FLEXURAL STRENGTH AND DURABILITY OF REINFORCED CONCRETE BEAMS STRENGTHENED WITH HIGH PERFORMANCE TEXTILE REINFORCED MORTAR

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Abstract

Different techniques have been implemented for repair or strengthening RC structures. Drawbacks and limitations in the form of cost, safety, durability and performance of these techniques made Textile Reinforced Mortar (TRM) a promising strengthening material. In spite of the merits of this material, it has limited applications due to the lack of information and the premature failure (debonding) of the TRM strengthening layer. In addition, as the corrosion of steel reinforcement has significant influences in reduction the performance of RC beams, that impact with higher extent applies to the strengthening RC beams, which has not been fully addressed for TRM. Thus, this study explores the behaviour of RC beams strengthened with TRM with and without the presence of corrosion under flexural loading. In particular, attention has been focused on improving the bond strength of the interface between the substrate RC beams and the TRM strengthening layer to prevent the premature failure. Novel chemical and physical improvements are examined on the existing TRM strengthening technique to improve the interface bond strength. The chemical improvement is included using high strength mortar (HSM) instead of regular strength mortar as a matrix of the TRM, which provides higher adhesion with the substrate. The physical improvement included an application of high strength cementitious connectors at the interface to resist the shear and tensile stresses. These improvements would improve the efficient use of textile fibres and allow achieving high enhancement of the strengthened members. For thickness of concrete cover of 20mm, the experimental results demonstrated the efficiency of the proposed improvements; however, the effectiveness of the cementitious connectors was found profoundly influenced by the strength of substrate concrete. Besides, the experimental data also were assisted in validating the numerical interface bond model used in the finite element models for the beams and good agreement was evident between numerical and experimental load-deflection curves. Moreover, case studies with different properties of substrate RC beams and strengthening layer was carried out and it was found that about 100% increasing in ultimate capacity can be achieved for high strength substrate concrete beams (C60). The effect of steel reinforcement corrosion on the flexural behaviour of RC beams was investigated in two phases. During the first phase, control RC beams were corroded using impressed current (to simulate aging and deterioration of beams) and then repaired by the application of TRM composites. The objective in this phase was to investigate the effectiveness of repair of the corrosion-damaged beams. It was found that the degree of corrosion less than 4% exhibited a non-considerable effect on the repair process. During the second phase, the strengthened RC beams were later subjected to the impressed current based accelerated corrosion. The objective for this phase was to investigate the durability and longevity of the TRM based strengthening technique. The results demonstrated that the degree of corrosion higher than 10% can lead to losing the effectiveness of strengthening due to cover separation and that should be considered in the design of RC beams by means of remove the cover before strengthening.
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Declaration

I declare that the research contained in this thesis, unless otherwise formally indicated within the text, is the original work of the author. The thesis has not been previously submitted to this or any other university for a degree, and does not incorporate any material already submitted for a degree.

Signed: Ameer Baiee
Dated:
Chapter One

Introduction

1.1 Preamble

Reinforced concrete (RC) is the most heavily used material in construction industry around the globe. Whilst, in general, the strength and durability of RC structures are adequate, these are often subjected to deterioration mechanisms such as alkali-aggregate reaction, chemical attack, freezing and thawing action, abrasion, fire and corrosion that reduces their service life. Moreover, a large number of existing RC structures were constructed based on old design codes since design life of such structures ranged from 70-120 years but many are still in service much longer than intended design life. Hence, these do not conform to the safety requirements current design codes due to changes in utility and load demand for such structures, in addition, the ageing and degradation over the time. Accordingly, a significant number of these structures around the world are in urgent need of replacement or rehabilitation.

In economic terms, the direct replacing cost of an RC structure is considerably higher than rehabilitation cost. In addition, there are higher indirect costs accompanied with the replacement such as results of delay, loss of service and lost productivity. In the US for example, according to Federal Highway Administration, the annual direct costs (replacement and maintenance) for highway bridges were estimated to be $8.3 billion; however, the indirect costs were estimated to be as high as ten times that of direct costs (Maierhofer et al., 2010). In addition, replacing RC structures increases the demand for cement, which has harmful impacts on the environment through the increase in emission of CO$_2$ as a result of cement production. Therefore, the production of new RC structures depletes natural resources and increases air pollution. Complete replacement of an existing structure can be considered uneconomic and a burden on the natural resources if rehabilitation is a viable alternative.
The durability of RC structures is essential to ensure long service life, which leads to conserving resources and reducing wastes and the environmental impacts of repair and replacement. Strong emphasis in current guidelines for design and construction is on the durability hence any repair and strengthening work should exhibit durability. Corrosion of steel reinforcement adversely affects the durability of RC structures. Figure 1.1 demonstrates the effect of corrosion in spalling the concrete cover of RC structures. Concrete separation occurs when the stresses resulted from expanding corrosion products exceeds the tensile strength of concrete (Luca Bertolini et al., 2013). In the presence of strengthening layer, additional tensile stresses are applied on concrete which accelerates concrete cover separation and losing the contribution of strengthening in resisting the applied loads. Hence corrosion has higher deterioration influences on strengthened structures. Since strengthening and/or repair is applied to old structures which have a high possibility of subjecting to deterioration due to corrosion. The durability of the cover has a crucial role in the life service of any strengthening technique. The investigation, the performance of strengthened RC members in the presence of corrosion environment, is essential for safety, economy and environment aspects.

Figure 1.1: Reinforcement cover spalling due to steel reinforcement corrosion of RC bridge (Galvanizer Association, 2018)

1.2 Strengthening of reinforced concrete structures

Many structural engineers are faced with the challenge of evaluating and implementing practical and economical repair and/or strengthening technique. There is no single solution offers a simple, straightforward and effective method for repair and/or strengthening all types of RC structures. Different techniques have been used for the strengthening of RC structures. Strengthening techniques include; section enlargement, steel plate bonding,
external post-tensioning, polymer fibre epoxy bonded and textile fibre cementitious methods. Strengthening materials are the main difference among these techniques.

Section enlargement is the oldest strengthening method. In this technique, extra material is added to critical parts of the structure. Reinforced concrete is usually added to an existing structural member in the form of layers or jackets. The main disadvantage of this method is that it increases dimensions and weight of the strengthened member. Problems related to corrosion of steel reinforcement are also reported as a result to the relatively thin cover provided to the reinforcement of strengthened layer (Thanoon et al., 2005).

Steel plate bonding is very effective in increasing the shear capacity of structural members. The bonded steel plates were used usually as an external stirrup. The main drawback of this method is the risk of corrosion of the bonded steel surface. In addition, steel plates may need lengthening by joints, when used for flexural strengthening, due to limited transportation length. In addition, it can be difficult to apply steel plates to curved surfaces and the enhancement for flexural strength is limited (Barnes R.A. et al., 2001).

Post-tensioning with longitudinal strands can be used in enhancing the flexural and shear strength of RC structures. However, the effectiveness of this method is limited by the compression stresses that applied at the end of the member. This technique requires specialised devices and anchorages for prestressing (Tan and Tjandra, 2007).

Fibre Reinforced Polymer (FRP) technique is a polymer fibre bonded to the concrete surface using epoxy adhesive. FRP composite could be in the form of sheets, plates or bars. This technique has many favourable properties such as high strength to weight ratio and resistivity to corrosion but also has drawbacks mainly attributed to the organic epoxy resins used to bind the fibres (Ombres, 2011). Epoxy has many weak points such as poor fire resistance (60 to 82 °C); high cost; and inapplicability on wet surfaces or at low temperatures; hazards for the manual worker; poor thermal compatibility with the base concrete (Si Larbi et al, 2010; Ombres, 2011; Elsanadedy et al., 2013). That leads in many cases to debonding of FRP layers from the strengthened members (Gamage et al., 2006).

Recently, the textile fibres cementitious technique has been investigated for the strengthening of masonry and RC structures. This technique is based on using textile polymer fibres bonded to the substrate concrete using cementitious matrix (Ombres, 2011). The main difference between this technique and epoxy bonded technique is the type of binder. The cementitious binder can exhibit higher compatibility with the substrate members than epoxy (D’Ambrisi and Focacci, 2011). Scattered studies investigated using this technique in the field of strengthening RC structures most of which exhibited a premature (debonding) failure mode of the strengthening layer with controversial results.
about the effect of fibre and mortar properties on the performance of strengthened member. Premature failure refers to the separation of the strengthening layer from the substrate member before reaching the ultimate capacity of the strengthened members. Moreover, the literature lacks investigations on the durability of the strengthened members using the textile fibres cementitious technique as well as the applicability of these techniques for a range of extent of corrosion on existing deteriorating structures.

1.3 Textile reinforced cementitious material

Textile reinforced cementitious material is a composite material usually constituted by layers of continuous polymer fibre mesh (textile) embedded into the cement-based matrix. These fibres can be in the form of two (2D) and three (3D) dimensions as shown in Figure 1.2. The continuous filaments work as a tensile reinforcement of the composite material. The first application of this composite material was in the form of construction materials to produce thin-walled construction and facade elements (Hegger and Voss, 2008; Mechtcherine and Lieboldt, 2011). During the past few years, cost effectiveness, high compatibility with the substrate concrete, the high strength to weight ratio and corrosion resistance of these composite material encouraged researchers to investigate its efficiency in the field of strengthening. Figure 1.3 shows examples of the applications of cementitious textile materials.

![Figure 1.2: Textile cementitious materials reinforced with 2D and 3D textile polymer fibres (BFT International, 2012)](image)
Textile cementitious composites are similar to fibre reinforced concrete (FRC) regarding using cementitious matrix reinforced with fibres, but it has additional advantages in terms of the ability to control the density and location within the mortar matrix which in turns affecting the mechanical characteristics.

There are different forms of textile cementitious composite material; Textile Reinforced Mortar (TRM), Textile Reinforced Concrete (TRC), Fibre Reinforced Cementitious Mortar (FRCM) and Mineral Based Composites (MBC). All these forms have similar components with a slight difference in property of the cementitious matrix. For TRM, polymer modified mortar is usually used as a matrix, whereas fine-grained concrete with the maximum aggregate size of 1mm is used for TRC. Cement based mortar is used for FRCM and MBC (Ombres, 2011).

Different types and configuration of carbon, basalt, glass, aramid, polypropylene, and Polypara-phenylene-benzo-bisthiazole (PBO) can be used as a textile reinforcement in the cementitious material. The selection of fibre depends on different factors such as material properties, temperature resistance, bond quality and cost. The most important material properties of a textile fibre are tensile strength and modulus of elasticity. In addition, configuration and size of the textile mesh impact the behaviour of textile cementitious composite since these effects the penetration of cementitious matrix within the textile fibre. The reinforcement ratio and placement also affect the behaviour of the composite material. For cementitious matrices, tensile strength and workability significantly affect the performance of the composite textile cementitious member. However, most of the available literature emphasis on the properties of the textile fibres rather than the cementitious mortar which has compressive strength less than 70MPa.
1.4 Rationale for research

Many RC structures around the world are in urgent need of replacing or strengthening. Different techniques have been investigated to address this global issue. However, drawbacks and limitations in the form of cost, safety, durability and performance of these techniques encouraged researchers to investigate new material, textile cementitious mortar (TRM), as an alternative strengthening material. Despite the advantages of this composite material, it has limited applications in the field of strengthening. This limitation could be attributed to several reasons such as: the lack of information about the efficiency of TRM as a strengthening technique, the low strength of the implemented TRM, the premature TRM bonding failure, no investigation on the durability of repaired members with TRM and no investigation on the applicability of such strengthening technique in relation to the degree of deterioration. In addition to the safety risks associated with its sudden failure, it is considered a costly problem.

