OUT-OF-PLANE STRENGTHENING OF UNREINFORCED MASONRY WALLS USING TEXTILE REINFORCED MORTAR SYSTEMS

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Summary

Masonry walls are popularly used in building envelopes because of their strength, durability, thermal resistance and aesthetical appearance. However, unreinforced masonry walls are vulnerable to out-of-plane loadings such as those resulting from earthquakes, gas explosions and blasts. In this study, the use of three different textile-reinforced mortar (TRM) strengthening systems to enhance the out-of-plane behavior of unreinforced masonry walls was investigated. These were polypropylene (PP) band-reinforced mortar, ferrocement and alkali resistant (AR)-glass textile reinforced mortar systems.

Material tests were conducted on the compression strength of brick, mortar and strengthening matrix and tensile strength of PP band, wire mesh and AR-fibreglass textile mesh. In addition, tests were performed on walls specimens and strengthening systems to obtain the stress-strain relation in compression and tension respectively. Four-point-bending tests were then carried out to examine the flexural behavior of masonry walls strengthened with the TRM systems under consideration. The walls were tested with the continuous mortar joint parallel or perpendicular to the loading span. For each TRM strengthening systems, the walls were tested in two orthogonal loading directions and the reinforcement ratio varied. In total, 22 wall specimens were tested.

Test results showed that ferrocement was highly effective in increasing the out-of-plane load carrying capacity but not the deformation capacity of the walls. AR-fibreglass reinforced mortar system provided comparable strength enhancement as
ferrocement and also led to higher deformation capacity of the walls. The use of PP-band reinforced mortar system resulted in the largest deformation of the walls but lower load-carrying capacity.

Analytical predictions based on the derived stress-strain relation of the masonry walls in compression and TRM systems in tension compares reasonably well with the test results. It was observed that the load-carrying capacity and energy absorption capacity based on the area under the load-deflection curve until peak load, increases with the reinforcement ratio or tensile capacity of the strengthening system, but were largely independent of the loading direction.
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\[ A_t \quad = \quad \text{cross section area of TRM system} \]

\[ b \quad = \quad \text{wall width} \]

\[ C \quad = \quad \text{compressive force in masonry wall} \]

\[ d \quad = \quad \text{effective depth of wall} \]

\[ E_t \quad = \quad \text{stiffness of TRM composite prior to crack initiation} \]

\[ E_2 \quad = \quad \text{elastic stiffness of TRM composite after crack initiation} \]

\[ E_3 \quad = \quad \text{slope of TRM composite in the plastic region} \]

\[ f_{s, i} \quad = \quad \text{stress and strain in TRM composite with subscripts a, b, c and d} \]

\[ f_{l, d} \quad = \quad \text{corresponding to A,B,C and D respectively; } f_{l, d} \text{ and } \varepsilon_{l, d} \text{ are also refer} \]

\[ f_{l, d} \quad = \quad \text{to as } f_{u, u} \text{ and } \varepsilon_{u, u} \]

\[ f_t , \varepsilon_t \quad = \quad \text{tensile stress and strain in TRM composite} \]

\[ f_{m}, \varepsilon_{m} \quad = \quad \text{compressive stress and strain in masonry} \]

\[ f'_{m}, \varepsilon'_{m} \quad = \quad \text{peak stress and corresponding strain respectively in masonry} \]

\[ f_{mu}, \varepsilon_{mu} \quad = \quad \text{ultimate compressive strength of masonry (defined as 90% of } f'_{m} \text{)} \]

\[ h \quad = \quad \text{full depth of wall} \]

\[ kd \quad = \quad \text{neutral axis depth} \]

\[ L \quad = \quad \text{effective span of the wall specimen} \]

\[ M_u \quad = \quad \text{ultimate moment of resistance of TRM strengthened wall} \]

\[ P_u \quad = \quad \text{ultimate load capacity of TRM strengthened wall} \]

\[ T \quad = \quad \text{tensile capacity of TRM system} \]
Introduction

1.1 GENERAL

Masonry is one of the oldest construction materials. Masonry was used world-wide as the predominant building material before materials such as concrete and steel have been introduced in construction. It has been used in a variety of structural applications, such as arch bridges, walls of buildings, parapets and monuments (Bartoli and Blasi 1997; Hobbs et al. 2009; Melbourne and Tomor 2006). Brick and block masonry are still the most popular building material particularly in developing countries due to its easy handling and cheap costs in construction. Besides, brick masonry provides many additional advantages such as aesthetics, effective heat and sound isolation, fire resistance and economical construction. Due to its many advantages, brick masonry is still well used as envelope in both commercial and residential buildings.

Typically, most of the existing masonry walls in developing countries are in the form of unreinforced masonry (URM). These URM walls are highly vulnerable to out-of-plane loading which may result due to seismic action, high speed winds and blast explosion. In such situations, in-plane shear failure and/or out-of-plane failure can result. In the case of in-plane shear failure, diagonal cracking may occur. However, out-of-plane failure will lead to catastrophic collapse. The out-of-plane failure of URM walls is the main cause of personal casualties and fatalities (Ehshani et al. 1999).

The strengthening of URM structures to enhance the out-of-plane behavior is therefore important. There have been numerous efforts (Albert et al. 2001; Almusallam

1.2 STRENGTHENING METHODS

Common traditional strengthening methods for URM walls include: (a) grout and epoxy injection to fill voids and cracks; (b) re-pointing; (c) confinement using RC elements; (d) post-tensioning; and (e) centre core technique.

It has been reported by ElGawady et al. (2004) that injection of grout or epoxy can restore the initial stiffness and strength of walls by filling voids and cracks. Further, this study recommends that the epoxy resin injection is suitable for small cracks while cement-based grout for large cracks, voids and empty collar joints. This technique is effective at restoring the initial stiffness and strength of masonry. Moreover cement-based grout injection is capable of restoring up stiffness and strength 0.8-1.1 and 0.8-1.4 of the unstrengthened wall respectively. In epoxy injection they were about 0.1-0.2 and 2-4 receptively.

Repointing mortar joints is another traditional method which has been particularly used when mortar joints are weak while bricks are in good quality. As shown in Figure 1-1, this involves replacing the deteriorated mortar layer by higher-strength bonding material. It is usually necessary to repoint when the depth of the open joint is approaching the thickness of the mortar bed. The work is generally straightforward but labour intensive, and though materials are cheap, the ultimate cost of employing a builder may be considerable. Successfully completed repointing should last 50 or 60 years of the mortar joint, the wall and historical structures (Mark et al. 2004).
As shown in Figure 1-2, confinement of URM walls by introducing reinforced concrete tie elements, have been widely used in Asia and Latin America. Particularly, in China, this method has been used in new masonry walls and existing URMs. Usually, URM walls confined with this system are consider to have significant positive effect (Karantoni and Fardis 1992). The confinement of URMs with RC elements prevents disintegration and improves ductility and energy dissipation (ElGawady et al. 2004). However, confined masonry construction is more expensive than URM construction and requires somewhat higher level of labor skills (Brzev 2007).

Post-tensioning of masonry is achieved by applying pre-compressive force to masonry which can counteract the tensile stress. Different types of materials have been used for post-tensioning of masonry such as alloy steel thread bars, scrap rubber tyres as a low cost material (Turer et al. 2007). For instance, as shown in Figure 1-3, shortening the chain of scrap tyre ring will provide the post tensioning forces in the wall. Post-tensioning of masonry improves out-of-plane resistance; also it does not provide additional mass to the original structure. However, post-tensioning is an expensive method due to the requirement of anchorage system and also it is susceptible to corrosion.

As another traditional method, the center core method is achieved by vertically core drilling into masonry walls and placing reinforcement steel into the cores followed by grouting of the cores with a specialized resin grout. This method has been used predominantly in California for seismic rehabilitation of URM buildings (Council 1997). It does not effect the space reduction and improves ultimate lateral load resistance.
The above strengthening methods for masonry structures have been proven to be effective, but have many drawbacks. They are always time consuming to apply, add heavy mass to the structures, and affect the aesthetic appearance of original structure. To overcome most of these problems, external application of overlays such as ferrocement (Tan and Samsu 2007), engineered cementitious composites (ECC) (Lin 2007) and fiber reinforced polymers (FRP) (Albert et al. 2001; Almusallam et al. 2001; Gilstrap and Dolan 1998; Marshall et al. 2000; Mosallam 2007; Nanni and Tumialan 2003; Tan and Patoary 2004; Tan and Patoary 2009; Triantafiliou 1998) have been investigated as successful methods in out-of-plane strengthening up to date. The advantages of their applications include easy installation and minimal additional weight on the structure.

In addition, polypropylene (PP) bands (Macabuag et al. 2009; Paola et al. 2006; Sathiparan et al. 2005) and other textile reinforced mortar (Papanicolaou et al. 2007, 2008) have been introduced as strengthening overlays. Particularly for developing countries, PP bands offer a comparatively cheap and easily available material for strengthening walls.

The choice on the suitability of a strengthening system does not only depend on the degree of damage or required strengthening but also material cost, labor and fabrication cost, availability of technology and workmanship. Considering these factors, this study has been carried out to investigate the flexural characteristics of URM walls strengthened with PP mesh reinforced mortar, ferrocement and Alkali-resistant (AR)-fibreglass textile reinforced mortar system.
1.3 OBJECTIVE AND SCOPE

The main objective of this research is to investigate the effectiveness of different types of textile reinforced mortar systems in out-of-plane strengthening of URM walls to resist lateral loading. To achieve this objective, the scope of study had been set up as summarized in Figure 1-4.

The failure modes and load-carrying capacity in out-of-plane behavior of masonry walls strengthened with PP mesh-reinforced mortar; ferrocement and AR-fibreglass textile reinforced mortar were experimentally investigated. Wall specimens were tested in four-point bending with the continuous mortar joint either parallel or perpendicular to the loading span.

The flexural capacity was calculated using conventional flexural theory incorporating strain compatibility, force equilibrium and constitutive models of the materials.

1.4 THESIS STRUCTURE

In this thesis, Chapter 1 gives an introduction to the research project which is about the necessity of strengthening URM walls to resist lateral loading, existing strengthening methods, and the objective and scope of this study.

Previous research studies on the strengthening of URM walls with the proposed strengthening systems which include PP band reinforced mortar, ferrocement and AR-fibreglass textile reinforced mortar system are reviewed in Chapter 2.

Chapter 3 describes the test to obtain material properties of masonry, brick, mortar and the reinforcement. Test on masonry walls under compression and
strengthening systems under tension are also described which form the basis for the constitutive models for theoretical calculations.

Theoretical formulations to determine the flexural strength of strengthened masonry walls are given in Chapter 4. The failure modes are examined and applications to TRM strengthened walls are described.

The test program for flexural testing of TRM strengthened masonry walls are described in Chapter 5. The discussion of the test results including comparison with theoretical predictions are also presented in Chapter 5. The effect of test parameters that is loading direction, type of TRM strengthening systems and reinforcement amount are also evaluated.
Chapter 1: Introduction

(a) Hammer out the old mortar  
(b) Brush out loose mortar  
(c) Soak the brick with water  
(d) Slide the mortar in

Fig. 1-1: Repointing steps in masonry

Fig. 1-2: Confinement of brick masonry wall by placing of new RC elements  
(Paikara and Rai 2006)
(a) Two steel bolts placed through those holes are used to connect the two pipes and scrap tyre ring (STR)

(b) Shortens the STR chain while generating an adjustable tensile force

(c) The post-tensioning forces on the wall

Fig. 1-3: Constructing the post tensioning straps (Turer et al. 2007)
Strengthening of unreinforced Masonry Wall with thin layer of cement matrix with reinforcement mesh (TRM)

Laboratory tests

Analysis

- PP reinforced Mortar
- Ferrocement
- AR-Fiberglass TRM
- Longitudinal direction
- Transverse direction

Different types of TRM strengthening systems

Loading direction

Amount of reinforcement

Identify failure modes

Verify the model with experimental results

Fig. 1-4: Scope of research
Chapter 2: Literature Review

Literature Review

2.1 GENERAL

In many disasters, casualties and fatalities due to collapse of masonry structures are common because of their poor performance under lateral loading. Various strengthening methods for masonry walls had been studied. This chapter summarizes previous works on strengthening of URM structures that have been done using PP-band mesh, ferrocement and AR-fibreglass textile reinforced mortar, that are relevant to the present study.

Polypropylene (PP) band is a universal cheap packing material having considerable elongation capacity. It is of more practical use in developing countries, since it is a low-cost material and can be simply installed with available resources and skills. Up to date, it has been applied only in seismic strengthening of URM walls. By encasing the walls with PP-band meshes, it is possible to contain debris of the collapsed walls from flying off.

