MASTER

Seismic retrofitting of low-rise unreinforced masonry structures in the Groningen region

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SEISMIC RETROFITTING OF LOW-RISE UNREINFORCED MASONRY STRUCTURES IN THE GRONINGEN REGION

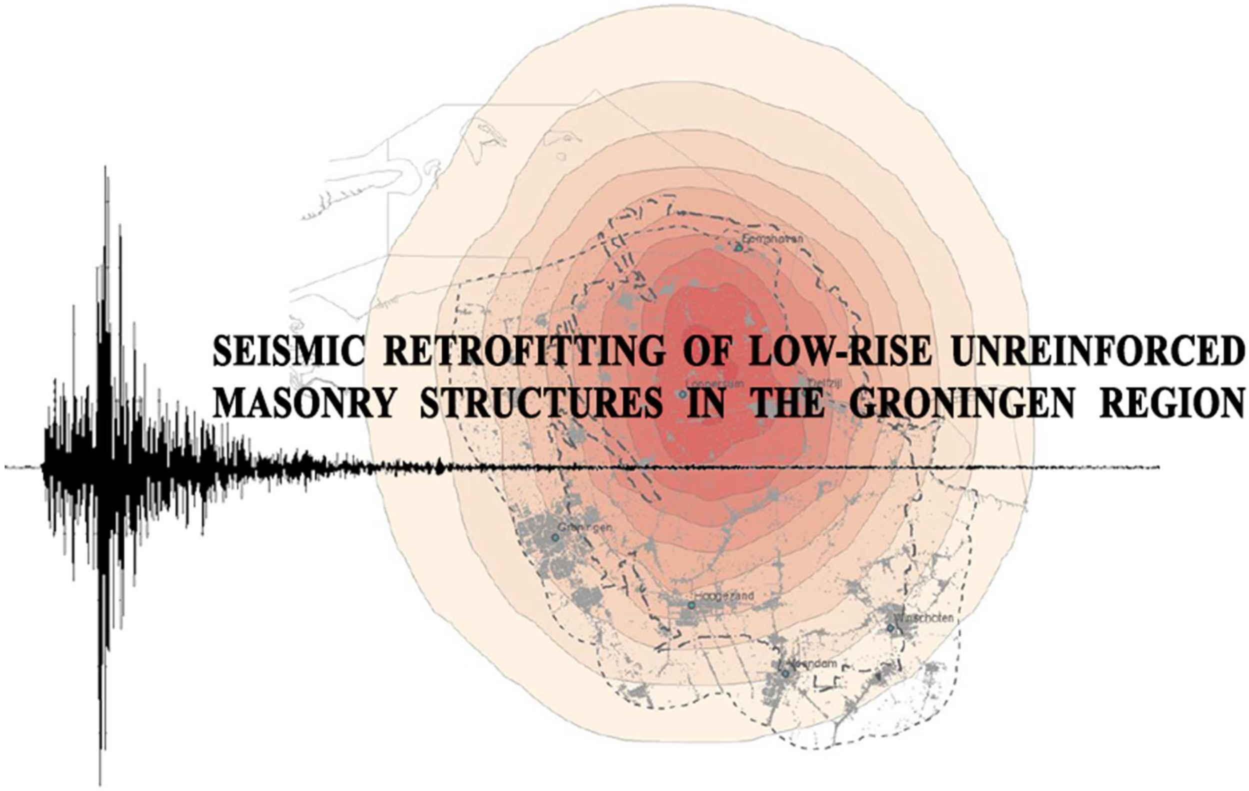

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Table of Contents

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1 INTRODUCTION

This chapter briefly introduces the aim of the thesis describing the problems and motivations which encouraged the study.

This thesis was conducted as final project of the Master of Science in Structural Design. The aim of the research project was to accomplish a series of practical solutions based on theoretical assumptions for systematic seismic retrofitting of the Netherland's built environment, located in the Groningen region.

Groningen is an area of interest for seismic retrofitting because of the confirmed induced seismicity as effect of natural gas extraction. While upgrading, the Dutch built environment presents a remarkable design challenge, the beforehand literature review introduces possible solutions and techniques already used in an international context.
The study includes an analysis of building codes and guidelines about earthquake resistant buildings in "earthquake-experienced" regions, namely California, Italy and New Zealand.
Specifically, to the deficiencies of masonry buildings to withstand seismic actions, number of solutions have been defined in the past. The proposal is to adapt to the Groningen context the best solutions among already existing techniques and new possible alternatives.

1.1 Problem Statement

Effects of decades of natural gas extraction in the Groningen region in the north of the Netherland have become apparent. Increase of soil sinking and more frequent seismic events compelled the Dutch government to take provisions regarding possible consequences for the building stock in the interested area. Despite earthquakes were already present in the past, only from 2012 they became a threat for the building stock in Groningen. Indeed, in August of 2012 a magnitude 3.6 earthquake with epicentre in Huizinge caused damages to surrounding constructions. Due to the fact that it is predicted a rise in the frequency and magnitude of these earthquakes, a research for a structural upgrading strategy is necessary in this context.

On the behalf of the Dutch government, the professional firm ARUP carried out an investigation on the Groningen region building stock. ARUP collected a building database which reveals a massive presence of unreinforced masonry structures in and around the region of Groningen.
Since unreinforced masonry finds its largest application in residential buildings, and is the construction material mostly exposed to damage in case of earthquakes, the structural upgrading study is focused on this building type.

Unreinforced masonry dwellings present on the Dutch territory differ for geometry, floor construction material and location of bearing elements. Masonry house buildings are generally prone to seismic damages, however it is possible to define a more vulnerable and a less vulnerable class depending on three parameters. Factors such as wall openness, wall type and building mass, strongly influence the behaviour of a masonry structure when subjected to dynamic horizontal loads.

Terraced and semi-detached house, which are typically two storeys buildings, represent the most vulnerable typology. Because of their initial design, which considers only the wind load as horizontal action, these structures display a strong direction and a weak direction. Horizontal loads acting perpendicularly to the long side of the buildings are well restrained by stiff shear walls. However, load acting parallel to the long sides are restrained by long walls characterized by abundance of openings, which reduce the bearing elements to slender piers.

1.2 Motivation of the Research

Unreinforced masonry low-rise buildings can be found in many places around the world. This material is widely used because of economy, ease in construction, eco-efficiency. However, the building typologies are vulnerable to damage under seismic loads.

Despite several decades of research, the best approach for a seismic retrofitting remains a primary controversy among structural engineers and researchers nowadays.

Given the decision of the Dutch government to continue with natural gas extraction, and the consequent increase of seismic events, a defined seismic structural upgrading for dwellings in the Groningen region is necessary. The interest to develop a “tool box” of retrofitting solutions specific for low-rise URM buildings concerns many other situations around the world. The Groningen scenario may be result as a testing lab where many eyes are watching.

1.3 Dissertation Outline

Chapter 2 presents a brief review of the seismic scenario in the Groningen area and describes the typology of URM buildings object of the study. In Chapter 3 the retrofitting strategy adopted is detailed and an introduction about seismic retrofitting is presented. Chapters 4,5,6,7,8 are similarly shaped, each chapter firstly introduces design informations regarding its level of seismic retrofitting, than displays possible solutions of the seismic weak aspect, and propose eventually a calculation example of the chosen technique. Chapter 9 provides a summary of the findings of this research. Appendices contain assumptions considered in the design examples, further calculation examples, extra data assumed in the retrofitting calculations and informations relative to applied materials.

2 BACKGROUND

This chapter briefly introduces the seismic scenario in the Groningen area and presents the typology of URM building analysed and its seismic weak aspects.

2.1 Geology and Seismicity in Groningen

The Netherlands in its subsurface is rich of numerous gas fields and several oil reservoirs, mostly situated in the northeaster part.

In the 1943 oil production started with the discovery of the Schoonebeek oil field, whereas the first gas field, the Coevorden field has been found in 1948. The Groningen gas reservoir, discovered in the 1959 represents the largest gas field in Western Europe and the tenth in the world.

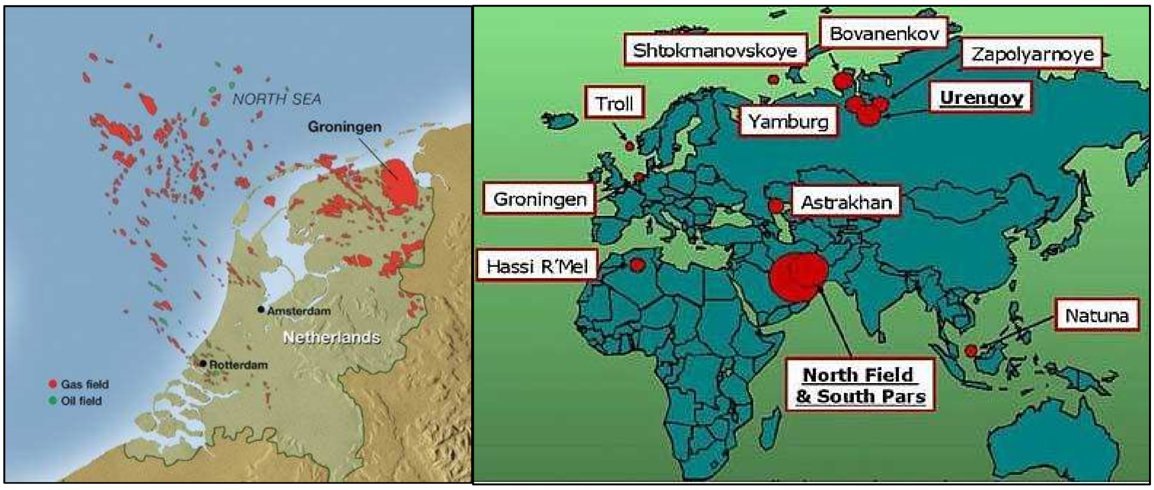


Figure 2-1 Gas fields in Netherlands Area; Largest gas fields in the worlds

2.2 Earthquake Statistics

KNMI [1](Koninkijk Nederlands Meteorologisch Instituut) published a survey about the recorded induced earthquakes between the 1995 and the 2013. During these years, most of the earthquakes have been registered in the north of the country, where 720 of them are thought to be related to the Groningen gas field. However, only 234 events had a magnitude of 1.5 or higher. It is assumed that below the magnitude of 1.5 an earthquake is seldom notice by human. In other words, the 32.5% of the registered earthquakes during the 18 years of survey have been felt by the Groningen area citizens.

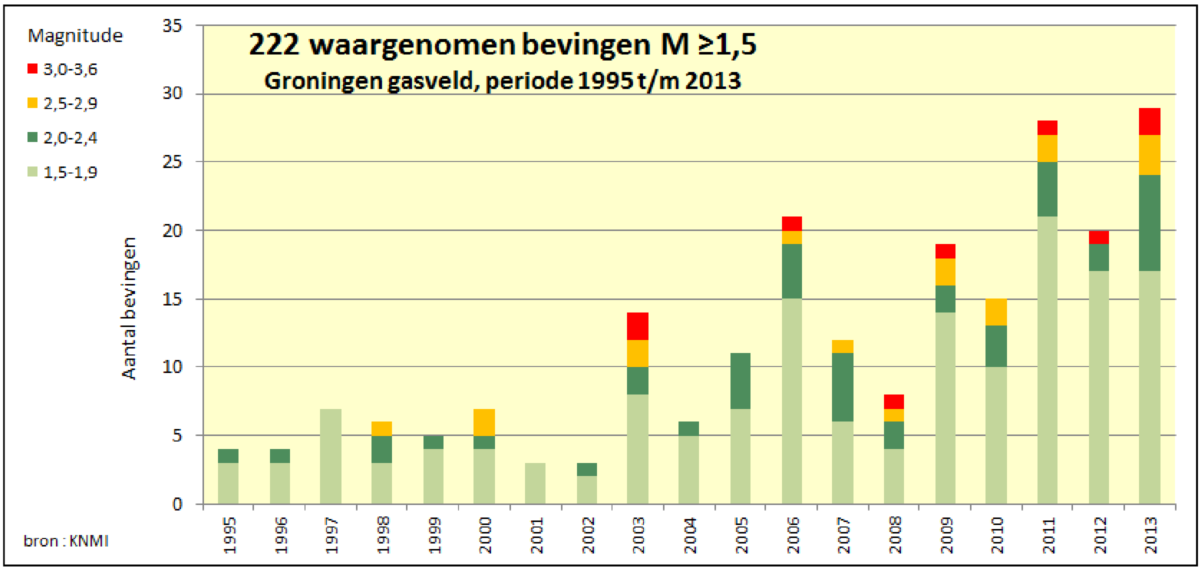


Figure 2-2 Registered earthquakes between 1995 and 2013 by KNMI

KNMI detected the earthquake epicentres during these years. Surface projections of the recorded earthquakes are placed in the northern area of the region, and mostly above the northern part of the gas reservoir.

2.3 Building database

The number of buildings and their relative locations has been furnished by the Basisregistratie Adressen en Gebouwen (BAG), the informations regarded the building typology, usage, value and year of construction have been collected.

The building database has been further completed assigned to the total number of the analysed buildings the construction material.

The most used construction material in the Groningen area is the unreinforced masonry, which composes the 77% of the total building, follow the buildings made of reinforced concrete that are the 4%. The wooden building are only the 0.2%, whereas the steel frame construction are around the 1%. The remaining 18% is composed by unclear buildings, which includes objects with unknown functions and building under construction.

2.4 Vulnerable Dwelling Typologies

Vulnerability tests on the Groningen house building stock show a damage of the 12% of the unreinforced masonry dwellings in the area in the scenario of a magnitude 5 earthquake located in Huizinge.
By means of a deeper elaboration of the data resulted by the estimation tests, it is possible to define two groups of URM house building typologies in relation to their seismic vulnerability. The vulnerability of the building sub-typologies is characterized by factors as: wall openness, wall type and building mass.

The more vulnerable house group comprises terraced houses and semi-detached houses. The original design process for these building typologies considers a collaboration between the several connected dwellings, especially for lateral loads acting in the direction parallel to the front and rear facades.

Designed to withstand (as lateral force) only the wind load, they present small shear walls in the direction of the longest sides. Consequently, resistance of shear walls in this direction might be not adequate to withstand seismic loads, even taking into account the collaboration between consecutive structures.

2.5 Weak Aspects Target of Retrofitting

Seismic deficiencies that characterize the URM house building can be described by aspects which make these dwelling prone to structural and non-structural element damages in case of seismic events.

- *Overall strength*: Every wall is characterized by a weak direction and a strong direction, the global strength of a masonry building is directly dependent to the in-plane shear capacity and out-of-plane bending capacity of structural walls. The ideal performance of a URM structure is the collaboration of load-bearing walls which if adequately tied together act as a box against lateral loads.
- *Overall stiffness:* URM bearing walls are generally characterized by small deformation due to the short period of vibration. However, perforated façades with relative slender piers confer flexibility to the building. Consequently, the presence of punctured façade in addition with strength lacking may enhance displacements and hence damages.
- *Mortar joints:* Poor mortar joints can represent an issue for the seismic performance of a masonry building. The joint composed by poor mortar materials or just old, causes disintegration of masonry wall units and loss of supports for horizontal element as floors and roof.
- *Irregularities in plan and in elevation:* Load bearing walls of masonry building shall follow a regular geometry in plan and in elevation. Rectangular or square plan shapes with load bearing elements arranged in plan respect to the two main axes are preferred for their seismic performance. Irregularities in plan can cause torsional moment due to the ground motion, and therefore concentration of stress occurs in correspondence of the connection.
- *Wall-diaphragm connection:* One of the most important issue for URM buildings is the connection between floor and roof diaphragms and walls. The horizontals forces, triggered by the seismic motion on the diaphragm level can cause a concentration of load in absence of an adequate bond between floor and walls.
- *Diaphragm deficiencies:* Diaphragms are essential for ensuring the transfer of lateral loads to vertical elements and for contributing to tie the building together. Timber floors could lack both strength and stiffness, which leads to an excessive flexibility of the diaphragm

and therefore to an inadequate distribution of the loads. When large displacements of these diaphragms are possible, wall damages might occur.

- *Foundation deficiencies:* Foundations are not considered to be one of the most critical aspect of the seismic behaviour for URM buildings. Anyway, it has been proved that buildings supported by shallow foundations suffer more than those with adequate deep foundations. Foundation could be susceptible to soil spreading and landslides having a detrimental effect on the URM structure. *Dogangun et al.* [2]

3 SEISMIC UPGRADING

This chapter briefly describes the meaning of seismic upgrading regarding its strategy, design approach and how it is regulated by international codes.

3.1 Upgrading Strategy

For the Groningen building stock measures of preliminary structural upgrading related to the weak aspects have been formulated. The measures have been thought in order to satisfy both life safety and damage mitigation. Particular attention has been given that during the realization process social disturbance will be minimized, and the final result will show the slightest possible aesthetic incongruences.

The upgrading measures for both structural and non-structural elements can be distinguished in two categories of intervention. Temporary upgrading measures are intended to be applied on the URM building stock which requires a rapid risk reduction. In the case of a high vulnerable building, an external stiffness upgrade, which supports laterally the building can be considered a short-term risk mitigation until a permanent measure has not been developed.

Permanent measures have been divided in seven levels ordered from 1 to 7 regarding the efficiency of the measure and the rapidity of completion respect to the risk mitigation, but also taking into account the inhabitant repercussion. The rising of the measure level aims to highlight the increment in complexity, duration and people impact.

- *Level 1, structural upgrading of falling hazards:* Reduce and abolish the risk of falling elements which are risky even for low level ground motion acceleration. The intervention can be also external to the building in order to minimize higher risk building elements
- *Level 2, tying of floors and walls:* The aim is to introduce a better distribution of load between load-bearing walls. Connection between wall and diaphragms improves the overall building robustness, and restrains walls from their out-of-plane displacements during the seismic event.
- *Level 3, stiffening of flexible diaphragms:* In order to transfer lateral load to walls in their in-plane direction, diaphragms need a certain stiffness. Stiff diaphragms guarantee an equal distribution of horizontal loads in bearing members running parallel to the seismic force. This kind of measures is the step forward after the level 2, which together enhance the overall building capacity by ensuring box-like action.

- *Level 4: Strengthening of existing walls in their out-of-plane direction:* URM walls are weak to restrain load in their out-of-plane direction. During a seismic event walls connected at diaphragm levels experience a horizontal load perpendicular to their faces. Consequently the member shall be strengthened in order to be able to withstand the solicitation due to the earthquake.
- *Level 5: Strengthening of existing walls in their in-plane direction:* When diaphragms are both adequately connected to the walls and stiff enough to transfer loads, the building assumes a favourable behaviour during the seismic event. If the capacity in the in-plane direction of the URM load-bearing walls is not sufficient, these shall be strengthened to limit damages.

3.2 Seismic Retrofit Design-Approach

A seismic retrofit for a generic building is the addition of one or more structural or not structural enhancements that provide an increase in stiffness or ductility to the existing structure.

During an earthquake elements or variations to existing members introduced by the retrofitting design will fulfil the deficiencies of the building to withstand seismic loads.

The seismic retrofit of a masonry building is generally based on one of two design approaches, or a combination of them:

- Damage Limitation Retrofit (DLR)
- Near Collapse Retrofit (NCR)

3.2.1 Damage Limitation Retrofit

It represents the conventional engineering approach of seismic retrofit. The design is based on the strengthening of the structure in its vulnerable aspects, such as connections, diaphragms flexibility and wall resistance.

The design approach assumes the elastic behaviour of the building, and focusing on different techniques aims to delay cracking. The retrofitting techniques should enhance the strength of structural elements, assuring the necessary resistance to withstand forces generated by the response of the building during ground motions. Generally, the required resistance is calculated on the design-level earthquake, while in case of major earthquakes, the additional energy will be assumed to be dissipated by the post elastic deformation of both materials and connections. In case of upgrading the out-of-plane capacity of a wall, for example, DLR approach would introduce vertical pre-stressed tie rods inserted into drilled holes. This will increase the axial load in the masonry wall element decreasing the formation of cracks during the seismic event.

3.2.2 Near Collapse Retrofit

NCR reduces the risk for severe structural damage and avoids the collapse after that post-elastic behaviour has occurred.

The design is focused on the overall performance of the structure by means of structural stability during the post-yielding phase. This approach requires a good understanding of the dynamic characteristics of the structure in order to develop adequate interventions that prevent severe damage or collapse.

In comparison to the previous example, regarding the lack of out-of-plane capacity of a URM wall, a NCR intervention would prevent the overturning of the element. This could be obtained by means of vertical non-prestressed rods which, introduced into drilled holes, come into play only when the structure has developed cracks and has displaced enough to engage the stabilizing elements. Measures for a NCR provide a reduced response of the building by increasing the structural damping via friction across the cracks and by lowering the response frequency due to the wall rocking.

The two design strategies are not intended to be mutually exclusive, contrary they can be complementary. The DLR approach addresses the elastic performance of the structure, while the NCR approach addresses the post-elastic behaviour.

The difference in the seismic damage behaviour of the two design strategies can be better understood by the graphic representation in the Figure 3-1, which refers to a generic masonry building.

In the graph the horizontal axis is an increasing function of earthquake intensity, whereas the vertical axis indicates the damage index.

The line represents different building performance:

- Line ABC depicts the performance of the building in its original condition
- Line DEF represents the damageability of the structure with a Damage Limitation Retrofit
- Line GHI represents the damageability of the structure with a Near Collapse Retrofit

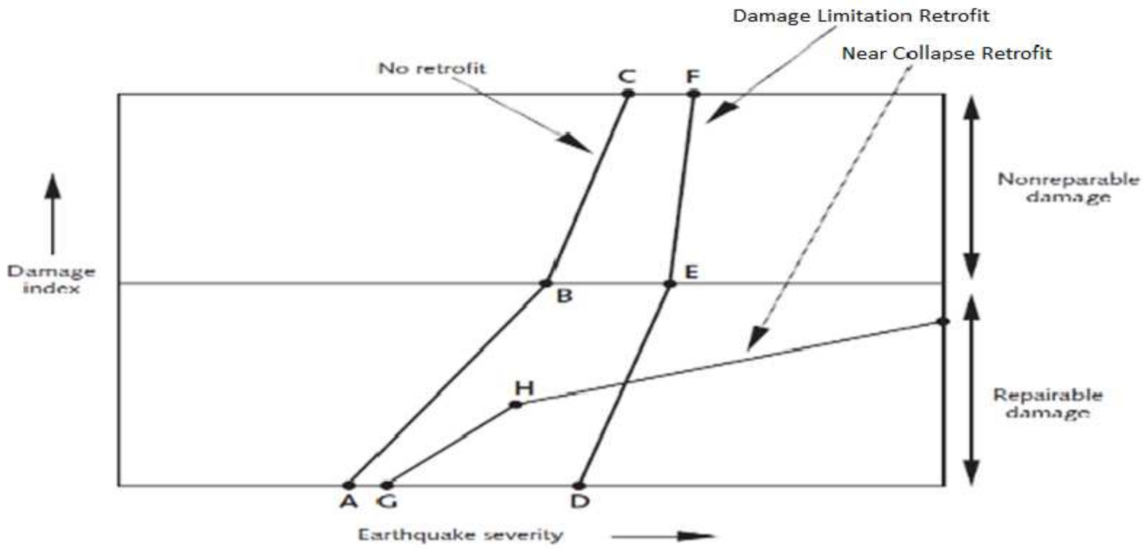


Figure 3-1Plot of damage-progression index versus earthquake severity for non-retrofitted structures (ABC) and for DL (GHI) and NC (DEF) retrofitted structures. Tolles [3].

Point A represents the threshold earthquake intensity for the damaging of the masonry building. When the point A is reached, the increase in the earthquake intensity will produce an increment of the reparable damage, which is limited by the point B. The line between B and C represents non-reparable damage, and it ends in point C with the collapse of the building.

A classic DLR has the effect to translate the threshold of initial damage. The distance between point A and D shows the improvement introduced by the upgrading of the building. Once the seismic intensity overruns the point D, the damage progresses follow the line until point E. It can be understood from the graph that, once the strengthened addition on structural elements and connections fail (line E-F), the behaviour of the structure as a whole becomes dominant. Collapse occurs on F, which is reached by a little increment in the seismic intensity respect to E.

In the case of a NC retrofitted structure, the point of initial damage G corresponds to a small increment of earthquake intensity respect to the non-retrofitted structure, point A. This is a consequence of the fact that no attempts have been done to avoid cracks. Indeed, this strategy adopts the no-linear behaviour as advance, focusing on displacement constraints, instead of strength improvements. The yielding of the material advances to point H, where the overall behaviour starts to dominate the performance. After the point H, the stabilization retrofit elements are activated, and the structure shows an increasing rate of repairable damage. The point I indicates the limit of relative high earthquake intensity for which the structure does not collapse.

In conclusion, while a DLR results in a better damage control at lower earthquake intensities, the NCR aims to life-safety and preventing collapse, Tolles [3].

3.3 Seismic Retrofitting Guidelines

Seismic codes aim to provide standards and published guidelines ensuring that in case of a seismic event, human are protected, damages are limited and relevance buildings remain operational. Indications on the codes are given for both new buildings and retrofitting designs in order to upgrade existing constructions.

3.3.1 Europe

In the European seismic zones the design of new buildings and the upgrading techniques for existing buildings are regulated by *Eurocode 8*[4] principally, with the addition of Eurocode 0 till 7 and 9.

Eurocode 8 provides a reduced regulation for low levels of PGA (peak ground accelerations), and in the case of very low PGA levels, it does not require any application of these guidelines.
When the URM building can be classified by parameters as "simple masonry building" only simple criteria need to be satisfied. The design criteria for simple masonry buildings regard building aspects such as plan geometry, wall configuration, minimal dimension and details.

3.3.2 Italy

The Italian regulation for the design of new structures and retrofitting of the existing buildings is described in the *NTC' 08*[5] (Norme Tecniche per le Costruzioni) with the addition of *CIRC'09* [6](Istruzione per l' applicazione delle NTC'08). The design of an URM building in seismic areas needs to respect guidelines given for the structural elements requirements. In addition to the regularity in plan and in elevation, the structure should satisfy conditions for diaphragm stiffness, diaphragm connection, wall dimensions and max height, respect to risk class zone. NTC' 08, as well as Eurocode, considers differently the case of "simple building". In the case of simple building no verifications are needed in the building design, but after the retrofitting procedure others conditions must be satisfied.

3.3.3 U.S.A.

In 2014 the American Society of Civil Engineers released the *ASCE 41-13*[7] named Seismic Evaluation and Retrofit of Existing Buildings. ASCE 41- 13 is a combination of two previous American standard guides, so called Seismic Evaluation of Existing Buildings (ASCE 31-03) and Seismic Rehabilitation of Existing Buildings (ASCE 41-06). The new ASCE 41-13 preserves the three tiers approach given by the ASCE 31-03, while the technical provisions have been taken from the 41-06 as the principles for analytical procedures.
ASCE 41-13 provides evaluation guidance for component capacity specifically for URM buildings and allows different ductility factors in relation to different failure modes. The first Tier consists in a screening of deficiencies to resist seismic action for an evaluation only. Tier 2 is the following step, which is intended for both evaluation and retrofit of the building. It leads the user to go over

the Tier 1 deficiencies with an appropriate upgrading design. The last Tier, the third, is named Systematic Evaluation. It gives the possibility to choose the analysis method between Linear Static, Linear Dynamic, Nonlinear Static and Nonlinear Dynamic.

3.3.4 New Zealand

The New Zealand Society for Earthquake Engineering (*NZSEE* [8]) published in 2006 a document so called "The Assessment and Improvement of the structural Performance of Buildings in Earthquakes" focused in the existing URM buildings guidance assessment. It also refers to structural upgrading methodologies without providing any design procedure for retrofitting. Two stages of evaluation are proposed in the document for the seismic assessment.
The first, the initial evaluation procedure, is a coarse screening involving reasonably resources, which aims to the identification of all those buildings that could be potentially earthquake prone. This step is supposed to be carried out by experienced earthquake engineers on the behalf of territorial local authorities and building owners.

The second stage is a detailed evaluation procedure for the assessment of the ULS level of existing buildings reached during earthquakes. It provides information, guidance and formulas in order to provide assistance in the evaluation of strength and ductility of structural components, elements and systems.

3.4 Adopted Guidelines for the Retrofitting Design

Retrofit designs of Groningen URM buildings should be based on Eurocode recommendations. Despite Eurocode 8 like the Italian code suggests guidelines for the seismic upgrading of masonry structures, it rarely recommends precise calculation procedures or empirical formulas for designing of retrofitting intervention. Consequently, it relies on the designer judgment for the determination of loads and deformations of particular cases.

Differently American and New Zealand codes define much more strictly procedures that a designer shall follow in the retrofitting of a masonry structures. These two codes suggest equations to determine solicitations and propose retrofitting techniques to solve the deficiencies of the building.

Consequently the most used codes in the calculation examples of this report are the American code Asce 41-13[7] and both Eurocodes 8[4] and 6[9]. Attention have been placed about the use of safety coefficients in both cases of solicitations and resistances, since different codes adopt different assumptions. However, resistances have been determined using procedures and relative coefficents suggested by Eurocodes, when information about design of solicitations were not present on the Eurocode, guidelines of Asce 41-13 have been followed.

Annex A lists load combinations suggested by the Eurocode and the American code, which have been adopted in this report.

4 LEVEL I RETROFITTING STRATEGY

This chapter explains the design approach and calculation procedure for upgrading of elements such as parapets and chimneys.

The first level of retrofitting strategy indicates permanent measures to reduce risk for non-structural building elements. Although these elements do not affect the structural behaviour of the building, they represent a hazard even for a low level of ground acceleration. Level 1, therefore aims to provide solutions for seismic upgrading for building members such as parapets and cantilever walls, chimneys and stacks.

In Eurocode 1998.1.1[4]is present a verification guideline for non-structural elements is present. It determines the effect of the seismic action defining the horizontal force F_a acting on the element by Eq. [4.1]

$$F_a = \frac{(S_a \cdot W_a \cdot \gamma_a)}{q_a} \quad [4.1]$$

Where:

F_a is the horizontal seismic force, acting at the centre of mass of the non-structural element in the most unfavourable direction

W_a is the weight of the element

S_a is the seismic coefficient applicable to the non-structural element

γ_a is the importance factor of the element (equal to 1 in case of no life safety risk)

q_a is the behaviour factor of the element tabled for non-structural element case

Type of non-structural element	q_a
Cantilevering parapets or ornamentations Signs and billboards Chimneys, masts and tanks on legs acting as unbraced cantilevers along more than one half of their total height	1,0
Exterior and interior walls Partitions and facades Chimneys, masts and tanks on legs acting as unbraced cantilevers along less than one half of their total height, or braced or guyed to the structure at or above their centre of mass Anchorage elements for permanent cabinets and book stacks supported by the floor Anchorage elements for false (suspended) ceilings and light fixtures	2,0

Table 4-1 EN 1998.1.1 [4] *Table 4.4: Values of q_a for non-structural elements*

The seismic coefficient S_a takes into account both ratios between height of the element and building height, and between fundamental period of the member (T_a) and of the building (T_1).

It shall be determined using Eq. [4.2] provided by Eurocode 8:

$$S_a = \alpha \cdot S \left[\frac{3\left(1+\frac{z}{H}\right)}{\left(1+\left(1-\frac{T_a}{T_1}\right)^2\right)} - 0.5 \right] \qquad [4.2]$$

Where:

α is the ratio of the design ground acceleration on type A ground, a_g, to the acceleration of gravity g

S is the soil factor

T_a is the fundamental vibration period of the non-structural element

T_1 is the fundamental vibration period of the building in the relevant direction

z is the height of the non-structural element above the level of application of the seismic action (foundation or top of a rigid basement)

H is the building height measured from the foundation or from the top of a rigid basement

Furthermore Eurocode 8 advises that the seismic coefficient S_a may not be taken less than αS.

To determine the fundamental vibration period for non-structural element T_a, the Eurocode does not suggest any procedure.

Differently in the American code ASCE 41-13 [7] a simple analytical approach to determine T_a is proposed. In section 13.4.3.1 concerning the Horizontal Seismic Force for non-structural elements, it is suggested to estimate the fundamental period T_a (on ASCE 41-13 so-called T_p) by Eq. [4.3].

$$T_a = 2\pi\sqrt{\frac{W_p}{K_p g}} \tag{4.3}$$

Where

W_p is the operating weight of the element

g gravitational acceleration

K_p Approximate stiffness of the support system of the component, its bracing, and its attachment, determined in terms of load per unit deflection at the centre of gravity of the component.

Plugging both definitions of seismic coefficient S_a (Eq.[4.2]) and of fundamental vibration period for non-structural element T_a (Eq. [4.3]) into the seismic horizontal force equation, follows:

$$F_a = \frac{\alpha \cdot S \cdot W_a \cdot \gamma_a}{q_a}\left[\frac{3\left(1+\frac{z}{H}\right)}{\left(1+\left(1-\frac{2\pi\sqrt{\frac{W_p}{K_p g}}}{T_1}\right)^2\right)} - 0.5\right] \tag{4.4}$$

Therefore, the retrofitting solution to reduce the effect of the horizontal load acting on the member may be based on one of two approaches:

- Lowering of the operative weight of the element W_p
- Increasing the stiffness of the element supporting system K_p

Two of the most complete and update guidelines for this type of seismic retrofitting has been thought to be the FEMA E-74 [10] and FEMA 547 [11]. The American guideline FEMA E-74 named *Reducing the Risk of Non-Structural Earthquake Damage-A Practical Guide*, upgraded in the 2011, provides practical solutions for the retrofitting of non-structural building parts. Whereas FEMA-547 named *Techniques for the Seismic Rehabilitation of Existing Buildings* treats both structural and non-structural elements.

A selection of the practical solutions presented in the FEMA E-74 and FEMA-547 has been collected on the basis of the Groningen case. Indeed, the non-structural elements taken into consideration are building parts which are considered to be generally present in the URM building stock situated in the Groningen area.

4.1 Parapets and cantilever walls

Unreinforced masonry parapets due to their significant falling hazard represent causes of injuries and expensive repairs in case of seismic events. The inadequate bending strength and ductility makes these elements the first parts of the building to be damaged. In case of no rigid roof diaphragm, parapets show the greater damage at mid-span of diaphragm due to the higher accelerations and displacements. Whereas, when the roof diaphragm is adequately rigid, the parapet performs equally along the entire span.

URM parapets are considered causes of further damage in case of seismic event due to their typical configuration.

Heavy and unbraced parapets placed at the roofline might involve failure of parts of walls caused by their out-of-plane collapse.
The out-of-plane failure of a parapet may occur either inwards or outwards. In case of inwards falling, it might involve further damage of the building roof. On the other hand, outwards falling could include damage of adjacent property.

4.1.1 Description of the proposed rehabilitation techniques

International guidelines propose as most effective techniques three kinds of approaches to minimize the falling hazard risk:

- Removing of the parapet
- Replacement of the parapet by a light weight solution
- Anchoring of the parapet to the roof by steel bracing

Remove the whole parapet or reduce its height means a decrease of vertical compressive stress at the roof-to-wall anchor locations. Typically when the intention is to remove the element, this is then replaced by a concrete cap or a bond beam to ensure the anchorage of the diaphragm. The

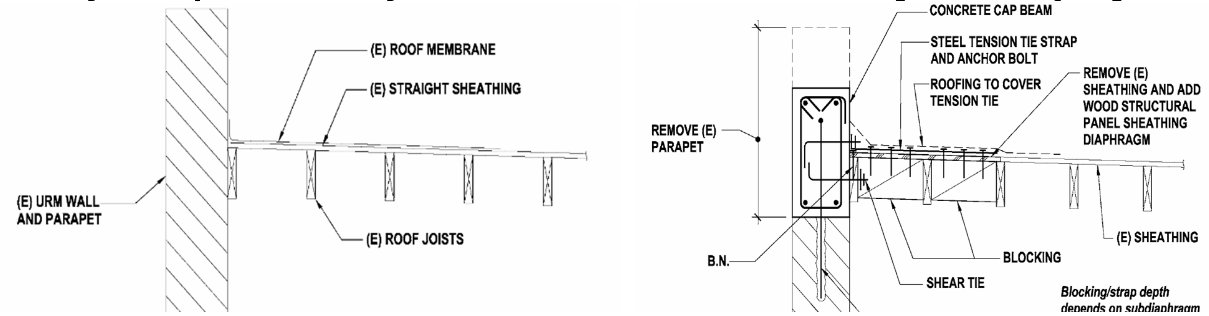


Figure 4-1 Parapet removal and Concrete Cap Beam for the rehabilitated condition FEMA 547 [11].

original reason of extending the masonry walls up forming parapets was to limit spreading of fire. Therefore, if the parapet is removed, precautions shall be taken for fire protection of adjacent building.

Due to the fact that one primary condition for retrofitting is to preserve the architectural aspect of the building, replace the parapet by a light weight element is not considered.
The mitigation solution by the installation of bracing represents the suggested technique in this report.

In order to minimize solicitations due to horizontal seismic loads, the parapet is anchored. Respect to the original configuration of a cantilever beam, when braced, the parapet behaves a doubly supported beam. As it can be seen from Figure 4-2, the bracing is usually realized by a steel angle brace anchored near the top face of the parapet and to the roof framing. The present roof framing may need a localized strengthening to withstand the reaction from the brace. Whereas in the low part, the parapet is restrained by the roof-to-wall tension anchor.

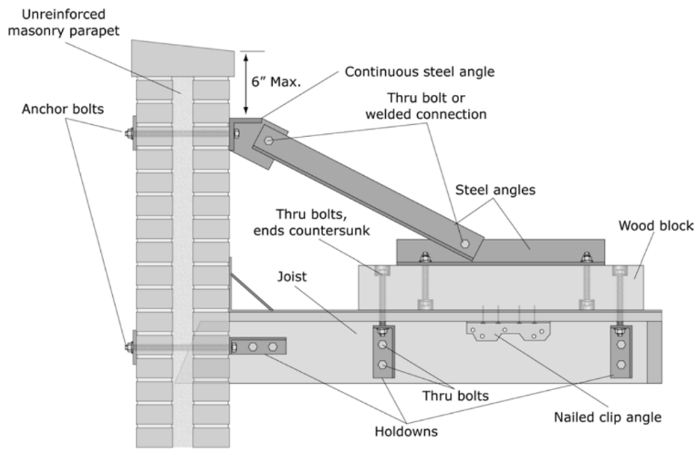


Figure 4-2 Typical brace configuration for URM parapet FEMA E-74 [10]

4.1.2 Detailing and construction considerations

Parapet and roof configuration may vary for different cases. Different roof slope with diverse parapet sizes involve small variations respect to the details.

- *Parapet anchorage types:* A continuous steel angle profile is placed horizontally along the parapet at the location where the brace is supposed to be connected. The profile can be fixed by means of anchor bolts or invisible adhesive anchors. The horizontal profile will provide a uniform distribution of stress introduced by the bracing.
- *Top angle:* It is normally assumed a continuous angle running between braces in the roof.
- *Roof framing modification:* Vertical loads acting on the base connection of the brace is typically hold by the roof framing. Especially for tall parapets, the existing roof joists may

defect to resist the substantial brace loads. In these cases the stiffness at the base of the brace workpoint should be increased. This can be obtained by additional joists, by the application of more bracing to distribute the load or by adding blocking beneath the base of the workpoint as shown in the Figure 4-3. When a stiffening of the diaphragm is planned, a concrete topping overlay may represent the anchorage for the brace workpoint.

- *Waterproofing:* The brace anchor at the roof workpoint is attached to the structural framing of the roof, so a penetration of the waterproof layer will occur. Therefore waterproof solution must be provided for any roof connection.

4.1.3 Cost, disruption and aesthetic modification:

Since the anchorage of parapets by bracing involves mostly external modification, disruption is relatively low, occupants can remain in place. Combine the parapet retrofitting with roof-to-wall ties rehabilitation can reduce the total cost of the intervention. The anchorage of the parapet will not result externally visible and no substantial aesthetic modification of the façade will occur.

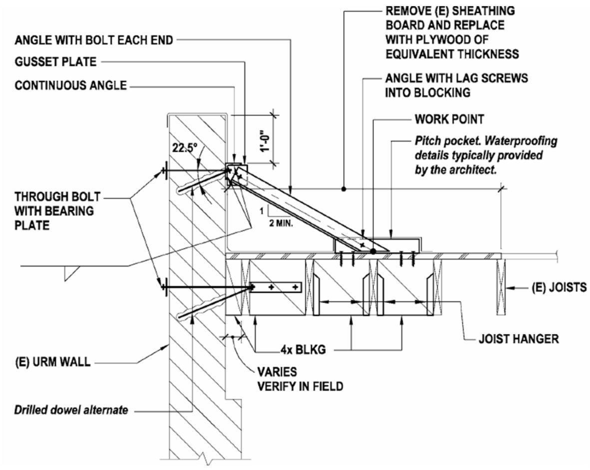


Figure 4-3 Parapet bracing example FEMA 547 ***[4]***

4.2 Parapet seismic upgrading example calculation

In the following calculation example, the parapet of a Dutch terraced house has been selected. Figure 4-4 displays plans views of the building with the parapet highlighted by the colour red.

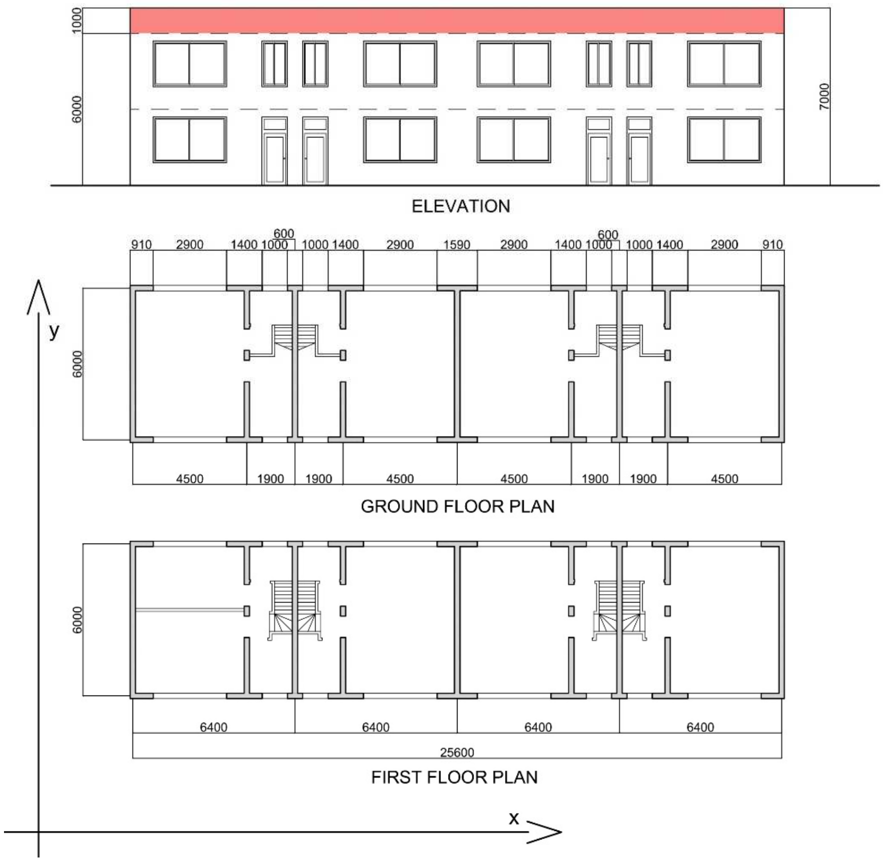


Figure 4-4 Dutch Terraced House with the Selected Parapet

4.2.1 Parapet seismic load calculation

In order to determine seismic loads acting on the parapet, the fundamental period of vibration of the building has been estimated. Periods in the two directions have been determined by means of the Rayleigh's method, calculation is shown in Section 6.6.4.

BUILDING	Building height	$H := 6000\ mm$
	Building fundamental period in the relevant direction	$T_{1.x} := 0.143\ s$ $T_{1.y} := 0.044\ s$

Figure 4-5 shows the top view of the terraced house with the indexes assigned to the analysed parapets depending on their orientation, and the seismic loads acting on the elements.

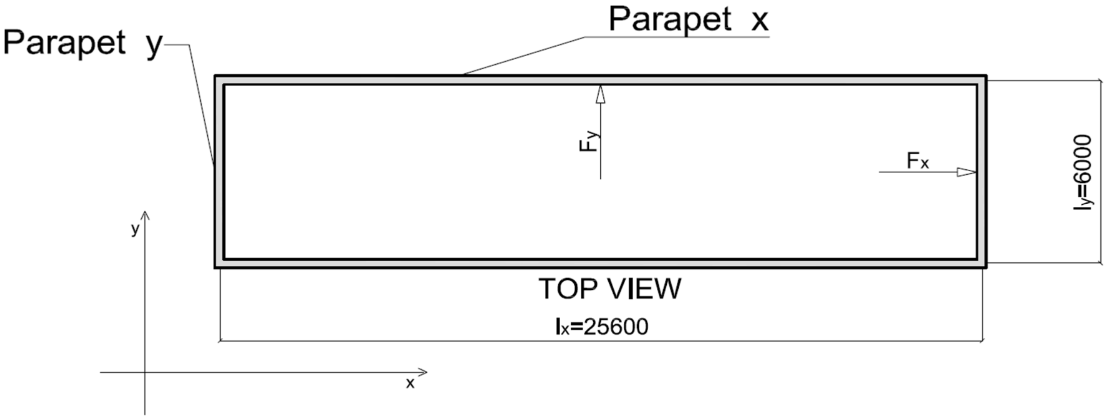


Figure 4-5 Building Top View with Parapet Reference Index used in the Design

PARAPET

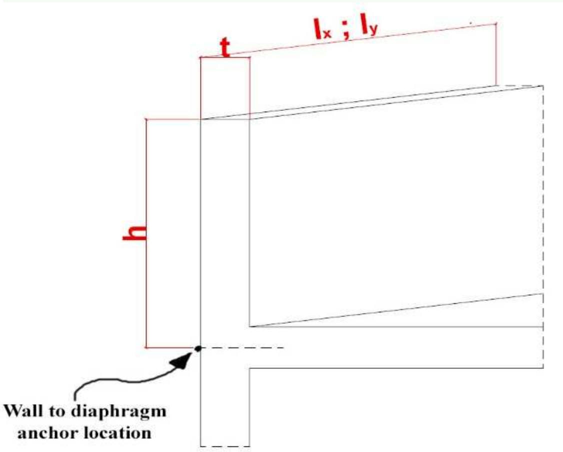


Figure 4-6 Parapet Geometry

Thickness $t := 210\ mm$

Height $h := 1000\ mm$

Length $l_x := 25600\ mm$
$l_y := 6000\ mm$

Height-to-thickness ratio: $\frac{h}{t} = 4.76$

Height of the parapet from foundation level $z := 7000\ mm$

MASONRY

	Modulus of Elasticity	$E_m := 4410\ MPa$
	Compressive strength	$f_{ck} := 4.4\ MPa$
MASONRY	Bending strength parallel to the bed joint	$f_{xk.1} := 0.2\ MPa$
	Bending strength perpendicular to the bed joint	$f_{xk.2} := 0.4\ MPa$
	Partial factor	$\gamma_m := 1.7$
	Weight (clay bricks)	$\rho_m := 18\ \frac{kN}{m^3}$

The bracing system to restrain the parapet is designed to be composed by an L profile running along the parapet and placed at h_L below the top face of the wall. Vertically, the parapet is restrained by a series of L profiles which are connected to supports on the roof by means of L profiles. L profiles which connect parapet and roof are considered to be hinged at both ends. The described solution is shown in Figure 4-7.

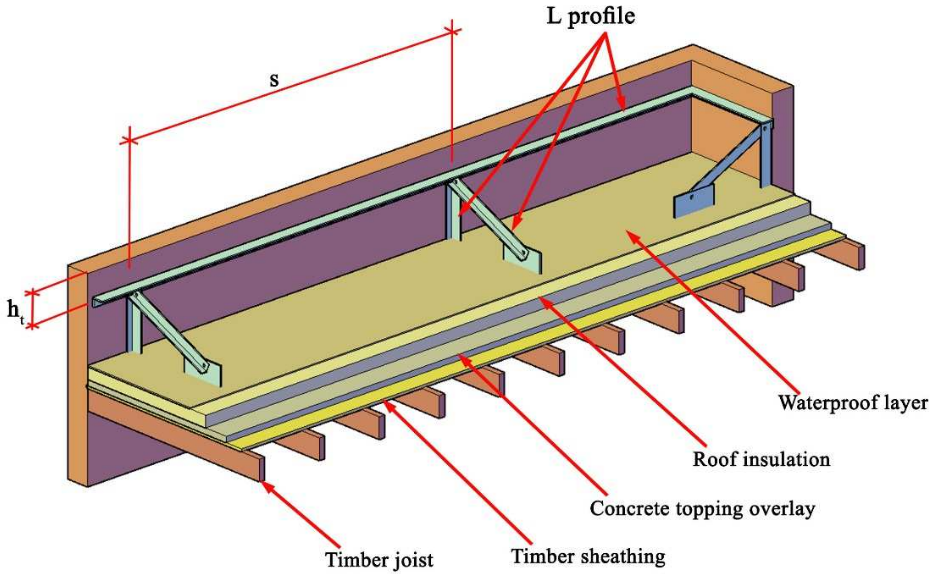


Figure 4-7 Parapet Bracing Configuration

The parapet without seismic upgrade may be approximated by means of an uniformly loaded cantilever beam when subjected to the seismic load. However, the bracing introduced by the retrofit changes the boundary conditions of the element. The restrained parapet shall be represented by a doubly supported beam.

The element, as displayed in Figure 4-8 , is assumed to be composed by two parts. Part 1 behaves as a beam clamped at one end and supported on the other end, with span equal to h-h_L.
Whereas, Part 2 is approximated by a cantilever beam with length equal to h_L.