1.5 Aims and objectives

This study aims to investigate improving the bond strength between substrate RC beams and TRM strengthening layer to prevent the premature bond failure by means of conducting comprehensive experimental and numerical investigations of RC beams strengthened with TRM composites. In addition, to study the performance of the TRM composites in the presence of steel reinforcement corrosion as a repair material as well as investigation the durability of the strengthened members. The aim of this study can be achieved through the following objectives:

1. Design mix proportions of ultra-high strength cementitious matrix for TRM with investigation the effect of supplementary cementitious materials (SCMs) on the performance of the matrix.

2. Investigate the effect of the cementitious matrix properties on the performance of TRM composites.

3. Investigate and characterise the effects of high strength cementitious connectors’ configurations on the bond strength between RC beams and TRM strengthening layer.
4. Investigate the effect of amount and type of textile fibre layers on the behaviour of RC beams strengthened with TRM by using different cementitious matrix strength.

5. Investigate the efficiency of the TRM strengthening technique for the repair of corroded RC beams.

6. Investigate the durability of the strengthening technique in relation to the performance degradation of the strengthened RC members with particular focus on the failure mode.

7. Develop a numerical model to simulate the behaviour of RC structures strengthened with the TRM strengthening technique.

1.6 Structure of thesis

The present study is divided into six chapters and three appendices.

**Chapter two** review previous studies of strengthening methods with focusing on the limitation and drawbacks of these techniques. A detailed review of the available works on the study of strengthening members using TRM is presented. In addition, the bond strength between two cementitious materials cast at different age is reviewed to identify the factors affecting the bond strength. Moreover, corrosion aspects were also studied by means of the mechanism, methods of acceleration and consequences of corrosion on strengthened RC beams.

**Chapter three** presents the detailed experimental investigation of developing high-performance TRM composites. Materials properties, mix proportions of normal and high strength mortar, and the effect of SCMs on the performance of high strength mortar are explained. Moreover, the mechanical properties and shrinkage behaviour of TRM are presented and discussed.

**Chapter four** presents the experimental investigation of bond, strengthening, repair and corrosion impacts on the strengthening system. The bond investigation included examined the interface bond strength under direct tensile and combined shear and compression stresses. RC beams strengthened with different properties of mortar, textile fibres and
connectors were investigated under the strengthening aspect. Repair of corroded RC beams were also investigated. Moreover, the effect of corrosion on the durability of strengthened RC beams was examined.

**Chapter five** presents the implemented numerical model which included three parts: modelling, validation and investigation case studies. Modelling part consists of materials modelling, finite element characterisation, loading procedures and boundary conditions and nonlinear solution method. The comparison between the experimental and predicted results was demonstrated in the validation part. Finally, case studies were conducted to examine different properties of concrete, steel reinforcement, textile fibres and connectors ratios.

**Chapter six** summarises the general conclusions based on the experimental and analytical investigations. In addition, the recommendations for further research is presented.

**Appendix A** presents the procedures of determination the area of textile fibres, the stress-strain relationship of the TRM composites under direct tensile stresses and the load deflection responses under flexural loading.

**Appendix B** presents the design of RC beams and the calculations of the required impressed corrosion.

**Appendix C** presents the load slip behaviour of the strengthened RC beams with TRM under flexural loading.
Chapter Two

Literature Review

2.1 Introduction
Rehabilitation of RC structures usually refers to either strengthening or repair. The purpose of the repair is to restore the original design strength that was lost due to ageing deterioration. While strengthening increases the existing strength of a structure beyond its original design strength to satisfy new demands. Different techniques have been used for the strengthening and repair of RC structures. The interface bond strength between the substrate member and the strengthening layer is considered a decisive parameter in selection the efficient strengthening method. A brief review of the strengthening techniques with emphases on interface bond strength, the drawbacks and limitations of these techniques are the subject of the first part of this chapter.

The cost-effectiveness, durability and compatibility of cementitious textile materials encourage many researchers to investigate the efficiency of this material in the strengthening field. A detailed review of the available literature of strengthening structures using cementitious textile materials was conducted in the next section. Within this section, the characteristics of the cementitious textile materials by means of the mortar and textile fibres properties were prescribed. Moreover, the details of the previous analytical and numerical studies regarding cementitious textile materials are also explained. In addition, a critical review is conducted to study the bond strength of cementitious materials with substrate concrete. The interface bond assessment methods are also reported.

Corrosion is considered one of the major deteriorating agents of RC members. Depending on the degree of corrosion, corrosion products cause cracking, spalling and delamination of concrete cover. In fields of strengthening, additional stresses are applied on the concrete cover resulted from the loading of the strengthening layer, which increases the impacts of corrosion in reducing the durability of the strengthened member. A brief review is presented on the mechanism and induced corrosion methods. The effects of corrosion on the behaviour of RC structures through reducing the bond between the steel and concrete and the strength of RC members were also reviewed.
2.2 Strengthening of RC members

2.2.1 Section enlargement

Section enlargement is considered the oldest strengthening/repair technique for RC structures. It is performed by adding new material to the critical part of a structure. Different materials have been used such as reinforced concrete, ferrocement and fibre reinforced concrete. These materials are usually applied in the form of jacketing to provide sufficient bond strength at the interface.

Many researchers investigated the effectiveness of this method for strengthening/repairing RC structures with a focus on improving the interface bond strength. Thanoon et al. (2005), for instance, investigated the efficiency of repair cracked RC slabs with new 50mm reinforced concrete layer. The cracked surface of specimens was roughened and holes drilled to fix 10 mm diameter steel bars (155mm long) using Sikadur30 epoxy adhesive. The configuration of the shear anchors is illustrated in Figure 2.1. The experimental results demonstrated an enhancement in ultimate load capacity despite the horizontal cracks at the interface due to the relative displacement between the two layers.

It is interesting to notice the contribution of the steel anchors in transmitting the stresses between the substrate concrete and the additional layer which delays the debonding to achieve higher capacity.

![Shear connectors of the strengthening layer of RC slabs](image)

Figure 2.1: Shear connectors of the strengthening layer of RC slabs (Thanoon et al., 2005)

Recently, Tsioulou et al. (2013) used 50mm reinforced concrete strengthening layer to improve the performance of RC beams without using shear connectors. Only about 8% enhancement in ultimate load was observed due to the debonding of the strengthening layer. However, Chalioris et al. (2014) investigated the flexural behaviour of RC beams repaired with self-compacted concrete jacketing (SCC). The experimental work included
the addition of 5mm diameter of steel shear connectors in the form of L-formed dowels to improve the bond strength between substrate RC and 25mm SCC layers. Despite the relative slip observed for the repaired specimens, about 45% enhancement in the ultimate load was observed. In addition, the results indicated to the efficiency of increasing the contact area between the substrate concrete and the repaired layer in enhancing the resistance of stresses at the interface.

Another set of studies investigated the applicability of reducing the thickness of strengthening layers using Ferro cement composites. These composites are made up of cement mortar reinforced with uniformly distributed steel wire mesh layers. Sivagurunathan and Vidivelli (2012) investigated repair RC beams loaded initially to 75% of the ultimate load of control specimen with 25mm layer of Ferro cement at the tension face. The experimental work included repairing two beams with the cast in situ Ferro cement layer reinforced with one steel mesh and four beams with precast Ferro cement laminates reinforced with 2 and 3 steel meshes. Epoxy resin was used to bind the precast Ferro cement layers. The repaired beams with the cast in situ Ferro cement demonstrated the higher enhancement in ultimate load (14%) than precast layers (9%). The failure of both cases was due to the debonding of Ferro cement layers from the substrate beams.

In a similar study, Khan et al. (2013) investigated the effectiveness of repairing RC beams with 18 mm cast in situ and precast Ferro cement laminates. The specimens initially loaded to 45% of the ultimate theoretical load, which represents the service load. For cast in situ specimens, steel nails were used to bind the substrate RC beams after preparing their surface using a wire brush. The samples exhibited flexural failure mode with 18% and 15% enhancement for cast in situ and precast Ferro cement laminates, respectively.

In the same field, ultra-high-performance fibre reinforced concrete (UHPFRC) has been investigated as the strengthening layer for RC structures. For instance, Martinola et al. (2010) examined the behaviour of full-scale beams (4.55m length) strengthened with 40mm UHPFRC jackets. The increase in the ultimate flexural load of the strengthened RC beam was about 115%. This high enhancement in ultimate load was due to two main reasons. Firstly, the strengthening layer is extended after the support points, which provided a compression stress on the strengthening layer, which improve the friction at the interface. In addition, the supports worked as two anchors at the ends of the beams effective length to prevent the debonding of the strengthening layer during loading. Secondly, the tensile reinforcement of the control beam was relatively low (less than the minimum reinforcement required for reinforced concrete beams) which led to present high enhancement in comparison with strengthened members. Similar strengthening material
has been investigated by Mohammed et al. (2016) to study the torsional behaviour of RC beam. The strengthening layer has been also extended over the supports. The results demonstrated the enhancement in torsional strength of about 95% with 25mm of UHPFRC added on all sides.

It can be stated from the above studies that the bond between new and old concrete is the key to successful strengthening method. Higher enhancement of capacity can be achieved by using anchors and high strength matrix.

Despite the enhancement of capacity for members strengthened using section enlargement, following drawbacks accompanies this method:

- Increase the weight of the strengthened member.
- The strengthening layer is susceptible to the corrosion as a result of insufficient cover of the steel reinforcement.
- Some materials are costly such as steel fibres and epoxy resins which increase the cost of strengthening and or repair process.
- Careful execution and vibration of concrete is required, and extra materials are required such as epoxy resins.

### 2.2.2 Steel plate bonding

Steel plates or straps have been used to strengthen RC structures. Steel plates can be bond to the tensile face of RC members to enhance the flexural capacity. For shear strengthening, steel plates are bonded on the sides of the RC members. These steel sheets are attached to the substrate RC members using adhesive bonding, bolting or both. Figure 2.2 illustrates strengthened RC beams with bolted steel plates.

In this field, Arslan et al. (2008) investigated the flexural behaviour of repaired RC beams using steel plates. The strengthened specimens were loaded up to failure then steel plates were bond using epoxy resin on the tensile face. The samples failed due to peeling of the steel plates. That may refer to insufficient bond strength at the interface despite the relatively high adhesive properties of the epoxy.

On other hands, using steel bolts anchors to bond steel plates for strengthening RC beams were found have a negligible effect on the failure mode (Aykac et al., 2013). It was concluded that epoxy with U collars to attach steel plates at the tensile face better than bolts in preventing the shear peeling. The increase in the ultimate load using 1.5, 3 and 4.5 mm thick epoxy adhesive steel plates were found to be 35%, 70% and 60%, respectively. The reduction in the enhancement of 4.5 mm steel plates was due to peeling of steel plates.
Barnes (2001) investigated the effect of bonding type used to attach steel plates on the shear strengthening of RC beams. Socket anchorages and epoxy were used to attach steel plates to the vertical sides of RC beams. Socket anchorages were positioned before casting the beam and after curing time, welded with the plate to prevent any slip. Specimens strengthened with 4mm steel plates using epoxy showed 64% enhancement in ultimate shear capacity with debonding failure mode. Whereas, strengthened specimens with welded plates explained 142% increase in ultimate load due to the improved bond strength.

In another study, Altin et al. (2005) investigated the effectiveness of strengthening deficient reinforced concrete T-section beams in shear with 4mm thickness steel plates at vertical sides using epoxy after roughening the sides of beams and steel plates. About 60% improvement in ultimate shear capacity with respect to the control beam and flexural failure mode was observed.

