obtained by the three specimens of walls (unreinforced, plastered and FRCM) and the shear strengths that are supposed to be obtained by the theoretical formulas, in order to verify the increase of shear strength and understand how much the theoretical solution differs from the reality.

The amount of shear strength of the unreinforced wall SC-2 was obtained by the formula given by the Peruvian code E.070 -26.3:

$$Vm = 0.5v_m \alpha t L + 0.23P_g$$

Where v_m is the characteristic shear strength of masonry, P_g the service gravity load, with reduced overhead, t is the effective thickness of the wall, L the total length of the wall, α the reduction factor for shear strength slenderness effects (in this case α=1 for higher safety standards).

v_m (kN/m²)	350
α (/)	1
t (m)	0,22
L (m)	1,2
P_g (kN)	218,75

Tab. 2.3 – Values of to obtain shear strength of SC-2

The amount of shear strength of the plastered wall SC-5 was obtained by adding to shear strength of the unreinforced wall, the amount of strength of the mortar. To obtain this value we have assumed that the behavior of the mortar is similar or almost equal to that of the concrete, being also a conglomerate of binder, water and sand. So we decided to use the formula given by the Peruvian code E.060 -13.2:

$$V_{mo} = 0.53\sqrt{f'c}\, bd$$

Where *f'c* is the compressive strength of the plaster, *b* is the thickness of the plaster and *d* is length of the wall.

v_m (kN/m²)	350
α (/)	1
t (m)	0,22
L (m)	1,2
P_g (kN)	267,46
f'_c (kN/m²)	7120
b (m)	0,02
d (m)	1,2

Tab. 2.4 – Values of to obtain shear strength of SC-5

The amount of shear strength of polymer grid reinforced wall SC-10 was obtained by the formula suggested by ACI 549.4R – 13.2:

$$V_f = 2nA_f L f_f$$

Where A_f is the area of the mesh reinforcement by unit width effective in shear, n is the number of layers of mesh reinforcement and L is the length of the wall in the direction of applied shear force.

To obtain the area of the mesh reinforcement by unit width effective in shear we have study the case of the grid which tension resistance in longitudinal direction in 47 kN/m. This specimen had a length of 21 cm, a Young's modulus E of 3207 MPa. After the test the ultimate load P_u result 4,7 kN and its strain δ was 19,5 mm. Since we know the definition of stress and the elastic law:

$$\sigma = \frac{P}{A}$$

$$\sigma = E\varepsilon$$

we can obtain, mixing these two formula, the area of the mesh reinforcement by unit width effective in shear:

$$A = \frac{PL}{E\delta}$$

In this way we obtained an area of A=0,157 cm²/m. This is almost the same value that we obtained knowing the width of grid and the amount of filaments present in 1 meter. In this case the specimen has a length of 90 mm, which means that in 1 meter we find 11 filaments.

b (mm)	h (mm)	A (mm²)	n	A_t (mm²)
1,1	1,2	1,32	11	14,52

Tab. 2.5 – Values of to obtain area of mesh

Now that we have all the values, we can calculate the amount of shear strength given by the polymer grid.

v_m (kN/m²)	350
α (/)	1
***t* (m)**	0,22
***L* (m)**	1,2
P_g (kN)	233,26
f'_c (kN/m²)	7120
***b* (m)**	0,02
***d* (m)**	1,2
n	2
A_g (m²/m)	1,4 E-05
f_{fv} (kN/m²)	412370

Tab. 2.6 – Values of to obtain shear strength of SC-10

Having all the values to obtain the shear strength of the three types of wall by theoretical formulas, now can be made the comparison between the shear strength obtained by the three specimens of walls and the shear strengths that are supposed to be obtained by the theoretical formulas.

Wall	V_{th}	V_{re}	Difference	%
Unreinforced	96,51	94,25	2,26	2,34
Plastered	109,86	106,81	3,05	2,77
FRCM	133,24	167,21	33,97	25,50

Tab. 2.7 – Difference of shear strength between the specimens and theory

From this comparison it is noted that the two formulas that express the value of shear strength of unreinforced wall and plastered wall describe almost the reality. In fact the difference is only about 2-3%. For the case of polymer grid reinforced wall the difference is about 25% which is a relevant amount, but reading the table is seen that the theoretical formula underestimate the real increase of shear, which means that the rest of strength not defined can be used for higher safety standards.

For this reason we consider the formula proposed by ACI 549.4R – 13.2 acceptable.

Chapter 3 – RISK PREVENTION PROGRAM IN PERU

Masonry is the oldest building system of humanity and especially in Peru, the brickwork is the most widely used for housing construction. Ease of construction using hand without computers or sophisticated machine tools, the economic cost of materials and flexibility of the construction process that allows do this in stages, has allowed hundreds of thousands of families have housing progressively according to their means. The brick masonry alone resists very well the gravity loads but when it is located in seismic areas such as Peru, requires additional reinforcements to adequately resist the horizontal loads of an earthquake.

In this area, the reinforcement system most commonly used is the confinement of masonry columns and beams of reinforced concrete that are intended to maintain the integrity of the masonry walls after his endurance limit shear strength is exceeded and produces cracking of the wall. The reinforcement beams and columns of reinforced concrete substantially increases the cost of housing by using additional materials such as crushed stone and steel and includes the intervention of skilled labor in the assembly and construction of reinforced concrete elements work.

Thus the vast majority of houses on the outskirts of the urban area were built informally without the participation of professionals or the application of building regulations. As a result they do not have the backup system or these are poorly constructed and not fully comply with its function, making them vulnerable to seismic risk and high forces in the event of large magnitude earthquakes occur.

3.1 - Bonus for protection of vulnerable housing earthquake risk

The country of Peru is located in a highly seismic area, where the probability of a major event occurring is very high. In this regard, seismic risk studies carried out by the CISMID -UNI show that off the coast of Peru and Chile are the most vulnerable areas, that due to the accumulation of energy that exists.

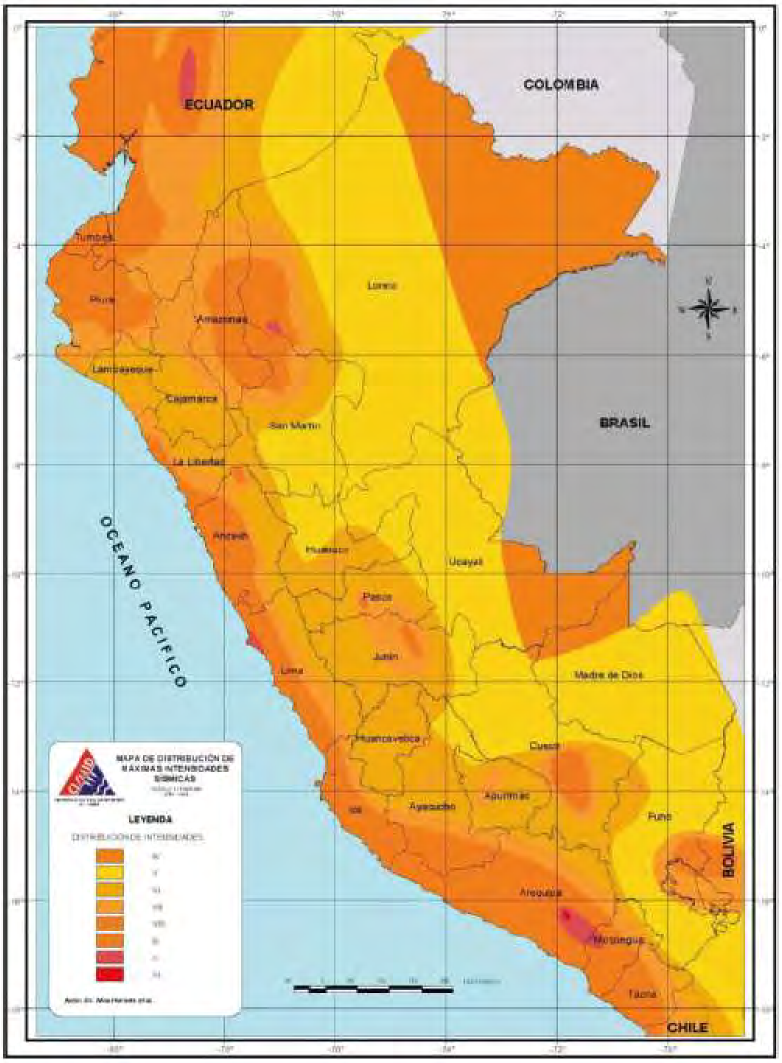


Fig. 3.1 – Distribution map of maximum seismic intensities

In Peru the possibility that buildings are affected by a seismic event is really high. The risk occurs as a result of uncontrolled growth of urban areas, of bad construction practices because are made by not qualified persons, of misuse of soil and inadequate implementation of foundations because the foundation structure of the houses are not for the soil which they have been implemented.

All this coupled with the construction of houses without permission and a financial system that fails to cover the majority of poor families and in extreme poverty, creates conditions that in the unlikely event of an earthquake, seismic waves would be amplified making that demands of dynamic forces increase on these edifications.

In this context, *Article 9 of Law N° 30191*, that establishes measures for the prevention, mitigation and adequate preparation for disaster response, provides a “Bonus for Protection of Vulnerable Housing Earthquake Risk” as part of the sectoral policy of the *Ministry of Housing, Construction* *and Sanitation* to reduce the vulnerability of the effects of the earthquake risk in favor of households in poverty, without charge to restitution by them, exclusively for interventions of structural reinforcement of houses located in soils vulnerable to seismic risk or that have been built in conditions of fragility.

The objectives of this project is to protect the lives of families living in poverty, reduce the material and personal damage caused by earthquakes, reinforcing a home environment.

Fig. 3.2 – Reinforcement of part of a house

The requirements and conditions for families to access the bonus are being owners of a house that need intervention, being in socioeconomic level classification 1 to 5 established by SISFOH (Sistema de Focalizaciòn de Hogares) and be within the standard housing to intervene, according to diagnosis of *Ministry of Housing, Construction and Sanitation*.

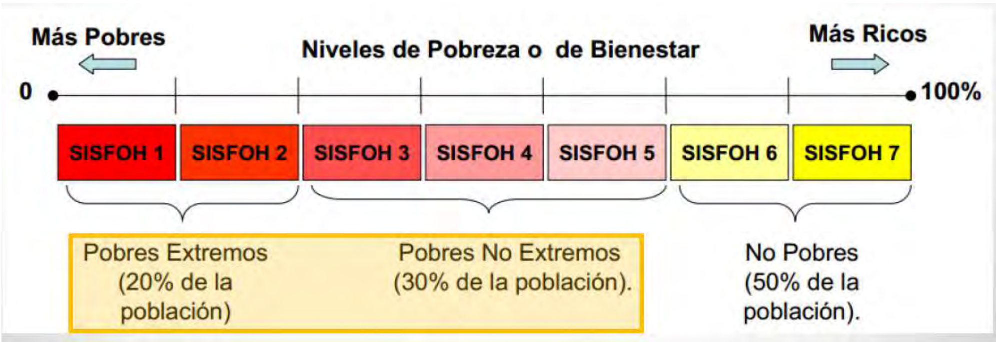


Fig. 3.3 – SISFOH classification

Due to the instability and fragility of informal housing constructions built on unstable soil and seismic risk which thousands of homes are exposed, it is proposed the reinforcement of brick masonry walls with structural polymer grid in a home environment masonry.

3.2 - Description of buildings

Within the project to reduce seismic vulnerability in the informal housing promoted by the government, the *Ministry of Housing* has conducted a campaign to identify homes that could access the Bonus of reinforcement from the technical, economic and legal standpoint.

The architectural style of informal housing in study due to the economic situation of owners. They are generally houses in progressive height growth and the type corresponds to the growth stage whose the building is.

One-story houses probably do not have reinforcement system that, as mentioned, consist in concrete confinement columns and beams. It is also probably that the roof is made by lighter provisional calamine. Households of two or more story are commonly provided with reinforced concrete columns and beams as elements of confinement and the last floor closed by a calamine corrugated lighter roof.

The houses are characterized by having no space to retreat and gain public roads by using projecting parts. This cause that the upper facade walls do not have confining columns because of the discontinuity. These homes are built without any parting between them that in case of earthquakes involves an interaction between several consecutive houses.

Fig. 3.4 – Discontinuity between first and second floor

The foundations of the walls is made by strip foundation and because many of the informal housing are located in slopes of the hill, there is the need to train platforms using retaining walls. It is common in the construction of these embankments the use of simple concrete or stone cairns.

Because the lots are not very large, they have small rooms with lights between 3-5 meters in length and heights ranging from 2,40 m and 2,50 m.

Usually they possess good wall density in the direction perpendicular to the front and lower density in the direction parallel to the facade. The walls are constructed of solid, artisanal and/or industrial brick on the first floor and is very usual to use hollow bricks on the upper floors, whose use for bearing walls is forbidden in seismic areas of the Peruvian coast according to code E.070. The finishes range from exposed brick, plastered and/or painted.

Fig. 3.5 – House with plastered and painted facade at first floor and exposed brick at second floor

Fig. 3.6 – House with just painted facade

It's not new that informality reigns in Peru, but that only in Lima there is about a million informal structures is a critical and worrying situation, putting on latent risk the lives of thousands persons. It was estimated that this amount of housing does not accomplish building standards, as being monitored during execution and design by architects and engineers. In addition, they are located in risky areas where the soil is rocky and are subject to flooding.

Examples of bad seismic performance in informal houses that can be prevented with surface reinforcement system are the following:

- Use of hollow brick for building bearing walls - This type of bricks are prohibited from using in load-bearing walls in earthquake zones because of their low bearing capacity and extreme fragility that makes them spray when they fail.

Fig. 3.7 – House with just painted facade

- No connection between roof and wall (light calamine roof) - Often last floor remain unbuilt and thus the walls of the top floor are not braced at the top. This produces the emptying side walls or parts thereof in case of earthquakes

Fig. 3.8 – Wall failures due the absence of rigid diaphragm

- Unconfined facade up the second level - A common construction defect not only in informal buildings but also in those with license, is the absence of elements of confinement and bracing in walls of the facades of buildings or houses with two or more floors. This is because on the second floor, the facade "flies" on the wall of the first floor to take advantage of more construction area, which results in that the columns of the facade of the first floor brake on the upper floors and the facade is simple masonry.