Ferrocement is a thin layer of cementitious composite which is reinforced with closely and uniformly spaced wire mesh with square or rectangle grid. In the beginning, ferrocement was very popular in liquid-retaining structures such as water tanks and casing for wells and sedimentation tanks. Later, ferrocement has been extensively used as a structural element and strengthening material in the field of civil engineering due to advantages such as high tensile strength to weight ratio, crack control capability, high ductility, and impact resistance. Ferrocement is ideal for low
cost housing in developing countries since it is cheap and can be done with unskilled workers. It improves both in-plane and out-of-plane behavior of URM walls (ElGawady et al. 2004).

Textile reinforced concrete has been introduced as an alternative to fiber reinforced polymer (FRP) system (Papanicolaou et al. 2007; 2008; Triantafillou and Papanicolaou 2006). It has additional advantages such as ability to be produced in thinner layers and also high strength to weight ratio. Although application of TRM in civil engineering structures started few years ago, considerable number of studies can be found in literature because of its advantages as a strengthening material. The main components of TRM are textile reinforcement and fine-grained concrete. The most popular textile in textile reinforced concrete is AR-fibreglass (Bruckner et al. 2008; J.Hegger 2006; Moller et al. 2005; U.Haubler-Combe and JHartig 2007).

2.2 PP-BAND REINFORCED MORTAR SYSTEM

Polypropylene bands have been proposed as a cost-effective retrofitting material in Japan. The suitability of this material in the form of mesh to seismically retrofit URM walls has been verified experimentally (Mayorca 2004). Figure 2-1 shows the tensile characteristics of a typical PP band (Sathiparan et al. 2005). To determine the resistance to in-plane and out-of-plane loading, diagonal compression (Figure 2-2) and flexural bending (Figure 2-3), tests for PP mesh reinforced wallets and unreinforced wallets have been conducted (Sathiparan et al. 2005). The diagonal compression tests showed that PP mesh strengthened walls provide higher residual strength after formation of the first diagonal shear cracks. The out-of-plane tests also indicated the effectiveness of PP mesh after the walls have cracked. The strength and deformation of PP mesh reinforced walls were 2.5 times and 45 times, respectively,
those of the un-retrofitted wallets, in diagonal compression tests. In out-of-plane bending tests, they were 2 times and 60 times respectively. As shown in Figure 2-4, the behavior of walls strengthened with various PP band mesh arrangements in diagonal compression have been studied (Macabuag and Bhattacharya 2008). These tests proved that initial failure stress is unaffected by the presence of the PP mesh due to the much lower stiffness of PP mesh compared to masonry.

On the other hand, in-plane lateral behavior of PP band strengthened walls have been studied by Mayorca (2004) using medium-scale walls as shown in Figure 2-5. In this study, inclined PP mesh has been employed. It was observed that, immediately after the peak load, corresponding to the diagonal cracking, the unreinforced wall strength dropped to 10 to 40% of the peak value. On the other hand, the reinforced walls exhibited a 60% residual strength after the peak, which was sustained for at least 2% lateral drift.

2.3 FERROCEMENT

Ferrocement has also been used as a strengthening system. This is a cementitious composite layer laminated with metallic mesh and has advantages such as a high tensile strength-to-weight ratio and superior cracking behavior (Tamer et al. 2005).

Prawel and Lee (1988) showed that ferrocement overlays increased the efficiency of diagonal tensile strength, stiffness and deformation capacity of masonry panels. Kabir and Hasan (1999) have studied the strength enhancement in brick masonry columns by encasing with precast ferrocement. Based on their investigations, the cracking and failure stresses of column with precast ferrocement jackets have substantially been increased compared to control specimens while exhibiting much
ductile response. According to the study of Tan and Samsu (2007), ferrocement is found to be an effective system in out-of-plane strengthening of unreinforced two-way masonry walls.

Although, few studies are available in the literature on strengthening of masonry structures with ferrocement, considerable research works have been done on strengthening of reinforced concrete structures with ferrocement. Al-Kubaisy and Zamin Jumaat (2000) have studied the flexural behavior of reinforced concrete slabs with ferrocement which was used as a tension zone cover to reinforcement. The study has considered volume fraction of the longitudinal reinforcement in the ferrocement cover, thickness of ferrocement cover and method of structural connection between the concrete slab and ferrocement cover as test variables. It concluded that ferrocement cover can be a feasible method for tension zone cover of reinforced concrete slabs providing superior crack control, higher stiffness and higher first crack moment compared to similar slabs with normal concrete cover.

Nassif and Najm (2004) have studied composite beams made of reinforced concrete overlaid on thin section of ferrocement. They have particularly studied the method of shear transfer between composite layers. Their study concluded that the full composite action between concrete beam and ferrocement overlay cannot be achieved by roughening surface without using shear studs. Furthermore, beams having shear studs with hooks exhibited better pre-cracking stiffness as well as cracking strength than L-shaped shear studs. (Nassif and Najm 2004) further stated that as shown in Figure 2-6, beams strengthened with square mesh shows better cracking capacity than the unstrengthened. The same applied to beams strengthened with hexagonal mesh when compared to the respective unstrengthened beam. However, the change in the
ultimate capacity was not significant. Furthermore, Ong et al. (1992) also studied the strengthening of RC beams with ferrocement laminates and showed that full composite action can be obtained by roughening the interface between ferrocement and concrete and providing loosely spaced shear connectors.

Abdullah and Takiguchi (2003) studied the behavior and strength of reinforced concrete columns strengthened with ferrocement jackets. A total number of six column specimens have been strengthened with circular or square ferrocement jackets (see Figure 2-7) with ratio of axial load and wire mesh layers as test variables. The specimens were tested under cyclic and constant axial loads. The study showed that by providing external confinement over the entire length of the RC columns, the ductility is significantly increased.

2.4 AR-GLASS REINFORCED TEXTILE SYSTEM

As shown in Figure 2-8, typical stress-strain curve for textile reinforced concrete can be characterized by three states (Haubler-Combe and Hartig 2007). In the first state, stress and strain are linearly related because concrete is un-cracked. With the formation of the first crack, the stiffness decreases suddenly in state-IIa due to multiple cracking. After multiple cracking (i.e. in state IIb), the stiffness of the stress-strain curve, increases to a value close to but lesser than the stiffness of reinforcement. This occurs because of incomplete and inhomogeneous load carrying effect of all filaments of the textile roving and imperfect bonding between matrix and rovings. Compared to rebars, the stress-strain curve of TRC does not show a state of yielding prior to ultimate failure.

The main reason for the reduction of strength of the roving in composites than the individual filament strength is the ineffectiveness of the total cross section of the
rovings due to the insufficient bond between filaments and the matrix. As discussed by Schleser et al. (2006), there are three methods of polymer application to TRM to improve the load transfer behavior by bond. They are impregnation of roving before embedding them in concrete, addition of polymers to matrix and combination of both methods. The third method shows the best tensile results as shown in Figure 2-9.

As an another improvement to TRM, Hinzen and Brameshuber (2007) have proposed adding ductile short fibers to further improve serviceability and load bearing capacity, as well as to optimize the crack development in TRM. As shown in Figure 2-10, this study investigated the effect of application of different short fibers (steel, glass, carbon and PVA) on AR-glass textile reinforced concrete. Figure 2-11 shows the effect of the addition of these short fibers on the cracked area of tensile specimens with reference specimen of AR-glass textile reinforced concrete. Therefore, the study concluded that the cracking pattern can be significantly improved by the addition of all short fibers except carbon fibers.

Owing to several remarkable properties, TRM has become popular as a strengthening material. Compared to short fibers, the reinforcement can be placed in the desired direction, thus achieving optimization in the amount of reinforcement (Schneider and Bergmann 2005). Furthermore, because of the smaller diameter of the reinforcement and small requirement for reinforcement cover to protect against corrosion, very thin concrete elements (of 10-20mm thick) can be constructed. The higher strength to weight ratio is also another beneficial property of TRM.

It has shown that the use of AR-glass TRM system increase both the flexural capacity and shear carrying capacity of RC (slabs and beams) (Bruckner et al. 2006). As shown in Figure 2-12, the load-deflection curve of a TRM strengthened slab rises
much more sharply than the non-strengthened slab due to the larger moment of inertia resulting from additionally applied TRM layer in the non-cracked region (Bruckner et al. 2006). After multiple cracking, the steeper rise of the curve is provided by textile reinforcement. The study further reported on TRM shear strengthening of reinforced concrete rectangular and T beams. As shown in Figure 2-13, the ultimate load of the beam strengthened with only fine grained concrete, showed very little increment over that of the reference beam. However, beams strengthened with two or three layers of textile considerably increased the shear capacity of the beams. In the case of T beam, with up to two layers of textile reinforcement, the ultimate load is about the same with or without mechanical anchoring. However, as can be seen in Figure 2-14, without mechanical anchoring, the specimens with four layers of textile reinforcement failed by almost the same ultimate load as the specimens with two layers of textile reinforcement. Bruckner, et al.(2008) have also studied the anchoring of TRM in shear strengthening of T beam. As shown in Figures 2-15, T beam strengthened with four numbers of textile layers without mechanical anchoring, has debonded by showing large increment of the deformation at about 350 kN and also the achieved ultimate load is about the same as unstrengthened beam. However, it further shows that T beams strengthening with mechanical anchorage, has considerably increased the ultimate load capacity.

Among the few studies on TRM strengthening of URM walls, Papanicolaou (2007; 2008), have studied the in-plane and out-of-plane behavior of TRM strengthened masonry walls and compared them with FRP strengthened masonry walls. In their out-of-plane strengthening study, ten medium-scale specimens were used under two series as shown in Figure 2-16: (a) Series A specimens were tested out-of-plane, such that the plane of failure would form parallel to the bed joints; and (b)
Series B specimens were tested out-of-plane, such that the plane of failure would form perpendicular to the bed joints. Each series consisted of one control specimen, two specimens each strengthened with one or two layers of textile bonded with commercial polymer-modified cement mortar (M) and two identical specimens where the textile were bonded with a epoxy adhesive (R). All specimen were subjected to cyclic out-of-plane loading under three point bending arrangement as shown in Figure 2-17. As can be seen in the Figure 2-18, load-displacement envelopes show that textile reinforced mortar jackets were extremely effective than FRP jackets and all strengthened specimens in Series A failed in flexure-shear in the push direction. The average strength and deformation of walls strengthened with TRM jackets were 2 times and 1.2 times, respectively, those of walls strengthened with FRP. However, as shown in Figure 2-19, in Series B where there was inadequate reinforcement, the failure was controlled the tensile fracture of textile in TRM jacket, with the specimens showing slightly less strength and deformability than that with FRP jacketing. The investigation concluded that TRM jacketing is a suitable for seismic retrofitting of URM subjected to out-of-plane bending.
(a) PP band  
(b) stress-strain relation in tension

**Fig. 2-1:** Tensile characteristics of PP band (Sathiparan *et al.* 2005)

**Fig. 2-2:** Masonry wall specimens under diagonal compression (Sathiparan *et al.* 2005)
Fig. 2-3: Masonry wall specimens under out-of-plane bending (Sathiparan et al. 2005)

Fig. 2-4: Effect of the mesh layout on behavior of masonry walls (Macabuag and Bhattacharya 2008)
Fig. 2-5: PP-band Retrofitted wall before mortar overlay setting and after test (Mayorca 2004)

Fig. 2-6: Load-deflection curves for beams strengthened with ferrocement that contains square wire mesh and hexagonal mesh (Nassif and Najm 2004)
Fig. 2-7: (a) Reference column; (b) column with square ferrocement jacket; (c) column with circular ferrocement jacket  (Abdullah and Takiguchi 2003)

Fig. 2-8: Typical stress-strain relation of TRM (Haubler-Combe and Hartig 2007)
Fig. 2-9: Tensile specimens test with modified and unmodified concrete and rovings (Schleser et al. 2006)

(a) Steel short fibers     (b) Glass short fibers

(c) Carbon short fibers  (d) PVA short fibers

Fig. 2-10: Tensile stress-strain characteristics of AR-fibreglass TRM with addition of short fibers (Hinzen and Brameshuber 2007)
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Fig. 2-11: Crack pattern of tensile specimen of AR-fibreglass TRM with addition of short fibers (Hinzen and Brameshuber 2007)

(a) Without addition of short fibers
(b) PVA short fibers
(c) Carbon short fibers
(d) Steel short fibers
(e) Glass short fibers

Fig. 2-12: Load-displacement diagram one-way RC slab (Bruckner et al. 2006)
Fig. 2-13 : Load Displacement Diagram of rectangular Beams (Bruckner et al. 2006)

Fig. 2-14 : Load Displacement Diagram of T Beams (Bruckner et al. 2006)
Fig. 2-15: Load-displacement diagram of TRM strengthened T beams (Bruckner et al. 2008)

Fig. 2-16: Specimens detail series (a) A specimens (b) Series B Specimens (Papanicolaou et al. 2008)
Fig. 2-17: Cyclic out-of-plane test set-up under three point bending

(Papanicolaou et al. 2008)

Fig. 2-18: Envelope curve of Load versus mid-span displacement hysteresis for Series A (Papanicolaou et al. 2008)
Fig. 2-19: Envelope curve of Load versus mid-span displacement hysteresis for Series B (Papanicolaou et al. 2008)
Chapter 3: Material Properties

Material Properties

3.1 GENERAL

This chapter discusses the material properties of the strengthening systems which will be used for the theoretical predictions of the ultimate load-capacity of the strengthened walls. To obtain material properties, laboratory tests have been performed both on the constituent materials as well as on the composites systems.