It can be concluded that strengthening and or repairing RC members with steel plates can provide significant shear and/or flexural enhancement if peeling of steel plates can be prevented. Moreover, steel anchors improved the stresses transferring at the interface, which was evident from the increase in the capacity compared with strengthened members without connectors. In addition, the following drawbacks may be attributed to steel plates bonding:

- The risk of corrosion of the bonded steel surface and the anchors.
- Inapplicability to the rounded sections or curves.
- Connectors are required to prevent the peeling of steel plates.
- Steel plates might need lengthening by joints due to limited transportation length when it is used for strengthening long beams.
2.2.3 Post tensioning

High strength steel strands have been used for strengthening of RC structures through post tensioning. This method is based on setting initial deflection opposite to the deflection produced from the applied loads through the application of compression forces at the ends of the member. For strengthening RC members, special steel plates need to be fixed at the ends and middle of the strengthened member to apply the external compression force from pulling the high strength strands, as shown in Figure 2.3. For this purpose, holes are drilled through the member to place transverse connectors. As the transverse drilling may cause damage to the concrete or existing reinforcement, a specialist contractor is required to detect the reinforcement location.

Figure 2.3: (a) anchorage end plate (b) deviator middle plate (Tan and Tjandra, 2007)

Tan and Tjandra (2007) investigated strengthening RC beams using 7-wires steel prestressing steel strands, which were applied on both sides of the strengthened beams. All strands were post-tensioning up to 950MPa tensile stress. It was observed that parabolic tendons profile increased the ultimate load up to 44% while only 33% enhancement was achieved for straight tendons profile counterpart.

For shear strengthening, Kim, Yang et al. (2007) investigated the behaviour of RC beams repaired with prestressing strands. The study included repaired RC beams failed in shear with vertical and diagonal wire rope units. All specimens were designed without shear reinforcement to fail in shear and then repaired using vertical and diagonal prestressing strands. The repaired samples with shear span to depth ratio (a/h) equal to 1.5 exhibited 71% and 92% enhancement in ultimate shear capacity for vertical and diagonal wire rope. While a negligible increase for 2.5 a/h counterparts was observed and a decreasing in shear capacity for repaired specimens with 3.25 shear span to depth ratio. It can be found that this technique is more suitable for deep beams rather than slender RC beams. As a result, the following drawbacks can be concluded:
Special devices and materials for prestressing are required.
- Special technicians are required to allocate the prepare location for anchorages setup. In addition, it needs very stiff and powerful anchorages.
- Steel tendons are corrosion sensitive.
- The use of high strength steel strands is costly compared with normal strength steel.

### 2.2.4 Fibre reinforced polymer (FRP)

Fibre reinforced polymer (FRP) has been widely used for repair and or strengthening reinforced concrete structures. This material has many favourable properties such as high strength to weight ratio, corrosion resistance, ease and speed of application with minimal change of geometry (Ombres, 2011).

Several experimental investigations have been reported on the performance of concrete structures strengthened with FRP. Aram et al. (2008) investigated the behaviour of RC beams strengthened for flexural with a carbon fibre reinforced polymer (CFRP). Different tensile strength of CFRP; 1300, 2000 and 2700MPa were applied. Specimens strengthening with low CFRP tensile strength (1300MPa) failed due to CFRP rapture while the other samples failed due to the CFRP debonding. All strengthened specimens exhibited about 100% increase to the ultimate load. The results showed that despite application of strengthening layers with different properties, all strengthened specimens exhibited the same enhancement, which reveals a difficulty in predicting the enhancement of strengthening in presence of debonding.

In a similar study, Kim and Shin (2011) investigated the effect of using hybrid carbon and glass fibre reinforced polymer for strengthening RC beams. It was demonstrated that hybrid strengthening layer present higher increase in ultimate load due to delay the debonding in comparison with specimens strengthened with CFRP or GFRP. Increasing in ultimate capacity due to strengthening with two carbon, glass and one glass and one carbon fibres were 32%, 35% and 41%, respectively. By comparison the amount of strengthening enhancement with the work of Aram et al. (2008), it can be observed a high difference as a result of the difference in the strength of control beams of both studies.

The effect of application anchors to improve the performance of specimens strengthening with CFRP has been investigated by Anil and Belgin (2009). Mechanical steel fasteners and anchors prepared from CFRP strips were used as shown in Figure 2.4. Specimens with steel anchors exhibited CFRP rupture with increasing in ultimate capacity about 180% while samples without anchors failed due to debonding. The rupture of CFRP
strips was near to the steel anchors due to the stress concentration at the holes of anchors. The results demonstrated that specimens with CFRP anchors delayed the debonding but failed in preventing the debonding of the strengthening layer.

![Figure 2.4: a. Mechanical steel anchors. b. CFRP anchors (Anil and Belgin, 2009)](image)

To avoid fibres damages due to the effect of anchors, Wu et al. (2011) investigated using a fastening technique with steel plates to apply a vertical pressure on CFRP sheets. Figure 2.5 shows the mechanical fasteners and plates used to improve the bond between CFRP and RC beams. The results explained that strengthened specimens without fastening technique failed due to debonding of CFRP strip with about 55% of ultimate load capacity of strengthened samples with inclusion fastening technique. In addition, all strengthened RC beams with fastening technique failed due to tensile rupture of CFRP strips.

![Figure 2.5: Fastening technique for strengthening RC beams with CFRP (Wu et al., 2011)](image)

Similar results have been observed by Zhou et al. (2013) when investigating RC beams strengthening with CFRP using different types of anchorages. Strengthened beams with mechanical fastening technique exhibited tensile CFRP failure mode with increasing in ultimate capacity to about 75%. While U shaped CFRP anchors counterparts failed due to debonding of the CFRP strips. The results demonstrated that mechanical fastening anchors
are more effective than other anchors in preventing the debonding of FRP strengthening layer.

Different studies reported durability issues of FRP materials that may lead to premature failures. Yun and Wu (2011) investigated the long-term durability of fibre reinforced polymer (FRP) strengthening systems under freeze-thaw cycling. The study consisted of single pull-off shear test to evaluate the degradation in bond strength after subjected to different cycles of freeze and thawing. It was observed a significant decrease in bond strength between CFRP sheets and substrate concrete up to about 50% of control specimens. This reduction in bond strength was due to high fluency of epoxy with freeze and thawing cycles.

In the same field, Firmo et al. (2012) investigated the effect of fire on the performance of RC beams strengthened with CFRP. The study examined different insulation materials to protect CFRP layers. The results showed that non-insulation strengthened specimens failed due to debonding under 55°C within 23 minutes. While with insulation materials, the samples can sustain the same load with the same conditions up to 167 minutes. That explains the sensitivity of epoxy resin to the temperature, which in turn affects the performance of strengthened RC members by means of debonding failure mode.

Similarly, Lai et al. (2013) found a similar deficiency of epoxy relating to the temperature. Externally-bonded CFRP concrete composites had been deteriorated through exposing groups of specimens into three water baths with different elevated temperatures (25°C, 40°C and 60°C) for a period of 5, 15 and 30 weeks. The results showed that there was an early start of losing adhesive bond for all the 60°C specimens at lower force levels (<5 KN). In addition, recently, Raoof and Bournas (2017) found that CFRP composites loss about 83% of the bond strength with the substrate concrete at 150°C while carbon fibres with a cementitious adhesive loss only about 15% of the bond strength up to 400°C.

It can be stated that despite the high bond strength that provided by epoxy resins, the strengthening layers need anchors to avoid the debonding failure mode. Moreover, many researchers reported drawbacks of this method, which is attributed to the organic epoxy resins used to bind the fibres. Epoxy resin has many weak points such as poor fire resistance, high cost, hazards for the manual worker, inapplicability on wet surfaces or at low temperatures, and poor thermal compatibility with the substrate concrete (Si Larbi et al., 2010; Ombres, 2011; Elsanadedy et al., 2013). In addition, Gamage et al. (2006) reported that epoxy resin loss its ability to bind fibres and became a viscous martial with weak tensile properties under temperature 60-70°C.
In addition, the following main points can be drawn from the brief review of FRP strengthening materials:

- FRP sheets need protection from environmental effects.
- Premature debonding failure is a critical issue in many strengthening application and mechanical anchors are necessary to achieve sufficient bond conditions.
- Insulation layers are required to provide sufficient protection of FRP layer against fire.
- High cost and hazard effects of epoxy resin.
- Environmental limitations regarding temperature and humidity.
- Low compatibility between the FRP-epoxy and substrate concrete.

2.3 Textile reinforced mortar (TRM)

Textile reinforced mortar (TRM) materials represent promising materials in the field of repair/strengthening due to their advantages. Among these advantages, the ability to produce thin-walled structural members with high load carrying capacity, and resistivity to corrosion. Moreover, no concrete cover is required to protect the reinforcement against corrosion, and the fibre material is placed only where necessary in the direction of the tensile stresses.

TRM is a composite of a cementitious matrix reinforced with textile polymer fibres. The first application of these composites was in the field of facade construction but also use in the production bearing structural elements for minor vertical loads (Hegger and Voss, 2008). Recently, the efficiency of different forms of these composite materials in strengthening masonry and concrete structures has been investigated. Different terms have been used for textile reinforced mortar (TRM) in literature: Textile reinforced concrete (TRC); textile-reinforced mortar (TRM); mineral-based composites (MBC); and fibre reinforced cement (FRC). The difference between these forms is mainly attributed to the matrix properties. The matrix of TRC is a fine-grained concrete with a maximum aggregate size of 1mm. Cement mortar with and without polymers are used with TRM, FRCM, FRC and MBC (Ombres, 2011; D’Ambrisi et al., 2012; Schladitz et al., 2012; Bernat et al., 2013).
2.3.1 Cementitious matrix

The cementitious matrix can range from mortar to fine gained concrete. For TRM applications, fine-grained concrete refers to a cementitious material with maximum size aggregate equal to 1 or 2 mm (Häußler-Combe and Hartig 2007). This type of cementitious materials is also considered mortar because according to BS EN 12620:2002+A: 2008 the aggregate is considered fine if the maximum size is less than 4mm.

Gopinath et al. (2011) pointed out that the performance of the cementitious matrix depends on its ability to provide sufficient penetration within textile reinforcement, adequate bond strength, enough setting time to ensure the required workability during preparing specimens and sufficient dimensional stability. Table 2-1 summarises mechanical properties of the commonly used matrix for TRM. Most of the available studies of TRM composites used normal strength matrix (mortar) as are discussed in section 2.3.6.

<table>
<thead>
<tr>
<th>Author</th>
<th>Matrix type</th>
<th>Compressive strength (Mpa)</th>
<th>Direct tensile strength (Mpa)</th>
<th>Flexural tensile strength (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Di Tommaso et al. (2008)</td>
<td>Cement based</td>
<td>28</td>
<td>--</td>
<td>4</td>
</tr>
<tr>
<td>D’Ambrisi et al. (2012)</td>
<td>Cement based</td>
<td>16.1</td>
<td>2.55</td>
<td>--</td>
</tr>
<tr>
<td>Trapko (2014)</td>
<td>Cement based</td>
<td>29</td>
<td>3.5</td>
<td>--</td>
</tr>
<tr>
<td>Al-Salloum et al. (2012)</td>
<td>Polymer modified mortar</td>
<td>56.4</td>
<td>3.4</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>Cement mortar</td>
<td>23.9</td>
<td>2.77</td>
<td>--</td>
</tr>
<tr>
<td>Ombres (2011)</td>
<td>Cement mortar</td>
<td>23</td>
<td>3.5</td>
<td>--</td>
</tr>
<tr>
<td>Elsanadedy et al. (2013)</td>
<td>Cement mortar</td>
<td>23.9</td>
<td>2.77</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>Polymer modified mortar</td>
<td>56.4</td>
<td>3.4</td>
<td>--</td>
</tr>
<tr>
<td>Tetta et al. (2016)</td>
<td>Cement mortar</td>
<td>36</td>
<td>--</td>
<td>8.5</td>
</tr>
<tr>
<td>Raoof et al. (2017)</td>
<td>Polymer modified mortar</td>
<td>39.2</td>
<td>--</td>
<td>9.8</td>
</tr>
</tbody>
</table>
2.3.2 Textile reinforcement

Textile fibres are used to carry the tensile forces in the TRM composites arising from the applied loads. Textile fibre reinforcement consists of two sets of textile fibre yarn, warp and weft perpendicularly to each other (Silva et al., 2011). Yarns are composed of hundreds to some thousands of filaments with diameters of a few μm to obtain the desired thickness of the yarn. Different types of textile fibres are available as TRM reinforcement such as glass, basalt, carbon and polypropylene fibre mesh (Escrig et al., 2015). Figure 2.6 shows different examples of textile fibres.