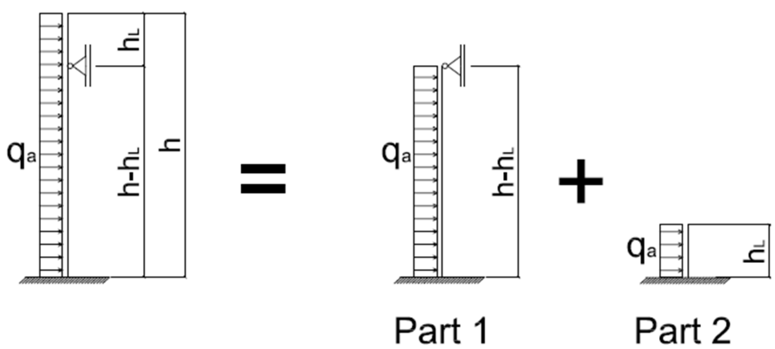


Figure 4-8 Assumptions for the Analysis of the Parapet

Distance between L profile and parapet top face		$h_L := 200\ mm$
Parapet stiffness part 1 (considered fixed at both ends)	x direction	$K_{p1.x} := \dfrac{E_m \cdot I_x}{4 \cdot \left(\dfrac{h-h_L}{t}\right)^3} = \left(5.11 \cdot 10^5\right)\ \dfrac{kN}{m}$
	y direction	$K_{p1.y} := \dfrac{E_m \cdot I_y}{4 \cdot \left(\dfrac{h-h_L}{t}\right)^3} = \left(1.2 \cdot 10^5\right)\ \dfrac{kN}{m}$
Parapet stiffness part 2 (considered as a cantilever)	x direction	$K_{p2.x} := \dfrac{4 \cdot E_m \cdot I_x}{\left(\dfrac{h_L}{t}\right)^3} = \left(5.23 \cdot 10^8\right)\ \dfrac{kN}{m}$
	ydirection	$K_{p2.y} := \dfrac{4 \cdot E_m \cdot I_y}{\left(\dfrac{h_L}{t}\right)^3} = \left(1.23 \cdot 10^8\right)\ \dfrac{kN}{m}$
Parapet fundamental period part 1	x direction	$T_{a1.x} := 2 \cdot \pi \cdot \sqrt[2]{\dfrac{\left(\rho_m \cdot t \cdot h \cdot l_x\right)}{K_{p1.x} \cdot g}} = 0.028\ s$
	y direction	$T_{a1.y} := 2 \cdot \pi \cdot \sqrt[2]{\dfrac{\left(\rho_m \cdot t \cdot h \cdot l_y\right)}{K_{p1.y} \cdot g}} = 0.028\ s$
Parapet fundamental period part 2	x direction	$T_{a2.x} := 2 \cdot \pi \cdot \sqrt[2]{\dfrac{\left(\rho_m \cdot t \cdot h \cdot l_x\right)}{K_{p2.x} \cdot g}} = 0.001\ s$
	y direction	$T_{a2.y} := 2 \cdot \pi \cdot \sqrt[2]{\dfrac{\left(\rho_m \cdot t \cdot h \cdot l_y\right)}{K_{p2.y} \cdot g}} = 0.001\ s$

Seismic hazard data for design purpose have been taken from preliminary study conducted by ARUP[12] and described in the report: Groningen 2013 – Structural Upgrading Study – Section 2.2 Seismic Evaluation.

Design ground acceleration	$a_g := 0.243\ g$
Ground type E, Soil factor	$S := 1.6$

Seismic coefficient Sa part 1

x direction $$S_{a1.x} := \frac{a_g}{g} \cdot S \cdot \left(\frac{3 \cdot \left(1+\frac{z}{H}\right)}{\left(1+\left(1-\frac{T_{a1.x}}{T_{1.x}}\right)^2\right)} - 0.5 \right) = 1.34$$

y direction $$S_{a1.y} := \frac{a_g}{g} \cdot S \cdot \left(\frac{3 \cdot \left(1+\frac{z}{H}\right)}{\left(1+\left(1-\frac{T_{a1.y}}{T_{1.y}}\right)^2\right)} - 0.5 \right) = 2.03$$

Seismic coefficient Sa part 2

x direction $$S_{a2.x} := \frac{a_g}{g} \cdot S \cdot \left(\frac{3 \cdot \left(1+\frac{z}{H}\right)}{\left(1+\left(1-\frac{T_{a2.x}}{T_{1.x}}\right)^2\right)} - 0.5 \right) = 1.08$$

y direction $$S_{a2.y} := \frac{a_g}{g} \cdot S \cdot \left(\frac{3 \cdot \left(1+\frac{z}{H}\right)}{\left(1+\left(1-\frac{T_{a2.y}}{T_{1.y}}\right)^2\right)} - 0.5 \right) = 1.09$$

Behaviour factor of the non-structural element $q_a := 1.0$
Importance factor $\gamma_a := 1.0$

Horizontal seismic force acting at the centre of mass of part 1 along the whole parapet

x direction $$F_{a1.x} := \frac{S_{a1.x} \cdot (\rho_m \cdot t \cdot (h - h_L) \cdot l_y) \cdot \gamma_a}{q_a} = 24\ kN$$

y direction $$F_{a1.y} := \frac{S_{a1.y} \cdot (\rho_m \cdot t \cdot (h - h_L) \cdot l_x) \cdot \gamma_a}{q_a} = 157\ kN$$

Horizontal seismic force acting at the centre of mass of part 2 along the whole parapet

x direction $$F_{a2.x} := \frac{S_{a2.x} \cdot (\rho_m \cdot t \cdot h_L \cdot l_y) \cdot \gamma_a}{q_a} = 5\ kN$$

y direction $$F_{a2.y} := \frac{S_{a2.y} \cdot (\rho_m \cdot t \cdot h_L \cdot l_x) \cdot \gamma_a}{q_a} = 21\ kN$$

Horizontal seismic forces can be represented by means of equally distributed loads acting along the parapet:

Part 1 $$q_{a1.x} := \frac{F_{a1.x}}{l_y \cdot (h - h_L)} = 5.05\ \frac{kN}{m^2} \qquad q_{a1.y} := \frac{F_{a1.y}}{l_x \cdot (h - h_L)} = 7.66\ \frac{kN}{m^2}$$

Part 2 $$q_{a2.x} := \frac{F_{a2.x}}{l_y \cdot h_L} = 4.07\ \frac{kN}{m^2} \qquad q_{a2.y} := \frac{F_{a2.y}}{l_x \cdot h_L} = 4.14\ \frac{kN}{m^2}$$

4.2.2 Check of the Parapet Resistance

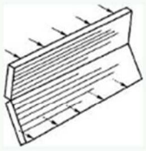
Check of the parapet running in the x direction

The resistance of the parapet shall be checked for both parts. Firstly part 1 is analyzed. This part may be considered supported on its four boundaries as shown in Figure 4-9. Hence, the resistance of this part has been determined for both its possible failure mechanism. In order to proceed a number of braces has been assumed.

Number of bracings $n_x := 20$

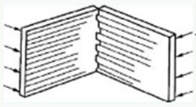
Bracing spacing $s_x := \frac{l_x}{n_x} = 1280\ mm$

Design bending moment resistance when the plane of failure is parallel to the bed joint



$$M_{Rd1.x1} := \left(\frac{f_{xk.1}}{\gamma_m}\right) \cdot \left(\frac{s_x \cdot t^2}{6}\right) = 1.11\ kN \cdot m$$

Design bending moment resistance when the plane of failure is perpendicular to the bed joint



$$M_{Rd1.x2} := \left(\frac{f_{xk.2}}{\gamma_m}\right) \cdot \left(\left(\frac{(h - h_L) \cdot t^2}{6}\right)\right) = 1.38\ kN \cdot m$$

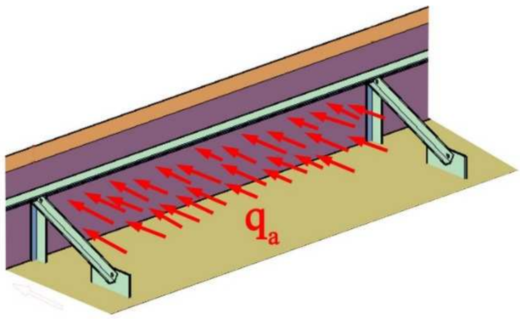


Figure 4-9 Seismic Load Acting in the Area Enclosed between two Contiguous Braces

The portion of parapet enclosed between two contiguous bracing (part 1) shall be considered as simply supported on each side. The out-of-plane resistance of this portion of parapet against the failure along the plane parallel to the bed joint may be checked using the yield line theory presented in Eurocode 6.

By the ratio h/l (where l is the spacing between braces) others parameters can be found on Annex E of Eurocode 6.

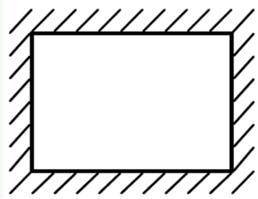


Figure 4-10 Wall Support Condition

h/l ratio $\frac{h - h_L}{s_x} = 0.625$

Orthogonal ratio $\mu := \frac{f_{xk.1}}{f_{xk.2}} = 0.5$

Bending moment coefficient $\alpha := 0.036$

The maximum allowable spacing between two contiguous is governed by the resistance of the parapet when the plane of failure is perpendicular to the bed joint. Therefore, the spacing before assumed shall be checked using the yield line theory, which determines the upper bound s_{max}.

$$s_{x.max} := \sqrt{\frac{M_{Rd1.x2}}{\mu \cdot \alpha \cdot q_{a1.y} \cdot (h - h_L)}} = 3543\ mm$$

where the assumed spacing is $s_x = 1280\ mm$

Sollecitations for both cases of failure shall be derived by means of the yield line theory:

$$M_{Ed1.x1} := \mu \cdot \alpha \cdot q_{a1.y} \cdot s_x^{\ 3} = 0.29\ kN \cdot m$$

Where

$$M_{Ed1.x2} := \alpha \cdot q_{a1.y} \cdot s_x^{\ 2} \cdot (h - h_L) = 0.36\ kN \cdot m$$

MEd1,x1 < MRd1.x1

MEd1,x2 < MRd1.x2

As before mentioned, part 2 of the element shall be considered as a cantilever beam subjected to a distribute load. Therefore the resistance of this portion of parapet has been checked considering the entire length of the element l_x.

$$M_{Rd2.x1} := \left(\frac{f_{xk.1}}{\gamma_m}\right) \cdot \left(\frac{l_x \cdot t^2}{6}\right) = 22.14\ kN \cdot m$$

$$M_{ed.2.x1} := \frac{F_{a2.y} \cdot h_L}{2} = 2.12\ kN \cdot m$$

MEd2,x1 < MRd2.x1

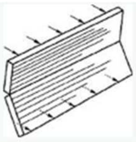
Check of the parapet running in the y direction

Consequently, the procedure carried out to design the bracing for the parapet running in the x direction has been repeated for the bracing of the parapet along the y direction.

Distance between L profile and parapet top face	$h_L := 200\ mm$
Number of bracings	$n_y := 6$
Bracing spacing	$s_y := \frac{l_y}{n_y} = 1000\ mm$

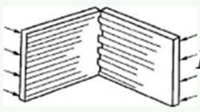
Similarly as for the parapet running in the x direction, the resistance of the element has been checked for two possible failure mechanisms.

Design bending moment resistance when the plane of failure is parallel to the bed joint



$$M_{Rd1.y1} := \left(\frac{f_{xk.1}}{\gamma_m}\right) \cdot \left(\frac{s_y \cdot t^2}{6}\right) = 0.86\ kN \cdot m$$

Design bending moment resistance when the plane of failure is perpendicular to the bed joint



$$M_{Rd1.y2} := \left(\frac{f_{xk.2}}{\gamma_m}\right) \cdot \left(\left(\frac{(h - h_L) \cdot t^2}{6}\right)\right) = 1.38\ kN \cdot m$$

The resistance for the portion of wall spanning between two contiguous supports is verified by defining the maximum spacing:

h/l ratio $\frac{h}{s_y}=1$

Orthogonal ratio $\mu := \frac{f_{xk.1}}{f_{xk.2}}=0.5$

Bending moment coefficient $\alpha := 0.048$

$$s_{y.max} := \sqrt{\frac{M_{Rd1.y2}}{\mu \cdot \alpha \cdot q_{a1.x} \cdot (h-h_L)}} = 3777\ \boldsymbol{mm}$$

where the assumed spacing is $s_y = 1000\ \boldsymbol{mm}$

$$M_{Ed1.y1} := \mu \cdot \alpha \cdot q_{a1.x} \cdot {s_y}^3 = 0.12\ \boldsymbol{kN \cdot m}$$

MEd1,y1 < MRd1.y1

Where

$$M_{Ed1.y2} := \alpha \cdot q_{a1.x} \cdot {s_y}^2 \cdot (h-h_L) = 0.19\ \boldsymbol{kN \cdot m}$$

MEd1,y2 < MRd1.y2

Then the bending moment acting on the basis of part 2 is checked

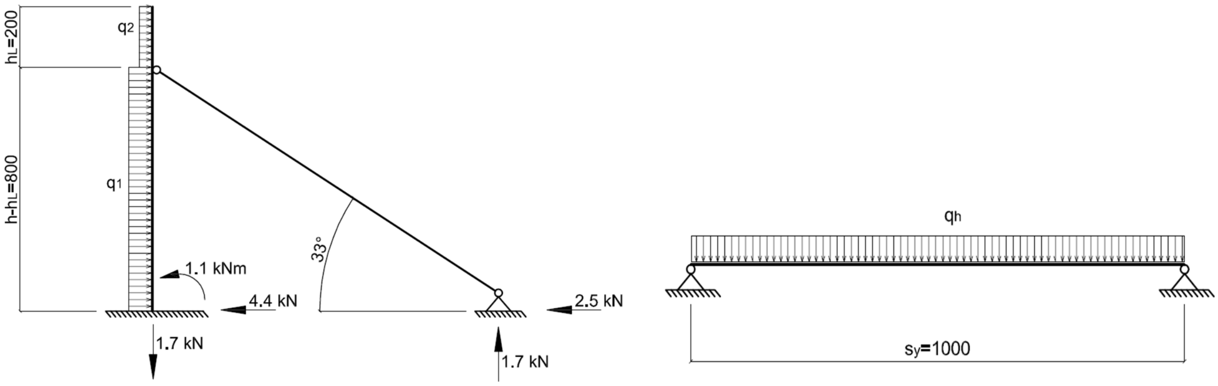
$$M_{Rd2.y1} := \left(\frac{f_{xk.1}}{\gamma_m}\right) \cdot \left(\frac{l_y \cdot t^2}{6}\right) = 5.19\ \boldsymbol{kN \cdot m}$$

MEd2,y1 < MRd2.y1

$$M_{ed.2.x1} := \frac{F_{a2.x} \cdot h_L}{2} = 0.49\ \boldsymbol{kN \cdot m}$$

4.3 Parapet Bracing Design

Bracing of the parapet is composed by a system of steel profiles as shown in Figure 4-7. The steel brace is designed to be inclined of 33° respect to the horizontal and it is assumed to be hinged at both ends. Load acting on the bracing have been determined on the basis of the greater load condition, which is respect to the y direction.



Total height of the parapet	$h := 1000\ mm$
Distance between L profile and parapet top face	$h_L := 200\ mm$
Brace spacing of the parapet running in the y direction	$s_y := 1000\ mm$
Load acting on part 1 on the parapet running in the y direction	$q_{a1.y} = 7.66\ \frac{kN}{m^2}$
Load acting on part 2 on the parapet running in the y direction	$q_{a2.y} = 4.14\ \frac{kN}{m^2}$
Distributed load acting on part 1	$q_1 := q_{a1.y} \cdot s_y = 7.66\ \frac{kN}{m}$
Distributed load acting on part 2	$q_2 := q_{a2.y} \cdot s_y = 4.14\ \frac{kN}{m}$
Distributed load acting on the horizontal steel profile	$q_h := q_{a1.y} \cdot \left(\frac{h - h_L}{2} + \frac{h_L}{2}\right) = 3.83\ \frac{kN}{m}$

Material properties of steel profiles:

Yield strength of chosen steel	$f_y := 355\ MPa$
Young modulus of steel	$E_s := 210000\ MPa$
Safety factor for steel	$\gamma_{M0} := 1.00$

In the previous design of the parapet bracing, the restrain system has been considered to be infinitive stiff, due to the fact that no stiffness has been accounted. However, an infinitive stiff bracing means large and heavy cross section of the elements. Hence in order to satisfy the boundary conditions set for the previous calculation only a small deformation of the bracing has been allowed, where this deformation has been set of 0.1mm. Consequently, all three steel elements have been designed for both resistance and deflection conditions.

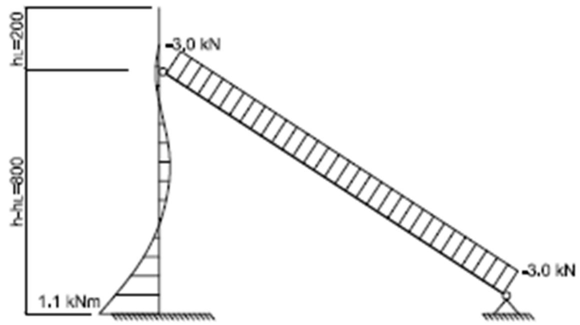


Figure 4-12 Bending Moment on the Vertical Element and Axial Load on the Brace

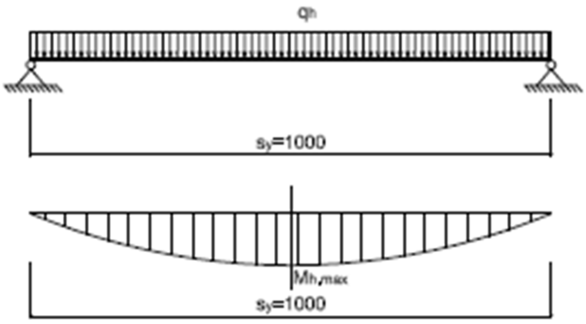


Figure 4-11 Bending Moment on the Horizontal Element

Vertical L profile

Calculation of the minimum plastic modulus of the profile due to the bending moment:

Max bending moment acting on the vertical steel element $M_{v.max} := 1.1\ kN \cdot m$

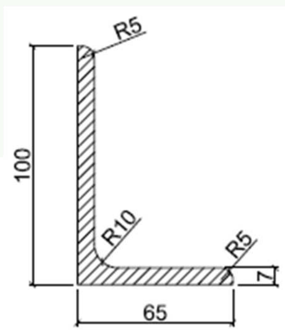
Minimum profile plastic modulus based on the resistance of the element $W_{pl.min} := \dfrac{M_{v.max} \cdot \gamma_{M0}}{f_y} = 3.1\ cm^3$

The L profile is assumed to behave as a beam fixed at one end and supported on the other subjected by a uniformly distributed loaded. The maximum deflection is at 0.4422 L.

Maximum deflection at 0.4422(h-hl) $\delta_{v.max} := \dfrac{q_1 \cdot (h - h_L)^4}{185\ E_s I_v}$

Minimum second moment area of L profile to assure a maximum deflection of 0.1 mm. $I_{v.min} := \dfrac{q_1 \cdot (h - h_L)^4}{185 \cdot E_s \cdot 0.1\ mm} = 80.71\ cm^4$

Chosen profile designation 100x65x7:



Section properties: $I_v := 112.5\ cm^4$

$W_{el} := 16.61\ cm^3$

Figure 4-13 L Steel Profile 100x65x7

Horizontal L profile

The L profile running along the parapet is assumed to behave as a beam double supported by adjacent braces subjected by a uniformly distributed loaded. The design of the minimum plastic modulus of the profile is based on the maximum bending moment at mid-span.

Max bending moment acting on the horizontal steel element $\quad M_{h.max} := \frac{q_h \cdot s_y^2}{8} = 0.48\ kN \cdot m$

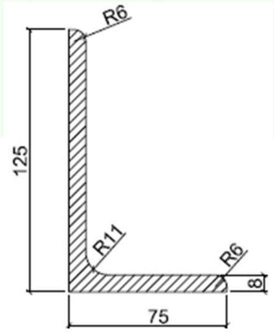
Minimum profile plastic modulus based on the resistance of the element $\quad W_{pl.min} := \frac{M_{h.max} \cdot \gamma_{M0}}{f_y} = 1.35\ cm^3$

Using the same assumption of the boundary condition, the maximum deflection at mid-span may be determined.

Maximum deflection at sy/2 $\quad \delta_{max} := \frac{5}{384} \frac{q_h \cdot s_y^4}{E_s I_h}$

Minimum second moment area of L profile to assure a maximum deflection of 0.1 mm. $\quad I_{h.min} := \frac{5}{384} \frac{q_h \cdot s_y^4}{E_s \cdot 0.1\ mm} = 237\ cm^4$

Chosen profile designation 125x75x7:



Section properties: $\quad I_h := 274.3\ cm^4 \qquad W_{el} := 28.57\ cm^3$

Figure 4-14 L Steel Profile 125x75x8

Brace L profile

The brace is what supports both horizontal and vertical elements. The member is considered hinged at both ends, consequently, the design is based on the axial load which the element is subjected.

Angle of the L profile $\quad \alpha := 33 \qquad \alpha := \pi \cdot \frac{\alpha}{180} = 0.58$

Max axial force acting along the brace L profile $\quad N_{b.max} := 3\ kN$

Minimum area of the profile based on the resistance of the element $\quad A_{b.min} := \frac{N_{b.max} \cdot \gamma_{M0}}{f_y} = 8.45\ mm^2$

Similarly as for the other bracing elements, a small deflection has been set in order to satisfy the boundary condition of stiffness used in the parapet retrofitting calculation.

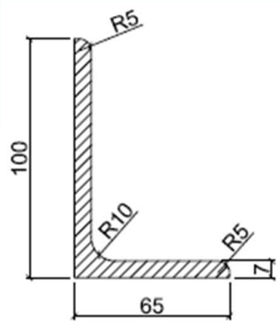
Maximum displacement of the steel profile along its axis

$$\delta_{max} := \frac{N_{b.max} \cdot l}{E_s A_b}$$

Minimum area of the profile based on the displacement of the element

$$A_{b.min} := \frac{N_{b.max} \cdot \dfrac{h - h_L}{\cos(\alpha)}}{E_s \cdot 0.1\ mm} = 114\ mm^2$$

Chosen profile designation 100x65x7:



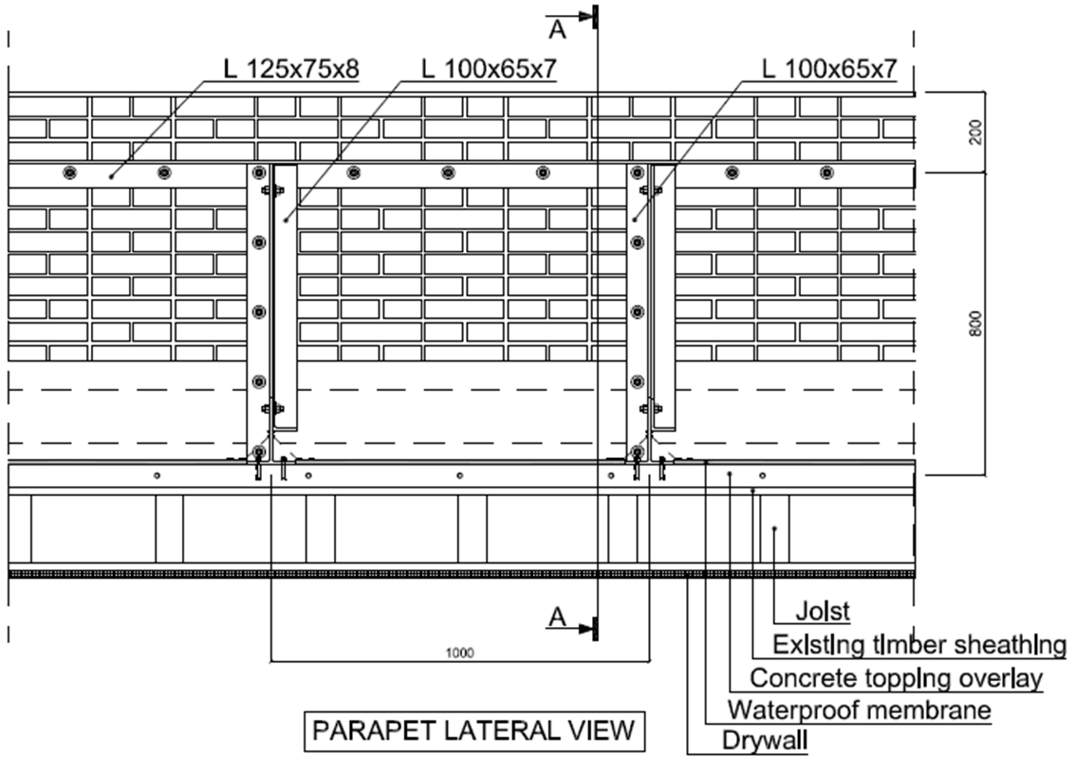
Section properties:

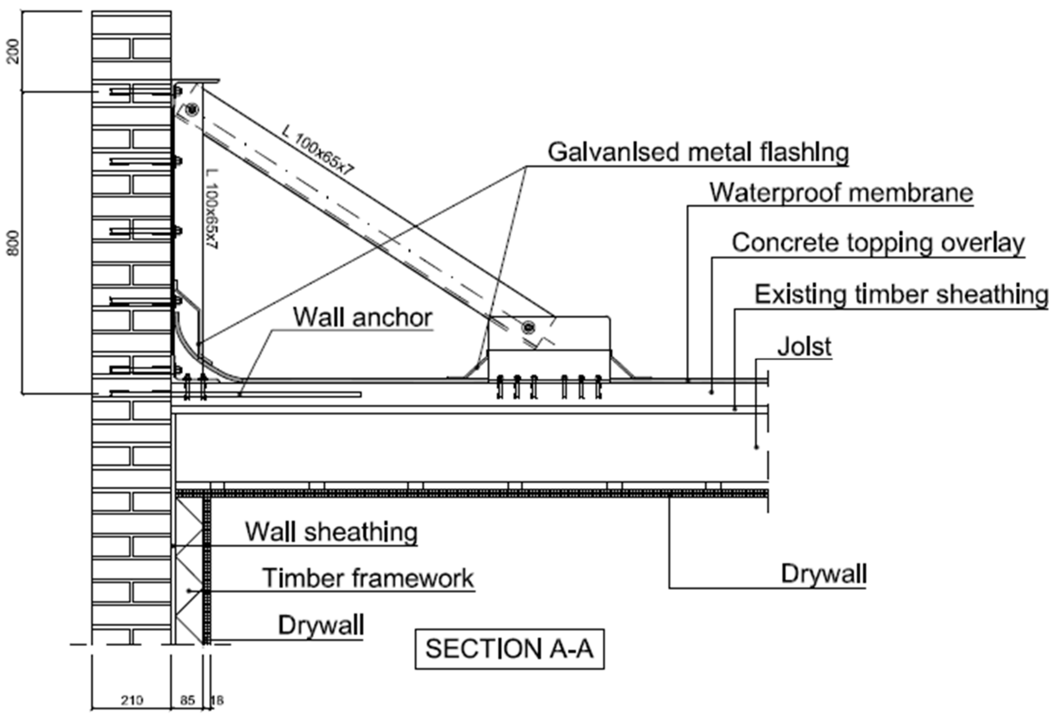
$$I_v := 112.5\ cm^4$$

$$W_{el} := 16.61\ cm^3$$

Figure 4-15 L Steel Profile 100x65x7

4.3.1 Detailed Design Assembly





4.4 Chimneys and Stacks

Unreinforced masonry chimneys result to be extremely weak for seismic load due to their characteristic slenderness. Even for relatively low levels of ground motion, URM chimneys during the earthquake may crack, separate from the main structure and collapse. The collapse of the chimney may involve injuries of occupants in case of failure through the roof structure, or anyway further damages.

4.4.1 Description of the Proposed Rehabilitation Techniques

The possible solutions to mitigate risk for unreinforced and unbraced chimneys are:

- Removal of the whole URM chimneys and firebox entirely.
- Filling of the chimney
- Removal of the chimney and replace with a lightweight solution
- Anchorage of the chimney to the building

Clearly both removal and filling techniques may be adopted only in particular cases, such as when the fireplace is not used anymore. Nevertheless, if the occupants do not show the will to use it, reducing its height to not more than 50 cm above the roofline will result to limit the potential damage.

The most reliable seismic retrofit for the mitigation of URM chimneys is the replacement of it with a lightweight solution. Indeed, the use of a metal flue inside a framed enclosure results to minimize the mass and therefore the seismic force acting on the chimneys.

On the other hand, the replacement of the whole chimney may represent an expensive intervention for typical URM house buildings. Whereas the bracing of it reduces the falling hazard by a lower price.

Eurocode 8 part 6[13] in Annex E refers to masonry chimneys giving a minimum seismic anchorage. When a masonry chimneys passes through floors and roof a building, it shall be anchored at each level which is more than 2 meters above the ground. Anchoring should be provided by means of two 5 mm by 25 mm steel straps embedded into the chimney for a minimum length of 300 mm. Each strap should be fastened to a minimum of four floor joists with teo 12 mm bolts.

The same procedure is mentioned on the American code Fema 547[11] as shown in Figure 4-16.

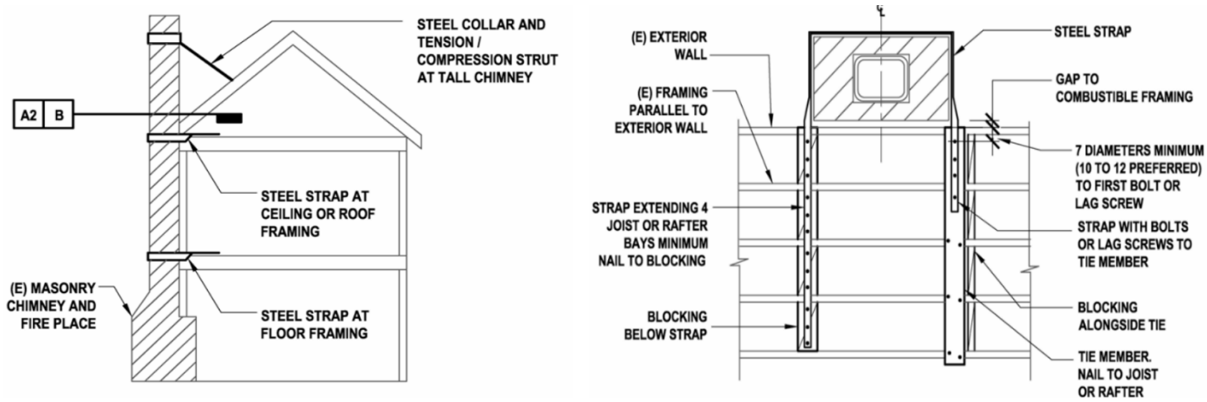


Figure 4-16 Bracing of masonry chimney along its height FEMA 547 [11].

4.4.2 Design considerations

In the retrofit of chimneys is recommended the assumption of the worst-case regarding the construction. This due to the fact that it is common to find chimneys ungrouted, poorly grouted and unreinforced.

4.4.3 Detailing and construction considerations

Alike for the retrofitting of parapets, the retrofit of chimneys via bracing supports needs an engineering design due to the vary of the roof-chimney configuration. Consideration regarding the bracing are the same before mentioned for the upgrading of the parapet support system.

Attention must be posed in the choice of the anchors type. Expansion anchors may induce splitting tensile stresses which result in cracking of the masonry. Adhesive anchors change properties when exposed to elevate temperature, therefore it might be experienced for this use. The use of embracing metal connection is suggested.

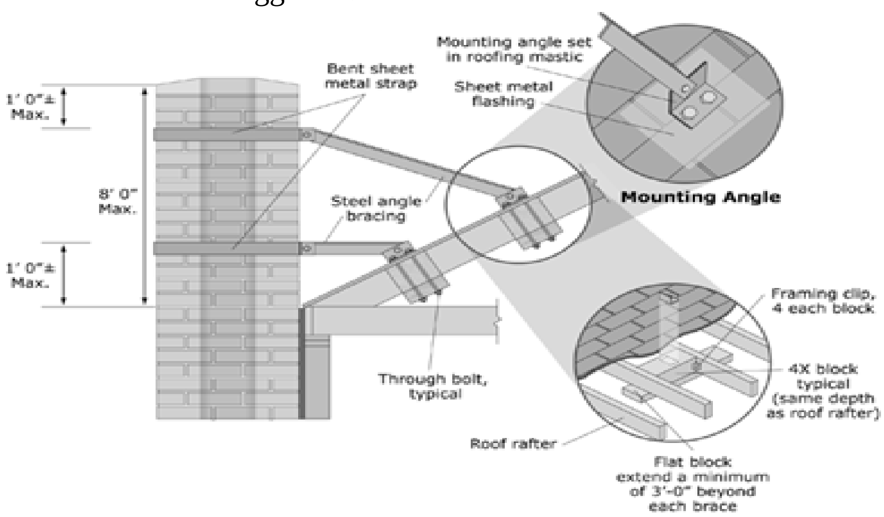


Figure 4-17 Typical brace configuration for chimneys FEMA E-74 [10]

4.5 Chimney Seismic Load Calculation Example

In the following calculation example, a chimney placed on the roof of a Dutch terraced house has been selected. The reference Dutch terraced house is the same adopted for the calculation example for the parapet retrofitting, it is displayed in Figure 4-4.

The fundamental period of the building in both directions has been determined by Rayleigh's method, calculation is shown in Section 6.6.4.

The following data for the chimney geometry and for the masonry properties have been assumed.

BUILDING	Building height	$H := 6000\ mm$
	Building fundamental period in the relevant direction	$T_{1.x} := 0.143\ s$ $T_{1.y} := 0.044\ s$
MASONRY	Modulus of Elasticity	$E_m := 4410\ MPa$
	Partial factor	$\gamma_m := 1.7$
	Weight (clay bricks)	$\rho_m := 18\ \frac{kN}{m^3}$
CHIMNEY	Thickness	$t := 210\ mm$
	Height	$h := 1200\ mm$
	Length	$l := 760\ mm$
	Height of the chimney from foundation level	$z := 7200\ mm$

Weight of the chimney $W_c := (\rho_m \cdot h \cdot (2 \cdot t \cdot l + 2 \cdot t \cdot (l - 2 \cdot t))) = 10\ kN$

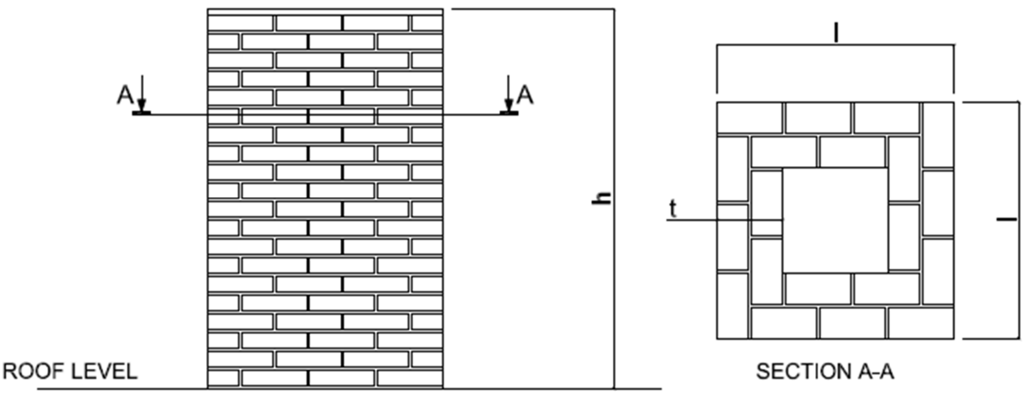


Figure 4-18 Chimney Elevation and Cross Section View

When subjected to a horizontal load the chimney resists the force by the entire area of its cross section. In this calculation example, in order to ease the computation, only the in-plane capacity of the elements composing the chimney has been considered. Consequently, the cross section of the member has been divided in four sub-elements of cross section lxt.

Figure 4-19 shows the selected sub-element highlighted by the colour green.

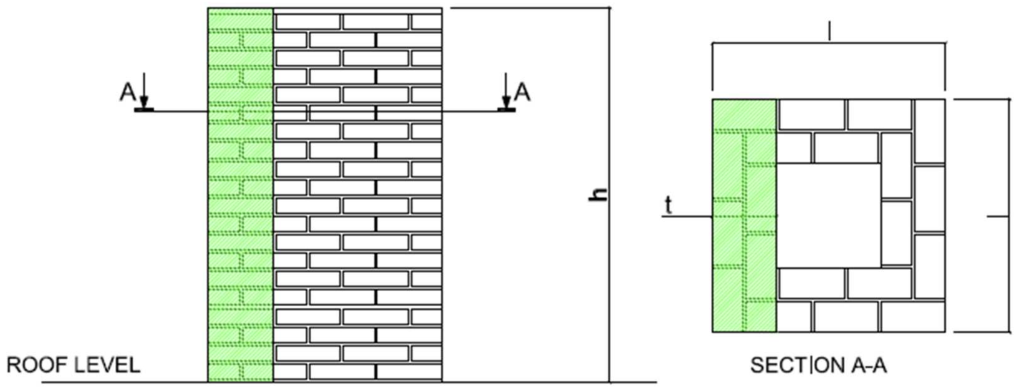


Figure 4-19 Selected Sub-Element to Determine the Lateral Stiffness

The selected sub-element has been considered to behave as a cantilever beam of length h. Hence, the lateral stiffness of the sub-element has been derived.

Weight of the chimney	$W_c := (\rho_m \cdot h \cdot (2 \cdot t \cdot l + 2 \cdot t \cdot (l - 2 \cdot t))) = 10\ kN$
Moment of inertia of selected part	$I := \frac{t \cdot l^3}{12} = (7.68 \cdot 10^5)\ cm^4$
Shear area of selected part	$A_v := \frac{5}{6} \cdot t \cdot l = (1.33 \cdot 10^5)\ mm^2$
Stiffness of selected part (considered as cantilever beam)	$K_c := \frac{3 \cdot E_m \cdot I \cdot A_v}{h^3 \cdot A_v + 7 \cdot 5 \cdot I \cdot h} = (2.45 \cdot 10^4)\ \frac{kN}{m}$

Since in both directions the lateral stiffness of the chimney is given by the in-plane stiffness of two sub-elements, the total stiffness can be determined. Consequently, also the fundamental period of vibration of the chimney has been assessed.

Stiffness of the chimney in both directions	$K_{tot} := 2 \cdot K_c = (4.89 \cdot 10^4)\ \frac{kN}{m}$
Period of the chimney in both directions	$T_c := 2\ \pi \sqrt{\frac{W_c}{K_{tot} \cdot g}} = 0.03\ s$

By means of the period of vibration of the member, it is possible to determine the seismic coefficient necessary to compute the seismic load acting on the chimney. The followed procedure is the same used to determine the seismic load acting on the parapet described in the previous section.

Seismic hazard data for design purpose have been taken from the preliminary study conducted by Arup and described in the report: Groningen 2013 - Structural Upgrading Study - Section 2.2 Seismic Evaluation

Design ground acceleration $a_g := 0.243\ g$

Ground type E, Soil factor $S := 1.6$

Seismic coefficient in the x direction $S_{a.x} := \frac{a_g}{g} \cdot S \cdot \left(\frac{3 \cdot \left(1 + \frac{z}{H}\right)}{\left(1 + \left(1 - \frac{T_c}{T_{1.x}}\right)^2\right)} - 0.5 \right) = 1.37$

Seismic coefficient in the y direction $S_{a.y} := \frac{a_g}{g} \cdot S \cdot \left(\frac{3 \cdot \left(1 + \frac{z}{H}\right)}{\left(1 + \left(1 - \frac{T_c}{T_{1.y}}\right)^2\right)} - 0.5 \right) = 2.09$

Behaviour factor of the non-structural element $q_a := 1.0$
Importance factor $\gamma_a := 1.0$

Horizontal seismic force acting at the centre of mass of the chimney

x direction $F_{c.x} := \frac{S_{a.x} \cdot W_c \cdot \gamma_a}{q_a} = 14\ kN$

y direction $F_{c.y} := \frac{S_{a.y} \cdot W_c \cdot \gamma_a}{q_a} = 21\ kN$

5 LEVEL II OF RETROFITTING STRATEGY

This chapter describes the importance of connection between diaphragms and walls, and provides calculation approaches to design connections.

Second level of retrofitting aims to improve the overall robustness of the building.

In order to enhance the overall robustness of an URM building, the seismic retrofitting design shall principally improve the monolithic three-dimensional behaviour of the structure against lateral seismic loads.

Monolithic three-dimensional behaviour is meant to be the capacity of a building to transfer both in-plane and out-of plane forces between walls and diaphragms. This behaviour can be achieved by means of adequate connections tying wall to wall, and diaphragms to wall.

5.1 Connection diaphragms to walls

Connections between diaphragms and walls are important in seismic retrofitting of masonry buildings because:

- They provide a load path between diaphragms and shear walls, and restrain diaphragms against dangerous displacements due to the seismic action.
- They avoid unfavourable failure mechanisms of walls in the out-of-plane direction by changing their boundary conditions, Figure 5-2.
- They prohibit disconnection between wall-wall, wall-roof and wall- floor. Especially the separation of the wall from the floor allows an unfavourable observed failure mechanism such as out-of-plane tipping of wall and floor collapsing, Figure 5-1.

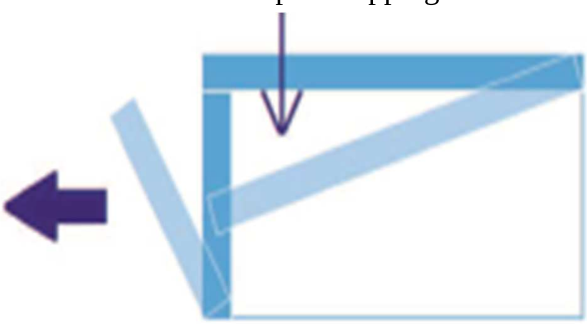


Figure 5-1 Separation of walls leads to out-of-plane tipping and floor fallina

Figure 5-2 Out of plane wall failure

A well designed connection is on one hand a connection able to avoid walls to disconnect from the diaphragms in the out-of-plane direction. On the other hand it is able to transfer loads from the diaphragm to the wall where they are restrained in the in-plane direction of the wall.

Eurocode 8[4] in the section related to design criteria and construction rules for masonry buildings states that floors and walls shall be connected in two orthogonal horizontal direction and in the vertical direction. However the European code does not suggest any procedure to design the relative connections.

Differently, FEMA 41-13[7] provides a guideline to determine the shear resistance of connection between bearing walls and diaphragms. Connections are designed to transfer the load due to the motion of the diaphragm. This load is assumed to be transferred only by means of shear connections, which act in the in-plane wall direction.

5.1.1 Procedure for Hazard Caused by Ground Shaking

The selected procedure is the BSE-2E[1] from the American code ASCE 41-13 [7], which considers existing buildings with seismic hazard of 5% probability of exceedance in 50 years.

The seismic hazard shall be determined by means of response spectrum ordinates for short (0.2s) and long (1s) periods, in the direction of maximum horizontal response. Spectrum ordinates are indicated by:

Sxs : design short-period spectral response acceleration parameter
Sx1: design long-period response acceleration parameter

Design spectral response parameters may be obtained from modified values taken from spectral response acceleration contour maps. Values found on contour maps are then adjusted respect to the site class as shown in Eq. [5.1] and Eq. [5.2]

$$S_{xs} = F_a S_s \qquad [5.1]$$

$$S_{x1} = F_v S_1 \qquad [5.2]$$

Where F_a and F_v are the modification factors determined respectively from Table 5-2 and Table 5-1, values are based on both site class and values of the response acceleration parameters S_S and S_1 for the selected return period.

[1] Basic Safety Earthquake-2 for use with the Basic Performance Objective for Existing Buildings.

	Mapped Spectral Acceleration at Short-Period S_S[a]				
Site Class	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	b	b	b	b	b

[a]Straight-line interpolation shall be used for intermediate values of S_S.
[b]Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Table 5-2 Values of Fa as a Function of Site Class and Mapped Short-Period spectral Response Acceleration Ss. ASCE 41-13[4]

	Mapped Spectral Acceleration at 1-s Period S_1[a]				
Site Class	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	b	b	b	b	b

[a]Straight-line interpolation shall be used for intermediate values of S_1.
[b]Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Table 5-1 2Values of Fv as a Function of Site Class and Mapped Spectral Response Acceleration at 1-s Period S1 ASCE 41-13[4].

5.1.2 Diaphragm Shear Transfer

Diaphragms shall be connected to shear walls at each chord in order to transfer horizontal loads due to the ground motion. If shear connections between walls and diaphragms are properly designed, it can be assumed that diaphragms transfer loads only to members running parallel to the seismic force. This assumption allows to determine the required shear load capacity for connections by means of the minimum of Eq. [5.3] and Eq. [5.4] as described in ASCE 41-13[7].

$$V_d = 1.25 S_{X1} C_p W_d \qquad [5.3]$$

$$V_d = v_u D \qquad [5.4]$$

Where:

S_{X1} is the design spectral acceleration at 1-second period, in g units

C_p is the horizontal force factor, given by the Table 5-3

W_d is the total dead load tributary to a diaphragm level

v_u is the unit shear capacity value for a diaphragm, given by Table 5-5 and Table 5-4

D is the depth of the diaphragm

CONFIGURATION OF MATERIALS	C_p
Roofs with straight or diagonal sheathing and roofing applied directly to the sheathing, or floors with straight tongue-and-groove sheathing.	0.50
Diaphragms with double or mulitple layers of boards with edges offset, and blocked plywood systems.	0.75
Diaphragms of metal deck without topping:	
Minimal welding or mechanical attachment.	0.6
Welded or mechanically attached for seismic resistance.	0.68

Table 5-3Horizontal Force Factor Cp IEBC [6]

EXISTING MATERIALS OR CONFIGURATION OF MATERIALS[a]		STRENGTH VALUES × 14.594 for N/m
Horizontal diaphragms	Roofs with straight sheathing and roofing applied directly to the sheathing.	300 lbs. per ft. for seismic shear
	Roofs with diagonal sheathing and roofing applied directly to the sheathing.	750 lbs. per ft. for seismic shear
	Floors with straight tongue-and-groove sheathing.	300 lbs. per ft. for seismic shear
	Floors with straight sheathing and finished wood flooring with board edges offset or perpendicular.	1,500 lbs. per ft. for seismic shear
	Floors with diagonal sheathing and finished wood flooring.	1,800 lbs. per ft. for seismic shear
	Metal deck welded with minimal welding.[c]	1,800 lbs, per ft. for seismic shear
	Metal deck welded for seismic resistance.[d]	3,000 lbs. per ft. for seismic shear

Table 5-4 Strength values For Existing Materials IEBC [17]

NEW MATERIALS OR CONFIGURATION OF MATERIALS		STRENGTH VALUES
Horizontal diaphragms	Plywood sheathing applied directly over existing straight sheathing with ends of plywood sheets bearing on joists or rafters and edges of plywood located on center of individual sheathing boards.	675 lbs. per ft.
Crosswalls	Plywood sheathing applied directly over wood studs; no value should be given to plywood applied over existing plaster or wood sheathing.	1.2 times the value specified in the current building code.
	Drywall or plaster applied directly over wood studs.	The value specified in the current building code.
	Drywall or plaster applied to sheathing over existing wood studs.	50 percent of the value specified in the current building code.
Tension bolts[e]	Bolts extending entirely through unreinforced masonry wall secured with bearing plates on far side of a three-wythe- minimum wall with at least 30 square inches of area.[b,c]	5,400 lbs. per bolt 2,700 lbs. for two-wythe walls
Shear bolts[e]	Bolts embedded a minimum of 8 inches into unreinforced masonry walls; bolts should be centered in $2^1/_2$-inch-diameter holes with dry-pack or nonshrink grout around the circumference of the bolt.	The value for plain masonry specified for solid masonry in the current building code; no value larger than those given for $^3/_4$-inch bolts should be used.
Combined tension and shear bolts	Through-bolts—bolts meeting the requirements for shear and for tension bolts.[b,c]	Tension—same as for tension bolts Shear—same as for shear bolts
	Embedded bolts—bolts extending to the exterior face of the wall with a $2^1/_2$-inch round plate under the head and drilled at an angle of $22^1/_2$ degrees to the horizontal; installed as specified for shear bolts.[a,b,c]	Tension—3,600 lbs. per bolt Shear—same as for shear bolts

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm^2, 1 pound = 4.4 N.
a. Embedded bolts to be tested as specified in Section A107.4.
b. Bolts to be $^1/_2$ inch (12.7 mm) minimum in diameter.
c. Drilling for bolts and dowels shall be done with an electric rotary drill; impact tools should not be used for drilling holes or tightening anchors and shear bolt nuts.
d. No load factors or capacity reduction factor shall be used.
e. Other bolt sizes, values and installation methods may be used, provided a testing program is conducted in accordance with UBC Standard 21-7. The useable value shall be determined by multiplying the calculated allowable value, as determined by UBC Standard 21-7, by 3.0, and the useable value shall be limited to a maximum of 1.5 times the value given in the table. Bolt spacing shall not exceed 6 feet (1829 mm) on center and shall not be less than 12 inches (305 mm) on center.

Table 5-5 Strength Values Of New Materials Used in Conjunction with Existing Construction IEBC [17]

5.1.3 Anchorage of Walls to the Diaphragm in the Out-Of-Plane Direction

During seismic events walls are subjected to horizontal loads proportional to their mass acting perpendicular to the wall face. Consequently, every wall shall be positively connected to diaphragms, which provide lateral restrain to walls.

Connection is provided by a number of anchors placed along the diaphragm line, where each anchor is designed respect to the portion of wall tributary to it. Distance between two consecutive anchors shall not exceed 2.4 meters, unless capacity of the wall has been checked to be adequate to span for greater distance. The design load acting on a typical anchors shall be determined by means of Eq. [5.5] but not less than the loads determined by Eq. [5.6] as described in ASCE 41-13 [7].

$$F_p = 0.4 S_{xs} k_a k_h \chi W_p \qquad [5.5]$$

$$F_{p,\min} = 0.2 k_a \chi W_p \qquad [5.6]$$

$$k_a = 1.0 + \frac{L_f \cdot 0.30}{100} \qquad [5.7]$$

$$k_h = \frac{1}{3}\left(1 + \frac{2 z_a}{h_n}\right) \qquad [5.8]$$

Where:

F_p is the seismic force for anchorage of walls to diaphragms.

k_a is the factor to account for diaphragm flexibility, equal to 1.0 for rigid diaphragms and need not exceed 2.0 for flexible diaphragms

L_f is the span, of a flexible diaphragm that provides the lateral support for the wall between vertical primary seismic-force-resisting elements that provide lateral support to the diaphragm in the direction considered.

k_h is the factor to account for variation in force over the height of the building when all diaphragms are rigid-for flexible diaphragms, use 1.0.

z_a is the height, of the wall anchor above the base of the structure, not to exceed h_n.

h_n is the height, above the base to the roof level

χ is the factor for calculation of out-of-plane wall forces, from Table 5-6 for the selected Structural Performance Level.

S_{xs} is the spectral response acceleration parameter at short periods for the selected hazard level and damping, adjusted for site class, without any adjustment for soil-structure interaction

W_p is the weight of the wall tributary to the wall anchor.

Structural Performance Level	χ
Collapse Prevention	1.0
Life Safety	1.3
Immediate Occupancy	2.0

Table 5-6 Factor X for Calculation of Out-of-Plane Wall Forces FEMA E-547[7]

5.2 Diaphragm Shear Transfer Calculation Example

In the following calculation example, two different horizontal loads due to the seismic ground shaking are determined. Firstly, the load supposed to be transferred to shear walls due diaphragm acceleration is calculated. Then, forces acting at diaphragm levels due to the oscillation of wall panels are determined. Both horizontal loads are assumed to be resisted by connections elements which have been designed in section.