3.2 CONSTITUENT MATERIALS

3.2.1 Compressive strength

3.2.1.1 Brick elements

All masonry walls specimens were fabricated using solid clay bricks with average dimensions of 70 mm × 95 mm × 215 mm. Following the test method in BS EN 772-1:2000, the compressive strength of brick was established from six specimens as 30 MPa. Test brick was done with size of 70 mm × 95 mm × 100 mm which was obtained by cutting from normal brick unit. The loading was applied at a rate of 200 kN/min.

3.2.1.2 Mortar

All masonry specimens were built with 10 mm thick mortar with a 1:3 cement: sand proportion by volume. River sand was used. The water cement ratio for the mortar mix was 0.45. Compressive strength of mortar in each wall specimen was measured using 100 mm cubes made from the same batch mix used in the fabrication
of the masonry specimen. The average compressive strength based on 3 cubes for each walls varies from 25 to 30 MPa.

### 3.2.1.3 Fine grained mortar

Fine-grained mortar was used as matrix in the PP band reinforced mortar and ferrocement strengthening systems. The maximum size of the aggregate was 1 mm, which was obtained by sieving sand and the proportion of cement: sand: water is 1:1.5:0.45. The compressive and flexural tests were carried out according to standard of (BSEN12190:1999 1999) and (BSEN196-1:2005 2005) respectively as shown Figure 3-1. The average compressive strength based on three 40 mm cubes varies from 55 to 65 MPa from wall to wall. Correspondingly, the flexural strength, also based on three 40 mm × 40 mm × 160 mm prisms varies from 5 to 6 MPa.

### 3.2.1.4 Polymerized fine grained concrete

In this study, the AR-fibreglass textile was embedded in a commercially available polymerized fine-grained concrete which combined two products of high strength cementitious powder and polymer liquid. According to the manufacturer, this mortar has high-bond strength with concrete and masonry surfaces. Once it is hardened, it forms a tough and compact layer which is impermeable to water and gases that may be present in the atmosphere. The mortar shows higher flexural strength to compressive strength ratio compared to normal fine grained concrete. The average compressive strength and flexural strength were measured as 33 MPa and 8 MPa respectively using specimens as discussed in section 3.2.1.3.
3.2.2 Tensile strength

PP bands with a cross-sectional area measuring 11.85 mm (width) × 0.85 mm (thickness) were interwoven in two orthogonal directions and they were connected with stapling at the joints to form PP band meshes as shown in Figure 3-2. Square wire mesh (used in ferrocement) consisted of band having a diameter of 1.22 mm welded orthogonally at 12.5 mm spacing (Figure 3-3). The alkali-resistant fiberglass mesh was a commercially fabricated mesh with bundle of glass fibers woven at 25 mm spacing in orthogonal directions as shown in Figure 3-4. The weight of AR-fibreglass mesh is specified as 225 g/m².

Tensile properties of the reinforcement were determined as shown in Figure 3-5. The PP band was tested in the form of single strip while the other two reinforcements were tested in the form of mesh in which they were manufactured. The width of the mesh for tensile tests was 50 mm. The ends of the mesh were glued on to 1mm thick aluminum plates to facilitate the gripping of the specimens and preventing slip during the tests.

To measure the strains, two methods were used; the first directly using strain gauges installed on the reinforcement and from displacement measurements. In the case of PP bands, an extensometer was used to measure the elongation (Figure 3-5 (a)) while in the case of AR-Glass mesh and wire mesh, two LVDTs in a frame were used as shown in Figures 3-5 (b) and(c). The PP band strip was tested with a loading rate of 0.5 mm/min initially and increasing to 5 mm/min in the later stages. The other two meshes were tested using a loading rate of 0.1 mm/min throughout the test.
3.2.2.1  **PP bands**

The stress-strain characteristics of PP bands used in this study is shown in Figure 3-6(a). The material has a low stiffness, equal to 1.4 GPa in the initial stage. It can be seen that the ultimate stress is 85 MPa. PP bands show a very large strain capacity of approximately 30%.

3.2.2.2  **Wire mesh**

Tensile stress-strain characteristics of welded wire mesh used in ferrocement is shown in Figure 3-6 (b). The Young’s modulus based on the initial shape of the curve is 160 GPa. Yield strengths are approximately 400MPa. The figure indicates an average 0.3% yield strain capacity and 0.7% ultimate strain capacity for the welded wire mesh.

3.2.2.3  **AR-fibreglass textile mesh**

Results of this study show that AR-fibreglass mesh is highly brittle compared to wire mesh and PP band in Figure 3-6 (c). The Young’s modulus of AR-fibreglass is about 40GPa. The strength capacity of AR-fibreglass is closer to that of welded wire mesh and it is 400MPa. The area of one roving of AR-Fiberglass mesh was calculated by multiplying the measured average width and thickness. The mesh area was then obtained by multiplying the number of roving across the section and area of one roving.

3.3  **CONSTITUTIVE MODELING**

To obtain theoretical predictions for TRM strengthened walls, the stress-strain characteristics of masonry and all strengthening systems under appropriate loading action are required.
3.3.1 Masonry walls under compression

The most important parameter in the structural analysis and design of masonry is the stress-strain relation in compression, including the behavior beyond the elastic limit. Few studies have been done on the stress-strain relations of masonry. Based on experimental data, Kaushik et al. (2007) have proposed an analytical model as shown in Figure 3-7. According to their proposed model, the curve follows a parabolic variation up to stress level of 90% of peak stress beyond the peak stress. Thereafter, the relation shows a linear variation until a stress level of 20% of peak stress.

Since masonry is an anisotropic composite, its material properties are dependent on the loading direction. Experimental investigations carried out in this study on the compressive behavior of masonry wall in loading directions parallel and perpendicular to the bed joint are shown in Figure 3-8. As can be seen from this figure, the peak compressive stress normal to the bed joint is higher (17.8 MPa) than that in parallel (11.5 MPa) to the bed joints while the peak strain remains the same.

3.3.2 TRM strengthening systems under tension

In this study, URM walls were strengthened with three types of TRM overlays. They were namely polypropylene band-reinforced mortar, ferrocement and AR-fibreglass reinforced textile mortar. Both PP band-reinforced mortar system and ferrocement contained normal fine-grained mortar while AR-fibreglass TRM systems contained polymerized fine-grained mortar as matrices.

Tests were carried out using dog-bone shaped specimens to determine the tensile capacity of the strengthening systems. Specimens were cast with the same reinforcement amount as they were applied on masonry wall specimens. Test specimens were designated as TP1, TP2 and TP3 for 1, 2 and 3 layers of PP band mesh.
respectively in the case of PP band reinforced mortar system, TF1, TF2 and TF3 for 1, 2 and layers of wire meshes respectively in the case of the ferrocement system and TT1, TT2, TT3, TT4 and TT6 for 1,2,3,4 and 6 layers of AR-fiberglass textile meshes respectively in the case of AR-fiberglass textile mortar.

Tensile tests on strengthening specimens were performed on the same day as wall testing. Details of specimen fabrication and test procedure are explained in following sections.

3.3.2.1 Specimen preparation

Tensile specimens were cast in dog-bone shaped moulds. The length of the specimens was 300 mm and the ends were 75 mm in width and thickness of specimens are mentioned in Table 3-1. To prevent from undesirable cracking outside the gauge length, all specimens’ ends were internally reinforced with additional wire meshes. Mortar overlays and meshes were placed alternatively and to ensure proper compaction, the specimens were placed on a small vibration table. The fabrication steps are shown in Figure 3-9. After 24 hours, the specimens were de-molded and covered with plastic sheets similar to the curing of the walls. To facilitate gripping during tests, aluminum plates measuring 75 mm × 75mm and 1mm thickness were glued using epoxy.

3.3.2.2 Test set-up and instrumentation

The test set-up and arrangement of measuring instrument are shown in Figure 3-10. The average thickness of specimens was measured with a Vernier caliper before the test. To measure the strains using displacement method, two linear variable differential transducers (LVDTs) were mounted on a frame, over a gauge length of 80
mm, as shown in the test arrangement. Three tensile specimens were tested for each reinforcement ratio.

### 3.3.2.3 Test procedure

The load was applied to the tensile specimens by gripping on the end plate over an area of measuring 50 mm × 50 mm, using a hydraulic jack with displacement control. The initial loading rate was 0.05 mm/min until the matrix has cracked, and then gradually increased till the specimen failed. The load and deflection readings were recorded.

### 3.3.2.4 Test results and discussion

The stress-strain characteristics of all strengthening systems are shown and characterized by piecewise-linear relations in Figures 3-11 (a), (b) and (c) for PP-band reinforced mortar, ferrocement and AR-fibreglass textile reinforced mortar system respectively. The corresponding load-strain relations are further shown together with those of the reinforcement alone in Figures 3-12 (a), (b) and (c).

Point A defines the cracking load of the composite systems, and is governed by the reinforcement ratio. The reason for this is that the proportion of tensile load taken by reinforcement increased with the amount of reinforcement. The cracking strains were independent of the amount of reinforcement. Immediately after first cracking, the applied load dropped to point B; the drop being larger in the case of ferrocement and AR-fibreglass reinforced mortar system and smaller in PP-band reinforced mortar system, due to a much higher reinforcement ratio. The reinforcement ratio is defined as $A_r/bh$ where $A_r$ is the area of reinforcement and $b$ and $h$ are the width and thickness of the original wall respectively.
At Point B, the load started to increase linearly again, but at a slower rate, until point C which corresponds to the first yield of reinforcement in the case of ferrocement. In the case of PP-band-reinforced mortar and AR-fibreglass textile reinforced mortar systems, point C corresponds to a change from linear elastic behavior to plastic behavior of the reinforcement. Thereafter, the specimens continued to elongate under more or less the same applied load in the case of ferrocement. Whereas the load increased further until it reached the peak value at point D, where the specimen broke into two in the case of PP-band and AR-fibreglass reinforced mortar specimens.

The tensile strain capacity is defined as the strain at which the load dropped drastically due to rupture of reinforcement. The tensile strain capacity of PP-band strengthened mortar system was highest at about 45%, followed by AR-fibreglass textile reinforced mortar at about 2.5% and ferrocement at about 0.7%.

All of the strengthening systems show improvement in tensile load-carrying capacities with an increase in reinforcement ratio. Figure 3-13 summarizes the tensile capacities of strengthening systems. It is seen that the tensile capacity increased almost linearly with the number of reinforcement meshes.

### 3.3.2.5 Material Model

The tensile stress-strain characteristics for all strengthening systems can be modeled by piece-wise linear relations as shown in Figure 3-14. Five reference points (A, B, C, D and E) have been used to describe the complete behavior.