Fig. 3.9 – Seismic effects in unconfined walls

- Lack of joint between the wall and column confinement - This construction defect occurs when the column is built first and then the wall is raised between columns and emptied. The correct procedure is to first lift the wall leaving the indented edge and then pour the concrete between meshed walls. This defect does not allow the column fasten to the wall overturning itself.

Fig. 3.10 – Lack of joint between columns and walls in the upper storeys

- Absence of confinement elements like columns and beams - It is recognized that simple masonry is not suitable for use in seismic areas unless you have a backup system to prevent the disintegration of the walls. In this area the most common system is the use of columns and beams of reinforced concrete for confining walls.

Fig. 3.11 – Partial collapse of brick masonry house due to the absence of confinement

- Bad quality of materials or execution – Even if there are present confinement elements as beams and columns of reinforced concrete, it is no guarantee of good behaviour if other factors, such as poor quality of the masonry wall caused by faulty materials or bad execution.

Fig. 3.12 – Insufficient shear strength of brick wall

Fig. 3.13 – Partial emptying of several walls that cause the collapse of the second floor

3.3 - Technical proposal with FRCM

In seismic areas as Peru walls must withstand horizontal inertia forces, on its own plane and out of plane, caused by earthquakes. Since they have low tensile strength it should be included adequate reinforcement elements to prevent the collapse of the masonry when tractions exceed the bearing capacity of the wall. The slender unreinforced walls are very vulnerable to the forces of inertia caused by earthquakes.

3.3.1 - Advantages of polymer grid reinforcement

Masonry walls are subjected, in case of earthquakes, to inertial forces in its plane and inertia forces perpendicular to its plane. Coplanar inertia forces come from the total mass of the building and are distributed according to the relative rigidity of walls. The forces of inertia perpendicular to the plane of the wall come from the mass of each wall.

In Peru the technique of seismic reinforcement most used is confining masonry walls with beams and columns of reinforced concrete, which have the function of maintaining the unity of the wall avoiding dislocations and maintaining its ability to withstand vertical loads after the wall has started to crack because of shear force in their plane.

It should be noted that the confinement of masonry columns and reinforced concrete beams does not increase the ability to bear vertical load or the ability to resist shear forces caused by the earthquake, which depend only on the characteristics of the masonry.

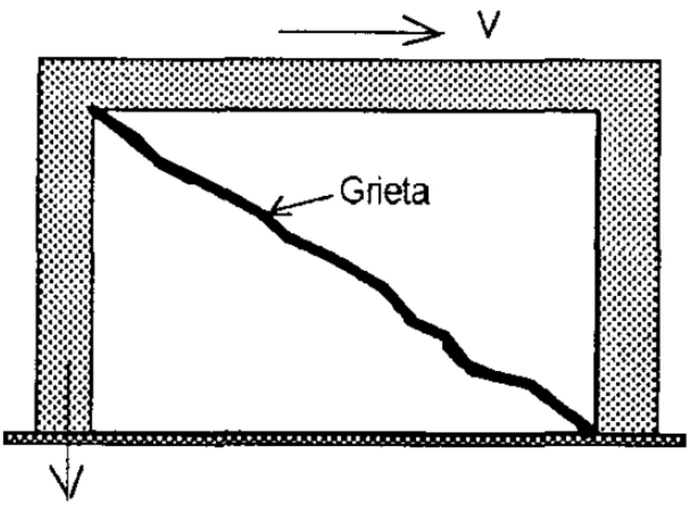


Fig. 3.14 – Crack scheme of confined wall

The surface reinforcement of masonry walls has been applied for several decades, being the most common the application of electro welded steel mesh embedded in mortar. In the last decade have been incorporated other materials of surface reinforcement as FRP, fibers reinforced polymer, applied with special resins, polymeric grid applied with a mortar, and other polymer materials embedded in resin or cement matrices.

Steel grids, polymeric or fibers meshes used as surface reinforcement have high tensile strength and reinforcement system are designed to transfer this resistance to masonry wall through mortar or resin adhered to the wall forming a composite resin.

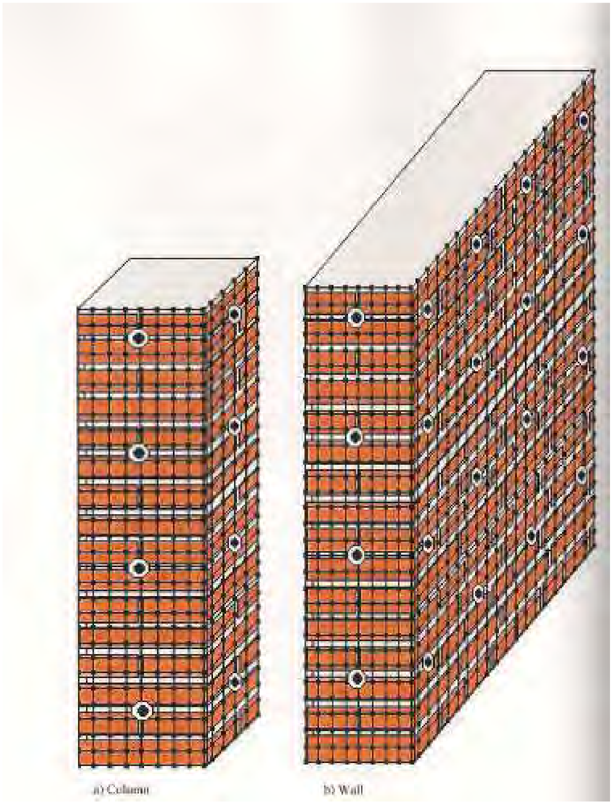


Fig. 3.15 – Superficial reinforcement with polymer grid

Depending if the transfer material is cement mortar or resin, the reinforcing effect is different. In the case of cement mortars, the original wall increases its thickness between 4-6 cm if applied on both sides of the wall which generates an increase in their ability to bear vertical and horizontal loads as has been corroborated by multiple experiences of laboratory, as seen in chapter 2. The relative importance of this increase of strength depends on the original wall thickness and characteristics. Lower thickness give less percentage of increase of resistance.

The most important contribution of surface reinforcement is the continuity of the wall beyond the limit shear strength preventing the disintegration of the wall and subsequent partial collapse. In Figures 3.16, 3.17 and 3.18 show the influence of the mesh in the type of failure of a wall with openings when there is no reinforcement and when it is reinforced with polymer grid. In this case, it shows that cracks are controlled by the reinforcement and cracks are more distributed than in the case of simple wall.

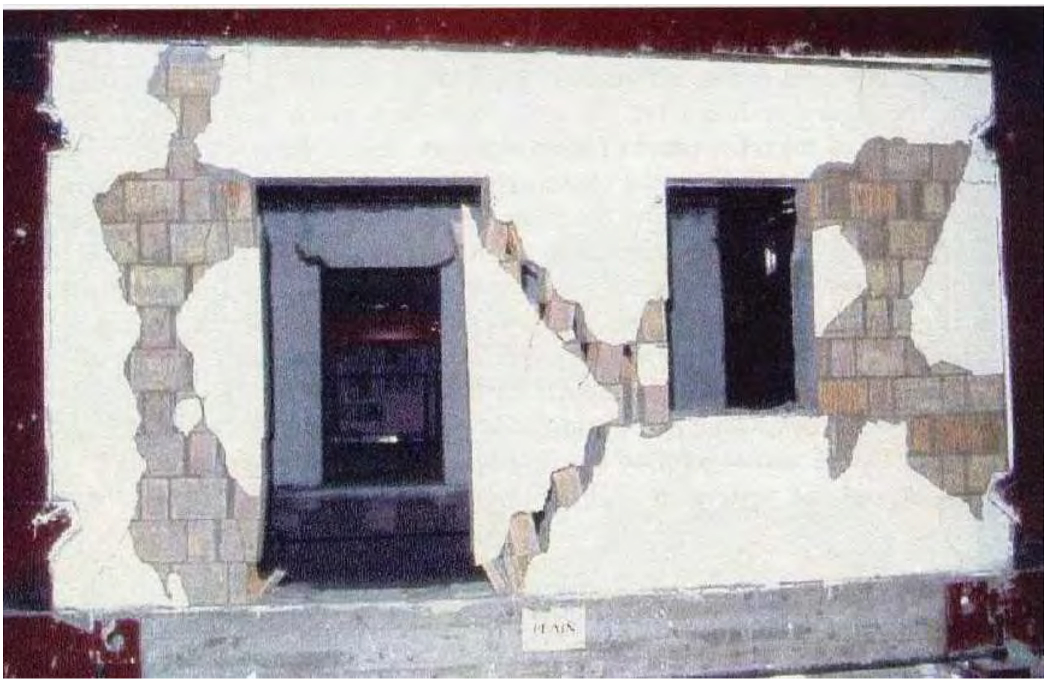


Fig. 3.16 – Unreinforced wall with openings submitted to horizontal cyclic force

Fig. 3.17 –Wall with openings reinforced with polymer grid

Fig. 3.18 – Grid reinforced wall with openings submitted to horizontal cyclic force

In the case of forces of inertia perpendicular to the plane of the wall that produced the overturning out of the plane, the polymer grid or surface reinforcement adhered on one or both sides of the wall, controls the out of plane bending keeping attached the wall and preventing lateral emptying.

Fig. 3.19 – Grid reinforced wall submitted to out of plane bending

When the reinforcement is placed on the side of the traction, it is produced and increasing of bending capacity of the wall, which depends on the strength and stiffness of the polymer grid used.

To carry out the objective of reducing the seismic vulnerability is enough that the wall be strengthened by one of the sides and that if the wall is affected by salt in its bottom, it must be ensured the permanence of the polymer grid at least in the upper half of the wall.

3.3.2 - Documents for validation

Affirmed the usefulness and practicality of this type of reinforcement, as well as to the economic benefits, it is to propose the polypropylene grid as a reinforcement for the selected houses that can take advantage bonus.

The aim is to officially validate the system of surface reinforcement with structural geogrid for new construction of brick masonry up to two levels as an alternative to confined masonry with reinforced concrete elements and validate also its application for the non-structural masonry (walls and parapets) in multi-storey buildings.

To validate the construction of new houses according to the "Regulations for approval of using construction systems unconventional" of *Ministry of housing, construction and sanitation* there are being made the following activities:

1. Collection of background investigations of surface reinforcement: Rules and regulations using other materials of surface reinforcement, implementation experiences, experimental research works.
2. Definition of the type of brick on which is applicable the reinforcement system. It involves the study of the adhesion of the mortar (mortar type) to the masonry.
3. Defining the minimum mechanical properties of the geogrid reinforcement.
4. Definition of architectural style using brick walls as the only support system of vertical and seismic loads. It involves appropriate density of walls in both directions.
5. Experimental determination of the behaviour and ultimate capacity of shear strength of reinforced walls by one and two surfaces with geogrid.
6. Define the level of energy dissipation in cyclic testing and comparing levels of confined walls to justify and use the same reduction factor R of seismic force of standard E.030

7. Seismic simulation demonstration test of a two storey construction
8. Analysis, design and construction of a full-scale model. It involves making construction plans with all details.
9. Development of technical specifications of the construction system. Much of this corresponds to the brickwork masonry exposed in E.070 standard, incorporating the surface geogrid reinforcement properties.

Particularly in section 3.2 of the “Regulations for approval of using construction systems unconventional” the technical documentation necessary for approval:

1. General system specification;
2. Technical specifications and Constructive;
3. Complete plans;
4. Structural Design Report, including the calculations;
5. Structural Testing Certificates awarded by a competent laboratory, and interpretive report of those results.

In particular, in the next chapter, it was decided to develop the point 4 of the requirements for the approval of the construction system (structural design), so it can be also demonstrate the usefulness of the reinforcement surface in the real case of entire house.

Chapter 4 – STRUCTURAL ANALYSIS

Within the project to reduce seismic vulnerability of informal housing promoted by the government, the Ministry of Housing has conducted a campaign to identify homes that could access the bonus of enforcement from an economic, technical and legally point of view. In order to satisfy the requirement for the approval of using construction systems unconventional, a structural analysis of a house that accomplish with the characteristic mentioned in the previous chapter, was made.

The selection of the prototype house was made in order to represent as much as possible the reality of Peruvian informal houses characteristic. This is also practical way to understand the benefit of this type of superficial reinforcement in a real case.

4.1 - Architectural distribution

For the analysis it was selected a prototype of informal house which encompasses all the characteristics mentioned above.

It is a two storey house, whose height between floors is 2,40 m and have a total height of 5,0 m.

The plan of house has a rectangular shape where the side of the house facing the street is wide 8,0 m meanwhile the other side is 13,87 m long. It has a total area of 111 m^2 for each floor.

The house consist of a living room, a kitchen, a dining room, a bedroom, two bathroom and two patios, in the ground floor. The second floor has instead 3 bedrooms and 2 bathrooms.

