Seismic in-plane behavior of post-tensioned existing clay brick masonry walls

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Seismic in-plane behavior of post-tensioned existing clay brick masonry walls

A thesis submitted to attain the degree of

DOCTOR OF SCIENCES of ETH ZURICH

(Dr. sc. ETH Zurich)

presented by

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July 1, 2015

Abdollah Sadeghi
Abstract

Existing masonry buildings are seismically vulnerable. Many residential buildings as well as historical structures made of unreinforced masonry walls need to be retrofitted against seismic actions. The post-tensioning technique has been used in various structural applications to provide compression and to postpone cracking in the structural elements. This research aims at studying the post-tensioning technique as a seismic retrofitting solution for existing masonry buildings. Solid clay bricks and weak cement-lime-sand mortar were used in the construction of the masonry buildings at the beginning of the 20th century. In this thesis, the aforementioned masonry type and its constituents are characterized using small size tests. Using a modern 3D image correlation system allows for a better understanding of the failure mechanism in the masonry. The in-plane behavior of unreinforced and post-tensioned masonry walls is investigated by a series of large-scale quasi-static cyclic tests. The effects of aspect ratio of the walls, existing normal stress on them, and applied post-tensioning stress are taken into account in the experimental program. Post-tensioning was found to result in a significant increase in the shear resistance of masonry walls and a moderate decrease in their displacement capacity. An extra increase in the shear resistance was observed in the post-tensioned masonry walls with a rocking failure mode due to the elongation of the tendons. The requirements of the Eurocode regarding the displacement capacity of the masonry walls at the significant damage limit state were mostly met. In slender masonry walls with high amount of existing axial stress, e.g. the walls located at lower stories of a multi-story building, a high post-tensioning stress results in changing the failure mode from a ductile rocking mode to a brittle diagonal cracking. The level of post-tensioning must be selected carefully to avoid brittle failure modes. Various numerical models with different complexities for predicting the in-plane behavior of post-tensioned masonry walls are investigated. The shortcomings of the methods in providing a proper estimation of the in-plane behavior of the post-tensioned masonry walls are noted. A 2D finite element model is developed in this thesis for a better prediction of the in-plane response of the tested walls. The model uses the experimental results of the small size tests carried out in this study. It provides a reasonable prediction of the in-plane behavior for slender walls. The developed FE model is also able to capture the change in the failure mode due to high amount of post-tensioning applied to the post-tensioned walls.
Zusammenfassung

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1 Introduction

1.1 General

Masonry buildings are one of the common types of residential building all over the world. In addition to buildings, there are many valuable historical structures made of this ancient construction material. In these structures, unreinforced masonry (URM) walls are the main load carrying elements. The vertical forces applied to masonry structures cause compressive stresses in the URM walls. In terms of compressive stress, masonry is a reliable material. Nevertheless, tensile stresses are generated in the URM walls as a result of the action of lateral loads such as wind and earthquake on masonry buildings. Due to the very small and unreliable tensile strength of masonry, tensile stresses can easily cause cracking, failure and collapse in the URM walls. Being the most critical lateral load in the areas prone to earthquake, it seems inevitable to upgrade the masonry buildings against seismic actions.

For overcoming the intrinsic tensile weakness of masonry, post-tensioning can be applied to URM walls to develop compressive stresses in them. This can be done by prestressing vertically oriented tendons that are anchored at both ends of a URM wall. The prestressing tendons can be placed either inside the existing walls in newly drilled holes or adjacent to them. In either case, the tendons remain unbonded to the wall over their entire length. This allows a uniform distribution of tensile strain over the entire length of the tendons. By this method, the concentration of strain that may occur in bonded post-tensioning systems is prevented. Additionally, a large elongation capacity before yielding is expected for the tendons that results in a large deformation capacity of the wall. The reaction of the generated tension in the tendons is introduced to the wall by the anchors in form of compression. By increasing the normal force on the URM walls in a masonry building using the post-tensioning technique, the in-plane shear resistance of the walls is expected to increase and therefore a higher lateral load resistance is provided to the masonry building.

In comparison with other retrofitting techniques such as the application of fiber reinforced polymer (FRP) sheets, reinforced shotcrete overlay or the introduction of new walls, post-tensioning is a rather minor intervention. It is therefore suitable for cases where the existing structure shall be left as close as possible to its original state as possible such as for historical constructions. Also, in many cases, the tendons are fairly easy to install minimizing the disruption of the function of the building. Another advantage is that the applied prestressing force acts as a restoring force that reduces residual deformations after excessive loading (Ganz 2003).

It is well known and experimental studies (Abrams et al. 2007; Magenes et al. 2008) have repeatedly shown that high normal forces reduce the deformation capacity of the walls which is detrimental for seismic loading. Cyclic-static tests on post-tensioned concrete-block masonry (Laursen 2002) and modern hollow clay brick masonry (Rosenboom and Kowalsky 2004) have shown, however, that the deformation capacity of the masonry walls is still satisfactory as long as a “rocking” behavior is ensured.
So far, research projects on post-tensioned masonry have focused mainly on the out-of-plane and in-plane behavior of new structures (masonry with hollow concrete blocks or hollow clay bricks). Although often applied in practice, very little is in fact available on the post-tensioning of existing masonry (masonry with solid clay bricks and poor quality mortar) for an improved seismic in-plane behavior; the inelastic deformation capacity being more or less unknown.

### 1.2 Motivation

In order to assess the seismic vulnerability of existing masonry buildings and to do a feasibility study on using post-tensioning as a seismic rehabilitation method the first step was to evaluate a common type of residential masonry building against seismic actions. Critical URM walls in terms of shear resistance were to be recognized and the implementation of post-tensioning on them to overcome the lack of shear resistance was to be studied. For this purpose Merkblatt SIA 2018 (2004) “Assessment of existing structures with regard to earthquake” was followed. The force-controlled assessment procedure is based on comparing the resistance of various components of a masonry building at design level, with the seismic action applied to them. The resistance over action ratio is called compliance factor. The compliance factor determines to what extent an existing building fulfills the design requirements of SIA standard. When the building partially fulfills the design requirements, the decision of retrofitting is made based on risk-based criteria.

A typical multi-story residential masonry building in Zurich, Switzerland, is shown in Fig. 1.1. The building is 50 m long and 13 m wide. Along its length, the building is separated by expansion joints into six separate parts with a plan shown in the same figure. Such buildings have been constructed by unreinforced masonry walls as the vertical and horizontal load carrying elements. The floors are made of reinforced concrete. They transfer the gravitational forces to the URM walls which are located both externally at the perimeter of the building and internally in the living area as seen in the plan of the building (Fig. 1.1). Both types of walls are considered as structural walls and contribute in transferring vertical forces to the ground. The thickness of the structural walls varies between 10 cm to 15 cm. The openings in the external walls, however, decrease the length of these walls and therefore their in-plane stiffness. As a result, their contribution in carrying the horizontal forces decreases significantly. In Fig. 1.1, the walls that contribute in resisting the horizontal forces are illustrated in blue.

![Fig. 1.1](image)

Fig. 1.1 (a) A typical masonry building in Switzerland and (b) a typical plan (plan area = 101.2 m²).
In order to find the existing normal stress in the URM walls, details of the building are considered as follows. The thickness of the concrete floors is 160 mm. Specific weights of 25 kN/m$^3$ and 13 kN/m$^3$ are considered for the concrete floors and the masonry walls, respectively. In addition, both the surcharge and live load are assumed to be equal to 2 kN/m$^2$. Based on (SIA 261 2003), Switzerland’s standard of “Actions on structures”, the permanent vertical load is calculated by taking into account the dead load of floors and walls and 30% of the live load. This results in a 670 kN vertical load per story level. By lumping the dead load of the walls at the story levels, the total vertical load at each story level becomes about 920 kN. The normal stress of each wall at different story levels can be calculated by taking into account the contributing load area. The normal stress applied to the walls at different story levels is shown in Fig. 1.2. The mean value of the normal stress at different story levels, shown in the figure, can be used as a basis for designing experiments.

According to Switzerland’s standard for designing masonry structures (SIA 266 2003), the compressive strength of masonry in two directions is required for estimating the shear strength of a masonry wall, perpendicular to the bed joints and parallel to them. For this example, the design value of the compressive strength of masonry is taken as 3.5 MPa in the direction perpendicular to the bed joints. In the direction parallel to the bed joints, the compressive strength is considered to be 30% of the other direction as suggested by the standard for clay brick masonry walls. The modulus of elasticity is also suggested by the standard to be equal to 1000 times compressive strength for standard masonry.

The equivalent force method of SIA 261 is followed for finding the shear force and bending moment applied to every URM wall. An importance factor of $\gamma_I=1$, a response factor of $q=1.5$, an earthquake zone of Z1 with a horizontal ground acceleration of $a_{gd}=0.6$ m/s$^2$ and a soil type C are assumed. This results in the design spectrum shown in Fig. 1.3. Based on an elastic finite element model of the building, the stiffness of the walls, the natural period of the structure is equal to $T_{1X}=1.57$ and $T_{1Y}=0.48$ seconds in the X and Y directions, respectively. Therefore, the ratio between the base shear force and the building weight equals 0.045 in X direction and 0.117 in Y direction. The base shear is distributed over the height of the building. Subsequently, the shear force and bending moment applied to each wall is calculated. The distributions of shear force and bending moment as well as normal force over the height of the building are shown in Fig. 1.4 for W5 and W11.
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Fig. 1.3 Design spectrum of the building and its natural periods in X and Y directions.

(a)

(b)

Fig. 1.4 Distribution of normal force, shear force, and bending moment along the height of the building for (a) W5 and (b) W11.

The stress field method of SIA 266 is followed for calculating the shear resistance of the walls. This method is based on the lower bound theorem of the theory of plasticity. A vertical and an inclined stress fields are assumed in a masonry wall as struts to transfer the applied axial and shear forces as well as the bending moments through the wall. A uniaxial stress field is considered for each strut. The uniaxial stresses in the struts as well as their combination shall remain within the boundaries of the failure criterion. A detailed description of the method is provided in Chapter 4. Fig. 1.5 shows the nondimensional shear resistance of W5 and W11 as a function of the normal force in different stories of the building. Each curve represents a story by taking into account the applied bending moment and shear force at the corresponding level. The points stand for the state of critical walls under certain normal force. As seen in the figure, application of higher normal force up to a certain level results in an increase in its shear resistance. The additional normal force to a story can be provided by post-tensioning.
Motivation

Fig. 1.5 Nondimensional shear resistance versus normal force for (a) W5 and (b) W11 in the stories.

Having the shear demand and capacity of each wall, the compliance factor is calculated for each masonry wall as the ratio between the shear resistance (capacity) and the design shear force (demand). Fig. 1.6 illustrates the compliance factor of the critical walls in both directions. The most critical walls in X and Y direction are W5 and W11 in the ground floor with compliance factors of 0.39 and 0.61, respectively. The maximum reachable compliance factor, shown in the figure, is calculated solely by increasing the axial force, by assuming post-tensioning, to reach the maximum shear capacity of the walls. The lower values of compliance factors in the lower stories are due to higher shear forces in the lower stories on one hand, and, on the other hand, higher bending moments in the lower stories that result in a decrease in the shear resistance.

Fig. 1.6 Existing and maximum reachable compliance factor of (a) W5 and (b) W11 in different stories.

The reasonable\(^1\) and commensurable\(^2\) thresholds of the compliance factor, seen in Fig. 1.6, are defined according to Merkblatt SIA 2018 (2004) to decide about the seismic retrofitting of the building. The thresholds are a function of the importance of the building as well as its expected remaining useful life,

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\(^{1}\) Zumutbar in German  
\(^{2}\) Verhältnismässig in German
see Fig. 1.7. Assuming a normal residential building with 30 years remaining useful life time, the reasonable and commensurable thresholds are equal to 0.25 and 0.66 respectively. If the compliance factor is lower than the reasonable threshold, reasonable measures are required for seismic retrofitting of the building. Based on the standard, the lifesaving cost, to be at most CHF 100 million per human life saved is viewed as reasonable. If the compliance factor is between the two thresholds, the risk to people through seismic retrofitting measures is reduced so that the costs remain commensurable. Lifesaving costs to a maximum of CHF 10 million per human life saved is viewed as commensurable (Wenk 2008). Higher compliance factors express that the current state of the building is acceptable.

As seen in Fig. 1.6 increasing the normal force on both critical walls by post-tensioning brings the compliance factor to the region where no measures are recommended. Therefore, post-tensioning of the critical walls in the first story could be considered as a solution for retrofitting the building.

As seen in the force-based seismic assessment of a masonry building, post-tensioning was considered only as a constant increase in the normal stress acting on the masonry walls. However, depending on the failure mode of a post-tensioned masonry wall, the amount of stress in post-tensioning elements may increase as a function of the horizontal displacement. Therefore, a variable normal force is expected in post-tensioned masonry walls, which is not considered in the seismic evaluation of masonry buildings.

If the displacement-based method of Eurocode 8-3 (2004) is considered for the seismic evaluation of an existing masonry building, the displacement capacity of URM walls shall satisfy the specified limits depending on the considered performance level. In the significant damage (SD) performance level, the displacement capacity of a URM wall should satisfy the limit of 0.004 and 0.008-height/length for the shear and bending failure modes, respectively. For the near collapse (NC) performance level, the limits are multiplied by 4/3. In the literature, there is a lack of a robust formula for displacement capacity of masonry walls. In practice, some empirical equations are used that inversely relate the displacement capacity of masonry walls to the normal stress to strength ratio.

Motivation of the research derives from the fact that (1) there is limited research on post-tensioning as a seismic retrofitting method of existing masonry structures all over the world and particularly in Switzerland, (2) in a simplified way, post-tensioning on URM walls is considered only as an increase in their normal stress without taking into account the effect of post-tensioning tendons during horizontal...
loading, (3) there is little knowledge about the in-plane displacement capacity of post-tensioned masonry walls made of clay bricks and weak mortar, and (4) the seismic structural behavior is likely to be improved by applying the post-tensioning technique.

1.3 Thesis organization

This thesis is organized in six chapters. The structure of the remainder is as follows:

Chapter 2 presents a literature review of post-tensioned masonry applications around the world and the current state of research on the in-plane and out-of-plane behavior of post-tensioned masonry walls. The review of the numerical models for predicting the in-plane behavior of post-tensioned masonry walls is described separately in Chapter 5.

In Chapter 3 the results from a comprehensive experimental program are presented. The aim for conducting the experimental program is to characterize the masonry used for construction of masonry buildings at the beginning of the 20th century worldwide particularly in Switzerland. Weak lime-cement-sand mortar and solid clay bricks constitute the components of the masonry. Axial compression tests on the mortar with various geometries are described. The uniaxial behavior of mortar and its confined behavior, which is experienced by the mortar in a masonry assemblage, can be observed by this set of experiments. Uniaxial compression and lateral tensile compression tests performed on the solid clay brick are explained. In addition to the tests on masonry constituents, uniaxial centric and eccentric compression tests as well as triplet shear tests were performed on masonry assemblages. A modern 3D image correlation system was used in all tests for a better understanding of the behavior of masonry. The results are explained in detail.

Chapter 4 presents a series of static cyclic in-plane shear tests carried out on large-scale cantilever masonry walls without and with post-tensioning. The walls were constructed in two different geometries with a height to length ratio of more than unity (squat) and less than unity (slender). Various normal forces were applied on the slender wall in order to take into account the effect of normal force on the in-plane behavior of masonry walls with a rocking failure type. The in-plane behavior and crack pattern of the walls during different steps of loading are described and classified. The most relevant results concerning the structural behavior of unreinforced and post-tensioned masonry walls are discussed, including the shear resistance and displacement capacity. The seismic behavior of the walls is investigated.

In Chapter 5, common numerical models for simulating post-tensioned masonry walls are described. The tested post-tensioned masonry walls, described in Chapter 4, have been simulated using different methods with various complexities. The mechanical properties of masonry and mortar obtained in Chapter 3 have been used in the simulation of the walls. The ability of existing semi-analytical and fiber element methods in predicting the seismic in-plane response of post-tensioned masonry walls is investigated. A new 2D finite element (FE) model is developed to explain the seismic in-plane response of post-tensioned masonry walls.

Chapter 6 summarizes the main conclusions of the thesis and presents recommendations for future research.
1.4 References


2 Literature review

2.1 Post-tensioned masonry and its applications

Masonry has been used for construction of a variety of structures such as residential, commercial, office, and industrial buildings, towers, arches, viaduct bridges, etc. Major advantages in using masonry in ancient and modern times have been in its overall availability of raw materials, its easy and economical construction, and its natural beauty and durability. Masonry has a relatively large compressive strength but a low tensile strength. Post-tensioning offers the possibility to actively introduce a desired level of axial load in a wall to enhance strength, performance, and durability of masonry (Ganz 1990).

Post-tensioning masonry combines an advanced construction technique with an old building material to overcome its weaknesses. This idea is not new. Early research and applications of post-tensioned masonry have been reported primarily in England by innovative engineers in the early 19th century. Ganz (2003) reported that in 1825 a post-tensioning method was used in a tunneling project under the River Thames in England. In this project vertical tube caissons of 15 m diameter and 21 m height were constructed with masonry. The 0.75 m thick brick walls were post-tensioned with 25 mm diameter wrought iron rods.

Since the 1960's more applications of prestressed masonry appeared in practice. Schultz and Scolforo (1991) gave a comprehensive review on various applications especially in the U.S.A. Some other applications have also been described in the literature (Ganz 2003; Lissel et al. 1999; Foti and Monaco 2000; Wight 2006). For the sake of brevity, only a few applications of post-tensioned masonry are mentioned here. The applications of post-tensioned masonry in practice can be categorized into two groups, design and retrofitting.

2.1.1 Applications for design of new structures

Salvation Army Citadel, Warrington, UK

The Salvation Army asked W. G. Curtin and Partners of Liverpool to design a new citadel in Warrington, Cheshire, England in the early 1980's (Curtin et al. 1982). This is the first known application of highly stressed post-tensioned diaphragm walls (Wight 2006).

Only the main hall was designed with post-tensioned masonry. It was 25 m long by 15 m wide and had a height of 8.5 m. The boundary condition of the wall was considered as a free standing cantilever due to the inclusion of a clerestory window running around the top of the wall, shown in Fig. 2.1(a). The post-tensioning resulted in an absence of tensile stresses in the bottom of the walls under the full design wind loading.

A brick finish was used for the exterior of the hall, whereas white concrete blocks were implemented for the internal structure to reflect light. This resulted in a cavity diaphragm wall section as illustrated in
Fig. 2.1(b). Consequently, the blocks were jointed off from the external clay masonry and cavity ties were installed to ensure adequate transfer of shear stresses.

The post-tensioned wall resisted the in-plane and out-of-plane loads mostly due to the lateral actions of wind. The creep characteristics of the blockwork governed the estimation of the prestress losses. Two 32 mm diameter bars located in each cavity were prestressed using a conventional torque wrench to reach a force of 100 kN. The bars were protected from corrosion by wrapping some anti-corrosion tapes. Threaded couplers were used to extend the bars to assist in construction. The work was found to be within the capabilities of a small local builder, and the project was completed on time and within budget (Curtin et al. 1982).

(a) Salvation Army Citadel (Curtin et al. 1982) (a) Citadel and (b) prestressed cavity diaphragm wall.

Tring Bridges, Hertfordshire, UK

In 1994, two 7 m long pedestrian bridge decks were constructed of post-tensioned masonry in Tring, UK (Caine 1998). Theses bridges are believed to be the first post-tensioned brick box girder decks ever built.

Each deck was 1.5 m wide and had a 440 mm deep box section formed by top and bottom flanges connected with five webs in the section (Fig. 2.2). Four voids were provided to accommodate the longitudinal post-tensioning tendons. Negative deflection was provided to the deck in transverse and longitudinal directions to drain water and to improve the durability of the masonry deck.

The tendons inside the box were straight. The negative deflection of the deck provided the required eccentricity at mid-span to resist the out-of-plane loading. Concrete capping beams were used for anchoring the tendons at either end of the deck. One of the two bridge decks used stress bars for the post-tensioning tendons; the other used aramid fiber-reinforced polymer (AFRP) tendons.

The masonry decks were built and prestressed standing vertically and then placed in their final horizontal position as seen in Fig. 2.2(b). Stressing the tendons was done by a hydraulic jack to 200 kN per tendon.
Post-tensioned masonry and its applications

Kindergarten, Zurich, Switzerland

In 1990, VSL International reported two applications of post-tensioned masonry in Switzerland (Ganz 1990). As one of the first applications, the VSL post-tensioning system was used in the masonry walls of a kindergarten in Zurich made of cavity bricks. The exterior walls were post-tensioned aiming at providing the required out-of-plane resistance to lateral wind loads. The system utilizes self-activated, dead-end anchorages at one end, and stressing anchorages at the other end, that are cast in the foundation and capping beam, respectively, (Fig. 2.3a). During the construction of the wall, ducts were placed in the cavity bricks at the location of the post-tensioning tendons. After the wall construction was finished, the tendons were placed in the ducts through the capping beam. Post-tensioning was achieved using prestressing jacks.

In the kindergarten, the interior clay brick leaves of walls were 140 mm thick and up to 4 m high, with large window openings. The walls were laterally supported on top by a steel frame in the roof. Five monostrand tendons were used for each wall. The dead-end anchorages were placed in a 250 mm thick floor slab. The stressing anchorages were placed in prefabricated concrete elements with a 130 mm height. Truss-type bed joint reinforcement was placed below the capping to take the horizontal bursting forces. Each tendon was stressed to 180 kN, i.e. 70% of ultimate strength.

Fig. 2.2 Post-tensioned masonry pedestrian bridge “Tring Bridge” (a) designed section and (b) post-tensioning and installation (Caine 1998).
2.1.2 Applications for retrofitting of existing structures

General Post Office, Sydney, Australia

As reported by Ganz (1990) towards the end of the 1980s, the General Post Office (GPO) in Sydney, Australia, needed strengthening for seismic loading. The tower was strengthened using four vertical post-tensioning tendons, nineteen 13-mm strands each, and a number of horizontal stress bars, 35 mm in diameter, at floor level. The vertical post-tensioning tendons were placed in 100-mm-diameter holes core drilled from the top of the tower through the sandstone columns at the tower corners (Fig. 2.4). Special steel chairs were used to anchor the tendons and distribute the high anchorage force of 1771 kN. The entire restoration took 5 years to complete and installation and stressing of the tendons was done in 1990. The force in unbonded tendons was monitored and could be compensated if there was any loss due to volume changes of the sandstone.
Administration building at Empa, Duebendorf, Switzerland

The administration building of Empa in Duebendorf was constructed of reinforced concrete frames, reinforced concrete walls, and masonry in 1960 (Fig. 2.5a). The floors were made of reinforced concrete. The building is approximately 50 m long and 18 m wide. In a seismic retrofitting program in 2007, the seismic behavior of the buildings was assessed. It was found that the weakness of the structure is in the transverse direction, where the horizontal load resisting system contains end masonry walls together with a concrete core. Only one end was critical because the concrete core was constructed close to the other end of the building.

For retrofitting, the 220-mm-thick masonry wall located at the northern end of the building was strengthened by five 13 m long tension rods placed only on one side of the wall. The vertical post-tensioning force from the tendons was transferred to the center of walls by means of a steel construction on the roof and a concrete slab as counter weight (Fig. 2.5b). The rods were made of Carbon Fiber Reinforced Polymer (CFRP). This was the first known application of CFRP rods for seismic retrofitting of masonry walls (Bachmann 2007).

The Arts Center, Christchurch, New Zealand

In 1984, two stone masonry buildings within The Arts Center of Christchurch, i.e. the Chemistry building and the College hall, received post-tensioning seismic retrofits, which were subsequently subjected to design level seismic loads during the 2010/2011 Canterbury earthquake sequence. Bailey et al. (2014) observed and reported the performance of such retrofits in the actual design-level earthquake.

The retrofit of the Chemistry building (as shown in Fig. 2.6) was part of a strengthening scheme that also involved diaphragm strengthening and steel frames being applied to the top story level. Vertical tendons installed on the ground level were located in pairs on either side of the buttresses and were anchored into the concrete slab above the basement, stretching up to the bottom of the top level of the building. To enhance a frame-type action of building response, horizontal tendons were applied to the top and bottom of the walls in the interior and exterior of the building. In the analysis of the Chemistry building, the premier walls were considered as frames, with the required prestress level of 0.27 MPa, 40% of the total allowable stress on the section.
Fig. 2.5 (a) Administration building at Empa showing the northern masonry wall retrofitted with post-tensioning, (b) longitudinal section through the building showing the retrofit, and (c) top and bottom anchorage of post-tensioning CFRP rods (Wenk 2008).
Comparing the performance of the post-tensioned seismically retrofitted building with other unretrofitted buildings in the Arts Center highlighted the favorable performance of post-tensioned stone masonry and its lateral displacement restoring capabilities. While many signs of failure such as loss of mortar and residual displacement were observed in unretrofitted buildings at the Arts Center, the condition of the Chemistry building showed no evidence of hinge formation, loss of grout and residual displacement. It must be noted that both post-tensioned retrofitted buildings sustained moderate damage during the earthquake sequence, however, the overall performance of these structures was favorable resulting in significant cost saving. It must also be acknowledged that these retrofits were subject to budgetary constraints and that they were undertaken as part of a global scheme that also included other strengthening and securing techniques.

### 2.2 Current state of research

Besides summarizing the applications of post-tensioned masonry, Schultz and Scolforo (1991) reviewed the research carried out on this topic before 1990. In that era, various tests were conducted on masonry beams and walls to assess the effectiveness of post-tensioning in improving the overall in-plane and out-of-plane behavior. It can be concluded from their work that post-tensioning corroborates the strength, stiffness and integrity of the elements because the precompression delays crack formation and even if cracks do form, precompression closes them upon unloading. Additionally, the increase in the in-plane shear resistance of walls due to post-tensioning arises from the increased frictional resistance in the bed joints provided by precompression.