A further simplified model as shown in Figure 3-15 is obtained by ignoring point A and E and considering points B, C and D only. That is, the simplified tensile
stress-strain curve for the prediction of the ultimate load-carrying capacity of strengthened walls in this study is as follows:

**Regime OB**

\[
f_t = E_1 \varepsilon_t \quad \text{if } \varepsilon_t \leq \varepsilon_b, \quad \text{where } E_1 = \frac{f_b}{\varepsilon_b}\]

(3-1)

**Regime BC**

\[
f_t = f_b + E_2 (\varepsilon_t - \varepsilon_b) \quad \text{if } \varepsilon_b < \varepsilon_t < \varepsilon_c \quad \text{where } E_2 = \frac{f_c - f_b}{(\varepsilon_c - \varepsilon_b)}
\]

(3-2)

**Regime CD**

\[
f_t = f_c + E_3 (\varepsilon_t - \varepsilon_c) \quad \text{if } \varepsilon_c < \varepsilon_t \leq \varepsilon_d \quad \text{where } E_3 = \frac{f_d - f_c}{(\varepsilon_d - \varepsilon_c)}
\]

(3-3)

where

\[f_t, \varepsilon_t = \text{tensile stress and strain in TRM composite}; E_1 = \text{stiffness of TRM composite prior to crack initiation (i.e. slope of stress-strain relation in regime OB)}; E_2 = \text{elastic stiffness of TRM composite after crack initiation (i.e. slope of stress-strain relation in regime BC)}; E_3 = \text{slope of TRM composite in the plastic region; (i.e. slope of stress-strain relation in regime CD)}; f_b, \varepsilon_b = \text{stress and strain in TRM composite with subscripts b, c and d corresponding to B, C and D respectively}; f_d \text{ and } \varepsilon_d \text{ are also referred to as } f_{tu} \text{ and } \varepsilon_{tu} \text{ in subsequent chapters.}

The values of these parameters for each strengthening system are shown in Table 3-1.
### Table 3-1: Parameters defining the simplified tensile stress-strain curve for TRM strengthening systems

<table>
<thead>
<tr>
<th>Strengthening system</th>
<th>Specimen</th>
<th>Thickness</th>
<th>$f_b$ (MPa)</th>
<th>$\varepsilon_b$ (%)</th>
<th>$E_1$ (GPa)</th>
<th>$f_c$ (MPa)</th>
<th>$\varepsilon_c$ (%)</th>
<th>$E_2$ (GPa)</th>
<th>$f_d$ (MPa)</th>
<th>$\varepsilon_d$ (%)</th>
<th>$E_3$ (GPa)</th>
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<tr>
<td>PP-band reinforced mortar</td>
<td>TP1</td>
<td>14.2</td>
<td>13.2</td>
<td>13.9</td>
<td>1.5</td>
<td>0.15</td>
<td>1.00</td>
<td>3.3</td>
<td>20</td>
<td>0.01</td>
<td>3.9</td>
</tr>
<tr>
<td></td>
<td>TP2</td>
<td>16.6</td>
<td>16.4</td>
<td>17.0</td>
<td>2.5</td>
<td>0.15</td>
<td>1.67</td>
<td>5</td>
<td>20</td>
<td>0.01</td>
<td>6.9</td>
</tr>
<tr>
<td></td>
<td>TP3</td>
<td>15.8</td>
<td>15.9</td>
<td>15.8</td>
<td>4</td>
<td>0.15</td>
<td>2.67</td>
<td>8</td>
<td>20</td>
<td>0.02</td>
<td>10.25</td>
</tr>
<tr>
<td>Ferrocement</td>
<td>TF1</td>
<td>12.5</td>
<td>12.6</td>
<td>12.4</td>
<td>1.8</td>
<td>0.05</td>
<td>3.60</td>
<td>3.5</td>
<td>0.15</td>
<td>1.70</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>TF2</td>
<td>16.5</td>
<td>15.7</td>
<td>15.4</td>
<td>2.5</td>
<td>0.05</td>
<td>5.00</td>
<td>4.8</td>
<td>0.18</td>
<td>1.84</td>
<td>4.8</td>
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<tr>
<td></td>
<td>TF3</td>
<td>16.0</td>
<td>14.7</td>
<td>16.2</td>
<td>3.2</td>
<td>0.05</td>
<td>6.40</td>
<td>6.7</td>
<td>0.2</td>
<td>2.33</td>
<td>6.7</td>
</tr>
<tr>
<td>AR-fibreglass reinforced mortar</td>
<td>TT1</td>
<td>13.3</td>
<td>12.5</td>
<td>12.3</td>
<td>2.4</td>
<td>0.3</td>
<td>0.80</td>
<td>3.25</td>
<td>1</td>
<td>0.12</td>
<td>4.1</td>
</tr>
<tr>
<td></td>
<td>TT2</td>
<td>17.0</td>
<td>14.5</td>
<td>16.4</td>
<td>3.2</td>
<td>0.3</td>
<td>1.07</td>
<td>4.2</td>
<td>1</td>
<td>0.14</td>
<td>5.25</td>
</tr>
<tr>
<td></td>
<td>TT3</td>
<td>16.2</td>
<td>16.1</td>
<td>15.8</td>
<td>4</td>
<td>0.24</td>
<td>1.67</td>
<td>5.2</td>
<td>1</td>
<td>0.16</td>
<td>6.2</td>
</tr>
<tr>
<td></td>
<td>TT4</td>
<td>17.3</td>
<td>17.1</td>
<td>15.9</td>
<td>4.25</td>
<td>0.35</td>
<td>1.56</td>
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<td>1.25</td>
<td>0.24</td>
<td>7.25</td>
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<td></td>
<td>TT6</td>
<td>15.2</td>
<td>16.4</td>
<td>16.2</td>
<td>4.5</td>
<td>0.23</td>
<td>2.00</td>
<td>8.8</td>
<td>1.25</td>
<td>0.42</td>
<td>11.25</td>
</tr>
</tbody>
</table>

* refer to Figure 3-15 for definitions of symbols

* Width of specimen = 50 mm
Fig. 3-1: Compressive test and flexural test configuration (all dimension in mm.)
Fig. 3-2: Fabricated PP band mesh

Fig. 3-3: Welded Wire mesh

Fig. 3-4: Woven AR-fibreglass mesh
Chapter 3: Material Properties

Fig. 3-5: Reinforcement meshes - tensile test arrangement

(a) PP-band

(b) Wire-Mesh

(c) AR-fibreglass mesh
Fig. 3-6 (a): Stress–strain curves for reinforcement materials-
PP band

Fig. 3-6 (b): Stress–strain curves for reinforcement materials-
Wire mesh
Chapter 3: Material Properties

Fig. 3-6 (c): Stress–strain curves for reinforcement materials- AR-fibreglass mesh

Fig. 3-7: Analytical model for stress-strain of masonry  (Kaushik et al. 2007)
Fig. 3-8: Uni-axial compressive stress-strain relation of masonry obtained from current tests

(a) Casting of bottom mortar layer
(b) Placing of end wire mesh pieces
(c-1) Placing of PP mesh (TP)
(c-2) Placing of steel wire mesh (TF)
(c-3) Placing of AR-fibreglass textile mesh (TT)
(d) Casting of top mortar layer
(e) Specimens after de-moulding
(f) Gluing of aluminum plates to ends of specimen

Fig. 3-9: Casting of dog-bone shaped TRM tensile specimens
Fig. 3-10: Geometry of tensile specimens and test set-up (all dimension in mm.)
Fig. 3-11(a) : Tensile stress-strain characteristics of PP-band reinforced mortar system (TP)
Fig. 3-11(b) : Tensile stress-strain characteristics of Ferrocement (TF)
Fig. 3-11 (c) Tensile stress-strain characteristics of AR-fibreglass reinforced mortar (TT)
Fig. 3-12 (a) : Load-strain curve of PP-band reinforced mortar system with PP band reinforcement

Fig. 3-12 (b) : Load-strain curve of ferrocement system with Steel wire mesh
Fig. 3-12 (c) : Load-strain curve of AR-fiberglass TRM system with corresponding AR-fibreglass textile

Fig. 3-13: Comparison of tensile capacities of TRM strengthening systems
Fig. 3-14: Simplified tensile stress-strain model of TRM strengthening systems

Fig. 3-15: Generalized tensile stress-strain Curve with further simplification
Theoretical Considerations

4.1 GENERAL

The study has experimentally investigated the out-of-plane bending behavior of URM walls strengthened with three different types of textile reinforced mortar systems. All of the strengthened specimens were tested under four-point bending. Loading direction and type and amount of reinforcement were the test parameters. This chapter describes a simplified analytical model to predict the ultimate load carrying capacity of the strengthened walls in out-of-plane bending. The model has been derived as analogous to the flexural section analysis of reinforced concrete beams. Basically, flexural rupture of reinforcement and crushing of masonry can be considered as the failure types of the strengthened walls.

4.2 ULTIMATE LOAD CARRYING CAPACITIES OF TRM STRENGTHENED MASONRY WALLS

As shown in Figure 4-1, all strengthened walls were subjected to four-point bending. Moreover, the wall specimens were categorized into two groups according to the loading arrangement, that is with the plane of failure parallel or perpendicular to the continuous mortar joints. However, as discussed in the Chapter 3, the main compressive stress-strain relations depend on the direction of load with respect to the mortar joint. On other hand, the tensile resistance of the walls can be neglected. As shown in Figure 4-2, the strengthened wall may fail either due to crushing of masonry in compression or tensile rupture of reinforcement in the TRM strengthening layer. The load carrying capacity of the strengthened walls is derived as follows.
4.2.1 Method of strain compatibility

4.2.1.1 Flexural failure

The ultimate moment capacity of strengthened wall specimens is calculated based on strain compatibility and internal force equilibrium using the relevant material constitutive models. Furthermore, the following assumptions have been.

(a) plane section remains plane after bending;

(b) strains vary linearly across the section;

(c) tensile resistance of masonry can be neglected;

(d) each layer of reinforcement is placed in the mid-depth of the strengthening layer; and

(e) perfect bond exists between strengthening layer and masonry;

The parabolic stress-strain distribution of masonry as shown in Figure 3-7 was considered in the prediction. The compressive failure is when the maximum strain reached an ultimate strain value of 0.0035 (see Figure 3-8).

The derivations are summarized below. Consider a section of the strengthened wall with a width \( b \), and thickness \( d \), and subjected to bending as shown in Figure 4-3. The thickness of TRM system is \( t_t \) and the reinforcement ratio is \( \rho_t = (A_{mt}/bd) \) where \( A_{mt} \) is the area of reinforcement mesh across the width of the wall specimen. In the case of meshes, \( A_{mt} \) is equal to \( nA_r(b/s) \), in which \( n \) is the number of layers of reinforcement mesh, \( A_r \) is the area of a single strip/wire/roving and \( s \) is the spacing between the strips/wires/rovings in the mesh. The total area of strengthening system is \( A_t \) which is equal to the width times the thickness of strengthening layer \( t_t \). The tensile capacity
provided by strengthening system is calculated based on the stress-strain curve corresponding to the type of reinforcement and number of reinforcement layers as discussed in section 3.3.2.

(a) Balanced failure

Balanced failure will result if the masonry crushes and strengthening reinforcement ruptures simultaneously. The corresponding stress and strain distributions over the section are shown in Figure 4-4.

From strain compatibility, the neutral axis depth ratio ($k$) can be obtained as:

$$\frac{\varepsilon_{tu}}{\varepsilon_{mu}} = \frac{(1 - k)}{k}$$

or

$$k = \frac{\varepsilon_{mu}}{\varepsilon_{mu} + \varepsilon_{tu}} \quad (4-1)$$

where $\varepsilon_{mu}$ is the ultimate strain of masonry in compression and $\varepsilon_{tu}$ is ultimate strain of TRM strengthening system.

Compressive force carried by masonry can be obtained by integrating the compressive stress $f_{m}bdx$ over an area $bdx$ at a particular distance $x$ from neutral axis that is,

$$C = \int_{0}^{k} f_{m}bdx \quad (4-2)$$

As shown in Figure 4-3, the compressive stress in masonry at a particular strain can be expressed as:
\[
\frac{f_m'}{f_m''} = \left(\frac{\varepsilon_m}{\varepsilon_m'}\right)^2 + 2\left(\frac{\varepsilon_m}{\varepsilon_m'}\right)
\]  \tag{4-3}

where \(f_m'\) and \(\varepsilon_m'\) are the peak compressive stress and corresponding strain.