The calculation has been carried out in accordance with the design procedure present in the American code ASCE 41-13[7]. For this reason, the peak ground acceleration (a_g) has been converted in the Short and Long Period Spectral Acceleration. These have been determined using the converting relations elaborated by Lubkowsky [14] shown in annex A.

Anchorage of diaphragm to walls

Peak ground acceleration (PGA) $a_g := 0.243$ [g]

Short period spectral acceleration $S_s := (0.3386 \cdot a_g + 2.1696) \cdot a_g$

Long period spectral acceleration $S_1 := (0.5776 \cdot a_g + 0.5967) \cdot a_g$

$$S_s := 0.5472\ g$$

$$S_1 := 0.1791\ g$$

Considering a Site Class E for the Groningen area, values for Fa and Fv can be determined by Table 2-3 and Table 2-4 respectively on ASCE 41-13.

$F_a := 1.6$ $\qquad S_{XS} := F_a \cdot S_s = 8.6\ \frac{m}{s^2}$

$F_v := 3.0$ $\qquad S_{X1} := F_v \cdot S_1 = 5.3\ \frac{m}{s^2}$

	Storey height	$h := 3000\ mm$
WALL	Wall thickness	$t := 210\ mm$
	Masonry weight	$\rho_m := 1800\ \frac{kg}{m^3}$

Computation data have been assumed with reference to a Dutch terraced house already introduced in the report. For this calculation solid walls have been considered, the analysis of loads transferred from diaphragms to walls in case of cavity walls is presented in Annex B.

Diaphragms have been assumed to be a standard timber floor one single sheathings supported by timber joists. In Figure 5-3 the two diaphragm regions with different spans have been highlighted by colours.

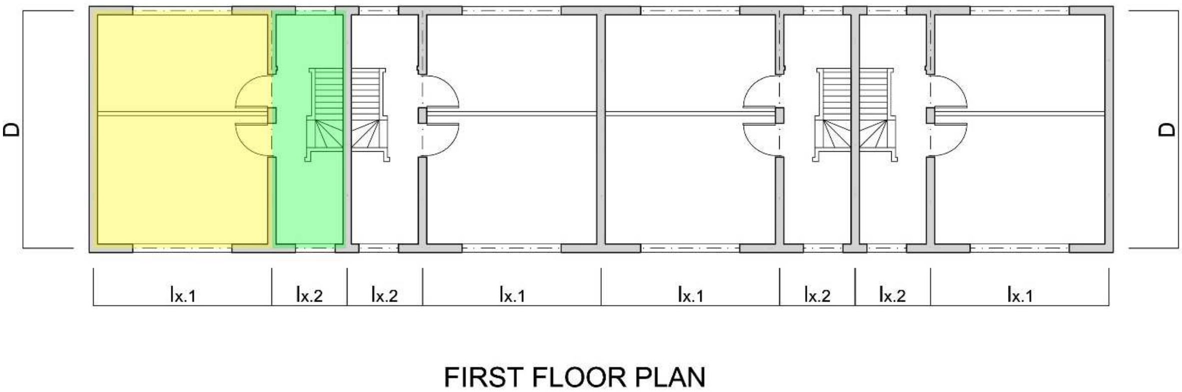


Figure 5-3 Examined Diaphragms in the Calculation Example

Diaphragms are supposed to be rigid in this calculation example. Hence, timber diaphragms are assumed to have been retrofitted by means of a concrete topping overlay of 60 mm thickness.

DIAPHRAGM Ceramic tiles and glue: 50 kg/m2
Concrete topping overlay: 80 kg/m2
Timber floor single sheathing: 35 kg/m2
Live load: 200 kg/m2

Diaphragm dimensions (see Figure) $l_{x.1} := 4500\ mm$ $l_{x.2} := 1900\ mm$ $D := 6000\ mm$

Diaphragm gravity load (see Annex A) $q_{G.d} := 1.1 \cdot \left(165 \dfrac{kg}{m^2} + 0.25 \cdot 200 \dfrac{kg}{m^2}\right) = 236.5 \dfrac{kg}{m^2}$

Dead load tributary to diaphragm 1 $W_{d.1} := (q_{G.d} \cdot D \cdot l_{x.1}) + (2 \cdot 1.1 \cdot \rho_m \cdot l_{x.1} \cdot h \cdot t) = 17612\ kg$

Dead load tributary to diaphragm 2 $W_{d.2} := (q_{G.d} \cdot D \cdot l_{x.2}) + (2 \cdot 1.1\ \rho_m \cdot l_{x.2} \cdot h \cdot t) = 7436\ kg$

Unit shear capacity value for the diaphragm taken from Table 3-4 (Table A1-D IEBC 2006) $\nu_u := 21891 \dfrac{N}{m}$

Horizontal Force Factor Cp taken from Table 3-3 (Table 15-3 ASCE 41-13) $C_p := 0.5$

Storey force distributed to shear walls in the in-plane direction

Horizontal loads transferred from the diaphragm to shear walls in their in-plane direction are determined by the minimum of the following values:

Diaphragm 1 (yellow area)

$V_{d;1} := 1.25\ S_{X1} \cdot C_p \cdot W_{d.1} = 58\ kN$

$V_{d;1} := \nu_u \cdot D = 131.3\ kN$

Diaphragm 2 (green area)

$V_{d;2} := 1.25\ S_{X1} \cdot C_p \cdot W_{d.2} = 24.5\ kN$

$V_{d;2} := \nu_u \cdot D = 131.3\ kN$

Anchorage of wall to the diaphragm in the out-of-plane direction

Semic load due to acelleration of wall panels has been determined for a contiguos wall with no openings. The panel is assumed to be retrained by anchors at diaphragms levels. Figure 5-4 shows the wall panel involved in the calculation highlighted by the colour red with indicated its dimensions.

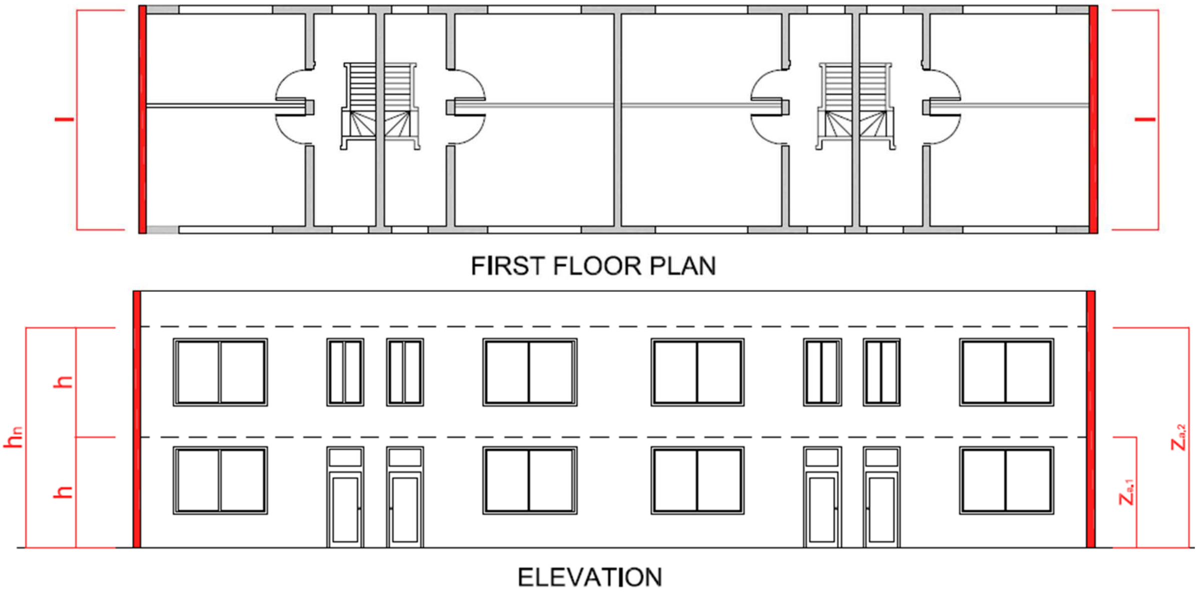


Figure 5-4 Wall Panels to Retrofit

Wall length	$l := 6000\ mm$
Storey height	$h = 3\ m$
Factor to account diaphragm flexibility (assumed rigid diaphragm)	$k_a := 1$
Height of the wall anchors above the base of the structure	$z_{a.1} := 3000\ mm$ $z_{a.2} := 6000\ mm$
Height above the base to the roof level	$h_n := 6000\ mm$
Factor for calculation out-of-plane wall forces from Table 3.-6 (Table 7-2 ASCE 41-13)	$\chi := 2$
Factor to account for variation in force over the eight of the building	$k_{h.1} := \frac{1}{3}\left(1 + 2 \cdot \frac{z_{a.1}}{h_n}\right) = 0.7$ $k_{h.2} := \frac{1}{3}\left(1 + 2 \cdot \frac{z_{a.2}}{h_n}\right) = 1$

A number of anchors has been assumed for both levels of anchoring. This assumption should be adjusted iteratively in order to match the required horizontal load to the effective pull out resistance of the designed anchor.

The Figure 5-5shows the assumed disposition of anchors and their spacing.

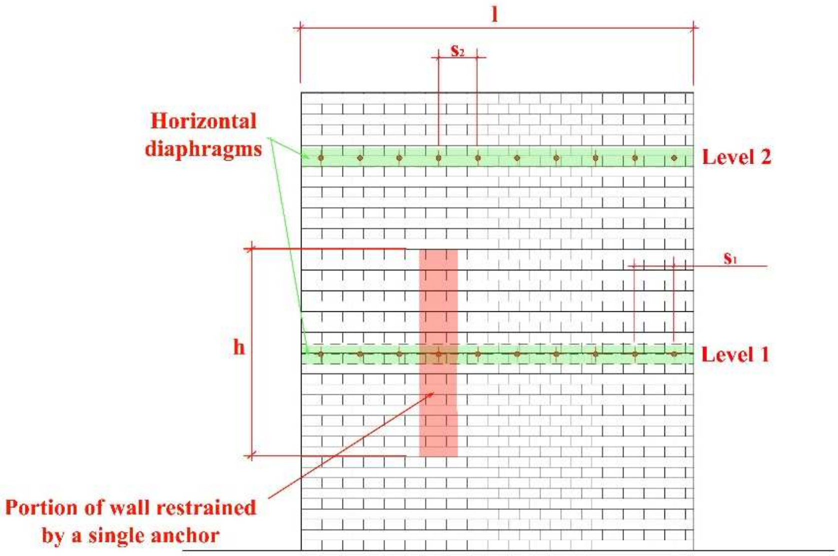


Figure 5-5 Anchor Locations

Values of shear and tensile load which each connection is assumed to resist need to be checked with the real capacity of each anchors. Calculation in order to determine the shear and tensile capacity of a single anchor has been carried out in section 5.7.2.

Number of anchors between wall and diaphragm	To the lower diaphragm To the upper diaphragm	$n_1 := 8$ $n_2 := 8$
Distance between consecutive anchors in the relative direction		$s_1 := \frac{l}{n_1} = 750\ mm$ $s_2 := \frac{l}{n_2} = 750\ mm$
Weight of the wall tributary to the wall anchor in the relative direction		$W_{p.1} := t \cdot s_1 \cdot h \cdot \rho_m = 850.5\ kg$ $W_{p.2} := t \cdot s_2 \cdot h \cdot \rho_m = 850.5\ kg$

Horizontal load due to the portion of wall relative to the anchor is determined by the value Fp, but not less than Fp.min. Where this value represents the axial load which each anchor needs to resist.

Load acting on anchors at level 1	$F_{p.1} := 0.4\ S_{XS} \cdot k_a \cdot k_{h.1} \cdot \chi \cdot W_{p.1} = 3.9\ kN$ $F_{p.1.min} := 0.2\ S_{XS} \cdot k_a \cdot \chi \cdot W_{p.1} = 2.9\ kN$
Load acting on anchors at level 2	$F_{p.2} := 0.4\ S_{XS} \cdot k_a \cdot k_{h.2} \cdot \chi \cdot W_{p.2} = 5.8\ kN$ $F_{p.2.min} := 0.2\ S_{XS} \cdot k_a \cdot \chi \cdot W_{p.2} = 2.9\ kN$

Ring Beam

This retrofitting solution is normally used to provide a stiff connection from wall to roof, which is minimal in the upper part of the building, due to the low compressive force in the masonry. The effectiveness of a ring beam strongly influences the building behaviour during a seismic event. A ring beam increases connection between walls, reduces outwards displacements, and enhance box-like behaviour. Different construction materials and techniques can be adopted for the realization of ring beams.

Reinforced masonry ring beam allows to maintain the original aesthetic characteristic of the masonry wall. At the top of the existing wall the ring beam is built using bricks and the steel reinforcement bonded via cement mortar. Bonding with the upper part of the existing wall can be obtained in a second moment by installation of grouted reinforcing steel bars. In case of poor quality of the existing masonry, a strengthening of the upper part of the wall is needed. This intervention requires to remove the roof, and do not show issues of thermal bridges.

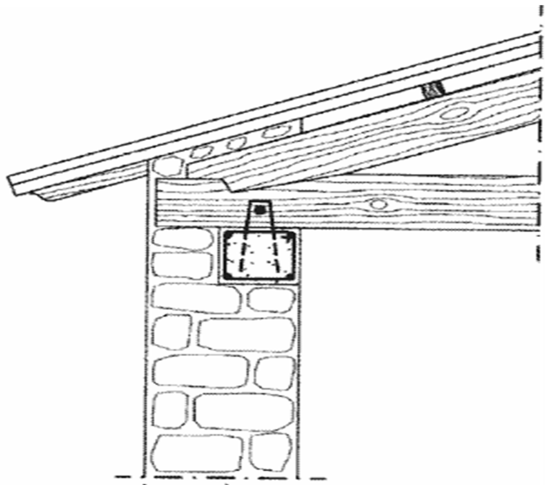


Table 5-7 Reinforce Masonry Ring Beam

Ring beams made out of steel result to have some advantages respect to reinforced masonry beams. Thanks to their lightweight, the mass increment in the building is negligible. Invasiveness is limited, since they can be placed on both external and internal face of the wall. Connection to diaphragm timber members is direct, and as crucial advantage, they can be placed without remove the roof. When the ring beam is applied internally, it may result invisible if placed in the crawl space.

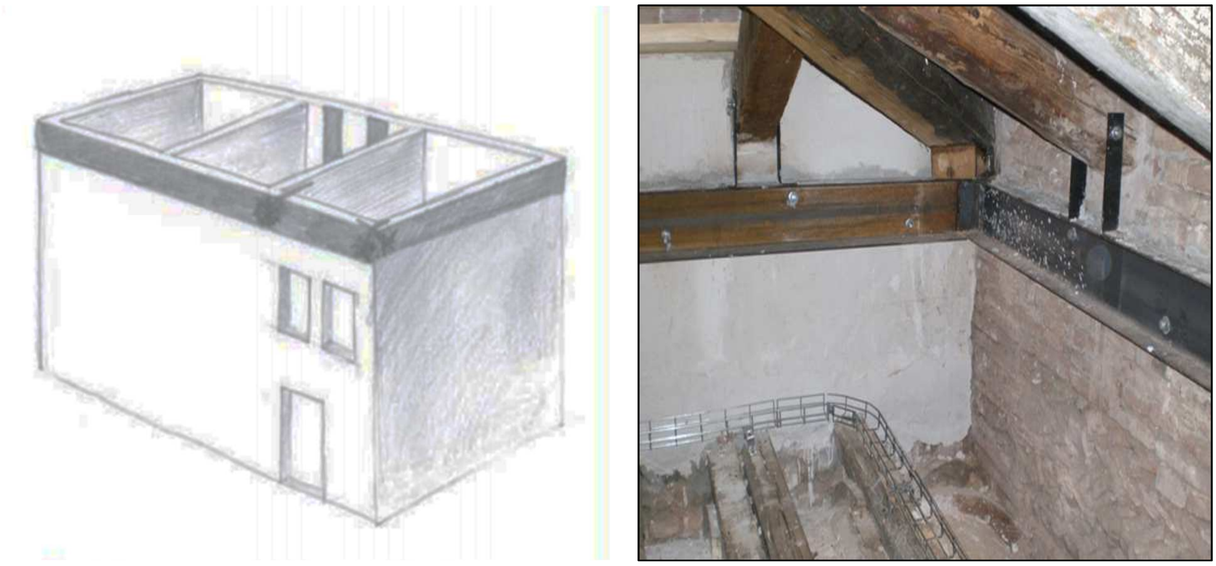


Table 5-8 Steel Ring Beam

5.3 Metallic or Different Material Wall Tie-Rods

Metallic or different material tie-rods are usually located at diaphragm level. Acting in both principal directions of the structure, they provide a valuable connection between orthogonal walls, besides favour the development of a three-dimensional behaviour of the structure. Typically connections are realized by means of anchoring plates, which are visible on the façade of the building. The typology of anchor plate should be wisely chosen by the designer respect to the quality of the masonry, which may be locally reinforced in the portion subjected to concentration of stress.

Table 5-9 Possible Typology of Anchor Plates

In order to experience an effective improvement in the seismic behaviour of the building due to the upgrading, a certain stiffness is required to the tie-rods system.

This can be achieved by rather using big diameters and limited length bars or pre-stressed bars.

In case pre-stressed bars are adopted, attention must be paid to avoid local damage on the wall.

Where the retrofit is applied along walls of significant length, to hold ties in position, elements such as stirrups, should be fixed into drilled holes.

The best performance is obtained when they are placed on both sides of walls, just below the floor, and fixed on the same external anchor. Consequently, ties involve two different reinforcing mechanisms depending on the direction of the seismic force, from Frumento [6]:

- When the seismic force acts transversally respect to the ties direction, the upper part of the wall and ties perform like a bond-beam. Consequently, ties should be designed as longitudinal bond-beam reinforcement acting against the bending moment due to the out-of-plane wall vibration.
- When the seismic force acts longitudinally respect to the ties direction, a global truss system develops. Compression components transferred from the compressed masonry diagonals run from storey to storey thanks to the horizontal truss members composed by the steel ties.

Therefore, both scenarios should be considered for the sizing of tie elements.

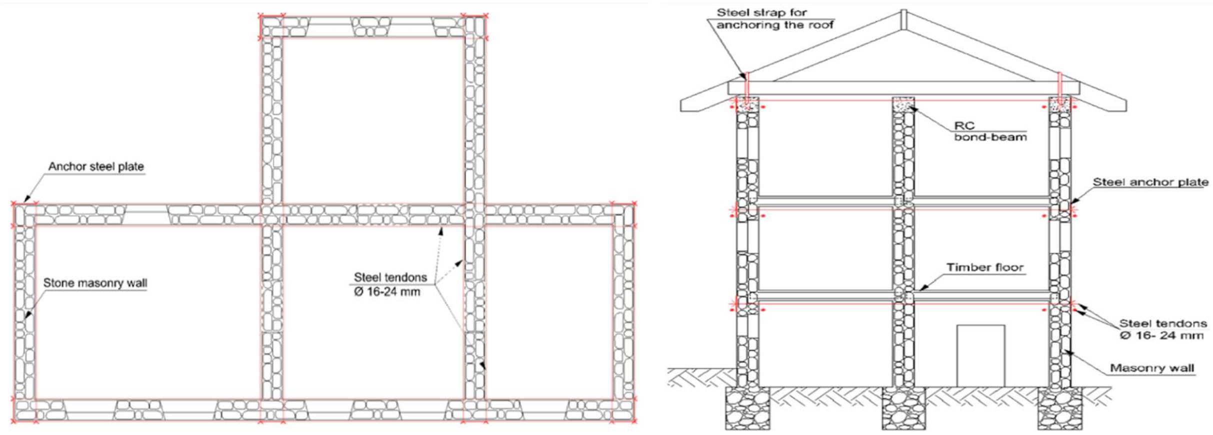


Figure 5-6 Steel Tendons Running on Both Sides of the Wall

This technique is often supported by anchoring diaphragms to walls. When distance between transverse cross walls is significant, steel ties do not ensure the wanted three-dimensional behaviour. Consequently, to reduce the uncoupled wall vibrations and minimize possible out-of plane cracking or failure, walls are anchored to diaphragms along their spans. A thorough description about connection of diaphragm to walls is given in the section 3.5 of this report.

5.4 External Circumferential Bandage

External circumferential bandage provides an effective connection between orthogonal walls. This enhance the three-dimensional behaviour of the structure and therefore increasing a box like behaviour for horizontal loads. The bandage can be realized by means of different material such as metallic elements (tie-rods), fibre reinforced polymers or reinforced concrete strips

The use of tie-rods is suggested for buildings with limited wall length, since no anchorage are expected along the facades. Attention should be paid for concentration of stress due to tensioning and possible pre-stress, which can be cause of damage in corners. This problem can be avoided by means of repartition plates, or by smoothing of corners.

Figure 5-8 Fibre Reinforced Polymers circumferential bandage

Figure 5-9 Steel ties along the facade

Figure 5-10 Repartition plates

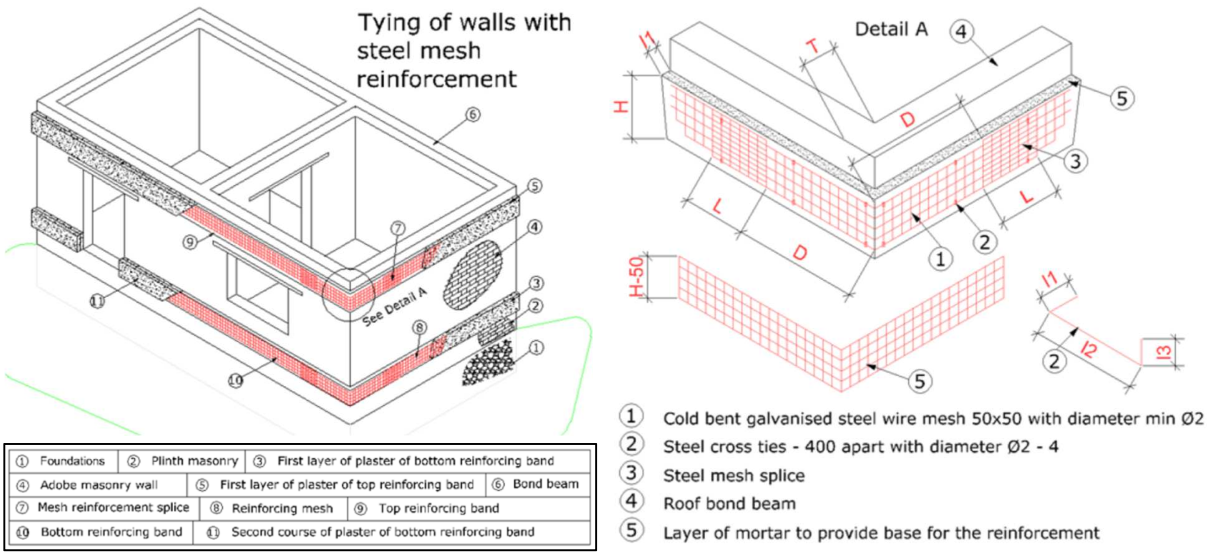


Figure 5-7 External Circumferential Bandage by means of Reinforced Concrete Strip

5.4.1 Design Consideration:

Reinforced concrete bandage as shown in Figure 5-7 is composed by a steel mesh embedded in a mortar strip. The steel mesh runs along walls embedded into cast strip about 15 mm thick of mortar. It works as a series of parallel flat ties at diaphragm level. In case the strip runs along long walls, the welded steel mesh shall be fixed to the mortar basis by means of steel cross ties, which are attached to the wall through purposely drilled holes.

On the wall surface, a base of cement of 15 mm thickness is installed. This layer is needed for the application of the steel mesh, which should be attached to the cast strip of mortar by embedded ties. The flat steel ties should be embedded in the masonry wall through purposely drilled holes. After, a second mortar layer of 15 mm thickness is installed to ultimate the reinforcing bandage.

Reinforced concrete bandage is thought to be able to reduce cracks on the wall due to the seismic load. Besides it provides an increase of the wall bonding through the tensile strength of the steel mesh, the layer of the concrete strip can be considered as a local stiffening of the URM masonry wall. Which may results advantageous for the installation of steel ties between floor and wall with external anchors.

5.5 Connection Wall to Diaphragm

One of the most affecting deficiency to the overall robustness of URM buildings is the lack of adequate mechanical connection between masonry walls and floor and roof diaphragms.

Bond between walls and diaphragms may be realized by means of connection steel ties.
Ties which connect the wall to the diaphragm play two roles during the seismic events:

- When the seismic force acts perpendicular to the wall direction, ties transfer out-of-plane inertial loads back into the diaphragm. This resisting mechanism helps to hold walls from falling away from the building.
- Ties transfer loads from diaphragms into shear walls where they are resisted by in-plane action of them. This avoids diaphragms from sliding along walls.

5.5.1 Design Considerations

Anchors when embedded in masonry walls shall be considered force-controlled components. Minimum effective embedment length shall be determined through consideration of pull-out and shear strength. Anchors are typically installed using one of three different configurations as shown in the following figures. In Figure 5-13and in Figure 5-11 the dowel is drilled and grouted into the masonry. The angle of the dowel in Figure 5-11 allows to engage more courses of brick, which theoretically should improve the reliability. Figure 5-12 shows a through bolt anchor using a steel sleeve [7].

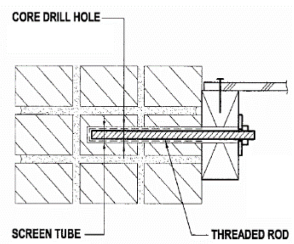


Figure 5-13 Drilled straight dowel FEMA 547 [4]

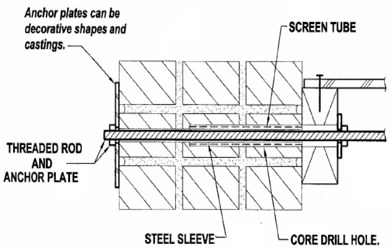


Figure 5-12 Trough bolt anchor FEMA 547 [4]

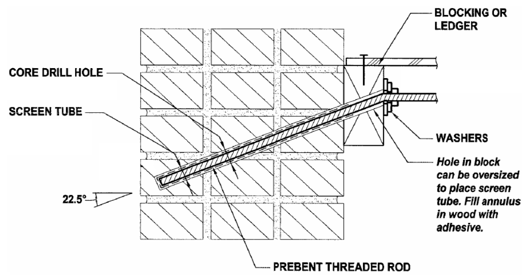
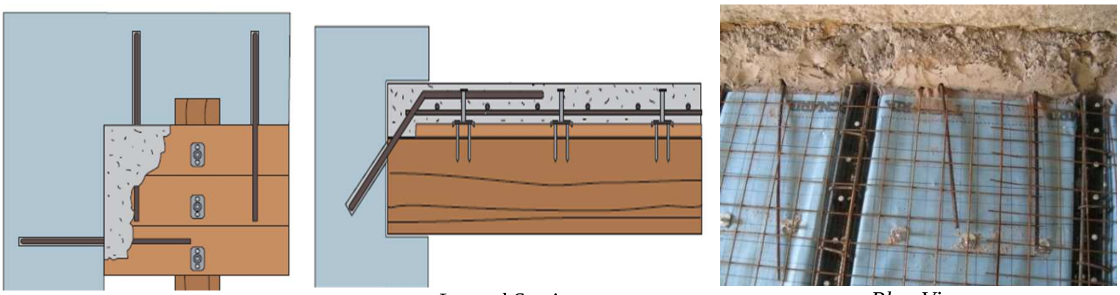


Figure 5-11 Drilled inclined dowel FEMA 547 [4]

5.5.2 Detailing and Construction Considerations

- *Aesthetics:* Anchors as shown in Figure 5-12 have a visible bearing plate on the exterior face of the wall. Many different anchor plates are present on the market, some of them present a countersunk hole which can be recessed into the wall and finished by stucco. When this approach is not possible, drilled dowels represent the best solution. Drilled dowels if properly set results totally invisible.
- *Installation approach:* The first stage is drill the hole and clean it by means of brush and compress air. Than a screen tube filled with adhesive is inserted. The screen tube looks like a test tube made out of wire mesh and can be in nylon, carbon or stainless steel. Then, when the threated rod is pushed into the screen tube it forces the adhesive out of the tube into the annulus between tube and masonry.

- *Adhesive types*: In the past, cementitious non-shrink grout were used, but they require large diameter holes. Chemical adhesives are nowadays preferred, there are many typologies, though most are epoxy. This because epoxy products have longest track record. The main aspects which affect an adhesive are the length of time it has been used, extent and quality of testing, ability to bond to damp, cost. When adhesives are curing, precautions for ventilation should be taken to avoid unpleasant off-gassing.
- *Dowel material type:* Threated rod is commonly specified as s355. Rebars are used as well, but they need to be threaded at the ends in case of connection between timber floor frame and wall.
- *Access:* The installation of anchors can be done either from above the diaphragm or below, this depends on whether there are finishes that need to be removed, whether strengthening of diaphragm is part of the retrofitting process, and which kind of diaphragm strengthening is planned. In case of a concrete topping overlay the anchors can be embedded into the concrete layer as shown in Figure 5-14.



Plan View *Lateral Section* *Plan View*

Figure 5-14 Connection Dowels Embedded into the Concrete Topping Overlay Tecnaria [34].

- *Issues for anchors at the top of the wall:* In most of URM buildings, the perimeter wall continues above the roof level forming a parapet which provides a fire protection and serves as a guardrail during roof maintenance. In some buildings, instead, the roof exceed the wall line. In this situation the reliability of the drilled dowel is reduced by the low overburden pressure at the top of the wall. Making reliable connections is usually dependent on the specific geometry and characteristics of the existing details. One of the most common strategies, even if it is an invasive solution, is to employ a concrete bond beam at the top of the wall, which provides anchorage for ties.
- *Reuse of existing ties:* In many old URM buildings ties so called government or “dog” anchors are already present. These elements normally only occur in the wall face where the joist are perpendicular and they probably are not at a sufficient spacing. Therefore a test of them is needed if the intention is to use them as wall-to-diaphragm tension anchors.
- *Dowel spacing and edge distance:* The American code provides some maximum spacing requirements on shear and tension dowels. At roof and floor level, anchors shall be provided within 610 mm (2 feet) horizontally from the inside of the corners of the walls, and the maximum spacing along the diaphragm line is 1830 mm (6 feet). When walls become thick, the out-of-plane demand and the relatively low capacity lead to tight dowel

spacing. From a practical considerations dowels should not be placed closer than 305 mm (12'') o.c..

- *Corrosion consideration:* In the case of drilled dowels, they are installed from the interior, where both the masonry and the epoxy bonding creates a corrosion protection, hence mild steel anchors are considered sufficient. When bolt connections are used, a more direct path for moisture intrusion is present. The anchor plate can be galvanized, made from stainless steel or painted with exterior grade paint, the through bolt can be made from stainless steel as well.

5.5.3 Cost and Disruption

The cost is firstly dependent on the difficulty of access given by the extent of finishes that are installed. Secondly it depends on the amount and type of the anchors. Through bolt solution results to be cheaper than adhesive anchors.

Drilling is loud and can be disruptive to occupants. It is suggested to apply this retrofitting techniques together with the strengthening of the diaphragm, when it is planned.

5.6 Conclusion about Level II Retrofitting Strategy

The second level of retrofitting strategy aims to increase the overall robustness of masonry buildings by means of enhancement of connection between walls and horizontal diaphragms.
This may be achieved via solutions mentioned in the previous sections, namely ring beam, wall tie-rod, external circumferential bandage and direct connection of the diaphragm to the surrounding walls.

Connection of diaphragms to wall by means of steel bar has been evaluated as the most technically reliable retrofitting technique for this type of weakness aspect of the exanimated URM buildings. This because respect to the other possibilities it does not affect the aesthetic condition of the construction, when dowels are grouted into the masonry. It appears to be easily repeatable and scalable on numbers of buildings since no special or complex members are involved. Furthermore, when the strengthening of the diaphragm is planned as seismic upgrading, these two interventions may be realized concurrently resulting in a minimal disruption for the occupant and a decrease of costs.

5.7 Connection Wall to Diaphragm Calculation Example

It has been assumed that connection between diaphragm and shear walls is executed together with the strengthening of the diaphragm by means of concrete topping (further described in the next chapter).
Connections are realized by means of steel bars which are embedded in the concrete overlay topping and grouted in the masonry shear walls, steel bars are supposed to behave as dowels.

5.7.1 Load Bearing Capacity of a Single Steel Bar

For the determination of the load bearing capacity of laterally loaded single bar connection, the Johansen-Meyer theory has been adopted. Equations based on this theory are present on several codes to determine the load-carrying capacity of a timber connection. The resistance of a single fastener connection for timber elements is dependent on the material properties of the timber and of the fastener and on the geometry of the connection itself. The theory assumes for both timber under embedding stresses and dowel under bending a rigid-plastic ideal behaviour.
The maximum load connection can carry depends on both bearing strength of materials of connected parts and plastic bending moment of connecting element (steel bar).
For bearing strength, or embedment strength is intended the limit resistance that the bar meets when pressed into the masonry or into the concrete layer. Since the purpose is to avoid cracks in both materials, the bearing strength has been limited to the elastic compressive resistance of both masonry and concrete.
The second parameter to define the connection strength is the yield bending moment of the bar. If neither concrete nor masonry fail prematurely, the steel rod will fail in bending at position where the plastic hinge has occurred.

The first failure mechanism occurs when the steel rod does not deform, but remains straight or only rotates (Figure 5-15). In this mechanism the embedment strength of the connected parts is decisive. The rod behaves as a knife pushed into the material and therefore a translation of the connection occurs. The maximum shear force in this mechanism is given by the embedment strength applied on the whole surface of the bar.

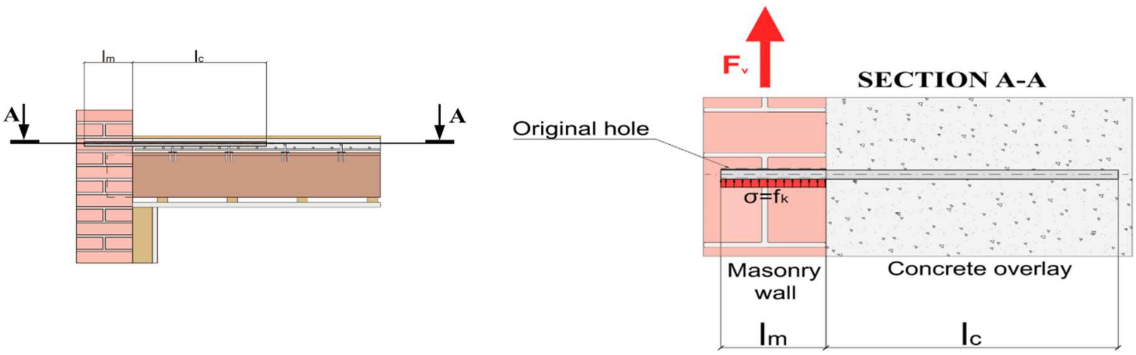


Figure 5-15 First Failure Mechanism for Shear Connection

The maximum shear force $F_{v,1}$ is given by the minimum values between the failure mechanism occurring in the masonry side and in the concrete side.

$$F_{v,1} = \min \begin{cases} l_m \cdot d \cdot f_{cd,m} \\ l_c \cdot d \cdot f_{cd,c} \end{cases}$$

When the first failure mechanism does not occur, the bending moment in the dowel will increase. With the steel rod embedded in the concrete slab, it can be assumed that the dowel is fully fixed and considered as a cantilevered beam. For a certain shear load the bending moment in the rod reaches the yield moment. Everywhere the steel rod is in contact with the masonry, the maximum embedment stress in reached (Figure 5-16). Assuming the rod represented as a cantilevered beam (Figure 5-17) the bearing capacity can be derived by means of equilibrium.

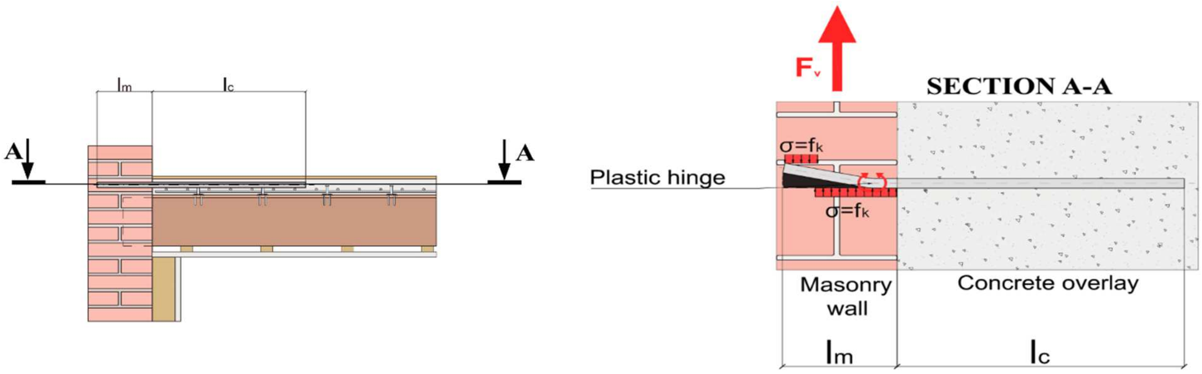


Figure 5-16 Second Failure Mechanism in Shear Connection

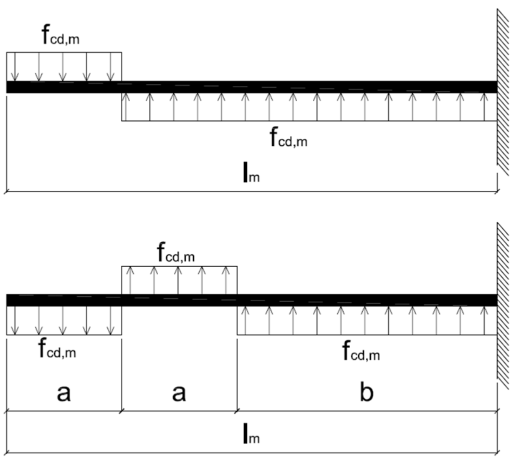


Figure 5-17 Assumed Behaviour of the Bar

$$M_{y,d} = df_{cd,m}a\left(\frac{3}{2}a+b\right) - df_{cd,m}a\left(b+\frac{a}{2}\right) - f_{cd,m}\frac{b^2}{2}$$

$$M_{y,d} = df_{cd,m}\left(a^2 - \frac{b^2}{2}\right)$$

$$where \;\; a = \frac{l_m - b}{2} \quad \rightarrow \quad M_{y,d} = df_{cd,m}\left(\frac{l_m^{\;2}}{4} - \frac{l_m b}{2} - \frac{b^2}{4}\right)$$

$$solving \;\; for\, b \quad \rightarrow \quad b = l_m\left(\sqrt{2 + \frac{4M_{y,d}}{\left(df_{cd,m}l_m^{\;2}\right)}} - 1\right)$$

$$F_{v,2} = df_{cd,m}b \quad \rightarrow \quad \boxed{F_{v,2} = f_{cd,m}\, l_m\left(\sqrt{2 + \frac{4M_{y,d}}{\left(df_{cd,m}l_m^{\;2}\right)}} - 1\right)}$$

Consequently to the heavily deformation of the rod, axial forces develop along the axis of the bar, due to both friction and the so called chord effect. The bar is grouted at both ends, while the middle part of the bar is deformed. Both components of friction stresses and embedment stresses act in the direction of the shear load. This contribution has not been taken into account in the calculation, since tests are needed in order to determine the effective contribution.

The third and last possible failure mechanism regards the failure of the tie itself. The shear strength of the bar shall be defined by Eq. [5.9]

$$F_{v,3} = \frac{A_v \cdot \frac{f_y}{\sqrt{3}}}{\gamma_s} \qquad [5.9]$$

Where

A_v is the shear area of the connection element

f_y is the yielding strength of the tie

γ_s is the partial safety factor

The maximum shear load that a single bar can transfer shall be determined by the minimum of the before mentioned resistances:

$$F_{v,Rd} = \min \begin{cases} l_m \cdot d \cdot f_{cd,m} \\ l_c \cdot d \cdot f_{cd,c} \\ f_{cd,m}\, l_n \left(\sqrt{2 + \frac{4M_{y,d}}{\left(df_{cd,m}\, l_n^{\,2}\right)}} - 1 \right) \\ \frac{A_v \cdot \frac{f_y}{\sqrt{3}}}{\gamma_s} \end{cases}$$

5.7.2 Shear Connections and Wall Anchors Calculation Example

CONCRETE TOPPING OVERLAY C30/37	Thickness	$t := 60\ mm$
	Compressive strength	$f_{ck} := 30\ \frac{N}{mm^2}$
	Tensile strength	$f_{ctk.0.05} := 2.0\ \frac{N}{mm^2}$
	Partial factor	$\gamma_c := 1.5$
	Anchorage length	$l_c := 500\ mm$
MASONRY	Compressive strength	$f_k := 4.4\ \frac{N}{mm^2}$
	Partial safety factor	$\gamma_m := 1.7$
	Anchorage length	$l_m := 160\ mm$
	Anchorage partial safety factor	$\gamma_M := 2.2$
STAINLESS STEEL ROD UNS S32304 - ISO 4362-323-04-I	Proof strength	$f_{p.0.2} := 400\ MPa$
	Partial safety factor	$\gamma_s := 1.15$
	Diameter	$d := 16\ mm$
	Yield moment	$M_{y.d} := \frac{f_{p.0.2}}{\gamma_s} \cdot \pi \cdot \frac{d^3}{32} = 139.9\ N \cdot m$
	Characteristic anchorage strength between mortar M20 and concrete 30/37 (Table 3.6 Eurocode 6)	$f_{bok} := 3.4\ MPa$

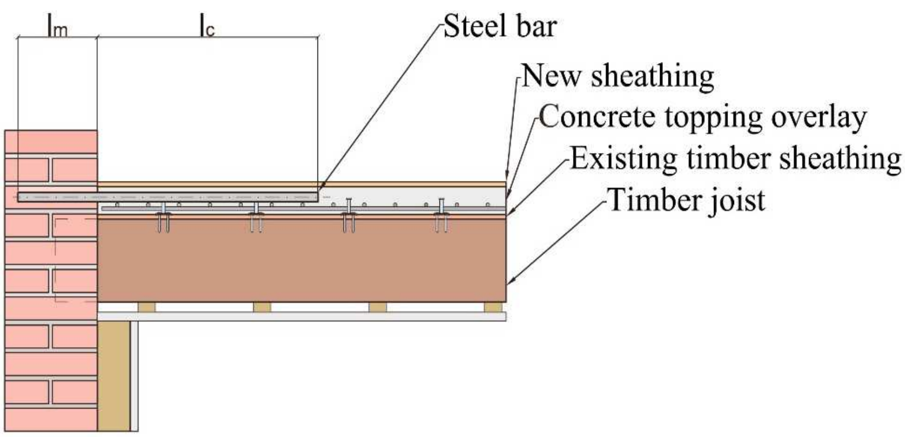


Figure 5-18 Shear Connection Configuration

5.7.3 Shear Bearing Capacity of a Single Stainless-Steel Bar Connection

The shear capacity of a connection bar of 12 mm of diameter results to be 2.6 kN. Therefore the total amount of shear connectors for every floor diaphragm can be determined.

Shear load bearing capacity of a single steel bar connection

$$F_{v.Rd} = \min \begin{cases} l_m \cdot d \cdot \dfrac{f_k}{\gamma_m} = 6.6\ kN \\ l_c \cdot d \cdot \dfrac{f_{ck}}{\gamma_c} = 160\ kN \\ \dfrac{f_k}{\gamma_m} \cdot l_m \cdot d \cdot \left(\sqrt{2 + \dfrac{4\, M_{y.d}}{\left(d \cdot \dfrac{f_k}{\gamma_m} \cdot {l_m}^2 \right)}} - 1 \right) = 3.9\ kN \\ \dfrac{\pi \cdot \dfrac{d^2}{4} \cdot \dfrac{f_{p.0.2}}{\sqrt{3}}}{\gamma_s} = 40.38\ kN \end{cases}$$

Bar shear capacity	$F_{v.Rd} := 3.9\ kN$	
Storey seismic load diaphragm 1 (yellow area)	$V_{d;1} := 58\ kN$	
Storey seismic load diaphragm 2 (green area)	$V_{d;2} := 24.5\ kN$	
Number of shear connectors necessary for diaphragm 1 on one side	$n_1 := \dfrac{V_{d;1}}{F_{v.Rd}} = 14.87$	$n_1 := 15$
Number of shear connectors necessary for diaphragm 2 on one side	$n_2 := \dfrac{V_{d;2}}{F_{v.Rd}} = 6.28$	$n_2 := 7$

5.7.4 Tensile Bearing Capacity of a Single Stainless-Steel Bar Connection

The tensile bearing capacity of a steel bar is determined by the minimum resistance given by the debonding in the masonry and in the concrete.

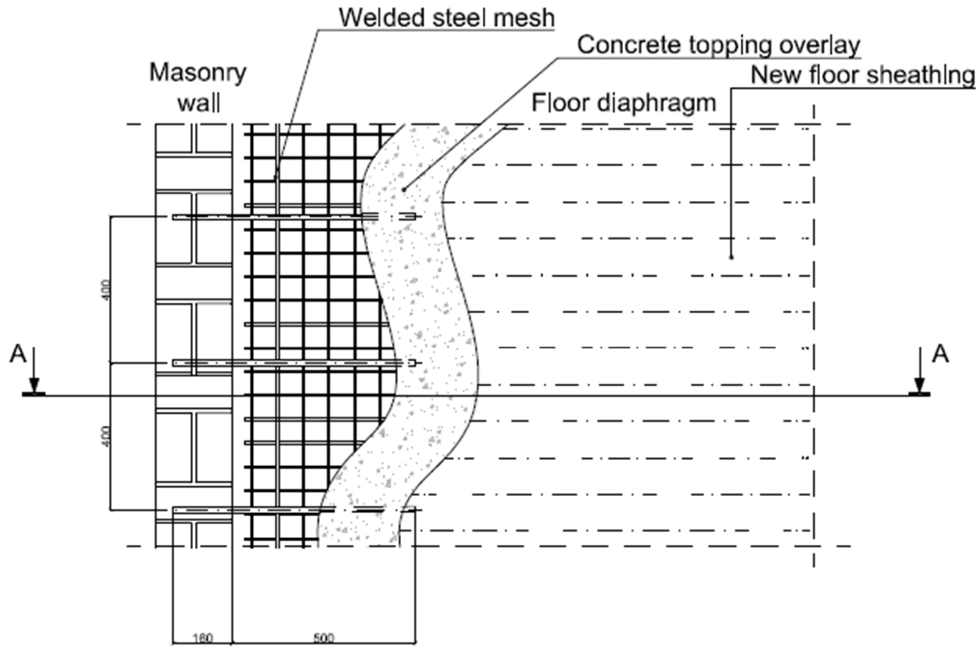
Firstly, the ultimate tensile load has been determined assuming the debonding in the masonry. The assessment of the capacity of the connection has been carried out by means of the procedure related to the reinforcement for reinforced masonry suggested by the Eurocode 6 in Section 8.2.5.

Anchorage length in the masonry	$l_m = 160\ mm$
Ultimate bond stress for high-bond stainless steel bars	$f_{bod} := \frac{f_{bok}}{\gamma_M} = 1.55\ \frac{N}{mm^2}$
Design stress of the bar at the position in the masonry from where the anchorage is measured from	$\sigma_{sd.m} := l_m \cdot 4 \cdot \frac{f_{bod}}{d} = 61.82\ \frac{N}{mm^2}$
Tensile load bearig capacity of a single bar given by the bond in the masonry	$F_{p.d.m} := \sigma_{sd.m} \cdot \frac{d^2}{4} \cdot \pi = 12.4\ kN$

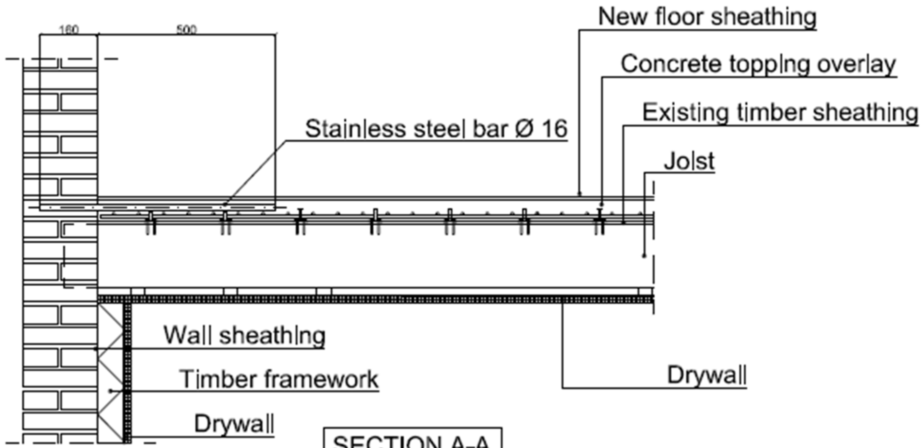
Consequently, the ultimate resistance of the bar given by the bonding with the concrete has been determined by means of method for anchorage length of longitudinal reinforcement described in Eurocode 2 Section 8.4

Coefficients related to the quality of the bond condition	$\eta_1 := 0.7$ $\eta_2 := 1.0$
Coefficient accounting of long term effects	$\alpha_{cc} := 1.0$
Design tensile strength	$f_{ctd} := \alpha_{cc} \cdot \frac{f_{ctk.0.05}}{\gamma_c} = 1.33\ \frac{N}{mm^2}$
Ultimate bond stress for ribbed bars	$f_{bd} := 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 2.1\ \frac{N}{mm^2}$
Anchorage length in the concrete	$l_c = 500\ mm$
Design stress of the bar at the position in the concrete from where the anchorage is measured from	$\sigma_{sd.c} := l_c \cdot 4 \cdot \frac{f_{bd}}{d} = 262.5\ \frac{N}{mm^2}$
Tensile load bearig capacity of a single bar given by the bond in the concrete	$F_{p.d.c} := \sigma_{sd.c} \cdot \frac{d^2}{4} \cdot \pi = 52.8\ kN$

5.8 Sketch of the solution



PLAN VIEW OF THE FLOOR DIAPHRAGM



SECTION A-A

6 LEVEL III RETROFITTING STRATEGY

This chapter introduces issues related to flexible diaphragms in masonry structures and it provides solutions and designs of stiffening techniques for timber diaphragms.

Once diaphragms are fully connected to walls, diaphragms necessity to be enough rigid to distribute equally loads to the linked walls. Consequently, the following level of retrofitting strategy regards measures to enhance the stiffness of diaphragms. Interventions of strengthening diaphragms aim to ensure the box-like response of the building, which would not be possible with flexible diaphragms.