Lissel et al. (1999) summarized research conducted during the 1990s on post-tensioned masonry. In the out-of-plane loading, the research showed that low levels of prestress are effective in significantly increasing the resistance of cantilever diaphragm retaining walls. Research conducted in the 1990s has shown consistently that prestressing masonry improves the flexural cracking moment capacity as
Literature review

well as the ultimate moment capacity of walls. It has also been shown that prestressed masonry is simple to construct and that no special training is required. Other findings included low deflections in the uncracked walls, and high residual lateral load resistance after cracking. Prestressed masonry construction had also been shown to be cost-competitive with reinforced concrete and masonry construction.

The other important parameters that attracted the researcher’s attention were the anchorage of tendons as well as the prestress loss. In order for the system to work, on the one hand, the transfer of high post-tensioning stresses must be done without damaging the masonry wall. On the other hand, loss of prestress due to long-term masonry deformations and steel relaxation must be managed. Anchoring of post-tensioning elements in the foundation on one end and in a capping beam or concrete slab on the other end was used to solve the anchorage problem in real applications. Foti and Monaco (2000) reviewed the articles on the effects of creep, shrinkage and deformations due to humidity variations, apart from the loads applied to the masonry wall. For instance, Lenczner (1988) observed that for concrete masonry the creep coefficient (ratio of creep deformation to elastic deformation) depends on the stress level. For clay masonry, however, the creep coefficient is approximately constant in relation to the applied stress in the service load range. Based on the review by Ganz (1990), clay brick masonry was found to have low values of volume changes. The coefficient of thermal expansion is only about 60% of the value of concrete. Shrinkage shortening is usually compensated by expansion due to increase in humidity and final creep deformations have the same order of magnitude as the elastic deformations. For concrete masonry, the corresponding values are similar to those of concrete.

The need for investigation of post-tensioned masonry was emphasized by research in the last 20 years. They pointed out that the method seems to have economic benefits such as reduction in the enormous economic losses caused by earthquakes. Furthermore, this technique might be used in the protection and preservation of historical heritage where the structure must remain close to its original state.

Main research on the out-of-plane and in-plane behavior of post-tensioned masonry walls are described below. The research has been mostly done on evaluating the response of post-tensioned masonry walls in the design of masonry buildings. On the other hand, seismic vulnerability of the existing masonry buildings was well recognized by many investigations.

2.2.1 Post-tensioning for improving the out-of-plane behavior of unreinforced masonry walls

Within the framework of a research project on the response of masonry subjected to combined in-plane axial load and out-of-plane bending rotation, Mojsilovic and Marti (1996) performed experiments on six post-tensioned masonry walls made of calcium-silicate blocks and hollow clay bricks. The walls were constructed in single wythe with two different heights of 2.6 m and 5 m and a uniform length of 1.03 m. Post-tensioning was done using the VSL system with two vertical monostrands located symmetrically in length at the mid-thickness of walls. The axial load was selected to produce compressive stresses of either 0.65 or 1.94 MPa based on the gross cross sectional area of the walls. The base rotation was applied by a hydraulic jack installed to a lateral frame, which was fixed to the base slab. It was seen that post-tensioning enhances cracking loads, improves the cracking behavior and results in an increased flexural resistance of masonry walls. Furthermore, in post-tensioned walls the curvature was more uniformly distributed along the height of walls and several narrow cracks developed. The level of axial stress was also studied. It was seen that larger values of axial force correspond to larger bending strength but a more brittle response.
Lissel et al. (2000) examined the out-of-plane behavior of post-tensioned masonry walls but with geometric cross sections having two wythes and two webs connecting them together (Fig. 2.7). The geometric cross sections are normally used in retaining masonry walls. Post-tensioning was done with CFRP as an attractive solution to potential corrosion problems with unbonded tendons. Due to the high cross sectional area in this type of cross sections, the compressive stress provided by post-tensioning becomes lower as compared to single wythe walls. The out-of-plane load was applied by a hydraulic jack and was distributed along the height of the wall using spreader beams. In this geometry, it was also concluded that the shear resistance of the masonry wall increased, deflections remained small and cracks fully recovered after unloading.

![Cross section of a diaphragm wall with post-tensioning tendons (Lissel et al. 2000).](image)

Bean Popehn et al. (2007) studied the post-tensioning on slender masonry walls (height to thickness ratio as large as 40). These walls are normally found in tall commercial and single-story industrial buildings with relatively light roof loads. In this situation, where the axial loads are low and the transverse loads (i.e., perpendicular to wall surface) such as wind, earth pressure, and inertia forces from seismic excitation, are significant, using post-tensioning seems to be advantageous by increasing the axial load. A series of tests was conducted on slender masonry walls and the effects of prestress magnitude and tendon bond condition (bonded or unbonded) on the lateral load behavior of the walls was assessed. In addition to the positive effect of prestressing on the out-of-plane flexural resistance, all prestressed specimens showed large displacements before losing their load carrying capacity. This is beneficial in terms of seismic evaluation where the acceptability of a component is determined not only by its strength, but also by its displacement capacity.

### 2.2.2 Post-tensioning for improving the in-plane behavior of unreinforced masonry walls

One of the early investigations on the in-plane behavior of post-tensioned masonry walls was carried out by Page and Huizer (1988). Three types of reinforcement were used in various unreinforced masonry walls including one reinforced and grouted, one vertically prestressed and one vertically and horizontally prestressed. The masonry was constructed from hollow clay units and Type S mortar, which has a compressive strength of more than 10 MPa. Horizontal load was applied monotonically on top of the wall in the plane of wall and besides post-tensioning no vertical load was considered in the experiments. The tests showed that vertical prestressing results in a substantial increase in the shear strength and stiffness. In theory, horizontal prestressing would result in an increase in the shear capacity due to additional suppression of diagonal cracking; however, this was not proven by the experiment due to premature local failure at the location of the horizontal jack.
To be regarded as a new construction form in seismic areas, post-tensioned concrete masonry (PCM) walls were first studied in New Zealand by Laursen (2002). At the time, there was a worldwide lack of understanding of the in-plane seismic behavior of PCM walls, which differs considerably from the behavior of reinforced masonry. Therefore, Laursen performed various tests to study the in-plane behavior of PCM walls. A close look into this study helped in the design and execution of the experiments in this study. Therefore, it is explained here in more details.

Three series of large-scale tests were carried out at the University of Auckland to investigate various parameters such as shear resistance, displacement capacity and energy dissipation of PCM walls. The first series consisted of eight single-story walls with a height of 2.6 m. Six of them had fully grouted masonry, one partially grouted and one ungrouted. The variable parameters consisted of the wall length and thickness, number and location of post-tensioning tendons, and level of post-tensioning. Cyclic in-plane tests were performed under displacement control with increasing amplitude. It was seen that rocking is the dominant mode in fully grouted walls exhibiting mostly a nonlinear elastic response. A large displacement capacity was observed in the grouted walls with drifts beyond 1.4%. Damage was localized at the bottom corners and the amount of dissipated energy was limited. Partially grouted and ungrouted walls also developed significant shear strength, but the initial rocking mode concluded with shear failure in the walls.

The second series of experiments aimed at reaching a drift capacity of 1.5% to 2% by enhancing the PCM walls with several methods such as bed joint confining plates, energy dissipater “dog-bone” type bars, and special high-strength concrete blocks. Five unbonded fully grouted walls were tested with uniform dimensions of 3 m long, 2.6 m high and a thickness of 140 mm. Confining steel plates were installed at the toe and heel regions of the walls in several mortar layers. A high-strength fiber reinforced concrete block was used in one of the experiments right at the bottom corners of the wall. Special energy dissipaters were also used in another test. It was observed that the in-plane cyclic behavior of all walls was dominated by rocking reaching drifts in the range of 0.7% to 2.6%. The highest drift was observed in the PCM wall with confining steel plates located at the bottom corners with a vertical spacing of 100 mm. The shear strength was not much affected by the confining plates though. Hysteretic damping exhibited a significant increase by using the confining steel plates together with energy dissipaters. Use of the high strength concrete blocks only resulted in the shift of the damage to the neighboring blocks which was not an effective option.

To validate the use of PCM walls in a realistic structural configuration, the third series of tests was done by Laursen on two 67% scale models of a typical cantilever wall from a 4- or 5-story building. The walls contained unbonded prestressing tendons and confinement plates in the lower wall corners. Localized damage was observed together with ductile response with drifts up to 1.5% and relatively little energy dissipation.

It can be concluded that post-tensioned concrete masonry walls with confining plates can be used in seismic areas as a competent material combination for ductile structural wall systems.
Further research was carried out by Rosenboom and Kowalsky (2004) in the U.S.A. to assess the feasibility of usage of post-tensioned masonry for modular housing in seismic areas. Regarding the common materials in the region, clay bricks with holes were used together with a high compressive strength mortar for construction of the walls. Five double wythe masonry walls were constructed with the aspect ratio of two and a cantilever boundary condition incorporating post-tensioning tendons. In the study, the variables of interest were bonding of post-tensioning steel, confinement of masonry, grouting of masonry and application of supplemental mild steel for energy dissipation. Pseudo-static cyclic in-plane test was done on the post-tensioned walls. Findings similar to those of Laursen et al. (2002) were achieved here stating that the best performing wall was fully grouted with unbonded prestressing and embedded confinement plates at the compression zones. Mild steel reinforcing bars did provide additional hysteretic
damping. About the grouting of masonry and bonding of post-tensioning, it was recommended to avoid these conditions due to the significant damage sustained and poor structural performance observed during testing.

In a follow-up project at the University of Auckland, post-tensioned concrete masonry walls were evaluated by dynamic shaking table tests (Wight et al. 2006). The self-centering nature of post-tensioned walls that had been demonstrated previously using pseudo-static loading histories was confirmed by dynamic testing. The test results also demonstrated the ability of PCM walls with confining plates to undergo a large number of cycles with minimal accumulation of damage. Small residual displacements could be expected due to post-tensioning, provided that the residual prestressing force is sufficiently large to bring the wall to its original position.

2.2.3 Seismic evaluation and retrofitting of masonry buildings

History has clearly highlighted the vulnerability of masonry buildings to earthquakes all over the world. In April 2009, the Aquila earthquake in Italy with a magnitude of 6.3 showed the necessity of retrofitting existing buildings in Europe (Fig. 2.9). Switzerland has experienced destructive earthquakes throughout its history. Most notable were the events of 1356 in Basel and 1855 in Valais. Although such events are very rare, their intensity is comparable to the major earthquakes of Northridge, California 1994, and Kobe, Japan 1995 (Lang 2002). Extensive research has been conducted to understand the seismic behavior of masonry buildings and design earthquake resistance masonry buildings (Tomazevic 1999; Paulay and Priestley 1992; Bruneau 1994).

Failures of unreinforced masonry (URM) structures can be categorized into connection failure, diaphragm failure, out-of-plane wall failure and in-plane wall failure. The in-plane wall failure includes rocking, sliding, diagonal cracking and toe crushing (Applied Technology Council (ATC) 2000). The last two failure modes are counted as brittle.

Lately, a coordinated European research was executed by Magenes et al. (2008) to gain additional information on the cyclic behavior of masonry structures, with regard to their response to seismic excitations. Hollow clay, calcium silicate, and lightweight aggregate concrete blocks were used in the construction of unreinforced masonry walls. The variables of interest in this program included the aspect ratio of walls and vertical loads, aiming at investigating the cyclic behavior of walls affected by different
failure mechanisms such as flexure, shear, and mixed mode. Two vertical servo-hydraulic jacks were utilized in a way to provide a double fixed boundary condition for the walls. The results exhibited a wide variation in ductility and drift capacity depending on the failure mode, which is in turn influenced by masonry typology, geometry, level of axial load, and boundary conditions. When diagonal cracking in the walls is avoided, high drift capacities can be attained, sometimes exceeding 1.0% (in a fixed-fixed boundary condition) or more, whereas very brittle behavior is reported when diagonal cracks develop through the units. In particular, a very low drift capacity (below 0.25%) was reported in the presence of high vertical compression stress (0.68 MPa). Limiting compression stresses in walls appears to be an important seismic design criterion to avoid poor performance, because the increase in shear strength due to axial compression may not compensate the dramatic reduction in deformation capacity.

Different methods have been studied in the literature for seismic retrofitting of masonry walls as the main lateral load carrying elements in masonry buildings. ElGawady et al. (2004) reviewed conventional retrofitting methods including surface treatment, grout and epoxy injection, external reinforcement using steel plates or tubes, confining URM walls using RC tie columns and post-tensioning. Traditional methods, such as surface treatment and grout injection, have been shown to be effective; however, they are typically expensive, disruptive to building occupants, and have a negative impact on building aesthetics. The latter is particularly of high importance for historic structures. To alleviate these shortcomings, modern techniques which involve the use of fiber reinforced plastic FRP overlays, near surface mounted (NSM) rods, and vertical post-tensioning have received attention. Some research in which different retrofitting techniques have been implemented is reviewed here.

Elgawady (2004) performed a set of cyclic in-plane tests on URM walls and then repaired the damaged walls by applying mostly GFRP composite layers and also post-tensioning. The walls were then retested. It was found that both methods increased the shear resistance of the walls more than twice. However, improvement in displacement capacity was much less significant.

Moon et al. (2007) investigated the effectiveness of several seismic strengthening techniques on a full-scale two-story URM building. The masonry was composed of clay bricks and type K mortar, a low strength mortar, to be consistent with the material properties obtained from common construction practices in the U.S.A. prior to 1950. First, cyclic tests were conducted on the building using horizontal hydraulic jacks installed at each story level in the plane of walls. Then, in the second phase, the cracked structure was strengthened and retested. The building had four URM walls, on which different retrofitting methods were applied including fiber reinforced plastic overlays, near surface mounted rods, and vertical post-tensioning (Fig. 2.10). The repairs applied to Wall A (post-tensioned) and Wall B (bonded GFRP strips and NSM GFRP rods) were the most effective resulting in strength gains in excess of 50%. In the other two walls, in contrast, the repairs resulted in modest strength gains below 20%. Due to uplift associated with local piers and global rocking, the force in the post-tensioning elements increased. The compressive stress on piers was, therefore, increased and as a result diagonal cracking, a brittle failure, occurred in the piers. It was recommended for the piers to be retrofitted with additional horizontal shear reinforcement.

In a seismic assessment program, Abrams et al. (2007) studied several rehabilitation techniques for improving the seismic behavior of masonry piers. The effectiveness of each technique is judged relative to the strength as well as ductility of control piers with no rehabilitation, which is governed by a nonlinear elastic rocking mode of behavior. The retrofitting methods considered in this study were bonded FRP strips, reinforced shotcrete overlay, ferrocement surface coating, and grouted reinforcing bars within
drilled cores. The results showed that even though the FRP rehabilitated specimen had high strength, the overall seismic capacity, defined as the product of strength and ductility factor, was the lowest of all specimens, including those without any rehabilitation, due to the low ductility.

(a) ![Diagram](image1.png)  (b) ![Diagram](image2.png)

(c) ![Diagram](image3.png)  (d) ![Diagram](image4.png)

Fig. 2.10 Retrofitting techniques on (a) Wall 1 with bonded unidirectional GFRP strips, (b) Wall 2 with bonded bidirectional glass fiber reinforced cement layers, (c) Wall A with unbonded post-tensioning, and (d) Wall B with bonded unidirectional GFRP strips on the interior face and NSM GFRP rods bonded into the bed joints every third course on the exterior face (Moon et al. 2007).

The shotcrete rehabilitated specimen behaved as a reinforced concrete pier with little or no evidence of composite action with the masonry. This rehabilitation displayed a large displacement capacity and energy dissipation. The effectiveness of shotcrete, ferrocement overlay, and grouted reinforced core techniques was roughly equal to that of the non-rehabilitated piers because of the extremely high ductility of the non-rehabilitated pier due to rocking. Therefore, it was recommended that the best approach in rehabilitation of URM piers and walls might be to do nothing to the structural elements, provided that rocking is the failure mode.
2.3 Summary and conclusions

As described in this chapter, the post-tensioning technique has been used successfully in practice for the design and strengthening of masonry structures. During the last two decades, there has been ample research on the in-plane and out-of-plane behavior of post-tensioned walls with different geometries and materials. The construction system of post-tensioned masonry walls with confined compressive corners has also attracted the attention of researchers to be used as a competent system for construction of buildings in seismic areas. The seismic vulnerability of masonry buildings was emphasized and several conventional retrofitting methods investigated in the literature were assessed. Compared to other retrofitting methods, post-tensioning was found to be a promising method for enhancing the shear resistance of URM walls and for increasing their integrity. However, this method might change the failure mode of the URM walls and piers to a brittle diagonal cracking mode by applying high axial stress. The following gaps can be found in the state of research and application of post-tensioning technique in retrofitting existing URM walls:

- Post-tensioning has not been studied as a seismic retrofitting method for enhancing the in-plane behavior of weak masonry walls and piers.
- Compared to the other retrofitting methods, only few applications of the post-tensioning technique are known; there is a lack of knowledge on the seismic response of post-tensioned masonry walls with different prestressing levels, aspect ratio, etc.
- An experimental study is required to understand the effectiveness of the method on the seismic behavior of existing masonry walls.

2.4 References


3 Material tests

Masonry is a composite material composed of mortar and brick. The mechanical behavior of a composite material depends on its constituents as well as the bonding between them. In order to understand the mechanical behavior of masonry, it is fundamental to study its components and the interface between them. Since this study aims at existing residential buildings constructed in the beginning of the 20th century, the corresponding materials must be investigated. These buildings are characterized by solid clay bricks and lime or lime-cement mortar. A series of tests were conducted on mortar and clay bricks as well as masonry wallets to understand the behavior of the material.

It should be mentioned that two series of large-scale experimental studies were carried out during this project in order to assess the effect of post-tensioning on unreinforced masonry walls. The first study aimed at understanding the feasibility of applying post-tensioning on URM walls. In the second study, a complementary set of tests was done on URM walls in order to assess the effect of normal stress on the walls. During each study, several material tests were carried out, which are described in this chapter. The material tests regarding each study are named according to their set of study.

3.1 Mortar

The mortar mix was designed to represent the weak mortar used about 100 years ago for residential buildings. For this purpose, various mortar mixtures were considered and tested for their compressive strength. Finally, a lime-cement mortar with a volume proportion of cement-lime-sand of 1-2-9 was found to have a low compressive strength representing the desired weak mortar. The mortar ingredients are shown in Table 3.1. The Cellulose Ether was used in order to decrease the compressive strength.

<table>
<thead>
<tr>
<th>Ingredients in a 100 liter mix</th>
<th>weight [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland Cement, Type II</td>
<td>13500</td>
</tr>
<tr>
<td>Lime, Filler 93% CaCO3</td>
<td>26000</td>
</tr>
<tr>
<td>Sand (50% 0-1 mm, 50% 1-4 mm)</td>
<td>175000</td>
</tr>
<tr>
<td>Water</td>
<td>27778</td>
</tr>
<tr>
<td>Cellulose Ether</td>
<td>44</td>
</tr>
</tbody>
</table>

Two types of tests were conducted on the mortar. The first type consisted of standard tests for obtaining the main characteristics of the mortar including the compressive strength, tensile strength and Young’s modulus. The geometry of the samples followed the rules specified by the standards. The second type, however, included nonstandard compression tests with various geometries. In this type, some samples had the same geometry as the standard tests, in which the goal was to capture the stress-strain behavior. Other samples were designed to have a layer like geometry in order to investigate the behavior of mortar under 3D stresses similar to those in a masonry assemblage.
3.1.1 Standard tests on mortar prisms (first type)

The specimens of this series of experiments were cast into standard steel molds. The specimens were 160 mm long and had a cross section of 40x40 mm². The flexural-tensile strength and the compressive strength of mortar were determined according to (EN 1015-11 1999). Based on this standard, a three point bending test shall be conducted on the samples to measure the flexural-tensile strength. Thus, the samples split at mid-length into two parts. Subsequently, a compression test shall be performed on each part using a compression machine with a loading platen of 40x40 mm². The compressive strength of mortar shall be calculated as the average of the compressive strength of the two parts. Fig. 3.1 illustrates the test set-up.

![Test setup for measuring (a) the flexural-tensile strength and (b) the compressive strength of mortar prisms.](image)

The results of the compressive and flexural tensile tests for all samples are shown in Fig. 3.2. As seen in the figure, the average strengths in the first study were generally lower than those obtained in the second one. This is presumed to be due to the type of cellulose used for the studies. In the second study, methyl cellulose was used instead of cellulose ether; this was the only ingredient that changed from the first study. The other ingredients as well as the curing conditions were kept the same in both studies. A detailed investigation of the mortar mixture was not conducted in this study.

A compressive strength of 2.5±0.73 MPa and 4.5±0.31 MPa and a flexural-tensile strength of 0.9±0.27 MPa and 1.5±0.39 MPa were obtained from the experiments in the first and the second study, respectively.

The Young’s modulus was only determined in the second study. The test setup is shown in Fig. 3.3. According to (SIA 262 2003), Switzerland’s standard for designing concrete structures, a cyclic compression load producing a stress of 1/3 of the compressive strength was applied to the specimen. A minimum level of stress was chosen as 0.6 MPa. The vertical displacement was measured during the third cycle at the minimum and maximum compression. The measurement was made by a 100 mm compressometer, which was placed symmetrically at mid-height of the specimens. Testing on five samples resulted in a mean value of 7800 MPa of Young’s modulus with a standard deviation of 2.7%. Table 3.2 summarizes the test results of the standard tests.
Fig. 3.2 (a) Compressive strength and (b) flexural-tensile strength of mortar samples.

Fig. 3.3 Young’s modulus test of mortar.

Table 3.2 Experimental results of standard tests on mortar prisms

<table>
<thead>
<tr>
<th>Study</th>
<th>Compressive strength [MPa]</th>
<th>Tensile strength [MPa]</th>
<th>Young’s modulus [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>First study</td>
<td>2.5 ± 0.73</td>
<td>0.9 ± 0.27</td>
<td></td>
</tr>
<tr>
<td>Second study</td>
<td>4.5 ± 0.31</td>
<td>1.5 ± 0.39</td>
<td>7800 ± 210</td>
</tr>
</tbody>
</table>

3.1.2 Nonstandard compression tests on mortar (second type) using a 3D image correlation system (ICS)

The standard tests on mortar prisms provide useful information about the characteristics of the mortar in a one-dimensional state. Only the strength values and Young’s modulus of the mortar are measured in the standard tests and the stress-strain behavior is not recorded. Moreover, these experiments do not represent the three-dimensional behavior of the mortar confined with clay bricks in a masonry wall. Therefore, several tests were designed to be done on the mortar with a geometry which is closer to the state of a mortar layer in a masonry wall. A 3D Image Correlation System (ICS) was used to measure the full-field
displacement of the front surface of specimens at different load steps. Therefore, the stress-strain curves could be captured by this method.

The 3D image correlation system functions by taking photos of a deformable object using stereo cameras at different stages of deformation. By comparing the photos of the object during deformation with the photo of the initial state, the deformation is obtained quantitatively. The system encompasses two 4-mega pixel cameras with a focal length of 20 mm. A random pattern is painted on the wall that allows comparing the two stereo photos of a specimen in a deformed state with those of the initial state. The ICS compares the so-called facets of 15×15 pixels. With this setup, the displacement resolution is in the range of 0.1 to 0.01 pixels.

Table 3.3 shows the specifications of different specimens of this series of experiment. Note that the height mentioned in the table is in fact the mortar thickness. The dimensions of the first and second specimens were equal to those used in the standard Young’s modulus and compression tests, respectively. The geometry of the third specimen was selected in order to see the effect of confinement on the compressive behavior of the mortar. Finally, the aim for selecting the last geometry was to have a similar boundary condition and geometry of the mortar layer in a masonry wall.

Fig. 3.4 shows the specimens with various geometries placed in the compression machine. The specimens were painted with a stochastic pattern for the optical measurement. The first three specimens were placed between two steel plates painted in a stochastic pattern. The reason is that during the experiment, the outer surface of the specimens may experience a significant out-of-plane deformation. Eventually, the outer surface, which is the measuring surface, is lost and the measurement becomes impossible. However, the outer surface of the steel plates remains relatively undeformed even after the failure of the mortar. The axial strain of the mortar can therefore be calculated based on the relative displacement of the steel plates up to the failure of the mortar.

<table>
<thead>
<tr>
<th>Name</th>
<th>length [mm]</th>
<th>height [mm]</th>
<th>width [mm]</th>
<th>number of samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mo40x160</td>
<td>40</td>
<td>160</td>
<td>40</td>
<td>2</td>
</tr>
<tr>
<td>Mo40x40</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>2</td>
</tr>
<tr>
<td>Mo200x20</td>
<td>200</td>
<td>20</td>
<td>120</td>
<td>5</td>
</tr>
<tr>
<td>BMo250x20</td>
<td>250</td>
<td>20</td>
<td>120</td>
<td>3</td>
</tr>
</tbody>
</table>

The axial force was measured by the load cell of the compression machine. A 1 MN compression machine was used for testing the specimens Mo200x20 and BMo250x20. For the specimens with lower cross sectional area, i.e. Mo40x160 and Mo40x40, a 200 kN compression machine was used because a lower compressive strength was expected for these specimens. For the specimen Mo200x20, several tests were executed using the lower capacity compressive machine (200kN) in order to capture a more accurate cyclic behavior of the confined specimen at lower axial forces. The normal stress was calculated by dividing the axial force with the entire cross sectional area.

For each specimen several tests were carried out under various load histories. To determine the elastic stiffness, the specimens were unloaded and reloaded at certain displacements. The displacement at which the unloading was applied was not the same in all tests.
3.1.2.1 Specimen Mo40x160

Fig. 3.5(a) illustrates six points considered for measuring the vertical displacement and strain of the specimen Mo40x160. The relative displacement between two points can be considered as a virtual strain gauge connected to the front surface of the specimen. These strain gauges are shown in Fig. 3.5(b) on the unloaded state of the specimen. In this figure, the contour shows the 3D position of different points.