Substituting the value of \(f_m'\) from Equation (4-3) into (4-2) gives,

\[
C = b \int_{0}^{kd} \left(\frac{1}{\varepsilon_m} \right)^2 \left(\frac{\varepsilon_m}{\varepsilon_m'}\right)^2 + 2\left(\frac{\varepsilon_m}{\varepsilon_m'}\right) dx
\]  \tag{4-4}

Further, from linear strain distribution across the section;

\[
\varepsilon_m = \varepsilon \frac{x}{kd}
\]  \tag{4-5}

Hence, substituting Equation (4-5) into (4-4) and simplifying;

\[
C = b f_m' \left[ -\left(\frac{\varepsilon_m}{\varepsilon_m'}\right)^2 \left(\frac{1}{kd}\right)^2 \left(\frac{x^3}{3}\right) + 2\left(\frac{\varepsilon_m}{\varepsilon_m'}\right) \left(\frac{1}{kd}\right)^2 \left(\frac{x^2}{2}\right) \right]_{0}^{kd}
\]

\[
= b f_m' \left[ \left(\frac{\varepsilon_m}{\varepsilon_m'}\right) \left(\frac{1}{kd}\right)^2 \left(\frac{x^3}{3}\right) + 2\left(\frac{\varepsilon_m}{\varepsilon_m'}\right) \left(\frac{1}{kd}\right)^2 \left(\frac{x^2}{2}\right) \right]
\]

\[
C = b f_m' \left(\frac{\varepsilon_m}{\varepsilon_m'}\right) \left[ 1 - \frac{1}{3} \left(\frac{\varepsilon_m}{\varepsilon_m'}\right) \right]
\]  \tag{4-6}

Since peak compressive strain \(\varepsilon_m' = 0.003\) (see Figure 4-3) and ultimate strain \(\varepsilon_{mu} = 0.0035\) (Triantafiliou 1998).

\[
\frac{\varepsilon_{mu}}{\varepsilon_m'} = 1.17
\]  \tag{4-7}

Hence, Equation (4-6) gives in view of Equation (4-7)
The Tensile force provided by strengthening system can be obtained as:

\[ T = f_{tu} A_t \]  \hspace{1cm} (4-9)

The moment for a balanced section is therefore,

\[ M_{u, bal} = \frac{kd}{0} \int_{m} bdx + T(1-k)d \]  \hspace{1cm} (4-10)

By substituting Equation (4-3) into (4-10)

\[ M_{u, bal} = b \int_{m} f' \left( \frac{\varepsilon_m}{\varepsilon_m'} \right)^2 + 2 \left( \frac{\varepsilon_m}{\varepsilon_m'} \right) \left( \frac{e_{mu}}{kd} - x \right) dx + T(1-k)d \]  \hspace{1cm} (4-11)

Also, substituting Equation (4-5) into (4-11) gives

\[ M_{u, bal} = bf' \int_{m} b \left( \frac{1}{\varepsilon_m'} \right)^2 \left( \frac{e_{mu}}{\varepsilon_m'} \right)^2 + 2x \left( \frac{e_{mu}}{\varepsilon_m'} \right) \left( \frac{e_{mu}}{kd} - x \right) dx + T(1-k)d \]  \hspace{1cm} (4-12)

Further, substituting from Equation (4-7) to (4-12) and simplifying;

\[ M_{u, bal} = bf' \left[ - \left( \frac{e_{mu}}{\varepsilon_m'} \right)^2 \left( \frac{1}{kd} \right)^2 \left( \frac{x}{4} \right)^2 + 2 \left( \frac{e_{mu}}{\varepsilon_m'} \right) \left( \frac{1}{kd} \right) \left( \frac{x^3}{3} \right) \right]_{0}^{kd} + T(1-k)d \]

\[ = bf' \left[ - \left( \frac{e_{mu}}{\varepsilon_m'} \right)^2 \left( \frac{(kd)^2}{4} \right) + 2 \left( \frac{e_{mu}}{\varepsilon_m'} \right) \left( \frac{(kd)^2}{3} \right) \right] + T(1-k)d \]

\[ M_{u, bal} = 0.44b(kd)^2 f'_m + T(1-k)d \]  \hspace{1cm} (4-13)

Since equilibrium condition gives \( T = C \); and \( C = 0.714 k f'_m b d \) Equation (4-8) gives
\[ M_{u,\text{bal}} = 0.44b(kd)^2 f'_m + 0.714bkdf'_m (1-k)d \]
\[ = \left[ 0.44k^2 + 0.714k(1-k) \right] f'_m bd^2 \]

\[ M_{u,\text{bal}} = \left[ 0.714 - 0.274k \right] f'_m bd^2 \quad (4-14) \]

Substituting the value of \( k \) from Equation (4-1) into (4-14), the ultimate moment for a balanced section is given as;

\[ M_{u,\text{bal}} = \left[ 0.714 - 0.274k \right] f'_m bd^2 \]

\[ (4-15) \]

From equilibrium condition \( C = T \),

\[ 0.714k f'_m bd = f'_{tu} A_t \quad (4-16) \]

Substituting the value of \( k \) from Equation (4-1) in (4-16)

\[ 0.714 \left( \frac{\varepsilon_{mu}}{\varepsilon_{mu} + \varepsilon_{tu}} \right) f'_m bd = f'_{tu} A_t \]

\[ (4-17) \]

The tensile capacity of the strengthening system that will lead to a balanced failure of the strengthened wall is therefore given by;

\[ \eta_{\text{bal}} = \left( \frac{A_t}{bd} \right)_{\text{bal}} = 0.714 f'_m \left( \frac{\varepsilon_{mu}}{\varepsilon_{mu} + \varepsilon_{tu}} \right) \]

\[ (4-18) \]

Depending on the actual tensile capacity of the strengthening system, the wall can fail in masonry crushing or reinforcement rupture. If the tensile capacity ratio \( \eta \), defined as \( A_t f'_{tu} / bd \), is greater than the balanced value; \( \eta_{\text{bal}} \), then failure would be by
masonry crushing; otherwise, it would be by rupture of reinforcement in the strengthening system.

(b) Flexural compressive failure \((\eta > \eta_{bal})\)

In this case, the stress and strain distributions across the section are as shown in Figure 4-4. The maximum compressive strain in the masonry is \(\varepsilon_{mu} = 0.0035\). The compressive force carried by masonry can be obtained from Equation (4-8).

\[
C = 0.714 k_f m b d
\quad (4-8)
\]

The strain in strengthening system \((\varepsilon_t)\) can be obtained from the strain compatibility:

\[
\varepsilon_t = \frac{(1 - k)}{k} \varepsilon_{mu} < \varepsilon_{tu}
\quad (4-19)
\]

The tensile stress \((f_t)\) corresponding to \(\varepsilon_t\) can be obtained from the proposed simplified material model; (see section 3.3.2.5), that is,

\[
\text{if } 0 < \varepsilon_t \leq \varepsilon_b, \quad f_t = E_t \varepsilon_t \quad \text{where } E = \frac{f_b}{\varepsilon_b}
\quad (4-20)
\]

\[
\text{if } \varepsilon_b < \varepsilon_t < \varepsilon_c, \quad f_t = f_b + E_2 (\varepsilon_t - \varepsilon_b) \quad \text{where } E_2 = \frac{f_c - f_b}{(\varepsilon_c - \varepsilon_b)}
\quad (4-21)
\]

\[
\text{if } \varepsilon_c < \varepsilon_t < \varepsilon_d, \quad f_t = f_c + E_3 (\varepsilon_t - \varepsilon_c) \quad \text{where } E_3 = \frac{f_d - f_c}{(\varepsilon_d - \varepsilon_c)}
\quad (4-22)
\]

The tensile force provided by the strengthening system can be obtained by:

\[
T = f_t A_t
\quad (4-23)
\]

where \(A_t\) is the cross-sectional area of the strengthening system.
To obtain the value of $k$, assume a value ($<1$) and obtain $\epsilon_t$ from Equation (4-19). Depending on the value of $\epsilon_t$, the force equilibrium condition ($C = T$) can be checked by one of the following equations.

If $0 < \epsilon_t \leq \epsilon_b$,

$$A t \frac{E \epsilon}{1} \frac{m u}{k} \left(1 - \frac{k - 1}{k}\right) = 0.714 k f' b d \quad (4-24)$$

If $\epsilon_b < \epsilon_t < \epsilon_c$

$$A t \left[f_b + E_2 \left(\frac{\epsilon}{m u} \left(1 - \frac{k - 1}{k}\right) - \epsilon_b\right)\right] = 0.714 k f' b d \quad (4-25)$$

If $\epsilon_c < \epsilon_t < \epsilon_d$

$$A t \left[f_c + E_3 \left(\frac{\epsilon}{m u} \left(1 - \frac{k - 1}{k}\right) - \epsilon_c\right)\right] = 0.714 k f' b d \quad (4-26)$$

These steps are repeated until the force equilibrium is satisfied.

The ultimate bending moment capacity of the section under flexural compressive failure is then calculated as:

$$M_u = \int_0^{kd} f_b x dx + T(1 - k)d \quad (4-27)$$

Substituting Equation (4-3) into (4-27)

$$M_u = b \int_0^{kd} f' \left(-x \left(\frac{\epsilon}{m e'}\right)^2 + 2x \left(\frac{\epsilon}{m e'}\right)\right) dx + T(1 - k)d \quad (4-28)$$

Also, substituting Equation (4-5) to (4-28) gives

$$M_u = b f' \int_0^{kd} \left[-x \left(\frac{\epsilon}{m e'}\right)^2 + 2x \left(\frac{\epsilon}{m e'}\right)\right] dx + T(1 - k)d \quad (4-29)$$

Further, simplifying and substituting Equation (4-7) into (4-29);


\[ M_u = b f'_m \left[ \frac{e_{mu}}{e'_m} \right]^2 \left( \frac{1}{kd} \right)^2 \left( \frac{1}{4} \right) x^4 + 2 \left( \frac{e_{mu}}{e'_m} \right) \left( \frac{1}{kd} \right) \left( \frac{x^3}{3} \right) \right]_{0}^{k} + T(1-k)d \]

\[ = b f'_m \left[ \frac{e_{mu}}{e'_m} \right]^2 \left( \frac{(kd)^2}{4} \right) + 2 \left( \frac{e_{mu}}{e'_m} \right) \left( \frac{(kd)^2}{3} \right) + T(1-k)d \]

\[ M_u = 0.44b(kd)^2 f'_m + T(1-k)d \]

Since under equilibrium condition; \( T = C \) and from Equation (4-8), \( C = 0.714kf'_m b d \), therefore

\[ M_u = 0.44b(kd)^2 f'_m + 0.714bkd f'_m (1-k)d \]

\[ = \left[ 0.44k^2 + 0.714k(1-k) \right] f'_m b d^2 \]

Thus, the ultimate bending moment capacity of the section under flexural compressive failure;

\[ M_u = 0.714 \cdot 0.274k f'_m b d^2 \]

(c) **Flexural tensile rupture of Reinforcement in TRM system** \((\eta < \eta_{bal})\)

This section evaluates the flexural capacity of the strengthened wall if the wall fails by rupture of TRM overlay before the crushing of masonry (Figure 4-5). This failure occurs due to a lower tensile capacity of the strengthening system. The moment capacity of the strengthened wall can also be derived in the same manner as for the flexural compression failure discussed in the previous section. The difference is that in the case of flexural tensile failure, the TRM system has reached to its ultimate rupture tensile strain prior to masonry crushing. In all three strengthened TRM systems, point D represents this failure stage.

The tensile force provided by strengthening system can be obtained by:
\[ T = f_{tu} A \]  \hspace{1cm} (4-32)

From the strain compatibility condition;

\[ \varepsilon_m = \frac{\varepsilon_{tu}}{1 - k} x \]  \hspace{1cm} (4-33)

The compressive force acting on the masonry wall is

\[ C = \int_0^{kd} b dx \]  \hspace{1cm} (4-34)

Substituting Equation (4-3) into (4-34) gives

\[ C = b \int_0^{kd} f' m \left(-\frac{\varepsilon_m}{\varepsilon_m'} + 2 \frac{\varepsilon_m}{\varepsilon_m'} \right) dx \]  \hspace{1cm} (4-35)

Further, substituting from Equation (4-33) into (4-35) and simplifying

\[ C = b f' m \left[-\left(\frac{1}{\varepsilon_m'} \right)^2 \left(\frac{\varepsilon_{tu}}{(1-k)d} x \right)^2 + 2 \left(\frac{1}{\varepsilon_m'} \right) \left(\frac{\varepsilon_{tu}}{(1-k)d} x \right) \right]^{kd}_{0} \]

\[ = b f' m \left[-\left(\frac{\varepsilon_{tu}}{\varepsilon_m'} \right)^2 \left(\frac{k^3 d}{3(1-k)^2} \right) + 2 \left(\frac{\varepsilon_{tu}}{\varepsilon_m'} \right) \left(\frac{k^2 d}{2(1-k)} \right) \right] \]

which gives

\[ C = b df' m \left(\frac{\varepsilon_{tu}}{\varepsilon_m'} \right)^2 \left[ -\left(\frac{k^3}{3(1-k)^2} \right) + 2 \left(\frac{k^2}{2(1-k)} \right) \right] \]  \hspace{1cm} (4-36)

From equilibrium condition \( C = T \), and \( T = f_{tu} A \),

\[ f_{tu} A = b df' m \left(\frac{\varepsilon_{tu}}{\varepsilon_m'} \right)^2 \left[ -\left(\frac{k^3}{3(1-k)^2} \right) + 2 \left(\frac{k^2}{2(1-k)} \right) \right] \]  \hspace{1cm} (4-37)

from which the neutral axis depth factor \( k \) can be solved by trial and error.
The ultimate moment capacity of the section under flexural tensile failure is:

\[ M_u = \frac{kd}{b} \int_{0}^{f_{m}} \frac{bxdx}{\mu M} + T (1 - k)d \]  \hspace{1cm} (4-38)

Substituting Equation (4-3) into (4-38) gives

\[ M_u = b \frac{kd}{f_{mu}} \left( -x \left( \frac{e_{m}}{e_{m}'} \right)^2 \left( \frac{e_{tu}}{1 - k} \right)^2 + 2x \left( \frac{e_{m}}{e_{m}'} \right)^2 \left( \frac{e_{tu}}{1 - k} \right) x \right) dx + T (1 - k)d \]  \hspace{1cm} (4-39)

Then, substituting Equation (4-33) into (4-39) and simplifying

\[ M_u = b f'_{m} \left[ -x \left( \frac{1}{e_{m}'} \right)^2 \left( \frac{e_{tu}}{1 - k} \right)^2 \left( \frac{k^4}{4(1 - k)^2} \right) + 2x \left( \frac{e_{m}}{e_{m}'} \right)^2 \left( \frac{k^3 d^2}{3(1 - k)} \right) \right]_0^{kd} + T (1 - k)d \]  \hspace{1cm} (4-40)

Thus, the ultimate bending moment capacity of the section under flexural tensile failure is

\[ M_u = b d f' \left[ -x \left( \frac{e_{tu}}{e_{m}'} \right)^2 \left( \frac{k^4}{4(1 - k)^2} \right) + 2x \left( \frac{e_{m}}{e_{m}'} \right)^2 \left( \frac{k^3 d^2}{3(1 - k)} \right) \right] + f_{tu} A t (1 - k)d \]  \hspace{1cm} (4-41)

It can be shown that Equations (4-31) and (4-41) give the same value when k equals to the value that corresponding to balanced failure ( i.e. Equation (4-1)) (See Appendix A)

### 4.2.2 Application to TRM strengthened walls

As summarized in Table 4-1, Series I specimens strengthened with PP-band reinforced mortar system had tensile strengths greater than that for a balanced failure.
Therefore, it is predicted that all walls strengthened with PP-band reinforced mortar system would fail by flexural compression. The flexural moment of resistance is given by Equation 4-31.