![Figure 2.6: Textile reinforcement: (a) glass, (b) basalt, (c) carbon and (d) polypropylene (Escrig et al., 2015)](image)

The properties of interest of the textile reinforcement are the ultimate tensile strength, modulus of elasticity and elongation at breaking (rupture). In addition, the configuration and mesh size of the textile fibre affects the performance of TRM composites especially with the low workability of the matrix.

Some researchers express the tensile strength of the textile fibres as a tensile force per width, while others use tensile load per unit area (tensile strength). This attributes to the difficulty in many cases of accurate determination the area of fibres. Table 2-2 summarises the properties of the common textile fibres used in the field of strengthening in literature. Direct tensile tests are usually used to assess the tensile mechanical properties of the textile fibre mesh (Al-Salloum et al., 2011; Al-Salloum et al. 2012; Contamine et al., 2013; Razavizadeh et al., 2014; De Santis and De Felice, 2015). However, the possibility of rupture of the fibres at the grips of the tensile machine, and the difficulty of applying the tensile load equally on the filaments within the specimens are the main reasons for low accuracy in the measurements. De Santis and de Felice (2015) pointed out that a linear behaviour of glass fibre mesh tested under direct tensile test can be achieved with a special attention to the specimen’s preparation. Aluminium tabs were used at the ends of the samples to ensure equal loading of the textile filaments and avoid the specimens rupture at the tensile machine grips as shown in Figure 2.7. Selection of the type of fibres for specific applications depends on the following factors (De Santis and De Felice, 2015):
- Cost and environmental impacts.
- Resistivity to temperature and alkali.
- Mechanical materials properties.
- Mesh configuration.

Table 2.2: Material properties of textile reinforcement

<table>
<thead>
<tr>
<th>Author</th>
<th>Fibre type</th>
<th>Tensile strength</th>
<th>Modulus of elasticity (GPa)</th>
<th>Ultimate strain %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gopinath et al. (2011)</td>
<td>AR-glass</td>
<td>45 kN/m width</td>
<td>--</td>
<td>3</td>
</tr>
<tr>
<td>Bernat-Maso et al. (2014)</td>
<td>Glass</td>
<td>45 kN/m width</td>
<td>90</td>
<td>3</td>
</tr>
<tr>
<td>Bernat-Maso et al. (2014)</td>
<td>Basalt</td>
<td>92 kN/m width</td>
<td>95</td>
<td>3.15</td>
</tr>
<tr>
<td>Bernat-Maso et al. (2014)</td>
<td>Carbon</td>
<td>160 kN/m width</td>
<td>240</td>
<td>1.8</td>
</tr>
<tr>
<td>A. Di Tommaso et al. (2008)</td>
<td>PBO</td>
<td>5800 MPa</td>
<td>270</td>
<td>2.15</td>
</tr>
<tr>
<td>Al-Salloum et al. (2012), Elsanadedy et al. (2013)</td>
<td>Basalt</td>
<td>623 MPa</td>
<td>31.94</td>
<td>--</td>
</tr>
<tr>
<td>Tetta et al. (2016)</td>
<td>Heavy carbon</td>
<td>4800 MPa</td>
<td>225</td>
<td>--</td>
</tr>
<tr>
<td>Tetta et al. (2016)</td>
<td>Light carbon</td>
<td>3800 MPa</td>
<td>225</td>
<td>--</td>
</tr>
<tr>
<td>Tetta et al. (2016)</td>
<td>Glass</td>
<td>1400 MPa</td>
<td>74</td>
<td>--</td>
</tr>
</tbody>
</table>

Figure 2.7: (a) testing set up (b) stress strain curve of textile glass mesh (De Santis and De Felice, 2015)
2.3.3 Mechanical properties of TRM composites

TRM composites are subjected to the tensile stress through either direct tensile stress or flexural tensile stress. The tensile strength of these composites plays a crucial role in defining the performance of TRM composite. The behaviour of TRM composites is considered complex due to the difficulty of determining the bond between textile fibres and surrounding mortars. Häußler-Combe and Hartig (2007), demonstrate that the yarn of the textile fibres cannot be considered as homogenous over its cross section. The behaviour of TRM composite under direct tensile is controlled by the bond between the filaments that compose the yarn and the bond between the yarn and the surrounding mortar.

Different specimens’ designs have been used to investigate the tensile properties of TRM composites. For example, plate specimens of cementitious materials reinforced with textile fibre and subjected to direct tensile have been used by researchers to assess the tensile properties (Häußler-Combe and Hartig, 2007; Hegger and Voss, 2008; Silva et al., 2011; Si Larbi et al., 2012) as shown in Figure 2.8. However, Silva et al. (2011) found the for constant properties of TRM, the tensile strength and strain capacity decrease as the free sample length increases.

![Figure 2.8: Tensile test of TRC (Si Larbi et al., 2012)](image)

Other researchers have used dog bone specimens reinforced with textile fibres to determine the tensile and bond strength of TRM composites (Mumenya et al. 2010; Kim and Yun, 2011) as shown in Figure 2.9.
The tensile stress-strain relationship of TRM specimens can be divided into three stages (Hegger and Voss, 2008; Mechtcherine, 2013). The first stage increases linearly according to the properties of the matrix until the ultimate tensile strength of the matrix is reached and the cracks are initiated. Textile reinforcement bridges the cracks and depending on the reinforcement ratio, stage two starts by retransmitting tensile forces to the matrix. More cracks appear at various locations of the sample due to reaching the ultimate tensile strength of the matrix. The stress-strain curve of this stage is flatter compared to the first stage with a saw tooth course depending on the reinforcement ratio. During the third stage, the slope of the stress-strain curve becomes steeper depending on the properties of the textile reinforcement with increasing the bearing capacity until the ultimate tensile strength of reinforcement is reached, see Figure 2.10.
However, Larrinaga et al. (2013) and (2014) pointed out that the tensile behaviour of basalt TRM specimens can be divided into four stages. Stage one and two are similar to the Hegger and Voss (2008) model. However, in stage three, the composite may present a perfect bond or debonding performance. The perfect bond phase exhibited linear behaviour while debonding explained non-linear behaviour. Stage four represents the maximum load can be reached by rupturing of all filaments of the textile fibre.

2.3.4 Strengthening masonry structures using TRM composites

Masonry members are generally designed to resist in-plane compressive loads. Therefore, any out-of-plane forces or diagonal shear forces may lead to failure of these members. To avoid that, strengthening with different materials such as FRP can be used. However, these materials have low compatibility (high difference in the modulus of elasticity) with masonry members, which led to looking for alternative materials can provide proper compatibility to avoid drawbacks associated with using an organic binder (epoxy). Textile reinforced mortar (TRM) presents high compatibility with masonry members due to use inorganic (cementitious) binder. Recently, some researchers investigated the efficiency of using different forms of textile reinforced cementitious materials to strengthening masonry members.

Garmendia et al. (2011) investigated the effectiveness of using basalt textile reinforced mortar in strengthening masonry arches with inclusion spike anchors made of basalt fibre see Figure 2.11. Significant enhancement was observed, and the failure load increased from 1.45kN to 19.3KN, when arches were strengthened with two layers of basalt fibre. Also, the spike anchors changed the failure mode from deioned to basalt breakage with 15% enhancement in capacity in comparison with specimens without anchors.

Similarly, 12% enhancement due to use steel mechanical anchors was observed by Ismail and Ingham (2014) in strengthening masonry walls with glass TRM subjected to a diagonal shear load. They also achieved about 350% increase in diagonal shear strength for specimens strengthened in both faces. However, Bernat et al. (2013) observed a reduction to about 15% in the enhancement of the capacity of masonry brick walls strengthened with carbon textile reinforced mortar in the presence of carbon spike anchors.

Babaeidarabad et al. (2014) investigated the effectiveness of improving the diagonal shear strength of concrete masonry walls using carbon FRCM. The results exhibited about 95% and 135% increase in the ultimate capacity as a result of application one and four carbon FRCP composites, respectively. The failure of the strengthened specimens was due
to crushing the toe of the walls near the applied load, which demonstrated the efficiency of the strengthening layer in reaching the ultimate capacity of the walls.

Figure 2.11: Spike anchors of masonry aches strengthened with basalt fibre (Garmendia et al., 2011)

The effect of the properties of the TRM matrix on the strengthening performance was reported by Bernat-Maso et al. (2014) through their experimental study on masonry walls strengthened with glass TRM. It was found that for the same textile reinforcement, the strengthening enhancement increased about 12% when using mortar with compressive strength equal to 42.2 MPa instead of 14.5MPa. In a similar study, Cevallos et al. (2015) achieved about 53% increase in the ultimate eccentric compressive load of masonry elements strengthened with polyparaphenylene BenzobizOxazole (PBO) fabric-reinforced cementitious matrix (FRCM) composite. However, in spite of the high increasing (about 300%) in comparison with un-strengthen specimens, in the diagonal shear strength of masonry walls strengthened with glass TRM on both vertical faces, a debonding failure was observed by Yardim and Lalaj (2016).

2.3.5 Strengthening RC structures using TRM composites
The available studies regarding strengthening RC members using TRM are reviewed and the main parameters affect strengthening performance are identified and presented in the following subsections. Most of these parameters relate to the new strengthening layer, and only the surface preparation is attributed to the substrate RC members. Table 2.3 summarises the results of the available relevant investigations of RC beams strengthened with cementitious textile materials.
2.3.5.1 Substrate surface preparation
Proper surface preparation is a critical factor in a successful performance of strengthening RC structures. All surfaces receive strengthening should have sufficient roughness to increase the interlocking between the substrate member and the strengthening layer. Rough surface improves the bond strength through increasing the interlocking and contact area at the interface. Ortlepp et al. (2006) discovered that surface pre-treatment of the substrate concrete could prevent the failure in the bond between substrate concrete and new textile reinforced concrete. Ombres (2011) found that the best enhancement in ultimate strength of RC beams strengthened with PBO-FRCM and with sandblasting treatment was 34%. Most of the previous studies adopted the sandblasting treatment to prepare the substrate surface of the strengthened concrete members. Despite the efficiency of sandblast treatment in comparison with other treatments (such as chipping, wire brushing and grooving) the strengthened concrete members are still suffering from the debonding of the strengthening layer (A. Di Tommaso et al., 2008; Ombres, 2011; Si Larbi et al., 2012; Jabr et al., 2017; Escrig et al., 2017; Raoof et al., 2017). In particular, in high enhancement is required which needs interface bond strength higher than the maximum bond provided by the rough surface alone. Therefore, more improvement to the substrate surface is required to provide more interlocking between the substrate concrete and the new strengthening layer.

2.3.5.2 Amount of textile reinforcement
In the presence of sufficient interface bond strength, the amount of textile fibres controls the capacity enhancement of strengthened member through the axial tensile strength and the bond properties of fibres. The higher amount of fibres with a given mortar cross-section leads to reduce the bond strength between the fibres and mortar which affects the efficiency of strengthening. However, the enhancement is limited by the strength and failure mode of the substrate member. For instance, strengthening of RC beams for the flexural purpose should be limited by the shear strength of the substrate beam to avoid losing the full contribution of the strengthening in resisting the applied stresses.