Strain gauge 1 takes into account the axial deformation of the whole sample including the local displacements at the upper and lower interfaces. These local displacements are measured by the strain gauges 4 and 5. Strain gauge 2 records the vertical strain of the sample over its entire height. Finally, strain gauge 3 measures the strain of the samples far from the loading plates. The points were chosen at the same location as for the standard Young’s modulus test.

The stress-strain behavior of Specimen Mo40x160 using strain gauges 1, 2, and 3 is depicted in Fig. 3.6. Markers are drawn at the measurement stages. It is seen in the figure that there is a clear difference between the strain measured by strain gauge 1 and strain gauges 2 and 3 but not a notable difference was observed between strain gauges 2 and 3, which were installed on the specimen. This is due to the settlement.
settlement of the steel plates and the specimen. For more clarification, Fig. 3.7 depicts the history of the cumulative relative vertical displacements of the strain gauges 2, 4, and 5 at different load steps. It is obviously equal to the history of the relative vertical displacement measured by strain gauge 1. As seen in Fig. 3.7, the local vertical displacement at the interfaces does not remain constant after the initial settlement. It can be concluded that the contact between the steel plates and the specimen does not have an infinite stiffness. Therefore, in the stress-strain behavior measured by strain gauge 1, the stiffness of the interface is influential. As a result, the slope of the stress-strain curve in the linear part measured by strain gauge 1 is lower than the one measured by strain gauges 2 and 3, see Fig. 3.6.

Care would have to be taken in interpreting the results. For instance, Fig. 3.6(a) shows a negative strain at the initial loading steps. This could be attributed to two reasons. First, at low stresses the amount of vertical displacement measured optically by the ICS is in the range of measurement error. The other and more important reason is that at early stages of loading during the settlement of the steel plates, some displacements occur in the horizontal and also out-of-plane directions. Since the ICS functions on the basis of the images taken from different states of the specimen, these deformations may affect the measured vertical deformation and result in instabilities of the measurement at low stresses. After the settlement of the pieces in the compression machine, increasing the stress results mostly in increasing the strains in the same direction compared to the other directions and the measurement becomes stable.

The modulus was calculated based on the secant slope at stress states similar to those used in the standard Young’s modulus test. The lower stress limit was considered as 0.6 MPa similar to the standard test. Based on SIA 262/1, the upper stress limit was considered as 1/3 of the compressive strength of the specimen, which was approximately equal to 4.5 MPa in this series of experiments. The modulus of the monotonic sample was equal to 6600 MPa. The modulus of the cyclic test in the unloading and reloading was equal to 8300 and 8800 MPa, respectively.
These moduli are not equal to the modulus of elasticity measured in the standard test because of the different loading protocol applied to the samples in the nonstandard test. Moreover, as the number of samples was limited, a statistical value cannot be reported for the modulus of elasticity of the samples using the nonstandard tests. However, it can be concluded that the secant slope of the stress-strain curve in the monotonic test is lower than the modulus of elasticity. This can be attributed to the deformations in the porous structure of the mortar that results in a history dependent secant slope. The porous structure of the mortar is not the same in the third cycle and the first cycle of loading due to the probable collapses in the pores of the mortar as well as the initiation and propagation of micro-cracks. As defined in the standard, the loading branch in the third cycle of axial test shall be considered for capturing the modulus of elasticity of the mortar, which has experienced an axial stress of 1/3 of the compressive strength.

The mean compressive strength of the monotonic sample, 4.2 MPa, was considerably close to the result of standard test. However, in the sample under cyclic loading, Fig. 3.6(b), a lower compressive strength was observed, which was about 3.5 MPa. This can be justified by comparing the failure mode of the samples,
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see Fig. 3.8. It is most likely due to an existing weak plane in the specimen, which resulted in a premature shear failure close to the steel plates.

3.1.2.2 Specimen Mo40x40

Seen in Fig. 3.4(b), the specimen Mo40x40 was in fact one of the two split parts of a standard mortar prism after being tested for the flexural tensile strength. One part was used for the standard compression test and the other one for the nonstandard compression test using ICS in order to find the stress-strain behavior.

Two measurement points on the steel plates, points 1 and 4 in Fig. 3.9(a), were considered for measuring the axial strain that include the local displacement at the upper and lower interface levels. Two other measurement points on the specimen, points 2 and 3 in Fig. 3.9(a), were used for calculating the strain of the specimen. The measurement points where selected far from each other in order to increase the accuracy of the strain measurement. The strain gauges corresponding to the measurement points are shown in Fig. 3.9(b).
The stress-strain curves of both samples are drawn in Fig. 3.10. Since the axial strain occurs due to elastic and inelastic deformations, the initial slope of the stress-strain curve is considered as the modulus of deformation. The modulus of deformation was calculated as a secant slope of the stress-strain curve at load stages similar to those described for Specimen Mo40x160. Table 3.4 gives a summary of the modulus of deformation in various loading branches. Strain gauge 2 takes into account the relative displacement of the specimen and it excludes the local displacement at the lower and upper interfaces between the steel plates and the sample. Therefore, similar to Specimen Mo40x160, the modulus of deformation measured by this strain gauge at a certain load history represents the elastic modulus of the specimen.

Table 3.4 Modulus of deformation [MPa] of Specimen Mo40x40 measured with different strain gauges

<table>
<thead>
<tr>
<th>Load type</th>
<th>Monotonic loading</th>
<th>Cyclic loading</th>
<th>Cyclic reloading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strain gauge 1</td>
<td>2100</td>
<td>3900</td>
<td>6100</td>
</tr>
<tr>
<td>Strain gauge 2</td>
<td>6400</td>
<td>7800</td>
<td>9200</td>
</tr>
</tbody>
</table>

Fig. 3.10 Stress-strain behavior of Specimen Mo40x40 in (a) monotonic and (b) cyclic tests.

The amount of local displacement at the interfaces can be seen in Fig. 3.11. Due to settlement of the steel plates at the first stages of loading, a high amount of displacement is measured by strain gauge 1. By increasing the axial stress, there is an increase in the compressive strain of the specimen as well as local displacements. However, the contribution of the local displacement to the total deformation reduces. In both cases this contribution decreased to about 40% of the total strain measured by strain gauge 1, which is equivalent to about 0.1 mm. As stated earlier, the existence of the local displacement results in the lower slope of the stress-strain curve of strain gauge 1 in comparison with that of strain gauge 2 installed on the specimen.

It should be noted that when a specimen reaches its compressive strength, it expands in horizontal directions and as a result out-of-plane deformations occur. As the out-of-plane deformation develops, the validity of the vertical deformation measurements of the points located on the specimen is reduced. The shaded area in Fig. 3.11 represents the zone with a high out-of-plane displacement, in which the validity of the measurement of vertical strains is questionable. In order to have a realistic estimate of the axial strain of the specimen using the measurement points on the mortar surface, only the load stages should be
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considered in which the out-of-plane deformation is negligible in comparison with the vertical deformation.

For more clarification, the history of the out-of-plane displacement of the measurement points for the monotonic sample is shown in Fig. 3.12(a). The contour of out-of-plane deformation of the sample is also illustrated in Fig. 3.12(b).

It can be observed in Table 3.4 that the value of the modulus of deformation is higher during unloading and reloading. It is known that the nonlinear behavior of mortar is due to closure of pores and creation and propagation of micro-cracks. Therefore, during loading for the first time unrecoverable deformations occur in the mortar sample. During unloading, however, the normal stress at the micro-crack plane is high and the relative deformation decreases due to high frictional stress. Therefore, the deformation of mortar occurs mostly due to its elastic behavior. As a result, the unloading and reloading branches of the cyclic tests give a better estimate for the elastic behavior of mortar. This behavior has been reported in the literature for rock materials under compression, see Li et al. (1998).
The stress-strain curve shown in Fig. 3.10 gives an average of 4.4 MPa for the compressive strength of the mortar sample, which is close to the standard test results. The failure state of the specimen for monotonic and cyclic loading is depicted in Fig. 3.13. Macro-cracks were observed in the contour of the maximum principal strains.

![Fig. 3.13 Contour of maximum principal strain at failure state of Mo40x40 under (a) monotonic and (b) cyclic compression tests.](image)

### 3.1.2.3 Specimen Mo200x20
Specimen Mo200x20, Fig. 3.4(c), was designed to investigate the behavior of a mortar layer that is confined in a masonry wall. The mortar was molded in a wooden formwork in a way to produce smooth 200×120 mm² surfaces which were the loading surfaces, see Fig. 3.14. This was to minimize the roughness of the surface and have a better distribution of stress over the loaded area.

![Fig. 3.14 Formwork of the specimen Mo200x20.](image)
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Fig. 3.15 Illustration of (a) measurement points used for displacement and strain measurement and (b) strain gauges on the 3D position view of Mo200x20.

The mortar layer was placed between two steel plates with a stochastic pattern in order to measure the displacements using the 3D ICS. Similar to Specimen Mo40x40, four measurement points were considered for measuring the vertical displacements. They are shown in Fig. 3.15(a). Four virtual strain gauges were created for measuring the relative displacement of the measurement points. These strain gauges are illustrated in Fig. 3.15(b).

In total five experiments with different load protocols were conducted on Specimen Mo200x20. One monotonic and two cyclic tests were carried out using a 1 MN compression machine and two other cyclic tests using a 200 kN compression machine. As an example, the history of vertical displacement (Displacement Y) and out-of-plane displacement (Displacement Z) of the measurement points for one of the cyclic tests using the lower capacity compression machine is shown in Fig. 3.16(a) and (b), respectively. As seen in the figure, the out-of-plane displacement of the measurement points on the sample, i.e. measurement points 2 and 3, was significant at high load stages. During reloading, when the vertical displacement increases from its maximum level in the previous stages, a jump in the out-of-plane displacement is observed. However, during unloading and reloading to the previous amount of vertical displacement, the out-of-plane displacement remained relatively constant. It can be concluded that during

Fig. 3.16 Displacement history of measurement points in (a) Y direction (vertical) and (b) out-of-plane direction.
the increase in the vertical displacement, the outer surface gradually becomes separated from the specimen and its behavior is not compatible with the behavior of mortar confined between steel plates.

Fig. 3.17 shows the relative displacement of the measurement points obtained by four strain gauges. It can be seen in Fig. 3.17 that by increasing the relative vertical displacement of steel plates, measured by strain gauge 1, strain gauge 2 shows a negligible deformation whereas strain gauges 3 and 4 are dominant in the measured relative vertical displacement. It is of high importance to address the issue of dominancy of interface deformations when measuring the vertical strain of the mortar using strain gauge 1. Very low values of modulus of deformation result when this strain gauge is used for drawing stress-strain diagrams.

![Fig. 3.17 History of relative displacement of the measurement points of the specimen Mo200x20.](image)

The high value of local displacement at the interface measured by ICS can be explained considering two different loading levels, shown schematically in Fig. 3.18. It should be reminded that the ICS measures the deformations of the visible surface of objects. At the early load stages, state 1 in Fig. 3.18, the out-of-plane displacement of the measured surface is negligible and the surface remains vertical. As the strain in the specimen is low, the strain measured by strain gauge 1 represents mostly the local displacements at both interfaces measured by strain gauges 2 and 3. These local deformations are due to the settlement of pieces and also the roughness of the surfaces which are in contact. By increasing the axial force, state 2 in Fig. 3.18, the outer surface experiences a significant out-of-plane deformation. Strain gauge 2 measures the distance between measurement points on the outer surface of the mortar which is either curved or split from the mortar. So, strain gauge 2 installed on the mortar does not provide the vertical strain in the

![Fig. 3.18 Schematic illustration of the relative vertical displacement of mortar layer and steel plates at different loading states.](image)
confined mortar. Moreover, since the outer surface of the specimen is not at the same plane as the steel plates, the vertical deformations measured by strain gauges 3 and 4 include the overlap of the two surfaces and don’t stand exclusively for the local displacement at the interfaces.

In conclusion, strain gauge 1 shall be used for measuring the vertical strain because of the limited out-of-plane deformation in steel plates. At low values of axial force, the modulus measured by this strain gauge is not equivalent to the modulus of elasticity of the mortar because of the local deformations occurring at interfaces. At high values of axial force, however, the relative displacement of the steel plates, measured by strain gauge 1, provides a good estimation of the modulus of elasticity of the confined mortar.

The compressive stress-strain behavior of Specimen Mo200x20 under various loading schemes is shown in Fig. 3.19. The vertical strain was measured using strain gauge 1. As seen in the figure, all graphs show an upward concavity at the beginning of the loading. This is attributable to the settlement of the mortar layer between the steel plates. Therefore the modulus of elasticity of the mortar layer cannot be extracted from the initial loading part of the curve. Afterwards, a linear stress-strain behavior is observed in all cases, which can be recognized as hardening of the mortar. The average modulus of hardening is 300 MPa approximately. The unloading and reloading curve, however, shows a high slope. This shows that due to unloading and reloading only the recoverable or elastic deformation occurs and therefore, this region of the load story is suitable for capturing the modulus of elasticity of the confined mortar. The average value of the modulus of elasticity calculated based on the unloading/reloading branch of the stress-strain curves for Mo200x20S4 and Mo200x20S5 is approximately equal to 10000 MPa.

In summary, the stress-strain behavior of the confined mortar can be considered as a bilinear behavior. The modulus of 10000 MPa can be considered for the elastic part and a modulus of 300 MPa for the hardening. Based on the experimental results, the compressive strength of the mortar can be considered as the stress, at which the behavior changes from elastic to plastic. The compressive strength of mortar was measured to be equal to 4.5 MPa. In order to generate the settlement an exponential behavior must be assigned to the interface or contact between the steel plates and the mortar layer.

3.1.2.4 Specimen BMo250x20
Specimen BMo250x2, Fig. 3.4(d), was designed to investigate the behavior of a mortar layer confined in a masonry wall. For this purpose, the specimens were produced by constructing a stack bonded masonry assemblage using two solid clay bricks and a mortar layer between them. Surface treatment was done on the bricks in order to have smooth loading surfaces. The assemblages were adjusted in a way to make the upper and lower loading surfaces parallel to each other.

The outer surface of the specimens was painted to prepare a stochastic pattern required for the ICS. No treatment of the surface was done on the mortar. Therefore, the mortar surface was not smooth and small holes existed in the surface. Depending on the dimension of a hole, it might happen that some pixels of the image are not visible to both cameras. Therefore, the measurement of that region is not possible. Fig. 3.20 (a) shows the captured pixels in green. Similar to Specimen Mo200x20 measurement points and strain gauges were defined, see Fig. 3.20(a) and (b), respectively. The contour in Fig. 3.20 (b) stands for the 3D position of the specimen at the beginning of the experiment.
The stress-strain behavior of Specimen BMo250x20 under various loading schemes is illustrated in Fig. 3.21. The normal strain was calculated using strain gauge 1, which includes the local displacements at the upper and lower interfaces. Therefore, as long as the settlement of the specimens continues, the modulus of deformation captured from this curve is lower than the actual modulus of the mortar. But afterwards, the slope of the stress-strain curve represents the modulus of the mortar layer. The secant slope
between 10 and 15 MPa is equal to 600 MPa with a standard variation of 21%. This is considered as the hardening modulus of the mortar. Compared to the hardening modulus of Mo200x20, the mortar layer
between steel plates, the hardening stiffness is higher in BMo250x20. This is attributable to the higher friction and cohesion between clay bricks and mortar compared to the contact surface between steel plates and mortar. The modulus of elasticity of the mortar can be considered to be similar to that of Mo200x20 in a bilinear behavior. As opposed to the mortar layer test between steel plates, in which the slope of the plastic region was constant, in this series of test, the slope decreases. This is due to the initiation and propagation of tensile cracks in the clay bricks. It results in a reduction in the slope of the stress-strain curve.

The full-field contour of out-of-plane displacement as well as the axial compressive strain of the specimen for several stress states are shown in Fig. 3.22. The local strain at the upper interface is clearly seen even at the low stress level of 4 MPa. By increasing the normal stress to 10 MPa, the out-of-plane displacement

Axial stress = 4 MPa

Axial stress = 10 MPa

Axial stress = 20 MPa

Axial stress = 28 MPa

Fig. 3.22 Contour of out-of-plane displacement (left) and axial compressive strain (right) at various axial stress levels
remains limited, while the local strain at the interface increases. At higher values of the normal stress, even though the strain in vertical direction shows an increase at both interfaces, the out-of-plane contour displays that the outer surface of the mortar deforms significantly. As a result, the visible surfaces of the mortar and bricks do not remain in the same plane, see Fig. 3.22(c). Therefore, strain gauges 3 and 4 do not exclusively measure the relative vertical displacement at the interface between the brick and mortar but the overlap of the bricks and mortar. Fig. 3.22(d) illustrates the failure of the masonry assemblage. The failure in the specimens is due to lateral tensile stresses in the bricks. This is in agreement with failure mechanism of masonry described in the literature, e.g. (McNary and Abrams 1985).

3.2 Brick

Solid clay bricks common for construction of masonry buildings in the beginning of the 20th century. Even though clay bricks have been used in the construction of many old buildings, their use is poorly documented and the design was mainly based on experience and rough guidelines (Lang 2002). SIA 113-43 (1943) was the first provisional standard in Switzerland for the “Design and execution of natural stone and manufactured brick masonry structures”, which was published by the Swiss Society of Engineers and Architects. According to the standard, the compressive strength of the brick units was required to be 22 and 35 MPa for normal brick units (N) and high grade brick units (H), respectively.

The bricks used for this study were solid clay bricks manufactured in Switzerland with a guaranteed compressive strength of 28 MPa. These bricks are produced nowadays for façades and not for structural walls. Bricks with a lower compressive strength are not produced anymore in Switzerland. The bricks had a nominal size of 250 mm×120 mm×60 mm.

A series of experiments was conducted on the brick in order to obtain its mechanical characteristics. During the first study, only a standard compression test was carried out. In the second study, various tests were performed to obtain compressive behavior and lateral tensile behavior of the bricks. 3D ICS was employed to capture the stress-strain behavior of the specimens.

3.2.1 Standard compression test on bricks

To measure the compressive strength of the bricks various configurations were used throughout the first study:

1. The bricks were cut into two halves and then one half was placed on top of the other.

2. The bricks were tested as a whole in the direction perpendicular to the bed joints in a masonry wall which is known as X direction according to SIA 266.

3. The bricks were tested as a whole in the direction parallel to the bed joints in a masonry wall which is known as Y direction according to SIA 266.

4. Cubes with an edge length of 60 mm and a height of 54 mm were cut from the bricks and tested in X direction (a) and Y direction (b) according to the definition of SIA 266.
The test procedure followed the standard (EN772-1 2000). The load rate was 0.3 N/mm²s. Table 3.5 shows the result of the compression tests. The values of the compressive strength were already modified by the shape factor according to the standard.

The configurations 1, 2, and 4a provide the compressive strength of the brick in X direction and 3 and 4b provide that in Y direction. Considering the shape factors according to EN772-1, mean values of 32.0 MPa and 48.8 MPa resulted for the compressive strength of the brick in X and Y direction, respectively.

### 3.2.2 Nonstandard compression test on brick samples

Aiming at obtaining the stress-strain curve of bricks under axial compressive load in the X and Y directions, i.e. perpendicular and parallel to the bed joints in a masonry wall, respectively, a series of tests were conducted in the second study. The 3D image correlation system (ICS) was used for measuring the deformations.

To prepare the samples for testing in X and Y direction, a masonry unit was cut as shown in Fig. 3.23. Before cutting the brick, its expected loaded surfaces were grinded to become smooth. The slenderness ratio of 3 was chosen in order to reduce the confining effect of the loading platens of the compression machine. For doing the compression test in X direction, three parts were put on top of each other. However, for the same test in Y direction there was only one part. The front surface of both samples was painted to produce a stochastic pattern to be scanned by the ICS during the image analysis.

![Compression test in Y direction](image1.png)
![Compression test in X direction](image2.png)

**Fig. 3.23 Preparation of the bricks samples for testing in X and Y directions.**
Three tests were conducted for each direction. The axial strain measurement was done by installing virtual strain gauges connecting the measurement points on the front surface of the samples. Fig. 3.24 shows the measurement points and the virtual strain gauges considered for calculating the axial strain. Strain gauge 1 was installed on the upper and lower steel plates. As a result, it includes the local displacements at the interfaces due to the settlement. Strain gauge 2 used the measurement points which are far from the loading platens in order to avoid their confining effect. Strain gauge 3 excludes the local displacements that occur at the interfaces between the parts. This strain gauge was only used for the compression test in X direction.

The compressive stress-strain curves of the specimens are illustrated in Fig. 3.25. As seen in Fig. 3.25(a) and (b), when strain gauge 1 is used, an upward concavity is observed at the beginning of the test. This is due to the settlement of the loading steel plates. When strain gauge 2 is used, as seen in Fig. 3.25(d), these local deformations are excluded in Y direction. In X direction, however, the concavity still resulted which was due to the settlement of brick parts. The stress-strain curve corresponding to strain gauge 3 is shown in Fig. 3.25(e). It is seen in the diagram that the settlement was excluded using this strain gauge; however, the accuracy of the measurement is low because of the smaller length of the strain gauge. Moreover, the out-of-plane deformation at higher load levels resulted in a measurement which is influenced by the deformations in the other directions.

It can be concluded that at lower load levels the measurement points on the specimen should be used in order to exclude the local displacements at the interfaces. By increasing the load, an out-of-plane deformation occurs in the specimens. It decreases the validity of the axial strain measurement, see CompY2 in Fig. 3.25(d). Therefore, the measurement points on the loading steel plates should be used. At high load level, the settlement is also limited.

It can be seen in Fig. 3.25 that there is a considerable scatter in the compressive strength and modulus of elasticity of the brick in both directions. The mean value of the secant slope of the stress-strain curves between stresses of 5 and 10 MPa is 8800 MPa ± 42% for the X direction and 11600 MPa ± 32% for the Y direction. The average compressive strength of the brick is 18MPa ± 27% and 23MPa ± 56% in the X and Y directions, respectively.
An abrupt failure occurred in Y direction. This can be clearly seen in Fig. 3.25. In contrast to the X direction, no post-peak behavior could be captured in the Y direction. The behavior in Y direction can be characterized as elastic up to failure.

Fig. 3.25 Compressive stress-strain behavior of the brick in X direction using strain gauges (a) 1, (c) 2, and (e) 3 and in Y direction using strain gauges (b) 1 and (d) 2.
Fig. 3.26 shows the failure state of Specimen CompY3 after being unloaded. In the X direction the failure was not as abrupt as in the Y direction. The specimen kept bearing axial loads after reaching its maximum strength and the length of the cracks increased gradually until spalling of bricks occurred. The tensile principal strain of the specimens at their maximum strength state is compared with those at 20% degraded state in Fig. 3.27. Apart from the third specimen that showed a premature vertical crack, only small cracks were visible at the compressive strength of the specimens. Afterwards, the crack width gradually increased until failure.

As seen in the experimental results, a high scatter is observed in the modulus of elasticity and compressive strength of the clay brick samples. The compressive strength of bricks in both load carrying directions is in the same order of magnitude. The value of 10000 MPa is considered as an approximate value of the modulus of elasticity of the clay bricks.
3.2.3 Lateral tensile compression test

In the supplementary specifications of Switzerland’s standard for masonry (SIA 266/1 2003) an experiment is suggested for finding the lateral tensile strength of bricks using a compression test. In this experiment, specific rubber plates are placed between stack bricks simulating the lateral strain generated by mortar layers in a real masonry assemblage. By applying the compression to the specimen perpendicular to the rubber plates, they deform significantly in the lateral direction that finally results in tensile cracking in the bricks. This experiment was conducted in order to observe the behavior of bricks under lateral tension using 3D ICS.

Three specimens were prepared for the experiment in a similar way as the one described earlier for the compression test on brick samples in X direction. For each specimen, three cubic parts with a side length of 40mm were extracted from a brick. They were placed on top of each other. Between each pair of brick parts, the specific rubber plates were placed. Based on the recommendations of the standard, 3mm-thick rubber plates with a density of 1480kg/m³, hardness (shore A) of 70, a tensile strength of 5MPa and a tensile failure strain of 300% were chosen. The front surface of assemblage was painted to provide the stochastic pattern required for the optical measurement, see Fig. 3.28(a). The measurement points and virtual strain gauges are shown in Fig. 3.28(b). The test was conducted under displacement control with a rate of 0.005mm/s. At arbitrary levels of displacement, a cycle of unloading and reloading was followed.

The compressive stress-strain behavior of the lateral tensile specimens using strain gauges 1 and 2 is illustrated in Fig. 3.29. Strain gauge 2 measures the axial strain in the middle brick. Because of the high elastic modulus of bricks, a very low axial strain was measured by this strain gauge. The mean value of the modulus of elasticity of the brick recorded by strain gauge 2 was equal to 10000 MPa ± 44% which is close to that obtained from the compression test on brick samples in the X direction.

In contrast to strain gauge 2, strain gauge 1 included the axial deformation of the rubber plates, see Fig. 3.29(a). At lower normal stresses, the deformation of the rubber plates was higher but by increasing the normal stress it decreased. Due to the incompressibility of the rubber plates, increasing the vertical compressive strain results in an increase in their lateral strain. The generated strain in the rubber plates...
applied lateral tensile stresses to the bricks that resulted in cracking in vertical direction. The loss of axial stress, seen in Fig. 3.29, occurs due to cracking in the bricks.

In the first and second tests, the vertical crack occurred in a plane parallel to the front surface. Therefore, it was not visible by the cameras. Because of the 3D measurement, however, the change in the out-of-plane displacement represented the load stage at which the crack initiated. In both samples, the brick part located in the middle was cracked. The stress-strain curves of the two tests are quite comparable. In contrast to the first two tests, in the third one, the crack occurred in a plane perpendicular to the front surface in the part located on top. In this test, a higher compressive stress could be resisted by the sample. This is more likely due to the nonuniformity of the brick.