In the case of ferrocement strengthened wall specimens, the tensile strength ratios ($\eta$) of the ferrocement overlay were lesser than the corresponding balanced values ($\eta_{bal}$) as indicated in Table 4-1. Therefore, the model predicts that the ferrocement strengthened specimens fail by flexural tensile failure, and therefore, the resisting moment capacity can be obtained from equation 4-41.

Among the AR-fibreglass TRM strengthened specimens only specimens with six layer of AR-fibreglass TRM layer, in tested with continuous mortar joints parallel to the loading span was predicted to fail by crushing. The other specimens were predicted to fail by flexural tension.

From the obtained ultimate moment capacity, the ultimate load-carrying capacity can be calculated from

$$M_u = \frac{P_u}{2} \alpha L$$  \hspace{1cm} (4-42)

where $P_u$ is the total load applied on the beam and $\alpha L$ is the distance between support and the adjacent loading point (see Figure 4-1).

### 4.2.3 Summary

For the prediction, compressive strengths of masonry in longitudinal and transverse direction are used as 11.5 MPa and 17.8 MPa respectively which have been experimentally investigated and also ultimate compressive strain for both cases is
taken as 0.0035. The tensile properties were taken from proposed simplified linear stress-strain relationship resulted from dog-bone tensile testing.

The theoretical ultimate load capacities predictions of strengthened wall obtained by using the proposed simplified model have been summarized Table 4-1. It can be seen that ultimate load capacities of PP band strengthened walls have not been significantly increased with the reinforcement ratio while that of the other strengthened walls have been considerably increased with the amount of reinforcement ratio. The reason for this is that the failure of all PP strengthened walls is governed by compressive failure of the masonry.
### Table 4-1: Theoretical predictions of ultimate load capacity for strengthened wall

<table>
<thead>
<tr>
<th>Series</th>
<th>Strengthening System</th>
<th>Specimen</th>
<th>Reinforcement ratio (%)</th>
<th>$\eta_{bal}$</th>
<th>$\eta = \frac{f_{tu}A}{bd}$</th>
<th>Flexural Failure Mode*</th>
<th>Theoretical ultimate load capacity (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>PP-band reinforced mortar</td>
<td>PL1</td>
<td>0.36</td>
<td>0.06</td>
<td>0.38</td>
<td>FC</td>
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<td></td>
<td></td>
<td>PL2</td>
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<td>0.06</td>
<td>0.73</td>
<td>FC</td>
<td>63.4</td>
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<td></td>
<td></td>
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<td>0.06</td>
<td>1.28</td>
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</tr>
<tr>
<td></td>
<td></td>
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<td>0.1</td>
<td>0.4</td>
<td>FC</td>
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<td></td>
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<td>0.1</td>
<td>0.77</td>
<td>FC</td>
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</tr>
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<td></td>
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<td>0.1</td>
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<td>AR-fibreglass reinforced mortar</td>
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<td>1.53</td>
<td>1.29</td>
<td>FR</td>
<td>212.8</td>
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</table>

*FC: flexural compression; FR: flexural rupture of reinforcement
(a) Specimens (L) loaded with continuous joints parallel to span
(b) Specimens (T) loaded with continuous joints perpendicular to

**Fig. 4-1: Two main groups of walls specimens**

(a) Crushing of masonry in compression
(b) Rupture of reinforcement in tension in TRM

**Fig. 4-2: Flexural failure type of strengthened walls**
Fig. 4-3: Stress and strain distribution across the wall section – flexural balanced failure

Fig. 4-4: Stress and strain distribution across the wall section - flexural compression failure

Fig. 4-5: Stress and strain distribution across the wall section - flexural tensile failure
Test program on TRM strengthened masonry walls

5.1 GENERAL

This chapter describes the experimental investigation of the capability of proposed strengthened systems to enhance the out-of-plane strength of URM walls. To proceed with this investigation, the test program was performed with strengthened wall specimens and unstrengthened wall specimens. The test variables considered here were loading direction, type of TRM strengthening systems, and reinforcement amount in TRM systems. The chapter further discusses the test results on the static out-of-plane behavior of TRM strengthened masonry walls. The results are compared with analytical predictions using the simplified model previously discussed.

5.2 TEST SERIES

A total of 24 wall specimens were fabricated. Out of these specimens, 22 were strengthened while two were kept as control specimens. Strengthened specimens were divided into two series of 6 specimens and one series of 10 specimens, according to the type of TRM strengthening systems (designated by P, F, and T in the first prefix, for PP band-reinforced mortar, ferrocement, and AR-fibreglass textile reinforced mortar, respectively), as shown in Table 5-1. Within each series, half of the specimens were tested with the continuous mortar joints parallel to the loading span (denoted by L in the second prefix) while the other half were tested with the mortar joints perpendicular to the loading span (denoted by T).
In Series I specimens, wall specimens were strengthened with PP band reinforced mortar systems. In this series, three reinforcement ratios were used in loading directions; that is, 0.36% (1-layer mesh), 0.69% (2-layer mesh) and 1.16% (3-layer mesh) in Specimens PL1/PT1, PL2/PT2 and PL3/PT3 respectively. Series II specimens were strengthened with ferrocement system and three reinforcement ratios were also used for each loading direction, that is 0.07% (1-layer of mesh), 0.14% (2-layer mesh) and 0.19% (3-layer mesh) in Specimens FL1/FT1, FL2/FT2 and FL3/FT3 respectively. Series III specimens were strengthened with AR-fibreglass textile reinforced mortar system and this series consisted of five reinforcement ratios in each loading directions, that is 0.06% (1-layer mesh), 0.11% (2-layer mesh), 0.18% (3-layer mesh), 0.24% (4-layer mesh) and 0.33% (6-layer mesh) in Specimens of TL1/TT1, TL2/TT2, TL3/TT3, TL4/TT4 and TL6/TT6 respectively.

5.3 Fabrication of Wall Specimens

The wall specimens were fabricated using bricks with a size of 210 mm x 100 mm x 75 mm as shown in Figure 5-1. The bricks had an average compressive strength of 30 MPa, determined according to BSEN772-1-2000. The average compressive strength of mortar was 28 MPa.

Group L and T walls were fabricated vertically as in normal practice by skilled mason. Bricks were soaked in the water for 24 hours and kept for some time to ensure saturated condition so as to prevent the water absorption from the mortar and to maintain workability of the mortar. The mortar was hand mixed. After casting of the walls, plastering was done on the next day. Before and after plastering, walls were covered with plastic sheets to prevent water evaporation and early age cracking. The
walls were plastered on both sides to simulate actual condition prior to the installation of the strengthening system.

The walls were strengthened 7 days after they were constructed. The strengthening work was done with walls laid horizontally. To measure strain of the meshes during testing, four strain gauges were installed in each layer of mesh before the mortar was cast. Different types of tensile strain gauges were used for different reinforcement mesh depending on their strain capacity and dimensions. For the PP band mesh, 5mm width post-yield strain gauges with a measuring strain capability of about 15% were used. For the wire mesh and AR-fibreglass textile mesh, 2mm width steel strain gauges with a measuring strain capability of about 2% were used.

5.4 TEST SET-UP AND INSTRUMENTATION

The strengthened masonry walls were subjected to four point-bending as shown in Figure 5-2. LVDTs were placed under the two loading points and at mid-span to measure the displacements. As shown in Figure 5-3, the strains in the top extreme compressive fibers and in the reinforcement mesh at mid-span were measured using strain gauges. To measure the curvature, an in-house designed aluminum frame was mounted at mid-span, with two LVDTs each at the top and bottom, one in front and the other at the back of the specimen.

The load was applied by means of a hydraulic jack with displacement control at a loading rate of 0.05 mm/min until failure of the wall. For the post-cracking regime of the PP-band strengthened walls, a rate of 2mm/min was applied. All instrument readings were recorded using a data acquisition system.
5.5  TEST RESULTS AND DISCUSSION

5.5.1  Load-deflection characteristics

The load-deflection characteristics under four-point bending are shown in Figure 5-4 (a), Figure 5-5 (a) and Figure 5-6 (a) respectively for walls strengthened with PP-band reinforced mortar, ferrocement and AR-fibreglass TRM systems. The shape of the curves depends on the type of strengthening system. In all cases, the applied load increased linearly with deflections initially. For a particular strengthening system, the specimens exhibited similar elastic stiffness regardless of the reinforcement ratio and the loading direction. Series I and III specimens showed large deflections with considerable post cracking load capacity while Series II shows large load capacity with small ultimate deformations. Further details on the load-deflection characteristics of each series will be discussed separately below.

Series I: Walls strengthened with PP band reinforced mortar system

The load-deflection characteristics of PP-band reinforced mortar strengthened walls are shown in Figure 5-4 (a). This series includes six strengthened specimens with varying reinforcement ratios in each loading arrangement, i.e. whether the specimens were tested with the continuous mortar joints parallel or perpendicular to the loading span.

The elastic stiffness values of the six specimens were similar (see Figure 5-4(a)). Also, the ultimate load capacities of walls strengthened with the same reinforcement ratio are approximately the same in either loading direction. The applied load has significantly dropped after the peak value was attained. However, this drop in load was improved with the increase in reinforcement ratio. From this point onwards, the load again increased at a slower rate and then remained more or less the same load.
until the final failure. The load contribution from the PP-band is more significant at large deflections.

As can be seen from the Figure 5-4 (a), the load-deflection curves showed several drops in load-carrying capacity in the post-crack regime, particularly in the case of walls strengthened with two or three PP-band mesh layers and the number of drops increased with the reinforcement ratio. These correspond to the formation of multiple cracks throughout the wall slabs as can be seen from Figure 5-4 (b). The PP-band reinforced wall specimens showed large deformations of about 45~60 mm (see Figure 5-4 (b)).

**Series II: Walls strengthened with ferrocement system**

Figure 5-5(a) shows load-deflection characteristics of ferrocement strengthened wall specimens. This series includes six specimens with three reinforcement ratios tested in both transverse and longitudinal directions.

All six specimens followed the same shape in their load-deflection characteristics. Like Series I, the elastic stiffness of all ferrocement strengthened specimens were similar. The ferrocement strengthened specimens shows high ultimate load-carrying capacity which increased considerably (60 kN/m -160 kN/m) with an increase in the amount of reinforcement. Furthermore, specimens show small deflections of about 2~4 mm and they suddenly failed.

**Series III: Walls strengthened with AR-fibreglass textile reinforced mortar system**

Load-deflection characteristics of AR-fibreglass textile reinforced mortar strengthened walls are shown in Figure 5-6 (a). This series consisted of five
reinforcement ratios, that is, 1,2,3,4 and 6 numbers of layers of AR-fibreglass textile. For all these ratios, walls were tested in both loading directions.

The shape of the load-deflection curves are the same for all reinforcement ratios. Similar to Series I, this series also exhibited a post-crack regime. After the load has dropped which resulted from wall cracking, the AR-fiberglass textile was able to increase the load at a slower rate until rupture of textile. Similar to Series I, the drop in load reduced with an increase in the amount of reinforcement.