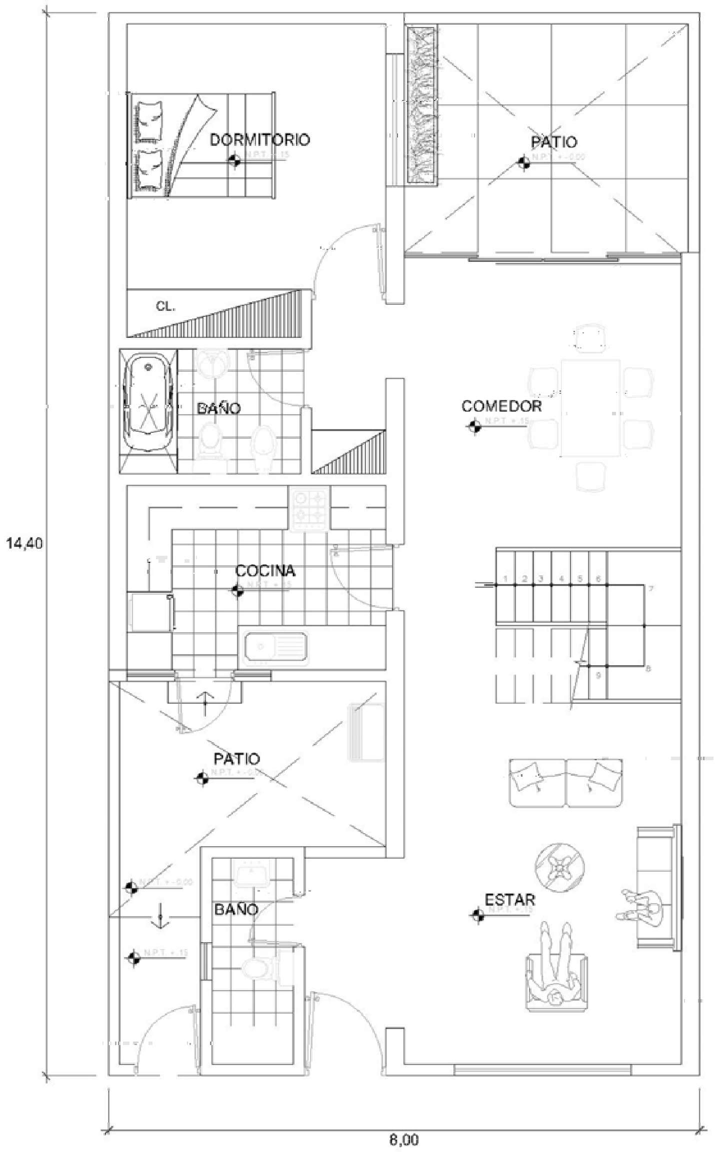


Fig. 4.1 – First floor architectural plan

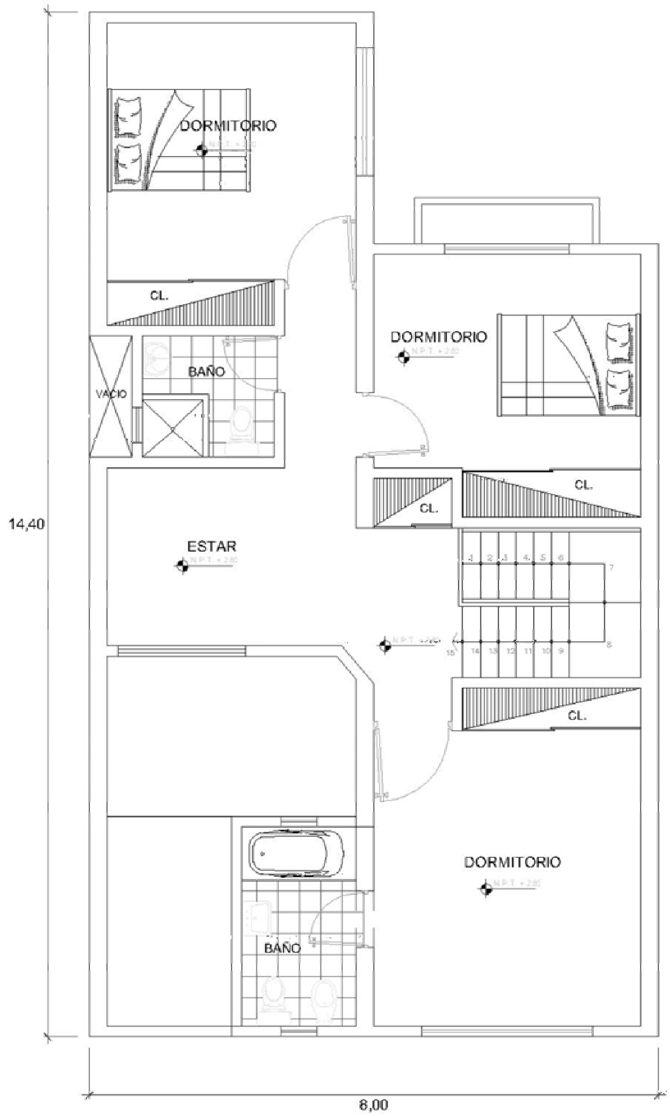


Fig. 4.2 – Second floor architectural plan

4.2 - Structural distribution

The house has a plant fairly regular from a structural point of view. Both floors have the same structural plan: it is composed longitudinally by 3 walls (3X) of a thickness of 25 cm while the transversely has only the outer walls of thickness of 15 cm.

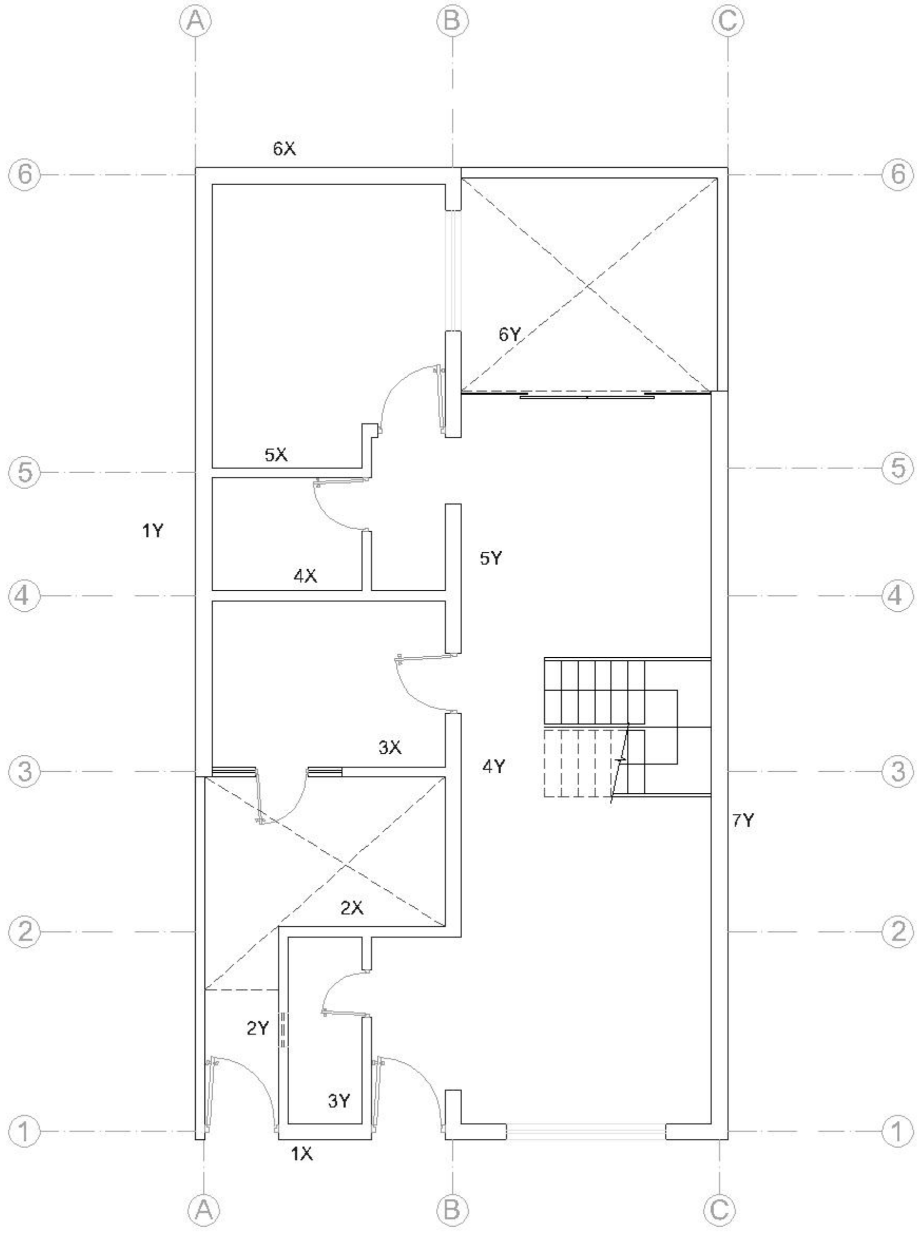


Fig. 4.3 –Structural plan

It is notice that, structurally, the house is developed mainly in the longitudinal direction. In fact it is constituted by three bearing walls with thickness of 25 cm. Moreover these walls are particularly long, that for masonry construction is a good feature. In transversal direction instead we find only two bearing walls, the outers.

Wall	*L* (m)	*t* (m)	*A* (m^2)
1X	1,40	0,25	0,35
2X	2,55	0,15	0,38
3X	1,68	0,15	0,25
4X	3,75	0,15	0,56
5X	2,45	0,15	0,37
6X	3,75	0,25	0,94
1Y	9,03	0,25	2,26
2Y	3,00	0,15	0,45
3Y	1,73	0,15	0,26
4Y	3,28	0,25	0,82
5Y	2,25	0,25	0,56
6Y	1,60	0,25	0,40
7Y	11,13	0,25	2,78

Tab. 3.1 –Length, thickness and section area of each wall

4.3 - Numerical modelling with FEM

To evaluate the performance of the building, like in the majority of cases, it was used, as now more and more frequently, a special software for structural analysis which is based on a numerical technique called FEM (Finite Element Method). This method solves the system of differential equations equilibrium, congruence and tie that govern the statics of continuous deformable.

The main problems related to the use of the FEM for masonry construction are the definition of the type of element to be used (mono-dimensional, frame equivalently method, bi-dimensional or three-dimensional) and the difficulty in identifying the mechanical behaviour of wall, which means to make an adequate choice of the constitutive law of the material (linear-elastic, elasto-plastic, material resistant or not to traction).

The model for the analysis considers a spatial distribution of mass and stiffness which are suitable for calculating the most significant aspects of the dynamic behaviour of the structure. For buildings in which one can reasonably assume that the systems work as rigid diaphragms floor, you can use a lumped mass model with three degrees of freedom diaphragm, associated with two orthogonal components of horizontal translation and rotation. In this case, deformations of the elements must be reconciled by rigid diaphragm condition and the physical layout of the horizontal forces shall be in terms rigidities of the resistant elements. In this case we suppose that the diaphragms have sufficient rigidity and strength to ensure the distribution mentioned otherwise be taken into account flexibility for the distribution of seismic forces.

The building was modelled with the aid of SAP2000 software, considered one of the best programs in the field of theory of construction for its flexibility in application to the most disparate real cases.

It is specified that for the modelling there have been taking only walls with structural function.

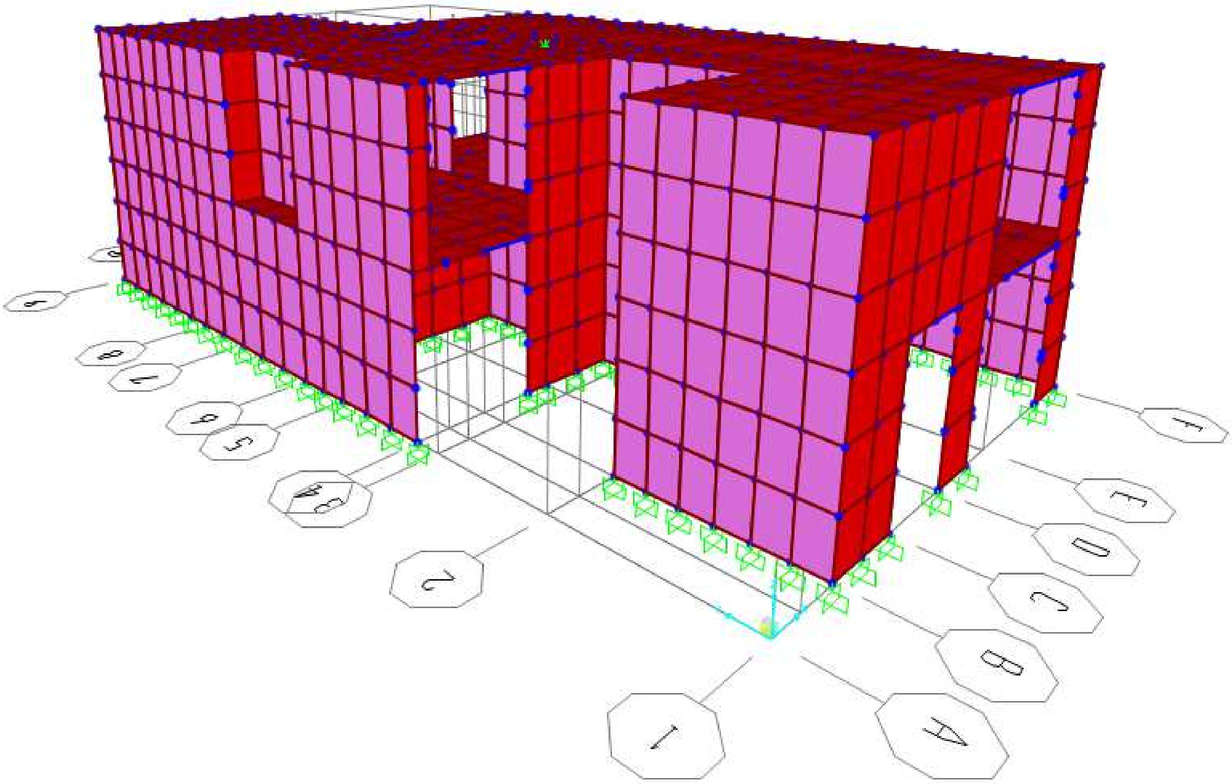


Fig. 4.4 – 3D view of the front prototype house

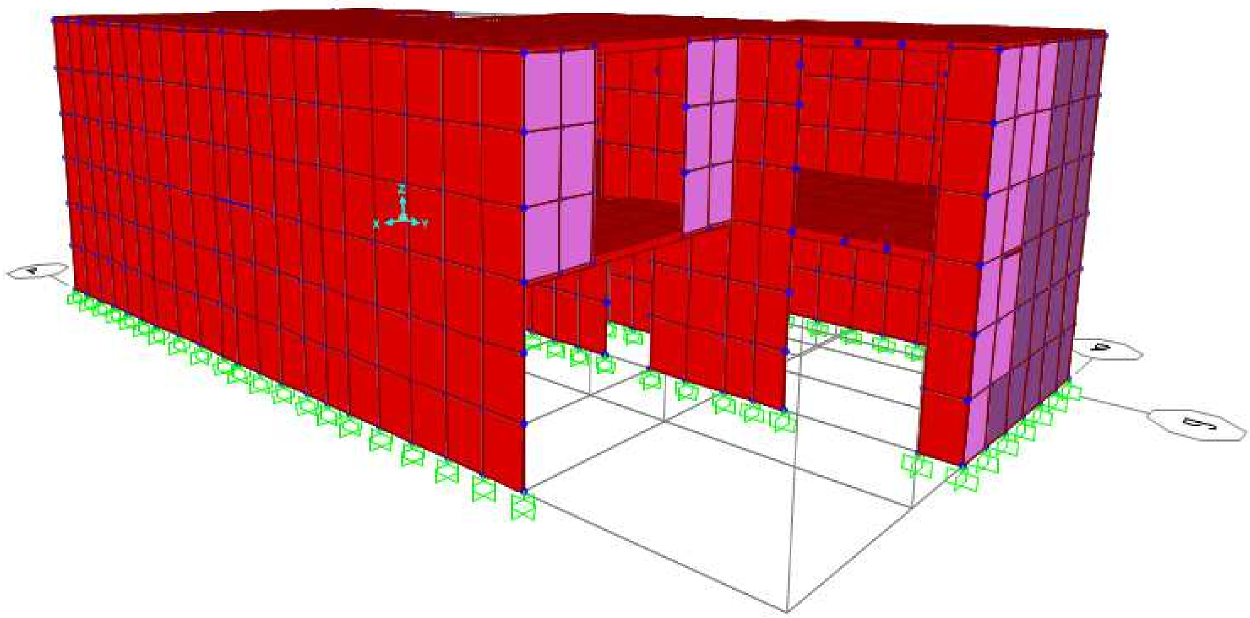


Fig. 4.5 – 3D view of the back of prototype house

For the modelling of the building it was used *Shell* elements, which, having a high computational burden, are suitable for this particular case because the building is not complex. The element's membrane response is treated with an assumed strain formulation that gives accurate solution to in-plane bending problems, is not sensitive to element distortion and avoid locking.