Fig. 3.30 shows the contour of out-of-plane displacement of the three tests after the loss in bearing the axial load. The out-of-plane displacement of the third sample hardly changed. The high amount of out-of-plane displacement of the rubber plates is noticeable in all three tests.

Fig. 3.29 Compressive stress-strain behavior of the lateral tensile specimens using (a) strain gauge 1 and (b) strain gauge 2.

Fig. 3.30 Contour of out-of-plane displacement of (a)-(c) first-third samples.
The mean value of the compressive strength corresponding to the lateral tensile failure was equal to 8.0 MPa ± 37%. This value is less than half of the compressive strength of the brick in X direction. In a masonry assemblage, even though the elastic modulus of mortar is expected to be higher than the rubber plates, this failure mode is highly expected rather than failure in compression. This phenomenon will be emphasized more in the masonry tests.

3.3 Masonry

As a composite material, the global behavior of masonry depends on the behavior of its constituents as well as the interaction between them. The compressive behavior of the mortar and clay bricks, as the constituents of the masonry, was studied in the previous sections. This section explains the experimental sturdy on the global behavior of the masonry. This is done by conducting compression and shear tests on masonry wallets.

3.3.1 Masonry centric compression tests

The specimens were prepared for measuring the compressive strength of masonry perpendicular and parallel to the bed joints according to SIA 266. These directions are denoted as X and Y directions, respectively. The specimens for testing in each direction are shown in Fig. 3.31. They were constructed in a single wythe using solid clay bricks. A weak mortar was designed in order to represent the existing masonry buildings. The mortar thickness was considered as 12 mm. More information about the characteristics of the brick and mortar was given in sections 3.1 and 3.2. The specimens were kept in a climate chamber with a temperature of 20 °C and a humidity of 50% and tested at the age of 28 days.

![Fig. 3.31 Geometry of the test specimens for compressive strength tests (a) perpendicular to bed joints (X direction) and (b) parallel to bed joints (Y direction).](image)

On one side of the specimens, the vertical displacements were measured using displacement transducers as shown in Fig. 3.31. They measured the relative displacement of two points in vertical direction. The points were spaced at 220 mm and 350 mm for the compression test in X and Y directions, respectively. On the
other side, 3D ICS was installed to measure the full-field displacement of the specimens. A speckle pattern was painted on the masonry specimens to measure the full-field displacement. The arrangement of the 3D ICS is shown in Fig. 3.32 for the test in X direction. A 5 MN compression machine was used to apply the axial compressive force.

![Configuration of 3D ICS for compression test on masonry prims in X direction.](image)

### 3.3.1.1 Perpendicular to the bed joints (X direction)

In X direction, the specimens were tested under three types of axial load protocol. In the first test, the load was increased monotonically up to failure. In the second one, a cyclic load protocol was applied in order to capture the stiffness of the specimen during unloading and reloading. Similarly, the last type of load protocol was cyclic but with the difference of repeating each cycle three times.

The tests were carried out under load control. The load rate for the monotonic tests was as low as 0.25 kN/s. In the cyclic tests, the load rate was chosen as 1 kN/s in order to shorten the duration of the test. Since the compression test was load-controlled, in order to avoid a sudden failure, the test was stopped just before the peak strength was reached.

Table 3.6 shows the experimental results in terms of compressive strength and modulus of elasticity of the samples. The modulus of elasticity was calculated from stress-strain curves by measuring the slope of a secant between ordinates corresponding to 50 kN, equal to 0.8 MPa, and 1/3 of the ultimate strength of the specimens. The final state was considered as the state in which the tangent slope of the stress-strain curves was relatively low, say 300 MPa. This state was experienced by all samples just before failure.

Apart from the very first experiment, MasCMonX1, the compressive strength of the monotonic tests did not show a significant scatter. However, the compressive strength of the specimens under cyclic loading with one cycle per amplitude was marginally higher than that under monotonic load. When loaded cyclically with three cycles per amplitude, Specimen MasCCyc3X1 showed an unexpectedly high compressive strength. The test was repeated using Specimen MasCCyc3X2. The resulting compressive strength of the specimen was still higher compared to the monotonic tests.

The strain corresponding to maximum compressive strength did not show a noticeable difference between monotonic and cyclic tests. Similarly, the modulus of elasticity of the masonry specimens indicated no trend under monotonic and cyclic axial loading.
Table 3.6 Compressive strength and stiffness of the masonry samples tested in X direction.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Mortar</th>
<th>Compressive strength [MPa]</th>
<th>Corresponding axial strain [%]</th>
<th>Modulus of elasticity [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>MasCMonX1</td>
<td>First study</td>
<td>5.8</td>
<td>0.38</td>
<td>4.8</td>
</tr>
<tr>
<td>MasCMonX2</td>
<td>First study</td>
<td>7.9</td>
<td>0.50</td>
<td>5.0</td>
</tr>
<tr>
<td>MasCMonX3</td>
<td>First study</td>
<td>8.1</td>
<td>0.59</td>
<td>6.0</td>
</tr>
<tr>
<td>MasCMonX4</td>
<td>First study</td>
<td>7.5</td>
<td>0.40</td>
<td>4.4</td>
</tr>
<tr>
<td>MasCMonX5</td>
<td>Second study</td>
<td>7.5</td>
<td>0.43</td>
<td>3.2</td>
</tr>
<tr>
<td>MasCCyc1X</td>
<td>Second study</td>
<td>8.4</td>
<td>0.41</td>
<td>4.0</td>
</tr>
<tr>
<td>MasCCyc3X1</td>
<td>Second study</td>
<td>12.3</td>
<td>0.59</td>
<td>5.1</td>
</tr>
<tr>
<td>MasCCyc3X2</td>
<td>Second study</td>
<td>10.4</td>
<td>0.50</td>
<td>5.7</td>
</tr>
<tr>
<td>mean</td>
<td></td>
<td>8.5</td>
<td>0.48</td>
<td>4.8</td>
</tr>
<tr>
<td>Coefficient of variation [%]</td>
<td></td>
<td>24</td>
<td>17</td>
<td>19</td>
</tr>
</tbody>
</table>

It is notable that even though the mean value of compressive strength of the mortar used for the first study was quite different from that used for the second one, refer to Fig. 3.2, the compressive strength of masonry was quite alike. The reason is that the failure in masonry occurs due to the lateral tension in the clay bricks resulting from the expansion of the mortar. Therefore, it is the volumetric deformation of the confined mortar that determines the failure in masonry.

Compressive stress-strain curves of the specimens are shown in Fig. 3.33. A nonlinear inelastic behavior
was observed in all specimens. As stated earlier, the modulus of elasticity of the specimens had a clear variation. Beyond the elastic region, the slope of the stress-strain curve of the samples decreased gradually up to failure. Higher values of the slope were measured when unloading and reloading in cyclic tests. During unloading, the slope decreased significantly when the axial compressive stress approached zero. Therefore, an upward concave stress-strain curve resulted during the unloading and reloading at low stress levels. At high levels of plastic strain, this behavior was more significant. A similar behavior was reported by Naraine and Sinha (1989) for brick masonry.

The stress-strain behavior of the masonry specimens can be explained using the results of image correlation measurement at different stages of loading. Fig. 3.34 shows the out-of-plane displacement as well as the strains in horizontal direction ($\varepsilon_{yy}$) and vertical direction ($\varepsilon_{xx}$). It can be seen in the figure that up to 33% of the compressive strength of masonry almost no out-of-plane displacement occurred. The horizontal strain shows that no tensile crack formed. In the vertical direction, the strain was maximum at the bed joints.

At 75% of the compressive strength, it can be seen that the mortar started to expand and as a result, an out-of-plane deformation as well as tensile cracks occurred in the bricks. The stress-strain behavior of the masonry became nonlinear.

By increasing the axial compressive stress to the maximum compressive strength of masonry, the density of cracking and the out-of-plane deformation increased. This is due to the high deformability in the mortar.

After degradation of the compressive strength, the expansion in mortar increased significantly. At this level, the cracks grew and the out-of-plane deformation in the bricks became larger until failure occurred in the sample.

It should be noted that the positive strain regions, shown in red in Fig. 3.34(c), are due to out-of-plane deformation in the front surface. When out-of-plane deformation occurs at some region, the plane becomes curve shaped. Therefore, the distance between points in the curve increases. The increase in the relative displacement results in positive strains in vertical direction even though the global strain is negative in compression.

It is emphasized here that the maximum compressive strains were at both interfaces between mortar layers and bricks. These locations can be observed clearly in Fig. 3.34(c) where parallel blue lines represent higher axial deformations at both interfaces. Similar to the mortar experiments, local deformations occur at the interface due to unevenness of the surfaces. These deformations decrease the axial stiffness of the specimens.

During the cyclic tests, the highest slope was reached when unloading started; conversely, when the axial compressive load approached zero, the slope was the lowest, see Fig. 3.33. This is attributable to the fact that under high normal stresses, the friction between the cracked surfaces that slide on each other increases. Therefore, the sliding deformations are prevented. Under low normal stress, sliding of the rough surfaces occurs and a high amount of axial strain is obtained.
Axial force/ strength ratio = 0.33

Axial force/ strength ratio = 0.75

Axial force/ strength ratio = 0.75

Axial force/ strength ratio = 1.0

Axial force/ strength ratio = 0.90 (post-peak)

Fig. 3.34 Contour of (a) out-of-plane displacement, (b) axial strain parallel to bed-joints ($\varepsilon_{yy}$), and (c) axial strain perpendicular to bed joints ($\varepsilon_{xx}$) in the masonry specimen under monotonic compressive axial test at various stages of loading.
3.3.1.2 Parallel to the bed joints (Y direction)

According to (SIA 266/1 2003) compressive strength tests parallel to the mortar bed joints were carried out for Specimens Y1 and Y2 under force-controlled. For Specimen MasCMonY3, however, a deformation-controlled procedure was chosen. Table 3.7 summarizes the test results.

Similar to the tests in the X direction, the moduli of elasticity were calculated using the secant slope between 50 kN and 1/3 of the maximum force. The mean value of the strains measured by LVDTs was used as the vertical strain. For the first specimen Y1, a lateral pressure equal to 10% of the compressive strength in the X direction ($f_{xm}$), was initially applied to the specimen according to (SIA 266/1 2003)(SIA266/1 2003). The lateral pressure was applied using two steel plates and four prestressing rods as shown in Fig. 3.35. This led to a very high compressive strength in Y direction which was in fact more than twice of $f_{xm}$. This is due the fact that by applying the vertical load in Y direction, the confining pressure that provides higher compressive strength increases accordingly.

Specimen MasCMonY2 was tested without lateral pressure. The obtained compressive strength parallel to the bed joints $f_{ym}$ was in the order of the compressive strength in the direction perpendicular to them. However, when looking at the failure mechanism of MasCMonY2 (Fig. 3.36), it can be seen that failure occurred due to the buckling of the outer layers. This instability led to a reduced measured compressive strength parallel to the mortar bed $f_{ym}$.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Mortar</th>
<th>Lateral pressure [kPa]</th>
<th>Compressive strength [MPa]</th>
<th>Corresponding axial strain [%]</th>
<th>Modulus of elasticity [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>MasCMonY1</td>
<td>First study</td>
<td>0.6 initially</td>
<td>17.8</td>
<td>0.39</td>
<td>7.3</td>
</tr>
<tr>
<td>MasCMonY2</td>
<td>First study</td>
<td>-</td>
<td>7.9</td>
<td>0.30</td>
<td>7.1</td>
</tr>
<tr>
<td>MasCMonY3</td>
<td>First study</td>
<td>0.6 constant</td>
<td>17.3</td>
<td>0.44</td>
<td>10.0</td>
</tr>
</tbody>
</table>

Fig. 3.35 Masonry specimen Y1 under compression with an initial lateral pressure.
It was therefore decided to carry out the test on specimen MasCMonY3 with a controlled constant lateral pressure of 0.6 MPa, similar to the initial lateral stress in MasCMonY1, in order to inhibit the buckling of the outer layers. For this, four small jacks were put in series with four prestressing rods, which held the forces constant during the test, see Fig. 3.37(a). This relatively small lateral pressure was sufficient to stabilize the specimen to such an extent that the measured strength was again more than twice the compressive strength perpendicular to the bed joints \(f_{xm}\). It can be seen in Fig. 3.37(b) that the small lateral pressure prevented the buckling failure mode and therefore, increased the compressive strength significantly. The axial stress-strain behavior of all specimens is shown in Fig. 3.38.

Fig. 3.37 Masonry specimen Y3 (a) before failure and (b) after failure.
It can be concluded that the compressive strength of masonry wallets made of clay bricks and weak mortar is higher in Y direction. This can be attributed to the fact that the expansion of mortars is smaller when the specimen is loaded in Y direction. Thus, the tensile stress produced in the clay bricks is less than its corresponding value in the specimen, which is loaded in X direction (perpendicular to the bed joints).

### 3.3.2 Masonry eccentric compression tests

It was seen in the centric compression tests that the mortar becomes confined by the adjacent bricks. This confinement increases in proportion to the axial compressive force. The expansion of mortar results in bilateral tension in the bricks that leads to lateral cracking in them and eventually to the failure of masonry. As opposed to the axial compressive load, when in-plane shear force is applied to a masonry wall, tensile stress might be produced in some region of the cross section. This is expected when in-plane horizontal forces are introduced to URM walls as a result of an earthquake. Under this type of loading, tensile cracks are expected to occur in the masonry at the bed joints because of the low tensile strength of masonry. Due to tensile cracking at the bed joints, the distribution of confinement in mortar layers changes. The mortar is confined only where it is in contact with bricks. To study this phenomenon a series of eccentric compression tests in the direction perpendicular to the bed joints was designed in a way to generate tensile stress in masonry samples using a compression machine.

The masonry specimens were produced similar to the specimens used for centric compression tests to be tested in the direction perpendicular to the bed joints. The test-set up is shown in Fig. 3.39.

The masonry specimen was placed between two steel beams which were connected to the compression platens of the machine using a hinge system. A pair of steel plates was used to stiffen the steel beams where the concentrated load was introduced to the beam. An eccentricity of 150 mm was provided for all samples. The eccentric tests were carried out under displacement control. Similar to the centric tests, three different protocols were considered for axial force. They were monotonic, cyclic with one cycle per amplitude and cyclic with three cycles per amplitude. For the cyclic tests four cycles were considered at various displacements. The vertical force and machine displacement were measured by the compression machine. Additionally, four LVDTs were installed on the bed joint located at the mid-height of the sample in order to measure the joint opening, see Fig. 3.39.
Fig. 3.39 Eccentric masonry test set-up (a) front view (b) side view.

Full-field displacement measurements were made on the other side of the specimens using 3D ICS. Fig. 3.40 shows the configuration of the system. The speckle pattern required for the optical measurement is illustrated more clearly in the photo taken by the right camera, see Fig. 3.41(a). In order to have the load-displacement diagram, the vertical displacement of the sample was measured by the ICS. For that purpose, the relative vertical displacement of the first and last row of bricks along the direction of the applied concentrated load was considered. The strain gauge can be seen in Fig. 3.41(b).

Fig. 3.40 Configuration of 3D ICS for eccentric compression test on masonry specimens.
Fig. 3.41 (a) Speckle pattern on the visible surface of a masonry specimen and (b) Measured surface and the points used for calculating the relative vertical displacement.

The cyclic test with one cycle per amplitude was repeated due to unexpectedly higher strength than the other two samples. The axial force-displacement diagram of eccentric tests is compared with that of centric tests in Fig. 3.42. For the centric tests, the vertical displacement was measured similarly between

Fig. 3.42 Axial force versus vertical displacement of eccentric and centric masonry tests
the first and last rows along the direction of the applied force. The test results are shown in Table 3.8 in more details.

**Table 3.8 Axial compression test results of centric and eccentric masonry specimens perpendicular to the bed joint**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load capacity [kN]</th>
<th>Corresponding vertical displacement [mm]</th>
<th>Initial stiffness [kN/mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>MasCMonX5</td>
<td>466</td>
<td>2.3</td>
<td>447</td>
</tr>
<tr>
<td>MasCCyc1X</td>
<td>525</td>
<td>2.2</td>
<td>476</td>
</tr>
<tr>
<td>MasCCyc3X1</td>
<td>767</td>
<td>2.6</td>
<td>610</td>
</tr>
<tr>
<td>MasCCyc3X2</td>
<td>647</td>
<td>2.8</td>
<td>589</td>
</tr>
<tr>
<td>mean</td>
<td>601</td>
<td>2.5</td>
<td>530</td>
</tr>
<tr>
<td>Coefficient of variation</td>
<td>22%</td>
<td>12%</td>
<td>15%</td>
</tr>
<tr>
<td>MasEMonX1</td>
<td>177</td>
<td>2.3</td>
<td>235</td>
</tr>
<tr>
<td>MasECyc1X1</td>
<td>298</td>
<td>2.3</td>
<td>390</td>
</tr>
<tr>
<td>MasECyc1X2</td>
<td>236</td>
<td>2.3</td>
<td>121</td>
</tr>
<tr>
<td>MasECyc3X1</td>
<td>192</td>
<td>2.7</td>
<td>178</td>
</tr>
<tr>
<td>mean</td>
<td>226</td>
<td>2.4</td>
<td>231</td>
</tr>
<tr>
<td>Coefficient of variation</td>
<td>24%</td>
<td>7%</td>
<td>50%</td>
</tr>
</tbody>
</table>

A nonlinear behavior was observed in all tests. As expected, a lower load capacity was reached for the eccentric tests than the centric tests due to the reduction of the compression zone. The stiffness was also decreased in the eccentric masonry tests. Similar to the centric tests, the stiffness is highest during unloading and reloading.

The vertical displacement corresponding to the load capacity was quite constant in the specimens. This emphasizes the fact that failure in masonry occurs due to vertical contraction of masonry at bed joints and expansion in the lateral direction.

To have a closer look at the phenomenon, the vertical and horizontal strains of the entire surface as well as its out-of-plane displacement are illustrated in Fig. 3.43. As seen in the figure, at an axial force/strength ratio of 0.33, a horizontal crack formed. Tensile deformation was concentrated about this crack until the end of experiment. Since this crack did not occur at the bed joint where the LVDTs were installed, the measurement by the LVDTs was not representing the main joint opening.

Vertical compressive deformation as well as out-of-plane deformation gradually increased at the compression region of the cross section that resulted in vertical in-plane and out-of-plane cracks in the masonry samples. In the post-peak region, the density of cracking increased significantly and spalling of bricks occurred.
3.3.3 Triplet shear test

In order to study the frictional behavior at the bed joints of the masonry, a series of triplet shear tests was carried out according to (EN 1052-3 2002). Stack bonded masonry specimens consisted of three bricks and two mortar layers between them. The geometry of the specimens is shown in Fig. 3.44(a). The test should be done under various pre-compression levels in order to find the correlation between the normal and shear stresses at bed joints. The lateral compression was generated using prestressing bars. It was measured by the load cell shown in Fig. 3.44(b). The lateral compression was introduced to the specimens
using steel and timber plates. The flexible timber plates were used in order to have a better distribution of the lateral stress. According to the standard, the eccentricity was kept less than 1/15 of the unit length. The vertical displacement was applied to the middle brick using a 200 kN compression machine. The adjacent bricks were supported by steel plates which were placed on rollers as seen in Fig. 3.44. The loading and supporting surfaces were treated using gypsum in order to provide a smooth surface. The rate of displacement was kept as small as 0.005 mm/s. Deformation of the entire front surface was measured using 3D ICS. The speckle pattern required for the measurement is shown in Fig. 3.44(b). On the back side of the specimens, the vertical displacement of each brick was measured using a vertically installed LVDT, see Fig. 3.44(c). The test set up is shown in Fig. 3.45 in more details.

Fig. 3.44 Triplet shear test: (a) geometry, (b) front surface measurement, and (c) back surface measurement.

Fig. 3.45 Triplet shear test set-up.
Fig. 3.46 illustrates the experimental results in terms of shear force versus shear displacement for various pre-compression levels. Shear displacement was calculated by subtracting the average vertical displacement of adjacent bricks from the middle one measured by LVDTs. In all cases, two drops in shear force were observed which were due to occurrence of sliding in the two bed joints. After the formation of cracks in both bed joints, a pure sliding occurred between the middle brick and the adjacent ones. Fig. 3.47 shows the full-field displacement at different steps of cracking.

![Fig. 3.46 Experimental results of triplet shear test in terms of shear force versus shear displacement.](image)

Fig. 3.46 Experimental results of triplet shear test in terms of shear force versus shear displacement.

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

![Fig. 3.47 Contour of vertical displacement in triplet shear test with 1 MPa pre-compression: (a) before and (b) after cracking in the left bed joint, (c) after cracking in the right bed joint, and (d) at relative sliding of 2 mm.](image)

Fig. 3.47 Contour of vertical displacement in triplet shear test with 1 MPa pre-compression: (a) before and (b) after cracking in the left bed joint, (c) after cracking in the right bed joint, and (d) at relative sliding of 2 mm.

Fig. 3.48 shows the relation between the shear strength and the normal stress for all tests, as well as a linear regression. Based on the Coulomb’s friction law, the cohesion and the friction coefficient are equal to 0.62 MPa and 0.63, respectively. The correlation coefficient $r^2$ of the linear regression is 0.99, which indicates an excellent correlation. Increasing the pre-compression level resulted in an increase in the shear resistance. In a standard masonry, the value of friction coefficient seems to range between 0.7 and 1.2, according to different combinations of units and mortars (Van der Pluijm 1999). The obtained value is at the lower end of the range reported in the literature.
3.4 Conclusions

In order to investigate the behavior of existing masonry buildings, a series of tests was conducted on masonry constituents. The masonry used in this study consisted of clay bricks and weak cement-lime-sand mortar. The most important conclusions from the experimental study are as follows:

- The axial compressive behavior of mortar is pressure-dependent. Under a uniaxial state, the mortar shows a distinctive compressive strength. Afterwards, the compressive strength drops. However, in a confined state, a bilinear stress-strain behavior is observed and the mortar carries relatively high compressive stresses by increasing the confining stress.
- The modulus of elasticity of mortar is higher than the secant slope of the linear part of the stress-strain curve in a monotonic uniaxial test. The secant slope at certain stresses of the unloading/reloading branch in a cyclic test is more representative of the elastic behavior. This is due to the unrecoverable deformations the mortar experiences under higher stresses. These deformations are attributed to the collapse of pores of the mortar as well as the initiation and propagation of micro-cracks.
- A high scatter occurs in the uniaxial compressive strength of the clay bricks in both load carrying directions. Nevertheless, the compressive strength of the two directions is in the same order of magnitude.
- The compressive strength of masonry is mostly affected by the lateral tensile strength of the bricks. This is because the failure in masonry under compression is due to the expansion of mortar, which generates bilateral tensile stresses in the bricks. The deformation of mortar and brick as well as the local deformation at the interface is well captured by 3D ICS.
- The deformation of a masonry assemblage in a uniaxial compression test is influenced by the stiffness of the interface between the bricks and the mortar. The contact stiffness is low at low uniaxial deformation due to the settlement of bricks and the mortar layer. The stiffness increases rapidly by increasing the deformation.
- The compressive strength of masonry wallets in the direction perpendicular to the bed joints (X direction) is lower than that in the direction parallel to the bed joints (Y direction), provided that a buckling failure mode is prevented in the compression tests in Y direction. The reason is that the expansion of mortar is greater when a masonry wallet is loaded in X direction.
• In an eccentric compression test on masonry wallets, only the compression zone of the section is confined. Under a constant bending moment, crack opening is not distributed among all bed joints; it is much higher in one of the bed joints. A lower load capacity is observed in the masonry under eccentric loading.

• The frictional behavior of the contact between the mortar and bricks has a linear relationship with the normal stress. In this study, the cohesion and the friction coefficient were found to be 0.62 MPa and 0.63, respectively.

3.5 References


SIA 113-43 (1943). "Provisorische Normen für die Berechnung und Ausführung von Mauerwerk aus natürlichen und künstlichen Bausteinen." Swiss Society of Engineers and Architects, Zurich, Switzerland.


4 Wall tests

4.1 Introduction

In order to understand the influence of post-tensioning on the in-plane behavior of unreinforced masonry walls, six large scale wall tests were conducted in the framework of two studies. The first study was a feasibility study in which two geometries were considered. As the failure mode of URM walls is highly influenced by the aspect ratio, one wall type was chosen to be fairly squat with an aspect ratio of less than unity and the other one was fairly slender with an aspect ratio of more than unity. The squat and slender walls were expected to undergo shear and flexural failure modes, respectively. The squat and slender walls were tested with and without post-tensioning. In total, four tests were carried out in the first study. In the second study two large scale walls were tested with and without post-tensioning to assess the effect of normal stress on unreinforced and post-tensioned masonry walls. The construction and testing details as well as the test results are described in this chapter.

4.2 Construction details

4.2.1 Wall specifications

Wall dimensions are specified in Table 4.1. The geometry of the walls was chosen as large scale as possible. However, due to the height limit of the reaction wall at the time, the height was limited to 1.90 m. Even though in the second study the reaction wall was modified, in order to be able to compare the results with the previous study, the wall height was kept constant. The geometries of the two wall types including the loading beam and the foundation are shown in Fig. 4.1 and Fig. 4.2, respectively.