6.1 Effects of Flexible and Rigid Diaphragms

Generally an URM building consists of horizontal elements such as floor and roof diaphragms and vertical members such as bearing walls. Lateral forces acting at diaphragm level, during seismic events are distributed among the vertical load resisting structure by the diaphragm itself.

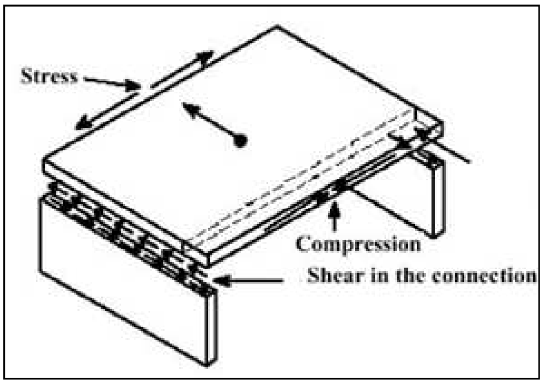


Figure 6-2 Concentration of Stress in Rigid Diaphragm

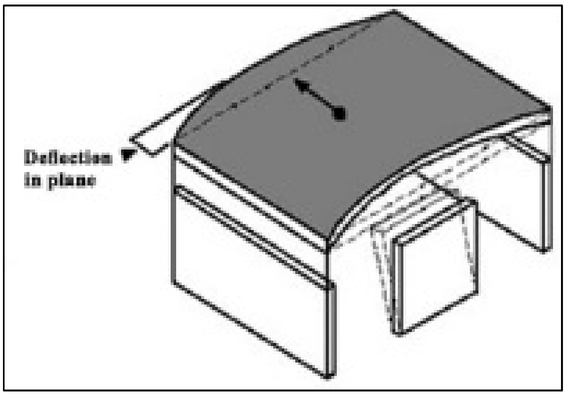


Figure 6-1 In-Plane Deflection of Flexible Diaphragm

Distribution of forces to vertical members depends on both geometry and rigidity of the diaphragm. The diaphragm may be considered as a plate girder, where flanges are boundary members called chords, and the deck or sheathing work as a web.

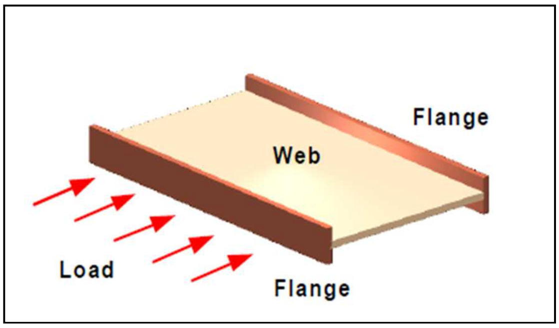


Figure 6-3 Plate Girder under the Horizontal Loading

Typically in diaphragms, the bending resistant element is assumed to be only the chords, whereas bending contribution of the web is neglected. The web instead, is supposed to carry only shear forces induced by the horizontal seismic action. If the web is assumed to be made of homogeneous isotropic elastic material, the stress distribution through the web assumes a parabolic shape, Sang-Cheol Kim [15].

Diaphragms for design purpose are generally treated either as flexible or rigid. Typically in URM buildings, timber diaphragms are considered as flexible, unless bracings are provided in the plane of them. While, cast in-place concrete floor system are usually considered as rigid diaphragms.
The manner of distribution of the total shear force into vertical members (walls) depends on the wall rigidity relative to the diaphragm rigidity.
In buildings with flexible diaphragms, the distribution of shear loads into walls is not dependent from their relative rigidity. These kind of diaphragms act like a series of horizontal beams spanning between walls, as shown in Figure 6-4.

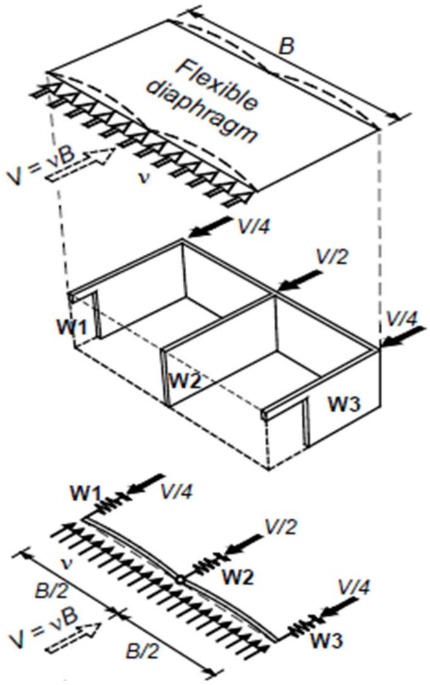


Figure 6-4 Distribution of Lateral Loads in Building with Flexible Diaphragm

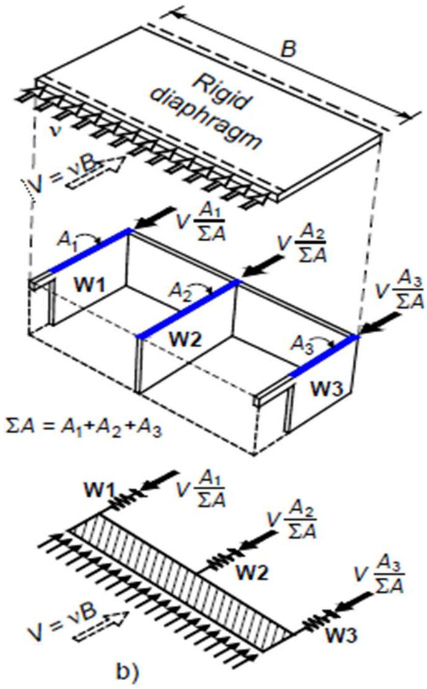


Figure 6-5 Distribution of Lateral Loads in Buildings with Rigid Diaphragms

Where a flexible diaphragm has proper strength, it is able to transfer horizontal loads to walls, but it does not distribute torsional forces to walls perpendicular to the earthquake ground motion.

In buildings with rigid diaphragms, shear loads transferred to walls are directly related to the wall rigidity. In URM low-rise buildings, the wall rigidity should be considered commensurate to the cross sectional area (A) of the element, as shown in Figure 6-5.

The aim of the third level of intervention is to ensure that diaphragms result enough stiff under the seismic excitation.

6.2 International Code Guidelines

6.2.1 Eurocode 1998.1.1

The Eurocode 8 [4] in Chapter 4 lists general rules for design earthquake-resistant buildings. In that section the European code mentions the issues of diaphragm behaviour at storey level.

Building floors and roof play a key role in the seismic behaviour of the structure. The inertia forces are collected and transmitter by the horizontal diaphragms to the vertical structural system by proper designed connections as described in the Chapter 3 of this report.
For an appropriate distribution of inertial forces into vertical structural system, diaphragms should show a sufficient in-plane stiffness. This aspects becomes more important where significant difference in stiffness (openings) or offsets are present in vertical elements above and below the diaphragm.

Furthermore, Eurocode points out that in case of floor diaphragms stiff in their planes, the masses and the moments of inertia of each floor can be concentrated at their centre of gravity.

Eurocode states the condition for a stiff diaphragm as following:

"*The diaphragm is taken as being rigid, if, when it is modelled with its actual in-plane flexibility, its horizontal displacements nowhere exceed those resulting from the rigid diaphragm assumption by more than 10% of the corresponding absolute horizontal displacements in the seismic design situation*".

6.2.2 Asce 41-13 and NZSEE

The American and New Zealand codes use exactly the same classification of diaphragms. They list an upper and a lower bound to distinct flexible and rigid diaphragms.

For a URM house building, the diaphragm should be classified as flexible where the maximum horizontal deformation of the diaphragm along its length is more than twice the average storey drift of the shear walls of the storey immediately below the diaphragm.
On the other hand, a diaphragm is considered as rigid where its maximum lateral deformation is less than half the average storey drift of shear walls of the storey immediately below the diaphragm.

In case that a diaphragms does not full fill the condition to be neither flexible nor rigid, it should be classified as stiff, ASCE 41-13[7].

6.3 Methods of Analysis

Building analysis including its retrofitting interventions shall be carried out to determine stresses and deformations introduced by ground motion relative to the selected seismic hazard level.

The analysis approach shall be computed by means of one of the following procedures:

- Linear Static Procedure
- Linear Dynamic Procedure
- Nonlinear Static Procedure
- Nonlinear dynamic Procedure

Static procedures result to be more appropriate for short and regular buildings where higher mode effects are not significant. Differently, where the design regards a tall building or a building with either torsional irregularities or non-orthogonal systems, a dynamic procedure is required.

In linear analysis procedure the term linear means linearly elastic, though, the analysis procedure may include implicit material nonlinearity of masonry components using properties of cracked sections.

The term non-linear in non-linear analysis approach implies explicit material nonlinearity or inelastic material response, despite, geometric nonlinearity can also be included.

Linear procedures incorporate adjustments to the overall building deformations and material acceptance criteria to allow better consideration of likely nonlinear characteristics of the seismic response, though, they still maintain the traditional use of a linear stress-strain relationship.

When the seismic analysis of the building is accomplished via linear static procedure, the seismic loads, and their distribution over the eight of the building, the corresponding internal forces and system displacements should be determined using a linearly elastic static analysis.

6.4 Period Determination for Linear Static Procedure

The fundamental period of a building may be calculated using one of the following analytical, empirical or approximated methods.

6.4.1 Analytical Method

This method results to be preferred for many buildings, including multi-storey buildings with well-defined framing system. By this method, the period is obtained via eigenvalue analysis by means of effective stiffness and not gross section properties of components. The flexible diaphragms of the building may be modelled as a series of aggregate masses and diaphragm finite elements.

6.4.2 Empirical Method

The use of empirical equations for the building period calculation deliberately underestimates the real period, and generally it results in conservative estimation of seismic loads. Calculation of the building period is based on the stiffness of vertical elements, which essentially underestimates the period of the actual dynamic response and overestimate the seismic action.

The empirical formula furnished by Eurocode 8 [4] results to be equivalent to those present on the American and New Zealand codes for buildings with eights up to 40 m, despite, the Eurocode 8 proposes further specifications in case of masonry buildings:

$$T_1 = C_t \cdot H^{\frac{3}{4}} \quad [6.1]$$

Where

$$C_t = \frac{0.075}{\sqrt{Ac}} \quad [6.2]$$

For concrete or masonry shear walls buildings only

$$A_c = \Sigma\left[A_i \cdot \left(0.2 + \left(\frac{l_{wi}}{H}\right)^2\right)\right] \quad [6.3]$$

Where

A_c is the total effective area of shear walls in the first storey of the building

A_i is the effective cross-sectional area of shear wall *i* in the direction considered in the first storey of the building

l_{wi} is the length of shear wall i in the first storey in direction parallel to the applied force.

H is the height of the building from the foundation or from the top of a rigid basement

6.4.3 Approximate Method

Approximate method is a proper procedure for systems characterized from rigid vertical elements and flexible diaphragms, in which, the dynamic response of the system is concentrated in the diaphragms.

The fundamental period of a building can be approximated by means of the following different approximate formulas.

- In case of one-storey building:

$$T = \left(0.1\Delta_W + 0.078\Delta_D\right)^{0.5} \quad [6.4]$$

Where Δ_W and Δ_D are respectively, wall in-plane and diaphragm displacements (in inches) because of a lateral force in the direction under consideration equal to the weight tributary to the diaphragm.

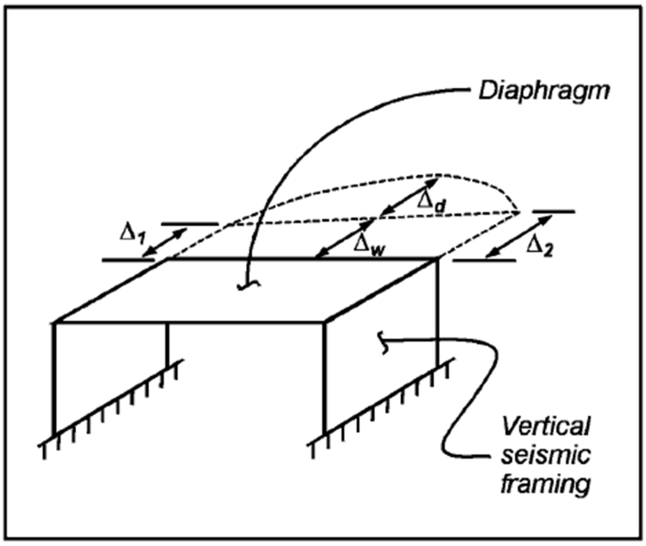


Figure 6-6 Diaphragm and Wall Displacement Terminology

In case of a multiple-span diaphragms, the period should be calculated for each diaphragm span, and the period which maximizes the seismic force shall be used for analysis of all diaphragm spans and walls in the building. In this situation, the lateral force equals to the weight tributary to the diaphragm span under consideration.

- In the situation of an unreinforced masonry building with single-span flexible diaphragms up to six stories:

$$T = \left(3.07\Delta_D\right)^{0.5} \qquad [6.5]$$

Where Δ_d is the maximum in-plane diaphragm displacement (in meters) caused by the lateral force in the direction under consideration, with the lateral force equal to the weight tributary to the diaphragm.
In this method, wall deformations are considered to be negligible compared with diaphragm deflection.

When diaphragm displacement is determined to estimate the fundamental period of the building using Eq. [6.4] and Eq. [6.5], the diaphragm should be assumed to conserve an elastic behaviour under the seismic lateral force.

- For every typology of building, the determination of the fundamental period of vibration can be approximated by the Rayleigh's method:

$$T = 2\pi \left[\frac{\sum_{i=1}^{n} w_i \delta_i^2}{\sum_{i=1}^{n} F_i \delta_i} \right]^{1/2} \qquad [6.6]$$

Where

w_i =Portion of the effective seismic weight located on or assigned to level *i*

δ_i =Displacement at floor *i* caused by lateral force F_i

F_i =Lateral force applied at level *I* defined by a linear distribution

n=Total number of stories in the vertical seismic framing above the base

6.5 Determination In-Plane Diaphragm Displacement

The behaviour of horizontal timber diaphragms is influenced by different parameters regarding the characteristic of diaphragms members and its geometry. Important factors result to be the type of sheathing, the amount of fasteners used, the presence of perimeter chord or flange members. Never the less, the ratio span to depth of the diaphragms besides the presence of openings also effect the behaviour and its shear capacity.

Sheathed diaphragms are typically affected by high flexibility with a long period of vibration when lacking of a proper retrofitting measure.
For the most common timber diaphragm sheathings, considering the typical configuration, the American code provides standard values of shear stiffness.
The in-plane displacement performed by the diaphragm in each direction can be evaluated using the following formula:

$$\Delta_D = \frac{V_u}{K_D} \qquad [6.7]$$

Where

V_u is the diaphragm lateral load

K_D is the diaphragm stiffness

Using the shear stiffness values, G_d, the diaphragms stiffness K_D can be calculated for each diaphragm in each direction using the relation in accordance to the American code Fema 356[16] (Eq.[6.8]), which assumes the diaphragm as a simply supported beam.

$$K_D = \frac{2bG_d}{L} \quad [6.8]$$

Where

b is the diaphragm width (m)
G_d is the diaphragm shear stiffness (N/m)
L is the diaphragm span between shear walls (m)

The diaphragm shear stiffness G_d given by the American code IEBC [17], for the most common timber diaphragm is presented in the Table 6-1:

Diaphragm Type		Shear Stiffness, G_d (KN/m)	Yield Strength, R_n (N/m)
Single Straight Sheathing		350	1750
Double Straight Sheathing	Chorded	2600	8750
	Unchorded	1200	5850
Single Diagonal Sheathing	Chorded	1400	8750
	Unchorded	700	6130
Double Sheathing with Straight Sheathing or Flooring Above	Chorded	3200	13100
	Unchorded	1600	9130
Double Diagonal Sheathing	Chorded	3100	13100
	Unchorded	1600	9130

Table 6-1 Default Strength Values For Existing Timber Diaphragms IEBC [17]

6.6 Diaphragm Seismic Load Calculation

After that the fundamental period of vibration of the building has been determined, design seismic loads may be determined following the approach described by Eurocode 8 [4].

6.6.1 Seismic Base Shear Force

Eurocode 8 estimates seismic actions by means of the seismic base shear force for every direction in which the building is analysed. The shear force acting on the building is determined using the Eq [6.9]

$$F_b = S_d(T_1) \cdot m \cdot \lambda \quad [6.9]$$

Where

$S_d\left(T_1\right)$ is the ordinate of the design spectrum at period T_1

m is the total mass of the building above the foundation

λ is the correction factor, the value of which is equal to: $\lambda=0.85$ if $T_1 \leq 2T_C$ and the building has more than two storeys, otherwise $\lambda=1$

6.6.2 Response Spectrum

The response spectrum is one of the most important tools in order to analyse the behaviour of a structure during a seismic event. It represents the peak response of a series of oscillators with different natural frequencies forced into motion by the same input. Therefore, the response spectrum relates the frequency of an oscillator to its response when loaded. The plot given by these oscillators is generally used to define the response of any linear system, given its natural frequency of oscillation.

Eurocode 8 represents the earthquake motion at any point on the surface by means of an elastic ground acceleration response spectrum, so called "Elastic Response Spectrum". It is shown in Figure 6-4.

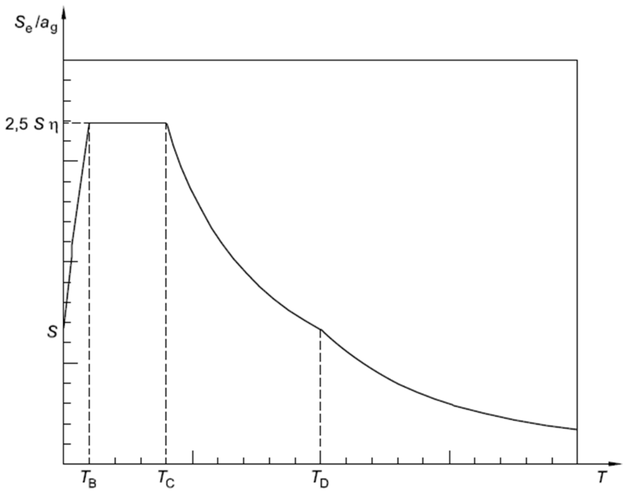


Figure 6-4 Shape of the elastic response spectrum present on the Eurocode 8

Hence, horizontal and vertical components of the seismic action may be derived by the elastic response spectrum where different values of T_B, T_C, T_D, and S depend on the ground type.

Since in reality a building does not behave has a perfect linear elastic structure, the elastic response spectrum is reduced to a "Design Spectrum for Elastic Analysis". The design spectrum takes into account the capacity of the structure to dissipate energy through ductile behaviour of its members or other mechanism. This is obtained by the introduction of the factor q.

The so-called behaviour factor q approximates the ratio of the seismic loads in case of a perfectly linear elastic structure with 5% of viscos damping, to the design seismic loads obtained by a conventional elastic analysis model. Values of the seismic behaviour factor are related to the material and structural system of the building.

Consequently, in seismic design, once the fundamental period of vibration of the structure is known, the design spectrum for horizontal components shall be determined by the following equations, which vary depending the value of the fundamental period.

$$0 \le T \le T_B : S_d(T) = a_g \cdot S \left[\frac{2}{3} + \frac{T}{T_B}\left(\frac{2.5}{q} - \frac{2}{3} \right) \right] \quad [6.10]$$

$$T_B \le T \le T_C : S_d(T) = a_g \cdot S \cdot \frac{2.5}{q} \quad [6.11]$$

$$T_C \le T \le T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q}\left[\frac{T_C}{T}\right] \\ \ge \beta \cdot a_g \end{cases} \quad [6.12]$$

$$T_D \le T : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q}\left[\frac{T_C T_D}{T^2}\right] \\ \ge \beta \cdot a_g \end{cases} \quad [6.13]$$

Where

a_g is the design peak ground acceleration
S, T_B, T_C, T_D are parameters regarding the ground type
$S_d(T)$ is the design spectrum
q is the behaviour factor
β is the lower bound factor for the horizontal design spectrum, recommended to be equal to 0.2

6.6.3 Distribution of the Horizontal Seismic Forces

Seismic design through linear static procedure involves the lateral force method analysis. It approximates the seismic action by means of static horizontal loads. Consequently, the effect of horizontal seismic actions are determined on the basis of equivalent horizontal loads applied at storey levels, where the masses of the building are concentrated.

When horizontal displacements increase linearly along the height of the building, horizontal loads shall be determined by means of Eq. [6.14]

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\Sigma\, s_j \cdot m_j} \qquad [6.14]$$

Where

F_i is the horizontal force acting on storey i
F_b is the seismic base shear
s_i; s_j are the displacements of masses m_i, m_j in the fundamental mode shape
m_i; m_j are the storey masses
If the fundamental mode shape is approximated by horizontal displacements increasing linearly along the eight of the building, horizontal loads shall be determined by means of Eq. [6.15].

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\Sigma\, z_j \cdot m_j} \qquad [6.15]$$

Where

z_i; z_j are the eight of the masses m_i; m_j above the level of application of the seismic action.

When horizontal loads F_i are determined in accordance with Eq [6.14], or Eq [6.15]., they shall be considered distributed on the lateral load resisting members assuming horizontal diaphragms behave as rigid in their in-plane direction.

6.6.4 Calculation Example Fundamental Period by means of Rayleigh's Method

In this example, the fundamental period of vibration of a typical Dutch terraced house is presented. In order to define the total stiffness of the building, the stiffness of every single pier and shear wall has been determined. Every element has been named by its in-plane direction as shown in Figure 6-5.

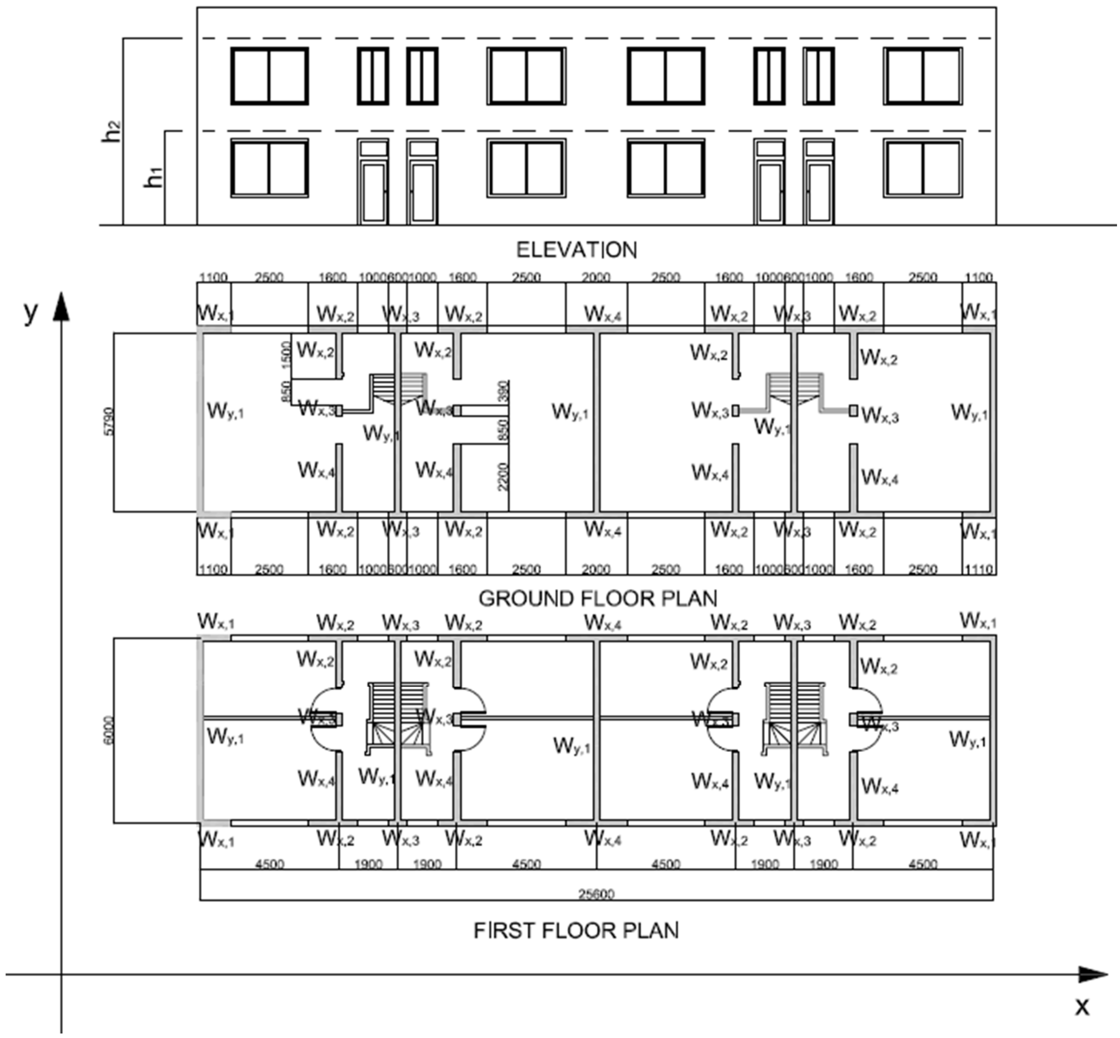


Figure 6-5 Building Plans and Elevation

Masonry modulus of elasticity	E	4410	N/mm^2
Height of the first storey	h1	3000	mm
Height of the second storey	h2	6000	mm

Table 6-2 2 Data used to determine the wall stiffness

Every wall or pier has been considered behaving as a cantilever beam. After that the stiffness for every element has been determined by means of Eq.[6.16], the total stiffness of storeys has been calculated by adding contribution of each element in the relevant direction.

$$k = \frac{3EIA_v}{h^3 A_v + 7.5Ih} \qquad [6.16]$$

Where:

E is the masonry modulus of elasticity
I is the moment of inertia of the pier
A_v is the shear area of the pier ($\frac{5}{6} b \cdot l$)
h is the height of the storey

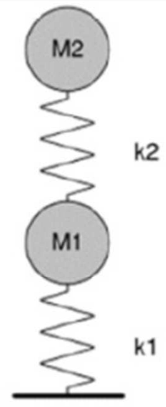
Wall stiffness considered as cantilever beam in the x direction										
$W_{x.1}$	$l_{x.1}$	1100	mm		$I_{x.1}$	2,329E+10	mm^4	$K_{x.1.1}$	1,037E+04	N/mm
	$b_{x.1}$	210	mm		$A_{vx.1}$	1,925E+05	mm^2	$K_{x.1.2}$	1,392E+03	N/mm
$W_{x.2}$	$l_{x.2}$	1600	mm		$I_{x.2}$	7,168E+10	mm^4	$K_{x.2.1}$	2,895E+04	N/mm
	$b_{x.2}$	210	mm		$A_{vx.2}$	2,800E+05	mm^2	$K_{x.2.2}$	4,168E+03	N/mm
$W_{x.3}$	$l_{x.3}$	600	mm		$I_{x.3}$	3,780E+09	mm^4	$K_{x.3.1}$	1,798E+03	N/mm
	$b_{x.3}$	210	mm		$A_{vx.3}$	1,050E+05	mm^2	$K_{x.3.2}$	2,298E+02	N/mm
$W_{x.4}$	$l_{x.4}$	2000	mm		$I_{x.4}$	1,400E+11	mm^4	$K_{x.4.1}$	5,145E+04	N/mm
	$b_{x.4}$	210	mm		$A_{vx.4}$	3,500E+05	mm^2	$K_{x.4.2}$	7,915E+03	N/mm
Stiffness ground floor in the x direction					$K_{1.x}$	383146	N/mm			
Stiffness first floor in the x direction					$K_{2.x}$	55661	N/mm			
Wall stiffness considered as cantilever beam in the y direction										
$W_{y.1}$	$l_{y.1}$	5790	mm		$I_{Y.1}$	3,397E+12	mm^4	$K_{y.1,1}$	4,387E+05	N/mm
	$b_{y.1}$	210	mm		$A_{vY.1}$	1,013E+06	mm^2	$K_{y.1.2}$	1,225E+05	N/mm
$W_{y.2}$	$l_{y.2}$	1500	mm		$I_{Y.2}$	5,906E+10	mm^4	$K_{y.2.1}$	2,437E+04	N/mm
	$b_{y.2}$	210	mm		$A_{vY.2}$	2,625E+05	mm^2	$K_{y.2.2}$	3,456E+03	N/mm
$W_{y.3}$	$l_{y.3}$	390	mm		$I_{Y.3}$	1,038E+09	mm^4	$K_{y.3.1}$	5,023E+02	N/mm
	$b_{y.3}$	210	mm		$A_{vY.3}$	6,825E+04	mm^2	$K_{y.3.2}$	6,338E+01	N/mm
$W_{Y.4}$	$l_{Y.4}$	2200	mm		$I_{Y.4}$	1,863E+11	mm^4	$K_{y.4.1}$	6,506E+04	N/mm
	$b_{Y.4}$	210	mm		$A_{vY.4}$	3,850E+05	mm^2	$K_{y.4.2}$	1,037E+04	N/mm
Stiffness ground floor in the y direction					$K_{1.y}$	2553462	N/mm			
Stiffness first floor in the y direction					$K_{2.y}$	668046	N/mm			

Table 6-3 Stiffness of wall piers

BUILDING PERIOD CALCULATION

Wall stiffness in x direction $K_{1.x} := 383146 \ \frac{kN}{m}$ $K_{2.x} := 55661 \ \frac{kN}{m}$

Wall stiffness in the y direction $K_{1.y} := 2553462 \ \frac{kN}{m}$ $K_{2.y} := 668046 \ \frac{kN}{m}$



Building dimensions $l_x := 25600 \ mm$
$l_y := 6000 \ mm$

Parapet height $h_p := 1000 \ mm$
First storey height $h_2 := 6000 \ mm$
Second storey height $h_1 := 3000 \ mm$
Wall thickness $t := 210 \ mm$

Diaphragm dead load $g_{k.j} := 100 \ \frac{kg}{m^2}$ Ceramic tiles and glue: 55 Kg/m2
Timber floor single sheathing: 45 Kg/m2

Live load $q_{k.i} := 200 \ \frac{kg}{m^2}$

Weight of diaphragm per sq meter (see Annex A) $q_{G.d} := g_{kj} + (0.3 \cdot 1.0 \cdot q_{k.i}) = 160 \ \frac{kg}{m^2}$

Masonry weight $\rho_m := 1800 \ \frac{kg}{m^3}$

Diaphragm mass
$m_d := l_x \cdot l_y \cdot q_{G.d} = (2.458 \cdot 10^4) \ kg$ Mass of floor slabs $m_1 := m_d$ $m_2 := m_d$

Building mass
(openings have not been considered)
$m_b := \rho_m \cdot t \cdot ((2 \cdot h_2 \cdot l_x) + (9 \cdot h_2 \cdot l_y) + (h_p \cdot (2 \cdot l_x + 2 \cdot l_y))) + (3 \cdot m_d) = (3.362 \cdot 10^5) \ kg$

In the first iteration of the Rayleigh's method to determine the natural period of the building, horizontal loads have been assumed. Horizontal loads are equals to the weight of the floor slabs, and storey displacements are obtained by applying the assumed horizontal loads at storey levels. A linear distribution of loads have been supposed along the height of the building.

Displacements due to loads equal to the diaphragm masses

$$\delta_{1.x} := \frac{(m_1 + m_2) \cdot g}{K_{1.x}} = 1.26 \ mm$$

$$\delta_{2.x} := \delta_{1.x} + \frac{m_2 \cdot g}{K_{2.x}} = 5.59 \ mm$$

Fundamental period of vibration

$$T_{1.x} := 2 \cdot \pi \cdot \sqrt{\frac{(m_1 \cdot \delta_{1.x}^{\ 2} + m_2 \cdot \delta_{2.x}^{\ 2})}{g \cdot (m_1 \cdot \delta_{1.x} + (m_1 + m_2) \cdot \delta_{2.x})}} = 0.103 \ s$$

With the period determined by the first iteration it is possible to calculate the response spectrum acceleration. Forces and displacements used in the second iteration are based on the response spectrum derived from the first period obtained in the first iteration.

Correction factor $\lambda := 0.85$ $\beta := 0.2$

Parameters describing the recommended Type 2 elastic response spectra for ground type E $T_B := 0.05\ s$ $T_C := 0.25\ s$ $T_D := 1.2\ s$ $S := 1.6$

$T_{1.x} = 0.103\ s$ Therefore $T_B \le T_{1.x} \le T_C$

Design ground acceleration $a_g := 0.234\ g$
Behaviour factor for URM $q := 1.5$

Response spectrum acceleration $S_{dx} := a_g \cdot S \cdot \frac{2.5}{q} = 6.12\ \frac{m}{s^2}$

Base shear force $F_{b.1} := S_{dx} \cdot m_b \cdot \lambda = 1749\ kN$

Horizontal force first storey $F_{x.1} := F_{b.1} \cdot \left(\frac{h_1 \cdot m_1}{(h_1 \cdot m_1) + (h_2 \cdot m_2)} \right) = 583\ kN$

Horizontal force second storey $F_{x.2} := F_{b.1} \cdot \left(\frac{h_2 \cdot m_2}{(h_1 \cdot m_1) + (h_2 \cdot m_2)} \right) = 1166\ kN$

Period in x direction 2nd iteration

Displacements due to the seismic loads

$$\delta_{1.x} := \frac{F_{x.1} + F_{x.2}}{K_{1.x}} = 4.564\ mm$$

$$\delta_{2.x} := \delta_{1.x} + \frac{F_{x.2}}{K_{2.x}} = 25.51\ mm$$

Fundamental period of vibration $T_{2.x} := 2 \cdot \pi \cdot \sqrt{\frac{\left(m_1 \cdot \delta_{1.x}^2 + m_2 \cdot \delta_{2.x}^2\right)}{\left(F_{x.1} \cdot \delta_{1.x} + F_{x.2} \cdot \delta_{2.x}\right)}} = 0.142\ s$

Similarly as for the first iteration, a response spectrum is determined on the basis of the natural period of the building obtained by the second iteration.

Response spectrum acceleration $S_{dx} := a_g \cdot S \cdot \frac{2.5}{q} = 6.12\ \frac{m}{s^2}$

Base shear force $F_{b.1} := S_{dx} \cdot m_b \cdot \lambda = 1749\ kN$

Horizontal force first storey $F_{x.1} := F_{b.1} \cdot \left(\frac{\delta_{1.x} \cdot m_1}{(\delta_{1.x} \cdot m_1) + (\delta_{2.x} \cdot m_2)} \right) = 265\ kN$

Horizontal force second storey $F_{x.2} := F_{b.1} \cdot \left(\frac{\delta_{2.x} \cdot m_2}{(\delta_{1.x} \cdot m_1) + (\delta_{2.x} \cdot m_2)} \right) = 1483\ kN$

Period in x direction 3rd iteration

Displacements due to the seismic loads

$$\delta_{1.x} := \frac{F_{x.1}+F_{x.2}}{K_{1.x}} = 4.56\ mm$$

$$\delta_{2.x} := \delta_{1.x} + \frac{F_{x.2}}{K_{2.x}} = 31.21\ mm$$

Fundamental period of vibration in x direction

$$T_{3.x} := 2 \cdot \pi \cdot \sqrt{\frac{\left(m_1 \cdot \delta_{1.x}^{\ 2} + m_2 \cdot \delta_{2.x}^{\ 2}\right)}{\left(F_{x.1} \cdot \delta_{1.x} + F_{x.2} \cdot \delta_{2.x}\right)}} = 0.143\ s$$

Period in y direction 1st iteration

Similarly, the same iterative procedure followed to determine the fundamental period of the building in the x direction has been adopted to calculate the period of the building in its y direction.

Displacements due to loads equal to the diaphragm masses

$$\delta_{1.y} := \frac{\left(m_1+m_2\right) \cdot g}{K_{1.y}} = 0.19\ mm$$

$$\delta_{2.y} := \delta_{1.y} + \frac{m_2 \cdot g}{K_{2.y}} = 0.55\ mm$$

Fundamental period of vibration in x direction

$$T_{1.y} := 2 \cdot \pi \cdot \sqrt{\frac{\left(m_1 \cdot \delta_{1.y}^{\ 2} + m_2 \cdot \delta_{2.y}^{\ 2}\right)}{g \cdot \left(m_1 \cdot \delta_{1.y} + \left(m_1+m_2\right) \cdot \delta_{2.y}\right)}} = 0.032\ s$$

Correction factor $\lambda := 0.85$ $\beta := 0.2$

Parameters describing the recommended Type 2 elastic response spectra for ground type E $T_B := 0.05\ s$ $T_C := 0.25\ s$ $T_D := 1.2\ s$ $S := 1.6$

$T_{1.y} = 0.032\ s$ Therefore $0 \le T_{1.y} \le T_B$

Design ground acceleration $a_g := 0.234\ g$

Behaviour factor for URM $q := 1.5$

Response spectrum acceleration

$$S_{d.y} := a_g \cdot S \cdot \left(\frac{2}{3} + \frac{T_{1.y}}{T_B} \cdot \left(\frac{2.5}{q} - 2\right)\right) = 1.65\ \frac{m}{s^2}$$

Base shear force $F_{b.1} := S_{d.y} \cdot m_b \cdot \lambda = 472\ kN$

Horizontal force first storey (see Figure below)

$$F_{y.1} := F_{b.1} \cdot \left(\frac{\delta_{1.y} \cdot m_1}{\left(\delta_{1.y} \cdot m_1\right) + \left(\delta_{2.y} \cdot m_2\right)}\right) = 121\ kN$$

Horizontal force second storey (see Figure below)

$$F_{y.2} := F_{b.1} \cdot \left(\frac{\delta_{2.y} \cdot m_2}{\left(\delta_{1.y} \cdot m_1\right) + \left(\delta_{2.y} \cdot m_2\right)}\right) = 352\ kN$$

Period in y direction 2nd iteration

Displacements due to the seismic loads

$$\delta_{1.y} := \frac{F_{y.1} + F_{y.2}}{K_{1.y}} = 0.18\ mm$$

$$\delta_{2.y} := \delta_{1.y} + \frac{F_{y.2}}{K_{2.y}} = 0.71\ mm$$

Fundamental period of vibration in x direction

$$T_{2.y} := 2 \cdot \pi \cdot \sqrt{\frac{\left(m_1 \cdot \delta_{1.y}^{\ 2} + m_2 \cdot \delta_{2.y}^{\ 2}\right)}{\left(F_{y.1} \cdot \delta_{1.y} + F_{y.2} \cdot \delta_{2.y}\right)}} = 0.044\ s$$

$T_{2.y} = 0.044\ s$ Therefore $0 \le T_{2.y} \le T_B$

Response spectrum acceleration

$$S_{d.y} := a_g \cdot S \cdot \left(\frac{2}{3} + \frac{T_{2.y}}{T_B} \cdot \left(\frac{2.5}{q} - 2\right)\right) = 1.37\ \frac{m}{s^2}$$

Base shear force

$$F_{b.1} := S_{d.y} \cdot m_b \cdot \lambda = 393\ kN$$

Horizontal force first storey

$$F_{y.1} := F_{b.1} \cdot \left(\frac{\delta_{1.y} \cdot m_1}{\left(\delta_{1.y} \cdot m_1\right) + \left(\delta_{2.y} \cdot m_2\right)}\right) = 81\ kN$$

Horizontal force second storey

$$F_{y.2} := F_{b.1} \cdot \left(\frac{\delta_{2.y} \cdot m_2}{\left(\delta_{1.y} \cdot m_1\right) + \left(\delta_{2.y} \cdot m_2\right)}\right) = 312\ kN$$

Period in y direction 3rd iteration

Displacements due to the seismic loads

$$\delta_{1.y} := \frac{F_{y.1} + F_{y.2}}{K_{1.y}} = 0.15\ mm$$

$$\delta_{2.y} := \delta_{1.y} + \frac{F_{y.2}}{K_{2.y}} = 0.62\ mm$$

Fundamental period of vibration in x direction

$$T_{3.y} := 2 \cdot \pi \cdot \sqrt{\frac{\left(m_1 \cdot \delta_{1.y}^{\ 2} + m_2 \cdot \delta_{2.y}^{\ 2}\right)}{\left(F_{y.1} \cdot \delta_{1.y} + F_{y.2} \cdot \delta_{2.y}\right)}} = 0.044\ s$$

6.7 Timber Panel Overlays above the Existing Timber Floor

Diaphragm flexibility can be reduced by superposition of a new layer of wood planks or plywood panels over the existing sheathing.
The new wood structural panels are placed over the existing straight or diagonal sheathing, and stapled or nailed to the existing framing members through the old sheathing.

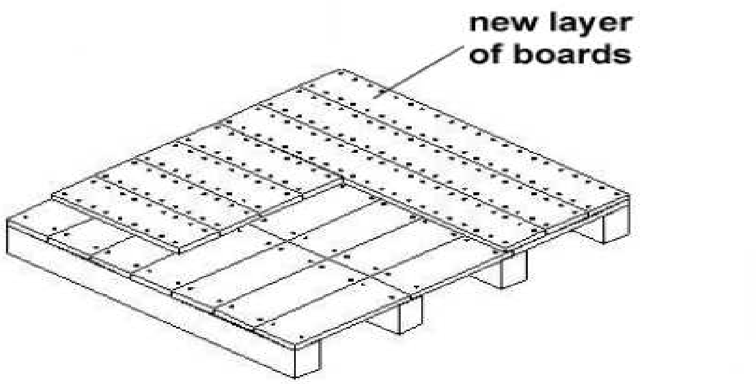


Figure 6-7 Timber Panel Overlay above the Existing Timber Floor

6.7.1 Design Considerations

In the design calculation of flexibility for existing floor and for new timber layer three different contributions shall be taken into account.

Brignola [18] explains the three different contribution by evaluation of the overall flexibility of a single straight sheathing. Under in-plane loading condition, every single board performs deformation due to its both flexural and shear flexibility, moreover it effects a rigid rotation due to the nail slip. The Figure 6-15 shown the in-plane deformation of a straight sheathing timber floor.

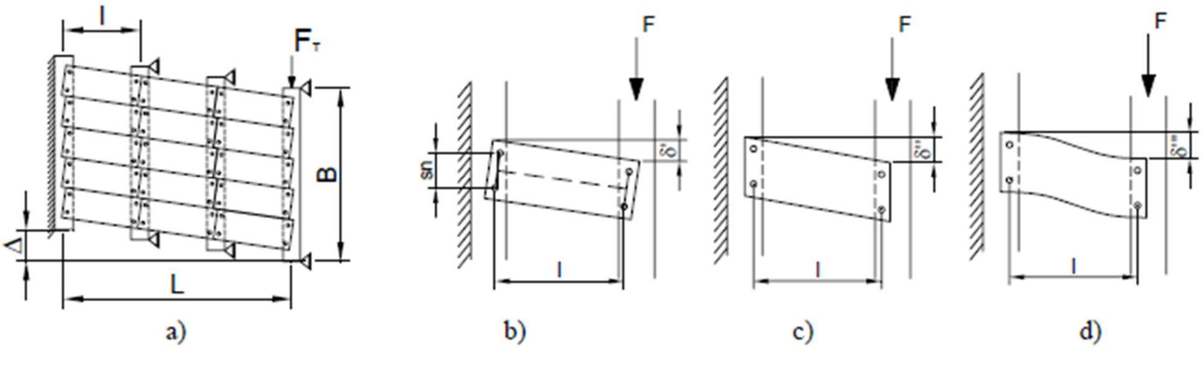


Figure 6-8 In Plane Deformation of a Single Straight Sheathing Timber Floor; B) Rigid Rotation of the Board due to Nails Slip; C) Board Shear Deformation; D) Board Flexural Deformation, Brignola [18].

6.7.2 Detailing and Construction Considerations

- *Staples, short nails, regular length nails:* Attachment of timber overlay is recommended the use of nail sizes of d8 and d10, which require respectively 35 and 40 mm of penetration depth in order to assure no slip. When the overlay is applied on a diaphragms that is exposed from below, particular attention must be paid to avoid nail penetrations.
- *Partitions:* The diaphragm provides stiffness and direct load path when is continuous between walls. Often in existing buildings is possible to find partition walls on the floor. Removing these partitions during the rehabilitation is recommended, in this way no interruptions in the overlay occur. If the partitions are to remain during the rehabilitation a shear transfer connection from one side to the other of the partition is needed. See Figure 6-6 for the shear connection detail.

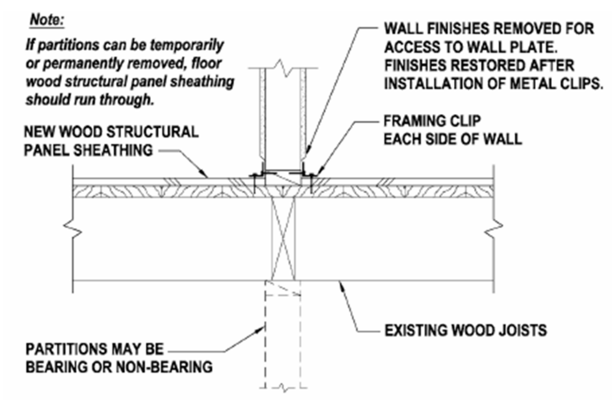


Figure 6-9 Shear Transfer Connection at Partition Basis

- *Weight*: The addition of wood panel sheathing over the existing sheathings increase the weight of the diaphragm. This typically does not represent an issue, but engineer should consider this aspect, especially in the calculation of the horizontal seismic load.
- *Location of diaphragm*: The following figure shows the retrofitting structural panel added at the top of the floor. This is the most used solution, but when there is the needing to preserve finishes on the top of the floor, the underside of the floor can be enhanced.

Figure 6-60 Shear Connection Details

6.7.3 Cost and Disruption:

Necessity of access to either the top or underside of the floor can result a significant disruption to occupants, besides, noise of sawing and hammering. When the building remains occupied during rehabilitation, the work can be phased by floor or wing to minimize disruption for occupants.

6.8 Concrete Topping Overlay Above of the Existing Floor

Where the diaphragms do not show proper stiffness in the in-plane direction, a concrete topping overlay may represent the solution in both cases of concrete and timber diaphragms. A thin concrete slab is sufficient to guarantee adequate in-plane stiffness of the diaphragm, but attention shall be paid about instability of the concrete plate when subjected to in-plane loads. Consequently, proper connection elements need to be designed in order to bond the concrete overlay with the main frame of the diaphragm.

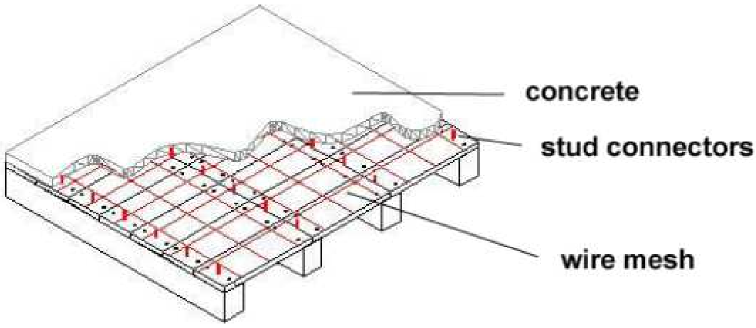


Figure 6-12 Concrete Topping Overlay above the Existing Floor

6.8.1 Design Consideration

The concrete overlay can reach high level of compression in the in-plane direction when the connection between concrete overlay and diaphragm frame is well designed.

It is assumed that connectors installed on the existing frame elements do not transfer shear forces. Seismic loads run directly in the concrete slab to shear walls without transfer any force to the existing diaphragm frame, connectors do not experience any horizontal force. Connectors play the role to hold the concrete overlay in its position, which, due the high compression may buckle. The connectors are indeed designed on the basis of the tensile and compression loads due to the concrete topping.

6.8.2 Detailing and Construction Consideration

- *Aesthetics:* The diaphragm does not change its aesthetic characteristic, since the intervention regards its top surface only which can be renovated by placing new tiles or timber planks. The only visible modification regards the height of doors, which shall be reduced after the retrofit.
- *Installation approach*: In the construction process of the concrete topping overlay above the diaphragm, a number of phases should be followed.
  1. *Propping.* The diaphragm frame should be supported. This phase is particularly important for composite slabs. The skipping of this phase means a reduction of increment in resistance of the diaphragm.
  2. *Removal* of non-structural elements carried by the diaphragm
  3. *Waterproofing.* A water proof sheet is placed to preserve the existing frame.

4. *Installation of the connectors.*
5. *Placing of steel welded wire mesh.* The mesh should intersect connectors.
6. *Casting of concrete slab*.
7. *Waiting of the hardening of the concrete.*
8. *Propping removal.*

- *Materials:* Increment of the diaphragm weight leads to a higher seismic load, therefore preventative measures must be taken. The use of a 50 mm thickness standard concrete overlay on an existing diaphragm involves an increase of 125 Kg/m^2. This increment can be reduced by the use of lightweight concrete, which results to be around 70 Kg/m^2. In the load computation of this solution others aspects should be accounted. When particular attention is paid on the reduction of weight due to finishing and filling, the retrofitted diaphragm may results lighter than the existing one. Often masonry buildings presents diaphragms with heavy concrete and sand layer, partition walls and ceramic tiles.
  On the market is possible to find lightweight solutions for many elements, therefore it is a good approach lightening not only structural elements, but finishing as well.
- *Connectors:* Typology of connectors changes depending on diaphragm frame material. For the retrofit of timber diaphragms, one possible type of connectors is shown in the Figure 6-7. This type of connectors are placed by mechanical union, without any chemical bonding, which allows a fast and reversible placing.
  In case of concrete diaphragms, one typology of connectors are shown in the Figure 6-8. Connectors are directly screwed in the concrete beams and fixed by a locking plate. All parts of the connector are galvanized.
- *Access and realization issues:* Where propping is not possible because the space under the diaphragm is not accessible. It is possible to support the diaphragm frame by means of wires hung to the above diaphragm.
- *Dimension:* The Italian code defines as rigid a retrofitted diaphragm with a concrete overlay of 50 mm thickness by the least. Where the overlay is connected to the frame by means of a proper designed number of shear connectors.