Only walls with structural function were designed, and areas where there were opening are not consider as structural wall. All the walls were connected by a rigid diaphragm that represents slab.

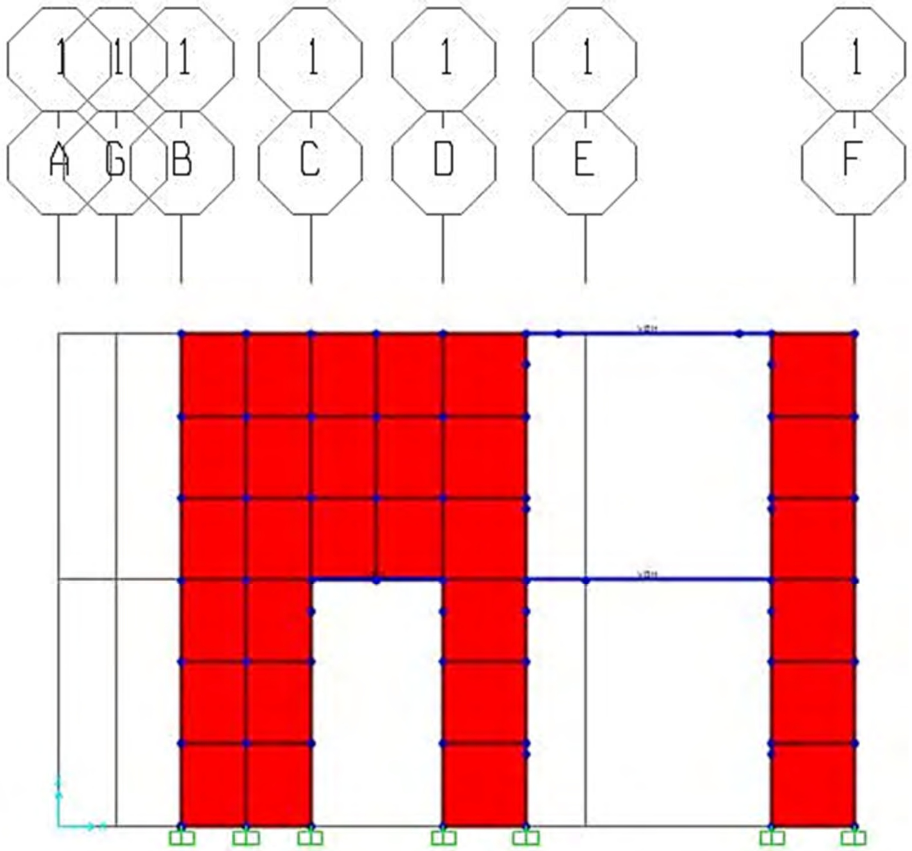


Fig. 4.6 – Particular of connection between wall with and without openings

To define the type of material, which identifies the mechanical behaviour of elements, it was decided to use a linear static constitutive law which, however, is not resistant to traction.

To simulate the soil effect it was decided to impose on ground joints a fixed restrain, which, even if it overestimates the stresses of walls, to the simplicity of the case is fine.

In order to obtain an acceptable solution, an appropriate mesh it has been made.

4.4 - Structural analysis

For structural analysis we assume that the house rigid diaphragm realized by a lightweight concrete roof. To begin the structural analysis it has been made a Load Analysis:

Specific weight of masonry	18,0 kN/m³
Dead load:	
Slab lightened 15+5	2,8 kN/m²
Finishes	1,0 kN/m²
Total	3,8 kN/m²
Live load:	
First floor	2,0 kN/m²
Rooftop	1,0 kN/m²
Total Area for each floor	85,7 m²

Referencing to the Masonry Peruvian Code E.070, the prototype house fulfils all minimum structural requirement.

According point *19.1.a* the effective thickness of a wall must be:

$$t \geq \frac{h}{20}$$

Were *h* is the clearance between the horizontal elements, which is 2,4 m and *t* result 12 cm. This value is less than thickness of all walls presented in the house (the finest element has 15 cm of thickness).

According to 19.1.b the maximum axial stress σ_m caused by the maximum gravity load, including the 100% overload must be:

$$\sigma_m = \frac{P}{Lt} \leq 0{,}2\ f'_m \left[1 - \left(\frac{h}{35t}\right)^2\right] \leq 0{,}15\ f'_m$$

Where *L* is the total length of the wall and f'_m is the characteristic compressive strength of masonry, that for this case is 3,5 MPa.

The values of axial load has been taking by results of the SAP2000 modelling with a linear static analysis when just subjected to the gravity load.

Since subjected to the maximum load, only the first floor was studied.

The values of maximum stresses of the two types of walls are σ_{25}= 687 kN/m² and σ_{15}= 554 kN/m² but since they are higher than 0,15 f'_m the value of maximum stress is σ= 525 kN/m².

In the following table it is seen that all walls fulfil this requirement.

Wall	L_x (m)	*t* (m)	*A* (m²)	*N* (kN)	*σ* (kN/m²)	*E.070*
1X	1,40	0,25	0,35	61,4	175,4	<525 kN/m² ✓
2X	2,55	0,15	0,38	86,4	225,8	<525 kN/m² ✓
3X	1,68	0,15	0,25	46,7	185,7	<525 kN/m² ✓
4X	3,75	0,15	0,56	112,3	199,6	<525 kN/m² ✓
5X	2,45	0,15	0,37	67,3	183,0	<525 kN/m² ✓
6X	3,75	0,25	0,94	138,3	147,5	<525 kN/m² ✓

Tab. 3.2 – Verification of walls in X direction

Wall	L_y (m)	*t* (m)	*A* (m²)	*N* (kN)	*σ* (kN/m²)	*E.070*
1Y	9,03	0,25	2,26	278,8	123,5	<525 kN/m² ✓
2Y	3,00	0,15	0,45	71,0	157,9	<525 kN/m² ✓
3Y	1,73	0,15	0,26	40,7	157,2	<525 kN/m² ✓
4Y	3,28	0,25	0,82	121,9	148,8	<525 kN/m² ✓
5Y	2,25	0,25	0,56	45,4	80,7	<525 kN/m² ✓
6Y	1,60	0,25	0,40	91,9	229,7	<525 kN/m² ✓
7Y	11,13	0,25	2,78	365,3	131,3	<525 kN/m² ✓

Tab. 3.3 – Verification of walls in Y direction

According point *19.2.b* brick masonry construction must have a minimum density of reinforced walls to ensure a state of repairable damage after a severe earthquake for which each main direction of the ground floor, the minimum density of reinforced walls is determined by the following expression:

$$\frac{\sum Lt}{A_p} \geq \frac{ZUSN}{56}$$

Where *Z* is the maximum ground acceleration with a 10% probability of being exceeded in 50 years, in the case of Peruvian coast Z= 0,4; *U* is the factor that define the use and importance of the building, in the case of house U=1; *S* is the soil profiles, in the case of intermediate soil S=1,2; *N* is the number of floors, in this case N=2. As said before, the total area of the plant is 111m².

	ΣLt/At	ZUSN/56
Longitudinal direction	0,026	>0,017 ✓
Transversal direction	0,068	>0,017 ✓

Tab. 3.4 – Verification of density walls

In both directions this requirement is satisfied.

With those verifications we consider the fulfil the minimum structural requirements according Peruvian Code E.070.

4.5 - Seismic analysis

The Peruvian code E.030 establishes the minimum requirements for the design of houses with a proper seismic behaviour in order to:

1. Protect life
2. Ensure continuity of basic services
3. Minimize property damage

It applies to the design of all new buildings, of evaluation and strengthening of existing buildings and of those resulting damaged by the action of earthquake.

It is recognised that give complete protection against all earthquakes is not technical and economical feasible for most structures. In keeping with that philosophy there are two principle for design:

- The structure should not collapse or cause serious damage to persons due to severe earthquake that can occur at the site;
- The structure should withstand moderate earthquake, which may occur at the site during its service life, experiencing damage within acceptable limits.

Said that, given the regularity of the building, it will make a linear static analysis where the seismic acceleration was simplified by equivalent static forces placed in the centre of gravity of each floor.

4.5.1 - Determination of the seismic force

To determine the equivalent static forces V various parameters are needed.

Peru is a country considered divided into three zones, as shown in Figure 4.7. The proposed zoning is based on the spatial distribution of the observed seismicity, the general characteristics of earthquakes and attenuation of these with the epicentral distance and in neotectonics information.

Fig. 4.7 – Three zone of Peru

Each zone is assigned a Z-factor as indicated in Table 3.5. This factor is interpreted as the *Maximum Ground Acceleration* with a 10% probability of being exceeded in 50 years.

Factors zone	
Zone	*Z*
3	0,4
2	0,3
1	0,15

Tab. 3.5 – Factors zone

Since we are considering informal houses on the outskirts of the urban area of Lima, we are in zone 3 which means have a Z=0,4

Each type of structure must be classified according to the categories listed in the E.030 code. The type of *Use and Importance Coefficient* (U) that is used is for public buildings, whose failure would cause loss of middle level as homes, offices, hotels, restaurants, warehouses and industrial facilities whose failure can not cause additional fire hazards, leaking contaminants, is U=1

The soil profiles are classified taking into account the mechanical properties of soil, layer thickness, the fundamental period of vibration and the speed of propagation of shear waves. In this case we have consider a S_2 profile which have a Intermediate soil. This type of sites have characteristics intermediate between rocks and very rigid soils with propagating wave rock similar to a court (where the key to low-amplitude vibrations period not exceeding 0,25 s), and soft soil or thick layers (where the fundamental period, for low amplitude vibrations, is greater than 0,6 s). For this type of soil, the Peruvian code gives the *Soil Parameter* S=1 and the *Propagation Period* T_p=0,6.

According to the characteristics of the site, the *Seismic Amplification Factor* (C) is defined by the following expression:

$$C = 2{,}5 \cdot \left(\frac{Tp}{T}\right) \leq 2{,}5$$

Where T_p is the propagation period, which in this case is 0,6 s, and T is fundamental period, which in our case in 0,08 s. Since the value of C is higher of the limit proposed by the code, the value of the seismic amplification factor taken is C= 2,5.

The structural systems are classified according to the materials used and the prevailing system of seismic structure. According to the classification made of the building a coefficient of *Seismic Force Reduction* R will be used.

In the legislation there are mentioned steel frames, walls or frames of reinforced concrete, masonry and wood. In particular there is a specification about masonry: it can be confined or steel reinforced. Not actually being in neither case it was decided to not adopt any reduction factor, obtaining a seismic force much higher. I want to point out that in any case, compared to the other values specified in the table, the reduction which would have had in the case of masonry is not so significant. So in this case R=1.

Tabla N° 6 SISTEMAS ESTRUCTURALES	
Sistema Estructural	Coeficiente de Reducción, R Para estructuras regulares (*) (**)
Acero	
Pórticos dúctiles con uniones resistentes a momentos.	9,5
Otras estructuras de acero:	
Arriostres Excéntricos.	6,5
Arriostres en Cruz.	6,0
Concreto Armado	
Pórticos(1).	8
Dual(2)	7
De muros estructurales (3)	6
Muros de ductilidad limitada (4).	4
Albañilería Armada o Confinada(5).	3
Madera (Por esfuerzos admisibles)	7

Tab. 3.6 – Different types of structural system

The weight P was calculated by adding to the dead load of building a percentage of the live load, which for roofs and ceilings in general is the 25% of live load. In this case the total weight, included the masonry permanent load, is P=1586 kN for the entire building.

Now it can be determine the total shear force at the base of the structure, corresponding to the direction of interest, by the following expression:

$$V = \frac{ZUCS}{R} P$$

This formula allows to obtain the total shear equivalent force from the values previously specified of V= 1586 kN.

Since the fundamental period T is greater than 0,7 s, a part of the shear V, called F_a, should be applied as a concentrated force on top of the structure. This force Fa is determined by the expression:

$$F_a = 0{,}07TV \leq 0{,}15V$$

The concentrated force F_a results 8,9 kN, which is less than 0,15V.