<table>
<thead>
<tr>
<th>Wall</th>
<th>length</th>
<th>height</th>
<th>thickness</th>
<th>aspect ratio</th>
<th>vertical force</th>
<th>dead load</th>
<th>initial post-tensioning force</th>
<th>initial normal stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$l_w$ [m]</td>
<td>$h_w$ [m]</td>
<td>$t_w$ [m]</td>
<td>$h_w/l_w$</td>
<td>[kN]</td>
<td>[kN]</td>
<td>$P$ [kN]</td>
<td>$\sigma_n$ [MPa]</td>
</tr>
<tr>
<td>Wall 1</td>
<td>2.85</td>
<td>1.90</td>
<td>0.12</td>
<td>0.67</td>
<td>114</td>
<td>20.2</td>
<td>0</td>
<td>0.59</td>
</tr>
<tr>
<td>Wall 2</td>
<td>2.85</td>
<td>1.90</td>
<td>0.12</td>
<td>0.67</td>
<td>114</td>
<td>20.2</td>
<td>8×20=160</td>
<td>1.29</td>
</tr>
<tr>
<td>Wall 3</td>
<td>1.58</td>
<td>1.90</td>
<td>0.12</td>
<td>0.67</td>
<td>114</td>
<td>20.2</td>
<td>0</td>
<td>0.32</td>
</tr>
<tr>
<td>Wall 4</td>
<td>1.58</td>
<td>1.90</td>
<td>0.12</td>
<td>0.67</td>
<td>114</td>
<td>20.2</td>
<td>8×21=84</td>
<td>0.69</td>
</tr>
<tr>
<td>Wall 5</td>
<td>1.58</td>
<td>1.90</td>
<td>0.12</td>
<td>0.67</td>
<td>114</td>
<td>20.2</td>
<td>0</td>
<td>0.71</td>
</tr>
<tr>
<td>Wall 6</td>
<td>1.58</td>
<td>1.90</td>
<td>0.12</td>
<td>0.67</td>
<td>114</td>
<td>20.2</td>
<td>4×37.5=150</td>
<td>1.36</td>
</tr>
</tbody>
</table>
Fig. 4.1 Geometry and section of Wall 2.

Fig. 4.2 Geometry and section of Wall 4.
4.2.2 Foundation slab and loading beam

For each wall type separate foundation slabs and loading beams were produced. The foundation slabs and the loading beams were designed to accommodate the anchor forces of the prestressing bars. The small foundation slab constructed for the slender walls is shown in Fig. 4.3. In the second study, the small foundation was modified with a step in order for the bolts not to block the view of the optical measurement system (Fig. 4.4c).

![Diagram of foundation slab and loading beam](image)

Fig. 4.3 Longitudinal section of the small foundation for slender walls.

4.2.3 Wall construction

The walls were directly constructed on the foundation slab, which was placed at the final position onto the strong floor. In this way, no transport of the walls, which could lead to premature damages, was required. Fig. 4.4 shows the construction of Wall 1 and Wall 3 (Table 4.1). The masoning of the walls took two days. At the uppermost bed joint, between the wall and the loading beam, in addition to a high strength cement mortar, a special adhesive (SikaDur 30) was used to prevent the sliding of the loading beam.

4.3 Testing details

4.3.1 Test setup

The test setup is shown in Fig. 4.5. The principal components are pointed out in the figure. Two 200-kN hydraulic jacks were connected vertically to the main frame. The jacks produce a vertical force which is representative of the gravitational loads of the upper stories in a real building. The vertical force is introduced to the concrete beam using a steel beam which is supported by three rollers. The rollers reduce the friction between the steel beam and the concrete loading beam. This prevents the vertical jacks from carrying any horizontal load. The horizontal load was applied to the wall using a 250-kN servo-hydraulic jack, which was connected to the reaction wall. The reaction wall, which was modified for the second study, is shown in Fig. 4.6. Finally, a restraining system was used in order to prevent any out-of-plane displacement of the loading beam.
Fig. 4.4 Construction of (a) Wall 1, (b) Wall 3, and (c) Wall 5.

Fig. 4.5 Test setup in the first study.
4.3.2 Measuring tools

The compression forces of the vertical jacks as well as the shear force introduced by the horizontal jack were measured by their load cells. Four load cells where used for measuring the post-tensioning force in the prestressing bars.

Two methods were used to measure the displacement. On one side of the walls, common Linear Variable Displacement Transducers (LVDTs) were used at certain points to measure the desired displacements. On the other side, the novel 3D Image Correlation System (ICS) was used to measure the full-field displacement of the walls. Both methods are explained in more detail in the next section.

4.3.2.1 Linear Variable Displacement Transducers (LVDTs)

Fourteen LVDTs were installed on each wall (Table 4.2). The arrangement of transducers for squat and slender walls is shown in Fig. 4.7 and Fig. 4.8, respectively. In the second study the transducers V5 and V6 were modified to measure the vertical displacement of the total height of the wall as opposed to the first study, in which the mentioned transducers measured the vertical displacement of the wall between the 6th brick row and the top of the wall (Fig. 4.7).

Table 4.2 Functionality of displacement transducers

<table>
<thead>
<tr>
<th>Name</th>
<th>Sensor type</th>
<th>Function</th>
</tr>
</thead>
<tbody>
<tr>
<td>H1, H2</td>
<td>Silvac 25 mm</td>
<td>measuring the slip between the wall and the loading beam and between the wall and the foundation slab, respectively.</td>
</tr>
<tr>
<td>V1, V4</td>
<td>Silvac 50 mm</td>
<td>measuring the uplift of the wall</td>
</tr>
<tr>
<td>V2, V3</td>
<td>inductive HBM WSF/ 50 mm</td>
<td>measuring the vertical displacement of the entire wall</td>
</tr>
<tr>
<td>V5, V6</td>
<td>inductive HBM W 50</td>
<td>measuring the horizontal displacement of the wall</td>
</tr>
<tr>
<td>H3</td>
<td>inductive HBM WSF/200</td>
<td></td>
</tr>
<tr>
<td>H4</td>
<td>inductive HBM W 100</td>
<td></td>
</tr>
<tr>
<td>H5</td>
<td>inductive HBM WSF/100</td>
<td></td>
</tr>
<tr>
<td>H6</td>
<td>inductive HBM WF/50</td>
<td></td>
</tr>
<tr>
<td>D1, D2</td>
<td>cable transducer</td>
<td>measuring the shear deformation of the wall</td>
</tr>
</tbody>
</table>
4.3.2.2 Image Correlation System (ICS)

3D ICS was used to measure the full-field displacement of the masonry walls. The arrangement of the measurement equipment is shown in Fig. 4.9.

For the squat walls, the length of measuring window of 4 m is mapped into 2000 pixels. This corresponds to 2 mm/pixel and results in a resolution of 0.2 to 0.02 mm. For the slender walls of the first study, a smaller measuring window was chosen, covering only the lower half of the wall and resulting in a resolution of 0.1 to 0.01 mm. For the other two slender walls, a measuring surface of 1750 mm×1750 mm was considered.

Prior to the measurement, the system must be calibrated using calibration objects to ensure the dimensional consistency of the measurement. The calibration cross, shown in Fig. 4.9, was used for that purpose. The size of the calibration cross was selected in a way to cover the whole measuring surface.

Lamps were used in order to increase the brightness of the photos. This increases the visibility of speckles and also results in a shorter shutter time required for taking photos.

4.3.3 Testing procedure

Before running the tests, all LVDTs were set to zero and the 3D ICS was calibrated. Then, the vertical loads were imposed by the vertical jacks. The total vertical load was kept constant over the duration of the test without imposing a bending moment. In Walls 2, 4, and 6, after the application of the vertical loads, the post-tensioning was applied by tightening the nuts used for anchoring the threaded prestressing bars on
top of the concrete beam. During this process, the post-tensioning forces were recorded by the load cells installed on them.

![Diagram of Wall 4](image1)

**Fig. 4.8 Instrumentation of Wall 4.**

![Diagram of Image Correlation System](image2)

**Fig. 4.9 Arrangement of Image Correlation System for Wall 5.**
The horizontal displacement protocol was then applied to the walls under a displacement control. At each load step, a specific displacement was reached by the horizontal jack. Three displacement cycles were completed for each load step. The displacement was increased in the next load steps. In every maximum and minimum displacement, the horizontal jack was stopped in order to report the new cracks and to perform the optical measurements. The tests were continued until the walls experienced a severe failure.

### 4.4 Prediction of shear resistance

An estimation of the theoretical horizontal force resistance was made using the stress field method proposed by SIA 266 (2003). The method developed by Ganz (1985) is based on the lower bound theorem of the theory of plasticity. The failure criterion was originally developed for masonry made of hollow bricks where the compressive strength of masonry in the direction perpendicular to the bed joints \( f_{xm} \) is higher than that parallel to the bed joints \( f_{ym} \). This is due to the fact that in a hollow brick masonry both web and rib parts contribute in carrying the compression when the load is applied perpendicular to the bed joints. However, when loaded in the parallel direction, only the web elements contribute. The tensile strength of masonry in both directions was neglected. Two material parameters were also considered for the mortar bed joints that are the angle of internal friction \( \phi \), with \( \mu = \tan(\phi) \), and the cohesion \( c \). The failure criterion is therefore given by:

\[
\begin{align*}
I & : \tau_{xy}^2 - \sigma_x \cdot \sigma_y \leq 0 & \text{no tension in bricks} \\
II & : \tau_{xy}^2 - (\sigma_x + f_{xm}) \cdot (\sigma_y + f_{ym}) \leq 0 & \text{resistance of web and rib elements} \\
III & : \tau_{xy}^2 + \sigma_y \cdot (\sigma_y + f_{ym}) \leq 0 & \text{resistance of web elements} \\
IV(a) & : \tau_{xy}^2 - (c - \sigma_x \cdot \tan(\phi))^2 \leq 0 & \text{sliding of bed joints} \\
IV(b) & : \tau_{xy}^2 + \sigma_x \cdot \left[ \sigma_x + 2 \cdot c \cdot \tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \right] \leq 0 & \text{no tension in bed joints}
\end{align*}
\] (4.1)

The definition of two compressive strengths depending on the direction demonstrates the anisotropic behavior of masonry. For the present case, the mechanical properties are considered based on the material tests described in Chapter 3 therefore: \( f_{xm} = 8 \) MPa, \( f_{ym} = 18 \) MPa, \( \tan(\phi) = 0.62 \), and \( c = 0.63 \) MPa. As observed in our experiments, in solid clay brick masonry, the compressive strength in Y direction was higher than that in the X direction. Therefore, regime II in Eq. (4.1) changes accordingly. Additionally, regime III is not relevant in the case of solid bricks. Fig. 4.10 shows the failure criterion considered for our masonry. A comparison is also presented in the figure with hollow clay brick masonry with \( f_{ym} = 0.3 f_{xm} \).

An inclined stress field is assumed in a masonry wall as a strut to transfer the applied axial and shear forces through the wall. A uniaxial stress field is considered in the strut; see Fig. 4.11 (a). The uniaxial stress in the strut shall remain within the boundaries of the failure criterion. The allowable uniaxial stress as a function of the inclination angle \( \alpha \) is shown in Fig. 4.11(b) based on the failure criterion.
Fig. 4.10 Anisotropic masonry failure criterion for (a) our experiment with solid bricks ($f_{ym} = 2.25f_{xm}$) and (b) sample masonry with hollow brick masonry ($f_{ym} = 0.3f_{xm}$).

Fig. 4.11 (a) Masonry wall with applied forces and corresponding stress field and (b) failure criterion of masonry.

The equilibrium considerations give:

$$V = (N + P) \cdot \tan \alpha$$  \hspace{1cm} (4.2)
In which $\alpha$ is the inclination angle of the stress field. Based on the assumed stress field, the relationship between the inclination angle and the length of the stress field is:

$$\tan \alpha = \frac{l_w - l_v}{2h_w}$$

(4.3)

In order for the stress field to satisfy the failure criterion, the following condition applies:

$$\frac{N + P}{l_v \cdot t_w \cdot \cos^2 \alpha} \leq \sigma_2$$

(4.4)

A compression failure occurs in the stress field when this condition is violated. The calculated shear resistance and the failure mode of the walls are shown in Table 4.3.

Table 4.3 Shear resistance and failure mode of walls predicted by stress-field method.

<table>
<thead>
<tr>
<th>Wall</th>
<th>initial normal force [kN]</th>
<th>predicted shear resistance [kN]</th>
<th>predicted failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall 1</td>
<td>134.2</td>
<td>94.6</td>
<td>compression failure in the inclined stress field</td>
</tr>
<tr>
<td>Wall 2</td>
<td>294.2</td>
<td>192.4</td>
<td>compression failure in the inclined stress field</td>
</tr>
<tr>
<td>Wall 3</td>
<td>73.2</td>
<td>28.9</td>
<td>compression failure in the inclined stress field</td>
</tr>
<tr>
<td>Wall 4</td>
<td>157.2</td>
<td>58.2</td>
<td>compression failure in the inclined stress field</td>
</tr>
<tr>
<td>Wall 5</td>
<td>161.2</td>
<td>59.5</td>
<td>compression failure in the inclined stress field</td>
</tr>
<tr>
<td>Wall 6</td>
<td>311.2</td>
<td>101.6</td>
<td>compression failure in the inclined stress field</td>
</tr>
</tbody>
</table>

4.5 Test results

4.5.1 Overall behavior and damage pattern

Four distinguished states were observed in the in-plane behavior of all walls. These states were: first horizontal cracking, first vertical cracking, maximum shear resistance and degraded state. Damage patterns of the walls are emphasized at these states. The final crack pattern of the walls is shown in Fig. 4.12.

It must be noted that the drift used for the explanation was considered as the horizontal displacement measured by displacement transducer H4 divided by the height of the walls. The applied cyclic displacement was at a higher level at the mid-height of the beam; the target drift was defined based on the displacement measured at this level.

4.5.1.1 Wall 1

The first horizontal crack due to flexure was formed in the 4th bed joint in the push direction at a drift of 0.05%. This crack was followed by a second horizontal crack in the bed joint between the wall and the foundation in the pull direction at the same drift. With increasing the loading steps, the two cracks were joined and the wall started rocking and sliding about the full-length crack.
The first vertical crack occurred in the compression zone of the pull direction at a drift of 0.3%. By increasing the load steps, more cracks occurred in the compression zone in the pull direction. The first vertical crack in the push direction occurred later at a drift of 0.5%. At this load step, a large diagonal stepped crack opened. This changed the global behavior of the wall from rocking to sliding about the diagonal crack.

The maximum shear resistance of the wall was reached at the load step in which the first vertical crack occurred. The drift was 0.3% and 0.5% in the pull and push directions, respectively. When diagonal cracking occurred, the shear resistance was stable and did not decrease significantly. The shear resistance started decreasing when the bricks located at the compression zones crushed.

By increasing the load steps, a new horizontal crack occurred at higher bed joints. This crack joined the diagonal crack and the wall started sliding and rocking about the new full-length crack. Due to rocking, new vertical cracks occurred and the crushed compression zones fell apart, which led to the degradation in the shear resistance of the wall. The test was stopped when the wall lost its out-of-plane stability. By this time the shear resistance of the wall was reduced to 83%. Due to an error in the control of the horizontal jack, the final drift was unsymmetrical at 0.83% and 0.51% in the push and pull directions, respectively.

4.5.1.2 Wall 2
First horizontal cracks occurred at a drift of 0.07% in both pull and push directions (compared to the 0.03% drift in Wall 1). The crack started in the pull direction initiated in the 4th bed joint at the wall end in tension and then stepped down to the bottom bed joint between the wall and the foundation slab. In the push direction, the horizontal crack started in the 3rd bed joint at the other wall end and also stepped down to the bottom bed joint between wall and foundation slab. Increasing the drift to 0.10% resulted in joining the cracks and formation of a full-length crack, about which the wall started rocking and sliding. In comparison with Wall 1, sliding was more significant in this wall.

At the final cycle of a drift of 0.14%, vertical cracks were formed in the compression zones in both pull and push directions (less than half of the drift in the corresponding state in Wall 1). By increasing the load steps, the vertical cracks formed closer to the center of the wall. The vertical cracks at the corners joined together and resulted in crushing of the bricks. At a drift of 0.3%, a horizontal full length crack occurred at the 3rd bed joint as an extension of the existing cracks. The plane, about which rocking and sliding occurred, was changed from the first bed joint to this bed joint.

The wall exhibited its maximum shear resistance at 0.3% drift in both pull and push directions. The compression zones full of vertical cracks were pushed outwards and as a result the shear strength reduced during the next load steps. The horizontal displacement was mostly due to sliding about the newly formed full-length crack and the rocking was limited.

Increasing the target displacement, the compression zones extended toward the center of the wall and the damage increased such that the bricks were partly destroyed. Subsequently, new horizontal flexural cracks formed at higher elevations and joined together to develop the full crack in the 9th bed joint. At a drift of 0.6% and 0.7%, respectively, in the pull and push directions, during the first cycle the shear resistance of the wall degraded to 80% in the pull direction and 65% in the push direction. In the third cycle of the same load step, the degradation was more pronounced. It was 63% and 43% in the pull and push directions, respectively. The test was stopped at this degradation level.
4.5.1.3 Wall 3

The first horizontal crack occurred at a drift of 0.03% in both pull and push directions. In the push direction, the crack was formed at the 3rd bed joint from the wall base but in the pull direction the crack started unexpectedly at the 8th bed joint from the bottom. At a drift of 0.07%, a new horizontal crack occurred at the base in the pull direction. Then, it was connected to the crack in the push direction and formed a full-length crack at the base. The wall started a pure rocking about this crack.

The first vertical crack was formed parallel to the wall plane at a drift of 0.86% in the push direction. First visible vertical cracks in the front surface of the wall were formed at 1.1% drift in both pull and push directions. At this load step, the masonry unit located at the corner in the compression zone split when loading in the pull direction. Increasing the load steps resulted in the formation of new cracks at the compression regions of the rocking wall. The cracks were concentrated at the corners and did not distribute toward the center of the wall.

The maximum shear resistance was reached at 1.3% drift in the pull direction. In the push direction, however, the shear resistance increased until a drift of 3.6%. This was due to the fact that the cracked region in the compression zone could still resist the applied compressive stresses in the push direction. However, in the pull direction the bricks located at the corner in the compression zone lost their integrity with the wall. The compressive behavior of the separated part was dependent on how it was located under the rocking wall. The shear capacity of the wall in the pull direction decreased when the separated part was fully crushed.

By increasing the horizontal displacement, the compression zones in both directions were largely destroyed. The shear resistance, however, did not decrease significantly. The degraded shear resistance was about 93% and 94% of the shear strength in the pull and push directions, respectively. This implies a very stable rocking behavior of the wall with a large displacement capacity which is very favorable in the case of earthquakes. The effective displacement capacity of the wall was not reached during the test as the test had to be stopped because of the limits of the test set-up (fixation of the steel beam and displacement capacity of the horizontal jack).

4.5.1.4 Wall 4

Wall 4 experienced its first horizontal cracks at the base at a drift of 0.04% in both directions (a greater drift than Wall 3). At 0.11% drift the cracks joined and formed a full-length crack. The wall started rocking about this crack.

The first vertical crack occurred in the compression corner in the pull direction. This crack opened at a drift of 0.22%, which was significantly lower than the drift in the corresponding state of Wall 3. In the push direction, the vertical crack occurred slightly later at a drift of 0.31%. By increasing the load steps, the rocking about the first crack continued and as a result more cracks were initiated. The cracks were concentrated at the corners but at higher elevations in the wall.

The maximum shear resistance was reached at 1.2% drift. In the same load step, a full length crack occurred at the 3rd bed joint and the wall started rocking about the newly formed horizontal crack. It resulted in the formation of new vertical cracks closer to the center of the wall. The vertical cracks at the corners joined and the compression zones were pushed outward. Therefore, the shear resistance of the wall degraded to 87% and 82% in the pull and push direction, respectively.
Increasing the horizontal displacement resulted in the movement of vertical cracks toward the center of the wall. The compression corners were completely destroyed. The vertical load was carried only by the central portion of the wall at its base. New full length cracks occurred at the 5th bed joint and rocking occurred about this crack. The test was stopped at a target drift of 2.5%. Due to an error in applying the horizontal displacement, the final drift in the wall was unsymmetrical and equal to 1.9% and 2.3% in the pull and push directions, respectively. The remaining shear resistance of the wall was 47% and 37% of the maximum shear resistance in the pull and push directions, respectively.

4.5.1.5 Wall 5
The first horizontal crack occurred at a drift of 0.06% in both pull and push directions at the 3rd bed joint. Increasing the horizontal displacement the cracks stepped down to the 2nd bed joint and joined together. The wall started rocking about the full-length crack.

Due to rocking the first vertical cracks in the compression zones occurred at a drift of 0.63% and 0.91% in the pull and push directions, respectively. By increasing the load steps, the density of vertical cracks increased.

The maximum shear resistance was reached at 1.4% drift in both directions; at this point the destroyed bricks at the outer corners were pushed outwards. At the same drift, during unloading from the pull direction in the second cycle, a diagonal tensile crack occurred at the bottom of the wall. Then, sliding and rocking occurred about this crack in the pull direction. In the push direction, however, the wall continued rocking about the previous full-length crack at the second bed joint. Under higher horizontal displacement, new horizontal and diagonal cracks occurred in the push direction as well. These cracks initiated at a drift of 1.9%.

The test was stopped when a major part of the compression zones was lost. This resulted in a degraded shear resistance of 79% and 76% of the maximum shear resistance in the pull and push directions, respectively.

4.5.1.6 Wall 6
The first horizontal crack was formed in the 2nd bed joint in the pull direction at 0.17% drift. At the same drift, this crack was followed by a similar crack at the 3rd bed joint in the push direction. The cracks were connected at higher load steps and the wall started rocking. Compared to Wall 5, the first horizontal crack occurred at a higher drift due to higher initial normal stress.

Vertical cracking in the compression zone started in the push direction at a drift of 0.4%. In the second and third cycles of the same load step, vertical cracks were also formed in the pull direction. By increasing the horizontal displacement, the density of vertical cracks increased at the compression corners. The new cracks formed at higher elevations.
At 0.84% drift the wall reached the maximum shear resistance. At this load stage, the compression corners were fully cracked but still not lost. In the second cycle of the same drift, a diagonal crack occurred in the entire wall in the push direction. It resulted in a drop in the shear resistance in the push direction. In the pull direction, however, the behavior of the wall did not change and it was still rocking about the full-length crack. At the end of this load step, the shear resistance of the wall was degraded to 86% in the pull direction and 79% in the push direction.

The horizontal drift was increased to 1.3% in the pull direction. It resulted in a severe diagonal cracking that reduced the shear resistance of the wall to 55%. The test was stopped at this drift before applying the same drift in the push direction because of the possibility of collapse.

### 4.5.2 Horizontal force-drift

The history of shear force vs. drift of the walls is shown in Fig. 4.13. Different histories were observed in the walls depending on their failure mode.
A nonlinear nearly elastic behavior was observed in Wall 1 prior to the diagonal cracking. The opening in the individual curves, which was due to sliding about the full-length crack at the bed joints, gradually increased. When the diagonal crack formed, the hysteretic curve opened and the residual displacement increased significantly. The degradation of the shear resistance during three cycles of the last load step was not significant showing that higher in-plane displacements could be resisted by the wall.

Wall 2 also showed a nonlinear elastic behavior at the early load steps similar to Wall 1 prior to toe crushing. However, its shear force vs. drift curve was slightly more open compared to Wall 1 as a result of sliding. When toe crushing occurred, much higher energy was dissipated by Wall 2 and the area of the cycle increased. Due to higher damage at the bottom of the wall, the stiffness of the wall decreased which can be seen in the horizontal force-displacement diagram by reduction in the slope. The degradation of the shear resistance during the last load step was significant due to the damage in the compression zones.

A very stable nonlinear elastic behavior was observed in Wall 3 due to pure rocking. Individual curves corresponding to load steps showed a “flag shape” behavior. The area inside the flag represents the dissipated energy due to cracking and crushing of the corner zones. With increasing damage in the compression zones the hysteresis opened more and more, indicating an increase in the energy dissipation. The damage remained concentrated in the compression zones at the bottom toes of the wall.

At the early steps of loading, Wall 4 also showed a nonlinear elastic behavior due to rocking about the tensile crack at the base bed joint. The hysteretic curves remained almost closed and residual displacement was negligible. By increasing the load steps and formation of vertical cracks in the compression corners, Wall 4 experienced a more pronounced initial stiffness reduction than Wall 3. Due to rocking, elongation occurred in the post-tensioning bars that resulted in a higher shear resistance and stiffer behavior compared to Wall 3, in which a plateau was observed in the shear force vs. drift curve. The hysteretic behavior in each load step was like a bilinear elastic behavior including energy dissipation. When crushing in the compression corners occurred, the energy dissipation increased resulting in open hysteretic curves. However, due to the action of the post-tensioning bars the wall returned to the origin and the residual displacement was insignificant, i.e. a “self-centering” behavior.

Wall 5 also showed a “flag shape” behavior similar to Wall 3 due to rocking about the full-length crack. The initial stiffness decreased gradually as a function of lateral displacement even before vertical cracks became visible. Prior to diagonal cracking, a plateau was observed in the force-drift diagram up to 1.4% drift. The residual displacement remained insignificant. By formation of the diagonal crack, the force-drift curve opened and sliding resulted in an increase in the residual displacement and energy dissipation. When the compression corners were partially destroyed, the shear force resistance decreased. Afterwards, even though the force-drift diagram showed a plateau during loading, the amount of degradation was significant that led to the stoppage of the test.

A nonlinear elastic behavior similar to Wall 4 was observed in Wall 6. As opposed to Wall 5, no plateau was observed in the force-drift behavior. In the last load step, when the diagonal crack formed, the hysteretic curve became open and the dissipation increased significantly. This change in the failure mode of the wall led to a high residual displacement after unloading.
4.5.3 Sliding

Sliding is considered as a ductile failure mode in unreinforced masonry walls. The contribution of sliding in the total horizontal displacement of the walls is investigated in this section.

As expected, no sliding was measured by Transducer H1 (Fig. 4.7) between the masonry wall and the load beam in all experiments. Transducer H2, however, showed some sliding between the wall and the foundation beam and in various tests. H2 measures the relative horizontal displacement only locally. The ICS results can be used to address this shortcoming and measure the relative horizontal displacement of the entire surface of the walls. But first, the sliding measured by ICS shall be verified by LVDT results. For that purpose, the horizontal displacement of the point corresponding to H2 in the ICS measurement is verified with the measurement taken by Transducer H2 in Wall 1. The comparison is shown in Fig. 4.14. The horizontal displacement in the pull direction is considered as positive. The comparison confirms the properness of the measurement of local sliding by ICS.