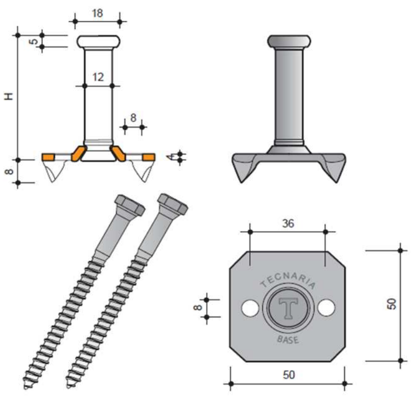


Figure 6-7 Connectors for Retrofitting of Timber Diaphragms

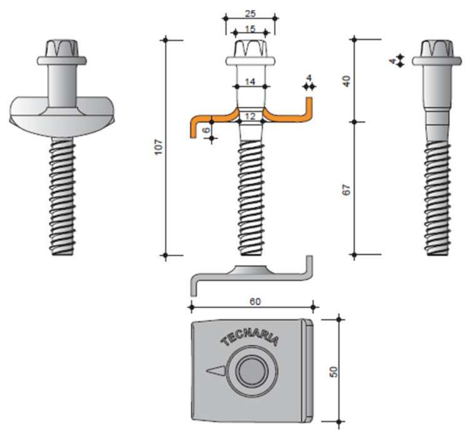


Figure 6-8 Connectors for Retrofitting of Concrete Diaphragms

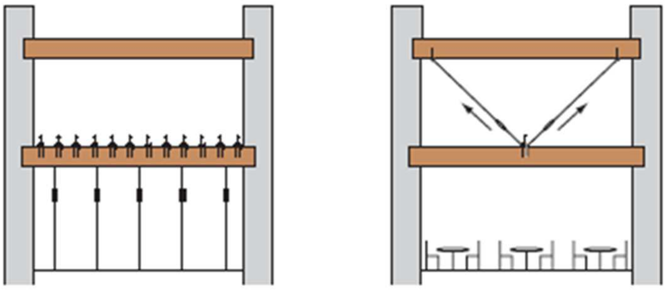


Figure 6-15 Possible Solutions to Support the Diaphragm to Retrofit

- *Corrosion and moisture consideration:* The use of a waterproof sheeting is recommended in the case of timber diaphragm. This is placed in order to avoid that the timber panel sheathing absorbs the hydration water of the concrete. The waterproof sheeting should be breathable in the upward direction to avoid vapour condensation on the below ceiling. Nespolo [19].

6.8.3 Cost and Disruption

Necessity of access to both the top and underside of the diaphragm can result an issue to the occupants. When the building remains occupied during rehabilitation the work can be phased by room to minimize the disruption for the occupants.

6.9 Conclusion about Level IV Retrofitting Strategy

URM buildings are considered to have their masses concentrated at floor levels. Consequently during a seismic event, the so-called seismic loads act at these levels. Resistant members are represented by walls which shall transfer the load from diaphragms to foundations.
Unreinforced masonry walls are characterized by a stronger and a weaker direction. A wall laterally loaded along its in-plane direction assume a more favourable behaviour than in the case subjected to out-of-plane load.
Consequently, diaphragms need enough stiffness to transfer equally horizontal loads to vertical elements in their in-plane direction.

Increase of stiffness in diaphragm may be achieved by different solutions, two of them have been listed in this report. In both cases the strengthening is reached by adding an extra layer above the existing floor sheathing.
The examined literature confirms that stiffening of an existing timber floor by means of concrete topping overlay is able to transform a flexible diaphragm into a rigid diaphragm.
Moreover, since this intervention is often realized together with the connection between diaphragms and walls, the concrete topping overlay allows to work only above the floor respect to the timber overlay. In which the connection between walls and floor should involve joists of the timber frame.

6.10 Reduction of Period of Vibration due to Diaphragm Stiffening

For the calculation example, a diaphragm of a typical Dutch terraced house has been selected. The chosen diaphragm is highlighted by the green colour in Figure 6-11. With the purpose of an easier understanding, the selected part has been assumed as detached by the rest of the building. Figure 6-9 shows the geometry of the structure used for the analysis of the diaphragm period.

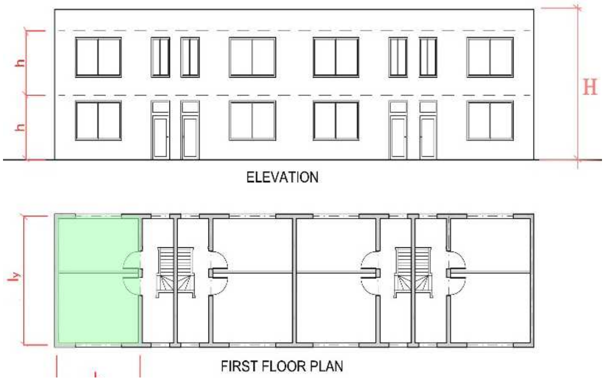


Figure 6-11 Dimensions Diaphragm subject of analysis

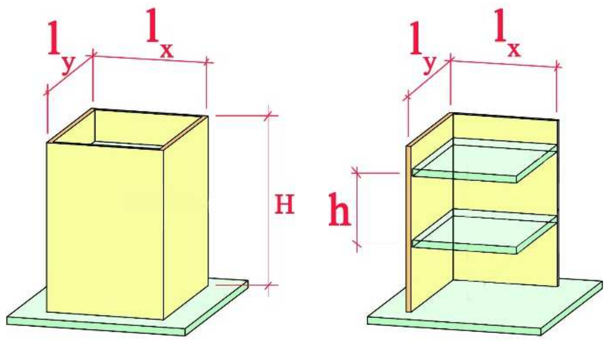


Figure 6-910Diaphragm and Fictitious Building

BUILDING

Building height $H := 7000\ mm$

Storey height $h := 3000\ mm$

Wall thickness $t := 210\ mm$

Masonry weight $\rho_m := 1800\ \frac{kg}{m^3}$

CONCRETE TOPPING OVERLAY C30/37

Concrete topping overlay thickness $t_c := 60\ mm$

Concrete modulus of elasticity $E_c := 30000\ MPa$

DIAPHRAGM

Shear stiffness for timber single straight sheathing (taken from Table 4-1) $G_d := 350\ \frac{kN}{m}$

Dimensions $l_x := 4500\ mm$ $l_y := 6000\ mm$

Diaphragm dead load $g_{k.j} := 100\ \frac{kg}{m^2}$ Ceramic tiles and glue: 55 Kg/m2

Timber floor single sheathing: 45 Kg/m2

Live load $q_{k.i} := 200\ \frac{kg}{m^2}$

Weight of diaphragm per sq meter (see Annex A) $q_{G.d} := g_{kj} + (0.3 \cdot 1.0 \cdot q_{k.i}) = 160\ \frac{kg}{m^2}$

Building mass
(no openings have been considered)

$$m_b := (H \cdot \rho_m \cdot ((2 \cdot l_y \cdot t) + (2 \cdot l_x \cdot t))) + (2 \cdot l_x \cdot l_y \cdot q_{G.d}) = (6.421 \cdot 10^4)\ kg$$

ANALYSIS BEFORE THE DIAPHRAGM RETROFIT

In order to evaluate the variation of the building behaviour during a seismic event due to the stiffening of horizontal diaphragms, two analysis have been carried out. In both analysis the period of vibration of the building was approximated by means of Eq.[6.5], where the diaphragm is assumed to perform as a simply supported beam spanning between shear walls.

In the first analysis, the natural period has been determined considering the non-retrofitted diaphragm composed by a timber frame single sheathing. Once the period was calculated, seismic loads acting at storey levels have been determined using the procedure suggested by Eurocode 8. After that, design seismic loads were applied to the building, hence it was possible to evaluate deformations performed by the building in terms of diaphragm and wall displacements. In the first analysis no connection between wall and diaphragm have been taking into account, therefore the diaphragm displacement is only attributed to the shear stiffness of the element.

The second analysis has been carried out following the same approach used in the first analysis. In this case the diaphragm has been considered retrofitted by means of a concrete topping overlay of 60 mm thick.

The walls running perpendicularly to the seismic forces have been assumed to be restrained by the diaphragm with anchors. With this assumption, the weight tributary to the diaphragm shall include the dead load of wall panel portions anchored to the diaphragm. This can be seen from the Figure 6-12 where the green colour represents the masses accounted as weight tributary to the diaphragm.

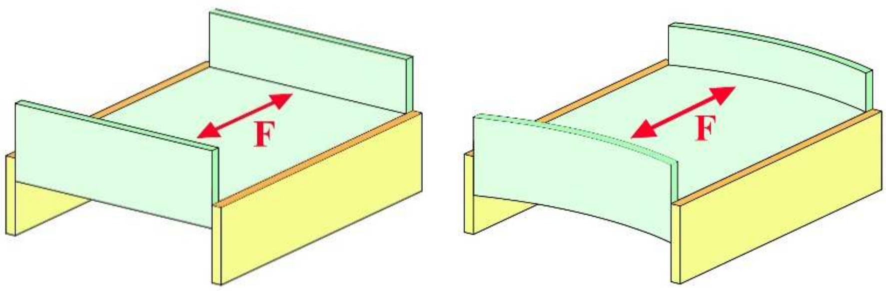


Figure 6-12 Weight Tributary to the Diaphragm

Diaphragm period calculation in the y direction

Self weight of the diaphragm and walls in the y direction	$W_{D.y} := ((q_{G.d} \cdot l_x \cdot l_y) + (2 \cdot (\rho_m \cdot t \cdot h \cdot l_x))) = 14526\ kg$
Diaphragm displacement	$\Delta_{D.y} := \dfrac{W_{D.y} \cdot g \cdot l_x}{l_y \cdot 2 \cdot G_d} = 152.6\ mm$
Diaphragm period	$T := \sqrt{3.07 \cdot \Delta_{D.y}} = 0.685\ m^{\frac{1}{2}}$ $\quad T_y := 0.685\ s$

Diaphragm period calculation in the x direction

Self weight of the diaphragm and walls in the x direction $W_{D.x} := \left(\left(q_{G.d} \cdot l_x \cdot l_y\right) + \left(2 \cdot \left(\rho_m \cdot t \cdot h \cdot l_y\right)\right)\right) = 17928\ kg$

Diaphragm displacement $\Delta_{D.x} := \dfrac{W_{D.x} \cdot g \cdot l_y}{l_x \cdot 2 \cdot G_d} = 334.9\ mm$

Diaphragm period $T := \sqrt{3.07 \cdot \Delta_{D.x}} = 1.014\ m^{\frac{1}{2}}$ $\quad T_x := 1.014\ s$

Diaphragm seismic load calculation in the y direction

Correction factor $\lambda := 0.85$ $\quad \beta := 0.2$

Parameters describing the recommended Type 2 elastic response spectra for ground type E $\quad T_B := 0.05\ s \quad T_C := 0.25\ s \quad T_D := 1.2\ s \quad S := 1.6$

$T_y = 0.685\ s$ $\quad$ Therefore $\quad T_C \le T_y \le T_D$

Design ground acceleration $a_g := 0.243 \cdot g$

Behaviour factor for URM $q := 1.5$

Response spectrum acceleration $S_{d.y}(T) := a_g \cdot S \cdot \dfrac{2.5}{q} \cdot \left(\dfrac{T_C}{T_y}\right) = 2.32\ \dfrac{m}{s^2}$

$\beta \cdot a_g = 0.477\ \dfrac{m}{s^2}$

Base shear force $F_{b.y} := S_{d.y}(T) \cdot m_b \cdot \lambda = 126.57\ kN$

Horizontal force first storey (see figure below) $F_{1.y} := F_{b.y} \cdot \left(\dfrac{h \cdot W_{D.y}}{\left(h \cdot W_{D.y}\right) + \left(2 \cdot h \cdot W_{D.y}\right)}\right) = 42.19\ kN$

Horizontal force second storey (see figure below) $F_{2.y} := F_{b.y} \cdot \left(\dfrac{2 \cdot h \cdot W_{D.y}}{\left(h \cdot W_{D.y}\right) + \left(2 \cdot h \cdot W_{D.y}\right)}\right) = 84.38\ kN$

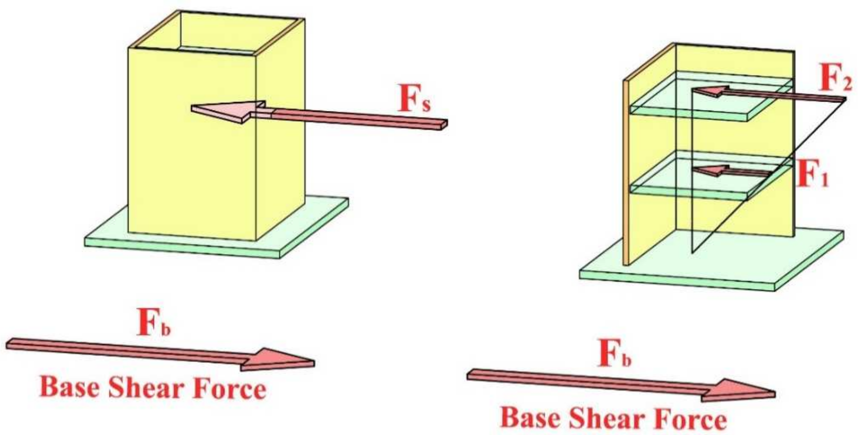


Figure 6-13 Seismic base shear force and seismic storey forces acting on the building

Diaphragm seismic load calculation in the x direction

$T_x = 1.014\ s$	Therefore $\quad T_C \le T_x \le T_D$
Response spectrum acceleration	$S_{d.x}(T) := a_g \cdot S \cdot \frac{2.5}{q} \cdot \left(\frac{T_C}{T_x}\right) = 1.57\ \frac{m}{s^2}$ $\beta \cdot a_g = 0.477\ \frac{m}{s^2}$
Base shear force	$F_{b.x} := S_{d.x}(T) \cdot m_b \cdot \lambda = 85.51\ kN$
Horizontal force first storey	$F_{1.x} := F_{b.x} \cdot \left(\frac{h \cdot W_{D.x}}{(h \cdot W_{D.x}) + (2 \cdot h \cdot W_{D.x})}\right) = 28.50\ kN$
Horizontal force second storey	$F_{2.x} := F_{b.x} \cdot \left(\frac{2 \cdot h \cdot W_{D.x}}{(h \cdot W_{D.x}) + (2 \cdot h \cdot W_{D.x})}\right) = 57\ kN$

After that seismic actions are known, it is possible to asses if the existing diaphragm is rigid or flexible. In order to evaluate the behaviour of the element, the dispalcement of walls (Δ_W) and of the diaphragm (Δ_D) due to the seismic action shall be determined.

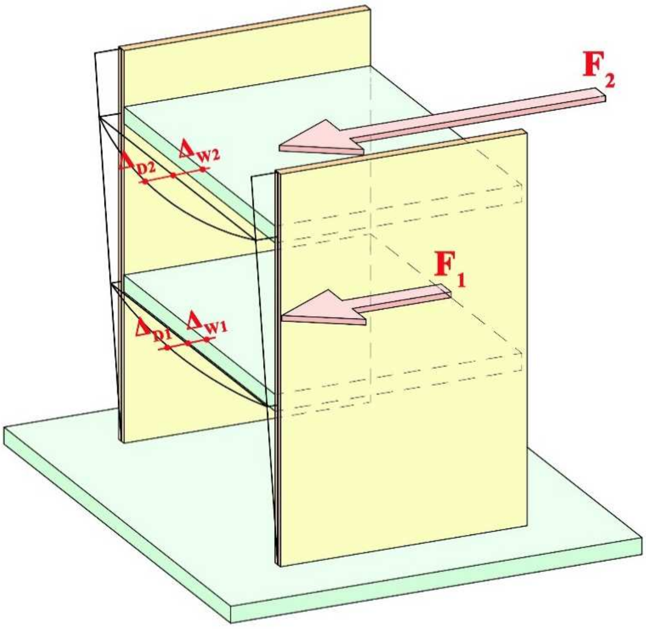


Figure 6-14 Displacement performed by walls and diaphragms during a seismic event

The assessment of diaphragm deformation has been conducted considering the second storey diaphragm, since it represents the element which performs the greater displacement respect to the two levels.

Diaphragm Behaviour Assessment in the y Direction

Masonry modulus of elasticity	$E_m := 4410\ MPa$
Masonry shear modulus	$G_m := 0.4\ E_m = 1764\ MPa$
First storey height	$h_1 := 3000\ mm$
Second storey height	$h_2 := 6000\ mm$
Shear wall cross sectional area	$A_{v.y} := \frac{5}{6} \cdot t \cdot l_y = 1.05\ m^2$
Shear wall modulus of inertia	$I_y := \frac{t \cdot l_y^{\ 3}}{12} = 3.78\ m^4$
Second storey wall displacement in the y direction	$\Delta_{W2.y} := \frac{\frac{F_{2.y}}{2} \cdot h_2^{\ 3}}{3 \cdot E_m \cdot I_y} + \frac{\frac{F_{2.y}}{2}}{A_{v.y}} \cdot \frac{h_2}{G_m} = 0.32\ mm$
ASCE and NZSEE upper bound	$\Delta_{Dmax2.y} := 2 \cdot \Delta_{W2.y} = 0.64\ mm$
ASCE and NZSEE lower bound	$\Delta_{Dmin2.y} := 0.5 \cdot \Delta_{W2.y} = 0.16\ mm$
Second storey diaphragm displacement in the y direction	$\Delta_{D2.y} := \frac{F_{2.y} \cdot l_x}{l_y \cdot 2 \cdot G_d} = 90.41\ mm$

Diaphragm Behaviour Assessment in the x Direction

Shear wall cross sectional area	$A_{v.x} := \frac{5}{6} \cdot t \cdot l_x = 0.788\ m^2$
Shear wall modulus of inertia	$I_x := \frac{t \cdot l_x^{\ 3}}{12} = 1.595\ m^4$
Second storey wall displacement in the x direction	$\Delta_{W2.x} := \frac{\frac{F_{2.x}}{2} \cdot h_2^{\ 3}}{3 \cdot E_m \cdot I_x} + \frac{\frac{F_{2.x}}{2}}{A_{v.x}} \cdot \frac{h_2}{G_m} = 0.41\ mm$
ASCE and NZSEE upper bound	$\Delta_{Dmax2.x} := 2 \cdot \Delta_{W2.x} = 0.83\ mm$
ASCE and NZSEE lower bound	$\Delta_{Dmax2.x} := 0.5 \cdot \Delta_{W2.x} = 0.21\ mm$
Second storey diaphragm displacement in the x direction	$\Delta_{D2.x} := \frac{F_{2.x} \cdot l_y}{l_x \cdot 2 \cdot G_d} = 108.58\ mm$

The diaphragm displacement results to be greater than the upper bond in both x and y direction. Therefore the diaphragm shall be classified as flexible in both directions.

REVISED DIAPHRAGM PERIOD CALCULATION

The overall stiffness of the diaphragm is given by the contribution of the in-plane flexural stiffness of the sole diaphragm and the stiffness of diaphragm-wall shear connectors. Since the two systems (diaphragm and connectors) are in series, the total deformation of the diaphragm shall be determined by the sum of contributions.

$$\Delta_D = \Delta_C + \Delta_f \qquad [6.17]$$

The two displacement contributions can be determined analysing the system by means of two ideal systems. In the ideal case of rigid connectors (Kc --> ∞) the overall deformation is only due to the flexural stiffness of the diaphragm. Similarly, when assuming rigid diaphragm (Kf --> ∞), only the stiffness of connectors contributes. The equivalent stiffness of the entire diaphragm system, which is used for the retrofit analysis is thus given by combination of both contribution.

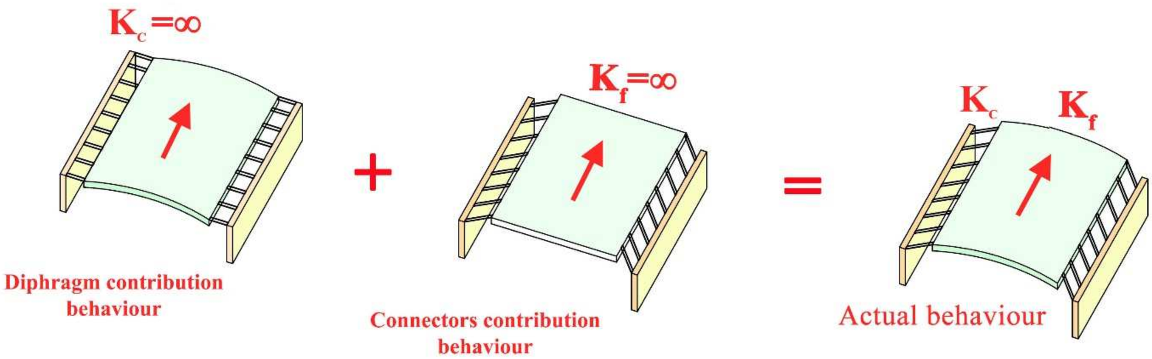


Figure 6-15 Stiffness contribution given by flexural diaphragm contribution and connectors contribution

The new diaphragm period has been determined assuming infinitive stiff connections, since the same assumption was taken in the calculation before the retrofitting.

Also in the assessment of the element behaviour (rigid or flexible), only the flexural stiffness of the member has been considered.

Eventually, the total displacement of the diaphragm element due to the seismic forces has been determined considering both contributions of stiffness.

Period Calculation in the y direction

Diaphragm flexural stiffness and flexural deformation

The diaphragm is assumed to behave as a beam subjected to a concentrated load applied in the center. The stiffness of the diaphragm is related to the stiffness of the concrete slab, without taking into account the stiffness of the existing timber frame.

Concrete slab moment of inertia $$I_{C.y} := \frac{t_c \cdot l_y^3}{12} = \left(1.08 \cdot 10^{12}\right)\ mm^4$$

Diaphragm flexural stiffness $$K_{f.y} := \frac{48 \cdot E_c \cdot I_{C.y}}{l_x^3} = \left(1.707 \cdot 10^{7}\right)\ \frac{kN}{m}$$

Diaphragm connection stiffness and connection deformation

Connections between diaphragm and walls have been realized by means of steel bars of diameter d. The assumed load condition of each steel as considered in section 5.7.1 to determine the load bearing capacity of the element has been considered to calculate the stiffness of the bar. In order to define the stiffness of the bar in its elastic behaviour, the principle of superposition of effects has been used. Figure 6-16 displays the two systems analysed.

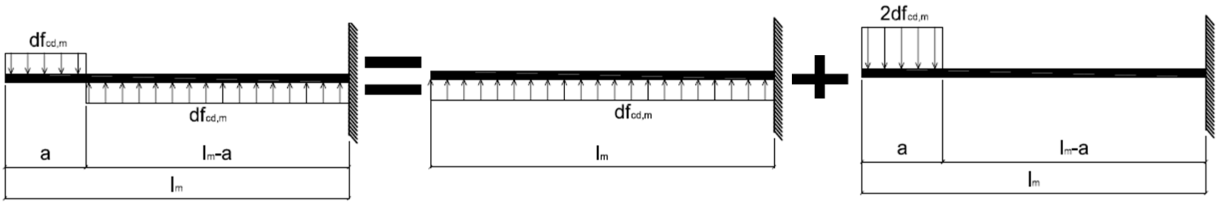


Figure 6-16 Superposition of Effects for Load Condition

The following data has been assumed to determine the stiffness of a single bar.

STAINLESS STEEL ROD
UNS S32304 - ISO 4362-323-04-I

Proof strength $$f_{p.0.2} := 400\ MPa$$

Partial safety factor $$\gamma_s := 1.15$$

Diameter $$d := 16\ mm$$

Yield moment $$M_{y.d} := \frac{f_{p.0.2}}{\gamma_s} \cdot \pi \cdot \frac{d^3}{32} = 139.9\ N \cdot m$$

Characteristic anchorage strength between mortar M20 and concrete 30/37 (Table 3.6 Eurocode 6) $$f_{bok} := 3.4\ MPa$$

Steel modulus of elasticity $$E_s := 210000\ \frac{N}{mm^2}$$

Moment of inertia of the bar used for connection $$I_b := \frac{\pi}{4} \cdot \left(\frac{d}{2}\right)^4 = \left(3.217 \cdot 10^{3}\right)\ mm^4$$

MASONRY

Compressive strength $f_k := 4.4 \dfrac{N}{mm^2}$

Partial safety factor $\gamma_m := 1.7$

Anchorage length $l_m := 160\ mm$

Design compressive strength $f_{cd.m} := \dfrac{f_k}{\gamma_m} = 2.6 \dfrac{N}{mm^2}$

Length b (equal to lm-a) derived from equilibrium condition $b := l_m \cdot \left(\sqrt{2 + \dfrac{4\ M_{y.d}}{\left(d \cdot f_{cd.m} \cdot l_m^2\right)}} - 1 \right) = 94\ mm$

Length a $a := \dfrac{l_m - b}{2} = 33\ mm$

The total stiffness of the bar has been derived by the sum of stiffnesses determined by the superposition of effects:

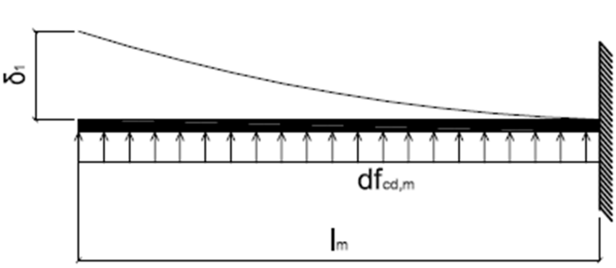


Figure 6-17 Load Condition 1

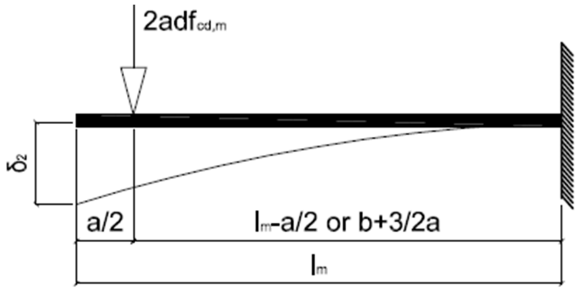


Figure 6-18 Load Condition 2 with Equivalent Load

$$\delta_1 := \frac{d \cdot f_{cd.m} \cdot l_m^4}{8 \cdot E_s \cdot I_b}$$

$$k_1 := \frac{8 \cdot E_s \cdot I_b}{l_m^3} = 1319 \frac{kN}{m}$$

$$\delta_2 := \frac{(2 \cdot d \cdot f_{cd.m} \cdot a) \cdot \left(b + \frac{3}{2}\ a\right)^2}{6 \cdot E_s \cdot I_b} \left(3\ l_m - b + \frac{3}{2}\ a\right)$$

$$k_2 := \frac{6 \cdot E_s \cdot I_b}{\left(b + \frac{3}{2}\ a\right)^2 \cdot \left(3\ l_m - b + \frac{3}{2}\ a\right)} = 452 \frac{kN}{m}$$

$k_{eq} := k_1 + k_2 = 1772 \dfrac{kN}{m}$ where $k_{eq} := k_{c.y}$

Consequently the equivalent diaphragm stiffness is:

Number of diaphragm connectors $n := 30$

Diaphragm equivalent stiffness $K_{D.y} := \dfrac{1}{\dfrac{1}{K_{f.y}} + \dfrac{1}{n \cdot K_{c.y}}} = \left(5.652 \cdot 10^4\right) \dfrac{kN}{m}$

Diaphragm seismic load and deflection after the retrofit

Diaphragm dead load $g_{k.j} := 180\ \frac{kg}{m^2}$

Timber floor single sheathing: 45 Kg/m2
Ceramic tiles and glue: 55 Kg/m2
Concrete topping overlay: 80 Kg/m2

Live load $q_{k.i} := 200\ \frac{kg}{m^2}$

Weight of diaphragm per sq meter (see Annex A) $q_{G.d} := g_{k.j} + (0.3 \cdot 1.0 \cdot q_{k.i}) = 240\ \frac{kg}{m^2}$

Self weight of the diaphragm and walls in the x direction $W_{D.y} := ((q_{G.d} \cdot l_x \cdot l_y) + (2 \cdot (\rho_m \cdot t \cdot h \cdot l_x))) = 16686\ kg$

Building mass $m_b := (H \cdot \rho_m \cdot ((2 \cdot l_y \cdot t) + (2 \cdot l_x \cdot t))) + (2 \cdot l_x \cdot l_y \cdot q_{G.d}) = (6.853 \cdot 10^4)\ kg$

Diaphragm period $T_y := \sqrt{3.07 \cdot \left(\frac{W_{D.y} \cdot g}{K_{f.y}}\right)} = 0.005\ m^{\frac{1}{2}}$ $T_y := 0.006\ s$

$T_y = 0.006\ s$ Therefore $0 \le T_y \le T_B$

Response spectrum acceleration $S_{e.y}(T) := a_g \cdot S \cdot \left(\frac{2}{3} + \frac{T_y}{T_B}\left(\frac{2.5}{q} - \frac{2}{3}\right)\right) = 3\ \frac{m}{s^2}$

Base shear force $F_{b.y} := S_{e.y}(T) \cdot m_b \cdot \lambda = 174.71\ kN$

Horizontal force first storey $F_{1.y} := F_{b.y} \cdot \left(\frac{h \cdot W_{D.y}}{(h \cdot W_{D.y}) + (2 \cdot h \cdot W_{D.y})}\right) = 58\ kN$

Horizontal force second storey $F_{2.y} := F_{b.y} \cdot \left(\frac{2 \cdot h \cdot W_{D.y}}{(h \cdot W_{D.y}) + (2 \cdot h \cdot W_{D.y})}\right) = 116\ kN$

Second storey wall displacement in the y direction $\Delta_{W2.y} := \frac{\frac{F_{2.y}}{2} \cdot h_2^3}{3 \cdot E_m \cdot I_y} + \frac{\frac{F_{2.y}}{2}}{A_{v.y}} \cdot \frac{h_2}{G_m} = 0.44\ mm$

ASCE and NZSEE upper bound $\Delta_{Dmax2.y} := 2 \cdot \Delta_{W2.y} = 0.88\ mm$

ASCE and NZSEE lower bound $\Delta_{Dmin2.y} := 0.5 \cdot \Delta_{W2.y} = 0.22\ mm$

Second storey diaphragm displacement in the y direction accounting only the flexural stiffness of the element $\Delta_{D2.y} := \frac{F_{2.y}}{K_{f.y}} = 0.01\ mm$

Second storey diaphragm displacement in the y direction accounting both stiffness of the element and of connections $\Delta_{D2.y} := \frac{F_{2.y}}{K_{D.y}} = 2.2\ mm$

In order to compare the diaphragm before and after its retrofit, the displacement performed by the element taking into account only its flexural stiffness results to be largely reduced. Since the flexural displacement results to be smaller than the lower bond, the retrofitted diaphragm shall be classified as rigid.

Period Calculation in the x direction

Similarly as for the period calculation in the y direction, the diaphragm flexural stiffness has been determined.

Concrete slab moment of inertia	$I_{C.x} := \frac{t_c \cdot l_x^3}{12} = (4.556 \cdot 10^{11})\ mm^4$
Diaphragm flexural stiffness	$K_{f.x} := \frac{48 \cdot E_c \cdot I_{C.x}}{l_y^3} = (3.038 \cdot 10^6)\ \frac{kN}{m}$

Diaphragm connection stiffness and connection deformation

The stiffness of connectors is the same determined for the period calculation in the y direction. Consequently the diaphragm equivalent stiffness may be calculated by means of the following equation.

Number of diaphragm connectors	$n := 30$
Connections stiffness	$K_{c.x} := 1772\ \frac{kN}{m}$
Diaphragm equivalent stiffness	$K_{D.x} := \frac{1}{\frac{1}{K_{f.x}} + \frac{1}{n \cdot K_{c.x}}} = (5.225 \cdot 10^4)\ \frac{kN}{m}$

Similarly as before described for the behaviour of the diaphragm in the y direction, the diaphragm flexural displacement results to be lower than the lower bond after its retrofit. Therefore also in the x direction the diaphragm shall be classified as rigid.

Diaphragm seismic load and deflection after the retrofit

Diaphragm dead load $g_{k.j} := 180 \ \frac{kg}{m^2}$

Ceramic tiles and glue: 55 Kg/m2
Timber floor single sheathing: 45 Kg/m2
Concrete topping overlay: 80 Kg/m2

Live load $q_{k.i} := 200 \ \frac{kg}{m^2}$

Weight of diaphragm per sq meter (see Annex A) $q_{G.d} := g_{k.j} + (0.3 \cdot 1.0 \cdot q_{k.i}) = 240 \ \frac{kg}{m^2}$

Self weight of the diaphragm and walls in the x direction $W_{D.x} := ((q_{G.d} \cdot l_x \cdot l_y) + (2 \cdot (\rho_m \cdot t \cdot h \cdot l_y))) = 20088 \ kg$

Diaphragm period $T_x := \sqrt{3.07 \cdot \left(\frac{W_{D.x} \cdot g}{K_{f.x}}\right)} = 0.014 \ m^{\frac{1}{2}}$ $T_x := 0.014 \ s$

$T_x = 0.014 \ s$ Therefore $0 \le T_x \le T_B$

Response spectrum acelleration $S_{e.x}(T) := a_g \cdot S \cdot \left(\frac{2}{3} + \frac{T_x}{T_B}\left(\frac{2.5}{q} - \frac{2}{3}\right)\right) = 3.61 \ \frac{m}{s^2}$

Base shear force $F_{b.x} := S_{e.x}(T) \cdot m_b \cdot \lambda = 210 \ kN$

Horizontal force first storey $F_{1.x} := F_{b.x} \cdot \left(\frac{h \cdot W_{D.x}}{(h \cdot W_{D.x}) + (2 \cdot h \cdot W_{D.x})}\right) = 70 \ kN$

Horizontal force second storey $F_{2.x} := F_{b.x} \cdot \left(\frac{2 \cdot h \cdot W_{D.x}}{(h \cdot W_{D.x}) + (2 \cdot h \cdot W_{D.x})}\right) = 140 \ kN$

Second storey wall displacement in the y direction $\Delta_{W2.x} := \frac{\frac{F_{2.x}}{2} \cdot h_2^{\ 3}}{3 \cdot E_m \cdot I_x} + \frac{\frac{F_{2.x}}{2}}{A_{v.x}} \cdot \frac{h_2}{G_m} = 1 \ mm$

ASCE and NZSEE upper bound $\Delta_{Dmax2.x} := 2 \cdot \Delta_{W2.x} = 2.04 \ mm$

ASCE and NZSEE lower bound $\Delta_{Dmin2.x} := 0.5 \cdot \Delta_{W2.x} = 0.51 \ mm$

Second storey diaphragm displacement in the x direction accounting only the flexural stiffness of the element $\Delta_{D2.x} := \frac{F_{2.x}}{K_{f.x}} = 0.05 \ mm$

Second storey diaphragm displacement in the x direction accounting both stiffness of the element and of connections $\Delta_{D2.x} := \frac{F_{2.x}}{K_{D.x}} = 2.7 \ mm$

7 LEVEL IV RETROFITTING STRATEGY

This chapter describes the out-of-plane loads that a wall necessities to withstand during a seismic event, and it provides a solutions of the issue.

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URM walls result weak when they are subjected to loads other than compression. When shear walls are fully restrained at each level, as described in Chapter 3, out-of-plane forces can cause significant wall bending. The bending moment resistance of the wall is determined by the ratio of height between levels of support and the thickness of the wall itself, in addition of the axial load.

7.1 Out-Of-Plane Wall Requirement

In Chapter 3 connection between walls and diaphragms has been treated. The seismic force F_p related to each anchorage is determined by the minimum of Eq. [7-1] and Eq. [7-2]. A wall restrained at storey levels has reduced spans of free panels, spans are equal to the distance between locations of out-of-plane anchorages in both vertical and horizontal directions. ASCE 41-13 [7].

The strength of the wall shall be adequate to withstand the force F_p which is applied at mid span between anchorage locations:

$$F_p = 0.4 S_{xs} \chi W_p \qquad [7\text{-}1]$$

$$F_{p,\min} = 0.1 \chi W_p \qquad [7\text{-}2]$$

Where:

χ is the factor for calculation of out-of-plane wall forces, from Table 7-1 for the selected Structural Performance Level.

S_{xs} is the spectral response acceleration parameter at short periods for the selected hazard level and damping, adjusted for site class, without any adjustment for soil-structure interaction

W_p is the weight of the wall for unit area[2]

Structural Performance Level	χ
Collapse Prevention	1.0
Life Safety	1.3
Immediate Occupancy	2.0

Table 7-1 Factor X for Calculation of Out-of-Plane Wall Forces FEMA 547[4]

7.2 Post-Tensioning Steel Rods Inserted into Vertical Holes through URM Wall

Post-tensioning is considered as one of the most effective ways of providing both out-of-plane and in-plane strength in URM walls. The post-tensioning retrofitting technique may be applied either internally to the masonry element or externally along the face of walls.

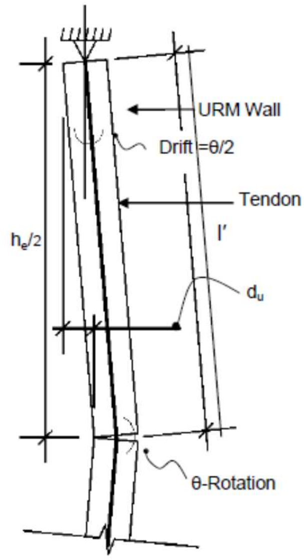


Figure 7-1 Possible instability due to the eccentricity of the steel rod

Performance of post-tensioned URM walls varies upon the post-tensioning stress initially applied, steel rod type and spacing, restraint conditions and confinement. Steel rods can either be bonded into cavities or left unbonded by not filling the cavities. In the case of bonded tendons, they may be considered as fully restrained. Lateral restrain of post-tensioning tendons is an important issue when second order effects are considered. This because, additional axial load aggravates bending stresses in walls as they displace due to P-Δ effect. For this aspect it is preferable to provide lateral restrained tendons to eliminate P-Δ effects. The restrained tension does not change its load line of action respect the neutral axis of the wall, thus no additional stress are introduced. On the other hand, unbonded post-tensioned steel rods are preferable for URM buildings having important heritage value due to reversibility of the intervention. Ismail [20].

7.2.1 Design Considerations

Insertion of post-tensioning steel rods is a retrofitting measure applicable if some requirements are fulfilled. The part of masonry subjected to the post-tensioning shall be able to withstand the additional axial force introduced by the tendon. This capacity should be checked for both global local resistance, where the anchorage are placed. The initial loss of compression due to the masonry deformation shall be taken into account.

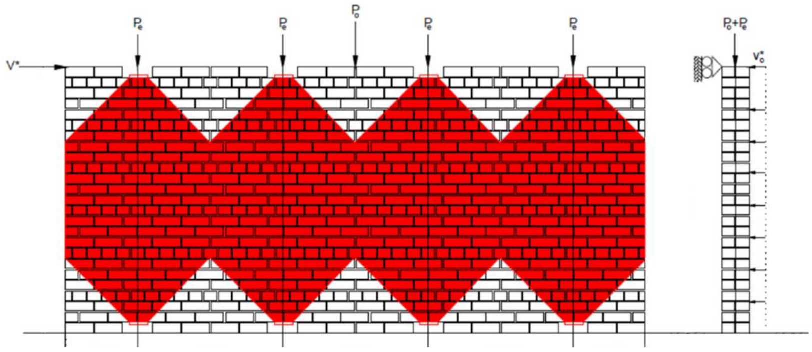


Figure 7-2 Additional axial force distribution introduced by tendons

7.2.2 Detailing and Construction Consideration

Figure 7-3 Anchorage of the tendon on the top of the retrofitted wall

- *Aesthetics:* Post-tensioning has a very little visual impact, since no signs are visible along the wall. When the threaded end of tendons is hidden in the upper part of the wall the intervention is almost invisible.
- *Installation approach*: The placing of tendons involves a sequence of steps:
    1. *Drilling.* The first procedure involves boring a cavity from the top of the URM wall right through to the foundations.
    2. *Placing* of the rod into a grout sock and fitted with grout tubes.
    3. *Insertion.* The rod wrapped in the grout sock is inserted into the cavity.
    4. *Grouting.* In case of unbounded tendon only the bottom part of the cavity is grouted, whereas in case of bounded tendon the whole cavity is grouted.
    5. *Placing* the top anchor plate.
    6. *Appling the required force* into the rod by means of hydraulic jack or by hand.
- *Materials:* Since a compressive stress is applied locally in the top anchor plate location, a local strengthening of the masonry may be required. Furthermore a direct path for moisture intrusion is present. The anchor plate can be galvanized, made from stainless steel or painted with exterior grade paint.
- *Access and realization issues*: In order to drill cavities the top part of the shear wall shall be free from obstruction. Therefore in case where the roof cover this part, precautions shall be taken.
- *Dimension:* specialized New Zealander constructors stated that it is possible to bore a cavity up to four storeys with a precision of +- 10mm. Recently this technique has been used to retrofit the Fort Leavenworth, USA, where 28 holes 13 meter long with a diameter of 20 mm have been drilled.

- *Anchorage:* anchorage of post-tensioning results for masonry retrofitting to be more difficult compared with other materials, due to its low compressive strength. This is typically realized by using a grout sock. In the case the building presents continuous reinforced concrete foundations a self-activating dead end can be encasing in them. Top anchorage devices and plates are typically in a recess of the surface and covered with shotcrete or cement mortar.

7.2.3 Cost and Disruption

Seismic retrofit by means of post-tensioning is considered an effective technique but expensive, due to the necessity of skilled contractor and particular equipment. Since rods are installed from the top of the wall to retrofit, disruption is dependent upon the accessibility to the top face of the wall.

7.3 Inter-Floor Wall Support

Strengthening of the wall is obtained by means of external steel reinforcement or other material elements. A series of either vertical or horizontal members can be bolted to the inside face of the wall in order to enhance the out-of-plane resistance.

Vertical steel sections aim to break up a large planar wall into a number of supported areas. Profiles spanning between diaphragms act as beams loaded in bending by the pressure of the wall during a seismic event. In the past vertical inter-floor supports have also been conceived as auxiliary support for the diaphragm in the event that the wall fails and collapses.

A similar result can be achieved by means of a horizontal steel member anchored at mid-height of the wall. The profile is than braced with diagonal struts up to the floor or ceiling diaphragm above. In this way the effective height of the masonry wall is reduced. Goodwin [21].

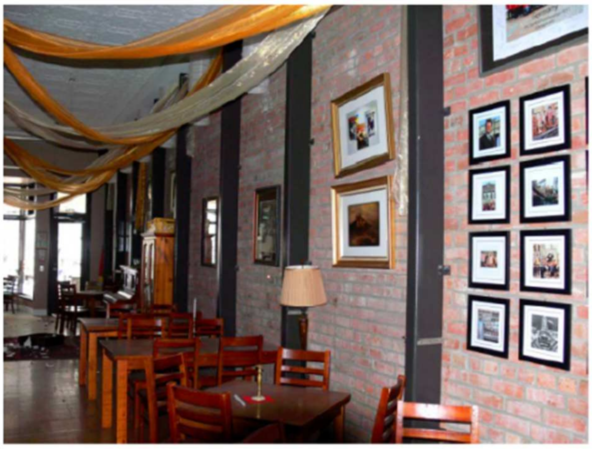


Figure 7-5 Internal vertical steel elements to restrain out-of-plane wall failure

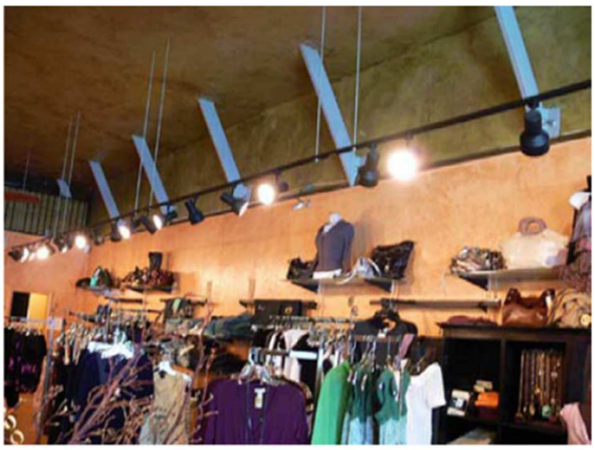


Figure 7-4 Diagonal struts from the floor above to improve out-of-plane performance of the wall

7.3.1 Design Consideration

The American code IEBC [17] furnishes standard indications regarding the spacing. Spacing of vertical bracing members should not exceed on half of the unsupported height of the wall or 3048mm (10 feet). Deflection for this bracing should not be greater than one-tenth of the wall thickness.

For horizontal elements, indications are given regarding spacing of diagonal struts, which should not exceed 1829mm (6 feet).

7.3.2 Detailing and Construction Consideration

- Aesthetics: It is considered as the least expensive approach but not suitable for every occupancy, since elements added to walls are remarkably noticeable. To minimize the visual impact, elements can be recessed into a cavity cut in the wall. Recessing the member requires more complex work and raises for cracking to propagation from the inside of the recess to the wall face.
- *Floor/roof framing capacity*: In the circumstance where the above diaphragm joists are parallel to the wall to retrofit, the horizontal anchorage force shall be developed out into the diaphragm. The existing diaphragm frame may need to be strengthened to distribute the load.
- *Materials*: Bracings can be realized by means of steel profiles, timber elements or reinforced concrete pilaster.

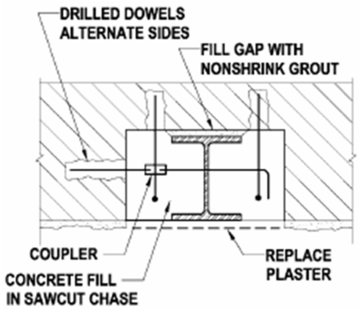


Figure 7-6 Steel Element Recessed Into Cavity

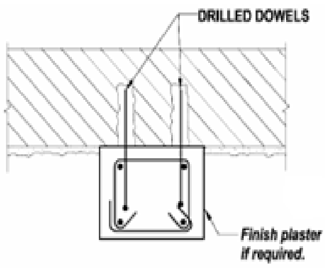


Figure 7-7 Concrete Pilaster

7.3.3 Cost and Disruption

The combination of horizontal element with diagonal struts is usually cheaper, but considered less reliable than vertical bracing. Exposed braces result less expensive than aesthetically sensitive options as recessed vertical braces. Installation of this technique is quite disruptive since bracing are normally placed around the entire perimeter connected to horizontal diaphragms with drilled dowels.

7.4 Non-Stressed Bounded Bars into Vertical Holes Drilled Through URM Walls

This technique adds reinforced cores along the URM walls to retrofit. Thus, afterward the wall can be considered as a reinforced masonry element. Although his technique has many aspects in common with the post-tensioned steel rods mentioned in the section 5.2, non-stressed steel bar set in grout become stressed only when the wall is loaded laterally.

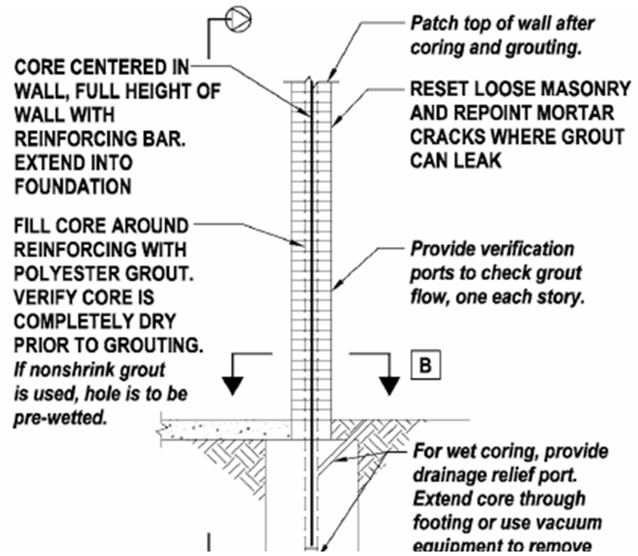


Figure 7-8 Non-stresses bounded bar inserted into masonry wall

7.4.1 Design Consideration

URM wall will result in a RM wall after the retrofit by means of reinforced cores. It is remarkable to not over-reinforce the masonry section. Which may cause a brittle failure of the masonry.

7.4.2 Detailing and Construction Consideration

- *Bar Size*: Size of bars depends on demand and design, but typically it ranges from ϕ 12 to ϕ 20
- *Grout type*: Researches show that the use of polyester grout provides the best dispersion into the masonry. In case of polyester grout attention must be taken to dry the hole before installing the grout. An alternative is high quality non-shrink cementitious grout, although the hole needs to be prewetted prior the grout pouring.
- *Installation approach*: The addition of reinforced cores is summarized in three steps:
    1. *Drilling*. The first procedure involves boring a cavity from the top of the URM wall right through to the foundations.
    2. *Lowering* of the bar into the hole.

3. *Pumping of grout*. A grout tube is loosely tied to the bar, as the grout is pumped into the hole, the tube is slowly withdrawn simultaneously with the increase of grout level.

- *Check points*: During the pouring of the grout, it will leak into the voids along the wall. To confirm that the grout is raising into the core, horizontal checking holes can be drilled on the wall. When the grout outcome from the hole, it is plugged and the grout is allowed to continuing raising along the core.

7.4.3 Cost and Disruption

Similarly as for post-tensioning, reinforced cores is considered an effective technique but expensive, due to skilled contractor and equipment needed. Disruption to interior and exterior faces is limited to sealing cracks, and to the necessity to access to the top of walls.

7.5 Composite Material Strips Fitted Into Vertical Saw Cuts in URM

Composite materials begun to be used about 40 years ago in aerospace and other industries. The material is made by means of strong fibres such as carbon, glass and aramid bound together by a matrix. The matrix is usually vinylester, polyester or epoxy resin. These materials are commonly so-called fibre reinforced polymer (FRP).

Mechanical properties of FRP depend on the fibre to matrix ratio. The matrix provides transfer of loads between fibres, whereas the strength and stiffness is provided by fibres. FRP are used in seismic retrofitting because of their high tensile strength, which typically exceeds that of metals by several times.

FRP are considered an effective technique for flexural or shear strengthening element to upgrade structural capacity. The retrofit of a masonry wall can be accomplished by means of either externally-bonded FRP laminates or near-surface-mounted (NSM) FRP bars.

Laminates are on the market in two forms: FRP sheets (fabric) and pre-cured strips (plates). FRP sheets are generally applied by manual wet lay-up and are attached with adhesive on the prepared surface of the masonry wall. Whereas, pre-cured strips are adhered to the substrate of walls with an epoxy or cement paste.

NSM FRP bars are round or rectangular. They are placed in grooves cut on the masonry surface and bound with epoxy or cement-based paste.

From the structural point of view, NSM and laminate FRP systems can be engineered to achieve similar objectives. In this report NSM FRP strips are further described as retrofit technique.

Figure 7-9 URM wall retrofitted using FRP NSM

7.5.1 Design Considerations

The lower elastic modulus of glass FRP (GFRP) respect to carbon FRP (CFRP) is not limiting in masonry strengthening application, since it results more compatible with the low elastic modulus of masonry.. On the other hand, CFRP show a better resistance to creep over a superior durability in most environments compared to GFRP and thus result to be a better choice. Aramid is not commonly used in masonry. The material properties of aramid are sensitive to moisture change, which is common in masonry structures.

Failure modes of masonry walls strengthened with FRP is typically governed by the debonding of FRP from the masonry. Thus, it precludes to exploit the high tensile strength on FRP. Because of these bond limitations, the usable design strengths of FRP applied to conventional masonry are typically in the range of 30% to 40% the ultimate tensile strength of FRP.