The remaining shear seismic force will be distributed between the two different level, in function of height and weight, according the following expression:

$$F_i = \frac{P_i h_i}{\sum_j P_j h_j}(V - F_a)$$

Where P_i is the weight of each floor, h_i the height of each floor and α_i is the ratio between $P_i h_i / \Sigma P_j h_j$.

Floor	h_i(m)	P_i (kN)	$P_i h_i$(kNm)	α_i(/)	F_i(kN)
2°	4,8	782,45	3755,74	0,660636	1042,11
1°	2,4	803,87	1929,30	0,339364	541,35
		$\Sigma P_i h_i$*(kNm)*	5685,04		

Tab. 3.7 – Determination of seismic force for each floor

These two values of forces were applied to the modelling and it was made a linear static analysis.

4.5.2 - Stresses of walls

As mentioned before, the storey equivalent forces, obtained by the formula given by the Standard, were applied to the model. Solving the problem with a linear static analysis, we have obtained different results.

By applying the force in the X direction, it is noted that only the walls with development in X have undergone considerable stresses, unlike those with longitudinal development which have stresses practically negligible.

Subsequently there are represent the results obtained for each wall in X direction.

Wall 1X

Shear stress F_{1x}= 91,77 kN

Bending moment M_{1x}= 7,27 kNm

TABLE: Joint Reactions								
Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
1	Sisma en X	LinStatic	-36,722	-10,296	-127,688	-0,1274	-1,8984	-0,0012
2	Sisma en X	LinStatic	-27,879	1,7	74,785	0,2786	-1,8209	0,000228
269	Sisma en X	LinStatic	-27,168	0,004326	-32,479	0,000142	-3,5522	-0,001

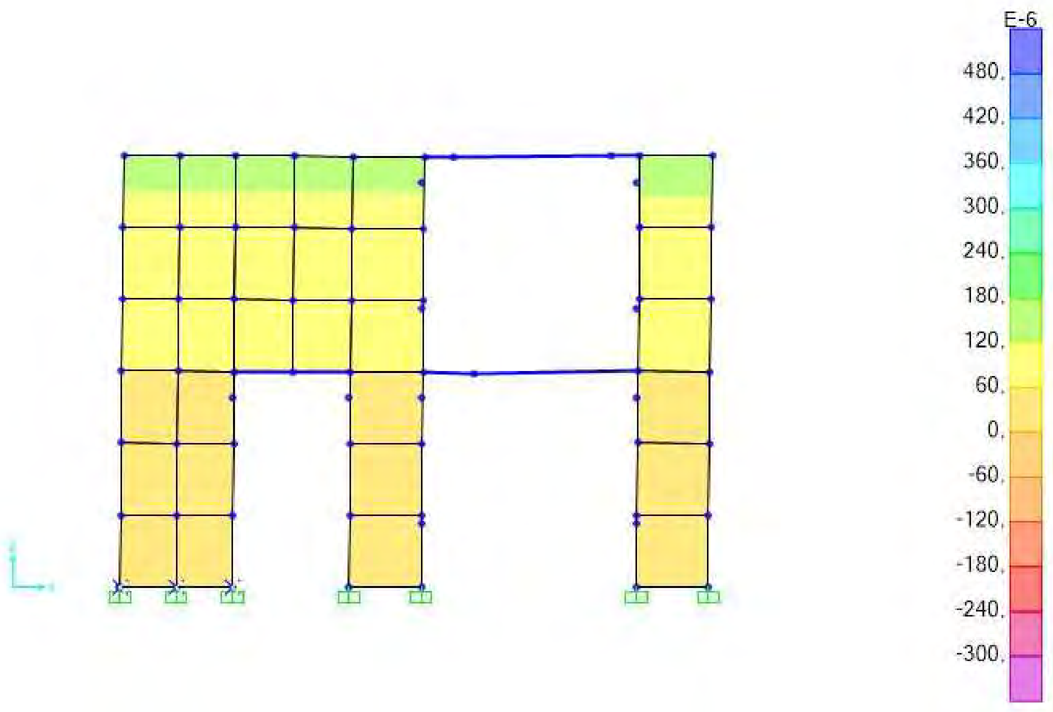


Fig. 4.8 – Shear stress of wall 1X

Wall 2X

Shear stress F_{2x}= 118,09 kN

Bending moment M_{2x}= 13,06 kNm

TABLE: Joint Reactions								
Joint	OutputCase	CaseType	F1	F2	F3	M1	M2	M3
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
16	Sisma en X	LinStatic	-32,735	10,982	-109,476	0,1206	-1,4713	-8,11E-05
17	Sisma en X	LinStatic	-41,613	-0,143	-3,794	0,0044	-2,8472	-1,60E-05
20	Sisma en X	LinStatic	-32,12	14,986	142,131	0,1476	-1,3684	-0,0021
352	Sisma en X	LinStatic	-38,337	-0,00101	-52,798	0,000144	-3,3903	-1,99E-05
360	Sisma en X	LinStatic	-44,144	-0,00526	44,525	-6,23E-05	-3,9832	-0,00028

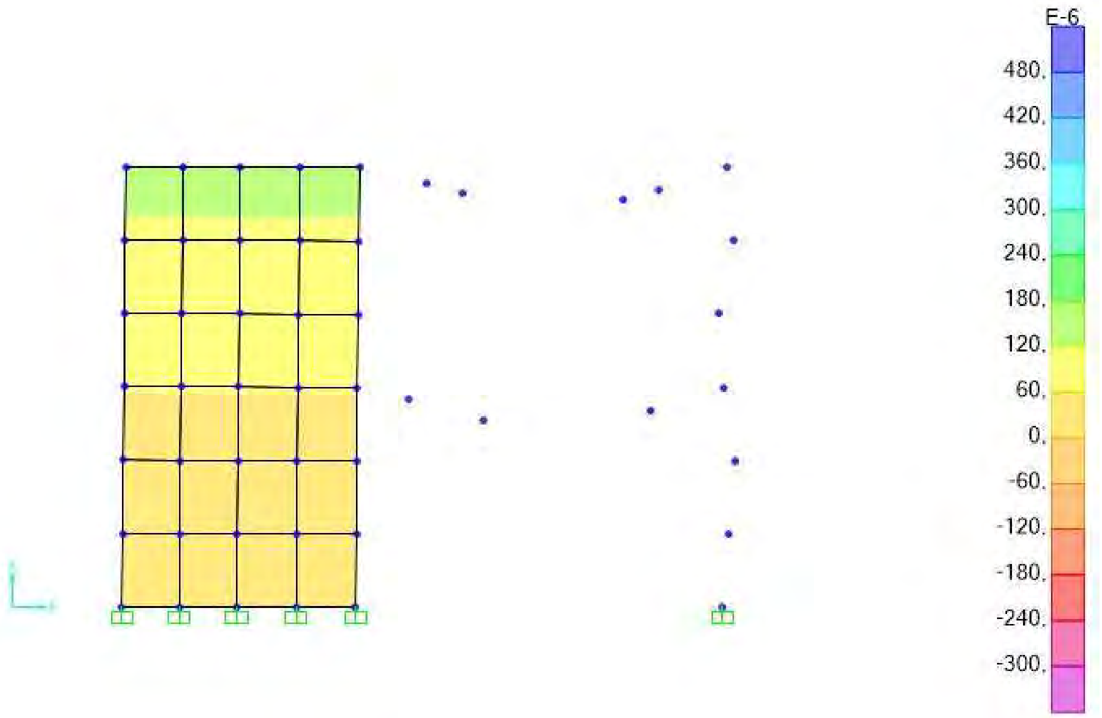


Fig. 4.9 – Shear stress of wall 2X

Wall 3X

Shear stress F_{3x}= 83,67 kN

Bending moment M_{3x}= 6,47 kNm

TABLE: Joint Reactions								
Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
29	Sisma en X	LinStatic	-13,807	-0,00099	-81,595	-3,55E-05	-1,5045	-2,98E-05
31	Sisma en X	LinStatic	-18,988	-9,668	79,272	0,494	-1,3035	0,0135
165	Sisma en X	LinStatic	-25,434	0,001176	-58,676	-0,00018	-0,5559	5,05E-05
378	Sisma en X	LinStatic	-25,442	0,000405	-22,526	0,000248	-3,1109	0,000107

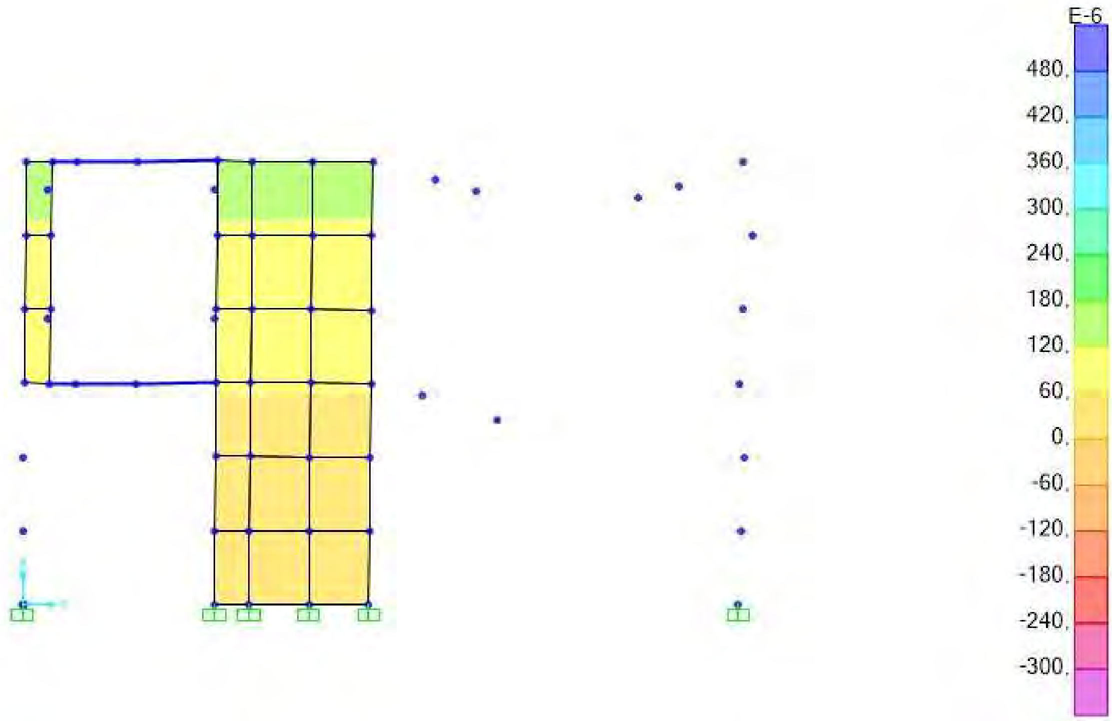


Fig. 4.10 – Shear stress of wall 3X

Wall 4X

Shear stress F_{4x}= 106,52 kN

Bending moment M_{4x}= 27,63 kNm

TABLE: Joint Reactions								
Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
37	Sisma en X	LinStatic	-34,214	-3,77	-115,272	0,2964	-1,806	0,000689
38	Sisma en X	LinStatic	-62,269	1,46E-05	4,877	3,22E-05	-3,8216	-1,77E-06
41	Sisma en X	LinStatic	-73,244	6,644	58,446	-0,0269	-4,0314	0,000118
43	Sisma en X	LinStatic	-46,845	-2,006	145,163	-0,1607	-2,0597	0,000564
442	Sisma en X	LinStatic	-83,433	0,0001	53,343	-6,37E-05	-6,4402	2,99E-07
450	Sisma en X	LinStatic	-57,789	0,000531	-19,631	-4,74E-05	-4,3901	-1,96E-06
458	Sisma en X	LinStatic	-68,294	0,000126	23,853	6,32E-06	-5,0804	3,09E-06

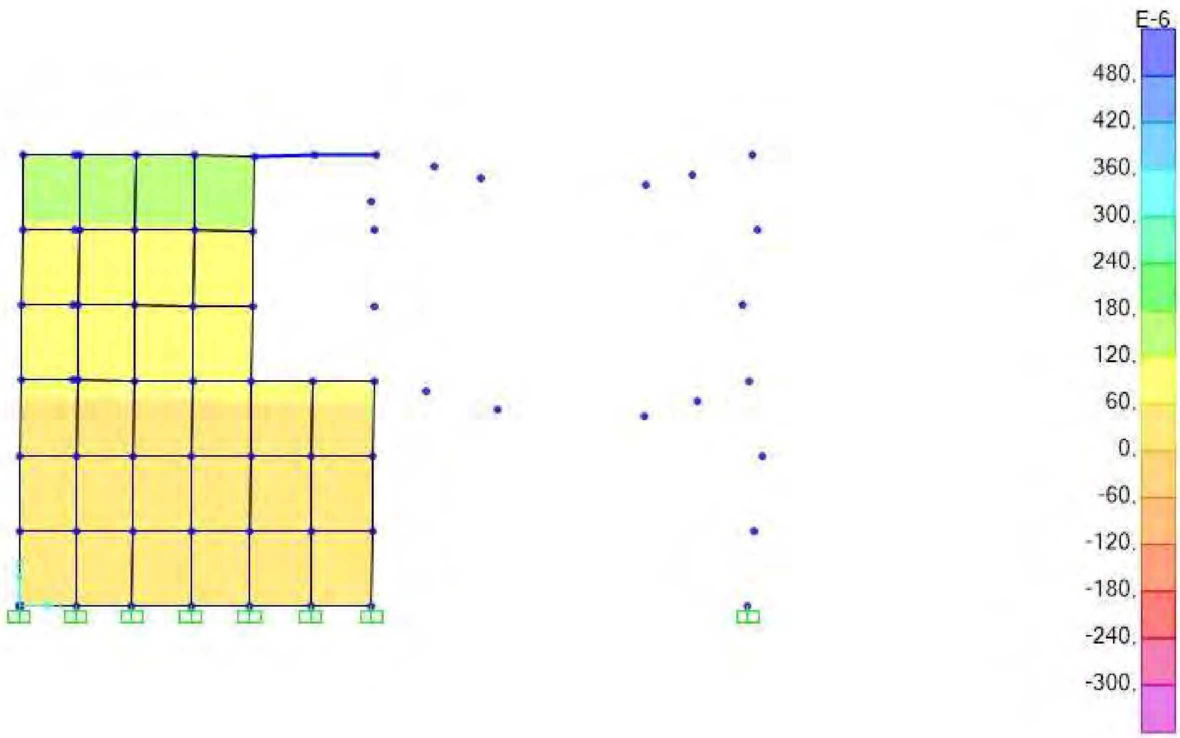


Fig. 4.11 – Shear stress of wall 4X

Wall 5X

Shear stress F_{5x}= 110,05 kN

Bending moment M_{5x}= 11,89 kNm

TABLE: Joint Reactions								
Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
48	Sisma en X	LinStatic	-22,572	-0,19	-104,535	-0,0275	-1,4215	0,000801
49	Sisma en X	LinStatic	-35,097	0,000464	59,008	-0,00065	-3,0683	5,13E-06
52	Sisma en X	LinStatic	-43,358	0,001623	121,073	-0,00091	-1,5785	-0,00022
474	Sisma en X	LinStatic	-33,974	-0,00195	6,018	6,09E-05	-2,8878	1,36E-05
482	Sisma en X	LinStatic	-30,07	0,002071	133,968	-0,0014	-2,9358	6,09E-06