Fig. 4.13 Horizontal force vs. drift.
In Wall 1, as seen in Fig. 4.14, sliding occurred in both pull and push directions at early steps of loading. The sliding gradually increased and resulted in a residual displacement of 2.5 mm in the pull direction. This is due to the fact that Transducer H2 was installed on a region of the wall that lost its integrity due to horizontal cracking. This can be clearly seen in Fig. 4.15(a) showing the contour of horizontal displacement at the last load step in the pull direction. In order to have a better estimation of the contribution of sliding in the total horizontal displacement in both pull and push directions, a region of the wall should be considered which remains integrated to the wall as long as the horizontal loading is continued. This region is found to be the sky blue region shown in Fig. 4.15(a), in which “Point A” is shown. The history of horizontal displacement at Point A is compared with that of Point H2 in Fig. 4.15(b). It can be seen that this region of the wall experienced the cyclic sliding longer. Consequently, the maximum sliding in both directions is estimated as 3.5 mm and 0.9 mm in the pull and push directions, respectively, which is equal to 36% of the total horizontal displacement in the pull direction and 9% in the push direction.

It should be noted that after the formation of the diagonal crack, the upper part of the wall slid about this crack and no more sliding was observed at the lowest bed joint. If we consider a point on the lowest row of the upper part of the wall, shown as “Point B” in Fig. 4.15(c), its horizontal displacement exceeds more than 80% of the total horizontal displacement in both push and pull directions. For more clarification, a comparison between the histories of the horizontal displacement of Point B and the horizontal displacement of the top of the wall, measured by H4, is illustrated in Fig. 4.15(d) for the last load step. A significant part of the total horizontal displacement is provided by the horizontal displacement of Point B.
In Wall 2, between the wall and the foundation slab, a cyclic horizontal movement coincided with the formation of a continuous crack throughout the length of the wall. After the formation of new full-length cracks at higher elevations, sliding occurred about the newly formed cracks and a residual displacement of 2.9 mm was observed in the push direction. The contribution of sliding in the total horizontal displacement can be found by measuring the horizontal displacement of the bottom row of the separated part located just above the highest full-length crack. Fig. 4.16(a) shows the contour of horizontal displacement in the push direction at the last load stage. “Point A” is considered on the lowest point of the upper part in order to find out the sliding between the separated parts. The horizontal displacement of Point A, H2 and H4 is exported from the ICS results and plotted in Fig. 4.16(b). The percentage of sliding in the pull and push directions reach to about 52% and 61% of the total horizontal displacement, respectively, during the last load step. It can be concluded that even though the sliding between the wall and foundation is negligible, formation of new continuous cracks through the wall length can result in a significant sliding. This affects the elongation of the post-tensioning bars which will be discussed later.
Test results

In contrast to the squat walls, a rocking behavior was expected in the slender walls and therefore sliding was considered to have a lower contribution in the total horizontal displacement. This was observed in the experimental results of slender walls. Fig. 4.17 shows the contour of horizontal displacement of the entire surface of walls at maximum horizontal displacement, as well as a comparison between the time histories of applied horizontal displacement and sliding at different points. It can be seen that even when the sliding about the newly formed cracks are taken into account, “Point A” for instance, the proportion of sliding to the total horizontal displacement remains limited. A detailed description of the sliding behavior of slender walls is as follows.

In Wall 3 sliding was negligible until the very end of the experiments and the rocking mode contributed mainly to the horizontal displacement. During the last load steps, sliding in the push direction increased gradually and reached to about 6% in this direction. In the pull direction, however, the amount of sliding remained negligible. This was due to the fact that the compression corner in the pull direction was split from the wall but remained connected to the foundation. Therefore, this part did not experience horizontal movement; see Fig. 4.17(a). It was therefore acting as a barrier to the horizontal sliding of the wall in the pull direction. The asymmetric sliding can be clearly seen in the time history of H2 plotted in Fig. 4.17(b).

Between Wall 4 and the foundation slab small cyclic displacements were observed. The wall, however, always returned to its origin at the end of a load step. The maximum amount of residual displacement at the bottom of the wall was about 0.7 mm in the pull direction. However, similar to Wall 2, new full-length cracks formed during the cyclic loading that resulted in sliding of the upper part of the wall about the lower part. Clearly, sliding about the newly formed cracks contributed more significantly to the total horizontal displacement of the wall. In the last load step, sliding of Point A was about 12% and 8% of the
Fig. 4.17 (a, c, e, and g) Contour of horizontal displacement of Wall 3 to Wall 6, respectively, at the last load step and (b, d, f, and h) their corresponding time history of horizontal displacements.
total horizontal displacement in the push and pull directions respectively, while almost no sliding was reported by H2. However, compared to Wall 2, the contribution of sliding in the total horizontal displacement is less significant. The residual displacement in the push direction measured by Point A shows a shift in the position of the wall. The asymmetric horizontal displacement applied to the wall was in fact influenced by this shift in the wall position.

In Wall 5, H2 showed almost zero displacement because the first cracking occurred at the second bed joint from the bottom. The amount of sliding was limited before the diagonal crack formed and joined to the horizontal crack of the other side of the wall. After the formation of a continuous crack, sliding started to increase. During the last load step, Point A showed a 3.7 mm horizontal displacement in the pull direction, which was about 10% of the total horizontal displacement. In the push direction, the region on which Point A is located was separated from the wall and therefore, a 6 mm residual displacement was measured by the ICS in the pull direction. Moreover, this residual displacement is not compatible with the residual displacement of the wall in the push direction. Another point must be considered on the first row of the upper part in order to investigate the amount of sliding in this direction; about 15% of the total displacement was calculated as the sliding of the wall in the push direction at the last load step.

Similar to Wall 5, the first horizontal crack occurred above the first bed joint in Wall 6. Therefore, no relative displacement was measured by H2. Point A, shown in Fig. 4.17(g), was selected above the first full-length crack. A small cyclic displacement was observed in this measurement before the diagonal crack occurred. Afterwards, the amount of sliding remained limited and reached to 1.3 mm in both directions at the last load step which is equal to 8% of total displacement.

In summary, it was confirmed that in the squat walls with a cantilever boundary condition, sliding contributed to the total horizontal displacement significantly. In the slender walls, however, the contribution of sliding remained limited. When sliding occurs in a post-tensioned masonry wall, the uplift in the wall as well as the elongation in the bars decreases. As a result, the effectiveness of post-tensioning decreases. The post-tensioning force in the bars is described in the next section in more detail.

4.5.4 Post-tensioning force

The post-tensioning force histories for the post-tensioned walls are plotted against the horizontal drift in Fig. 4.18. The force for different post-tensioning bars is plotted in separate diagrams. It is reminded here that in Wall 4 and Wall 6, the load cells K2 and K4 were installed on the bars located on the opposite side of the wall but at the same location as the bars with the load cells K1 and K3, respectively. Therefore, the forces measured by K2 and K4 are similar to those measured by K1 and K3, respectively.

An increase in the post-tensioning force was observed in all post-tensioning bars during applying the in-plane displacement. This increase is due to the uplift of the wall when rocking occurs about horizontal tensile cracks. The amount of increase in the axial force of the bars depends on the distance between a post-tensioning bar and the center of rotation, about which rocking occurs. When a horizontal displacement is applied to the wall, the post-tensioning bars located far from the rotation center experience a higher increase in their axial force. This can be clearly seen in Fig. 4.18(a), by comparing K1 and K2.
The failure mode of masonry walls is another parameter that influences the amount of increase in the post-tensioning forces. In Wall 4, in which rocking was very significant, the average post-tensioning force of K1 to K4 increased from 20.8 to 72.2 kN, that is about a 246% increase. In Wall 6, however, diagonal cracking occurred and therefore the uplift of the wall and the elongation of the post-tensioning bars remained limited. Therefore, the post-tensioning force increased from 36.8 to 80.9 kN, about 120%. Wall 2 experienced an increase of 188% in the average post-tensioning force of K1 and K4. The in-plane displacement of this wall consisted of sliding and rocking. The contribution of sliding to the increase in the post-tensioning force is less compared to the rocking because during sliding the uplift is insignificant and the elongation in the bars is only due to the change in the orientation of the bars. Since the drift ratio is not significant in Wall 2, this change in the orientation of vertical bars results in a negligible increase in the post-tensioning force.

Even when compression corners crushed and the maximum shear resistance was surpassed, the amount of post-tensioning force increased in the bars experiencing elongation. However, when the wall was unloaded to zero displacement state, the amount of post-tensioning decreased significantly. As seen in Fig. 4.18, the post-tensioning force decreased to zero in Wall 2 and Wall 4. This is due to the fact that the vertical stiffness of the walls decreased when compressive damages occurred at the bottom of them. In
contrast, post-tensioning forces did not reduce to zero in Wall 6 because of less compressive damages at its bottom part as a result of the diagonal tensile failure mode.

In summary, assuming a certain material for the post-tensioning bars, there are some parameters which greatly influence the amount of post-tensioning in post-tensioned masonry walls. The main factors are the failure mode, the distance of the bar from the center of rotation of the wall and the amount of uplift generated in the wall. The higher the elongation in the post-tensioning bars, the higher the increase in their tensile force. Increase in the density of cracking of a masonry wall lowers its vertical stiffness and results in a release in the post-tensioning force.

4.5.5 Masonry average vertical strain

As seen in the overall behavior and failure pattern of the masonry walls (section 4.5.1), most of the damage occurs at their bottom part in a cantilever geometry and the upper part remains undamaged if the diagonal cracking failure is avoided. Therefore, it is of high interest to measure the vertical strain in the masonry walls at their bottom part. The vertical strain can be calculated using the results of the image correlation system (ICS). By monitoring the vertical deformation of a desired section at the bottom part of a wall, the average vertical strain can be calculated. In this study, the section is considered to be above the last horizontal crack in all walls. Therefore, the vertical deformation of the total length of the wall could be captured until the end of experiments. For this purpose, the horizontal section was selected at the height of 450 mm which is located on the 7th row of bricks.

Fig. 4.19 shows the average vertical strain of the masonry walls in the push direction at four distinguished states of failure which are, first horizontal cracking, first vertical cracking, maximum shear resistance and degraded state. As seen in the figure, in all masonry walls the distribution of the average vertical strain at the bottom of the walls remained linear until the maximum shear strength was reached. In the post-peak region, however, depending on the type of failure the average vertical strain at the bottom varied. In walls 1-2, the failure in the corners reached the height of the considered section (x = 450 mm) and therefore, some discontinuities were observed in the vertical strain results.

Higher vertical compressive strain was observed in the walls with post-tensioning when the failure mode was toe crushing. In Wall 6, however, since the failure mode was diagonal cracking, the vertical strain was lower than in Wall 5 without post-tensioning. The amount of uplift, which is represented by positive vertical strain, was higher in the walls without post-tensioning. The uplift in the post-tensioned walls was less because of the constraints provided by the post-tensioning elements. By increasing the horizontal displacement, the distribution of vertical cracks at the bottom of the wall increased and as a result the vertical stiffness of the wall decreases and therefore, the uplift decreased.
In the degraded state of the post-tensioned walls, the compression zone extended toward the center of the wall, whereas in the walls without post-tensioning, the compression zone was concentrated at the corners. The movement of the neutral axis toward the center of the wall results in a self-centering behavior in the post-tensioned walls. It means that the wall returns close to its original state after a cyclic action.

Fig. 4.19 The average vertical strain in masonry over lowest 450 mm height of masonry walls (a)-(f) Wall 1-Wall 6 in the push direction.
4.6 Discussion

In this section, the influence of post-tensioning on the in-plane behavior of masonry walls is discussed. Shear resistance, displacement capacity and seismic behavior of the masonry walls are considered as desired parameters on which the effect of post-tensioning is evaluated. In order to observe the effect of post-tensioning on the shear resistance and displacement capacity of URM walls with weak mortar, the envelope of the in-plane force vs. drift curve of the squat and slender walls, without and with post-tensioning is shown in Fig. 4.20.

![Fig. 4.20 Envelope of the shear force vs. drift curve of all wall samples.](image)

4.6.1 Effect of post-tensioning on the shear resistance

The shear resistance of masonry walls showed a significant increase in all cases. In the squat masonry wall, the increase is due to the enhancement of normal stress in the wall. As seen previously in the failure pattern of the squat walls without post-tensioning (Wall 1) and with post-tensioning (Wall 2), the increase in the normal stress prevented Wall 2 from diagonal cracking and resulted in toe-crushing. It can be expressed that post-tensioning increases the integrity of the squat wall by increasing the normal stress on the possible diagonal cracks. Comparing Wall 3 (the slender wall without post-tensioning and with lower normal stress) and Wall 4 (the slender wall with post-tensioning and with lower normal stress), it is seen that post-tensioning resulted in a 4-time increase in the shear resistance, which is significantly higher than that in the squat geometry. The amount of increase in the shear resistance of Wall 6 (the slender wall with post-tensioning and with higher normal stress) from Wall 5 (the slender wall without post-tensioning and with higher normal stress) was not as significant as the previous case though. This can be explained by looking at the failure mode of the walls as well as the stress field method for evaluating the shear resistance.

The nondimensional shear capacity of the URM walls calculated based on the stress field method is shown by the black curve in Fig. 4.21 as a function of applied normal stress. Fig. 4.21(a) illustrates the curve for the squat walls whereas Fig. 4.21(b) is for the slender walls. The predicted and observed shear resistances are plotted on the figures based on the applied normal stress. As seen in the figure, the predicted shear stresses are close to the observed results in all walls without and with post-tensioning except for Wall 4. This is due to the fact that the stress field method considers post-tensioning only as an increase in the
normal stress. In Wall 4, rocking contributes significantly in the displacement of the wall, hence the uplift in the wall produces elongation in the post-tensioning elements and as a result, an extra normal force is generated. Moreover, a significant bending moment is generated in the opposite direction of the applied horizontal force, which increases the amount of shear force required to reach a certain horizontal displacement. The current stress field method cannot include such effects as it only takes into account the initial normal stress. In Wall 2, the amount of uplift was not significant because of the geometry of the wall. In wall 6, as the failure mode was diagonal cracking, the amount of uplift remained limited and therefore, the amount of increase in the shear resistance of the wall was comparable with a wall with an extra normal stress.

It can be concluded that post-tensioning generally results in an increase in the shear resistance of unreinforced masonry walls by increasing the normal stress. But nevertheless, an extra increase occurs only when the geometry and existing normal stress in the wall allow for a significant elongation in the post-tensioning elements. In this case, the increase in the shear resistance becomes more significant.

Fig. 4.21 Nondimensional shear resistance versus normal force (stress fields method) and experimental results for (a) squat walls and (b) slender walls.

4.6.2 Effect of post-tensioning on the displacement capacity

The ultimate displacement is considered to correspond to a specific limit of the strength degradation. It is common in the literature to use 80% of the shear resistance as the ultimate degraded state as suggested by Magenes and Calvi (1997). As described in section 4.5.1 in Wall 1 and Wall 3, even though the walls started to degrade, they did not experience the 20% degradation in their shear resistance. To be able to compare the results, the ultimate state in Wall 2 and Wall 4 (post-tensioned walls) was considered to have the same degradation as Wall 1 and Wall 3 (without post-tensioning), respectively. In Wall 5 and Wall 6, the degradation was more than 20% and the ultimate displacement was considered at this point. In Fig. 4.22, the ultimate states are illustrated. Table 4.4 summarizes the experimental results regarding the ultimate drift and the percentage of degradation, in which the ultimate drift is calculated.

It can be seen in the table that by applying the post-tensioning, the ultimate drift decreased in all walls both in pull and push directions except for the pull direction in Wall 2 (squat wall with post-tensioning) compared to Wall 1 (squat wall without post-tensioning). The amount of decrease in the ultimate drift in the squat walls is less than that in the slender walls. In summary, post-tensioning decreased the average
value of the ultimate drift in the push and pull directions to about 89% in the squat wall, to 36% in the slender wall with lower existing normal stress and to 42% in the slender wall with higher existing normal stress. It can be concluded that if the failure mode of a URM wall is rocking, post-tensioning results in a more significant decrease in the ultimate drift capacity.

Table 4.4 Experimental results of maximum shear resistance, degraded shear resistance and ultimate drift of the walls.

<table>
<thead>
<tr>
<th>Wall name</th>
<th>Maximum shear resistance [kN]</th>
<th>Degraded/Maximum shear resistance [%]</th>
<th>Ultimate drift [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>pull</td>
<td>push</td>
<td>pull</td>
</tr>
<tr>
<td>Wall 1</td>
<td>64.0</td>
<td>-65.0</td>
<td>83</td>
</tr>
<tr>
<td>Wall 2</td>
<td>156.5</td>
<td>-170.5</td>
<td>83</td>
</tr>
<tr>
<td>Wall 3</td>
<td>20.5</td>
<td>-22.2</td>
<td>93</td>
</tr>
<tr>
<td>Wall 4</td>
<td>84.9</td>
<td>-92.3</td>
<td>93</td>
</tr>
<tr>
<td>Wall 5</td>
<td>46.1</td>
<td>-46.0</td>
<td>80</td>
</tr>
<tr>
<td>Wall 6</td>
<td>108.9</td>
<td>-101.8</td>
<td>80</td>
</tr>
</tbody>
</table>

Fig. 4.22 Envelope of the first cycle of the shear force vs. drift curve of the (a) squat walls and (b) slender walls with the specification of the degraded limits considered for the ultimate drift.

4.6.3 Seismic behavior of masonry walls

Eurocode 8-3 (2004) is considered as the standard for the seismic assessment of masonry buildings and their elements. Eurocode 8-3 distinguishes two different behavior modes: a flexural mode and a shear mode. If the shear resistance of a masonry wall predicted by the flexural mode is greater than that predicted by the shear mode, the wall is assumed to be controlled by shear and vice versa. The Limit State (LS) of Significant Damage (SD) for the walls controlled by flexure and shear is expressed in terms of drift and taken equal to 0.008·h_w/l_w and 0.004 for primary walls, respectively. The capacity of an unreinforced masonry wall in the LS of Near Collapse (NC) may be taken as 4/3 of the values specified for the SD.

Fig. 4.23 depicts the envelope of shear force versus drift of the masonry tests again. The required drift capacity of masonry walls corresponding to the LS of SD is illustrated in the figure. In the post-tensioned squat wall (Wall 2), the displacement capacity satisfies the SD limit state when it is controlled by shear.
The post-tensioned slender walls satisfy the SD limit state for flexure when loaded in the pull direction. However, the post-tensioned slender wall with higher axial stress shows a lower drift capacity than that specified by Eurocode. It can be concluded that high amount of post-tensioning in a masonry wall, which is under high normal stress, results in a brittle failure considering the thresholds specified by Eurocode 8-3.

![Graph](image)

Fig. 4.23 Illustration of the significant damage limit states on the envelope of the shear force vs. drift curves of the (a) squat walls and (b) slender walls.

### 4.7 Conclusions

An experimental study was carried out on large-scale masonry walls in order to evaluate the influence of post-tensioning on the in-plane behavior of unreinforced masonry walls. The following conclusions can be drawn regarding the experimental results.

- Post-tensioning generally results in an increase in the shear resistance of unreinforced masonry walls by increasing the normal stress. An extra increase is expected when the geometry of the wall allows for a significant elongation in the post-tensioning elements. This is more pronounced in slender walls with low existing normal stress. Under high existing normal stress brittle diagonal cracking failure may occur earlier than ductile rocking failure.

- The displacement capacity of URM walls decreases by implementing the post-tensioning technique for the seismic retrofitting of the masonry walls. However, depending on the geometry of the wall, the existing axial stress and the post-tensioning level, the displacement capacity of the post-tensioned walls may satisfy the Eurocode requirements regarding the displacement capacity at the limit state (LS) of significant damage (SD). A level of post-tensioning can be found in order to satisfy the requirements.

- In squat walls, sliding contributes to the horizontal displacement more significantly than in slender walls. When sliding occurs in a post-tensioned masonry wall, there is less elongation in the post-tensioning bars and consequently, the increase in the shear resistance is limited to the increase in the normal stress.

- The stiffness of the post-tensioning tendons, their distance from the center of rotation of the wall and the amount of uplift produced in the wall are mainly influential on the shear resistance and
drift capacity of the wall. The higher the elongation in the tendons, the higher is the increase in their post-tensioning force. Increase in the density of cracking of a masonry wall lowers its vertical stiffness and results in a release in the post-tensioning force.

- A favorable self-centering behavior was observed in the in-plane response of post-tensioned masonry walls.
- Rocking is a very stable failure mode in the URM walls in terms of their displacement capacity, but their shear resistance remains limited. It was observed in the slender masonry walls with low existing normal stress. However, in the squat walls or walls with high existing normal stress, where the shear resistance was high, a limited displacement capacity was observed.

4.8 References

5 Numerical modeling

This chapter describes three types of models with different complexities for predicting the in-plane response of post-tensioned masonry walls. These methods are commonly used in the literature and here they are explained and verified with experimental data.

First, a simple semi-analytical model is evaluated. The model was previously fitted to a range of experimental and numerical data to predict the flexural strength, tendon forces, and in-plane displacement of post-tensioned concrete masonry walls (Wight 2006). The model has been the theoretical basis for design of post-tensioned concrete masonry walls in some of the international masonry standards.

Next, a commonly used 1D fiber element model for predicting the cyclic response of post-tensioned walls is described. The model has been vastly used in the literature for predicting the in-plane behavior of post-tensioned rocking systems with gaps especially in the post-tensioned precast concrete units. The main assumptions of the model are stated. The performance of the model is evaluated by the experimental data from the post-tensioned wall tests. The main shortcomings of the model are explained.

Subsequently, a 2D finite element model is described that was developed to predict the in-plane behavior of post-tensioned masonry walls. The developed model uses the available mechanical properties of the mortar and masonry based on the small scale tests that were explained in Chapter 3. The simulation of all post-tensioned walls using the FE model is described. The full-field displacement predicted by the model is compared with that of the image correlation results. The ability of the model to be used for a parametric study on post-tensioned masonry walls is assessed.

5.1 Semi-analytical model

In order to predict the in-plane response of post-tensioned concrete masonry (PCM) walls, a semi-analytical model was used based on the states that a PCM wall goes through under monotonically applied in-plane displacement. These states include but are not limited to cracking, maximum serviceability, and nominal strength. The mechanism of the behavior of PCM walls under in-plane displacement is considered and the stress distribution at the bottom of the wall is evaluated accordingly. Stress in tendons at each state must be assessed accurately to find the flexural strength of the wall as well as its shear resistance.

The model was developed by Laursen (2002). Wight (2006) modified it to give a better estimation of the tendon forces. Wight verified the model with a set of results obtained from a 2D finite element model. The modified model satisfactorily predicted the nominal shear resistance and its corresponding displacement of PCM walls. Wight’s limit state model was employed in this study and compared with our experimental results. A short explanation of the model is given in the next section. The reader is referred to the aforementioned two references for more detail.
5.1.1 Limit states

Flexural cracking

Due to normal force on a masonry wall, the entire length of the wall undergoes compression. By applying the horizontal loading on top of the wall, a bending moment is generated throughout the height of the wall. Increasing the horizontal load results in the reduction of the compressive stress at one of the bottom corners of the wall. The state in which the compressive stress become zero is considered as the flexural cracking state. It is obvious that the tensile strength of masonry is ignored in this definition.

The distribution of stress at the bottom of the wall is shown in Fig. 5.1(a). Based on this distribution, the bending moment corresponding to this limit state ($M_{cr}$) is calculated by Eq. (5.1).

$$M_{cr} = \frac{(P + N)l_w}{6} = \frac{f_m t_w l_w^2}{6} \quad \text{where} \quad f_m = \frac{(P + N)}{t_w l_w}$$

(N is the existing normal force on top of the wall and $P$ is the vertical force provided by post-tensioning. $f_m$ is the average compressive stress applied to the wall as a result of both vertical forces. The thickness and length of the wall are $t_w$ and $l_w$, respectively. This formula is obtained with the assumptions that the distribution of stress and strain at this state is linear, the axial force in the tendons remains constant, and the tensile strength of the masonry is ignored. The shear force ($V_{cr}$) is calculated by dividing the resisting bending moment by the height of the wall ($h_w$), as expressed in Eq.(5.2).

$$V_{cr} = \frac{M_{cr}}{h_w}$$

The lateral displacement of the wall has a flexural component ($d_{fl}$) and a shear component ($d_{sh}$). The flexural component is calculated by integrating the curvature of the wall ($\phi$) through its height. Based on ordinary beam theory, the curvature of the wall at the base is evaluated as shown in Eq. (5.3). Assuming a homogeneous elastic behavior for masonry, the curvature of the wall through its height is proportional to the bending moment, which is distributed linearly through its height. By integrating the curvature through the height of the wall the flexural part of the lateral displacement is obtained.

$$\phi_{cr} = \frac{M_{cr}}{EI} = \frac{2(P + N)}{E_m t_w l_w^2} = \frac{2f_m}{E_m l_w}$$

$$d_{cr} = d_{cfl} + d_{crsh} = \frac{2 h_w^2 (P + N)}{3 E_m t_w l_w^2} + \frac{2 (1 + \nu)(P + N)}{5 E_m t_w}$$

In the equation above, $EI$ is the flexural rigidity of the wall and the Poisson’s ratio of masonry is shown by $\nu$. 

\[96\] Numerical modeling
Maximum serviceability state

By increasing the horizontal displacement applied on top of the wall, the crack at the base section propagates. At the same time, the compressive stress at the toe of the wall increases. The state in which the masonry reaches its elastic limit is considered as the maximum serviceability moment. In this state the masonry is considered to remain elastic.

The elastic limit in the stress-strain behavior of masonry is considered as a percentage of the maximum compressive strength of masonry. Based on Laursen (2002), this limit typically ranges between 0.45 and 0.55 for the prestressed concrete masonry in New Zealand standards.