7.5.2 Detailing and Construction Considerations

- *Aesthetics:* In case of naked brick, this technique has some visual impact, but in case it is installed in plastered walls, it is totally invisible. Otherwise, in order to minimize the change in appearance for naked brick walls, a cement based paste is recommended. The paste can be mixed to match the original mortar in colour and texture
- *Installation approach:* The surface preparation for NSM FRP application is minimal. NSM FRP can be applied by means of these steps:
    1. *Cut groove.* Using a diamond blade saw or grinder, a grove 1.5 times the depth and 3 times the thickness of the FRP stripe is cut.

2. *Prepare groove*. The grove is prepared with masking tape or similar product to prevent surface damages due to excess adhesive. The groove is carefully cleaned using vacuum or compression air.
3. *Apply adhesive*. Epoxy adhesive or cement based paste is filled in the groove, taking care to avoid entrapped air voids.
4. *Place FRP strips into grooves*. After application of adhesive, the strip is placed and pressed into the groove to ensure proper location of it.
5. *Finish*. When the FRP strip is seated into the groove, the adhesive is smoothed and any additional adhesive is added. General clean up and removal of the masking tape is necessary.

- *Embedding paste:* Epoxy pastes provide better bond properties than cement based pastes. Cement based paste is the typical choice in case of aesthetic limitations. This choice causes a reduction in bond-development strengths between FRP and masonry. This reduction may result in an increase in number of strips. Tumialan [22].
- *Accessibility of wall faces:* Strengthening of masonry walls may require addition of NSM FRP on both sides of the wall, to gain flexural resistance against both inward and outward loads.

7.5.3 Cost and Disruption

This technique is characterized by a low installation cost given that the preparation of the wall is minimal. Disturbance to occupants can be reduced by dividing the wall to retrofit in small areas to minimize the loss of usable space.

7.6 Conclusion about Level IV Retrofitting Strategy

Walls running perpendicular to the direction of the ground motion are subjected to out-of-plane loads due to their own weight. Even if walls have been anchored at floor levels, walls are often subjected to loads higher than their resistant capacity.
Strengthening of URM walls in their out-of-plane direction is a challenge which nowadays is faced by means of the mentioned retrofitting techniques.

Steel rods inserted into drilled holes, in both cases of post-tensioning or not-stressed show good improvements in the strengthening of masonry members. However, these techniques present some limits, due to the complex drilling process, besides the difficulties connected to the anchorage at foundation level.

Inter-floor wall support, on the other hand, may represent a valid solution in case of industrial buildings. Since restrain of the wall is guaranteed by external elements, which would not been aesthetically acceptable in case of retrofitting of dwellings, or by recessed elements, which involves a more complex construction process.

Composite material strips applied into vertical cuts has been evaluated the most technically reliable solutions for this weakness aspect of URM buildings. This because on one hand, the calculation example shows the improvement in the resistance of the wall in its out-of-plane direction. On the other hand, this has been achieved by applying thin strips, which means a minimal disruption during the execution, a final acceptable aesthetic condition of the wall surface, and a total repeatability of the technique.

7.7 Design Example of Out-Of-Plane Strengthening of URM Wall using CFRP NSM

Design of URM retrofitted by means of CFRP NSM is based on the analytical model developed by Seracino [23], which predicts the intermediate crack (IC) debonding resistance of CFRP NSM to reinforced concrete.

7.7.1 Intermediate Crack Debonding Resistance

Strengthening of URM wall by means of vertical CFRP strips is based on the concept that the wall behaviour will be governed by the IC debonding of the strip, rather than the development of alternative more brittle failure modes. The aim is to avoid failure modes as the rupture of the CFRP strip, or horizontal bending failure of masonry between vertical strips, or masonry crushing.

The axial force in the CFRP strip required to cause the onset of IC debonding in reinforced concrete is defined as $P_{IC.}$ Hence, P_{IC} is generally limited by an upper limit, the limit is the rupture strain of the strip as shown in Eq. [7-3]

$$P_{IC} \leq \varepsilon_{rup} \left(EA\right)_p \qquad [7\text{-}3]$$

Where

ε_{rup} is the rupture strain of CFRP strip

$\left(EA\right)_p$ is the axial stiffness of the strip

Seracino [23] set Eq. [7-4] to determine the axial force transferable between concrete and the CFRP

$$P_{IC} = \sqrt{\tau_f \delta_f} \sqrt{L_{per} \left(EA\right)_P} \qquad [7\text{-}4]$$

Where

τ_f is the maximum interface shear stress

δ_f is the maximum slip in the bond-slip model

$L_{per} = 2d_f + b_f$ is the perimeter of the debonding failure plane cross section (see Figure 7-11)

In order to simplify Eq. [7-4], Seracino [23] using regression determined coefficients and exponents to express the maximum interface shear stress (τ_f) and the maximum slip (δ_f) in terms of debonding failure plane aspect ratio (φ_f) and the cylinder compressive strength of the concrete (f_c). The relation between these parameters is given by Eq. [7-5].

$$\tau_f \delta_f = 0.976\varphi_f^{\ 0.526} f_c^{\ 0.6} \qquad [7\text{-}5]$$

Eq. [7-5] is derived by relations given in Eq. [7-6] and Eq. [7-7], which refer to geometric parameters shown in the Figure 7-11 and material properties of concrete.

$$\tau_f = \left(0.802 + 0.078\varphi_f\right) f_c^{\ 0.6} \qquad [7\text{-}6]$$

$$\delta_f = \frac{0.976\varphi_f^{\ 0.526}}{0.802 + 0.078\varphi_f} \qquad [7\text{-}7]$$

Where

$\varphi_f = \dfrac{d_f}{b_f}$ is the IC debonding failure plane aspect ratio

f_c is the concrete cylinder compressive strength

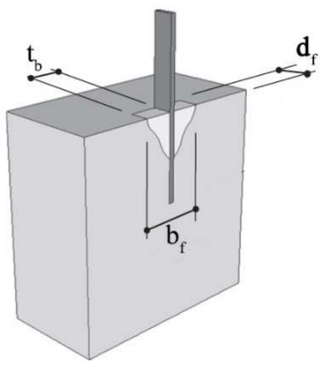


Figure 7-10 Prospective view of failure surface of CFRP NSM

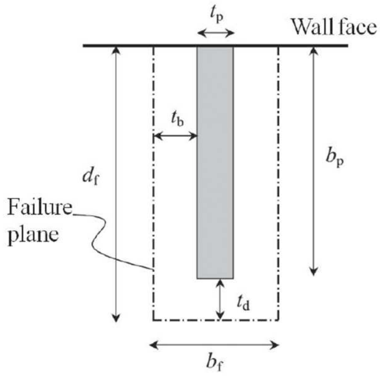


Figure 7-11 Failure surface cross section of CFRP NSM

Substituting Eq. [7-5] in Eq. [7-4], the following relationship is obtained for the mean IC debonding resistance, P_{IC} (Eq. [7-8])

$$P_{IC} = 0.988\varphi_f^{\ 0.263} f_c^{\ 0.3} \sqrt{L_{per}\left(EA\right)_P} \qquad [7\text{-}8]$$

7.7.2 Substitution of Material Parameters in Generic Model for Use in Masonry Retrofitting

The before mentioned model elaborated by Seracino [23] for concrete retrofitting is a function of the cylindrical compressive strength of the material.

In order to modify the model to use with masonry, Willis [24] expresses the concrete cylinder compressive strength in terms of tensile strength of the concrete. Generally the splitting tensile strength of concrete is possible to establish from its compressive strength. Willis [24] through the relation shown in Eq. [7-9] introduces this relationship.

$$\sqrt{f_c} = \frac{f_{ct}}{0.53} \qquad [7\text{-}9]$$

Therefore, substituting the tensile strength of the concrete f_{ct}, with the tensile strength of the brick unit f_{ut}, and substituting Eq. [7-9] into Eq. [7-8], the following relationship may be used in case of masonry retrofitting:

$$P_{IC} = 0.988\varphi_f^{\ 0.263}\left(\frac{f_{ut}}{0.53}\right)^{0.6}\sqrt{L_{per}\left(EA\right)_p} \qquad [7.10]$$

$$P_{IC} = 1.45\varphi_f^{\ 0.263} f_{ut}^{\ 0.6}\sqrt{L_{per}\left(EA\right)_p} \qquad [7.11]$$

7.7.3 Calculation Example of URM Wall Strengthening By CFRP NSM

The calculation example shows the retrofitting of an URM wall panel selected from a Dutch terraced house.

In Figure 7-12 the selected wall panel for the design is highlighted with the red colour. The wall panel is assumed to be beforehand retrofitted, and therefore restrained at diaphragm levels by means of wall to diaphragm connection anchors. On the basis of this assumption, the wall can be considered restrained at all its sides.

The seismic load acting on the wall panel has been determined on the basis of the approach given by the American code Asce 41-13[7]. The Peak Ground Acceleration (a_g) assumed in the design has been converted in the Short Period Spectral Acceleration S_S. This has been carried out by means of converting relations elaborated by Lubkowski [14] reported in Annex A.

WALL PANEL	Brick tensile strength		$f_{ut} := 3.5\ MPa$
	Masonry weight		$\rho_m := 1800\ \frac{kg}{m^3}$
	Masonry modulus of elasticity		$E_m := 4410\ MPa$
	Masonry compressive strength		$f_k := 4.4\ MPa$
	Masonry bending strength 1		$f_{xk.1} := 0.2\ MPa$
	Masonry bending strength 2		$f_{xk.2} := 0.4\ MPa$
	Partial safety factor for masonry		$\gamma_m := 1.7$
	Factor of structural performance level		$\chi := 2.0$
	Wall panel sizes	Thickness	$t := 210\ mm$
		Length	$l := 6000\ mm$
		Height	$h := 3000\ mm$
	Weight of the wall per unit of area		$W_p := t \cdot \rho_m = 378\ \frac{kg}{m^2}$

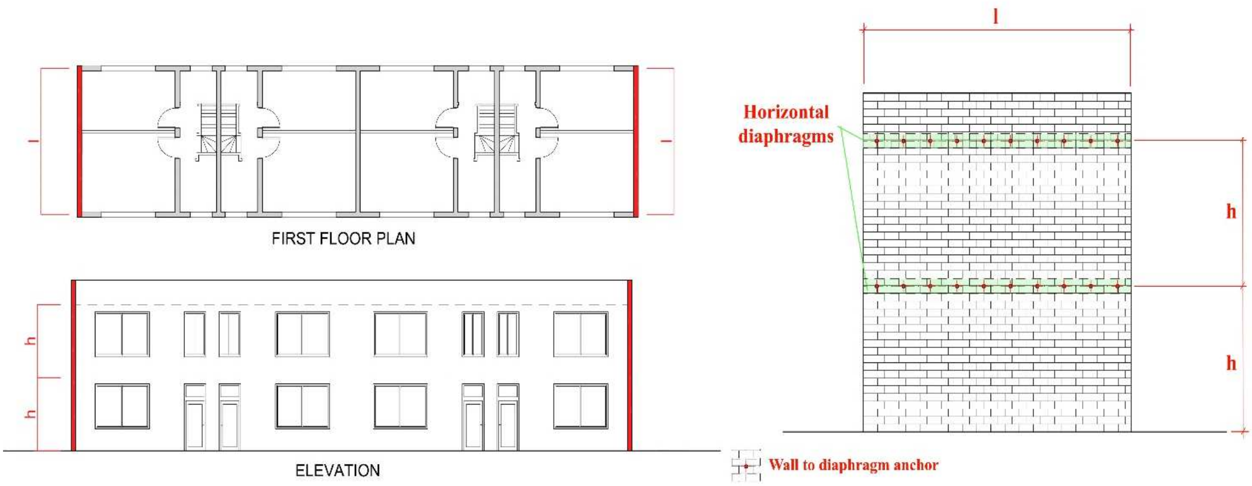


Figure 7-12 Selected wall panel from a typical Dutch terraced house

Peak ground acceleration (PGA) $a_g := 0.243$

Short period spectral acceleration $S_s := (0.3386 \cdot a_g + 2.1696) \cdot a_g = 0.547$

Long period spectral acceleration $S_1 := (0.5776 \cdot a_g + 0.5967) \cdot a_g = 0.179$

$S_s := 0.547 \ g$

$S_1 := 0.179 \ g$

Considering a Site Class E for the Groningen area, values for Fa and Fv can be determined by Table 2-3 and Table 2-4 respectively on ASCE 41-13.

$F_a := 1.6$

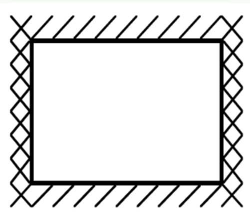
$S_{XS} := F_a \cdot S_s = 8.6 \ \frac{m}{s^2}$

$F_v := 3.3$

$S_{X1} := F_v \cdot S_1 = 5.8 \ \frac{m}{s^2}$

The wall panel shall be considered fully restrained along vertical edges due to the connection to others shear walls. Whereas along horizontal edges, the wall panel shall be considered hinged due to the presence of anchors between wall and diaphragms. The out-of-plane resistance of this portion of wall shall be checked using the theory of linear elasticity presented on Eurocode 6.

By the ratio h/l other parameters can be found on Annex E of Eurocode 6:



h/l ratio $\frac{h}{l} = 0.5$

Orthogonal ratio $\mu := \frac{f_{xk.1}}{f_{xk.2}} = 0.5$

Bending moment coefficient $\alpha := 0.021$

Design values of bending moments

Wall out of plane load due to the seismic acelleration is given by the minimum value between Fp and Fp.min

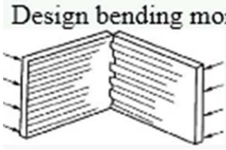
$F_p := 0.4 \cdot S_{XS} \cdot \chi \cdot W_p = 2.6 \ \frac{kN}{m^2}$

$F_{p.min} := 0.1 \cdot \chi \cdot W_p = 75.6 \ \frac{kg}{m^2}$

Design bending moment when the plane of failure is parallel to the bed joint

$M_{ed.1} := \mu \cdot \alpha \cdot F_p \cdot l^3 = 5.9 \ kN \cdot m$

Design bending moment when the plane of failure is perpendicular to the bed joint



$M_{ed.2} := \alpha \cdot F_p \cdot l^2 \cdot h = 5.9 \ kN \cdot m$

Lateral resistance using flexural strength

Bending moment resistance in the direction parallel to the bed joint	$M_{Rd.1} := \left(\frac{f_{xk.1}}{\gamma_m}\right) \cdot \left(\left(\frac{l \cdot t^2}{6}\right)\right) = 5.2\ kN \cdot m$
Bending moment resistance in the direction perpendicular to the bed joint	$M_{Rd.2} := \left(\frac{f_{xk.2}}{\gamma_m}\right) \cdot \left(\left(\frac{h \cdot t^2}{6}\right)\right) = 5.2\ kN \cdot m$

Strengthening of wall using NSM CFRP strips

Selected strips for the design are 0.125”x0.500” (3.2x12.7 mm) carbon fibre rectangular strip produced by Acp Composites. Material properties of strips have been taken by specification supplied by the manufacturer and attached in Annex C.

Horizontal spacing of strips has been based on the moment resistance of the wall in the direction perpendicular to the bed joint in the first place, than it has been adjusted to meet the material restrictions of the masonry in compression.

CFRP Modulus of elasticity	$E_p := 138\ GPa$
CFRP Tensile strength	$f_{up} := 1.72\ GPa$
Bending moment coefficient for wall support condition E (taken from Ec. 6 Annex E)	$\alpha := 0.028$
Horizontal spacing of CFRM strips	$s_{max} := \sqrt{\frac{M_{Rd.2}}{\alpha \cdot F_p \cdot h}} = 4878\ mm$
	$s := 500\ mm$

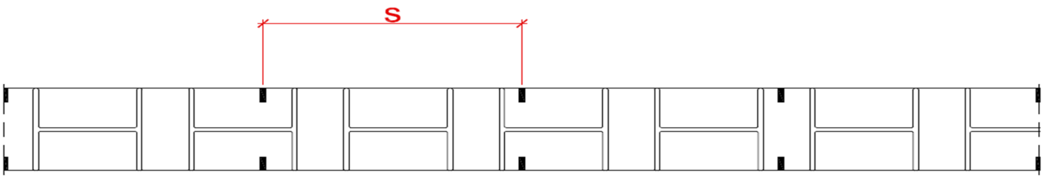
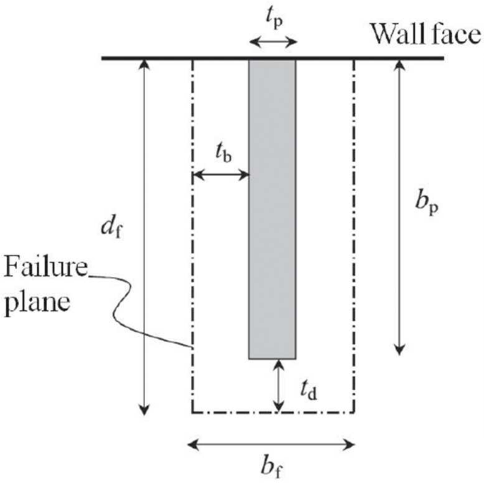


Figure 7-14 Wall panel cross section with strip spacing



$t_p := 3.2\ mm$

$b_p := 12.7\ mm$

$A_p := t_p \cdot b_p = 40.6\ mm^2$

$d_f := 20\ mm$

$b_f := 10\ mm$

$t_b := \frac{b_f - t_p}{2} = 3.4\ mm$

Figure 7-13 Strip and failure plane cross section

Axial stiffness	$EA_p := E_p \cdot A_p = 5608\ kN$
Failure plane aspect ratio	$\varphi_f := \dfrac{d_f}{b_f} = 2$
Perimeter of debonding failure plane	$L_{per} := 2 \cdot d_f + b_f = 50\ mm$
Intermediate crack debonding capacity	$P_{IC} := 1.45 \cdot \varphi_f^{\ 0.263} \cdot (f_{ut})^{0.6} \cdot \sqrt{L_{per} \cdot EA_p}$
$P_{IC} := 1.45 \cdot \varphi_f^{\ 0.263} \cdot (3.5)^{0.6} \cdot \sqrt{50 \cdot 5608000} = 61783$	$P_{IC} := 62\ kN$
CFRP Tensile rupture capacity	$P_{rupture} := f_{up} \cdot A_p = 70\ kN$

Since the aim is to avoid damage in the masonry wall due to the seismic loads, stress and strain distribution is assumed to be linear elastic in the loaded cross section of the wall. This is shown in Figure 7-15.

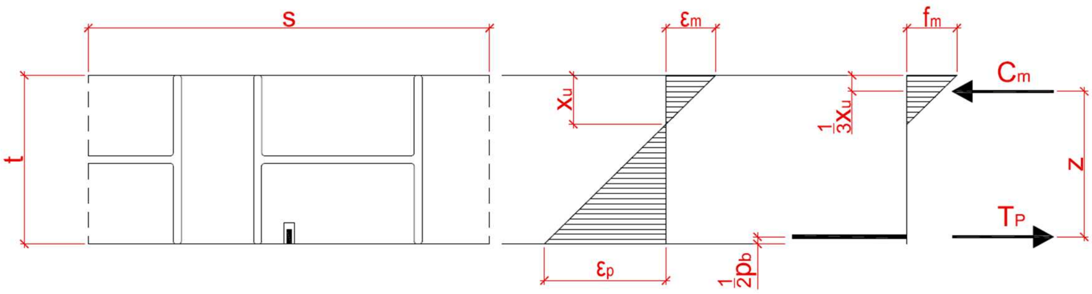


Figure 7-15 Linear elastic distribution of stress and strain in the masonry cross section

Masonry elastic strain limit	$\varepsilon_m := 0.0006$
Masonry compressive force	$C_m := P_{IC}$
CFRP tensile force	$T_p := P_{IC}$
Neutral axis location	$x_u := \dfrac{2 \cdot P_{IC}}{\varepsilon_m \cdot E_m \cdot s} = 93.7\ mm$
Effective compressive stress in masonry	$f_m := \dfrac{2 \cdot C_m}{x_u \cdot s} = 2.6\ MPa$
Maximum allowable stress in masonry	$f_d := \dfrac{f_k}{\gamma_m} = 2.6\ MPa$
Lever arm	$z := t - \dfrac{x_u}{3} - \dfrac{b_p}{2} = 172.4\ mm$
Bending moment resistance	$M_{Rd} := P_{IC} \cdot z = 10.7\ kN \cdot m$

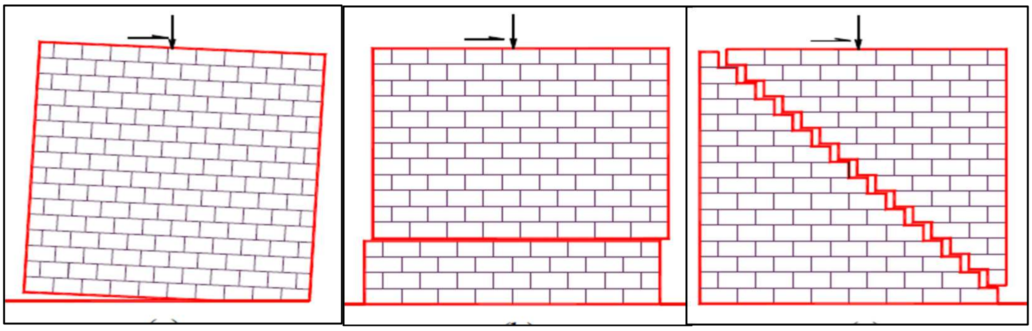
8 LEVEL V RETROFITTING STRATEGY

This chapter introduces possible failure mechanism of a masonry wall when laterally loaded, and it provides possible interventions for stiffening of walls affected by openings.

Lateral loads on masonry buildings are resisted primarily by in-plane action of walls oriented in the direction of loads. In case of seismic loads, the concern associated to URM walls to transfer load to foundations is about brittle behaviour of the material. Once the ultimate lateral loads is reached the capacity of the element rapidly decreases with very limited deformations.

Failure modes associated to the in-plane loading of URM walls is determined by the combination of the applied load, wall geometry and properties of the constituent materials.

Possible failure modes due to both lateral and vertical loads are shown in the Figure 8-1.



(a) Rocking failure *(b) Sliding/Shear Slip failure* *(c) Diagonal tension failure*

Figure 8-1 In-Plane Failure Mode of a Laterally Loaded URM Wall

8.1 Tension Controlled/Rocking Failure

This failure mode typically occurs when the applied axial compression load is low and the aspect ratio (slenderness) is high. Rocking is characterized by a rigid body rotation about or near the toe of the wall. The lateral force causes an overturning moment at the base of the wall, the introduced tension cracks the wall and propagates along the whole length. As the bearing area reduces, localized compression rupture may occur at the toe of the wall as shown in Figure 8-1(a).

8.2 Sliding/Shear-Slip Failure

Shear Slip failure is due to the sliding of the masonry above and below a mortar bed joint. Walls and wall piers with low slenderness are prone to this failure mode, especially if they are subjected to a low axial load. Typically slip occurs in the interface between masonry unit and mortar, rather than through mortar joints. The critical load for this failure mode is regulated by combination of adhesion and shear-friction resistance between mortar and masonry units.

8.3 Diagonal Tension Failure

For greater axial and lateral loads, the combination of shear and axial compressive stress leads in diagonal cracking. Whereas, both rocking and shear-slip failure are correlated to the strength of the mortar bond, diagonal tension failure also depends on the tensile strength of the masonry units. Diagonal cracking may assume a stepped pattern depending on the combination of vertical and shear forces. Stepped diagonal cracking path is also due to slip along one or more bed joints.

Mixed modes, or more likely, sequence of different behaviour modes are common in URM wall piers. The behaviour given different combination of load is resumed in Figure 8-2.

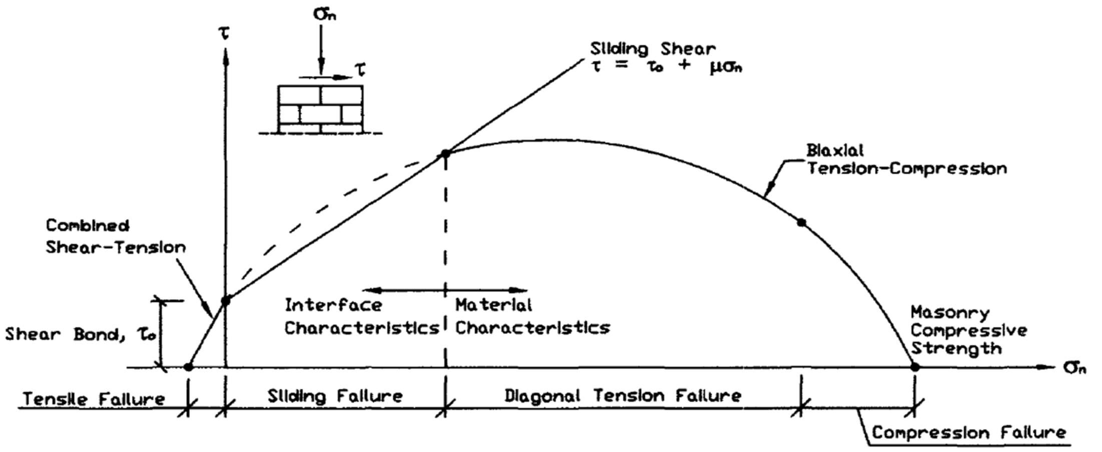


Figure 8-2 Behaviour of Ureinforced Masonry under Combined Shear and Normal Stress along the Mortar Bed Joints

Spandrels play an important role in the behaviour of piers in case of seismic loads. Spandrels stronger than piers can couple multiple piers and transfer overturning to adjacent piers, increasing axial stresses in end piers and probably changing their sequence of actions. The capacity of the spandrel to transmit vertical shear and bending moment regulates effects of element overturning and rocking. In case of weak wall spandrels respect to adjacent piers, they do not provide fixity at tops and bottoms of piers. This may result in piers acting as cantilevers.

8.4 In-plane wall stiffness

The magnitude of design lateral loads caused by the seismic event is dependent to the stiffness of the lateral bearing system of the structure besides other parameters regarding the seismic event itself. Stiffness of bearing elements such as solid masonry walls can be determined by means of the well-known standard deep beam theory. On the other hand, the presence of openings on a masonry wall radically changes the stress path and causes concentration of stress, which cannot be considered using the standard beam theory.

With the purpose to fill this theoretical gap, few methods have been developed to estimate the stiffness of a masonry shear wall with openings. Balasubramanian [25] elaborated a method, which is different respect others present in literature. It discretizes the masonry wall made out of spandrel and sill in a series of horizontal-system of piers instead of discretize spandrel and sill portions in a single solid element.

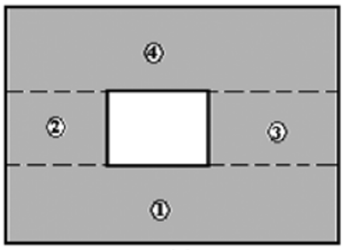


Figure 8-4 Discretization of Spandrel and Sill Portion in a Single Solid Element

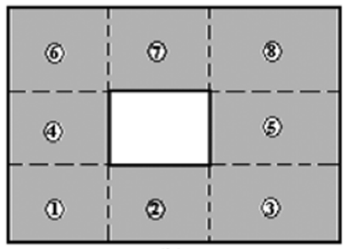


Figure 8-3 Discretization of Spandrel and Sill Portion Proposed by Balasubramanian [25]

The method proposed by Balasubramanian [25] show a better approximation since stress concentrations, causes of diagonal cracking, are likely to occur at corners of openings. Discretization methods which use a single solid element as shown in Figure 8-4 limits the possible failure within the pier assuming the condition of strong spandrel and weak pier. Differently the method proposed by Balasubramanian [25] assumes possible failure in both pier or spandrel portion.

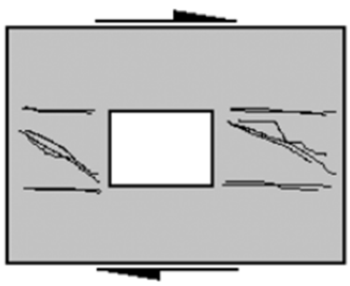


Figure 8-5 Possible Failure Limited in the Pier Portions

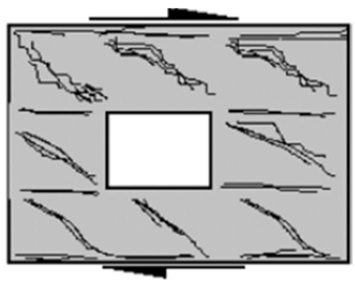


Figure 8-6 Possible Failure in both Spandrel or Pier Portion

However, Balasubramanian [25] points out that even if the discretization method is determined by possible failure mechanisms expected at limit state, the analysis conducted to determine the stiffness of a wall is limited to the linear elastic range.

After that the wall has been discretized as shown in Figure 8-3, piers are divided into four groups depending on the boundary conditions on their top and bottom faces. The wall before presented in Figure 8-3 is shown in Figure 8-8 with piers divided by type.

Discretization was carried out based on the following assumption:

1. Lateral deflection of every single pier is given by the sum of deflections due to bending and shear deflections.
2. Plane sections remain plane after the load is applied
3. Rotation of cross sections perpendicular to the longitudinal axis is given by bending rotation only
4. Top edge of the wall is considered restrained in the vertical direction
5. Shear interaction between horizontally consecutive piers is neglected

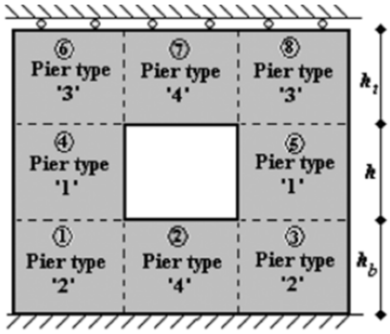


Figure 8-8 Different Types of Masonry Piers, Balasubramanian [25]

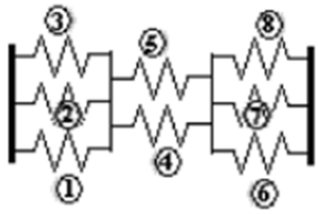


Figure 8-7 Representation of the Piers by Translational Springs, Balasubramanian [25]

For every pier type the stiffness is defined by Eq. [8-1]:

$$K = \frac{1}{pq^3 + 3q} \qquad [8\text{-}1]$$

Where p and q are parameters differently defined for every type of pier as listed in Table 8-1:

Pier type	Boundary conditions	Expressions for the parameters
1	Piers with both ends partially fixed	$p = \frac{q^4 + 4q^3 r + 4q^3 s + 3q^2 r^2 + 3q^2 s^2 + 14q^2 rs + 12qr^2 s + 12qrs^2 + 12r^2 s^2}{q^4 + q^3 r + q^3 s + 2q^2 rs}$ and $q = h/d$ where, $r = h_b/d$; $s = h_t/d$ and d = width of the pier
2	Piers with bottom end fixed and top end partially fixed	$p = (q+3s)/q$ and $q = h_b/d$ where, $s = (h+h_t)/d$ and d = width of the pier
3	Piers with top end fixed and bottom end partially fixed	$p = (q+3r)/q$ and $q = h_t/d$ where, $r = (h+h_b)/d$ and d = width of the pier
4	Cantilever piers	$p = 4$ and $q = h/d$ where, d = with of the pier

Table 8-1 Parameter Involved in the Expression for the Stiffness of Masonry Piers

8.5 Overlay Concrete On Shear Walls (Jacketing)

A new concrete overlay applied against an unreinforced masonry wall is considered a retrofitting technique for both in-plane and out-of-plane resistance of the element. The concrete overlay is typically reinforced with a welded wire fabric and attached to the masonry wall by means of adhesive anchors. It may either be cast-in-place or sprayed-in-place. Nowadays, sprayed concrete, known as shotcrete is preferred due to practical and fast application.

The ultimate load of one-side retrofitted wall with 90 mm thick shotcrete overlay tested in diagonal tension by Abrams and Lynch [26] in cyclic static test, was increased by a factor of 3.

8.5.1 Design Consideration

A masonry wall retrofitted by concrete overlay shows a strength three times greater than before, and a noticeable increase in ductility. Also the stiffness of the retrofitted wall at the peak lateral force is approximately three times the stiffness of the unreinforced one.

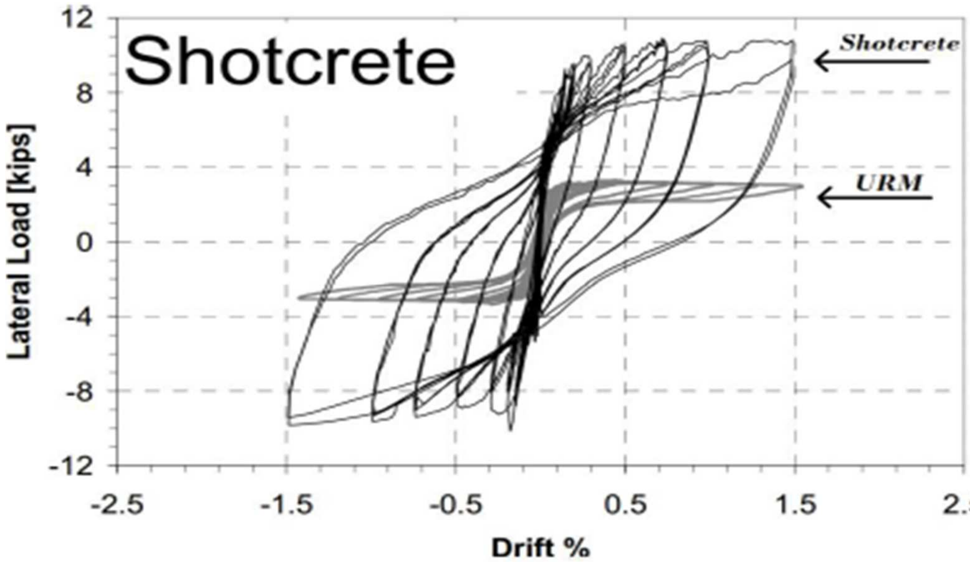


Figure 8-9 Hysteretic Curves for a Specimen before and after Retrofitting using Shotcrete. Amiraslanzadeh [27].

Design assumptions may be taken from different force-based design approaches given the relatively high strength of concrete compared to the masonry.

One of them, the most conservative, is to assume that the concrete overlay takes the 100% of demand tributary to the strengthened wall. This approach means that the masonry will be considerably damaged before the concrete reaches its design load.

Another approach is to distribute the load among both concrete and masonry elements, depending on their stiffnesses. This involves checking of both members to confirm that they are not overstressed.

In case of retrofitting a URM wall with openings, the concrete overlay is typically applied to wide piers. This means that the retrofitted pier shows high strength, but not enough stiffness to attract the load which it has been designed. Therefore the masonry will develop significant cracks at the ends of the overlay in the masonry spandrels before the concrete overlay takes the total load. This issue can be minimized by spreading out the influence of the overlay to the top and/or bottom spandrels as shown in Figure 8-10.

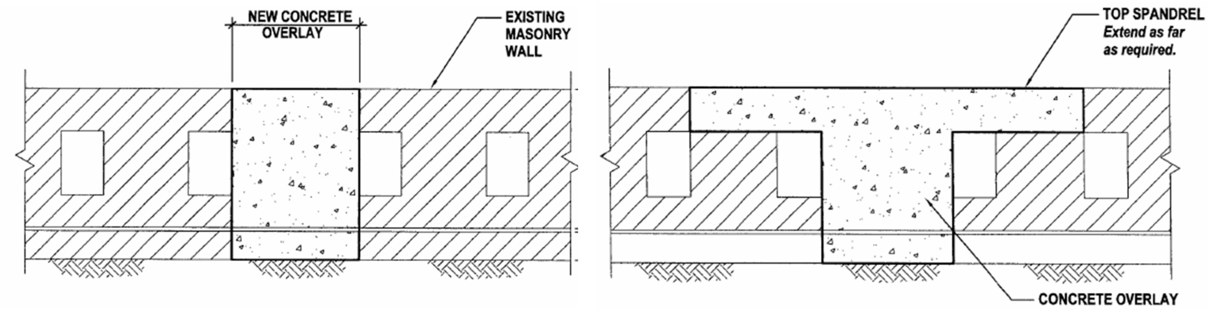


Figure 8-10 Concrete Overlay on Masonry Wall

If this technique is chosen as retrofit measure, considerations shall be taken for the wall out-of-plane action. The concrete overlay will increase the mass of the element, and therefore the seismic load will increase as well.

8.5.2 Detailing and Construction Consideration

- *Drilled dowels*: Connection between the overlay and masonry wall is typically realized by means of drilled dowels. Drilled dowels are able to transfer shear loads between the two materials.
- *Overlay base*: Precautions shall be taken about additional load of the shotcrete to the existing footing. The base of the shotcrete can be set on the ledge of the existing footing if possible, or a new footing should be provided.
- *Interface between diaphragms and wall:* Two possible scenarios may be designed for this interface. Conditions where floor joists run parallel to the wall are easier to address. The first joist closer to the wall is removed to apply the overlay, than a ledger is placed back and the floor sheathing can run over the ledge.

  When the joists are perpendicular to the wall, these can be embedded into the wall, but precautions shall be taken. Joists necessity an air gap on the top and sides, building paper on the bottom part should be present into the wall to minimize moisture effects. The typical solution is to cut joists and diaphragm in order to place the overlay, then joists are supported by steel connections between diaphragm and wall.

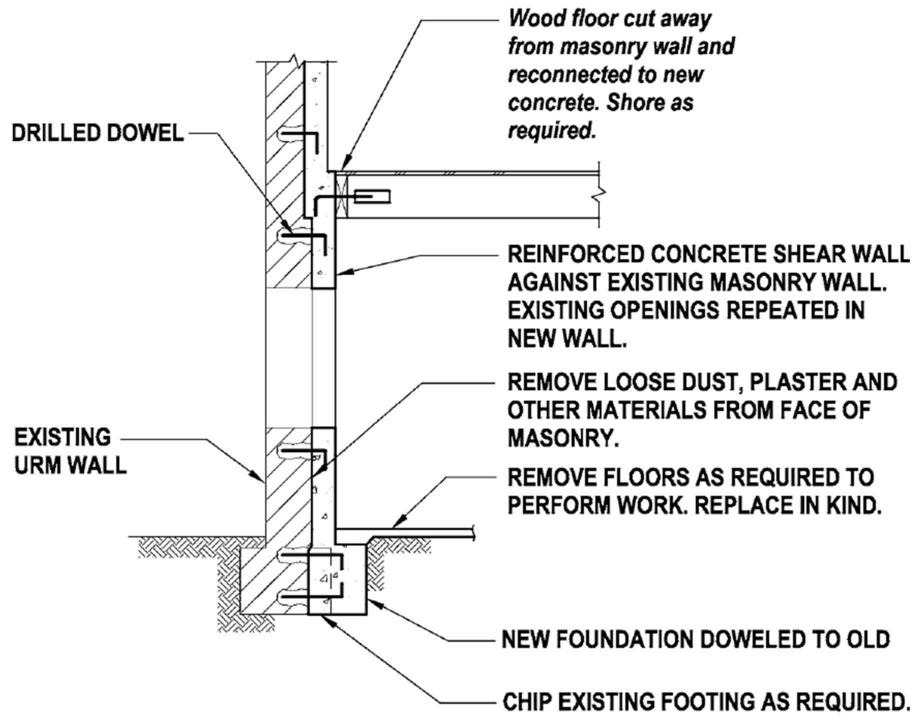


Figure 8-11 Concrete overlay on a Shear Wall

- *Additives and moisture:* concrete overlays can be affected to alkali salts leaching on the surface, which leads to white streaks or spots. These signs are due to both additives within the concrete and salts within the masonry wall. The use of low-alkali concrete is suggested.
- *Installation approach:* This technique of retrofitting consists of
  1. *Cleaned surface, watered and grinded*
  2. *Shear dowels placement*
  3. *Welded wire fabric placement*
  4. *Shrinkage control reinforcement setting*
  5. *Wall surface sprayed (in case of shotcrete)*

8.5.3 Cost and Disruption

Adding a new concrete layer can be quite disruptive, especially if applied with shotcrete. The choice between shotcrete and cast.-in-place concrete is regulated by conditions of access to the wall.

- *Shotcrete:* it is preferred when access for hose and concrete truck is possible and enough room is available to spray it. Since it is preferable to be sprayed downward, the use of scaffolding is necessary. Spraying is noisy and dusty, especially if it is done to an indoor wall, protection in the room is needed. Residue known as rebound forms at the base of the shoot, this must be cleaned away so that does not become part of the overlay.

- *Cast-in-place concrete:* it also necessities of access for hose and concrete truck, but it needs less front-side access. Furthermore, a front-side formwork is essential, which involves sawing and hammering noise of construction.

8.6 Composite Fibre Reinforcement (FRP strips)

The primary purpose of both externally-bonded FRP laminates and near-surface-mounted (NSM) FRP bars is to improve inadequate in-plane wall strength, besides they can also improve out-of-plane bending capacity. Strengthening of out-of-plane bending capacity of walls is described in Section 7.5 of this report. Fibre-reinforced polymer are typically made of glass or carbon fibres.

Externally-bonded FRP laminates are applied as an overlay on the wall to strengthening. The existing wall surface shall be prepared to the new material application, which shall be protected against ultraviolet rays afterward.

Near-surface-mounted FRP bars are typically applied vertically on the masonry wall. The wall face necessity less preparation, since the bars can be embedded in the horizontal mortar joints.

8.6.1 Design Considerations

Similarly to reinforced concrete overlay, when a FRP overlay is added to a rocking critical wall pier, the retrofit may be able to reduce cyclic degradation of the pier. Although, the strength and behaviour mode are not altered if FRP laminates are applied only on the pier surface. Whereas if the overlay crosses top and bottom of the pier into the spandrel, this may limit or prevent formation of a rocking mode. It will increase the strength, but the ductility will be reduced from that of a rocking-critical mode to that of a shear critical mode.

8.6.2 Detailing and Construction Considerations

- *Surface preparation:* for FRP overlay, the wall surface necessities to be cleaned of finishes and everything that prevents a proper adhesion. In some cases, sandblasting of the wall may be requested.
- *Continuity of FRP:* In the strengthening of walls and wall piers continuity is important. When in-plane loads need to be transferred from one storey to the next, continuity through the floor is necessary. It will require special attention for detailing in locations of shear connections between floor and diaphragm.
- *Aesthetics:* If FRP laminates are used, the wall surface necessities than to be plastered to hide the new overlay. When NSM FRP are used they are grouted into horizontal mortar joints. Which results in minimal appearance.
- *Moisture barrier:* When continuous overlays are used, moisture evaporation from the wall will be stopped, since fibre composites are impermeable. Special attention shall be taken for this issue. If the moisture accumulate underneath the overlay, it will begin to delaminate the fibre bond and lead to general building concerns.

8.6.3 Cost and disruption

Fibre composites are less expensive than in the past. Both FRP laminates and NSM FRP are to be applied along the total height of walls or wall piers. This means that the retrofit cannot be realized one storey at a time, but simultaneously for the total storey height and access is required at floor level. Furthermore, FRP laminates require wall surface preparation such as sandblasting. Fumes from adhesives used in application of the fibre composite are also cause of disruption. Consequently, occupants are to be relocated for the duration of intervention.

8.7 Exterior or Internal Axial Post Tensioning Ties

Vertical post-tensioning ties show considerable improvement in wall ultimate behaviour for both in-plane and out-of-plane, where the latter effect has been already explained in Section 0 of this report. It improves both cracks development and strength of the element due to the compressive force applied to the masonry, which counteracts the tensile stresses due to lateral loads.

Tendons are normally placed into holes drilled at mid-plane of the wall or along symmetrical groves on the wall surfaces. Those can be either bonded or un-bonded with the surrounding masonry depending if they are grouted or un-grouted. It has been shown by tests that if grouted post-tensioning is used in cavity walls, the lateral resistance increases of 40%, which it is higher than in the case of un-grouted post-tensioning. Furthermore, walls with un-bonded bars has more than double lateral drift than walls with bounded bars.

Horizontal post-tensioning requires further examination to state the effectiveness of this method. Although, a linear finite element model performed by Karantoni and Faradis [28]shows that if horizontal and vertical post-tensioning are combined together, the resulting improvement is higher than the sum of the individual effects.

For specifications regarding design consideration, detailing, cost and disruption see Section 7.2 of this report.

8.8 Reinforcing Ring at Opening Locations

In-plane strength of masonry walls is strongly affected by the presence of openings. Strength may be enhanced by rethinking an approach which consists of making RC or steel frames at new opening locations.

This technique is regularly used to restore the necessary stiffness of a wall when a new opening has been planned. Because, when the new opening is on a structural wall, the initial stiffness of the wall in its in-plane direction shall be restored.

It is a common practise to preserve the initial stiffness by means of steel or reinforced concrete frames introduced into the opening. The frame should be as stiff as the missing wall panel and well connected to the surrounding masonry to obtain its best performance.

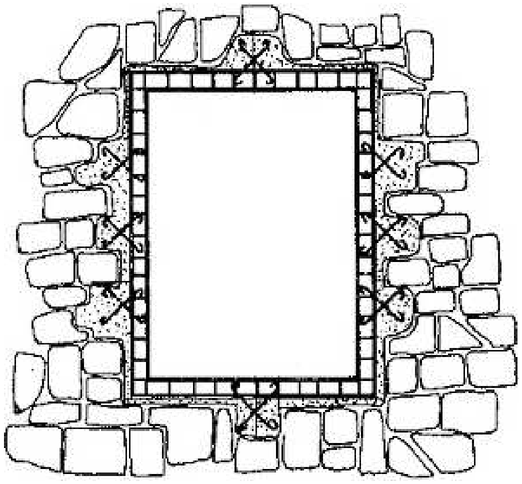


Figure 8-12 Concrete Reinforcing Ring

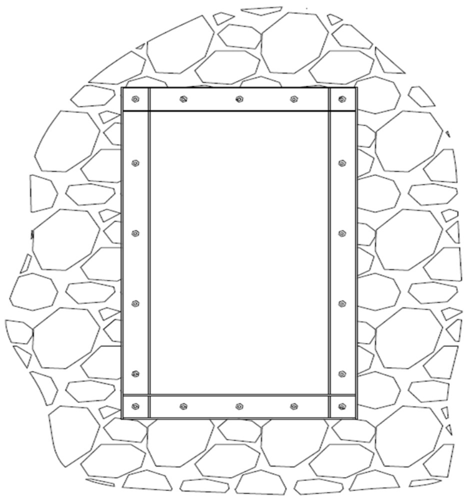


Figure 8-13 Steel Reinforcing Ring

Hence in practice, the introduction of frames as shown in Figure 8-12 and Figure 8-13 represents an addition of stiffness in the in-plane direction.

Based on this concept, this technique may be adapted as strengthening measure in case openings are already present along the wall. In this report the solution of reinforcing steel ring has been considered.

8.8.1 Design Consideration

Reinforcing rings are placed at opening locations, hence the possible increment of stiffness is strongly dependent on the quantity of openings present on the wall. Steel frames may be connected to the existing edge of the openings or be recessed into the surrounding masonry. When a wall is affected by a large amount of openings, the bearing system may be reduced only to a set of slender piers. In this case it is suggested the addition of frames exclusively into the biggest window openings without removing of masonry. Removing the masonry would increase the slenderness of piers making of the increment of stiffness a small variation in the total stiffness of the wall. When no removing of masonry around openings is intended, it is suggested to select the biggest openings.

The effectiveness of this solution is proportional to the quality of the masonry. When the building presents low quality masonry it is more effective than when the wall material shows good mechanical properties.

8.8.2 Detailing and Construction Consideration

- *Small windows and door openings:* In case of small windows the addition of frame would result in a considerably reduction of the opening area. Similarly, for door openings the consequence would be a narrower door and a less stiff frame given the greater height of it.
- *Connection to the masonry:* The aim of the design is to make the introduced frame resist in parallel with the surrounding piers when horizontal loads are acting on the wall. This would mean that it is possible to sum the resistance capacity of the frame with the capacities of adjacent piers. In order to obtain this behaviour the connection between frame and masonry shall assure total shear transfer and no sliding mechanisms. This may be obtained by means of bar grouted into the masonry and bolted to the steel frame
- *Cross sections:* Sizing of the resistant cross section of the steel frame is limited by the thickness of the wall and by the size of the opening. It is suggested the use of standard I section profiles.
- *Moment resisting connections:* The reinforcing steel frame is assumed to be composed by moment resisting joints. This can be achieved by means of both welded and bolted connections. The two solutions are shown in Figure 8-14 and Figure 8-15 respectively.

Figure 8-14 Welded Moment Resistant Connection

Figure 8-15 Bolted Moment Resistant Connection

- *Installation approach:* Construction phases may be summarized as:

1 Removal of the existing window frame
2 Preparation of the masonry wall around the opening
3 Possible removal of masonry
4Connection of frame with the existing masonry

8.9 Cost and Disruption

The cost of this solution is mostly affected by the assembly phase on the building site and the construction of the steel frame, which can be realized in advance in the shop. Disruption for occupants is relative to the number of openings which are planned to be hooped. The intervention may be realized in one room at the time, which would allow to do not relocate the occupants.

8.10 Conclusion about level V retrofitting strategy

The aim of a damage limitation retrofitting is to prevent damage of the structure during a seismic event. This in case of URM buildings may be achieved by increase the stiffness of bearing elements affected by their characteristic brittle behaviour.

Stiffening of URM walls in their in-plane direction may be achieved by means of different retrofitting techniques. Three of them have been described along the previous sections.

Overlay concrete so-called jacketing show a remarkable improvement in the resistance of the element. However it totally changes the appearance of the wall faces when this in not originally covered by plaster. Moreover, jacketing is preferable to be applied by shotcrete, which makes this solution often not possible for house building. The use of shotcrete, involves the necessity of access for truck and enough room to spray it, besides the needing of empty the house.

External or internal axial post tensioning ties, as already discussed for retrofitting of walls in their out-of-plane direction, show a considerable improvement in preventing cracks. Despite the improvements in the behaviour of the wall, they involves complex construction procedures, due to the drilling of long and straight holes, and anchoring at foundation level.

Reinforcing rings are typically adopted in the case of a new opening in an existing building. In order to restore the initial stiffness of the wall, a steel or concrete frame is inserted around the new opening.
Consequently, where openings are already present, introduction of frames may increase the total stiffness of the wall.
This has been assumed to be the most technically reliable retrofitting technique for this level of strategy due to the minimal disruption during execution compared with other techniques. Moreover, steel frames may be prefabricated in the shop and assembled at the building site in a shorter time, respect other solutions.