Trapko (2014) found that the capacity of plain concrete cylinder small-scale columns confined with PBO-FRCM increases as the number of strengthening layer increases. The results demonstrated the increase in the ultimate compressive capacity of specimens confined with one, two and three PBO-FRCM to about 45%, 90% and 150%, respectively. In a similar study, Colajanni et al. (2014) found that strengthening unreinforced concrete
short columns with two layers of carbon FRCM explained 29% increase in ultimate capacity while specimens with three layers explained 49%. It was also observed that all strengthened samples failed due to rupture of strengthening layer. In another study, Colajanni et al. (2014) achieved 29% and 51% increase in ultimate compressive strength of plain concrete confined with two and three layers of PBO-FRCM, respectively. In contrast, Trapko (2014) found that the enhancement in the ultimate axial capacity of RC columns strengthened with different layers of PBO-FRCM decreases as the number of layer increases. Specimens strengthening with one layer explained increasing about 17%, while two and three layers exhibited 10% and -8%, respectively.

For rehabilitation of RC slabs, Schladitz et al. (2012) found that as the number of carbon TRC strengthening layer increases, the flexural capacity of RC elements increases. The strengthened specimens of slabs with dimensions 230×1000×7000 mm with four layers exhibited an enhancement in ultimate strength up to 245% in comparison with non-strengthened samples. While 67% enhancement in strength was observed for one strengthening layer. It is interesting to note that higher interface area (such as in slabs) is effective in improving the interface bond strength which leads to achieving a higher enhancement in the capacity.

A. Di Tommaso et al. (2008) reported that strengthened RC beams with one layer of PBO-FRCM composites exhibited a negligible enhancement, about 0.4%, in flexural behaviour. However, it was also found that specimens strengthened with two and three layers explained a debonding failure mode with 29% and 28% enhancement compared to control sample, respectively. Similarly, Ombres (2011) observed that the increase in the ultimate strength of RC beam strengthened with one, two and three layer of PBO-FRCM were 9%, 31% and 34% respectively. The slight enhancement was observed due to the addition of three-layer in comparison with two, however, both specimens failed due to debonding of the strengthening layer. Similar results were obtained by D’Ambrisi and Focacci (2011). It was found that strengthening RC beams with two and three layers of FRCM presented an increase of 27%. While no enhancement was observed with one-layer counterparts.

Contamine et al. (2013) observed that RC beams strengthened with 5mm thickness shear strips of AR-TRM exhibited increasing in ultimate capacity about 31% in comparison with non-strengthened specimens. However, only 35% enhancement was achieved when double the thickness of strips due to the separation of strengthening layer. Similarly, Al-Salloum et al. (2012) found that the shear strength of RC beams strengthened
at the vertical sides with basalt TRM using cement mortar increased to about 36% and 46% for two and four basalt layers, respectively, in comparison with control specimens. Loreto et al. (2015) investigated the shear behaviour of RC beams strengthened with one and four U-wrap of PBO-TRCM for a different class of substrate concrete (28 and 40 MPa compressive strength. It was found in spite the partial delamination of strengthening layer for all specimens, strengthened specimens with 4 layers exhibited about 61% enhancement in comparison with 26% of one-layer counterparts. Elsanadedy et al. (2013) found that the percentage increase in ultimate strength of shear strengthened RC beams with 10 and 5 U-shaped layers of basalt textile mortar were 91% and 39% respectively. It was also observed that as the number of strengthening layer increases the strengthened specimens exhibited debonding failure mode. On the other hand, Al-Salloum et al. (2012) found that the difference in enhancement of ultimate strength of RC beam strengthened for shear was no more than 10% when use four basalt textile fibre mortar instead of two.

Similarly, Tetta et al. (2015) reported a noticeable difference in enhancement of RC beams strengthened with one and two carbon TRM layers. The results of shear strengthening of RC beam without shear reinforcement using one and two layers of side-bonding and one and two U-wrapped configuration were 9%, 71%, 51% and 132%, respectively. All strengthened specimens failed due to debonding of the strengthening layer in spite the high increase in ultimate load which attributed to the absence of shear reinforcement of control specimen. In the same field, Ombres (2015) found that the shear bond strength between PBO-FRCM and substrate concrete of single layer was about 40% of double layer counterparts.

It can be concluded from the above studies that a significant enhancement in strengthening for members strengthened with two layers in comparison with one layer counterparts depending on the mechanical properties of the textile fibres. This difference in improvement may decrease with the increase the number of layers depending on the substrate capacity and strengthening configuration. At the same time, the effectiveness of the number of layers depends mainly on the adhesive properties of the matrix and the mechanical properties of the fibres. Besides, it worth to note that failure mode of the strengthening layer bounds observed enhancement due to increasing the amount of fibres.
2.3.5.3 Textile fibre properties

The mechanical properties of the textile fibres govern the performance of the TRM strengthening layer by means of the maximum tensile strength and modulus of elasticity of the fibres. Si Larbi et al. (2012) reported that the enhancement in strength of RC beam strengthened with 3 layers of AR-glass TRC was about 28% in comparison with control specimen. The strengthened specimens explained a partial peeling of the strengthening layer at the mid-span of the beam. In a similar study, A. Di Tommaso et al. (2008) reported that two PBO-FRCM strengthening layer explained about double enhancement (29%) in the flexural behaviour of RC beams strengthened with carbon- FRCM counterparts. Both strengthening layers exhibited debonding failure mode. Ombres (2011) used Polypara-phenylene-benzo-bisthiazole (PBO) fibres to compose fibre reinforced cementitious mortar (FRCM) to strengthening reinforced concrete beams. It was found that in spite of using high strength fibers with 5800Mpa tensile strength and 270Gpa modulus of elasticity, the average enhancement in the ultimate strength of the strengthened beams was about 34%. In a similar study, D’Ambris and Focacci (2011) found that the improvement of strengthening RC beams using PBO fibres (31%) was double of carbon fibres counterparts (16%). That was due to the difference of the mechanical properties, where the tensile strength and modulus of elasticity of PBO fibres were higher than carbon fibres.

In contrast, Elsanadedy et al. (2013) achieved 91% increase in the ultimate flexural strength of RC beam strengthened for shear with ten U-shaped basalt textile mortar. The mechanical properties of the basalt fibre were 623Mpa tensile strength and 31.94 Gpa modulus of elasticity. Similar observation of the effectiveness of fibres with a low modulus of elasticity was reported by Tetta et al. (2016). It was found that textile fibres with a low modulus of elasticity can exhibit higher strengthening improvement than fibres with a high modulus of elasticity. The study included investigated strengthening deficient part in shear (without shear reinforcement) of T-beams using heavy carbon, light carbon and glass TRM U jackets. All TRM types showed debonding failure mode, TRM with glass fibres (75 GPa modulus of elasticity) exhibited 75% improvement in comparison with control specimens. However, samples strengthened with heavy and light carbon (225 GPa modulus of elasticity) showed 37% and 45% improvement, respectively. However, Pellegrino and D’Antino (2013) investigated the behaviour of strengthened prestressed concrete beams with steel fibre net and carbon fibre reinforced cementitious matrix (FRCM). It was found a similar enhancement in ultimate strength of both strengthening fibres due to the debonding of the strengthening layer. Tsesarsky et al. (2013) investigated the behaviour of plain concrete slabs strengthened with three different layers of textile reinforced mortar. It
was found that specimens strengthened with carbon fibres explained 21kN flexural ultimate load, while AR-glass and polyethene counterparts failed under 9.5 and 2.5 kN respectively. However, Escrig et al. (2015) found that the adherent bond between strengthening TRM layer and RC beams can be more decisive than fibre type on the shear behaviour of strengthened specimens. The increasing in ultimate load due to strengthening with U-shaped glass, basalt, carbon and PBO TRM single layer were 36%, 31%, 36% and 43%, respectively. It was also observed that in spite of the tensile strength of PBO fibre is about twice glass fibre, the difference in increasing the ultimate load is only 7%. In addition, carbon and glass fibres exhibited the same enhancement whereas the tensile strength of glass fibres is about 60% of carbon fibres.

A similar observation was reported by Jabr et al. (2017). It was found that U jackets PBO-FRCM strengthening layer explained enhancement in flexural strength of RC beams ranged between 25 to 33% depending on the original flexural strength of the beams. However, the ultimate improvements in strength were about 4% with glass and carbon fibres FRCM. Escrig et al. (2017) found that the efficiency of fibres in enhancing the flexural strength of RC beams strengthened with FRCM was depending mainly on the type of the fibres (modulus of elasticity) and secondly on the tensile strength of fibres. FRCM with steel grid improved the bearing capacity to about 21% while FRCM with carbon fibres which have higher tensile strength increased the capacity to about 6%. Recently, Raoof et al. (2017) found that strengthening layer with basalt fibres can explain the higher improvement in the ultimate capacity of RC beams than glass fibres counterparts as a result of the higher tensile strength of basalt fibres. In addition, it was found that coated fibres can increase the contribution of the strengthening layer because of improving the bond of fibres with mortar. Azam et al. (2017) found that coated the carbon fibre grid with epoxy improve the shear enhancement of carbon FRCM strengthening layer for RC beams. That was because of increasing the bond between the fibres and the surrounding mortar where the vertical tows provide an anchorage to the grid and resist the slippage.

It can be stated that the mechanical properties of the fibres have a significant impact on the strengthening enhancement. In addition, from the above studies, it can found there is a controversial information about the effect of the mechanical properties, in particular, the modulus of elasticity, on the performance of the strengthening layer. Some researchers demonstrated that textile fibres with low mechanical properties (for instance modulus of elasticity less than 75GPa) could exhibit higher performance than fibres with high mechanical properties (modulus of elasticity higher than 225GPa), while others prove the opposite.
2.3.5.4 Mesh size of textile reinforcement

The stress distribution in the mortar of TRM influences by the mesh configuration of the textile fibres. In addition, the mesh size affects the penetration of mortar between the filaments, which in turns could reduce the bond strength of fibre-mortar in case of fine mesh, and low flowability of mortar. Blanksvård et al. (2009) found that the textile opening with dimension 42x43 mm presented higher increase in ultimate strength of beams strengthened with mineral based composite (MBC) compared with 24x25mm and 70x72mm counterparts. Similarly, D’Ambrisi and Focacci (2011) found that the 20 mm opening size of PBO fibres increased the ultimate strength of RC beams to about 37% while 28% was observed for PBO fibres with 10mm mesh size. On the other hand, D’Ambrisi et al. (2013) observed a slip between carbon fibre nets and the matrix of masonry walls strengthened with carbon-FRCM and subjected to direct shear. The slip was because of insufficient interlocking between the carbon fibre and the matrix due to use carbon nets with about 6mm free space between rovings.

In the same area, Bernat-Maso et al. (2014) found that in spite of using textile fibre with higher tensile capacity, basalt fibre, than textile glass fibre, the late explained higher enhancement to the ultimate moment capacity of masonry wall strengthened with TRM. That was because the size of the opening of the textile fibres was 25x25mm while for basalt was 10x10mm which led to increasing the interlocking between the textile and the cementitious matrix.

It was found that the size of fibre openings crucially affects the efficiency of strengthening because of their influences on the mechanical interlocking between mortar and the fibres. Also, it can be concluded that high workability of mortar is required to ensure adequate penetration of mortar inside the textile fibres mesh.

2.3.5.5 Cementitious matrix strength

Properties of TRM mortar, in particular, tensile strength, affect the performance of the TRM composites by means of controlling the cracking and the chemical adhesion bond with textile fibres. Gopinath et al. (2011) reported that a cementitious matrix consisting of silica fume and fly ash with compressive strength equal to 56 MPa could be a comparable binder of plain concrete cylinders strengthened with AR-glass fibre mesh using an organic adhesive (epoxy).

Similarly, Al-Salloum et al. (2012) observed that polymer modified mortar matrix exhibited higher enhancement, about 10%, in comparison with cement mortar of RC beams.
strengthened for shear using TRM of basalt fibres. The higher increase may be attributed to the higher mechanical properties of polymer-modified mortar (56Mpa compressive strength) in comparison with cement mortar (23 MPa compressive strength) counterparts. The similar superiority of polymer-modified mortar in comparison with cement mortar was reported by Elsanadedy et al. (2013).