The elastic stiffness values of the all specimens in Series III were similar. The peak load capacities of the walls were recorded in the pre-crack regime for specimens strengthened with up to four layers of AR-fibreglass textile. However, wall specimens strengthened with six layers of textile showed their peak load in the post-crack regime. AR-fibreglass TRM strengthened wall specimens were also able to show load capacities as high as ferrocement strengthened wall specimens, particularly with larger number of textile layers. For instance, walls strengthened with six layers of AR-fibreglass textiles showed higher load capacity than that of walls strengthened with ferrocement having three layers of wire meshes. Moreover, Series III wall specimens showed about 4~10 mm displacements.

### 5.5.2 Ultimate load and energy absorption capacity

As shown in Figure 5-7, the ultimate moment of resistance of TRM strengthened specimens significantly and linearly increased with the increase in tensile capacity of strengthening systems in the case of ferrocement and AR-fibreglass TRM. In the case of PP-band reinforced mortar, the increment in ultimate moment of resistance was not significantly improved with the corresponding tensile capacity of the strengthening system.
The energy absorption capacity is derived as the area under the load-deflection curve up to the final failure. As can be seen from the Figure 5-8, wall specimens strengthened with PP-band reinforced mortar system showed largest energy absorption capacity. Furthermore, the energy absorption capacities of wall strengthened with the same type of TRM system having same amount of reinforcement, were approximately the same in each loading direction.

5.5.3 Strain development

The load-strain relations in masonry (compressive strain) and in mesh (tensile strain) are plotted in Figure 5-9 (a) and (b), Figure 5-9 (c) and (d) and Figure 5-9 (e) and (f) for Series I, II and III respectively. Strain was measured using the strain gauges and a curvature measuring device which were located in the pure moment zone. None of the tensile strain measurements (about 10%) of wall specimens in Series I (PP-band reinforced mortar) reached the ultimate tensile strain PP-band (i.e. \( \approx 30\% \)). On the other hand, some of the compressive strains (i.e in PT1 and PT3) have reached the ultimate compressive strain of masonry (i.e 0.0035). The major crack of the other four specimens in Series I formed outside the 30-mm gauge length of the compressive strain gauges. Therefore, the maximum compressive strain in these specimens at the failure section was not captured accurately. However, the smaller PP-band strain than its ultimate value proved that specimens had failed in flexural compression.

Figure 5-9 (c) and (d) show that none of the ferrocement strengthened wall specimens has reached the ultimate compressive strain of masonry. The tensile strain measurement obtained from the curvature measuring device showed that wall specimens strengthened with two or three layer of reinforcement in either Specimens L group or Specimens T group (i.e. FL2, FT2, FL3 and FT3), have reached to a strain of
over 0.5% in the welded wire mesh. Therefore, it was clear that those specimens failed in flexural tension. Wall specimens strengthened with single layer of ferrocement (FL1 and FT1) were not able to capture the strain measurement properly from either strain gauge or curvature measuring device because their failure crack line was formed outside the gauge length of them.

Figure 5-9 (e) and (f) show the load-strain relations of AR-fibreglass TRM strengthened specimens. In this series, five reinforcement ratios were used. None of compressive strain measurements of walls strengthened with AR-fiberglass TRM was reached to the ultimate strain of masonry. However, all tensile strain measurement of Series III wall specimens except TL1 have reached to ultimate tensile strain of AR-fibreglass TRM (i.e. \( \approx 2 \% \)) which indicated that they failed in flexural tension. The critical crack of TL1 formed out-of the gauge length of curvature measuring device and location of strain gauges.

### 5.5.4 Failure characteristics

The TRM strengthened wall specimens were tested under four-point bending. The failure was by a flexural compression, in all PP-band reinforced mortar strengthened specimens and by flexural tensile failure of reinforcement in all other specimens. The type of failure depends mainly on tensile capacity of the strengthening layer and hence on the type, area and elastic stiffness of reinforcement.

All PP-band reinforced mortar strengthened walls failed in flexural compression because it was observed that the top of the specimens were crushed but the PP-band remained intact without breaking as shown in Figure 5-4(a). These observations further confirmed that PP-band reinforced mortar strengthened specimens failed in flexural compression. Since PP-band has a very low stiffness, it has to be
largely elongated to provide its ultimate tensile capacity. However, due to over-
reinforced condition, failure resulted before PP-band could elongate adequately. As a
result, the load capacity of the walls has not been significantly increased with the
increase in the amount of reinforcement.

All specimens strengthened with ferrocement failed in flexural tension due to
rupture of reinforcement. As can be seen from the Figure 5-5 (b), all ferrocement
strengthened specimens were totally and suddenly broken into two pieces unlike PP-
band reinforced mortar strengthened walls. The steel reinforcement in ferrocement
system ruptured prior to masonry crushing as a result of under- reinforcement.
However, this system was able to increase the load capacity of the wall considerably
without any intermediate drop in load. The reason for is that the ferrocement was able
to control crack opening while maintaining a constant wall stiffness until the wall had
failed.

Specimens strengthened with AR-fibreglass TRM system also failed by
flexural tensile failure due to rupture of reinforcement. All specimens failed in similar
manner as ferrocement strengthened specimens. However, as can be seen from the
load-deflection curves, there were drops in the applied load before final failure. This
was due to large crack opening. Since AR-fibreglass TRM strengthening layer has
lower stiffness than ferrocement for a particular number of reinforcement layers, the
capability in crack control was lesser than ferrocement, which leads to the drop in load.
However, it can be seen from Figure 5-6 (a), the drop in load was reduced with the
increase in the amount of reinforcement. This is because a large amount of
reinforcement can control the sudden opening of cracks better. Furthermore, as shown
Figure 5-6 (c) in specimens TL6 and TT6, several small drops in load which resulted
from multiple cracking can be seen in both masonry and the strengthening layer. Multiple cracking leads to stiffer load-deflection response in post-crack regime with higher ultimate load and larger deflections.

5.6 COMPARISON BETWEEN TEST RESULTS AND THEORETICAL PREDICTIONS

Using the proposed model described in Chapter 4, the predicted ultimate load capacity and failure mode of strengthened walls were obtained and summarized in Table 5-3 with test results.

From Table 5-3, in, the predicted ultimate load capacity and failure modes for Series I specimens (PP-band reinforced mortar strengthened walls) agree with test results well. As introduced in Section 4.2.1.1 (a), the balanced tensile capacity of strengthened wall \( \eta_{bal} \), which can be calculated from Equation (4-18), have been used to gauge the failure mode of the strengthened walls. As summarized in Table 4-1, the failure modes of all PP-band reinforced mortar strengthened walls are predicted to fail by flexural compression since \( \eta \) is larger than \( \eta_{bal} \). This means that the available tensile capacity of the strengthening layer was greater than that for a balanced failure, which results in masonry crushing before reinforcement in TRM system ruptures. In the tests (see Section 6.2.4), it was observed that specimens in Series I have crushed while PP-band reinforcement remained unbroken, which confirmed that all specimens in Series I failed by flexural compression. The observed ultimate load capacities differed from the predicted load capacities by about ± 3-14 %.

As can be seen in Table 4-1, in contrast to PP-band reinforced mortar strengthened walls, the tensile capacities \( \eta \) of all ferrocement strengthened walls...
were lesser than that for a balanced failure ($\eta_{bal}$). Therefore, all ferrocement strengthened walls were predicted to be failed in flexural tension. As described in Section 5.5.3, tensile strain results of Series II specimens except FL1 and FT1 also confirmed that all observed ferrocement walls failed in flexural tension. As shown in Table 5-3, the observed ultimate load capacities of ferrocement strengthened walls differed from the predicted load capacities by about ±1-6%.

In AR-fibreglass TRM strengthened walls, the difference between the tensile capacities ($\eta$) of walls and that for a balanced failure ($\eta_{bal}$) are not as large as that in Series I and Series II as shown in Table 4-1.

However, except TL6, all other specimens have tensile strength less than that for a balanced failure. Therefore, according to predictions, except TL6, the others should fail in flexural tension and TL6 should fail in flexural compression. As described in section 5.5.3, the tensile strain results were able to confirm the failure mode of Series III specimens except TL1 which failed in flexural tension. However, from observed failure pattern, the failure modes of all specimens were established as flexural tension. Therefore, except specimen TL6, the observed failure modes agreed with predicted failure modes. As shown in Table 5-3, the observed ultimate strength of AR-fibreglass TRM strengthened walls differed from the predicted load capacities by about ±1-8%.

### 5.7 EFFECTS OF TEST PARAMETERS

In this study, the test specimens involved three test parameters; loading direction, type of TRM strengthening systems and reinforcement amount. The effect of these parameters on the characteristics of strengthened walls is discussed below.
5.7.1 Loading direction

To account for orthotropic behavior of masonry wall, loading was applied with the span of the specimens perpendicular and parallel to the bed joints. Figure 5-7 shows the load-deflection characteristics of URM wall under four-point bending with loading span of the specimens parallel (L0) and perpendicular (T0) to bed joints. The ultimate load capacities of L0 and T0 were 22 kN/m and 13.3kN/m respectively. Specimen L0 carried about 65% higher load capacity than T0. Although such a variation in ultimate load capacity can be observed in the control specimens, in wall specimens strengthened with PP-band reinforced mortar and ferrocement, the difference between load capacities in two directions was only about ±6 % while in the case of AR-fibreglass TRM system it was about ±11%. Therefore, loading direction has negligible effect on the ultimate load capacities of strengthened walls. The stiffness of load deflection curves is observed to be same for a particular strengthening system for both loading directions.

5.7.2 Type of TRM strengthening system

In this study, URM walls were strengthened with three types of TRM systems; PP-band reinforced mortar, ferrocement and AR-fibreglass TRM system. First, the failure modes of strengthened walls mainly depended on the type of reinforcement. For instance, all PP-band reinforced mortar strengthened walls failed in flexural compression while walls strengthened with the other types of reinforcement walls failed by flexural tension (see Table 6-1). Furthermore, Table 6-1 shows that for a given number of layers and for a given loading direction, the highest ultimate load carrying capacity is given by walls strengthened with ferrocement followed by AR-fibreglass TRM system and then PP-band reinforced mortar. However, as can be seen
from Figure 3-11 (a), uni-axial tensile tests showed that the PP-band reinforced mortar has the highest ultimate tensile capacity for a given number of layers. On the other hand, PP-band reinforced mortar showed the lowest elastic stiffness with a large strain capacity while ferrocement showed the largest elastic stiffness with a low strain capacity. This largest elastic stiffness of ferrocement led to the highest load capacity with lower deflection in the ferrocement strengthened walls. As shown in Figure 6-2 (a), the large stiffness of ferrocement strengthening system helps to control cracking and maintain a constant wall stiffness until failure. Due to the low stiffness, PP-band reinforced mortar was not able to provide significant tensile strength to the wall. As can be seen in Figure 3-11 (b), the stiffness of AR-fibreglass TRM strengthening system was lower than that of ferrocement but much higher than that of PP-band reinforced mortar system. Therefore, with the increase in number of AR-fibreglass TRM layers, that is, six layers, the ultimate load capacity significantly increased while the drop in load reduced in number (see Figure 6-3 (a)).

5.7.3 Reinforcement amount in TRM strengthening system

PP-band was unable to significantly improve the ultimate load capacity of walls (see Figure 6-4) with an increase in the reinforcement ratio compared to other systems. This is because the final failure of the walls was by flexural compression and also PP bands gave lower tensile contribution to the wall at small elongation due to its low stiffness.

In the case of ferrocement strengthened walls, the reinforcement ratio was such that the tensile capacity in the ferrocement system was lower than that for a balanced failure. Therefore, the flexural tensile failure was observed in all ferrocement strengthened walls. It can be clearly seen from Figure 6-2 (a) and Table 6-1, that the
ultimate load capacity, displacement and energy absorption capacity have been significantly increased with an increase in the reinforcement ratio.

Similar to ferrocement strengthened walls, AR-fibreglass TRM strengthened walls also indicated improvement in ultimate load capacity, displacement and energy absorption capacity with an increase in the reinforcement ratio (see Figure 6-3 (a)). Among these specimens, TL6 and TT6 showed a significant increase in the ultimate load, displacement and energy absorption capacity. Apart from TL6, all other specimens had a tensile capacity of the TRM strengthening system lower than that for a balanced section; hence they were predicted to fail in flexural tension. However, from the tests, a change in flexural failure mode with the increase in reinforcement ratio was not observed.