8.11 Calculation Example of the Chosen Technique

For decades buildings in the Netherland have been designed accounting only the wind load effect as lateral load. However, with arising of seismic events in the Groningen area, buildings show lack of stiffness in the bearing system to withstand seismic loads. This lack is particularly evident in terraced houses. Terraced houses with their elongated geometry are provided with strong bearing walls to resist lateral loads perpendicular to their longest sides, which is the case of greatest wind load. However, when lateral loads are acting in the other direction (x direction in Figure 8-16), the bearing system is represented by long walls which compose the façade of the house. These walls are typically affected by a considerable amount of openings which reduced the bearing structure to a set of slender piers.

For the calculation example of retrofitting URM walls in their in-plane direction, a wall affected by the before mentioned aspects has been selected from a two storeys terraced house.
Figure 8-16 shows the selected house building with sizes of piers indicated.
Masonry has been assumed to have Modulus of elasticity $\underline{E_m = 2500 \text{ Mpa}}$ and thickness $\underline{t = 210}$.

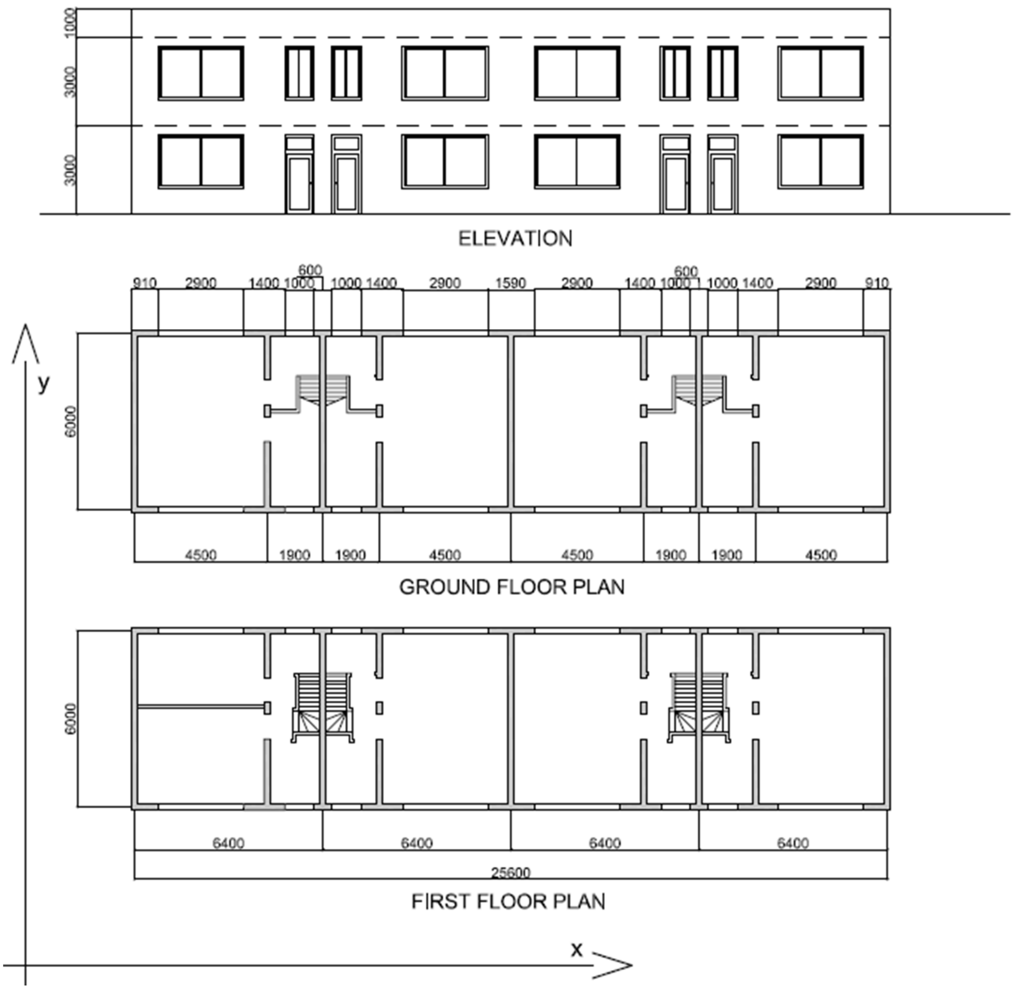


Figure 8-16 Selected Dutch Terraced House with Reference System

8.11.1 Elastic Lateral Stiffness of the Selected Wall

The URM wall for the analysis has been discretized in 80 pier portions following the procedure proposed by Balasubramanian [25] and described in this report in Section 8.4.

Firstly the wall has been divided in two macro elements which represent the bearing structures of each storey as shown in Figure 8-17. Secondly, each macro element has been further divided in pier portions.

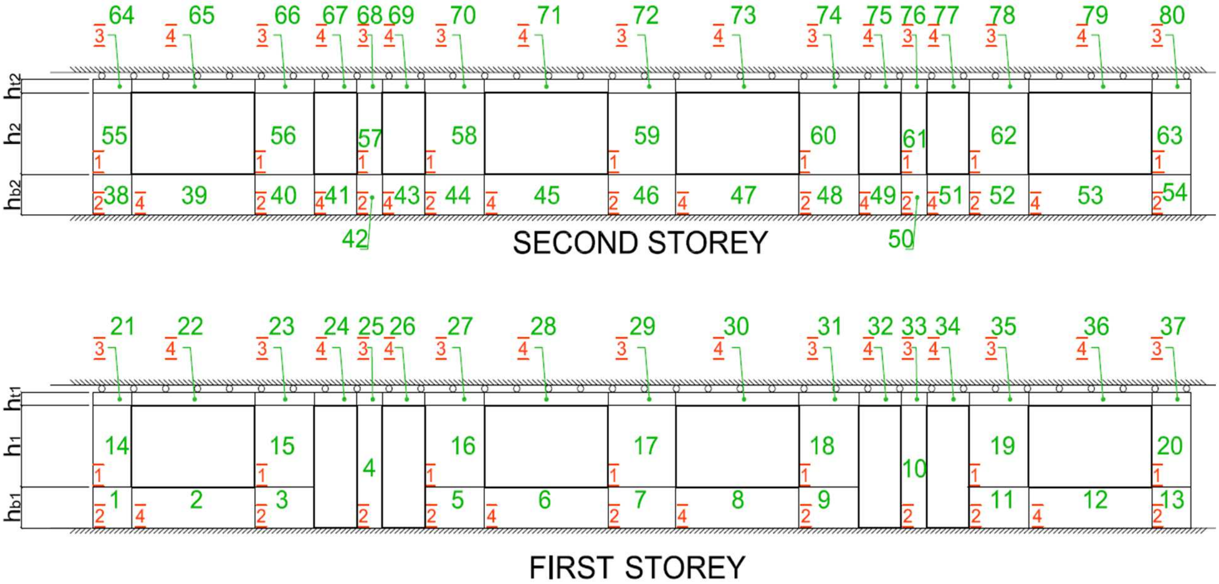


Figure 8-17 Discretization of Bearing Walls in the Initial Situation

At each pier portion and identification number from 1 to 80 has been assigned, and also every pier type has been defined depending on its boundary conditions.

Consequently, after that both geometry and boundary conditions for every wall element were assumed, the stiffness for each member has been determined.

Every member of the macro element may be represented by means of translational spring. Figure 8-18 displays the system of springs standing for the first storey bearing structure. Consequently the equivalent stiffness of the whole first storey $K_{1,1}$ can be determined by means of Eq. [8-2].

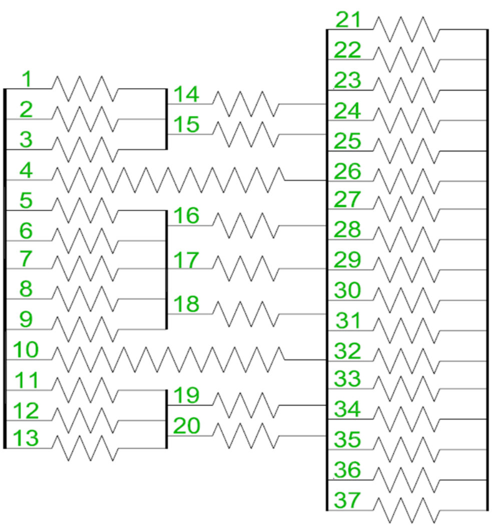


Figure 8-18 Assemblage of Masonry Piers of First Storey Wall in the Initial Situation

$$\frac{1}{K_{1.1}}=\frac{1}{\dfrac{1}{\dfrac{1}{K_1+K_2+K_3}+\dfrac{1}{K_{14}+K_{15}}}+K_4+\dfrac{1}{\dfrac{1}{K_5+K_6+K_7+K_8+K_9}+\dfrac{1}{K_{16}+K_{17}+K_{18}}}+K_{10}+\dfrac{1}{\dfrac{1}{K_{11}+K_{12}+K_{13}}+\dfrac{1}{K_{19}+K_{20}}}}+$$
$$+\frac{1}{K_{21}+K_{22}+K_{23}+K_{24}+K_{25}+K_{26}+K_{27}+K_{28}+K_{29}+K_{30}+K_{31}+K_{32}+K_{33}+K_{34}+K_{35}+K_{36}+K_{37}} \qquad [8\text{-}2]$$

Table 8-2 lists the data used to calculate the stiffness of every element besides the equivalent stiffness of the entire macro element.

Pier type	Pier number	Geometry				Parameters				Element stiffness		Stiffness of coupled elements	Eq. stiffness
		d [mm]	h_{b1} [mm]	h_1 [mm]	h_{t1} [mm]	r	s	q	p	K/Et	k [kN/m]	k [kN/m]	$K_{1.1}$ [kN/m]
2	1	910	900	1800	300		2.308	0.989	8.00	0.093	86501	643517	408526
4	2	2900	900	1800	300			0.621	4.00	0.355	328571		
2	3	1400	900	1800	300		1.500	0.643	8.00	0.247	228445		
2	4	600	2700	0	300		0.500	4.500	1.33	0.007	6860	6860	
2	5	1400	900	1800	300		1.500	0.643	8.00	0.247	228445	1408128	
4	6	2900	900	1800	300			0.621	4.00	0.355	328571		
2	7	1590	900	1800	300		1.321	0.566	8.00	0.318	294096		
4	8	2900	900	1800	300			0.621	4.00	0.355	328571		
2	9	1400	900	1800	300		1.500	0.643	8.00	0.247	228445		
2	10	600	2700	0	300		0.500	4.500	1.33	0.007	6860	6860	
2	11	1400	900	1800	300		1.500	0.643	8.00	0.247	228445	643517	
4	12	2900	900	1800	300			0.621	4.00	0.355	328571		
2	13	910	900	1800	300		2.308	0.989	8.00	0.093	86501		
1	14	910	900	1800	300	0.989	0.330	1.978	3.50	0.03	28046	110031	
1	15	1400	900	1800	300	0.643	0.214	1.286	3.50	0.089	81985		
1	16	1400	900	1800	300	0.643	0.214	1.286	3.50	0.089	81985	273255	
1	17	1590	900	1800	300	0.566	0.189	1.132	3.50	0.118	109284		
1	18	1400	900	1800	300	0.643	0.214	1.286	3.50	0.089	81985		
1	19	1400	900	1800	300	0.643	0.214	1.286	3.50	0.089	81985	110031	
1	20	910	900	1800	300	0.989	0.330	1.978	3.50	0.03	28046		
3	21	910	900	1800	300	2.967		0.330	28.00	0.502	464855	8005129	
4	22	2900	900	1800	300			0.621	4.00	0.355	328571		
3	23	1400	900	1800	300	1.929		0.214	28.00	1.089	1008420		
4	24	1000	900	1800	300			1.800	4.00	0.035	32237		
3	25	600	900	1800	300	4.500		0.500	28.00	0.2	185220		
4	26	1000	900	1800	300			1.800	4.00	0.035	32237		
3	27	1400	900	1800	300	1.929		0.214	28.00	1.089	1008420		
4	28	2900	900	1800	300			0.621	4.00	0.355	328571		
3	29	1590	900	1800	300	1.698		0.189	28.00	1.326	1228066		
4	30	2900	900	1800	300			0.621	4.00	0.355	328571		
3	31	1400	900	1800	300	1.929		0.214	28.00	1.089	1008420		
4	32	1000	900	1800	300			1.800	4.00	0.035	32237		
3	33	600	900	1800	300	4.500		0.500	28.00	0.2	185220		
4	34	1000	900	1800	300			1.800	4.00	0.035	32237		
3	35	1400	900	1800	300	1.929		0.214	28.00	1.089	1008420		
4	36	2900	900	1800	300			0.621	4.00	0.355	328571		
3	37	910	900	1800	300	2.967		0.330	28.00	0.502	464855		

Table 8-2 Calculation of Stiffness of the First Storey Wall

The same procedure followed to assess the stiffness $K_{1,1}$ of the first storey has been carried out for the second storey bearing structure.

Data regarding wall pier portions of the second storey wall are listed in Table 9-1 on Annex E

The stiffnesses of the two storeys are therefore:

$$K_{1.1} = 231591 \ {kN}/{m}$$
$$K_{2.1} = 233489 \ {kN}/{m}$$

The chosen retrofitting technique involves the stiffening of the wall by means of reinforcing steel rings at opening locations.
For this calculation example the introduction of four steel frames for each storey wall has been assumed.
Figure 8-19 presents the disposition of steel frames, which have been placed only in openings of largest windows. Reinforcing rings have been numbered from 81 to 88.

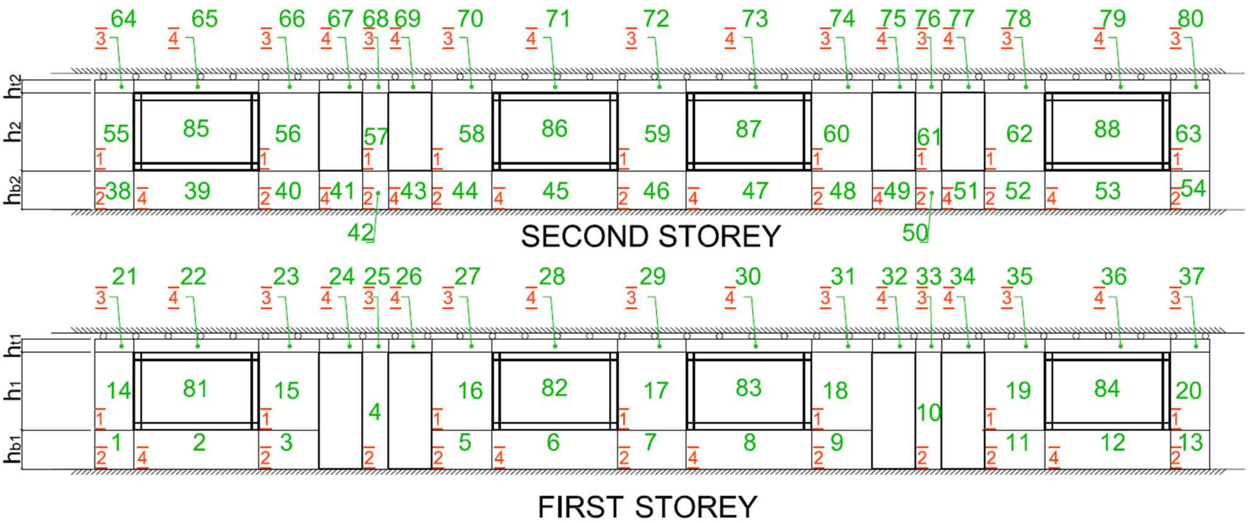


Figure 8-19 17 Discretization of Bearing Walls in the Retrofitted Situation

Hence, the new equivalent stiffness of the two macro elements was assessed.
Similarly how the spring system was computed in the case of the not retrofitted first storey macro element, Figure 8-20 shows the equivalent translational spring system relative to the new first storey resisting member set.

The stiffness contribution of reinforcing rings added to the wall is symbolized by red springs added in the set.

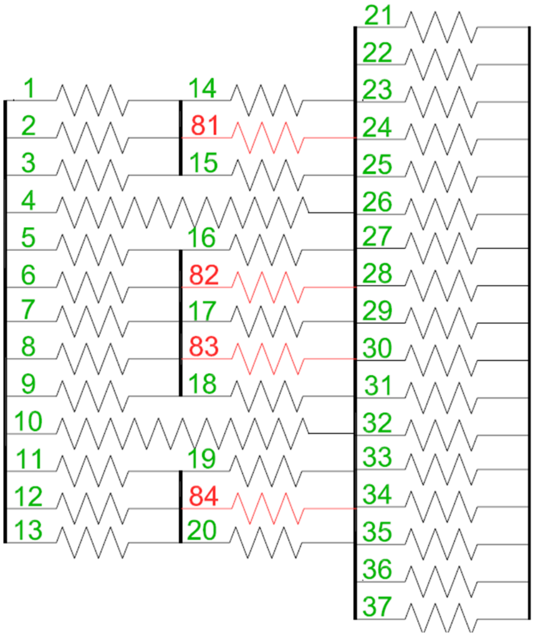


Figure 8-20 Assemblage of Masonry Piers of First Storey Wall in the Retrofitted Situation

The equivalent stiffness of the whole URM wall $K_{1.2}$ can be determined by Eq. [8-3].

$$\frac{1}{K_{12}}=\frac{1}{\frac{1}{\frac{1}{K_1+K_2+K_3}+\frac{1}{K_{14}+K_{81}+K_{15}}}+K_4+\frac{1}{\frac{1}{K_5+K_6+K_7+K_8+K_9}+\frac{1}{K_{16}+K_{82}+K_{17}+K_{83}+K_{18}}}+K_{10}+\frac{1}{\frac{1}{K_{11}+K_{12}+K_{13}}+\frac{1}{K_{19}+K_{84}+K_{20}}}}+\frac{1}{K_{21}+K_{22}+K_{23}+K_{24}+K_{25}+K_{26}+K_{27}+K_{28}+K_{29}+K_{30}+K_{31}+K_{32}+K_{33}+K_{34}+K_{35}+K_{36}+K_{37}} \quad [8\text{-}3]$$

The same procedure has been carried out for the retrofitted second storey wall, Table 9-2 Table 9-3 on Annex E displays the relative data for both retrofitted walls.

Therefore after the addition of 4 steel frames on both walls the stiffnesses of storeys are:

$$K_{1.2}=335328\ {kN}/{m}$$
$$K_{2.2}=338604\ {kN}/{m}$$

Comparing the values of stiffness before and after the retrofitting, it can be state that both first second storey bearing systems have experienced an increase of about the 45% respect to the initial stiffness.

In order to assess the efficacy of this retrofitting technique, a further comparison is necessary.

Additional steel reinforcing rings in the bearing walls of a masonry building enhance the lateral stiffness as well as increase the weight of the building.

Since the magnitude of seismic loads is dependent from the weight of the building, it is necessary to confirm that the total displacement of the building is effectively reduced in case of seismic event. This comparison is of crucial importance in order to define this technique as effective for damage limitation retrofit of URM buildings.

Hence, firstly the total mass of the building has been calculated in the condition before the retrofitting intervention. Consequently the period of vibration T_1 has been determined by means of Rayleigh's method.

After that the period T_1 was derived, seismic loads in the x direction of the building were determined on the basis of its design spectrum. Hence, horizontal seismic loads were applied at each storey level, relative displacements of walls were assessed.

Respect to the previous calculation example in this report, in this elaboration, the design ground acceleration has been raised to 0.4g. It is assumed representing a hazard value for the in-plane capacity of walls in terraced house.

BUILDING	Dimensions	$l_x := 25600\ mm$
		$l_y := 6000\ mm$
	Parapet height	$h_p := 1000\ mm$
	First storey height	$h_2 := 6000\ mm$
	Second storey height	$h_1 := 3000\ mm$
	Wall thickness	$t := 210\ mm$
	Masonry weight	$\rho_m := 1800\ \frac{kg}{m^3}$
	Masonry modulus of elasticity	$E_m := 2500\ MPa$

Diaphragm dead load $g_{k.j} := 180\ \frac{kg}{m^2}$

Timber floor single sheathing: 45 Kg/m2
Ceramic tiles and glue: 55 Kg/m2
Concrete topping overlay: 80 Kg/m2

Live load $q_{k.i} := 200\ \frac{kg}{m^2}$

Weight of diaphragm per sq meter (see Annex A)

$$q_{G.d} := g_{k.j} + (0.3 \cdot 1.0 \cdot q_{k.i}) = 240\ \frac{kg}{m^2}$$

Diaphragm mass

$$m_d := l_x \cdot l_y \cdot q_{G.d} = (3.686 \cdot 10^4)\ kg$$

Building mass
(openings have not been considered)

$$m_{b.1} := \rho_m \cdot t \cdot ((2 \cdot h_2 \cdot l_x) + (9 \cdot h_2 \cdot l_y) + (h_p \cdot (2 \cdot l_x + 2 \cdot l_y))) + (3 \cdot m_d) = (3.731 \cdot 10^5)\ kg$$

Storey stiffnesses

$$K_{1.1} := 231591\ \frac{kN}{m}$$

$$K_{2.1} := 233489\ \frac{kN}{m}$$

Storey displacements

$$\delta_1 := \frac{(m_d + m_d) \cdot g}{K_{1.1}} = 3.12\ mm$$

$$\delta_2 := \delta_1 + \frac{m_d \cdot g}{K_{2.1}} = 4.67\ mm$$

Fundamental period of vibration

$$T_1 := 2 \cdot \pi \cdot \sqrt{\frac{\left(m_d \cdot \delta_1^2 + m_d \cdot \delta_2^2\right)}{g \cdot \left(m_d \cdot \delta_1 + m_d \cdot \delta_2\right)}} = 0.128\ s$$

Diaphragm seismic load calculation in the initial situation

Correction factor $\lambda := 0.85$ $\beta := 0.2$

Parameters describing the recommended Type 2 elastic response spectra for ground type E $T_B := 0.05\ s$ $T_C := 0.25\ s$ $T_D := 1.2\ s$ $S := 1.6$

$T_1 = 0.128\ s$ Therefore $T_B \le T_1 \le T_C$

Design ground acceleration $a_g := 0.4\ g$

Behaviour factor for URM $q := 1.5$

Response spectrum acceleration

$$S_{d.1}(T_1) := a_g \cdot S \cdot \frac{2.5}{q} = 10.46\ \frac{m}{s^2}$$

$$\beta \cdot a_g = 0.785\ \frac{m}{s^2}$$

Base shear force

$$F_{b.1} := S_{d.1}(T_1) \cdot m_{b.1} \cdot \lambda = \left(3.32 \cdot 10^3\right)\ kN$$

Horizontal force first storey (see Figure below)

$$F_{1.1} := F_{b.1} \cdot \left(\frac{h_1 \cdot m_d}{(h_1 \cdot m_d) + (h_2 \cdot m_d)}\right) = \left(1.11 \cdot 10^3\right)\ kN$$

Horizontal force second storey (see Figure below)

$$F_{2.1} := F_{b.1} \cdot \left(\frac{h_2 \cdot m_d}{(h_1 \cdot m_d) + (h_2 \cdot m_d)}\right) = \left(2.21 \cdot 10^3\right)\ kN$$

First storey wall displacement

$$\Delta_{W1.1} := \frac{F_{1.1}}{2 \cdot K_{1.1}} = 2.39\ mm$$

Second storey wall displacement

$$\Delta_{W2.1} := \Delta_{W1.1} + \frac{F_{2.1}}{2 \cdot K_{2.1}} = 7.12\ mm$$

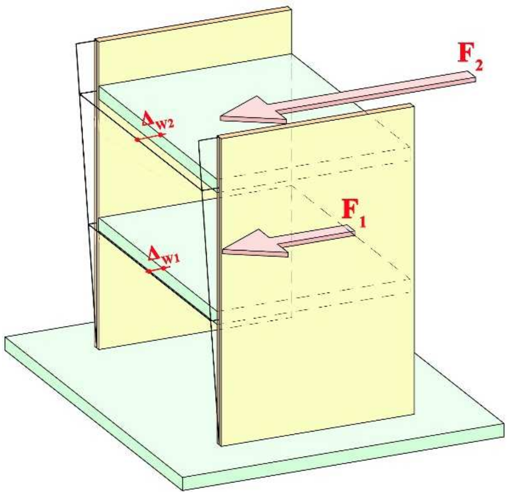


Figure 8-21 Seismic Loads and Wall Displacements

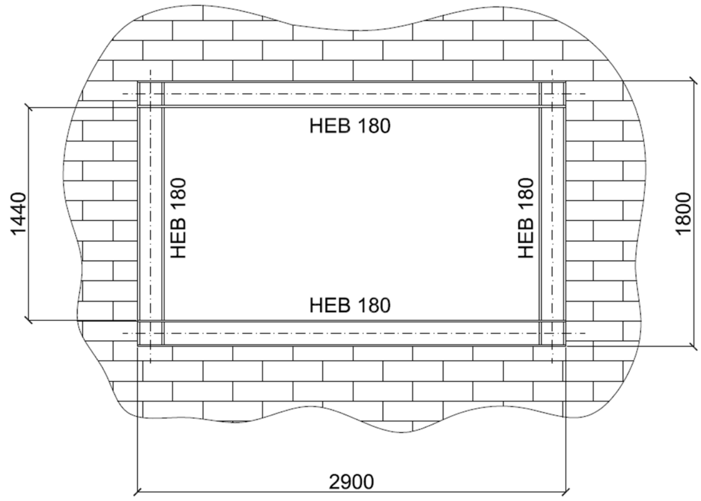


Figure 8-22 Steel Reinforcing Ring

The same procedure carried out for the assessment of in-plane wall displacement in the condition antecedent the seismic upgrading has been followed to assess displacement of retrofitted walls.

On the first stage, the additional weight of the reinforcing rings has been determined and added to the weight of the building. Figure 8-22 shows the steel reinforcing rings assumed in this calculation example.

STEEL FRAME

HEB 180

$E_s := 210000\ MPa$

$h_p := 180\ mm$

$I_p := 3831\ cm^4$

$\rho_p := 51.2\ \frac{kg}{m}$

OPENING

$h_o := 1800\ mm$

$b_o := 2900\ mm$

Steel frame mass

$$m_{fr} := \rho_p \cdot \left(\left(2 \cdot \left(h_o - 2 \cdot h_p\right)\right) + \left(2 \cdot b_o\right)\right) = 444\ kg$$

Building mass after retrofitting of masonry walls

$$m_{b.2} := m_{b.1} + \left(16 \cdot m_{fr}\right) = \left(3.802 \cdot 10^5\right)\ kg$$

Steel frame stiffness

$$K_{fr} := \frac{2 \cdot 12 \cdot E_s \cdot I_p}{\left(h_o - h_p\right)^3} = 45415\ \frac{kN}{m}$$

Storey stiffnesses	First storey	$K_{1.2} := 335328 \frac{kN}{m}$
	Second storey	$K_{2.2} := 338604 \frac{kN}{m}$
Storey displacements	First storey	$\delta_1 := \frac{(m_d + m_d) \cdot g}{K_{1.2}} = 2.156\ mm$
	Second storey	$\delta_2 := \delta_1 + \frac{m_d \cdot g}{K_{2.2}} = 3.224\ mm$
Fundamental period of vibration		$T_2 := 2 \cdot \pi \cdot \sqrt{\frac{\left(m_d \cdot \delta_1^{\ 2} + m_d \cdot \delta_2^{\ 2}\right)}{g \cdot \left(m_d \cdot \delta_1 + m_d \cdot \delta_2\right)}} = 0.106\ s$

Diaphragm seismic load calculation in the initial situation

Correction factor $\lambda := 0.85$ $\beta := 0.2$

Parameters describing the recommended Type 2 elastic response spectra for ground type E: $T_B := 0.05\ s$ $T_C := 0.25\ s$ $T_D := 1.2\ s$ $S := 1.6$

$T_2 = 0.106\ s$ Therefore $T_B \leq T_2 \leq T_C$

Design ground acceleration $a_g := 0.4 \cdot g$

Behaviour factor for URM $q := 1.5$

Response spectrum acceleration $S_{d.1}(T_2) := a_g \cdot S \cdot \frac{2.5}{q} = 10.46\ \frac{m}{s^2}$

Base shear force $F_{b.2} := S_{d.1}(T_2) \cdot m_{b.2} \cdot \lambda = \left(3.38 \cdot 10^3\right)\ kN$

Horizontal force first storey (see figure below) $F_{1.2} := F_{b.2} \cdot \left(\frac{h_1 \cdot m_d}{(h_1 \cdot m_d) + (h_2 \cdot m_d)}\right) = \left(1.13 \cdot 10^3\right)\ kN$

Horizontal force second storey (see figure below) $F_{2.2} := F_{b.2} \cdot \left(\frac{h_2 \cdot m_d}{(h_1 \cdot m_d) + (h_2 \cdot m_d)}\right) = \left(2.25 \cdot 10^3\right)\ kN$

First storey wall displacement $\Delta_{W1.2} := \frac{F_{1.2}}{2 \cdot K_{1.2}} = 1.68\ mm$

Second storey wall displacement $\Delta_{W2.2} := \Delta_{W1.2} + \frac{F_{2.2}}{2 \cdot K_{2.2}} = 5.01\ mm$

It can be state that this retrofitting technique besides increasing the stiffness, it decreases the total displacement for the structure. In this calculation example the reduction for both first and second storey walls is about 30% of the initial displacement.

As last check, the initial lateral resistance of the wall should be determined.

8.11.2 Strength of an Unreinforced Masonry Wall Affected By Openings

In order to state the necessity of retrofit for the URM wall analysed in the previous section, a check should be done in order asses if the initial resistance of the wall is sufficient to withstand seismic loads.

Prediction of ultimate strength of a shear walls with opening based on formation of plastic hinges has been carried out by Leiva[29],[30],[31]. The model for the prediction has been confirmed by Elshafie[32] by means of tests, which show that a flexural-dominated masonry walls with openings fails by forming plastic hinges at the ends of the members.

This model allows to determine the ultimate lateral load resistance by means of a plastic analysis for assumed plastic collapse mechanism. Where the sequence of plastic hinge formation is related to strength and stiffness of members.

Formation of plastic hinges leads to failure mechanism, for a flexural-dominated masonry walls with openings the possible failure mechanisms are:

- Strong pier/weak spandrel mechanism, where plastic hinges firstly form at both ends of spandrels, and then at the pier bases. Figure 8-25
- Strong spandrel/weak pier mechanism, in which the system fails by forming hinges at both ends of piers. Figure 8-24
- Mixed mechanism, which is a combination of the previous mechanisms. Figure 8-23

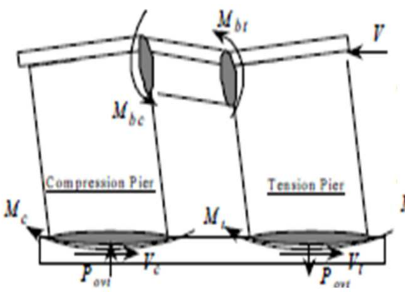


Figure 8-25 Strong Pier/Weak Spandrel Failure Mechanism Elshafie[32]

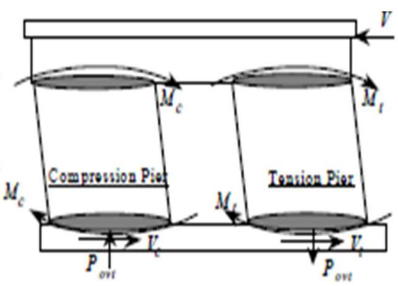


Figure 8-24 Strong Spandrel/Weak Pier Failure Mechanism Elshafie[32]

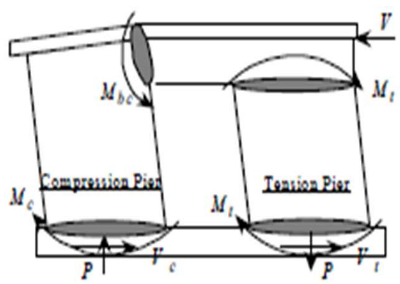


Figure 8-23 Mixed Failure Mechanism Elshafie[32]

As before mentioned, the correct choice of the failure mechanism depends on the strength of the wall elements. Once that the real failure mechanism is determined by means of the flexural strength of the elements, it is possible to determine the ultimate lateral load capacity of the wall by means of equilibrium.

8.11.3 Ultimate Lateral Load Capacity of the Analysed Wall

The analysed masonry wall has been divided in 4 wall panels of equal geometry for an easier computation. The adopted division of the wall is shown in Figure 8-26.

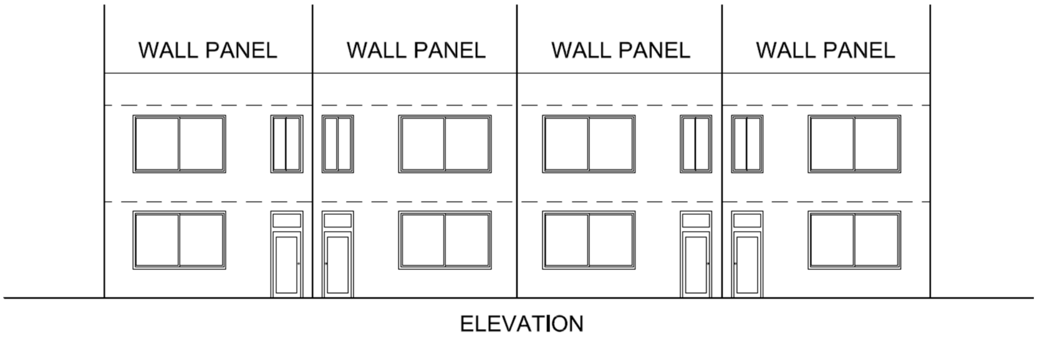


Figure 8-26 Division of Masonry Wall in Wall Panels

Consequently, each wall panel may be further divided in resisting members such as wall piers and wall spandrels. Figure 8-1 displays the assumed resistant members on the wall panels which have been accounted for the strength of the masonry wall, besides the lateral force at storey levels.

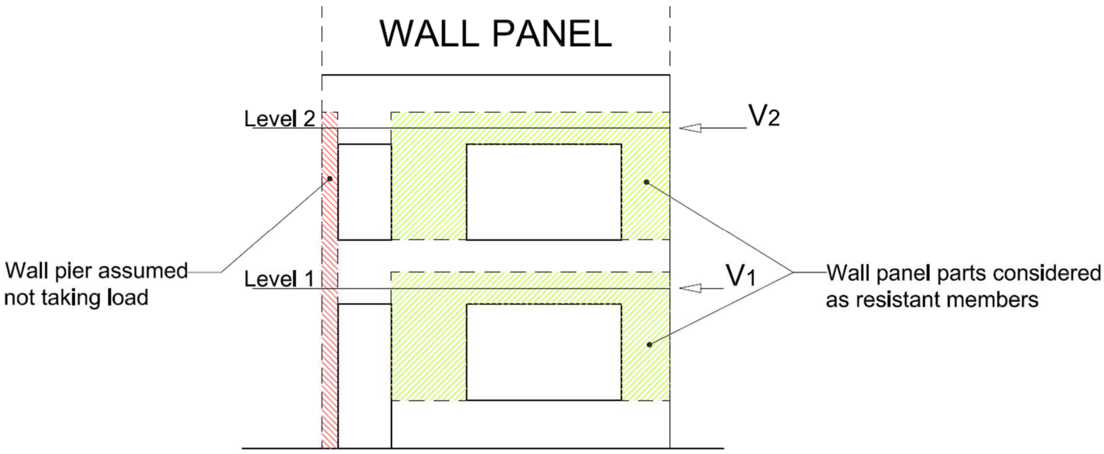


Figure 8-28Wall Panel Assumptions

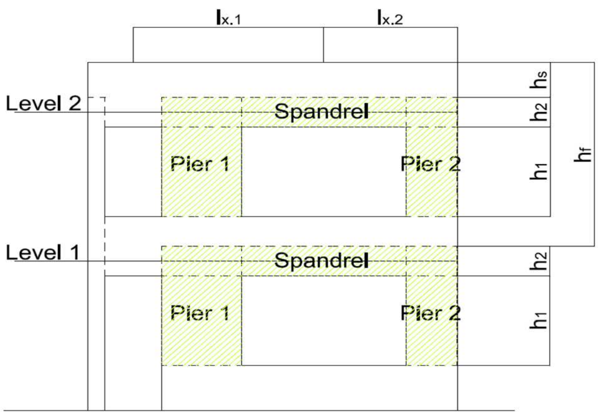


Figure 8-27 Wall Panel Geometry

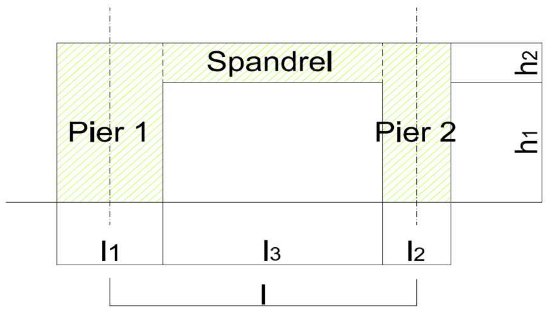


Figure 8-29 Resisting System Geometry

WALL PANEL GEOMETRY

Height of masonry above first storey piers	$h_f := 3700\ mm$
Height of masonry above second storey piers	$h_s := 700\ mm$
Length of masonry supported by pier 1	$l_{x.1} := 3350\ mm$
Length of masonry supported by pier 2	$l_{x.2} := 2360\ mm$
Wall thickness	$t := 210\ mm$
Height of piers	$h_1 := 1800\ mm$
Depth of the spandrel	$h_2 := 300\ mm \quad d_{sp} := h_2$
Total height	$h := h_1 + h_2 = \left(2.1 \cdot 10^3\right)\ mm$
Span between pier axes	$l := 4055\ mm$
Length of pier 1	$l_1 := 1400\ mm$
Length of pier 2	$l_2 := 910\ mm$
Distance between piers	$l_3 := 2900\ mm$

MASONRY CHARACTERISTICS

Modulus of Elasticity	$E_m := 4410\ MPa$
Compressive strength	$f_{ck} := 4.4\ MPa$
Bending strength parallel to the bed joint	$f_{xk.1} := 0.2\ MPa$
Bending strength perpendicular to the bed joint	$f_{xk.2} := 0.4\ MPa$
Average bond strength	$\nu_{te} := 0.1\ MPa$
Partial factor	$\gamma_m := 1.7$
Weight (clay bricks)	$\rho_m := 18\ \dfrac{kN}{m^3}$
Number of brick wythes	$NB := 2$
Brick length	$b_l := 210\ mm$
Brick width	$b_w := 100\ mm$
Brick height plus bed joint thickness (10mm)	$b_h := 80\ mm$

The resisting system as shown in Figure 8-29 is affected by a spandrel which is more slender than piers, hence the failure mechanism of strong pier/weak spandrel has been assumed. Given the failure mechanism, the ultimate lateral load capacity may be derived by equilibrium as shown in Figure 8-30.

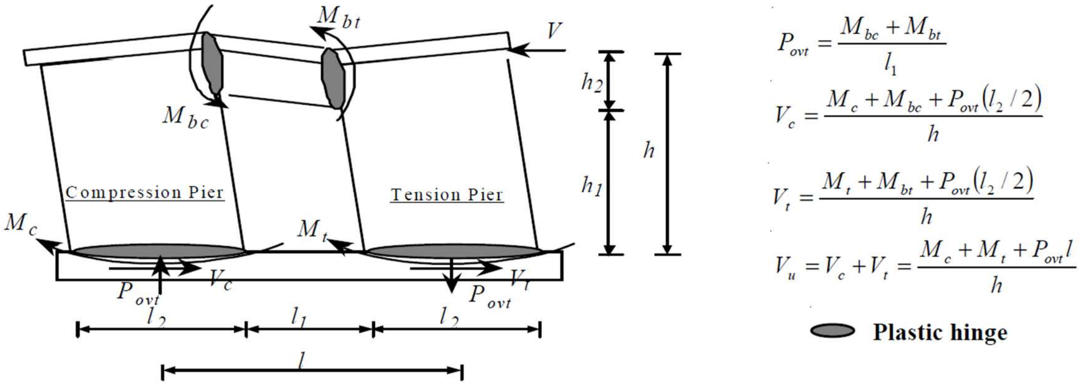


Figure 8-30 Strong Pier/Weak Spandrel Failure Mechanism Elshafie[32]

In order to determine the lateral load capacity of the wall panel, firstly, strengths of element are necessary to be calculated.

Flexural strengths for the pier end sections are determined for both compression and tension pier, which are named M_c and M_t respectively. These values may be derived by means of the method proposed by Eurocode 6 in section 6.3 which refers to the lateral moment of resistance for an unreinforced masonry wall subjected to lateral load.

The lateral moment of resistance of a pier is determined by Eq.[8.4].

$$M_{Rd} = f_{xd} \cdot z \qquad [8.4]$$

Where:

f_{xd} *is* the design flexural strength appropriate to the plane of bending, obtained from Eq.[8.5]
z is the elastic section modulus of the pier

$$f_{xd} = f_{xd.1} + \sigma_d \qquad [8.5]$$

Where:
$f_{xd.1}$ is the design flexural strength of masonry with the plane of failure parallel to the bed joints.
σ_d is the design compressive stress on the wall.

Flexural strengths for end sections of the coupling element (spandrel) are named respect to the connected pier, M_{bc} for the connection with the compression pier and M_{bt} for the connection with the tensile pier. Values for these flexural strengths may be determined by means of the procedure described on the American code Fema 306[16] in section 7.3.4 related to in-plane behaviour of perforated walls with spandrel damage.
The model assumes that the moment capacity of an uncracked spandrel is affected from the interlock between the bed joints and collar joints at the interface between the pier and the spandrel.

The model is based on an elastic stress distribution across the end of the spandrel with its neutral axis at the centre line of the spandrel height as shown in Figure 8-31.

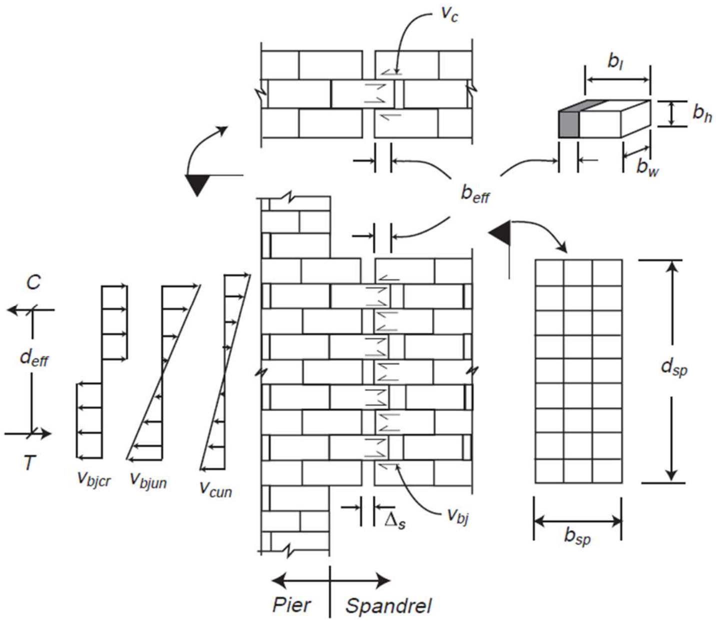


Figure 8-31 Spandrel Joint Sliding Fema 306[16].

The resultant force, both in tension and compression are assumed to be given by a linear superposition of bed joint and collar joint capacities, taking into account the mortar shear strength.

Consequently the uncracked moment capacity is the product of the resultant force and the effective distance between the resultant, given by Eq. [8.6].

$$M_b = T \cdot \frac{2}{3} d_{sp} \qquad [8.6]$$

Where

T is the resultant tensile and compressive forces (T=C) given by Eq. [8.9]
d_{sp} is the depth of the spandrel

- Uncracked bed joint shear stress:

$$v_{bjun} = \frac{0.75\left(0.75 v_{te} + \frac{\gamma P_{CE}}{A_n}\right)}{1.5} \qquad [8.7]$$

Where:

v_{te} is the average value of the masonry bond strength
P_{CE} is the expected vertical axial compressive force per load combination at the adjacent pier

A_n is the area of the net mortared section of the adjacent pier
$\gamma = 0.5$ this value assumes that the vertical axial stress on the spandrel bed joints at the end of the spandrel is half of the axial stress within the pier above the pier/spandrel joint.

- Uncracked collar joint shear stress

$$\nu_{bjun} = 0.375\nu_{te} \quad [8.8]$$

Consequently, the resultant tensile and compressive result force may be derived from Eq.[8.9]

$$T = \left(\nu_{bjun} \cdot b_w \cdot b_{effun}\right) + \left(\nu_{cun} \cdot b_h \cdot b_{effun} \cdot (NB-1)\right)\frac{NR}{2} \quad [8.9]$$

Where

b_w is the width of the brick unit
b_h is the height of the brick unit
NB is the number of brick wythes
$NR = 0.5\left(d_{sp}/b_h\right)$ is the number of rows of bed joints
$b_{effun} = b_l/2$ is the effective length of interface for an uncracked spandrel

Consequently the uncracked collar joint shear stress has been determined.

Effective length of interface for uncracked spandrel	$b_{effun} := 0.5 \cdot b_l = 105\ mm$
Number of rows of bed joints in the spandrel	$NR := 0.5\left(\frac{d_{sp}}{b_h}\right) = 2$
Uncracked collar joint shear stress between spandrel and pier	$\nu_{cun} := 0.375\ \nu_{te} = 0.04\ \frac{N}{mm^2}$

The load combination adopted to define the expected vertical axial compressive force on the piers is the combination suggested by American code Fema 306[16]. This has been done in order to use the procedure for the resistance of the pier-spandrel connection suggested by the beforementioned code.

In order to define the lateral strength of the selected wall, the tensile and compressive resultant force between spandrel and pier has been derived for both spandrel-pier connection. Thus, the uncracked moment of the connections has been calculated.

FIRST STOREY

Pier 1

Area of net mortared section of pier 1	$A_{n.1} = 294000\ mm^2$
Dead load on pier 1	$Q_{D1.1} := \rho_m \cdot t \cdot l_{x.1} \cdot h_f = 46.9\ kN$
Gravity load on pier 1	$Q_{G1.1} := 1.1 \cdot Q_{D1.1} = 51.5\ kN$
Expected vertical axial compressive force on pier 1	$P_{CE1.1} := Q_{G1.1} = 51.5\ kN$

Uncracked bed joint shear stress between spandrel and pier 1:

$$\nu_{bjun1.1} := \frac{0.75}{1.5} \cdot \left(0.75\ \nu_{te} + \frac{0.5 \cdot P_{CE1.1}}{A_{n.1}}\right) = 0.08\ \frac{N}{mm^2}$$

Tensile and compressive resultant force between spandrel and pier 1

$$T_{1.1} := \left(\nu_{bjun1.1} \cdot b_w \cdot b_{effun}\right) + \left(\nu_{cun} \cdot b_h \cdot b_{effun} \cdot (NB - 1)\right) \cdot \frac{NR}{2} = 1.1\ kN$$

Uncracked moment of connection spandrel pier 1

$$M_{bc.1} := T_{1.1} \cdot \frac{2}{3}\ d_{sp} = 0.2\ kN \cdot m$$

Pier 2

Area of net mortared section of pier 2	$A_{n.2} = 191100\ mm^2$
Dead load on pier 2	$Q_{D2.1} := \rho_m \cdot t \cdot l_{x.2} \cdot h_f = 33\ kN$
Gravity load on pier 2	$Q_{G2.1} := 1.1 \cdot Q_{D2.1} = 36.3\ kN$
Expected vertical axial compressive force on pier 2	$P_{CE2.1} := Q_{G2.1} = 36.3\ kN$

Uncracked bed joint shear stress between spandrel and pier 2:

$$\nu_{bjun2.1} := \frac{0.75}{1.5} \cdot \left(0.75\ \nu_{te} + \frac{0.5 \cdot P_{CE2.1}}{A_{n.2}}\right) = 0.08\ \frac{N}{mm^2}$$

Tensile and compressive resultant force between spandrel and pier 1

$$T_{2.1} := \left(\nu_{bjun2.1} \cdot b_w \cdot b_{effun}\right) + \left(\nu_{cun} \cdot b_h \cdot b_{effun} \cdot (NB - 1)\right) \cdot \frac{NR}{2} = 1.2\ kN$$

Uncracked moment of connection spandrel pier 1

$$M_{bt.1} := T_{2.1} \cdot \frac{2}{3}\ d_{sp} = 0.2\ kN \cdot m$$

SECOND STOREY

Pier 1

Area of net mortared section of pier 1 $A_{n.1} := l_1 \cdot t = 294000\ mm^2$

Dead load on pier 1 $Q_{D1.2} := \rho_m \cdot t \cdot l_{x.1} \cdot h_s = 8.9\ kN$

Gravity load on pier 1 $Q_{G1.2} := 1.1 \cdot Q_{D1.2} = 9.8\ kN$

Expected vertical axial compressive force on pier 1 $P_{CE1.2} := Q_{G1.2} = 9.8\ kN$

Uncracked bed joint shear stress between spandrel and pier 1:

$$\nu_{bjun1.2} := \frac{0.75}{1.5} \cdot \left(0.75\ \nu_{te} + \frac{0.5 \cdot P_{CE1.2}}{A_{n.1}}\right) = 0.05\ \frac{N}{mm^2}$$

Tensile and compressive resultant force between spandrel and pier 1

$$T_{1.2} := \left(\nu_{bjun1.2} \cdot b_w \cdot b_{effun}\right) + \left(\nu_{cun} \cdot b_h \cdot b_{effun} \cdot (NB-1)\right) \cdot \frac{NR}{2} = 0.8\ kN$$

Uncracked moment of connection spandrel pier 1

$$M_{bc.2} := T_{1.2} \cdot \frac{2}{3}\ d_{sp} = 0.2\ kN \cdot m$$

Pier 2

Area of net mortared section of pier 2 $A_{n.2} := l_2 \cdot t = 191100\ mm^2$

Dead load on pier 2 $Q_{D2.2} := \rho_m \cdot t \cdot l_{x.2} \cdot h_s = 6.2\ kN$

Gravity load on pier 2 $Q_{G2.2} := 1.1 \cdot Q_{D2.2} = 6.9\ kN$

Expected vertical axial compressive force on pier 2 $P_{CE2.2} := Q_{G2.2} = 6.9\ kN$

Uncracked bed joint shear stress between spandrel and pier 2:

$$\nu_{bjun2.2} := \frac{0.75}{1.5} \cdot \left(0.75\ \nu_{te} + \frac{0.5 \cdot P_{CE2.2}}{A_{n.2}}\right) = 0.05\ \frac{N}{mm^2}$$

Tensile and compressive resultant force between spandrel and pier 1

$$T_{2.2} := \left(\nu_{bjun2.2} \cdot b_w \cdot b_{effun}\right) + \left(\nu_{cun} \cdot b_h \cdot b_{effun} \cdot (NB-1)\right) \cdot \frac{NR}{2} = 0.8\ kN$$

Uncracked moment of connection spandrel pier 1

$$M_{bt.2} := T_{2.2} \cdot \frac{2}{3}\ d_{sp} = 0.2\ kN \cdot m$$

The laterla strength of the selected wall panel can now be evaluated by means of the equilibrium conditions beforementioned.