In the same field, Garcia et al. (2010) found that cement-based mortar explains higher strengthening enhancement, 32%, than Pozzolana mortar counterparts (24%). The study included plain concrete columns strengthened with basalt textile reinforced mortar. In a similar study, Al-Salloum et al. (2012) observed 36% enhancement of the ultimate capacity of RC beams strengthened with two layers of basalt textile reinforced cementitious and polymer modified cementitious mortar. With four strengthening layers, polymer modified cementitious mortar explained 58% increase in ultimate capacity in comparison with 46% of cementitious mortar counterparts. Jabr et al. (2017) found that FRCM strengthening layer with carbon and glass fibres explained a negligible enhancement in flexural strength of RC beams because of using mortar with 20MPa compressive strength, which led to explain low bond with the substrate RC beams, and debonding during the loading. Wu and Li (2017) found that using engineering cementitious mortar, which has PVA fibres, improved the bond strength between CFRP and RC beams.

In summary, the ordinary Portland cement mortar can provide similar impacts to other types of mortar such as modified mortar and modified polymer mortar. In addition, the efficiency of the mortar in binding fibres depends mainly on the mechanical properties of the mortar, and higher strength mortar exhibits better enhancement of strengthening.

2.3.5.6 Influence of using Anchors

Based on the debonding observation of TRM layers, which indicates insufficient interface bond strength, some researchers investigated using anchors to improve the interface bond strength. For instance, A. Di Tommaso et al. (2008) examine the effectiveness of applying U anchor shape of PBO-FRCM at the ends of RC beams strengthened with three layers of PBO-FRCM in comparison with full U anchor layer along the specimen. It was found both anchor configuration failed to prevent the separation of strengthening layer; however, samples with full U anchor explained double increasing in ultimate load. That may be attributed to decrease the stresses at the interface due to higher contact (interface) area that provided by full U anchor. In the same field, Tetta et al. (2015) observed a significant enhancement in the capacity of RC beams strengthened with single carbon TRM layer for
shear purpose. Three strengthening configurations were considered; side-bonding, U-wrapped and fully wrapped. In spite of the observed partial debonding of the strengthened specimens, side-bonding, U-wrapped and fully wrapped exhibited increasing in ultimate load about 9%, 51% and 115%, respectively, compared with the control beam. However, Bernat et al. (2013) found that using carbon fibre anchors decreased the ultimate strength of masonry walls strengthened with textile carbon mortar. The enhancement in ultimate strength of specimens without anchors was 116%. While, when using 6 and 9 carbon fibre anchors the increase were 96% and 106% respectively. In the same field, Bournas et al. (2015) investigated the efficiency of using carbon fibre spike anchors to improve the bond strength of TRM strengthening layer with substrate RC columns. The study included direct pull off test of the strengthened connection region of RC columns with footings. It was observed that all anchors failed in debonding failure mode and as the area of anchor increased the tensile bond capacity increased. The obtained tensile bond capacity of 30.67mm² and 67.2mm² anchors area were 9.59 and 14.6 kN, respectively.

In a similar study, Tetta et al. (2016) investigated the effectiveness of application textile – based anchors in improving the performance of U jackets shear strengthening of T beam using TRM, see Figure 2.12. The study included shear strengthening of deficient part (without shear reinforcement) of T beams using heavy carbon, light carbon and glass TRM. The results showed that all anchors failed either due to pull off or rupture. The enhancement in strength in comparison with specimens without anchors of heavy carbon with 14 anchors, heavy carbon with 30 anchors, light carbon with 14 anchors, and glass with eight anchors equal to 23%, 64%, 31%, and 6%, respectively. However, Gonzalez-Libreros, Sneed et al. (2017) observed that the effect of using spike aramid anchors with carbon and steel FRCM strengthening layers increase the enhancement of the ultimate bearing capacity of RC beams from 18% to 20%. That was due to the delamination of the mortar from the anchors. In spite of the achieved enhancement due to using carbon anchors but it is still insufficient to provide adequate bond between the substrate concrete and TRC.

![Figure 2.12: Textile based anchors of T-beams strengthened with TRM (Tetta et al., 2016)](image)
In the same field, Younis et al. (2017) investigated using mechanical anchors to improve the bond of the interface between the substrate RC beams and FRCM strengthening layer in shear. It was observed that the application of anchors improves the ultimate capacity of the strengthened member, but it does not prevent the debonding of the strengthening layer. The enhancement in ultimate capacity was varied depending on the properties of FRCM. Specimens with carbon and PBO fabric composite explained the low effectiveness of using anchors (improvement was ranged between 1 to 2.4%). However, samples with glass fibres exhibited improvement ranged between 7 and 30%. That was because the mortar strength of glass fibre specimens was higher than carbon and PBO counterparts.

Other researchers investigated the effectiveness of using anchors at the end of strengthening layer to prevent the debonding in the form of U-shape. Escrig et al. (2017), for instance, found that using U-jacket of carbon FRCM anchorage at the ends of RC beam strengthened with FRCM layers improves the performance of the strengthened member with one layer of textile fibre. However, no evidence is available about the efficiency of this configuration for a higher number of textile fibres layers. The maximum achieved improvement of this method was about 21% in comparison with the control beam.

Raoof et al. (2017) investigated using U-jacket carbon fibre TRM anchorage at the ends of RC beams strengthened with glass textile reinforced mortar. The results explained the improvement in the ultimate bearing capacity from about 25% for specimens without jackets to about 65% of specimens with jackets, but the debonding of the strengthening layer was observed. It is interesting to note that the control beams were reinforced with low longitudinal reinforcement ratio which led to demonstrate higher enhancement due to strengthening. However, Alabdulhady et al. (2017) found that full confinement of the FRCM strengthening layer of RC beam can improve the enhancement of the ultimate torsional capacity from 8% for three side strengthening configuration to 30%. That was due to reducing the slippage of the textile fibres. However, this configuration is considered difficult to apply in practice especially if the beam is part of RC slab.

Based on the above studies, it can be stated that implemented anchors at the interface improve the interface bond strength. However, that improvement was in most cases insufficient to prevent the debonding of the strengthening layer.
Table 2.3: Summary of the previous studies of flexural strengthening of RC beams with textile cementitious materials

<table>
<thead>
<tr>
<th>Authors</th>
<th>Fibres type</th>
<th>$F_{ty}$ (MPa)</th>
<th>Layers No.</th>
<th>Matrix strength (MPa)</th>
<th>Strengthening configuration</th>
<th>Anchors</th>
<th>Strength improvement (%)</th>
<th>Failure mode</th>
</tr>
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<tbody>
<tr>
<td>A. Di Tommaso et al. (2008)</td>
<td>PBO</td>
<td>5800</td>
<td>1</td>
<td>28</td>
<td>Bottom</td>
<td>U-jacket</td>
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<td>Debonding</td>
</tr>
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<td></td>
<td>PBO</td>
<td>5800</td>
<td>2</td>
<td>28</td>
<td>Bottom</td>
<td>U-jacket</td>
<td>29</td>
<td>Debonding</td>
</tr>
<tr>
<td></td>
<td>PBO</td>
<td>5800</td>
<td>3</td>
<td>28</td>
<td>Bottom</td>
<td>U-jacket</td>
<td>28</td>
<td>Debonding</td>
</tr>
<tr>
<td>Ombres (2011)</td>
<td>PBO</td>
<td>5800</td>
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<td>29</td>
<td>Bottom</td>
<td>--</td>
<td>9</td>
<td>Debonding</td>
</tr>
<tr>
<td></td>
<td>PBO</td>
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<td>2</td>
<td>29</td>
<td>Bottom</td>
<td>--</td>
<td>31</td>
<td>Debonding</td>
</tr>
<tr>
<td></td>
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<td>5800</td>
<td>3</td>
<td>29</td>
<td>Bottom</td>
<td>--</td>
<td>34</td>
<td>Debonding</td>
</tr>
<tr>
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<td>1102</td>
<td>3</td>
<td>--</td>
<td>Bottom</td>
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<td>28</td>
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<tr>
<td>Jabr et al. (2017)</td>
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<td>20</td>
<td>U shape</td>
<td>--</td>
<td>4.2</td>
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<tr>
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<td>20</td>
<td>U shape</td>
<td>--</td>
<td>0.9</td>
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<tr>
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<td>U shape</td>
<td>--</td>
<td>2.3</td>
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<tr>
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<td>U shape</td>
<td>--</td>
<td>33</td>
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<tr>
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<td>1518</td>
<td>5</td>
<td>39.2</td>
<td>Bottom</td>
<td>--</td>
<td>79.8</td>
<td>Debonding</td>
</tr>
<tr>
<td></td>
<td>Carbon-coated</td>
<td>2843</td>
<td>1</td>
<td>39.2</td>
<td>Bottom</td>
<td>--</td>
<td>19.4</td>
<td>Debonding</td>
</tr>
<tr>
<td></td>
<td>Basalt-coated</td>
<td>1190</td>
<td>7</td>
<td>39.2</td>
<td>Bottom</td>
<td>--</td>
<td>35.5</td>
<td>Textile rupture</td>
</tr>
<tr>
<td></td>
<td>Glass</td>
<td>794</td>
<td>7</td>
<td>39.2</td>
<td>Bottom</td>
<td>--</td>
<td>24.9</td>
<td>Debonding</td>
</tr>
<tr>
<td></td>
<td>Glass</td>
<td>794</td>
<td>7</td>
<td>39.2</td>
<td>Bottom</td>
<td>U-jacket</td>
<td>65</td>
<td>Debonding</td>
</tr>
<tr>
<td>Wu and Lie (2017)</td>
<td>CFRP</td>
<td>833.5</td>
<td>1</td>
<td>50.6</td>
<td>bottom</td>
<td>--</td>
<td>1.7</td>
<td>Debonding</td>
</tr>
</tbody>
</table>
2.4 Bond between two cementitious materials

Based on the review of the studies of strengthening RC members with TRM layers, it can be stated that the interface bond strength is one of the critical variables affecting the performance of the strengthened members. Therefore, the bond mechanism, factors affecting bond and interface bond measurements were reviewed.

The bond between substrate concrete and new cementitious layer depends mainly on the adhesive properties of the interfacial zone (interface) between the two layers. This adhesion is a result of chemical forces between the reactive matrix of the new cementitious layer and substrate concrete (Espeche and León, 2011). According to the micro model of Xie et al. (2002), the interfacial zone can be divided into three layers, see Figure 2.13.

The first layer is called penetration layer which consists mainly from of prickly calcium silicate hydrate (C–S–H) with smaller amounts of ettringite in the form of Al2O3·Fe2O3·tri (Aft) or calcium hydroxide (Ca(OH)2). This layer penetrates inside the concrete substrate and reacts with the active chemical components in the old concrete. The second layer, which is considered the weakest layer, is termed as the highly affected layer. It contains Ca(OH)2 and needle-shaped Aft crystals and is characterised by high porosity. The lower affected layer is the third layer, which mostly has the microstructure of the new layer.

Also, it has been found that the addition of supplementary cementitious materials (SCMs) like GGBS, silica fume and fly ash increases the amount of C-S-H gel through their pozzolanic reaction with the Ca(OH)2 (Neville, 2011; Sezer, 2012; Bagheri et al., 2013; Jalal et al. 2015). Accordingly, the mechanical properties of cementitious materials improved due to reducing the pores inside the hardened structure in addition to increasing the chemical adhesion resulted from the additional C-S-H.
A range of factors affect the bond strength, some of them regarding substrate concrete while others related to the repair layer. Assessment method of bond strength is important to investigate the parameters that affect the interface bond strength. In addition, bond strength measurement is required in the theoretical simulation by means of determining the required coefficients that describe the bond strength.

### 2.4.1 Factors affecting bond strength

Many researchers aimed at improving the interface bond strength between old and new concrete. Some of them investigated the effect of the substrate concrete properties while others emphasise the performance of the new layer. The following sections review the main factors affecting the bond strength between substrate concrete and new cementitious layer.