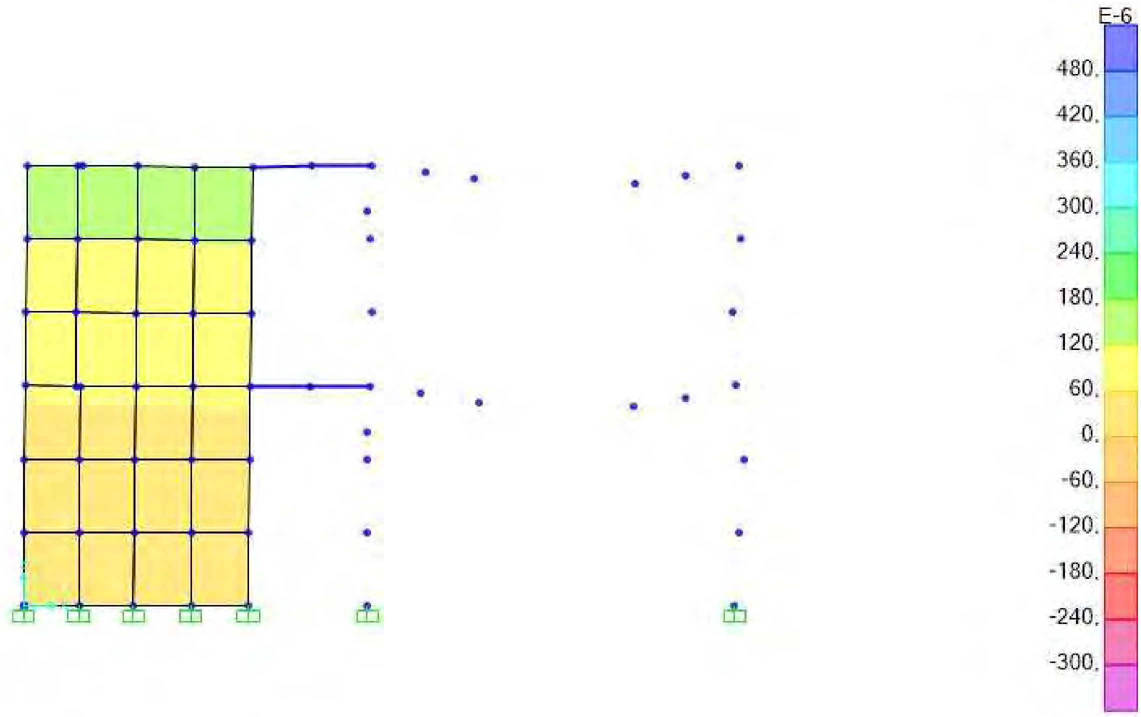


Fig. 4.12 – Shear stress of wall 5X

Wall 6X

Shear stress F_{2x}= 290,08 kN

Bending moment M_{2x}= 39,31 kNm

TABLE: Joint Reactions								
Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
68	Sisma en X	LinStatic	-61,201	12,476	-192,745	0,5631	-2,7985	-0,00027
69	Sisma en X	LinStatic	-80,488	-0,064	-105,357	0,0505	-6,3893	0,0183
72	Sisma en X	LinStatic	-90,149	-0,0083	-31,268	-0,0204	-6,5215	0,0079
74	Sisma en X	LinStatic	-89,639	-0,021	97,513	-0,0051	-6,4255	-0,0021
76	Sisma en X	LinStatic	-81,359	-26,774	309,954	-0,6703	-3,3678	0,0048
527	Sisma en X	LinStatic	-88,141	-0,071	31,804	0,0616	-6,5619	-0,002
535	Sisma en X	LinStatic	-89,173	-0,044	193,42	0,0323	-7,2475	-0,016

Fig. 4.13 – Shear stress of wall 6X

By applying the force in the Y direction, it is noted that only the walls with development in Y have undergone considerable stresses, unlike those with transversal development which have stresses practically negligible.

Subsequently there are represent the results obtained for each wall in Y direction.

Wall 1Y

Shear stress F_{1y}= 512,16 kN

Bending moment M_{1y}= 33,67 kNm

TABLE: Joint Reactions								
Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
25	Sisma en Y	LinStatic	-0,00571	-22,253	-57,01	0,7133	0,000801	-2,84E-05
37	Sisma en Y	LinStatic	0,91	-32,247	-2,624	2,1551	0,1378	0,001
48	Sisma en Y	LinStatic	2,585	-40,293	6,082	2,5958	0,1272	0,000435
68	Sisma en Y	LinStatic	6,717	-22,837	76,014	0,9577	-0,1151	-3,50E-06
87	Sisma en Y	LinStatic	0,017	-24,483	-17,571	1,7858	0,0133	-0,0015
88	Sisma en Y	LinStatic	0,016	-35,356	6,279	2,3242	0,0033	0,0026
583	Sisma en Y	LinStatic	0,051	-31,551	9,629	2,0896	0,0437	0,0017

588	Sisma en Y	LinStatic	0,024	-30,79	14,765	2,0993	0,0084	-0,00064
591	Sisma en Y	LinStatic	0,062	-30,199	21,042	2,0348	0,0548	0,0049
594	Sisma en Y	LinStatic	-0,0015	-29,861	29,304	2,1061	-0,0272	-0,0083
597	Sisma en Y	LinStatic	0,064	-27,294	45,203	2,0151	0,0544	-0,0144
610	Sisma en Y	LinStatic	-0,00601	-24,642	-7,77	1,7159	-0,0098	-0,0042
615	Sisma en Y	LinStatic	0,024	-39,271	4,038	2,5158	0,0224	0,0069
634	Sisma en Y	LinStatic	0,017	-18,374	-63,293	1,7032	0,0118	0,0036
639	Sisma en Y	LinStatic	0,007652	-23,823	-33,953	1,7694	-0,00079	-0,00028
652	Sisma en Y	LinStatic	-0,011	-39,055	-0,658	2,5187	-0,0171	0,001
656	Sisma en Y	LinStatic	0,014	-39,829	1,211	2,5735	0,0102	-0,0046

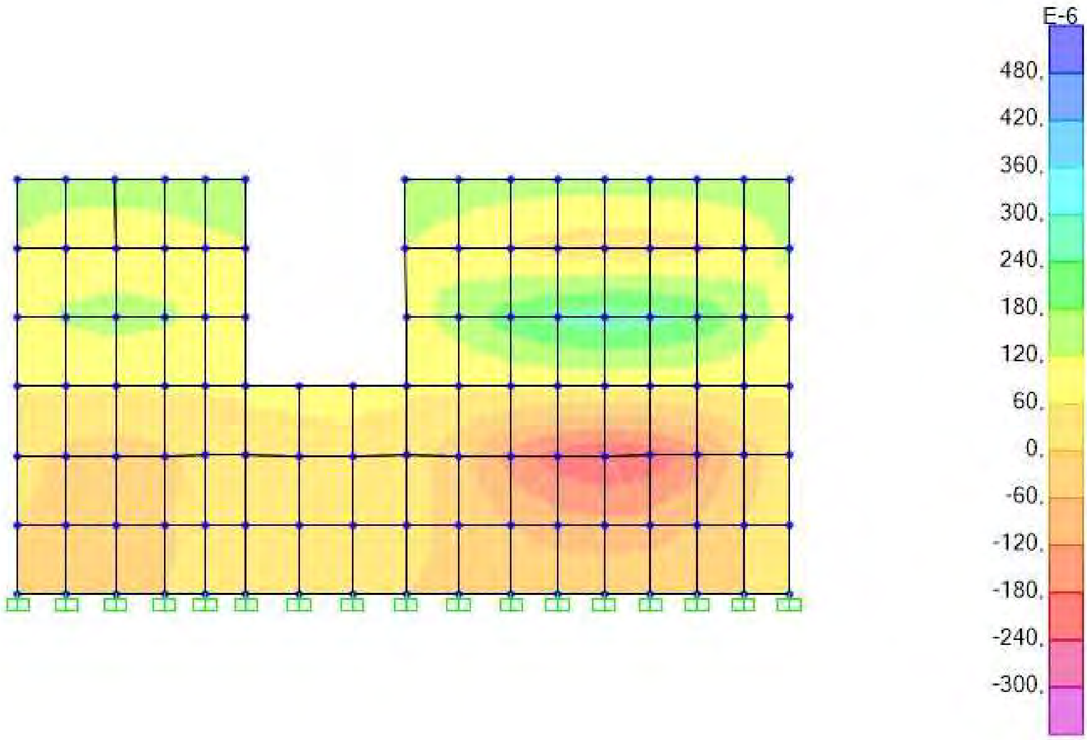


Fig. 4.14 – Shear stress of wall 1Y

Wall 2Y

Shear stress F_{2y}= 89,95 kN

Bending moment M_{2y}= 6,11 kNm

TABLE: Joint Reactions								
Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
1	Sisma en Y	LinStatic	-2,192	-9,563	-33,809	0,4966	0,1618	-0,00077
16	Sisma en Y	LinStatic	0,765	-11,154	35,265	0,491	-0,0647	-3,95E-05

660	Sisma en Y	LinStatic	0,000168	-13,276	-10,334	1,0015	8,64E-05	5,27E-06
664	Sisma en Y	LinStatic	0,000216	-14,036	-2,392	1,025	0,000188	-2,87E-06
668	Sisma en Y	LinStatic	0,000436	-14,197	3,635	1,0173	0,000247	-3,67E-07
672	Sisma en Y	LinStatic	0,000251	-14,305	9,813	1,0493	0,000188	2,25E-06
676	Sisma en Y	LinStatic	-0,00016	-13,421	18,823	1,0251	6,23E-05	1,31E-06

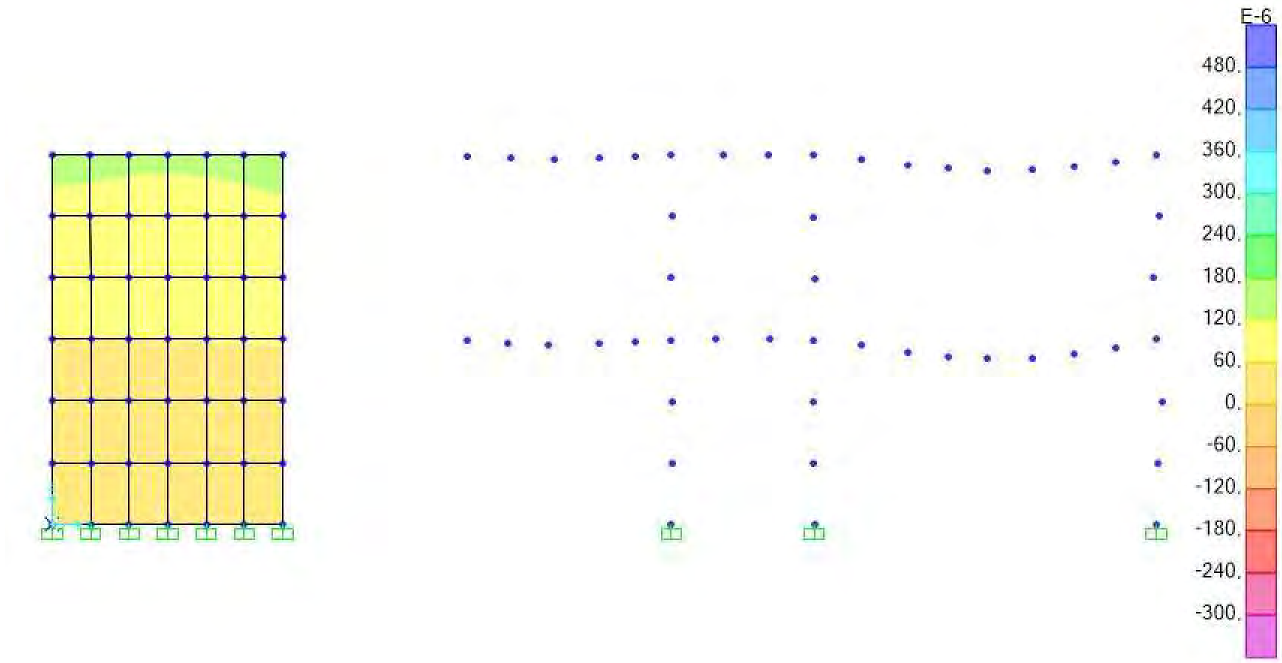


Fig. 4.15 – Shear stress of wall 2Y

Wall 3Y

Shear stress F_{3y}= 30,07 kN

Bending moment M_{3y}= 2,26 kNm

TABLE: Joint Reactions								
Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
2	Sisma en Y	LinStatic	5,833	-6,586	-33,067	0,3251	0,0831	0,000841
226	Sisma en Y	LinStatic	-0,00065	-9,87	25,916	0,4228	-0,00027	-6,72E-05
695	Sisma en Y	LinStatic	-7,41E-05	-7,48	-0,788	0,8772	-0,0001	-7,42E-06
699	Sisma en Y	LinStatic	-0,00032	-6,132	23,479	0,6398	-0,00033	4,94E-06