FIRST STOREY

$$M_{t.1} := \left(\frac{f_{xk.1}}{\gamma_m} + \frac{P_{CE2.1}}{A_{n.2}}\right) \cdot \left(\frac{t \cdot l_2^{\,2}}{6}\right) = 8.9\ kN \cdot m$$

$$M_{c.1} := \left(\frac{f_{xk.1}}{\gamma_m} + \frac{P_{CE1.1}}{A_{n.1}}\right) \cdot \left(\frac{t \cdot l_1^{\,2}}{6}\right) = 20.1\ kN \cdot m$$

$$P_{ovt.1} := \frac{M_{bc.1} + M_{bt.1}}{l_3} = 0.2\ kN$$

$$V_{c.1} := \frac{M_{c.1} + M_{bc.1} + P_{ovt.1} \cdot \left(\frac{l_1}{2}\right)}{h} = 9.7\ kN$$

$$V_{t.1} := \frac{M_{t.1} + M_{bt.1} + P_{ovt.1} \cdot \left(\frac{l_2}{2}\right)}{h} = 4.4\ kN$$

$$V_{u.1} := V_{c.1} + V_{t.1} = 14.1\ kN$$

SECOND STOREY

$$M_{t.2} := \left(\frac{f_{xk.1}}{\gamma_m} + \frac{P_{CE2.2}}{A_{n.2}}\right) \cdot \left(\frac{t \cdot l_2^{\,2}}{6}\right) = 4.5\ kN \cdot m$$

$$M_{c.2} := \left(\frac{f_{xk.1}}{\gamma_m} + \frac{P_{CE1.2}}{A_{n.1}}\right) \cdot \left(\frac{t \cdot l_1^{\,2}}{6}\right) = 10.3\ kN \cdot m$$

$$P_{ovt.2} := \frac{M_{bc.2} + M_{bt.2}}{l_3} = 0.1\ kN$$

$$V_{c.2} := \frac{M_{c.2} + M_{bc.2} + P_{ovt.2} \cdot \left(\frac{l_1}{2}\right)}{h} = 5\ kN$$

$$V_{t.2} := \frac{M_{t.2} + M_{bt.2} + P_{ovt.2} \cdot \left(\frac{l_2}{2}\right)}{h} = 2.2\ kN$$

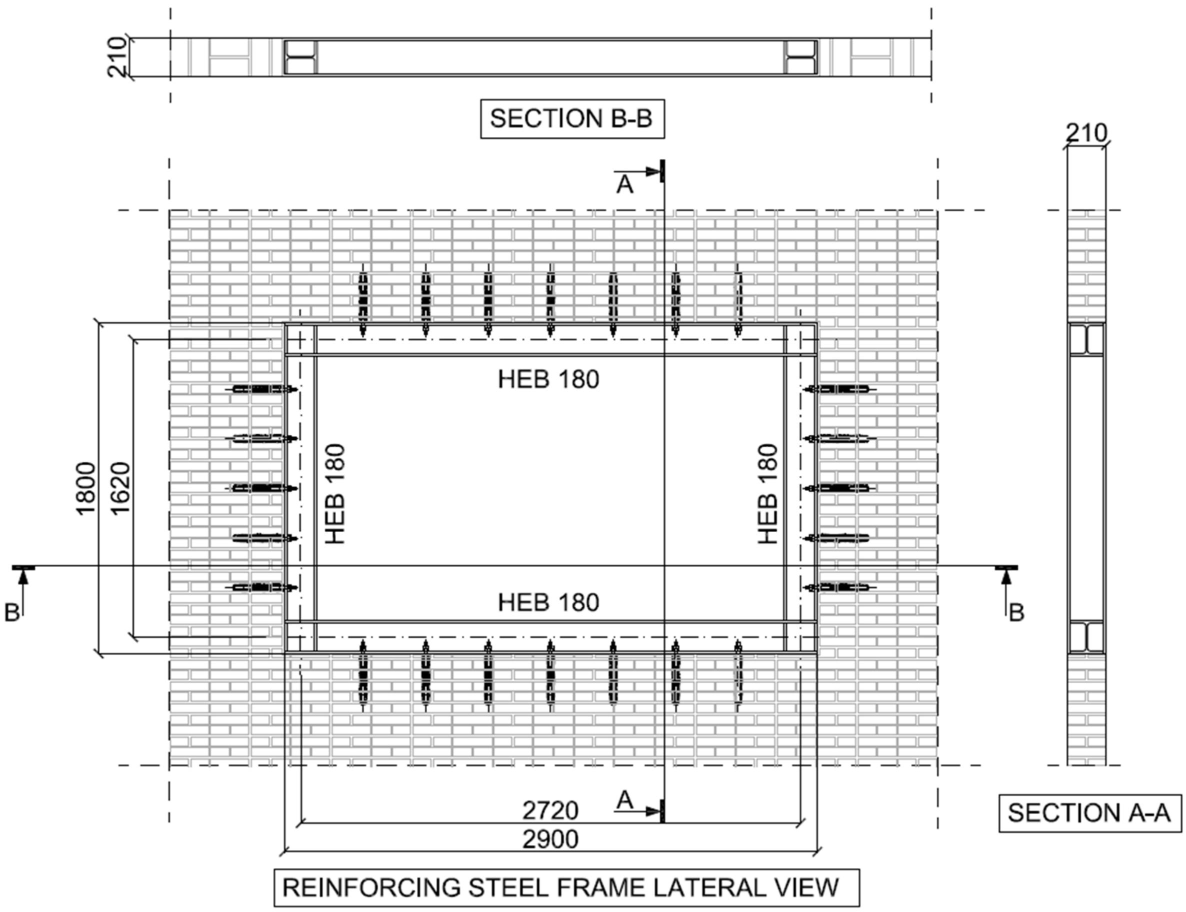
$$V_{u.2} := V_{c.2} + V_{t.2} = 7.3\ kN$$

The lateral resistance of the entire wall of the selected terraced masonry house may now be approximated. This may be obtained by counting 8 wall panels for each storey, and hence by sum their lateral strength.

$V_{u.1.tot} = 8 \cdot 14.1 = 112.8kN$ Approximated ultimate lateral resistance of the first storey.

$V_{u.2.tot} = 8 \cdot 14.1 = 58.4kN$ Approximated ultimate lateral resistance of the second storey

8.12 Sketch of the Solution



9 CONCLUSIONS

Summary of findings and recommendations of this research

This study was focused on the seismic retrofitting of URM structures found in Groningen area in the north of Netherlands. A summary of both the literature review and design examples of this research is presented in this section with highlight on findings. Furthermore, recommendations derived from the finding of the project are also introduced.

9.1 Summary of the Literature Review

Earthquake statistics show that the area surrounding the gas reservoir in Groningen is presently experiencing a rise in both frequency and magnitude of seismic events. Vulnerably tests performed on the basis of the predicted future earthquakes show possible damages for the Groningen buildings stock. Tests testify that the most vulnerable typology result to be URM buildings, which compose the 77% of total constructions in the analysed area.
URM building are particularly prone to seismic damages because of their diverse weak aspects, which are key points of this seismic upgrading. Consequently, based on the vulnerability of URM buildings a retrofitting strategy has been defined to design a damage limitation retrofit.

Although seismic retrofitting of URM buildings is a topic of international interest, differences may be found in different codes. European and Italian codes suggest guidelines and approaches about assumptions and considerations of seismic retrofitting of URM structures. More details are given by the American and New Zealand codes which often show relations and methods to determine loads and resistances.

The first level of retrofitting regarding the upgrading of non-structural elements (such as parapet and chimney) aims to reduce the risk of falling of these elements. Throughout the different solutions listed, bracing of members has been assumed the most technically reliable, given that other solutions would involve radical changing of the aesthetic appearance of the building or complex construction processes.

The second level of the strategy aims to increase the overall robustness of the building by strengthening of connection between horizontal and vertical elements. Diaphragms and walls may be connected by means of different solutions, such as ring beams, wall tie-rod, external circumferential bandage and steel bars. The latter solution which involves the use of stainless steel bars has been evaluated as the most technically reliable for the case of URM building. This because it shows advantages of repeatability, scalability and reduced aesthetic modifications respect to other techniques.

Due to the fact that URM masonry buildings present their masses concentrated at floor levels, floor diaphragms need a certain stiffness to transfer seismic loads, due to the seismic acceleration of these masses. Consequently, the aim of the third level of retrofitting strategy is the strength of horizontal diaphragms, in order to transfer seismic loads to vertical members. Nowadays the most used techniques for this seismic upgrading involve the use of an overlay above the existing floor sheathing. Comparison between timber overlay and concrete topping overlay shown how the concrete solution transforms a flexible diaphragm in a rigid element. Besides it presents advantages in case this level is planned together with the second level of strategy.

The fourth level of seismic strategy aims to strength masonry walls in their so-called weak direction. This because, URM walls in case of seismic events are prone to damages due to loads in their out-of-plane direction. Throughout different retrofitting techniques proposed in the relative chapter, composite material strips applied into vertical cuts have been evaluated as the most technically reliable for the case of URM building in Groningen. The construction process of this solution involves small cuts into the wall which may be translated into a minimal disruption during execution, final acceptable aesthetic appearance of the wall surface and repeatability of the technique.

Lateral loads in URM buildings are primarily resisted by in-plane action of masonry walls oriented in the direction of the loads. Due to the fact that masonry walls affected by numerous openings may show deficiencies to resist seismic loads, the fifth and last level of retrofitting strategy aims to provide enough stiffness to masonry wall respect seismic loads. For this upgrading level a not so common retrofitting technique has been selected. The solution involves the use of steel frame at opening locations which should add stiffness to the wall panel. Respect to other solutions this presents an easier construction process which involves less disruption for the occupants and in some cases minor aesthetic changes of the building.

9.2 Conclusions Related to Design Examples

In order to verify the reliability of the selected solutions, calculation example of techniques have been computed. In order to evaluate the efficiency on the Groningen URM building, a Dutch terraced house has been subjected of upgrading designs.

Calculation example of bracing of parapet show how the solution prevent the falling of the element besides to prevent cracks and damage of the member due to seismic loads.

Designs of level 2 and 3 of the retrofitting strategy proved the decreasing of the period of the building due to the stiffening of the floor diaphragms. Despite steel bars embedded in the masonry walls and in the concrete topping overlay provide mutual transfer of both in-plane and out-of-plane loads, the number of steel bars is determined by the shear load due to the mass of the diaphragm.

Design example of retrofitting of URM wall in their out-of-plane direction by means of CFRP NSM demonstrated the efficacy of this retrofitting solution. The selected wall which in the initial conditions would not resist the out-of-plane loads originated by the earthquake, after the upgrading intervention had doubled its bending moment resistance. Consequently it has been verified to be able to resist the seismic design loads.

Stiffening of masonry wall panels by means of reinforcing rings at opening locations indicated a remarkable increase of stiffness. The initial stiffness resulted to be incremented of about the 45% in the upgraded situation. As expected the increase of stiffness signifies a reduction of the displacement due to the lateral load of about the 30%.

9.3 Recommendation for Future Research

Based on the findings of this research the following recommendations are made:

- For the calculation of the initial stiffness of the diaphragm before to be retrofitted the shear strength of the connections should be neglected due to the fact that it relies on the friction between joists and masonry wall.
- Further tests are needed in order to assess an empirical formula to determine the intermediate crack debonding capacity of CFRP NSM, since presently the used relations is derived from test made on concrete specimens.
- Retrofitting technique which involves the place of steel frames at opening locations should be further exanimated in order to understand the behaviour of the solution after cracking of the masonry.

ANNEX A

Load Combinations used in the Calculation Examples

In Europe seismic retrofitting shall be designed taking into account actions defined by combinations listed on Eurocode 8.

- *Eurocode 8 combination of the seismic action with other actions:*

Inertial effects of the design seismic action shall be determined by accounting the presence of the masses associated with all gravity loads present in Eq.[10.1].

$$\Sigma G_{k,j} + \Sigma \psi_{E,i} \cdot Q_{k,i} \qquad [10.1]$$

Where:

$G_{k,j}$ are the dead loads

$Q_{k,i} = 2.0 \, kN/m^2$ are the variable load

$\psi_{E,i} = \varphi \psi_{2i}$ is combination coefficient for variable action *i*

In case of domestic and residential area, named category A on the Eurocode, the following values shall be assumed:

$Q_{k,i} = 2.0 \, kN/m^2$

$\varphi = 1$ for roof

$\psi_2 = 0.3$ for residential category

Hence, Eq.[10.1] may be rewritten as:

$$\Sigma G_{k,j} + 0.48 \, kN/m^2 \qquad [10.2]$$

In America, seismic designs are regulated by the code Fema 273[33].

- *Fema 273 combination of the seismic action with other actions:*

Load combination for seismic loads shall consider the component of gravity load defined by Eq.[10.3]

$$Q_G = 1.1(Q_D + Q_L) \qquad [10.3]$$

Where:

Q_D is the dead load effect

Q_L is the effective live load effect, equal to the 25% of the unreduced design live load

ANNEX B

Deriving S_s and S_1 Parameters from PGA Maps

Seismic hazard in the Groningen area has been defined by a probabilistic seismic hazard analysis. A map was developed for the level of peak ground acceleration approximately equivalent to the design basis earthquake ground motion in Eurocode 8 which corresponds to a return period of 475 years.

American codes for seismic design account the seismic hazard by means of coefficients for a return period of 2475 years.

Consequently a conversion was needed in order to adopt procedures mentioned in American codes.

Lubkowski[14] elaborated a methodology to convert 475 years PGA values to 2475 years values based on the level of seismicity. This has been possible by means of more than fifty probabilistic seismic hazard studies carried out by Arup low, moderate and high seismicity regions around the world.

Converting equations for Ss and S_1 are represented by Eq.[10.4] and Eq.[10.5] respectively.

$$\frac{S_S}{PGA} = 0.3386PGA + 2.1696 \qquad [10.4]$$

$$\frac{S_1}{PGA} = 0.5776PGA + 0.5967 \qquad [10.5]$$

ANNEX C

Diaphragm shear transfer in case of cavity wall

The following calculation example aims to design connection between diaphragm and cavity walls. In the first place forces assumed to be transferred from diaphragms to shear walls, due to the ground motion, are determined. Then, anchors to restrain walls in their out-of-plane direction are designed.

The analysis has been carried out in accordance with the method suggested by the American code ASCE 41-13 [7]. Hence, the Peak Ground Acceleration (a_g) has been converted in the Short and Long Period Spectral Acceleration. The conversion has been achieved by means of converting relations elaborated by Lubkowsky [14] and shown in Annex A.

Calculation data have been taken with reference to a Dutch terraced house already presented in the report. Diaphragms have been assumed to be a standard timber floors one single sheathing supported by timber joists. In Figure 9-1 the two analysed diaphragms are displayed.

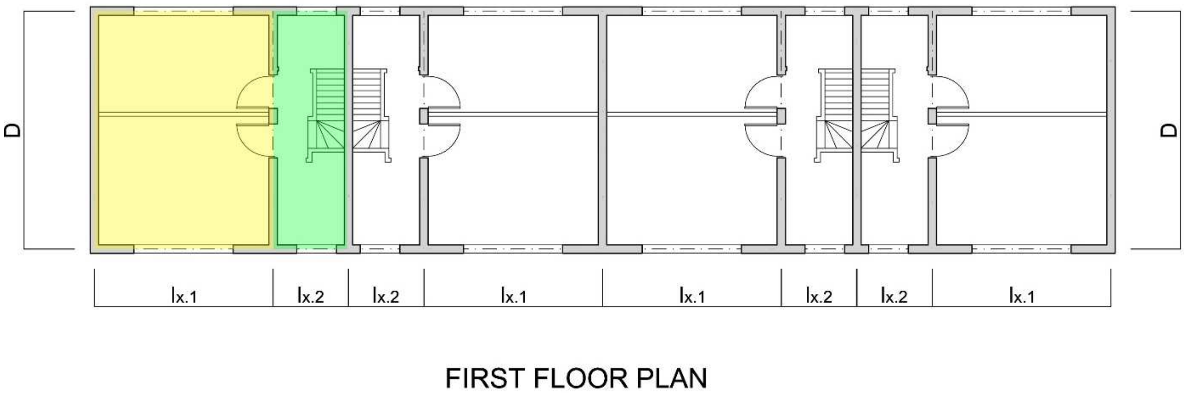


Figure 9-1 Examined Diaphragms in the Calculation Example

Diaphragm gravity load (see Annex A) $\quad q_{G.d} := 1.1 \cdot \left(165 \frac{kg}{m^2} + 0.25 \cdot 200 \frac{kg}{m^2}\right) = 236.5 \frac{kg}{m^2}$

Dead load tributary to diaphragm 1 $\quad W_{d.1} := (q_{G.d} \cdot D \cdot l_{x.1}) + (2 \cdot 1.1 \cdot \rho_m \cdot l_{x.1} \cdot h \cdot (t_o + t_i)) = 17078\ kg$

Dead load tributary to diaphragm 2 $\quad W_{d.2} := (q_{G.d} \cdot D \cdot l_{x.2}) + (2 \cdot 1.1\ \rho_m \cdot l_{x.2} \cdot h \cdot (t_o + t_i)) = 7211\ kg$

Unit shear capacity value for the diaphragm taken from Table A1-D IEBC 2006 $\quad \nu_u := 21891 \frac{N}{m}$

Horizontal Force Factor Cp taken from Table 15-3 ASCE 41-13 $\quad C_p := 0.5$

Peak ground acceleration (PGA) $a_g := 0.243$ [g]
Short period spectral acceleration $S_s := (0.3386 \cdot a_g + 2.1696) \cdot a_g$
Long period spectral acceleration $S_1 := (0.5776 \cdot a_g + 0.5967) \cdot a_g$

$$S_s := 0.5472\ g$$

$$S_1 := 0.1791\ g$$

Considering a Site Class E for the Groningen area, values for Fa and Fv can be determined by Table 2-3 and Table 2-4 respectively on ASCE 41-13.

$F_a := 1.6$ $\quad S_{XS} := F_a \cdot S_s = 8.6\ \frac{m}{s^2}$

$F_v := 3.0$ $\quad S_{X1} := F_v \cdot S_1 = 5.3\ \frac{m}{s^2}$

Storey height $h := 3000\ mm$

WALL

Wall outer leaf thickness $t_o := 100\ mm$
Cavity wall thickness $t_c := 60\ mm$
Wall inner leaf thickness $t_i := 100\ mm$

Masonry weight $\rho_m := 1800\ \frac{kg}{m^3}$

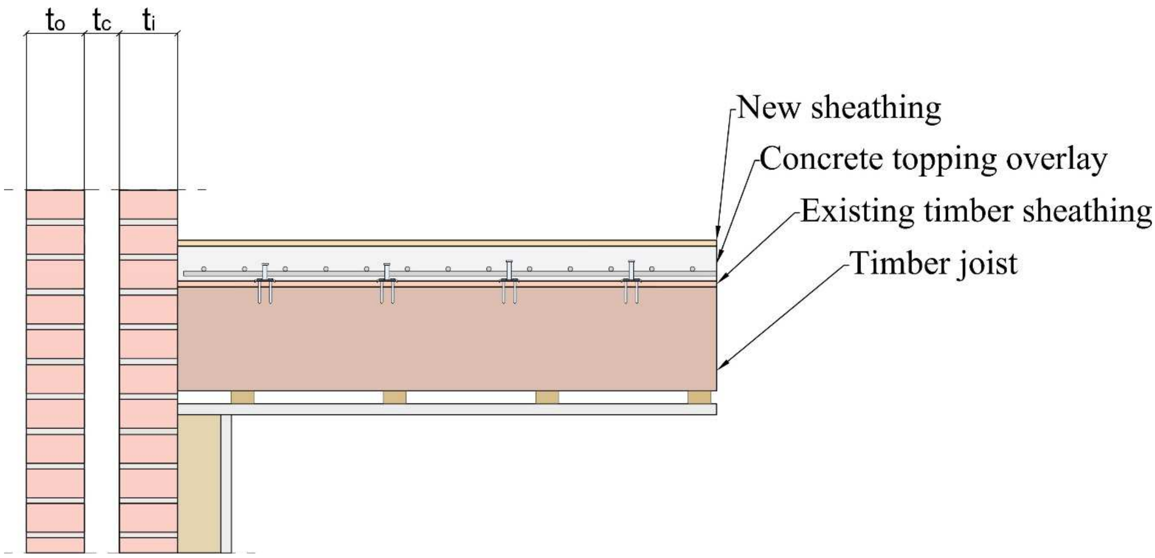


Figure 9-2 Detail of Cavity Wall and Timber Diaphragm

Storey force distributed to shear walls in the in-plane direction

Horizontal loads transferred from the diaphragm to shear walls in their in-plane direction are determined by the minimum of the following values:

Diaphragm 1 (yellow area)	$V_{d;1} := 1.25\ S_{X1} \cdot C_p \cdot W_{d.1} = 56.2\ kN$ $V_{d;1} := \nu_u \cdot D = 131.3\ kN$
Diaphragm 2 (green area)	$V_{d;2} := 1.25\ S_{X1} \cdot C_p \cdot W_{d.2} = 23.7\ kN$ $V_{d;2} := \nu_u \cdot D = 131.3\ kN$

The wall panel is assumed to be anchored at diaphragm level. Figure 9-3 displays the panel involved in the analysis highlighted by the colour red with indicated its sizes.

Anchorage of wall to the diaphragm in the out-of-plane direction

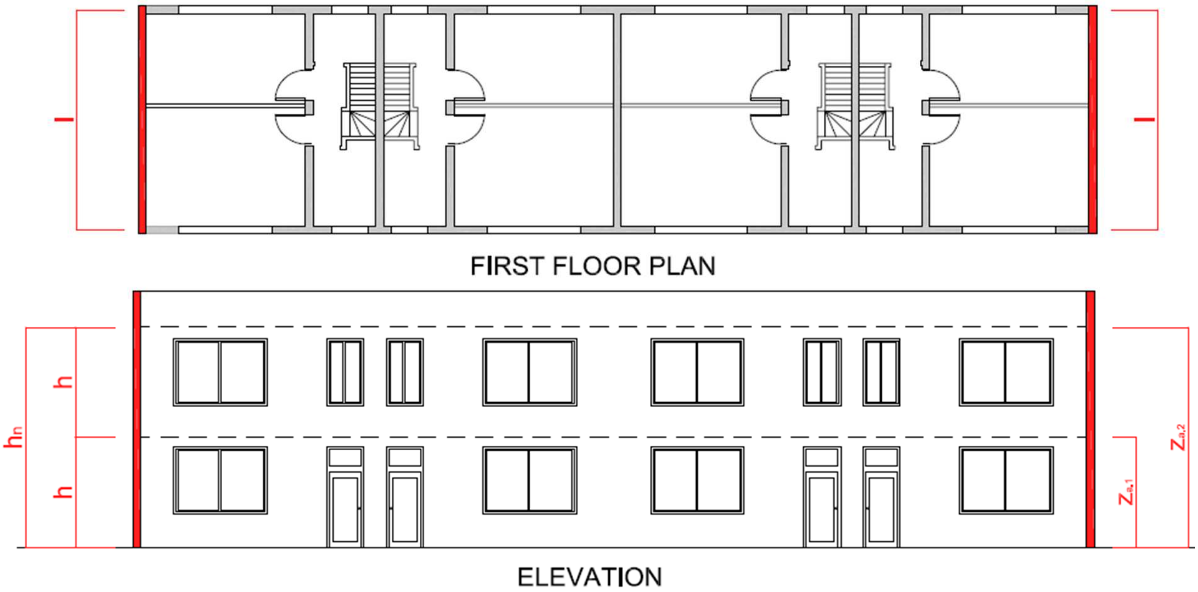


Figure 9-3 Wall Panels to Retrofit

Wall length	$l := 6000\ mm$
Storey height	$h = 3000\ mm$
Factor to account diaphragm flexibility (assumed rigid diaphragm)	$k_a := 1$
Height of the wall anchors above the base of the structure	$z_{a.1} := 3000\ mm$ $z_{a.2} := 6000\ mm$
Height above the base to the roof level	$h_n := 6000\ mm$
Factor for calculation out-of-plane wall forces (from Table 7-2 ASCE 41-13)	$\chi := 2$
Factor to account for variation in force over the eight of the building	$k_{h.1} := \frac{1}{3}\left(1 + 2 \cdot \frac{z_{a.1}}{h_n}\right) = 0.7$ $k_{h.2} := \frac{1}{3}\left(1 + 2 \cdot \frac{z_{a.2}}{h_n}\right) = 1$

At both diaphragm levels, a number of anchors has been assumed. The assumption is necessary in order to evaluate the size of the portion of wall which each anchor needs to restrain.

Figure 9-4 shows the assumed locations of anchors and their spacing.

Number of anchors between wall and diaphragm	To the lower diaphragm	$n_1 := 10$
	To the upper diaphragm	$n_2 := 10$
Distance between consecutive anchors in the relative direction		$s_1 := \dfrac{l}{n_1 + 1} = 545\ mm$
		$s_2 := \dfrac{l}{n_2 + 1} = 545\ mm$

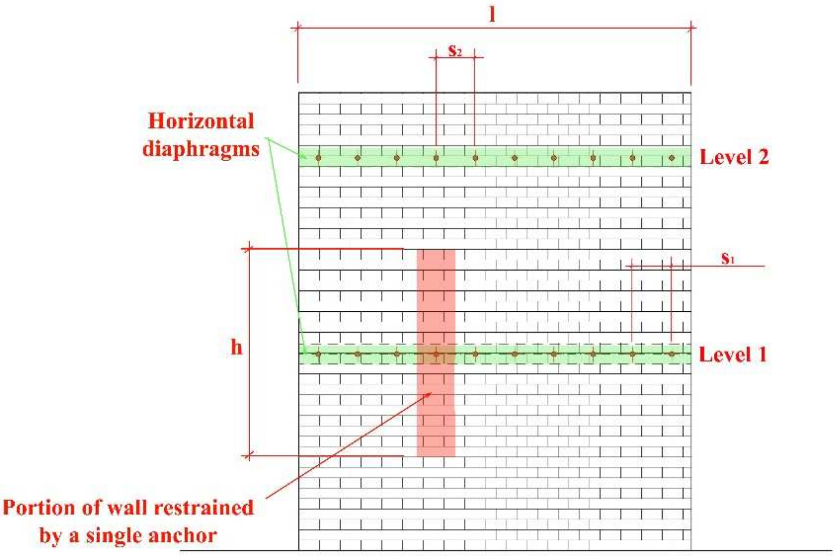


Figure 9-4 Anchor Locations

Horizontal load due to the portion of wall relative to the anchor is determined by the vale Fp, but not less than Fp,min.. The value represents the axial load which every anchor needs to resist.

Load acting on anchors at level 1

$$F_{p.o.1} := 0.4\ S_{XS} \cdot k_a \cdot k_{h.1} \cdot \chi \cdot W_{p.o.1} = 1.3\ kN$$
$$F_{p.i.1} := 0.4\ S_{XS} \cdot k_a \cdot k_{h.1} \cdot \chi \cdot W_{p.i.1} = 1.3\ kN$$
$$F_{p.o.1.min} := 0.2\ S_{XS} \cdot k_a \cdot \chi \cdot W_{p.o.1} = 1\ kN$$
$$F_{p.i.1.min} := 0.2\ S_{XS} \cdot k_a \cdot \chi \cdot W_{p.i.1} = 1\ kN$$

Load acting on anchors at level 2

$$F_{p.o.2} := 0.4\ S_{XS} \cdot k_a \cdot k_{h.2} \cdot \chi \cdot W_{p.o.2} = 2\ kN$$
$$F_{p.i.2} := 0.4\ S_{XS} \cdot k_a \cdot k_{h.2} \cdot \chi \cdot W_{p.i.2} = 2\ kN$$
$$F_{p.o.2.min} := 0.2\ S_{XS} \cdot k_a \cdot \chi \cdot W_{p.o.2} = 1\ kN$$
$$F_{p.i.2.min} := 0.2\ S_{XS} \cdot k_a \cdot \chi \cdot W_{p.i.2} = 1\ kN$$

Values of shear and tensile load which anchors are assumed to resist necessity to be compared with the shear and tensile resistance of the designed connectors. The next calculation example shows these capacities related to connection of cavity walls.

Calculation example for shear connectors between cavity walls and diaphragms

CONCRETE TOPPING OVERLAY C30/37

Thickness	$t := 60\ mm$
Compressive strength	$f_{ck} := 30\ \frac{N}{mm^2}$
Tensile strength	$f_{ctk.0.05} := 2.0\ \frac{N}{mm^2}$
Partial factor	$\gamma_c := 1.5$
Anchorage length	$l_c := 500\ mm$

MASONRY

Compressive strength	$f_k := 4.4\ \frac{N}{mm^2}$
Partial safety factor	$\gamma_m := 1.7$
Anchorage length outer leaf	$l_{m.o} := 60\ mm$
Anchorage length inner leaf	$l_{m.i} := 100\ mm$
Anchorage partial safety factor	$\gamma_M := 2.2$

STAINLESS STEEL ROD
UNS S32304 - ISO 4362-323-04-I

Proof strength	$f_{p.0.2} := 400\ MPa$
Partial safety factor	$\gamma_s := 1.15$
Diameter	$d := 16\ mm$
Yield moment	$M_{y.d} := \frac{f_{p.0.2}}{\gamma_s} \cdot \pi \cdot \frac{d^3}{32} = 139.9\ N \cdot m$
Characteristic anchorage strength between mortar M20 and concrete 30/37 (Table 3.6 Eurocode 6)	$f_{bok} := 3.4\ MPa$

Shear load bearing capacity of a single steel bar connection

The shear load due to the acceleration of diaphragms is assumed to be totally resisted by the inner leaf of the cavity wall. Hence the maximum shear load transferred by each anchor is determined on the basis of only the thickness of the inner leaf.

$$F_{v.Rd} = \min \begin{cases} l_{m.i} \cdot d \cdot \dfrac{f_k}{\gamma_m} = 4.1\ kN \\ l_c \cdot d \cdot \dfrac{f_{ck}}{\gamma_c} = 160\ kN \\ \dfrac{f_k}{\gamma_m} \cdot l_{m.i} \cdot d \cdot \left(\sqrt{2 + \dfrac{4\ M_{y.d}}{\left(d \cdot \dfrac{f_k}{\gamma_m} \cdot l_{m.i}^{\ 2} \right)}} - 1 \right) = 3.4\ kN \\ \dfrac{\pi \cdot \dfrac{d^2}{4} \cdot \dfrac{f_{p.0.2}}{\sqrt{3}}}{\gamma_s} = 40.38\ kN \end{cases}$$

Bar shear capacity	$F_{v.Rd} := 3.4\ kN$	
Storey seismic load diaphragm 1 (yellow area)	$V_{d;1} := 56.2\ kN$	
Storey seismic load diaphragm 2 (green area)	$V_{d;2} := 23.7\ kN$	
Number of shear connectors necessary for diaphragm 1	$n_1 := \dfrac{V_{d;1}}{F_{v.Rd}} = 16.53$	$n_1 := 17$
Number of shear connectors necessary for diaphragm 2	$n_2 := \dfrac{V_{d;2}}{F_{v.Rd}} = 6.97$	$n_2 := 7$

Tensile load bearing capacity of a single steel bar connection

The tensile bearing capacity of the rebar shall be determined with relation of the anchorage length in both outer and inner leaf and in the concrete overlay. It is assumed that the resistance due to the bond of the bar in the outer leaf ($F_{p.d.m.o}$) shall be able to resist the out-of-plane load generated by the vibration of the outer leaf portion ($F_{p.o}$). Similarly, for the inner leaf the resistance $F_{p.d.m.i}$ should be greater than the load $F_{p.i.}$

The assessment of the tensile resistance due to the bond between bar and mortar has been carried out by means of the procedure related to the reinforcement for reinforced masonry suggested by Eurocode 6 in Section 8.2.5

Anchorage length in the masonry	$l_{m.o}=60\ mm$ $l_{m.i}=100\ mm$
Ultimate bond stress for high-bond stainless steel bars	$f_{bod}:=\frac{f_{bok}}{\gamma_M}=1.55\ \frac{N}{mm^2}$
Design stress of the bar at the position in the masonry from where the anchorage is measured from	$\sigma_{sd.m.o}:=l_{m.o}\cdot 4\cdot\frac{f_{bod}}{d}=23.18\ \frac{N}{mm^2}$
	$\sigma_{sd.m.i}:=l_{m.i}\cdot 4\cdot\frac{f_{bod}}{d}=38.64\ \frac{N}{mm^2}$
Tensile load bearig capacity of a single bar given by the bond in the masonry	$F_{p.d.m.o}:=\sigma_{sd.m.o}\cdot\frac{d^2}{4}\cdot\pi=4.7\ kN$
	$F_{p.d.m.i}:=\sigma_{sd.m.i}\cdot\frac{d^2}{4}\cdot\pi=7.8\ kN$

Successively, the tensile resistance of the anchor due to the bond between bar and concrete overlay has been determined by means of anchorage length of reinforced concrete described in Eurocode 2 Section 8.4

Coefficients related to the quality of the bond condition	$\eta_1:=0.7$ $\eta_2:=1.0$
Coefficient accounting of long term effects	$\alpha_{cc}:=1.0$
Design tensile strength	$f_{ctd}:=\alpha_{cc}\cdot\frac{f_{ctk.0.05}}{\gamma_c}=1.33\ \frac{N}{mm^2}$
Ultimate bond stress for ribbed bars	$f_{bd}:=2.25\cdot\eta_1\cdot\eta_2\cdot f_{ctd}=2.1\ \frac{N}{mm^2}$
Anchorage length in the concrete	$l_c=500\ mm$
Design stress of the bar at the position in the concrete from where the anchorage is measured from	$\sigma_{sd.c}:=l_c\cdot 4\cdot\frac{f_{bd}}{d}=350\ \frac{N}{mm^2}$
Tensile load bearig capacity of a single bar given by the bond in the concrete	$F_{p.d.c}:=\sigma_{sd.c}\cdot\frac{d^2}{4}\cdot\pi=39.6\ kN$

Figure 9-5 shows the forces developed by the bonding in the two leafs

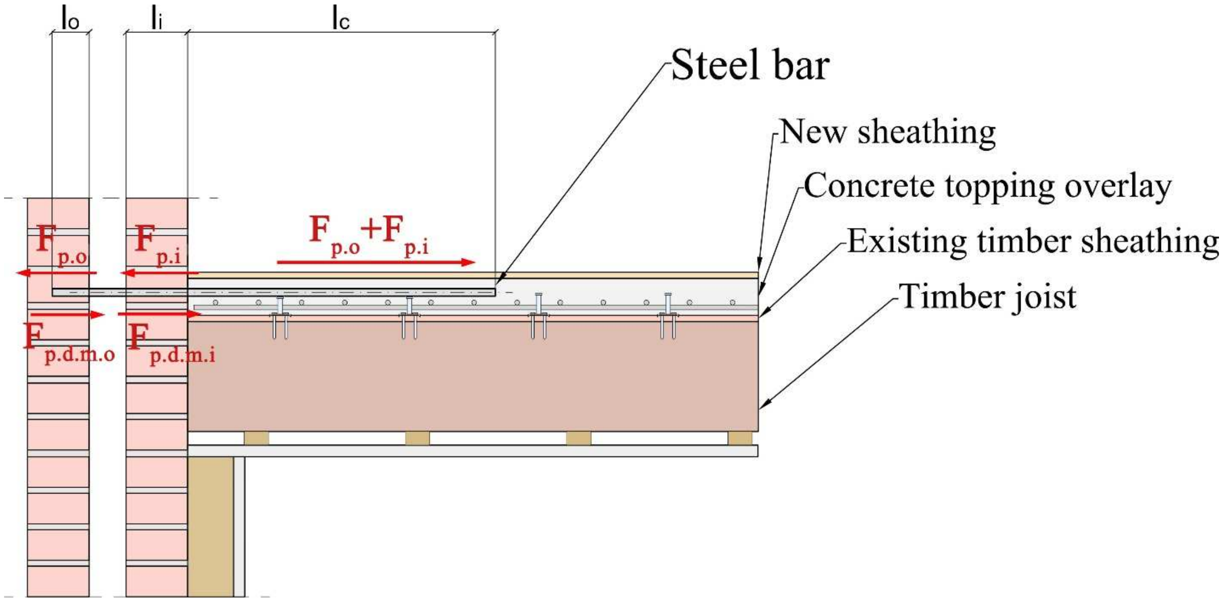


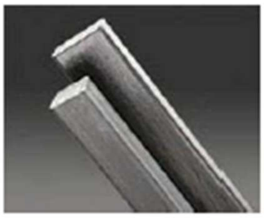
Figure 9-5 Cavity Wall Anchor Forces

ANNEX D
Carbon Fibre Rectangular Strip Manufacturer Details

ACP Composites

1-800-811-2009
www.acpsales.com

.125" x .500" Carbon Fiber Rectangular Strip



Carbon Fiber Rectangular Strips are manufactured through a process referred to as pultursion. Continuous fibers combined with a resin matrix are pulled through a heated steel forming die. As the carbon fibers are saturated with the resin mixture and then pulled through a rectangular die, the hardening of the resin is initiated by the heat from the die and a rigid, cured structure is formed in the shape and size of the die. The majority of the fibers are running in the 0 degree direction, along the length of the strip, to produce an extremely stiff and lightweight with incredible linear strength.

Physical Properties		
Width	.500" +/- .005"	Test Method-Caliper
Thickness	.125" +/- .005"	Test Method-Caliper
Straightness	.062" deviation from straight over 48" span in both X & Y direction	For reference only
Color	Natural dark gray to black	No color match
Surface Finish	Small scratches, surface defects, or blemishes may be apparent.	Mimimum-Visual
Composite Type	0° unidirectional orientation	For reference only
Resin Type	Premium grade bisphenol epoxy vinyl ester	For reference only
Fiber Type	33 to 35 MSI standard modulus carbon fiber	For reference only
Fiber Volume	60%	+/- 5%
Cuts	Rough abrasive cut both ends, small burrs may be apparent.	Mimimum-Visual

Technical Properties	
Tensile Strength	250 ksi / 1.72 GPa
Tensile Modulus	20.0 msi / 138 GPa
Ultimate Shear Strength	6.0 ksi / 41.3 Mpa
Ultimate Tensile Strain	1.50%
Flexural Strength	265 ksi / 1.83 GPa
Flexural Modulus	19.0 msi / 131 GPa
CTE	-0.1 ppm/cm3 / -0.2 ppm/°C
Thermal Properties	150°F maximum
Glass Transition Temp.	100° C
Density	.054 lbs/in3 / 1.5 g/cm3

Sample data is measured from a .156" diameter solid rod with standard modulus fibers and Bisphenol Epoxy Vinyl Ester

All the information contained in these properties is believed to be reliable. It is intended for comparison purposes only as each manufactured lot will exhibit variations. The user should evaluate the suitability of each product for their application. We cannot anticipate the variations in all end use and we make no warranties and assume no liability in connection with the use of this information.

ACP Composites
ISO 9001:2008 Certified I No. 49881

78 Lindbergh Ave I Livermore, CA 94551
1-800-811-2009 I P. 925-443-5900 I F. 925-443-5901
Revision: C I December, 2014 I Page: 1 of 1

ANNEX E
Calculation of wall stiffness in the initial and in the retrofitted condition

Pier Type	Pier number	Geometry				Parameters				Element stiffness		Stiffness of coupled elements	Eq. stiffness
		d [mm]	h_{b2} [mm]	h2 [mm]	h_{t2} [mm]	r	s	q	p	K/Et	k [kN/m]	k [kN/m]	$k_{1.2}$ [kN/m]
2	38	910	900	1800	300		2.308	0.989	8.00	0.093	49037		
4	39	2900	900	1800	300			0.621	4.00	0.355	186265		
2	40	1400	900	1800	300		1.500	0.643	8.00	0.247	129504		
4	41	1000	900	1800	300			1.800	4.00	0.035	18275		
2	42	600	900	1800	300		3.500	1.500	8.00	0.032	16667		
4	43	1000	900	1800	300			1.800	4.00	0.035	18275		
2	44	1400	900	1800	300		1.500	0.643	8.00	0.247	129504		
4	45	2900	900	1800	300			0.621	4.00	0.355	186265		
2	46	1590	900	1800	300		1.321	0.566	8.00	0.318	166721	1634303	
4	47	2900	900	1800	300			0.621	4.00	0.355	186265		
2	48	1400	900	1800	300		1.500	0.643	8.00	0.247	129504		
4	49	1000	900	1800	300			1.800	4.00	0.035	18275		
2	50	600	900	1800	300		3.500	1.500	8.00	0.032	16667		
4	51	1000	900	1800	300			1.800	4.00	0.035	18275		
2	52	1400	900	1800	300		1.500	0.643	8.00	0.247	129504		
4	53	2900	900	1800	300			0.621	4.00	0.355	186265		
2	54	910	900	1800	300		2.308	0.989	8.00	0.093	49037		
1	55	910	900	1800	300	0.989	0.330	1.978	3.50	0.03	15899		
1	56	1400	900	1800	300	0.643	0.214	1.286	3.50	0.089	46477		
1	57	600	900	1800	300	1.500	0.500	3.000	3.50	0.01	5072		
1	58	1400	900	1800	300	0.643	0.214	1.286	3.50	0.089	46477		
1	59	1590	900	1800	300	0.566	0.189	1.132	3.50	0.118	61952	289803	233489
1	60	1400	900	1800	300	0.643	0.214	1.286	3.50	0.089	46477		
1	61	600	900	1800	300	1.500	0.500	3.000	3.50	0.01	5072		
1	62	1400	900	1800	300	0.643	0.214	1.286	3.50	0.089	46477		
1	63	910	900	1800	300	0.989	0.330	1.978	3.50	0.03	15899		
3	64	910	900	1800	300	2.967		0.330	28.00	0.502	263523		
4	65	2900	900	1800	300			0.621	4.00	0.355	186265		
3	66	1400	900	1800	300	1.929		0.214	28.00	1.089	571667		
4	67	1000	900	1800	300			1.800	4.00	0.035	18275		
3	68	600	900	1800	300	4.500		0.500	28.00	0.2	105000		
4	69	1000	900	1800	300			1.800	4.00	0.035	18275		
3	70	1400	900	1800	300	1.929		0.214	28.00	1.089	571667		
4	71	2900	900	1800	300			0.621	4.00	0.355	186265		
3	72	1590	900	1800	300	1.698		0.189	28.00	1.326	696183	4538055	
4	73	2900	900	1800	300			0.621	4.00	0.355	186265		
3	74	1400	900	1800	300	1.929		0.214	28.00	1.089	571667		
4	75	1000	900	1800	300			1.8	4	0.035	18274.9		
3	76	600	900	1800	300	4.5		0.5	28	0.2	105000		
4	77	1000	900	1800	300			1.800	4.00	0.035	18275		
3	78	1400	900	1800	300	1.93		0.214	28.00	1.089	571667		
4	79	2900	900	1800	300			0.621	4.00	0.355	186265		
3	80	910	900	1800	300	2.97		0.330	28.00	0.502	263523		

Table 9-1 Calculation of Stiffness of the Second Storey Wall in the Initial Situation

Pier type	Pier number	Geometry				Parameters				Element stiffness		Stiffness of coupled elements	Eq. Stiffness
		d [mm]	h_{b1} [mm]	h_1 [mm]	h_{t1} [mm]	r	s	q	p	K/Et	k [kN/m]	k [kN/m]	$k_{1.2}$ [kN/m]
2	1	910	900	1800	300		2.308	0.989	8	0.0934	49037		
4	2	2900	900	1800	300			0.621	4.00	0.3548	186265	364806	
2	3	1400	900	1800	300		1.500	0.643	8.00	0.2467	129504		
2	4	600	2700	0	300		0.500	4.500	1.33	0.0074	3889	3889	
2	5	1400	900	1800	300		1.500	0.643	8.00	0.2467	129504		
4	6	2900	900	1800	300			0.621	4.00	0.3548	186265		
2	7	1590	900	1800	300		1.321	0.566	8.00	0.3176	166721	798258	
4	8	2900	900	1800	300			0.621	4.00	0.3548	186265		
2	9	1400	900	1800	300		1.500	0.643	8.00	0.2467	129504		
2	10	600	2700	0	300		0.500	4.500	1.33	0.0074	3889	3889	
2	11	1400	900	1800	300		1.500	0.643	8.00	0.2467	129504		
4	12	2900	900	1800	300			0.621	4.00	0.3548	186265	364806	
2	13	910	900	1800	300		2.308	0.989	8.00	0.0934	49037		
1	14	910	900	1800	300	0.99	0.330	1.978	3.50	0.0303	15899		
Reinforcing ring											45415	107791	
1	15	1400	900	1800	300	0.643	0.214	1.286	3.50	0.0885	46477		
1	16	1400	900	1800	300	0.643	0.214	1.286	3.50	0.0885	46477		
Reinforcing ring											45415		
1	17	1590	900	1800	300	0.566	0.189	1.132	3.50	0.1180	61952	245736	
Reinforcing ring											45415		
1	18	1400	900	1800	300	0.643	0.214	1.286	3.50	0.0885	46477		335328
1	19	1400	900	1800	300	0.643	0.214	1.286	3.50	0.0885	46477		
Reinforcing ring											45415	107791	
1	20	910	900	1800	300	0.989	0.330	1.978	3.50	0.0303	15899		
3	21	910	900	1800	300	2.967		0.330	28.00	0.5019	263523		
4	22	2900	900	1800	300			0.621	4.00	0.3548	186265		
3	23	1400	900	1800	300	1.929		0.214	28.00	1.0889	571667		
4	24	1000	900	1800	300			1.800	4.00	0.0348	18275		
3	25	600	900	1800	300	4.500		0.500	28.00	0.2000	105000		
4	26	1000	900	1800	300			1.800	4.00	0.0348	18275		
3	27	1400	900	1800	300	1.929		0.214	28.00	1.0889	571667		
4	28	2900	900	1800	300			0.621	4.00	0.3548	186265		
3	29	1590	900	1800	300	1.698		0.189	28.00	1.3261	696183	4538055	
4	30	2900	900	1800	300			0.621	4.00	0.3548	186265		
3	31	1400	900	1800	300	1.929		0.214	28.00	1.0889	571667		
4	32	1000	900	1800	300			1.800	4.00	0.0348	18275		
3	33	600	900	1800	300	4.500		0.500	28.00	0.2000	105000		
4	34	1000	900	1800	300			1.800	4.00	0.0348	18275		
3	35	1400	900	1800	300	1.93		0.21	28	1.0889	571667		
4	36	2900	900	1800	300			0.62	4	0.3548	186265		
3	37	910	900	1800	300	2.97		0.330	28.00	0.5019	263523		

Table 9-2 Calculation of Stiffness of the First Storey Wall in the Retrofitted Situation

Pier type	Pier number	Geometry				Parameters				Element Stiffness		Stiffness of coupled elements	Eq. Stiffness
		d [mm]	h_{b2} [mm]	h_2 [mm]	h_{t2} [mm]	r	s	q	p	K/Et	k [kN/m]	k [kN/m]	$k_{2.2}$ [kN/m]
2	38	910	900	1800	300		2.31	0.99	8	0.093	49037		
4	39	2900	900	1800	300			0.62	4	0.355	186265		
2	40	1400	900	1800	300		1.5	0.64	8	0.247	129504		
4	41	1000	900	1800	300			1.8	4	0.035	18275		
2	42	600	900	1800	300		3.5	1.5	8	0.032	16667		
4	43	1000	900	1800	300			1.8	4	0.035	18275		
2	44	1400	900	1800	300		1.5	0.64	8	0.247	129504		
4	45	2900	900	1800	300			0.62	4	0.355	186265		
2	46	1590	900	1800	300		1.32	0.57	8	0.318	166721	1634303	
4	47	2900	900	1800	300			0.62	4	0.355	186265		
2	48	1400	900	1800	300		1.5	0.64	8	0.247	129504		
4	49	1000	900	1800	300			1.8	4	0.035	18275		
2	50	600	900	1800	300		3.5	1.5	8	0.032	16667		
4	51	1000	900	1800	300			1.8	4	0.035	18275		
2	52	1400	900	1800	300		1.5	0.64	8	0.247	129504		
4	53	2900	900	1800	300			0.62	4	0.355	186265		
2	54	910	900	1800	300		2.31	0.99	8	0.093	49037		
1	55	910	900	1800	300	0.99	0.33	1.98	3.5	0.03	15899		
Reinforcing ring											45415		
1	56	1400	900	1800	300	0.64	0.21	1.29	3.5	0.089	46477		
1	57	600	900	1800	300	1.5	0.5	3	3.5	0.01	5072		
1	58	1400	900	1800	300	0.64	0.21	1.29	3.5	0.089	46477		
Reinforcing ring											45415		
1	59	1590	900	1800	300	0.57	0.19	1.13	3.5	0.118	61952	471462	338604
Reinforcing ring											45415		
1	60	1400	900	1800	300	0.64	0.21	1.29	3.5	0.089	46477		
1	61	600	900	1800	300	1.5	0.5	3	3.5	0.01	5072		
1	62	1400	900	1800	300	0.64	0.21	1.29	3.5	0.089	46477		
Reinforcing ring											45415		
1	63	910	900	1800	300	0.99	0.33	1.98	3.5	0.03	15899		
3	64	910	900	1800	300	2.97		0.33	28	0.502	263523		
4	65	2900	900	1800	300			0.62	4	0.355	186265		
3	66	1400	900	1800	300	1.93		0.21	28	1.089	571667		
4	67	1000	900	1800	300			1.8	4	0.035	18275		
3	68	600	900	1800	300	4.5		0.5	28	0.2	105000		
4	69	1000	900	1800	300			1.8	4	0.035	18275		
3	70	1400	900	1800	300	1.93		0.21	28	1.089	571667		
4	71	2900	900	1800	300			0.62	4	0.355	186265		
3	72	1590	900	1800	300	1.7		0.19	28	1.326	696183	4538055	
4	73	2900	900	1800	300			0.62	4	0.355	186265		
3	74	1400	900	1800	300	1.93		0.21	28	1.089	571667		
4	75	1000	900	1800	300			1.8	4	0.035	18275		
3	76	600	900	1800	300	4.5		0.5	28	0.2	105000		
4	77	1000	900	1800	300			1.8	4	0.035	18275		
3	78	1400	900	1800	300	1.93		0.21	28	1.089	571667		
4	79	2900	900	1800	300			0.62	4	0.355	186265		
3	80	910	900	1800	300	2.97		0.33	28	0.502	263523		

Table 9-3 Calculation of Stiffness of the Second Storey Wall in the Retrofitted Situation

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