#### 2.4.1.1 Moisture content within substrate concrete

The effect of water (moisture) content of substrate surface on the bond strength comes from the interaction of this water with the new layer. High water content may lead to form a separation layer between substrate concrete and the new layer and/or increase the voids of the interface after cement hydration. Santos et al. (2012) reported 50% reduction in shear bond strength of saturated concrete specimens tested under bi-surface shear test in comparison with dry samples. In addition, it was observed that this reduction was about 88% when using epoxy resin-based as a bonding agent. At the same time, a dry substrate surface may lead to absorption water from the new layer and reduce the cement hydration process.

Júlio et al. (2004) investigated the effect of pre-wetting of the substrate surface on the bond strength between two different age concrete layers. Specimens’ surface was prepared using chipping hammer and put in the water tank for 24 hours before casting the new concrete layer. Slant shear and pull-off tests were applied to examine the shear bond and tensile bond strength, respectively. The results explained a negligible effect of moisture content on the shear bond while the tensile bond strength reduced by about 30% when compared to non-pre wetting samples. From the above literature, the saturated surface dry surface could exhibit higher interface bond strength.
2.4.1.2 Surface preparation method

Many researchers concentrated on investigated the effect of substrate surface roughness on the bond strength between substrate concrete and a new cementitious layer by Xiong et al. (2004). The study included the splitting bond strength of specimens their surface treatment with a hydrochloric acid solution to improve the surface roughness, which in turns improves bond strength. The results showed that the effectiveness of this method depends on the concentration of acid and period of exposure. The ultimate bond strength (2.74) was obtained with 5% hydraulic acid solution and 5-minute etching period. The efficiency of using hydrochloric acid (chemical etching) as a treatment method in comparison with mechanical abrasion was investigated by Ray et al. (2005). It was found that for all specimens the mechanical surface treatment explained higher shear bond strength than chemical etching counterparts.

In a similar study, Júlio et al. (2004) investigated the effect of three different roughing methods; wire brushing, partially chipping and sandblasting on the bond strength of two concrete layers. Specimens with wire brushing and partially chipping explained about 75% and 45% of the shear bond strength of sandblasting specimens, respectively. Santos et al. (2012) observed the similar superiority of sandblasting treatment. They investigated the effect of three surface substrate preparations; wire brushing, shot blasting and left as cast on shear bond strength. The shear bond strength of specimens left as cast and wire brushing were about 38% and 52% of sandblasting treatment counterpart specimens, respectively.

Also, Tayeh et al. (2012) investigated the effect of different surface preparations of the substrate concrete on the bond strength between normal strength concrete and ultra-high performance fibre concrete (UHPFC). The types of surface preparation were as cast without roughing, sandblasting, wire brushing, drill holes and groves. The shear bond strength of as-cast, wire-brushing, grooving and drilling holes were about 47%, 68%, 76% and 68% of sandblasting counterparts specimens, respectively. Courard et al. (2014) found a negligible effect of the substrate concrete compressive strength on the bond with polymer cement layer in their study on the impact of surface treatment methods. They investigated effect of microcracks on the bond strength through four types of surface preparation. The results of the pull-off bond test explained that the highest bond strength was for high pressure (250MPa) water-jetting treatment. While the jackhammering treatment explained the lowest bond strength due to the cracks in substrate concrete from hammering. Momayez et al. (2005) found that the bond strength not only depends on the surface treatment method but also on the roughness level of the same way. For prepared specimens...
using wire brushing treatment different bond strength was obtained due to different roughness levels.

In the same area, Mohamad et al. (2015) found a significant effect of the substrate surface texture on the shear bond strength of the interface of two concrete layers cast at different ages. The study included investigating the effect of five different surface treatments; as cast, grooving, indented, longitudinal and transverse wire brushing. It was found that transverse bushing exhibits the highest shear bond strength (3.46MPa). While specimens without treatments (as cast) explained 0.67 MPa shear bond strength.

It can be concluded from the above studies that the surface roughness has a significant effect on the bond strength. In addition, based on the achieved bond strength, the best treatment can be obtained with sandblasting treatment, which provides sufficient roughness.

2.4.1.3 Mechanical properties of the cementitious layer

The mechanical properties of the repair material and their compatibility with the substrate concrete have a significant effect on the bond strength. Morgan (1996) reviewed the required compatibility between the repair material and the existing substrate concrete. The review included different types of repair materials and their properties. It was found that to provide structural compatibility between the substrate concrete and the repair layer, mechanical properties of the repair material should be equal to or greater than the substrate concrete.

Gohnert (2003) found that as the compressive strength of the new concrete layer increases the horizontal shear bond strength increases. The results demonstrated that the shear bond strength of specimens with 22 MPa compressive strength of the new layer exhibited about 79% of the 31MPa compressive strength of specimens counterparts. In addition, the same effect of the compressive strength on the ultimate load was observed for specimens tested under flexural load.

A similar effect of the new layer strength was observed by Júlio et al. (2006). It was investigated the impact of the new layer strength on the slant shear bond strength between new and old concrete. Three concrete compressive strengths were examined: 30/30 MPa, 30/50 MPa and 30/100 MPa. Specimens with 100MPa strength of new layer exhibited increasing in bond strength about 25% in comparison with 30MPa compressive strength counterparts. A similar study, Ray et al. (2005) found that the enhancement in bond strength due to use high strength repair material depends on the components of the repair
material and not only the value of compressive strength. It was found that repair material containing silica fume and fly ash with compressive strength 49.2 MPa presents higher bond strength than materials with compressive strength 60.7 MPa containing fibres. In the same field, Tayeh et al. (2013) investigated the bond strength between ultra-high performance fibre concrete (UHPFC) with compressive strength 170 MPa as a repair material and normal strength concrete. For sandblasting surface preparation, the average achieved the bond strength of the splitting bond and slant shear tests were 3.8 and 17.81 MPa, respectively. The effect of shrinkage of the repair material was reported by Qian et al. (2014). It was found that magnesium phosphate cement mortar present higher bond strength than ordinary Portland mortar because the later explained cracks in the interface zone due to shrinkage.

It can be concluded as the compressive strength of the repair material increases the bond strength increases. The increase in bond strength when using high compressive strength materials is due to either use high cement content which is considered responsible for the chemical adhesion bond strength or to use pozzolanic materials which react with the resulted Ca(OH)\(_2\) from the cement hydration to produce an additional C-S-H gel. Also, the shrinkage of the repair material should be considered to avoid cracks which reduce the bond strength.

### 2.4.1.4 Pozzolana materials effect

The concept of introducing Pozzolana materials to the repair materials is based on their efficiency in reaction with the weak part of cement hydration results (Ca(OH)\(_2\)). The effectiveness of addition Pozzolana materials depends on the equilibrium between their concentration and Ca(OH)\(_2\) content in the matrix.

Pu-Woei Chen (1995) observed that shear bond strength of two mortar layers in the presence of silica fume increased from 0.39 to 0.89 MPa. In the same field, Kuroda et al. (2000) investigated the bond effect of addition silica fume and fly ash on the bond strength between old and new cement past under direct tensile stresses. It was found that silica fume is more effective in increasing the bond strength than fly ash. Superiority may be attributed to the high content of SiO\(_2\) in silica fume, which produces more C-S-H and in turns improve the bond strength.

Similarly, Xiong et al. (2002) investigated the effect of addition fly ash to the mortar binder. The results were compared with other four types of the binder; neat cement paste, expansive paste, cement mortar and water-dispersible epoxy resin. It was found that the
highest splitting bond strength was observed with fly ash modified mortar. In contrast, Li (2003) found that the addition of fly ash to the repair material decrease the bond strength with ordinary concrete at the age of 28 days. It was found a slight increase in bond strength at one-year age. In another study, Xiong et al. (2004) found that the use of fly ash in the binder between new and old concrete increases the splitting bond strength about 15% of binder without fly ash specimens. Similar enhancement due to the addition of fly ash and silica fume was observed by Ray et al. (2005). It was found that the direct shear bond strength of repair concrete material contains silica fume, increased from 1.8 to 2.4Mpa with the addition of fly ash.

In the same field, Momayez et al. (2005) found that the addition of silica fume with replacement ratio 7 to 10% to the repair concrete layer improves the bond strength up to 15% in comparison with specimens without silica fume. While, Mohammadi et al. (2014) found that the addition of silica fume and metakaolin type A which have high SiO2 and low CaO content to the new layer decreased the bond strength. Also, the addition of metakaolin type B that has higher CaO content presented a 17% increase in bond with a replacement ratio 10%. While above or below this ratio the bond strength decreases. It was found that CaO has more influence on the bond strength than SiO2.

Zanotti et al. (2014) studied the slant shear bond strength of concrete specimens that contain fly ash with different contents of Polyvinyl Alcohol (PVA) fibres. The results explain that the addition of fibre increases the bond strength. The shear bond strength without fibre was 21.65 MPa while after addition 0.5% and 1% fibre the strength was 24 and 25 MPa respectively.

It can be concluded that the addition of pozzolana material has a variable influence on the bond strength. This influence may depend on replacement ratio and the way of addition to the repair material or the binder. No conclusive agreement can yet be found in literature about the effectiveness of supplementary cementitious materials (SCMs) (such as GGBS, silica fume and fly ash) on the bond strength, hence needs further investigation.

2.4.1.5 Polymers and additives effect
Some researchers investigated the effectiveness of addition expansive agents and polymers in the repair materials to increase the adhesive bond and to reduce the shrinkage of these materials. That will, in turn, increase the bond strength between the substrate concrete and the repair material.
Pu-Woei Chen (1995) found that the addition of short carbon fibre to the repair mortar could increase the shear bond strength between two cementitious mortar layers up to 89% in comparison with mortar without fibres. Their investigation of the drying shrinkage explained that the carbon fibre reduced shrinkage about 25%, which in turns improve the bond strength through reducing the initial cracks between the two layers. Xiong et al. (2002) found that the use of expansive cement paste binder explained higher slant shear bond strength than normal mortar and cement paste. In another study, On the other hand, Li (2003) found that the addition of the expansive agent to the concrete enhanced the short-term bond strength between new and old concrete but decreased the long-term bond strength.

In the same field, Cabrera and Al-Hasan (1997) observed that the slant shear bond strength of polymer-modified mortar was less than ordinary Portland cement counterpart. A debated observation was presented by Ray et al. (2005). It was observed through the experimental direct shear test of bond strength between two concrete layers, that the addition of latex to the new layer presented a perfected bond and made the failure in the substrate concrete. In contrast, Pellegrino et al. (2009) observed a debonding failure of repaired short reinforced concrete columns with reinforced and unreinforced polymer-modified cementitious mortar. The results also explain that the longitudinal reinforcement improves the contribution of the repair layer to the ultimate strength. The adverse effect of addition polymer was also observed by Mohammadi et al. (2014). It was found that the addition of latex (polymer) to the binder decrease the bond strength between two concrete layers.

To sum up, the addition of the expansive agent to the repair material enhanced the short-term bond strength while more investigations are required to demonstrate its long-term effect. In spite of the controversial impact of addition polymers to the repair material, it can be concluded as the mechanical properties of mortar increases the bond strength increases.

2.4.1.6 Binders effect

Different binder types were used to enhance the bond strength between old and new concrete. Xiong et al. (2002) found that in addition fly ash to the binder, improved splitting bond strength (3.21Mpa) between two concrete layers can be achieved. In a similar study, Li (2003) observed a negligible effect on the bond strength between old and new concrete when addition fly ash to new concrete. Xiong et al. (2004) investigated three types of
binders, cement mortar, expansive cement paste and fly ash mortar. The enhancements in splitting bond strength of these binders were 7%, 16% and 23% respectively. In the same field, Santos et al. (2012) found that the effectiveness of using an epoxy resin-based bond coat in improving the bond strength depends on the substrate surface roughness. The results showed that the bond shear strength of specimens without using bonding agent was about 48% for the smooth surface and 77% for rough surface of samples with bonding agent counterparts.