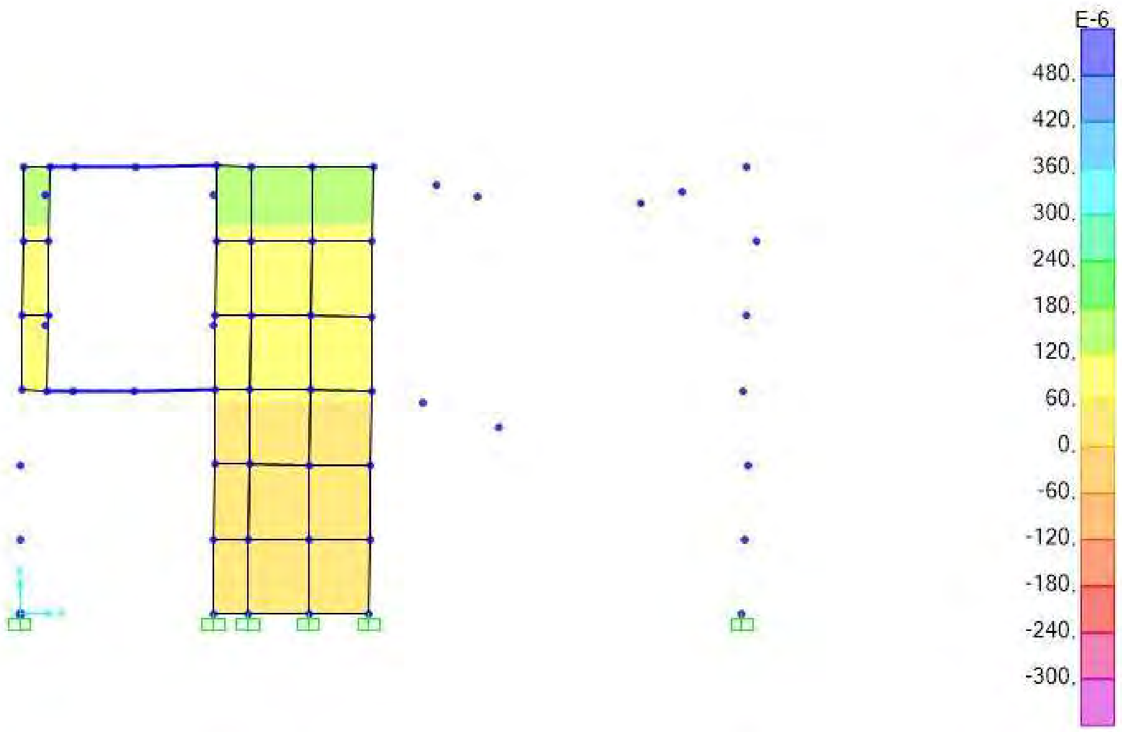


Fig. 4.16 – Shear stress of wall 3Y

Wall 4Y

Shear stress F_{4y}= 139,22 kN

Bending moment M_{4y}= 9,56 kNm

TABLE: Joint Reactions								
Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
20	Sisma en Y	LinStatic	-1,385	-16,573	-35,765	1,0239	0,1063	-0,003
31	Sisma en Y	LinStatic	-2,677	-16,547	49,21	1,4532	-0,035	0,0138
91	Sisma en Y	LinStatic	0,295	-22,32	29,061	1,5137	0,2624	-0,0296
186	Sisma en Y	LinStatic	-0,037	-20,583	48,46	0,5283	-0,0515	0,0307
730	Sisma en Y	LinStatic	0,159	-23,025	-10,811	1,6346	0,1104	0,0458
734	Sisma en Y	LinStatic	-0,069	-27,87	16,926	2,299	-0,138	-0,0306
774	Sisma en Y	LinStatic	0,188	-12,303	58	1,112	0,1646	-0,0234

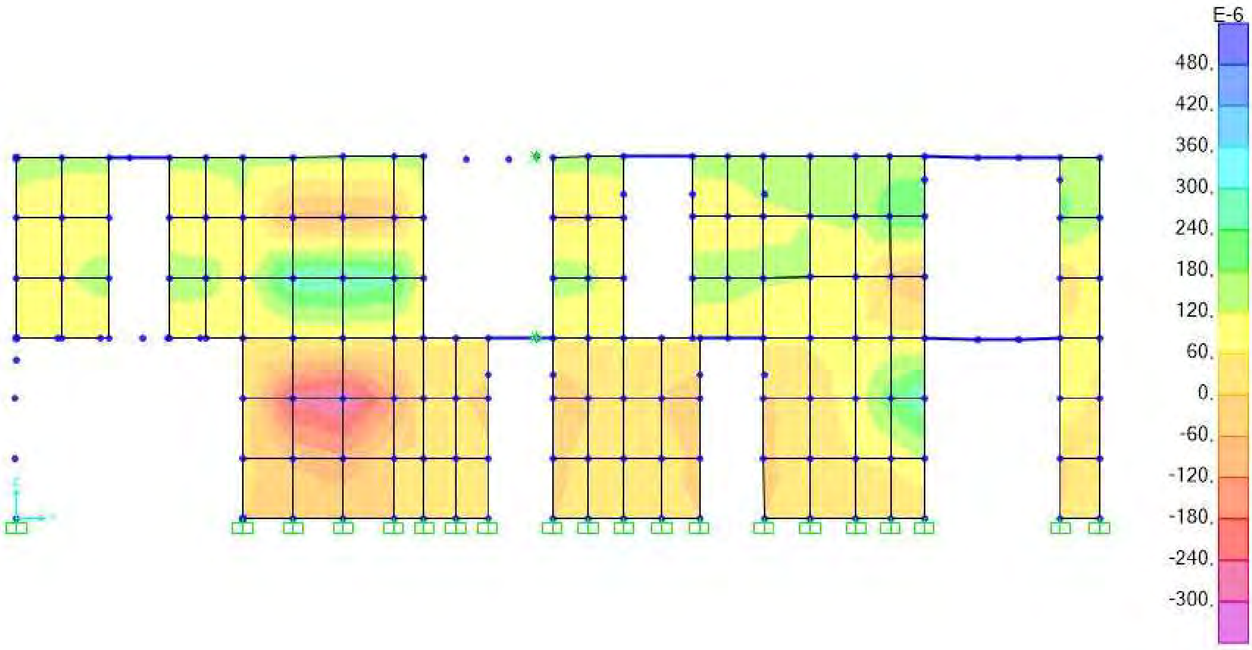


Fig. 4.17 – Shear stress of wall 4Y, 5Y and 6Y

Wall 5Y

Shear stress F_{5y}= 68,57 kN

Bending moment M_{5y}= 5,04 kNm

TABLE: Joint Reactions								
Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
43	Sisma en Y	LinStatic	3,937	-14,189	2,612	1,3154	0,666	0,000601
93	Sisma en Y	LinStatic	-0,00628	-14,054	-30,956	0,4538	-0,0114	-0,0091
191	Sisma en Y	LinStatic	-0,00664	-16,24	39,597	0,5157	-0,0102	0,0083
748	Sisma en Y	LinStatic	0,071	-12,221	-24,234	1,3853	0,0454	0,0023
780	Sisma en Y	LinStatic	0,069	-11,861	36,013	1,365	0,0423	-0,002

Wall 6Y

Shear stress F_{6y}= 57,36 kN

Bending moment M_{6y}= 4,40 kNm

TABLE: Joint Reactions								
Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
63	Sisma en Y	LinStatic	0,057	-11,662	40,251	1,1315	0,0448	-6,21E-05
95	Sisma en Y	LinStatic	0,01	-7,907	-17,88	0,4394	0,0108	0,0064
198	Sisma en Y	LinStatic	-0,0026	-20,238	51,654	0,3402	0,003	-0,0086
754	Sisma en Y	LinStatic	-0,014	-8,139	7,378	1,2181	-0,024	-0,0046
786	Sisma en Y	LinStatic	-0,076	-9,414	64,295	1,2661	-0,0576	0,0083

Wall 7Y

Shear stress F_{7y}= 662,09 kN

Bending moment M_{7y}= 43,48 kNm

TABLE: Joint Reactions								
Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
9	Sisma en Y	LinStatic	2,189	-21,676	-86,131	0,892	-0,3155	0,0055
60	Sisma en Y	LinStatic	0,008411	-29,811	78,492	1,0089	-0,0017	-0,0021
101	Sisma en Y	LinStatic	-0,105	-35,927	-4,018	2,3763	-0,0892	0,0048
103	Sisma en Y	LinStatic	-0,064	-34,885	5,999	2,2648	-0,0384	-0,00043
105	Sisma en Y	LinStatic	0,014	-31,561	6,473	2,1114	0,0101	0,0155
107	Sisma en Y	LinStatic	-0,127	-32,175	15,033	2,1309	-0,0938	-0,0072
109	Sisma en Y	LinStatic	-0,085	-31,339	24,04	2,1049	-0,0812	0,0096
111	Sisma en Y	LinStatic	-0,037	-30,419	62,103	2,2128	-0,023	9,35E-05
821	Sisma en Y	LinStatic	-0,024	-24,835	-42,717	1,8459	-0,0082	-0,0118
825	Sisma en Y	LinStatic	-0,032	-26,596	-27,786	1,9042	-0,0163	-0,0055
829	Sisma en Y	LinStatic	-0,025	-27,312	-18,954	1,8819	-0,01	0,009
833	Sisma en Y	LinStatic	-0,073	-28,494	-12,387	1,975	-0,0685	-0,0015
837	Sisma en Y	LinStatic	-0,00471	-29,541	-7,357	1,9906	0,0089	0,0045
868	Sisma en Y	LinStatic	-0,012	-28,437	16,838	1,9501	0,0169	-0,0013
876	Sisma en Y	LinStatic	-0,025	-25,378	94,178	2,1345	-0,0231	0,0028
896	Sisma en Y	LinStatic	-0,05	-33,849	35,153	2,306	-0,0344	-0,002
900	Sisma en Y	LinStatic	-0,064	-31,721	46,403	2,1667	-0,0529	0,0015
910	Sisma en Y	LinStatic	-0,024	-42,208	0,35	2,6786	-0,0214	-0,0044
914	Sisma en Y	LinStatic	-0,066	-43,31	4,111	2,7798	-0,0498	0,0049
918	Sisma en Y	LinStatic	-0,02	-36,65	10,15	2,3853	-0,06	-0,0031
922	Sisma en Y	LinStatic	0,021	-35,961	12,912	2,3784	0,0187	-0,0096

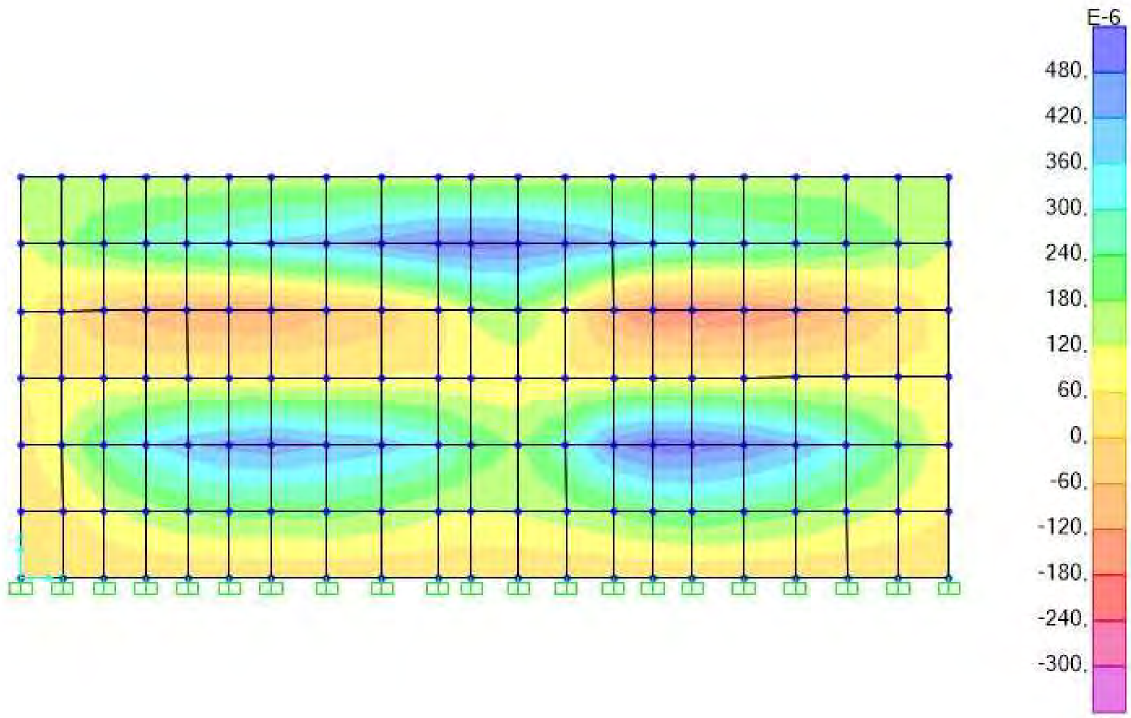


Fig. 4.18 – Shear stress of wall 7Y

It was made also an analysis based on the theory of the distribution of the forces in function to the stiffness of each wall. There were obtained values of the same order of magnitude as those cited above, but slightly different. As the distribution of forces does not take into account the interaction with other walls, it was decided to use for the strength verifications the values obtained from seismic analysis made with the software.

4.5.3 - Verification of shear strength

The design of a structural component necessarily require a verification phase during which we shall ensure that the external actions, which are supposedly subject during its operation, will not result in failure.

According to the Peruvian code E.070, for the design of shear walls is to considered that the section is rectangular, neglecting the contribution of the transverse walls.

For all masonry walls should be checked in each floor satisfies the following expression controlling the occurrence of cracks by shear forces:

$$V_e < 0.55V_m$$

where V_e is the shear force produced by the moderate earthquake on the wall in analysis and V_m is the diagonal shear cracking associated with masonry.