In this state, even though the first crack has already occurred and propagated in the wall, the stress-strain relationship in the compressive masonry is assumed to be elastic and linear. The stress distribution at the base of the wall is shown in Fig. 5.1(b). The length of the compression zone \( c_e \) can be found based on the force equilibrium in the vertical direction as:

\[
c_e = \frac{2f_m l_w}{k f'_m}
\]  

(5.5)

From the moment equilibrium and using Eq. (5.5), the resisting shear force is calculated as:

\[
V_e = \frac{1}{h_w} \left(3 - \frac{4f_m}{k f'_m}\right) \frac{f_m l_w^2}{6}
\]  

(5.6)

The coefficient \( k \) is the ratio of the elastic limit to the compressive strength of masonry \( (f'_m) \). The vertical strain in the extreme fiber located in the compression zone at the base of the wall can be found by:

\[
\varepsilon_{me} = \frac{k f'_m}{E_m}
\]  

(5.7)
where \( E_m \) is the modulus of elasticity of the masonry. The corresponding curvature is calculated by:

\[
\phi_e = \frac{\varepsilon_{me}}{c_e} = \frac{(kf'_m)^2}{2f_mE_ml_w}
\]  

(5.8)

This curvature occurs at the base of the wall. A section is shown in Fig. 5.1(b), in which the combination of the axial force and bending moment results in a stress distribution equal to the one from the cracking state. At this height, the curvature of the wall is equal to the cracking curvature. The curvature varies from \( \phi_e \) at the base to \( \phi_{cr} \) at the cracking height of the wall. Above this height, the curvature varies linearly and reaches zero at the top of the wall.

In the cracked region, the curvature at each section of the wall is related to the uncracked length of that section. In this region, the parameter \( \gamma \) is defined as the cracked length at each section normalized by the wall length. The relationship between the curvature at each section and the normalized cracked length can be written as:

\[
\phi = \frac{\varepsilon_m}{(1-\gamma)l_w} = \frac{2f_m}{E_ml_w(1-\gamma)^2}
\]  

(5.9)

The lateral displacement of the wall due to flexure is obtained by double integration of the curvature along the height of the wall, which is given by:

\[
d_{eff} = \frac{2f_mh_{cr}}{E_ml_w\gamma_e}\left[ (h_w-h_{cr}) \left( \frac{\gamma_e}{1-\gamma_e} \right) + \frac{h_{cr}}{\gamma_e} \left( \frac{\gamma_e}{1-\gamma_e} + \ln|1-\gamma_e| \right) \right] + \frac{\phi_{cr}}{3} (h_w-h_{cr})^2
\]  

(5.10)

The parameter \( \gamma_e \) is the normalized cracked length at the base of the wall. The lateral displacement due to shear is obtained in a similar way as in the cracking state based on the following formula:

\[
d_{esh} = \frac{12(1+v)h_wV_e}{5E_ml_wl_w}
\]  

(5.11)

Total displacement is calculated by summing the shear and flexural displacements using Eq. (5.12).

\[
d_e = d_{eff} + d_{esh}
\]  

(5.12)

It must be noted that at this limit state, it is assumed that the deformations of the wall are so small that they do not result in significant force increase in the tendons.
Nominal flexural strength state

This state is achieved when the compressive stress in the extreme fiber reaches the compressive strength of the masonry. At this limit state, similar to the design of concrete elements under bending at ultimate limit state, an equivalent stress block is assumed with a stress limit of $\alpha f'_m$ where the corresponding strain is obtained from the stress-strain curve from the masonry centric compression tests, see Section 3.3.

As opposed to the previous states, deformations at this state are significant and result in an increase in the tendon forces. Initially, Laursen (2002) did not consider the rocking deformations in this state. The deformation shape of the wall was considered as shown in Fig. 5.2(a). For a given axial stress on a wall, Laursen evaluated the vertical deformation of the ends of PCM walls, i.e. extension $u_e$ at one end and shortening $u_s$ at the other end, by integrating the curvature of the wall along its height. This evaluation at the nominal flexural strength state was contradicted by testing showing that only one single crack forms at the base of the wall. Wight (2006) modified the assumption of the deformation at this state to take into account the rocking deformation. The considered deformation shape of the wall is shown in Fig. 5.2(b).

![Deformation shape at nominal flexural strength state considered by (a) Laursen (2002) and (b) Wight (2006).](image)

Based on the rotation at the base of the wall shown in Fig. 5.2(b) and assuming that the wall rocks as a rigid block about the neutral axis, the tendon force can be calculated using the following equation:

$$f_{ps} = f_{se} + \frac{E_p}{l_p} \left( d - c \right) \theta$$  \hspace{1cm} (5.13)

where $c$ and $d$ are, respectively, the depth of the neutral axis and depth of tendons measured from the extreme compressive fiber, and $\theta$ is the rotation at the base due to rocking. By assuming a rectangular stress block of length $a$ and writing the equilibrium in the vertical direction, $c$ can be calculated as:
It is important to have a suitable evaluation for \( \theta \) to reach a proper estimation of the tendon forces as well as lateral in-plane displacement at this state. It is common to consider a plastic height at the bottom of the wall, in which the peak strain \((\varepsilon_{mu})\) is constantly distributed. For instance, the height of plastic region is considered as a constant equal to the compression zone \( c \) in the British masonry code (BS 5628-2 2000). Therefore:

\[
\theta = \frac{\varepsilon_{mu} c}{c} = \varepsilon_{mu}
\]  

(5.15)

In a similar study, Rosenboom and Kowalsky (2004) considered the maximum of the following three equations as the length of the plastic zone \( (L_p) \).

\[
L_{p1} = 0.2l_w + 0.044h_w
\]

\[
L_{p2} = 0.08h_w + 0.022 \frac{F_y}{d_{bl}}
\]  

(5.16)

\[
L_{p3} = 0.044 \frac{F_y}{d_{bl}}
\]

Wight and Ingham (2008) studied this parameter. By fitting the tendon force to a large set of data, they defined \( \theta \) to be in direct relationship with the aspect ratio of the wall and in inverse relationship with the axial force ratio, which is equal to:

\[
\theta = \frac{\varepsilon_{mu} h_e f_m'}{30 l_w f_m}
\]  

(5.17)

Substituting Eq. (5.17) into Eq. (5.13), they proposed the following equation for the tendon stress at nominal flexural strength in the post-tensioned masonry walls.

\[
f_{ps} = f_{se} + \left( \frac{\varepsilon_{mu} h_e f_m'}{30 l_w f_m} \right) \left[ d_i - \frac{f_{m} l_w}{\alpha \beta f_m'} \right] \leq f_{py} \quad [MPa]
\]  

(5.18)

In the above equation, the force in each tendon is calculated individually by considering the distance of each tendon from the extreme fiber in compression \( (d_i) \).

Knowing the tendon forces, the shear resistance of a post-tensioned wall at this state can be simply calculated by writing the moment equilibrium. Therefore, the resisting shear force is:
\[ V_n = \frac{M_n}{h_w} = \sum_{i=1}^{n} f_{p,i} A_{p,i} \left( d_i - \frac{a}{2} \right) + N \left( \frac{l_w}{2} - \frac{a}{2} \right) \]  
\[ (5.19) \]

where \( a \) is the length of equivalent compression stress block given by:

\[ a = \frac{\sum_{i=1}^{n} f_{p,i} A_{p,i} + N}{a f_{m} t_w} \]  
\[ (5.20) \]

where \( A_{p,i} \) is the area of the \( i \)th tendon.

The lateral in-plane displacement of the wall is composed of shear, flexural and rocking displacements. The flexural displacement is due to the bending moment applied to the wall before the wall undergoes rocking. Considering the deformation shape at the nominal flexural strength shown in Fig. 5.2(b), the displacement due to shear is not comparable with the other two terms. Therefore, the in-plane displacement is calculated as:

\[ d_n = d_{n,ro} + d_{n,fr} = \theta h_w + d_{n,fr} = \frac{\varepsilon_{mu} h_w^2 f_m}{30 l_w f_m} + \frac{2 h_w^2 f_m}{3 E_m l_w} \]  
\[ (5.21) \]

**Beyond nominal strength state**

After the nominal strength state, the wall is assumed to continue rocking about the plastic hinges formed in the wall corners until the tendons yield in tension. The ultimate displacement capacity of post-tensioned masonry walls is controlled by the strain capacity of the tendons and the crushing strain capacity of the masonry. Laursen (2002) discussed the maximum reliable masonry strain based on experimental results on post-tensioned masonry walls.

5.1.2 Verification of the method with experimental results

The semi-analytical limit state method was compared with the results of three post-tensioned wall tests. As seen in Chapter 3, the stress-strain curve of masonry is highly nonlinear from the beginning. Therefore, defining 50% of the maximum compressive stress as the elastic limit, similar to MSJC standard (MSJC 2005), does not seem applicable in this case. Hence, the serviceability limit state is not taken into account in this derivation. In addition, since no yielding was observed in the tendons, the limit state beyond nominal flexural strength is not evaluated. The mechanical properties of masonry was considered based on the average values of experimental results explained in Chapter 3. The parameters for the rectangular stress block at nominal flexural strength state was considered based on MSJC as \( \alpha \approx 0.8 \) and \( \beta \approx 0.8 \). Based on the experimental result, the strain at peak stress was considered as \( \varepsilon_{mu} = 0.0048 \).

A comparison is shown in Fig. 5.3 between the predicted shear force vs. drift curves of the semi-analytical limit state method with the experimental results for the post-tensioned walls. The comparison shows that the elastic stiffness of the walls is accurately captured and the prediction of wall response at the first
cracking state in terms of strength and deformation is excellent. At the nominal flexural strength, however, the strength and stiffness of all post-tensioned walls are considerably overestimated. This is attributed to the fact that the rectangular stress block based on MSJC does not provide a realistic estimation of stress distribution at the base of the wall. In other words, the eccentricity of the vertical reaction at the base of the wall is overestimated by considering the rectangular stress block. Therefore, a higher lever arm is calculated for the applied vertical loads as well as post-tensioning forces that results in the overestimation of the shear force at a certain drift.

Considering a realistic stress-strain relationship for masonry at the bottom of the wall is supposed to provide a better estimation of the stress distribution. A cross-sectional analysis can be done with a fiber element model including the stress-strain behavior of materials at their specific locations in the cross-section. This will be examined in the next section.

(a) Wall 2

(b) Wall 4

(c) Wall 6

Fig. 5.3 Comparison between the predicted response by the semi-analytical method and experiments

5.2 OpenSees fiber element model

The most common method in the literature for predicting the in-plane response of post-tensioned rocking systems is the fiber element model. This model was extensively used by Kurama (1997) to predict the
seismic response of post-tensioned precast concrete walls. The main source of nonlinear behavior in precast walls with unbonded post-tensioning is a gap opening along the horizontal joints. A similar nonlinearity is observed in unreinforced masonry walls when the first full-length horizontal crack from. Due to this similarity, the model was later used by Laursen (2002) for predicting the in-plane behavior of multi-story PCM walls. Wight (2006) used the model for single story PCM walls and compared the model by a series of dynamic shaking table tests. It was realized that the model overestimates the stiffness of post-tensioned masonry walls.

In this study, a fiber element model was created with the OpenSees software (McKenna et al. 2000) for each tested post-tensioned wall in order to assess its ability for predicting the in-plane response of post-tensioned clay brick masonry walls with weak mortar. The predicted response was then compared to the experimental results.

5.2.1 Description of model

In the fiber element method, the unreinforced masonry wall is considered as a 1D beam element. Its cross section is discretized into a finite number of fibers. The uniaxial stress-strain behavior of various materials can be assigned to their corresponding fibers. The principal assumption in this method is that, according to Euler’s beam theory, the plane sections of the element remain plane. Based on this assumption, the uniaxial strain in each fiber at a section can be calculated by a linear combination of two variables being the axial strain at the mid-length of the section and its curvature. Distribution of normal stress at a section can be calculated when the uniaxial strain as well as stress-strain behavior of each fiber is known. By integrating the stress distribution, the axial force and bending moment of the section are obtained.

It should be noted that as the fiber element is a macroelement with flexural behavior, it inherently ignores the shear deformations of the wall. In OpenSees, however, an elastic shear behavior can be aggregated in series with the fiber element.

All three post-tensioned masonry walls were modeled using the fiber element model. Fig. 5.4 shows Wall 4 and its associated fiber model. As shown schematically in Fig. 5.4(a), the density of the fibers at corners is higher than at the center part of the wall for a better estimation of the stress distribution. Fig. 5.4(b, c) display the entire model including all nodes and elements for the squat wall and slender walls, respectively. Since plasticity is only expected in the lower part, the rest of the wall was modeled by an elastic beam element, which incorporated shear deformations too. Similarly, the loading beam was modeled by a very stiff elastic beam element. Post-tensioning tendons were simulated using truss elements. The constrained nodes (i.e. Node 8 and Node 10 in Wall 4) were positioned lower than Node 1 to provide the entire unbonded length of tendons.

Two different types of fiber elements exist in the OpenSees library that take into account a distributed plasticity along the element. The elements are called displacement-based beam-column element and force-based beam-column element. The displacement-based element follows standard finite element procedure (stiffness method) where section deformations are interpolated from approximate end node displacement fields and then, the principle of virtual displacements is used to form the element equilibrium relationship. In such elements, a constant axial deformation and a linear curvature distribution are enforced along the element length (based on linear shape functions). Therefore, where the curvature is expected to be nonlinear along the element, e.g. in the cracked part, mesh refinement is inevitable. The force-based element, on the other hand, relies on the availability of an exact equilibrium solution within a beam-
column element, where section forces are interpolated from the end node force fields and the principle of virtual forces is used to formulate compatibility between section and element deformations (flexibility method). In the force-based element, a constant axial force and a linear distribution of bending moment are enforced. This can result in a variable curvature along the element. Even though the force-based element requires internal iterations, which makes it computationally expensive, it is used in this simulation for its distinct advantage in modeling regions with highly nonlinear curvature distribution along the element length.

(a)

(b)

(c)

Fig. 5.4 (a) Wall 4 and its associated fiber model, (b) model of Wall 2, and (c) model of Wall 4 and Wall 6.

A multi-linear compressive stress-strain curve was considered for masonry and assigned to the fibers of the force-based frame element. The compressive behavior of masonry was extracted from the uniaxial compression test results described in Chapter 3. In order to simulate the opening of cracks, and hence rocking, zero tensile strength was assigned to fibers. Fig. 5.5 shows the uniaxial stress-strain curve considered for masonry in fiber element analysis compared to the average of monotonic centric tests. The uniaxial pinching material model was selected from the OpenSees library and adopted in a way to provide the behavior. More information regarding the uniaxial pinching material model can be found in LeBorgne and Ghannoum (2014).
For truss elements, the material properties in tension were taken from the data provided by the manufacturer for the post-tensioning bars. As tendons are not load resisting elements in compression, zero compressive strength was considered for the material that was assigned to the truss elements. Post-tensioning was applied to the elements by introducing an initial strain in the material. The initial strain was considered such that it resulted in the same post-tensioning force in tendons as in the experiment after the stress release due to axial deformations in the walls. Table 5.1 summarizes the mechanical properties used for the simulation.

Table 5.1 Mechanical properties used for fiber element simulations.

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</tr>
</thead>
<tbody>
<tr>
<td>Masonry</td>
<td>4800</td>
<td>0.2</td>
<td>0</td>
<td>2.4</td>
<td>8.0</td>
<td>0.48</td>
</tr>
<tr>
<td>Post-tensioning bars (ϕ12)</td>
<td>210,000</td>
<td>0.3</td>
<td>1335</td>
<td></td>
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A fiber element is discretized into a number of “segments” by considering integration points along its length. The length of the segments of a fiber element is highly influential on its flexural behavior. Basically, the curvature along the length of the segments is assumed constant. For a certain relative displacement applied to a fiber element, by changing the length of segments, the generated curvature in them alters and therefore, the stress distribution in the fibers changes. In this study, we considered a force-based fiber element with 3-point Gauss-Lobatto integration scheme. Therefore, the length of the first segment, in which the curvature remains constant, is equal to 1/6 of the total length of the fiber element. Because of two reasons the height of this segment is assumed to be 200 mm: (1) in the uniaxial compression tests, the strain was measured over three bricks and mortar layers with a total length of 216 mm and (2) most of the damage and plasticity occurs in the first few rows or bricks located at the base of the walls in the wall tests. Thus, the height of Node 2 must be considered as 6×200=1200 mm.

Several steps were considered in this analysis. First, post-tensioning was applied to the structure. Then, a concentrated vertical load was applied on Node 4 that represents the axial force of the vertical jacks as
Numerical modeling well as the dead load of the walls. It can be noticed that the procedure of applying the vertical load in the model was not the same as in the experiment; however, the amount of initial post-tensioning in the model was chosen in a way to produce the same stress in the masonry as well as post-tensioning tendons at the end of this step. At last, a cyclic horizontal displacement was applied to the structure at Node 4. The elevation of this node was considered in the simulation in a way to provide the exact same lever arm for the horizontal force as in the experiment.

5.2.2 Results and discussion

Fig. 5.6 gives a comparison between the in-plane shear force versus horizontal drift of the post-tensioned walls predicted by the fiber model and observed in the wall experiments. It is seen that overall, the prediction of the in-plane behavior was better for the slender walls than the squat wall. In the squat wall, the shear resistance was considerably overestimated but the corresponding displacement matched closely the test results. In contrast, for the slender walls, the fiber model made a proper prediction of the shear resistance, whereas the corresponding displacement was underestimated.

![Graphs showing shear force vs. drift for different walls](image-url)
Fig. 5.7 displays the variation of the post-tensioning force in the tendons as a function of the applied horizontal drift. Due to the symmetry, the post-tensioning forces of the tendons located at the same distance from the mid-length of the walls are not displayed. It is seen that the fiber element model was able to capture the post-tensioning force in the slender walls quite satisfactorily, whereas it overestimated the post-tensioning force in both tendons in the squat wall.

(a) Wall 2 (squat wall)

(b) Wall 4 (slender wall, lower axial stress)

(c) Wall 6 (slender wall, higher axial stress)

Fig. 5.7 Post-tensioning force versus drift behavior predicted by fiber element model compared to the experimental results.

Due to the concentration of damage and plasticity, the bottom part of the walls controls the overall in-plane behavior, especially in our experiments, where the walls have a cantilever boundary condition. In order to explain the deviation of the predicted results of the fiber model from the experimental results, the strain and stress distributions were studied at the base of the walls. Fig. 5.8 demonstrates the strain and stress distribution at the base of the walls at the drifts corresponding to the maximum shear force predicted by the model. For Wall 4 and Wall 6, the strain distribution was also demonstrated at a higher drift which corresponds to the maximum shear force observed in the experiment. The numerical results were extracted from the first integration point (first segment) of the fiber model at the bottom of the walls. The experimental results for the strain distribution were taken out from 3D DIC measurements at a section in the height of 200 mm.
(a) Wall 2 (squat wall)

(b) Wall 4 (slender wall, lower axial stress)

(c) Wall 6 (slender wall, higher axial stress)

Fig. 5.8 Distribution of strain and stress at the base of the post-tensioned walls at maximum shear force predicted by the fiber model and observed in the experiments.

In Wall 2, the maximum shear force occurs approximately at the same drift in the model and in the experiment. It is seen in Fig. 5.8(a) that the compression zone predicted by the fiber model is much smaller than observed in the experiment. The center of rotation of the walls is closer to the wall end in the simulation than in the experiment. Therefore, the vertical strains in the tendons predicted by the model are higher than the experimental results that lead to higher force prediction in the tendons (Fig. 5.7a).
Moreover, the lever arm of the vertical forces is higher in the simulation than in the experiment; this results in a higher shear force prediction at a certain drift.

In Wall 4 and Wall 6, the length of the compression zone was estimated satisfactorily. Therefore the vertical displacement at the location of the tendons in the model was quite close to that in the experiment. So, the prediction of the post-tensioning force at this drift (i.e. 0.49% in Wall 4 and 0.36% in Wall 6) is quite accurate (Fig. 5.7 b and c). Nevertheless, the shear force was overestimated by the model at the aforementioned drifts. It can be concluded that the behavior of the compression zone of the wall is less stiff than observed in the uniaxial compression test.

It was previously stated that the shear resistance of the slender walls was estimated properly by the model. However, the drift corresponding to the shear resistance is much lower in the model than in the experiment. The distribution of strain at the drift corresponding to the shear resistance, i.e. 1.22% in Wall 4 and 0.83% in Wall 6, is shown in Fig. 5.8 (b and c). This is attributed to the fact that the kinematics of rocking is more complicated than considered by the plane sections remain plane assumption at the bottom of the walls. This assumption is obviously violated at the base of the walls where horizontal cracks occur. That is one of the main shortcoming of the 1D fiber element model.

Fig. 5.9 shows the shear force versus drift diagrams of the post-tensioned walls at early stages of horizontal loading when the applied drifts were small. At this stage of drift, the first horizontal full-length crack already formed at the base of the walls. The model is expected to give an excellent prediction of the shear force at such drifts because of the limited plasticity that is expected in the masonry. However, as seen in Fig. 5.9, although the stiffness of the walls predicted by the model remained expectedly unchanged, the experimental results show a significant decrease in the stiffness. The stiffness decrease in the cracked walls is obvious even when drifts are very small and the walls are completely under compression. This implies that cracking at the base of the wall and crushing of the mortar layer, over which the wall rocks, result in a degradation of stiffness in the next steps of loading. This is explained more in the next section. Experimental results show that the stiffness degradation accumulates by increasing the applied drift. This phenomenon cannot be captured by the fiber model.

5.2.3 Conclusions

In this section, the commonly used fiber element method was applied to the post-tensioned walls. The following conclusions can be made based on our observations:

- The assumption of plane sections remaining plane is highly violated at the base of post-tensioned masonry walls. Therefore, the model is not able to give a realistic estimation of the stress and strain distribution at the wall base.
- The numerical results fit better to the experiments for slender walls.
- In modeling strong nonlinearities at the base of the post-tensioned walls, the height of the fiber element is highly influential.
- The decrease in the stiffness observed in the experiments, even at very small drifts, cannot be captured by the fiber element model. This stiffness degradation accumulates under cyclic loading and results in inadequacy of the fiber model in predicting the shear resistance.
5.3 ABAQUS finite element model

5.3.1 Description of the model

For simulating the in-plane behavior of post-tensioned masonry walls, a 2D finite element (FE) model was developed in ABAQUS/CAE. For Wall 2, the geometry of the model and its constituents are shown in Fig. 5.10. Wall 4 and Wall 6 were simulated in the same way.

The foundation of the post-tensioned walls was modeled using elastic shell elements. The nodes on the bottom line of the foundation were rigidly tied to a reference point and the reference point was constrained in all directions.

Fig. 5.9 Shear force vs. drift curve for small drifts predicted by fiber element model compared to the experimental results.
Fig. 5.10 FE model of the post-tensioned squat wall (Wall 2).

Rocking is a rigid body motion that cannot be described by the conventional FE method because of the assumption of the continuity of deformations. In order to capture the in-plane rocking behavior of post-tensioned masonry walls, a special contact element was modeled between the walls and the foundation. The contact element of ABAQUS allows two surfaces to move relative to each other providing potential cracking and sliding to occur. Yi (2004) also used the same contact element in analyzing a two-story masonry building at the locations where cracking was most likely to occur.

The mechanical properties of the contact element in the normal direction is shown in Fig. 5.11(a). Two surfaces are initially attached. By applying a normal pressure to the contact, a linear relationship is considered between the applied pressure and the overclosure of the surfaces. A high value of stiffness ($k=100$ MPa/mm) was considered in our model. Upon applying a tension to the contact element, the surfaces separate and the normal force is not transferred between the two surfaces.

(a)  
(b)

Fig. 5.11 The behavior of the contact element in the (a) normal direction and (b) parallel direction to the bed joints adopted from ABAQUS theory manual (ABAQUS Inc. 2012).
Fig. 5.11(b) displays the frictional behavior of the contact element. A similar Coulomb friction model is used to determine the maximum shear stress that can be transferred between the surfaces in contact. The maximum shear stress is related to the normal force with the friction coefficient ($\mu$). Before reaching the maximum shear stress, the contact element is in the sticking state. In this state, elastic slip is allowed to occur. The elastic slip limit, $\delta$, can be defined in the ABAQUS contact element to define the slipping deformation prior to sliding. In our model, a full-length crack was considered at the base of the post-tensioned walls as their initial state. Thus, the elastic slip limit was neglected. The friction coefficient was extracted from the triplet shear test in the state where they were fully cracked. For this state, it was observed in the experiments that $\mu = 1$.

As mentioned in the fiber element model, crushing of the mortar layer on which rocking occurs changes the geometry of the post-tensioned walls at the base from a straight line to a round shape. This change is presumed to have a significant effect on the in-plane stiffness of the walls. Aiming at capturing the stiffness degradation, the first mortar layer of the walls was modeled separately using a plastic material model that is able to capture the permanent deformations in the mortar due to compression. As a reference for the mortar model, the results of the compression test on the mortar layer Mo200x20 was used, which was fully described in section 3.1.2.3.

A Von Mises material model with isotropic hardening was used for the mortar, see Fig. 5.12(a). A zero yield strength was assumed for the mortar and the entire nonlinear behavior was introduced as a bilinear hardening curve. As mentioned in the description of the uniaxial compression tests on the mortar, the initial slope of the stress-strain curve is affected by many parameters such as deformations at the interface, settlement of the mortar, etc. Therefore, an exact estimation of the slope is not possible. As at low stress levels the masonry is considered to be elastic, the initial slope of the stress-strain curve of the mortar is obtained based on the Young’s modulus of brick and masonry in an inverse procedure. The modulus of elasticity of clay bricks and masonry are equal to 10000 MPa and 4800 MPa, respectively (sections 3.2.2 and 3.3.1.1). Considering the geometry of the masonry assemblage as in Fig. 5.12(b), the initial modulus of mortar was evaluated as 1333 MPa. The slope of the second line was chosen based on the experimental results on the mortar layer that is equal to 300 MPa. The modulus of elasticity of the mortar was also taken as 10000 MPa according to the experiments. Using this modulus, the unloading-reloading behavior of the mortar is captured satisfactorily as seen in Fig. 5.12.