5.8 SUMMARY

The experimental investigation on the out-of plane behavior of TRM strengthened walls was discussed. The test results indicated that the failure occurred in unstrengthened walls by tensile failure of masonry while two different flexural failure modes, that is, flexural tensile failure of strengthening layer and flexural masonry crushing were observed in the TRM strengthened walls. Furthermore, it was shown that ferrocement and AR-fiberglass TRM systems were able to increase the ultimate load capacity, deflection and the energy absorption capacity of the URM walls. Furthermore, out of the test parameters considered in this study, the loading direction showed insignificant effect on the behavior of the walls. Further, it was concluded that type of strengthening systems affect the failure mode and ultimate load capacity of walls. An increase in the reinforcement ratio significantly improved the load-deflection
characteristics of the walls strengthened with ferrocement and AR-fibreglass TRM compared to PP-band reinforced mortar strengthened walls.
### Table 5-1: Details of test specimens

<table>
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<th>Strengthening system</th>
<th>Specimen Designation</th>
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### Table 5-2: Test specimens and failure characteristics

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*FT : tensile failure of masonry  FC: flexural compression; FR: flexural rupture of reinforcement  **: not applicable
Table 5-3: Comparison of test results with theoretical predictions

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<th>Series</th>
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<th>Test Ultimate load (kN/m) $P_{tu}$</th>
<th>Failure Mode*</th>
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FC: flexural compression; FR: flexural rupture
Chapter 5: Test Program on TRM strengthened masonry walls

Fig. 5-1: Plan view of masonry wall specimens (all dimensions in mm)

Fig. 5-2: Wall Test set-up (all dimensions in mm)

Fig. 5-3: Positions of tensile/compressive strain gauges in the walls

Specimens (L) loaded with continuous joints parallel to span
Specimens (T) loaded with continuous joints perpendicular to span

a) Specimens (L) loaded with continuous joints parallel to span

b) Specimens (T) loaded with continuous joints perpendicular to span

$\text{Loading}$

$\text{Curvature measuring device}$

$\text{100mm LVDT}$

$\text{25mm LVDT}$

$\text{320}$

$\text{775}$

$b = \text{width of wall} ; \ l = \text{distance between loading points}$

$b/3$

$b/2$

$b/4$

$b$

$l$

$l/2$
Fig. 5-4(a) : Load-deflection Characteristics of masonry wall strengthened with PP-band reinforced mortar system
Chapter 5: Test Program on TRM strengthened masonry walls

Fig. 5-4(b): Appearance after failure of masonry wall strengthened with PP-band reinforced mortar system
Fig. 5-5 (a) : Load-deflection characteristics of masonry wall strengthened with ferrocement system

Fig. 5-5 (b) : Appearance after failure of Masonry wall strengthened with ferrocement system
Fig. 5-6 (a) Load-deflection characteristics of masonry wall strengthened with AR-fibreglass TRM system

Fig. 5-6 (b) :Appearance after failure of masonry wall strengthened with AR-fibreglass TRM system
Fig. 5-6 (c) : Appearance after failure of masonry wall strengthened with AR-fibreglass TRM system
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Fig. 5-7: Ultimate moment capacity vs. tensile capacity of TRM strengthening system

Fig. 5-8: Energy absorption capacity vs. tensile capacity of TRM strengthening system
Chapter 5: Test Program on TRM strengthened masonry walls

Fig. 5-9 (a): Compressive and tensile Load-strain relations of PP-band mesh strengthened wall (series I- Specimens (PL))
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i. From Strain-Gauge

ii. From Curvature measuring device

Fig. 5-9 (b) : Compressive and tensile Load - strain relations of PP band mesh strengthened wall (series I- Specimens (PT))
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Fig. 5-9 (c) : Compressive and tensile Load - strain relations of Ferrocement strengthened wall (series II- Specimens (FL))
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Fig. 5-9 (d) : Compressive and tensile Load-strain relations of Ferrocement strengthened wall (series II- Specimens (FT))

i. From Strain-Gauge

ii. From Curvature measuring device
i. From Strain-Gauge

ii. From Curvature measuring device

Fig. 5-9 (e): Compressive and tensile Load - strain relations of AR-fibreglass TRM strengthened wall (series III- Specimens (TL))
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i. From Strain-Gauge

ii. From Curvature measuring device

*Note:  C-compressive strain  
T-Tensile strain

Fig. 5-9 (f) :Compressive and tensile Load-strain relations of AR-fibreglass TRM strengthened wall  (series III- Specimens (TT))
Chapter 5: Test Program on TRM strengthened masonry walls

Fig. 5-10: Load–deflection curves and Failure of control specimens
Conclusion

6.1 REVIEW OF WORK

In spite of being popular in building industry, URM walls suffer from poor performance against out of plane loading. Therefore, this project aims at investigating alternative options to strengthen URM walls with textile reinforced mortar (TRM) systems. To achieve this objective, three types of strengthening systems, namely, PP-band reinforced mortar, ferrocement and AR-fibreglass TRM were used. Four-point bending tests were performed on strengthened wall specimens. Since masonry is an orthotropic material, tests were carried out in two orthogonal bending directions, that is parallel and perpendicular to the bed joints. Furthermore, to evaluate the effect of amount of reinforcement, several reinforcement ratios were used in each strengthening system.

A simplified analytical model was proposed to predict the ultimate strength capacity and failure mode of the strengthened masonry walls. This model is based on basic bending theory as applied to reinforced concrete. Material testing was performed on each strengthening layer to obtain the tensile properties.

6.2 CONCLUSIONS

The test results mainly revealed that ferrocement and AR-fibreglass TRM systems were able to improve the ultimate load carrying capacity, energy absorption capacity and displacement of the walls significantly. PP-band reinforced mortar system on the other hand, led to improve energy absorption capacity and larger ultimate
displacement of the walls. These properties were further improved with an increase in
reinforcement ratio of the strengthening systems.

Due to a larger stiffness and tensile strength of steel wire mesh, ferrocement
showed the highest ultimate load capacity for a given tensile capacity of the
strengthening layer. Moreover, ferrocement overlays were further able to provide the
same stiffness to the wall until it reaches the ultimate load without any intermediate
drops in load and with controlled crack width opening. Therefore, it can be concluded
that ferrocement is an effective strengthening system to improve the out-of-plane
behavior of URM walls.

Although AR-fibreglass textile have a stiffness lesser than steel wire mesh,
with a larger number of reinforcement layers (i.e. six layers), AR-fibreglass TRM
strengthened walls showed larger ultimate strength capacity and energy absorption
capacity than ferrocement strengthened walls with three layers of wire mesh. For a
given tensile capacity of the strengthening system, AR-fiberglass TRM system provide
a slightly less ultimate load capacity compared to ferrocement.

PP-band reinforced mortar strengthening system was not effective in enhancing
ultimate load capacity of the walls. PP-band reinforced mortar system led to larger
energy absorption capacity of the strengthened walls due to a larger elongation
capacity.

On the other hand, all PP-band reinforced walls failed in flexural compression.
Even large tensile capacity with large reinforcement ratio did not contribute to increase
the wall strength prior to compressive failure of the walls due to a very low stiffness of
PP band. However, the observed results showed that although specimens failed in
flexural compression, PP band was able to safely contained failed wall specimens without dropping dangerously.

No distinguishable difference was observed in specimens with different loading directions, parallel or perpendicular to the bed joints. Therefore, under flexural loading TRM strengthened walls may be considered as having the same response regardless of loading direction.

These proposed strengthening systems are comparatively cheap particularly compared to FRP systems. Furthermore, materials are easily available and fabrication is simple. Therefore, these proposed strengthening systems are particularly useful for developing countries. Ferrocement and AR-fibreglass TRM can be considered as effective strengthening systems which can be appropriate economical solutions to out-of-plane strengthening of URM walls in the developing countries.

The proposed simplified analytical model agrees with observed ultimate load capacity and failure mode of the strengthened walls reasonably well.

### 6.3 RECOMMENDATIONS FOR FUTURE WORK

The present investigation evaluated the effectiveness of TRM systems in enhancing strength, energy absorption capacity and deformation in one-way spanning URM walls under static loading. However, in real application masonry walls are usually behave in two-way spanning to extreme lateral loads resulting from blast, gas explosion, high speed wind etc. To evaluate the real behavior of TRM strengthening URM walls under extreme lateral loads, it would be better to perform the laboratory
test under static out-of-plane loading on two-way URM walls and expand study to the field blast test of them.

Referring to the TRM strengthening of existing walls, anchorage of TRM layers to the wall would be an important factor which may govern the effectiveness of the TRM layers on resisting to lateral loads. Therefore, the study of anchorage method between TRM layer and wall would provide improvement in load resistance and would introduce easy installation method. This anchorage test would provide better understanding if the tests would perform on URM walls which are structurally connected to the beam-column frame as in real application.

Introducing a numerical model for the simulation of the out-of-plane behavior of TRM strengthened URM walls under static and dynamic loading condition would help for practical application with saving experimental cost.
References


References


Appendix-A

Ultimate moment at balanced condition

Balanced moment

From Equation (4-13)

\[ M_{u, bal} = 0.44b(kd)^2 f'_m + T(1 - k)d \]  \hspace{1cm} (A-1)

Since under equilibrium condition;

\[ T = C = 0.714 bkdf'_m \]  \hspace{1cm} (A-2)

\[ M_{u, bal} = 0.44b(kd)^2 f'_m + 0.714 bkdf'_m (1 - k)d \]

\[ = \left[ 0.44k^2 + 0.714k(1-k) \right] f'_m bd^2 \]

\[ M_{u, bal} = [0.714 - 0.274k] f'_m bd^2 \]  \hspace{1cm} (A-3)

On the other hand,

\[ T = f'_tu' A_t \]  \hspace{1cm} (A-4)

Hence, balanced moment can also be written as,

\[ M_{u, bal} = 0.44b(kd)^2 f'_m + f'_tu' A_t (1 - k)d \]  \hspace{1cm} (A-5)

Ultimate flexural compressive moment

From Equation (4-31)

\[ M_u = [0.714 - 0.274k] f'_m bd^2 \]  \hspace{1cm} (A-6)

Equations (A-3) and (A-6) are same for balanced neutral axis depth ratio (k)

\[ (M_{u, bal} = M_u \text{ at balanced neutral axis depth ratio}) \]

Ultimate flexural tensile moment
From Equation (4-41)

\[ M_u = b d^2 f' m \left( \frac{\varepsilon_{tu}'}{\varepsilon_m'} \right) - \left( \frac{k^4}{4(1-k)^2} \right) \left( \frac{\varepsilon_{tu}}{\varepsilon_m} \right) + 2 \left( \frac{k^3}{3(1-k)} \right) + f_{tu} A_t (1-k)d \] \quad (A-7)

Under balanced condition, Equation (4-1)

\[ \frac{\varepsilon_{tu}}{\varepsilon_{mu}} = \frac{1-k}{k} \] \quad (A-8)

Also, Equation (4-7) gives

\[ \frac{\varepsilon_{mu}}{\varepsilon_m'} = 1.17 \] \quad (A-9)

\[ \varepsilon_{tu} = 1.17 \left( \frac{1-k}{k} \right) \] \quad (A-10)

Substituting (A-8) into (A-9),

\[ \frac{\varepsilon_{tu}}{\varepsilon_m} = 1.17 \left( \frac{1-k}{k} \right) \]

Substituting (A-10) into (A-7)

\[ M_u = b d^2 f' m \left( \frac{\varepsilon_{tu}'}{\varepsilon_m'} \right) - \left( \frac{k^4}{4(1-k)^2} \right) \left( \frac{\varepsilon_{tu}}{\varepsilon_m} \right) + 2 \left( \frac{k^3}{3(1-k)} \right) + f_{tu} A_t (1-k)d \]

\[ = b d^2 f' m \left( \frac{\varepsilon_{tu}'}{\varepsilon_m'} \right) - \left( \frac{k^4}{4(1-k)^2} \right) 1.17 \left( \frac{1-k}{k} \right) + 2 \left( \frac{k^3}{3(1-k)} \right) + f_{tu} A_t (1-k)d \]

\[ = b d^2 f' m \left( \frac{\varepsilon_{tu}'}{\varepsilon_m'} \right) - \left( \frac{k^3}{4(1-k)} \right) 1.17 + 2 \left( \frac{k^3}{3(1-k)} \right) + f_{tu} A_t (1-k)d \]

\[ = b d^2 f' m 1.17 \left( \frac{1-k}{k} \right) \left( \frac{k^3}{4(1-k)} \right) - \left( \frac{1.17}{4} \right) + \left( \frac{2}{3} \right) + f_{tu} A_t (1-k)d \]

\[ = b d^2 f' m 1.17k^2 \left( \frac{1.17}{4} \right) + \left( \frac{2}{3} \right) + f_{tu} A_t (1-k)d \]

\[ = 1.17 \left( \frac{1.17}{4} \right) + \left( \frac{2}{3} \right) b d^2 f' m k^2 + f_{tu} A_t (1-k)d \]

\[ M_u = 0.44 b(kd)^2 f'_m + f_{tu} A_t (1-k)d \] \quad (A-11)
Equations (A-5) and (A-11) are same for balanced neutral axis depth ratio (k)

\[
M_{bal} = M_u \quad \text{at balanced neutral axis depth ratio.}
\]