4.5.3.1 - Strength of unreinforced walls

As suggested by legislation, and as previously mentioned in chapter 2.3, the value of maximum resistance of unreinforced wall is obtained by the formula:

$$v_m = 0.5v_m\alpha tL + 0.23P_g$$

Where v_m is the characteristic shear strength of masonry, P_g service gravity load, with reduced overhead, t is the effective thickness of the wall, L the total length of the wall, α the reduction factor for shear strength slenderness effects calculated as:

$$\frac{1}{3} \leq \alpha = \frac{V_e L}{M_e} \leq 1$$

Where V_e and M_e are the shear force and the bending moment of the wall obtained by a elastic analysis.

In the following table is shown for each wall, the correspondent shear force, bending moment, the reduction factor for shear strength slenderness effects, the service gravity load and the result of shear strength, when subjected to a seismic force in X direction.

Wall	*V* (kN)	*M (kNm)*	*α=VL/M*	*α(/)*	P_g (kN)	V_m(kN)	*0,55*V_m(kN)
1X	-91,77	-7,27	18	1	85,38	159,64	87,80
2X	-118,09	-13,06	37	1	20,59	157,74	86,75
3X	-83,67	-6,47	22	1	83,53	119,71	65,84
4X	-106,52	-27,63	58	1	150,78	259,68	142,82
5X	-110,05	-11,89	34	1	215,53	196,57	108,11
6X	-290,08	-39,31	55	1	303,32	444,76	244,62

Tab. 3.8 – Verification of shear strength of X direction walls

As you can see all the values of reduction α are higher than 1 because the moments resulting by solving the model were really low. Therefore, as suggested by the standard, we use a reduction factor of slenderness α=1.

Than you can see that for this specific value of seismic force, which was determined by using no reduction factor R, shear strength of almost all the walls in X direction, highlighted in red colour, do not resist the supposed earthquake. This is due to the fact that in X direction the most of walls have a thickness not enough to be consider as a bearing wall, even if respect standard.

In the following table is shown for each wall, the correspondent shear force, bending moment, the reduction factor for shear strength slenderness effects, the service gravity load and the result of shear strength, when subjected to a seismic force in Y direction.

Wall	*V* (kN)	*M (kNm)*	*α=VL/M*	*α*(/)	P_g (kN)	V_m(kN)	*0,55*V_m(kN)
1Y	-512,16	33,67	-137	1	30,69	909,56	500,26
2Y	-89,95	6,11	-44	1	21,00	184,83	101,66
3Y	-30,07	2,26	-23	1	15,54	107,07	58,89
4Y	-139,22	9,56	-48	1	155,08	363,17	199,74
5Y	-68,57	5,04	-31	1	23,03	230,30	126,66
6Y	-57,36	4,40	-21	1	145,70	193,51	106,43
7Y	-662,09	43,48	-169	1	212,89	1161,46	638,80

Tab. 3.9 – Verification of shear strength of Y direction walls

Also in this case all the values of reduction α are higher than 1 because the moments resulting by solving the model were really low. Therefore, as suggested by the standard, we use a reduction factor of slenderness α=1.

Instead in this direction we notice that for this specific value of seismic force, which was determined by using no reduction factor R, only shear strength of outer walls (1Y and 7Y), highlighted in red colour, do not resist the supposed earthquake. This is probably due to the fact that they are the two walls that carry most of part of the structure.

4.5.3.2 - Strength of polymer reinforced walls

Because of the non-resistance of certain walls saw in the previous point, it was decided to reinforce the walls that do not fulfil the values strengthening required by standard, through the aid of polypropylene grid proposed in previous chapters.

As suggested by *ACI*, and as previously mentioned in chapter 2.3, the value of maximum resistance of a polymer reinforced wall is obtained by the formula:

$$Vmax = Vma + Vmo + Vpg$$

Where V_{ma} is the contribution of shear strength of unreinforced masonry to the nominal shear strength as determined before , V_{mo} the contribution of plaster calculated as:

$$V_{mo} = 0.53\sqrt{f'c}\,bd$$

and V_{pg} the contribution of polymer grid composite material given by the formula:

$$V_f = 2nA_f L f_f$$

In the following table is shown for each wall, the correspondent contribution of shear force given by unreinforced masonry, contribution given by mortar, the contribution of the polymer grid used as reinforcement, and the total shear strength of the reinforced wall when subjected to a seismic force in X direction.

Wall	*L* (m)	*V*(kN)	V_{ma}(kN)	V_{mo}(kN)	A_g (m^2/m)	V_{pg}(kN)	V_{rw}(kN)	*0,55*V_{rw}(kN)
1X	1,40	-91,77	159,64	5,01	0,000014	32,33	196,98	108,34
2X	2,55	-118,09	157,74	9,12	0,000014	58,89	225,74	124,16
3X	1,68	-83,67	119,71	5,99	0,000014	38,68	164,38	90,41
4X	3,75	-106,52	259,68	13,42	0,000014	86,60	359,69	197,83
5X	2,45	-110,05	196,57	8,77	0,000014	56,58	261,91	144,05
6X	3,75	-290,08	444,76	13,42	0,000014	86,60	544,78	299,63

Tab. 3.10 – Verification of shear strength of X direction reinforced walls

Than you can see that for this specific value of seismic force, which was determined by using no reduction factor R, all the walls in X direction resist the supposed earthquake. It has to be pointed out that wall 6X is not far from the limit imposed by standard, so it suggested to reinforce the wall with steel bars or change the thickness of the wall, for higher safety standard.

In the following table is shown for the two walls that didn’t resist the shear force, the correspondent contribution of shear force given by unreinforced masonry, contribution given by mortar, the contribution of the polymer grid used as reinforcement, and the total shear strength of the reinforced wall when subjected to a seismic force in Y direction.

Wall	*L* (m)	*V*(kN)	V_{ma}(kN)	V_{mo}(kN)	A_g (m^2/m)	V_{pg}(kN)	V_{rw}(kN)	*0,55*V_{rw}(kN)
1Y	9,03	-512,16	909,56	16,14	0,000014	208,41	1134,11	623,76
7Y	11,13	-662,09	1161,46	19,90	0,000014	256,91	1438,27	791,05

Tab. 3.11 – Verification of shear strength of Y direction reinforced walls

Even in this case it can be seen that for this specific value of seismic force, which was determined by using no reduction factor R, also the two walls in Y direction (1Y and 7Y) resist the supposed earthquake.

CONCLUSIONS

In order to investigate the performance of brick masonry walls reinforced with polymer grid in the post elastic range, twelve panels of brick masonry were tested to in plane cyclic shear force at the Seismic Laboratory of *Catholic University of Perù*,

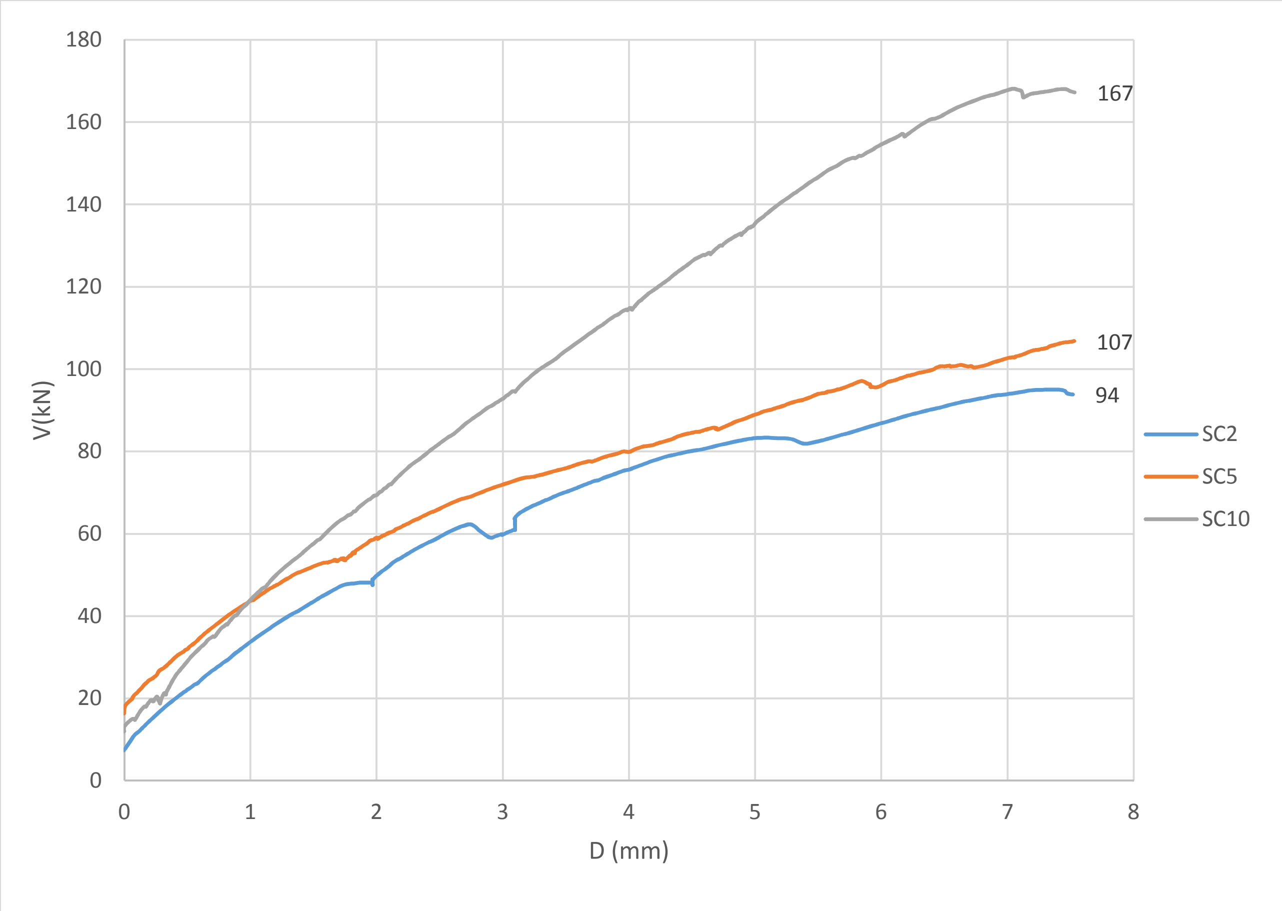
The panels had dimensions 1.2 x 1.2 m and 0.22 m thickness. Four walls were tested without plaster, four with sand cement plaster and the last four with the grid reinforcement embedded in the plaster on both sides.

In the unreinforced panel, reaching 4mm of horizontal displacement, with a horizontal force of the order of 90 kN, vertical small cracks appeared at both ends of the panels. The horizontal small cracks extended and enlarged, leading to the appearance of the first diagonal cracks at 7 mm of horizontal displacement, corresponding to a maximum horizontal load of approximately 94 kN, and the panels tried to rotate on their bases.

In plastered walls, vertical small cracks by compression appeared at both ends of the panels at 10 mm of horizontal cyclic displacement, being the maximum horizontal load approximately 107 kN. Diagonal small cracks appeared at 15 mm of horizontal displacement and grew till reaching the failure caused by the opening of a wide diagonal crack.

In polymer grid reinforced panels, reaching 8 mm of horizontal displacement, the maximum load reached was slighter bigger than the one of the previous tests. The small cracks pattern was scattered, diagonal small cracks appeared in a sequential way and followed the configuration of the reinforcement grid. For the maximum horizontal displacement, that was 20 mm, the plaster of both bottom corners detached from the walls due to the high concentrated compression forces.

The observation of the shear-deformation curves show an extremely good ductile behaviour of the panels subjected to cycling loadings. The increase in ductility is even more evident by comparing the curves of the plastered reinforced panels with those of the plastered-unreinforced ones or with the non-plastered ones.



Force – Displacement curve of the three types of tested walls

The graphs of the reinforced panels show a more pronounced hysteretic behaviour, with wider cycle areas corresponding to higher energy dissipation. Observation of crack pattern after shear-compression test shows that the grid reinforcement distributes the damage in several fine cracks in both diagonal directions, compared to the plastered unreinforced panels where one wide crack appears. In the non plastered and unreinforced panels also several fine cracks appear at lower levels of horizontal force, since the plaster itself increases the horizontal resistance.

From the theoretical point of view, the minimum tensile stress of the grid would have to be the same that the masonry wall in shear, since at the time of cracking, all the shear stress has to be transferred to the grid and at the same time to the wall with or without reinforcement. This means that the contribution of shear strength given by polymer grid must be added to the contribution of shear strength given by masonry and mortar. This is the same solution proposed by the *A*merican *C*oncrete *I*nstitute.

It was also made a comparison between the shear strength obtained by the three specimens of walls (unreinforced, plastered and FRCM) and the shear strengths that are supposed to be obtained by the theoretical formulas, in order to verify the increase of shear strength and understand how much the theoretical solution differs from the reality. From this comparison it is noted that the two formulas that express the value of shear strength of unreinforced wall and plastered wall describe almost the reality. In fact the difference is only about 2-3%. For the case of polymer grid reinforced wall the difference is about 25% which is a relevant amount, but the theoretical formula underestimate the real increase of shear, which means that the rest of strength not defined can be used for higher safety standards.

Then an example of informal house subjected to a seismic event was made. It was choose a prototype able to represent as much as possible the reality of Peruvian informal houses characteristic, and made a seismic analysis. In case of unreinforced house, it is shown that the house does not resist to a moderate earthquake. This because almost all the unreinforced walls do not have enough shear strength. For this reason a comparison was made, supposing to give a superficial reinforcement to those walls which didn’t resist. In effect after reinforcement all wall have enough shear strength to withstand to the supposed seismic event. We want to point out that for this seismic analysis no seismic force reduction was used. This because, according to standard, there is no specification about simple masonry building.

It is clear, after this comparison, that FRCM is a good alternative of reinforcement method to carry out the objective of reducing the seismic vulnerability.

Came at the end of this path , I actually say that this is not only an arrival point but the beginning, I hope, of a project that will really, in a developing country such as Peru, change the perspective and method of building, protecting what is for everyone the heart of life: our own home.

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Per ora torniamo a casa..

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Alma