![Fig. 5.12](attachment:image.png)

Fig. 5.12 (a) Compressive stress-strain behavior of the first mortar layer of the wall and (b) Geometry of masonry considered for the evaluation of the initial modulus of mortar.
The post-tensioned masonry walls were simulated using plain stress shell elements. The masonry was modeled using a Concrete Damage Plasticity (CDP) material model. This is a continuum, homogeneous, plasticity-based, and isotropic damage model that incorporates two main failure mechanisms: compressive crushing and tensile cracking. The uniaxial stress-strain can be introduced as shown in Fig. 5.13(a). The model is composed of a nonlinear softening behavior in tension. In compression, a hardening behavior can be defined up to the maximum compressive strength. This is followed by a post-peak softening behavior. Damage parameters of $d_t$ and $d_c$ can be considered in tension and compression, respectively. The damage parameters are used to lower the elastic modulus of the material as a function of the plastic strain.

In this study, the CDP model was adapted to the uniaxial compression test results on masonry. As seen in Fig. 5.13(b), the axial compression behavior was represented by a multilinear curve. The hardening up to the maximum compressive strength was modeled using two lines. The softening branches in the experiments are normally very steep for unconfined masonry. Considering a steep softening branch causes problems with solution convergence. Therefore, the slope of the strain softening curve was assumed in a way to avoid the convergence problems. In axial tension, a linear stress-strain relationship was considered. It was followed by a plateau. The tensile strength of the masonry was assumed to be equal to the cohesion that was obtained from the triplet shear tests ($c = 0.63$ MPa).

![Fig. 5.13 Uniaxial stress-strain behavior of the CDP model, (a) original curve and (b) adapted curve for this study.](image)

Similar to the foundation, the loading beam was simulated using plane stress shell elements. A very high modulus of elasticity was considered for the loading beam too. The loading beam was tied to the wall and no relative displacement was taken into account.

The tendons were simulated using truss elements. Elastic steel material properties were assigned to the tendons (Table 5.1). The tendons were tied to the top surface of the loading beam. The post-tensioning in the tendons was carried out by considering a “predefined stress field” in them. Similar to the fiber element method, the tendons were overstressed initially to address the stress release that occurs due the shortening of the masonry wall. As the zero compression limit was not defined for the tendons, the analysis of the walls was stopped when the first negative force was observed in the tendons.

The normal force was applied on top of the loading beam as a distributed load. Horizontal static cyclic loading was conducted in a sinusoidal form in a number of steps. The displacement amplitude increased in each step. To have a better convergence of the model, which includes high material plasticity, a dynamic
explicit analysis was carried out. The time step was chosen in a way to have negligible dynamic effects in the model.

5.3.2 Results and discussion

Fig. 5.14 shows the shear force versus drift behavior predicted by the FE model and compares it with the results of the fiber element model and experiments. It is seen that similar to the fiber element model, the shear resistance of the squat wall is significantly overestimated by the FE model. For Wall 4 and Wall 6, a reasonable prediction of the shear resistance and displacement capacity was provided by the FE model.

The post-tensioning force in the tendons is demonstrated in Fig. 5.15 as a function of the applied drift. The best prediction of the FE model was made for the slender walls and the post-tensioning force in both tendons of Wall 2 is considerably overestimated. This overestimation was observed before in the prediction of the shear force in Wall 2.
(a) Wall 2 (squat wall)

(b) Wall 4 (slender wall, lower axial stress)

(c) Wall 6 (slender wall, higher axial stress)

Fig. 5.15 Post-tensioning force vs. drift predicted by FE model compared to that of fiber element model and the experimental results.

Studying the failure mode helps to understand the reason for the inability of the FE model in predicting the in-plane response of the squat wall. Fig. 5.16 illustrates the contour of minimum in-plane principal stress as well as maximum in-plane principal plastic strain of the post-tensioned walls.

The contours are drawn for the maximum applied displacement at the step in which the failure occurs in the simulation. As seen in the contour of principal plastic strain in Fig. 5.16(a), a diagonal cracking failure is predicted by the model in Wall 2. It is seen in the contour of minimum in-plane principal stress that the maximum compressive stress is observed at the compressive corner of the wall and the reaction point does not move toward the center of the wall. This proves that no toe crushing failure is predicted by the model and the compressive stress at the corners do not reach the maximum compressive strength of the masonry. This is against the experimental results, where Wall 2 showed a toe crushing failure; see Fig. 4.12(b). In Wall 4, the rocking response is followed by toe crushing where the reaction point moves toward the center of the wall, see the blue zone in the contour of minimum in-plane principal stress in Fig. 5.16(b). Similar behavior was noticed in the experiment. In Wall 6, a combination of diagonal cracking and toe crushing is predicted by the model. High maximum principal plastic strain confirms the diagonal cracking of the wall. This is also in agreement with the experimental results, where a sudden diagonal cracking occurred.
(a) Wall 2

(b) Wall 4 (slender wall, lower axial stress)

(c) Wall 6 (slender wall, higher axial stress)

Fig. 5.16 Contour of minimum in-plane principal stress (left) and maximum in-plane principal plastic strain (right) after the failure drawn on the deformed shape of the post-tensioned walls (scale factor of deformation = 10).

Fig. 5.17 compares the contour of in-plane displacement obtained by the FE model with that measured in the experiment by the image correlation system (ICS). The comparison was carried out at the drift corresponding to the maximum observed shear resistance in the experiment. It is clearly seen that in the
experimental results of Wall 2, the center of rotation is closer to the mid-length of the wall than in the FE results. In contrast, the in-plane behavior of Wall 4 and Wall 6 obtained by the FE model closely matches the experimental results.

(a) Wall 2

(b) Wall 4 (slender wall, lower axial stress)

(c) Wall 6 (slender wall, higher axial stress)

Fig. 5.17 Contour of in-plane displacement obtained by FE model (left) and observed in the experiment by ICS (right) at the drift corresponding to the maximum shear resistance of the post-tensioned walls.
The poor performance of the Wall 2 model is attributable to the fact that the isotropic CDP material model, which was assigned to the masonry walls, is not representative for the masonry at the corners of the squat wall, because in the compression regions of the squat wall, the angle of inclination of the combined horizontal and vertical actions is higher than the friction angle of masonry. Therefore, inclined cracks occur at the bottom corners of the squat wall (Fig. 5.18a). It was observed in the stress field method in Chapter 4 that when the inclination angle of the applied load surpasses the friction angle of masonry, its compressive strength significantly decreases, see Fig. 4.11(b). In contrast, in Wall 4 and Wall 6, the cracks are more or less vertical, demonstrating a lower inclination angle of the applied stresses. Fig. 5.18 displays the crack pattern of all post-tensioned walls at 0.5% drift. The adapted CDP model based on the uniaxial stress-strain data provides a sound representation for the state of the slender masonry walls at their compression corners.

(a) Wall 2 (squat wall)

(b) Wall 4 (slender wall, lower axial stress)  (c) Wall 6 (slender wall, higher axial stress)

Fig. 5.18 Inclined crack pattern in the squat wall versus vertical crack pattern in the slender walls.

Even though the FE model provides a reliable prediction for the in-plane response of the slender walls, including the failure mode, shear resistance, post-tensioning force and displacement capacity, it still overestimates the shear resistance and stiffness of the walls especially at lower drifts. For instance, at a drift of 0.5%, the shear resistance of Wall 4 and Wall 6 is overestimated by about 38% and 19%, respectively. This is attributed to the fact that the behavior of the first mortar layer in a cyclic eccentric loading is different than its behavior under uniaxial compression. So, the axial stress-stain curve that was assigned to the mortar layer (Fig. 5.12a) is not representative of the situation of the mortar layer. The crushing of the mortar layer, on which rocking occurs, changes the bottom shape of the wall to a round shape. Therefore, the in-plane stiffness of the walls at low drifts is affected. Fig. 5.19 shows the crushing of the first mortar layer at low and high drifts.
ABAQUS finite element model

For the sake of calculation of the sensitivity of the FE results to the material behavior of the first mortar layer, the uniaxial behavior of the crushed mortar layer was assumed as shown in Fig. 5.20. The hardening modulus of the crushed mortar was reduced in order to have larger compressive plastic strain due to unloading. Under this assumption, there is a 10% reduction of the thickness of the first mortar layer at 8 MPa normal compressive stress.

Wall 4 and Wall 6 were simulated using the assumed mortar behavior. The global shear force versus drift behavior of the walls are compared with the experimental results as well as the simulations using the original mortar behavior in Fig. 5.21. Obviously, using the assumed crushed mortar behavior results in a reduced stiffness and strength, which is closer to the experimental results.
Fig. 5.21 Crushing of the first mortar layer in Wall 4 at a drift of (a) 0.22% and (b) 1.02%.

5.3.3 Conclusions

The following main conclusions can be made based on the developed finite element model of the post-tensioned walls:

- For the post-tensioned squat wall, the prediction of the in-plane response was poor. This is due to the fact that the mechanical properties considered for the masonry was only based on the uniaxial compression test results. The state of stress at the compression corners of the squat walls do not conform to that condition. In order to have a better estimation, the CDP model must be verified with multiaxial test results on the masonry.
- The FE model is able to provide a reasonable prediction for the in-plane response of the slender walls. The toe crushing and diagonal cracking failure modes were correctly predicted for Wall 4 and Wall 6, respectively. The shear resistance and stiffness of the walls is slightly overestimated by the model. This was found to be due to the degradation in the mortar layer on which the walls rock. Modifying the properties of the mortar resulted in a better prediction of the shear resistance and stiffness in post-tensioned slender masonry walls.
- Due to the complexity of the behavior of the masonry and lack of multiaxial test results, the developed FE model must be carefully used in parametric studies. The application is obviously limited to slender masonry walls.

5.4 Conclusions

In this chapter, several numerical methods with different complexities were studied. The ability of the models for predicting the in-plane response of the tested post-tensioned masonry walls was assessed. Based on the geometry and material properties used in this study the following conclusions can be made:

- The semi-analytical limit state model was found to be able to provide an excellent prediction of the shear stiffness and cracking shear force and displacement. However, it failed to give a proper estimation of the displacement capacity of the post-tensioned walls at the maximum flexural strength state.
The fiber element model, even though used extensively in the literature for predicting the in-plane response of post-tensioned walls, was found to be unable to provide a proper estimation for either the shear resistance or the displacement capacity of post-tensioned masonry walls. The prediction of shear resistance was reliable for slender masonry walls. Due to the assumption that plane sections remain plane, the application of the method must be limited to slender masonry walls or piers or multi-story walls.

In the fiber element model, the reduction of the in-plane shear stiffness of post-tensioned walls due to degradation of the mortar layer cannot be captured. It results in an overestimation of the shear resistance at low drifts.

Only rocking and toe crushing failure modes can be captured by the fiber element, unless a nonlinear shear behavior is defined for the walls.

Using the finite element model, where a predefined crack, a mortar layer at the base of the wall, and a damage plasticity material model for masonry are implemented, results in the best prediction of the in-plane response of the post-tensioned slender masonry walls.

A diagonal cracking failure mode was predicted by the FE model for Wall 2 (squat wall) as opposed to the toe crushing failure that was observed in the experiment. This is more likely due to the limitations of the material model which was formed based only on the uniaxial test results, whereas a multiaxial stress state is prominent at the compression corners of the wall. As a result, the masonry at the compression corners behaved stronger than expected and therefore, a diagonal cracking failure occurred before toe crushing.

Based on the uniaxial test results for the masonry and confined mortar, a sound prediction of the in-plane response of the post-tensioned slender masonry walls was made by the FE model. The change in the failure mode due to the increase in the vertical force and post-tensioning was captured satisfactorily by the model.

The applicability of the developed FE model for a parametric study is limited to slender walls.

5.5 References


ABAQUS Inc. (2012). ABAQUS theory manual, Providence, RI.
6 Conclusions and Future Research

6.1 Summary and conclusions

The seismic evaluation of a typical multi-story masonry building in Switzerland, demonstrated the vulnerability of old masonry structures to earthquakes. It also showed the feasibility of using post-tensioning as a seismic retrofitting technique. The motivation for this study arises because of limited research existing on the post-tensioning as a seismic retrofitting method. The in-plane response of post-tensioned old clay brick masonry walls with weak mortar is almost unknown.

An extensive literature review was carried out on the application of the post-tensioning on masonry structures and on the research conducted on post-tensioned masonry. It was found that there have been many applications of post-tensioned masonry. The method was already assessed to be used as a construction method in seismic areas. The properness of the method as a seismic retrofitting technique was confirmed in the actual design-level earthquake in New Zealand. The applications for seismic retrofitting of existing masonry buildings remained limited though, due to the lack of experimental and analytical investigations.

In order to investigate the masonry used in the residential buildings at the beginning of the 20th century, a series of small size tests was conducted on masonry and its constituents. The masonry used in this study was made of clay bricks and weak cement-lime-sand mortar. A series of uniaxial tests was carried out on the mortar with various geometries, on the brick placed under lateral tension, and on the masonry in two perpendicular directions. A series of eccentric axial tests and triplet shear tests was also performed on the masonry. The modern 3D image correlation system was successfully used in all experiments for measuring the full-field displacement of the specimens under various loadings. The most important conclusions drawn from the small size tests are as follows:

- The compressive behavior of the mortar is pressure-dependent. Under a uniaxial state, the mortar shows a distinctive compressive strength before the force resistance drops. In the confined state, which is more realistic in a masonry wall, hardening is observed in the axial stress-strain curve. The hardening slope depends on the confining stress.
- The initial secant slope of the stress-strain curve in the uniaxial monotonic tests on mortars, especially the confined ones, does not represent the modulus of elasticity of the mortar. Unrecoverable deformations occur initially when the mortar is loaded. The initial slope is called the modulus of deformability. The modulus of elasticity is better found in the unloading/reloading branch of the cyclic tests.
- In the clay bricks, a high scatter was observed in the uniaxial compressive strength in the two directions, parallel and perpendicular to the bed joints. However, the compressive strength in both directions was found to be in the same order of magnitude.
- The compressive strength of masonry is highly affected by the lateral tensile strength of the bricks, rather than their compressive strength. This is due to the fact that in the masonry under
compliance, the expansion of the mortar applies bilateral tensile stresses to the surrounding bricks.

- The interface between the mortar and bricks influences the deformability of the masonry assemblages. The stiffness of the contact is low at low uniaxial deformation due to the settlement of bricks and the mortar layer. The stiffness increases rapidly by increasing the deformation.

- The compressive strength of masonry wallets with solid clay bricks was observed to be lower in the direction perpendicular to the bed joints (X direction) than that in the direction parallel to the bed joints (Y direction), under lateral confining pressure. The reason is that the mortar expansion is more when a masonry wallet is loaded in X direction.

- In an eccentric compression test on masonry wallets, only the compression zone of the section is confined. A lower axial load capacity resulted in the eccentric tests.

- The frictional behavior of the masonry was obtained from the experimental results. The contact between the mortar and bricks has a linear relationship with the normal stress. In this study, the cohesion and the friction coefficient were found to be equal to 0.62 MPa and 0.63, respectively.

To understand the effect of post-tensioning on the in-plane behavior of existing masonry walls, an experimental study was carried out on large-scale masonry walls. As the failure mode of masonry walls is highly dependent on their geometry, the walls were constructed in two different aspect ratios including one less than unity (squat) and the other one more than unity (slender). The effect of axial load and the amount of post-tensioning was also studied on the slender masonry wall. In total, six masonry walls were constructed including two squat walls and four slender walls. One squat wall and two slender masonry walls were post-tensioned. The axial force on one of the slender masonry walls was higher representing a masonry wall located at the lower stories in a multi-story masonry building. All six walls were tested under static cyclic loading. LVDTs and 3D ICS were employed for measuring the in-plane displacements of the walls. Based on the experimental results, the following conclusions can be drawn:

- Post-tensioning generally results in an increase in the shear resistance of unreinforced masonry walls by increasing the normal stress. An extra increase is expected when the geometry and the existing normal stress on the wall allow for a significant elongation in the post-tensioning elements. This is more pronounced in slender walls with low existing normal stress.

- Post-tensioning of the unreinforced masonry walls results in a decrease in their displacement capacity depending on the geometry of the walls, the existing axial stress and the post-tensioning level. In slender walls, the reduction is more pronounced than in squat walls. A level of post-tensioning can be found in order to satisfy the requirements of Eurocode regarding the displacement capacity at the limit state (LS) of significant damage (SD).

- When sliding is the failure mode in post-tensioned masonry walls, a lower elongation occurs in the tendons resulting in a lower increase in the shear resistance of the walls; it is only due to the initial increase in the normal stress.

- Regarding the tendons, several parameters are mainly influential in the in-plane response of post-tensioned masonry walls. These parameters include the stiffness of the tendons, their distance from the center of rotation of the wall, and the amount of uplift produced in the wall (kinematics of rocking). The higher the elongation in the tendons, the higher is the increase in their post-tensioning force becomes. Stress release occurs in the tendons due to the reduction in the vertical stiffness of the walls as a result of increase in the crack density at the bottom of the walls.
• The movement of the neutral axis toward the mid-length of the wall results in a self-centering behavior in the post-tensioned walls. As a result, the wall returns close to its original state after a cyclic action.

• Rocking is a very stable failure mode in the URM walls in terms of the displacement capacity, but their shear resistance remains limited. In such walls, a high increase in the compressive stress by post-tensioning may change the failure mode to diagonal cracking. The amount of post-tensioning must be found carefully to avoid brittle failure modes.

In Chapter 5, all three post-tensioned masonry walls were analyzed using a simplified semi-analytical limit state method, a common 1D fiber element model and a more complex 2D finite element model. The ability of each method in predicting the in-plane response of the tested post-tensioned walls was investigated. Based on the geometry and material properties used in this study the following conclusions can be drawn.

• The semi-analytical limit state model failed to provide a proper estimation of the displacement of the post-tensioned walls corresponding to the nominal flexural strength. The rotation angle at the base of the walls, shown in Eq. (5.17), did not match our experimental results.

• The fiber element model was found to predict the shear resistance of post-tensioned slender masonry walls more satisfactorily. Their displacement capacity, however, was not estimated accurately. The main assumption in the fiber element is that plane sections remain plane. This assumption is highly violated at the cracked section of the walls especially in the squat walls. Therefore, the application of the method must be limited to slender masonry walls or piers or multi-story walls, in which flexural behavior is dominant.

• Stiffness reduction was observed in all masonry walls due to degradation of the mortar layer on which the masonry wall rocks. The 1D fiber element model cannot take this into account.

• The developed 2D finite element model included a predefined crack and a mortar layer at the base of the wall as well as a damage plasticity material model for masonry. The FE model provided the best estimation of the in-plane response of the post-tensioned masonry walls.

• The toe crushing failure mode happened in Wall 2 experiment due to the lower compressive strength of masonry under a combined stress state. This could not be captured by the FE model using the CDP model which was verified only with uniaxial tests result. Therefore, there is a need for comprehensive experimental data regarding the multiaxial behavior of the masonry.

• Based on the uniaxial test results for the masonry and confined mortar, a sound prediction of the in-plane response of the post-tensioned slender masonry walls was made by the FE model. The change in the failure mode due to the increase in the vertical force and post-tensioning was captured satisfactorily by the model. The full-field displacement of the post-tensioned slender masonry walls predicted by the FE model matched closely that measured by the image correlation system. The stress distribution in the system could be found using the FE model.

• The application of the developed FE model for a parametric study is limited to slender walls, unless the plastic damage masonry material is adapted to the mechanical properties of masonry in a multiaxial stress state. Likewise, the behavior of the mortar layer should be updated.
6.2 Future research

This research focused on the in-plane behavior of post-tensioned masonry walls constructed with solid clay bricks and weak mortar. A cantilever boundary condition was considered in the large-scale wall tests. Post-tensioning was applied to the walls using steel prestressing bars anchored to the foundation and the loading beam at both ends of the walls. Therefore, some aspects regarding the application of post-tensioning method on existing masonry walls were left out of the scope of this research. The following recommendations are made for future research:

- The prestress loss in the masonry due to long term deformations such as creep and shrinkage should be investigated experimentally.
- Further investigation is required on the anchorage system and the mechanism of transferring the post-tensioning force to the unreinforced masonry walls.
- Research should be performed on the effects of boundary condition on the in-plane behavior of post-tensioned masonry walls.
- Characterizing the masonry under multiaxial stress state would help finding a better material model for the FE model. For that purpose, it is suggested that a series of uniaxial tests should be performed on masonry with various inclination angles between the applied load and the bed joint.
- It is suggested that the influence of the first mortar layer on the global behavior of the wall should be analyzed using a simple analytical model considering a damaged mortar as suggested in Chapter 5.
- Monotonic failure tests on the URM walls are recommended to have a better reference for the cyclic tests.
Nomenclature

Abbreviations

AFRP  aramide fiber reinforced polymer
CFRP  carbon fiber reinforced polymer
FRP   fiber reinforced polymer
ICS   image correlation system
LS    limit state
LVDT  linear variable displacement transducers
NC    near collapse limit state
NSM   near surface mounted
PCM   post-tensioned concrete masonry
RC    reinforced concrete
SD    significant damage limit state
URM   unreinforced masonry
FE    finite element
CDP   concrete damage plasticity

Symbols

Roman characters

\( A_{ps} \)  cross sectional area of prestressing tendon
\( A_{ps,i} \)  cross sectional area of \( i^{th} \) prestressing tendon
\( E_m \)  masonry elastic modulus
\( E_{ps} \)  prestressing steel elastic modulus
\( F_y \)  reinforcement yield strength
\( I \)  moment of inertia of wall
\( L_p \)  plastic hinge length
\( M \)  bending moment
\( M_{cr} \)  bending moment at first cracking
\( M_{e} \)  bending moment at maximum serviceability
\( M_n \)  bending moment at nominal flexural strength
\( N \)  existing normal force
\( P \)  post-tensioning force
\( S_d \)  ordinate value of the design spectrum
\( T_{1X} \)  natural period in X direction
\( T_{1Y} \)  natural period in Y direction
\( V \)  shear force
\( V_{cr} \)  shear force at first cracking
\( V_e \)  shear force at maximum serviceability
\( V_n \)  shear force at nominal flexural strength
\( X \)  direction in masonry normal to bed joints
\( Y \)  direction in masonry parallel to bed joints
\( a_{gd} \)  horizontal ground acceleration
**Nomenclature**

- $c$: cohesion
- $c_e$: distance from extreme compression masonry fiber to flexural neutral axis
- $d$: distance from extreme compression masonry fiber to tendon
- $d_{id}$: diameter of post-tensioning bar
- $d_e$: damage parameter in compression
- $d_{eff}$: lateral flexural displacement at $h_u$ due to $V_{cr}$
- $d_{eh}$: lateral shear displacement at $h_u$ due to $V_{cr}$
- $d_c$: total lateral displacement at $h_u$ due to $V_e$
- $d_{eff}$: lateral flexural displacement at $h_u$ due to $V_e$
- $d_{eh}$: lateral shear displacement at $h_u$ due to $V_e$
- $d_i$: distance between the $i^{th}$ tendon and extreme compression wall fiber
- $d_u$: total lateral displacement at $h_u$ due to $V_u$
- $d_{ngf}$: lateral flexural displacement at $h_u$ due to $V_n$
- $d_{n, ro}$: lateral rocking displacement at $h_u$ due to $V_n$
- $d_t$: damage parameter in tension
- $f'_m$: masonry crushing strength
- $f_m$: average compressive stress on masonry
- $f_{ps}$: prestressing tendon stress at nominal flexural strength
- $f_{ps, i}$: prestressing stress in $i^{th}$ tendon at nominal flexural strength
- $f_{py}$: yield stress of prestressing tendon
- $f_{se}$: effective prestressing tendon stress after long term losses
- $f_{sd}$: dimensioning value of compressive strength of masonry normal to bed joints
- $f_{sm}$: mean value of compressive strength of masonry normal to bed joints
- $f_{sd}$: dimensioning value of compressive strength of masonry parallel to bed joints
- $f_{sm}$: mean value of compressive strength of masonry parallel to bed joints
- $h_b$: brick height
- $h_e$: location of the first cracking height for applied moment of $M_e$
- $h_{st}$: story height
- $h_w$: wall height
- $k$: ratio between the elastic limit stress and compressive strength of masonry
- $l_b$: brick length
- $l_v$: length of the stress field
- $l_w$: wall length
- $q$: seismic response factor according to SIA 261
- $t_b$: brick thickness
- $t_w$: wall thickness
- $u_e$: extension of the tensile end of wall
- $u_s$: shortening of the compressive end of wall

**Greek characters**

- $\alpha$: inclination angle of inclined stress field
- $\alpha$: defines equivalent rectangular stress block average stress $\alpha f'_m$
- $\beta$: defines equivalent rectangular stress block length $a=\beta c$
- $\gamma$: crack length normalized to wall length
- $\gamma_e$: normalized crack length at $M_e$
- $\gamma_f$: importance factor of a building according to SIA 261
- $\varepsilon_{me}$: extreme fiber strain in wall section due to $M_e$
- $\varepsilon_{mu}$: maximum dependable masonry strain
Nomenclature

\( \varepsilon_{xx} \)  
uniaxial strain in vertical direction \( X \)

\( \varepsilon_{yy} \)  
uniaxial strain in horizontal direction \( Y \)

\( \theta \)  
wall rotation due to rocking

\( \nu \)  
Poisson’s ratio of masonry

\( \sigma_1 \)  
1st principal stress

\( \sigma_2 \)  
2nd principal stress

\( \sigma_n \)  
normal stress

\( \sigma_x \)  
uniaxial stress in \( X \) direction

\( \sigma_y \)  
uniaxial stress in \( Y \) direction

\( \tau_{xy} \)  
shear stress

\( \phi \)  
angle of internal friction

\( \phi_{cr} \)  
curvature at wall base section at first cracking

\( \phi_s \)  
curvature at wall base section at maximum serviceability

\( \phi_n \)  
curvature at wall base section at nominal flexural strength