It can be concluded that the addition binder enhanced the bond strength between two hardened concrete layers. However, its efficiency fluctuates with repair the substrate concrete with a cementitious mortar. According to the reviewed parameters that are affecting bond strength between old and new cementitious materials, it can be stated that surface preparation and mechanical properties of the new layer dominate the performance of bond strength.

### 2.4.2 Measurement of bond strength

In general, bond tests of the interface between two cementitious layers can be classified into three categories. The first category measures the bond under tension stress state, which represents the adhesion coefficient. Pull-off and splitting tests represent the common tests under this category. In the pull-off test, Figure 2.14, a disc is usually bonded by epoxy resin to a prepared testing surface, and the disc is pulled off after a partial core has been cut around the disc. The bond strength is obtained by dividing the pull-off force on the cross-sectional area of the partial core. This test is widely used to assess the bond strength in either the field or laboratory (Li, 2003, Júlio et al. 2004, Momayez et al. 2005).

![Figure 2.14: Pull-off test (Li, 2003)](image)
Circular or square cross-section specimens are tested under compressive force for splitting test, see Figure 2.15. At the ultimate load, specimen splits into two halves due to the tensile stresses passing through the interface (Xiong et al. 2004).

![Figure 2.15: Splitting bond strength](image)

The second category measures the bond under pure shear stress state, which can represent the cohesion coefficient. Push-off and bi-surface triple surface shear tests are the main tests fall under this category (Gohnert, 2003; Santos et al., 2012). Specimens for these tests are subjected to direct shear stress as shown in Figures 2.16 and 2.17.

![Figure 2.16: Push-off test of composite two concrete layer](image)

![Figure 2.17: Bi-surface shear test](image)

The third category measures the bond strength under combined shear and compression stresses state. Slant shear test is applied to assess the bond strength for this category. Slant shear test of a square prism or a cylindrical sample made of two identical halves bonded at 30° and tested under axial compression was used, as shown in Figure 2.18. This test is
Sensitive to the surface roughness and significantly depends on the angle of the interface plane.

Slant shear test (Figure 2.18) has become the most widely accepted test and has been adopted by some international codes for evaluating the bond of resinous repair materials to concrete substrates (BS EN 1215:1999; ASTM C1042-1999). However, there is no general agreement among researchers of the suitability of this test for non-resinous materials. Such this test may be suitable to assess the bond between substrate concrete and overlay, where the interface under combined shear and compression.

Momayez et al. (2005) investigated the bond between old concrete and new cementitious layer using four tests methods; pull-off, splitting tensile, bi-surface shear and slant shear tests. The highest values were observed for slant shear test due to the effect of compressive stress in increasing the bond strength. It was also found a high consistency between the pull-off test and splitting tensile test. In the same field, Júlio et al. (2004) found a negligible effect of surface moisture content on the shear bond strength of two concrete layers tested under shear slant test. While for pull-off tested, a reduction in bond strength up to 30% was observed.

The bond strength of the first and second category can be determined based on the ultimate load that can be applied to the composite specimen and the area of the interface, according to the following equation:

\[
  f_b = \frac{P}{A_c} \tag{2.1}
\]

Where: \( f_b \) is the bond strength (MPa), \( P \) is the direct tensile applied load (kN) and \( A_c \) is the interface area (m\(^2\)).
For the third category of the bond test method, the shear and normal strength can be defined by analysis the stresses at the inclined surface and the area of the interface, according to the following formulas (Austin Simon, 1999):

\[
\sigma_o = \frac{p}{A} \quad (2.2)
\]

\[
\sigma_n = \sigma_o \sin^2 \alpha \quad (2.3)
\]

\[
\tau_n = \frac{1}{2} \sigma_o \sin 2\alpha \quad (2.4)
\]

Where: \(\sigma_o\) is normal axial stress, \(A\) is the cross-section area of specimen, \(\sigma_n \) and \(\tau_n\) are normal and shear stresses at the interface, respectively, and \(\alpha\) is the inclination angle of the interface. The relationship between the shear and normal stresses under compression can be determined using Coulomb theory (Austin Simon, 1999) according to following formula:

\[
\tau_n = C + \mu \sigma_n \quad (2.5)
\]

As a result, for the same materials properties of substrate, interface and addition layer, the value of bond strength varies according to the bond test method. The stress state of the interface controls that under loading. In addition, in the field of strengthening structures with the new cementitious layer, the stress state of the interface may change during the loading due to initiation of cracks and redistribution of stresses near the interface. Therefore, it is essential for theoretical simulations to consider all bond coefficients. To achieve that, two or more bond test methods are required. The most suitable bond tests method are the pull-off test to determine the adhesion coefficient and slant shear compression test to define the cohesion and friction coefficients. However, other types of bond test such as direct shear and slant shear tension test can be used to demonstrate the validation of the obtained bond coefficients from the first group.
2.5 Corrosion of steel reinforcement

Hydration products of cement in RC structures provide a protection layer for embedded steel reinforcement through covering the reinforcement with a thin protective oxide film (the passive film). This layer represents thermodynamical stable compounds of iron due to high alkaline pore solution (pH between 13 and 13.8) as a result of cement hydration.

The service life of RC structures can be divided into two phases; initiation of corrosion and propagation of corrosion phases, see Figure 2.19, (Luca Bertolini et al. 2013). The initiation stage ends by the carbonation or chloride penetration in the concrete cover which leads to the destruction of the passive layer of steel reinforcement and starts corrosion. Propagation phase begins when the steel reinforcement is de-passivated.

Depassivation of steel reinforcement usually occurs due to Carbonation and or penetration of chloride ions. Carbonation take places due to the presence of CO₂ at the surface of concrete and moving gradually inside inner zones of concrete which in turns leads to decrease the alkilanity (pH) and destroy the passive layer of steel reinforcement. Similarly, penetration of chloride ions into parts near reinforcement breakdown the passivity of steel reinforcement.

Carbonation of concrete leads to complete termination of the passive layer, which leads to corrosion the whole area of steel in contact with the carbonated carbon (leads to general corrosion). Limited Chloride penetration induces a local destroying of the passivity of steel reinforcement which leads to producing local corrosion (pitting corrosion). While high chloride concentration may lead to damage the passivity of a vast area of steel reinforcement and induce general corrosion (Luca Bertolini et al., 2013).

Figure 2.19: Initiation and propagation period of corrosion in RC structures (Luca Bertolini et al. 2013)
The duration of the initiation phase depends on the depth of cover and the penetration rate of the depassivation agents (CO$_2$, chloride ions). This penetration is highly influenced by the properties of concrete cover (permeability) and environments conditions (temperatures, humidity, cycles of wetting and drying). In the propagation phase (after depassivation), corrosion occurs in the presence of water and oxygen at the surface of steel reinforcement. Depending on the level of corrosion, different deterioration forms occur such as loss of reinforcement cross-section, cracking of concrete cover, spalling and delamination of concrete cover and collapse of RC structures (Luca Bertolini et al., 2013). It was observed that when the pH of concrete reduces to 9.5 (carbonation) or when the chloride concentration near the steel reaches 0.1% of the concrete mass, depassivation begins and the propagation of corrosion start (Dekoster et al., 2003; Belarbi and Bae, 2007).

The general reaction of the corrosion process is:

$$\text{Fe} + \text{H}_2\text{O} + \text{O}_2 \rightarrow \text{Fe(OH)}_2 / \text{Fe}_3\text{O}_4 / \text{Fe}_2\text{O}_3$$

(Steel + Water + Oxygen → Iron oxides/rust)

Corrosion is qualitatively described in terms of corrosion rate which represents the loss of metal of steel per unit time (Bhargava et al. 2007). Corrosion rate usually expressed in terms of penetration of corrosion with time (μm/year). Since corrosion of steel reinforcement embedded in concrete is an electrochemical process requiring an anode, a cathode, and an electrolyte solution (Spainhour and Wootton 2008). Therefore, electrochemical units (μA/cm$^2$) usually used to describe the rate of corrosion.

The adverse effect of corrosion on the performance of RC structures is not only attributed to the reduction in the reinforcing strength. But also associated to deterioration of concrete due to the stresses caused by corrosion products as a result of increasing the reinforcement volume from 2 to 6 times greater than that of original iron, depending on their composition and the degree of hydration (Luca Bertolini et al., 2013). Concrete cracks when these stresses exceed the ultimate plastic deformation of concrete. These cracks lead to weakening the region surrounding steel reinforcement which in turns reduce the bond strength between concrete and reinforcement and affect the strength and performance of the RC structure (Cabrera, 1996; Bhargava et al., 2007).
2.5.1 Effect of corrosion on behaviour of RC members

Many researchers investigated the effect of degree of corrosion on the behaviour of RC members. In this section, a brief review of the previous studies was conducted to demonstrate the relationship between the reduction in the performance of the corroded member and the amount of corrosion.

Almusallam et al. (1996) observed that the bond strength between corroded steel bars and concrete increased about 16% in the pre-cracking stage (degree of corrosion 0-4%). In post cracking stage, bond decreases with a sharp reduction in ultimate bond strength beyond 6% degree of corrosion. Similarly, Cabrera (1996) found a reduction in bond strength of RC specimens with corrosion level more than 2%. However, Fang et al. (2004) observed that the breakpoint was for corrosion levels of around 2–4%. In addition, it was found that at bond strength decreased rapidly as the corrosion level increased. For instance, bond strength at 9% corrosion was only one-third of that of non-corroded specimens.

Abosrra et al. (2011) found a significant effect of the concrete compressive strength on the propagation of corrosion steel reinforcement in concrete. It was found with increasing the compressive strength from 20 to 46 MPa, and the corrosion rate can be reduced to about 40% for high corrosion rate.

El-Maaddawy et al. (2014) found a negligible reduction in flexural strength of RC beams subjected to accelerated corrosion with 6% degree of corrosion. However, 21% reduction in ultimate capacity was observed with the degree of corrosion equal to 18%. Lachemi et al. (2014) observed that the reduction in shear strength of corroded RC beams depends on the level of corrosion. It was found 17% increase in shear strength of RC beams with corrosion level less than 6%. However, high corrosion level, 20%, exhibited 68% reduction in ultimate shear capacity with debonding failure mode.

Wang et al. (2015) found a slight increase in shear strength of corroded RC beams when the corrosion level less than 6%. However, high corrosion level (36%) of shear reinforcement caused a reduction in the shear strength provided by stirrups equal to 82.2% and the reduction in total shear strength of beam was about 17%.

Based on the above literature, it can be noted that degree of corrosion less than 4% has a non-considable effect on the performance of the corroded members and the reduction in performance appear after 6%. 

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2.5.2 Effect of corrosion on the behaviour of strengthened members

The concrete cover is considered the weakest part of the affected region. In the field of strengthening, an additional type of stresses are applied to substrate concrete and these stresses accompanied with stresses resulted from the corrosion lead to the separation the concrete cover and losing the enhancement of the strengthening layer. That demonstrates the importance of investigating the effect of degree of corrosion on the performance and failure mode of strengthening members.

Green et al. (2006) reported that the full wrapping of RC columns with CFRP layers decrease the rate of corrosion due to reduce the further ingress of water, chloride, and oxygen. In a similar study, Belarbi and Bae (2007) observed that the rate of corrosion significantly decreased from 4.51 g/day of unwrapped RC columns to 1.55 g/day of RC columns wrapped with CFRP. It was also observed about 66% reduction in ultimate compressive strength of the strengthened specimens in the presence of corrosion, and the failure mode changed from rupture the CFRP of non-corroded columns to concrete crushing for corroded counterparts.

El Maaddawy (2008) found that the enhancement due to repair corroded RC columns with CFRP composites depends on the amount of wrapping and eccentricity of loadings. Fully strengthened specimens with eccentricity (e/h) equal to 0.3 exhibited about 40% increase in capacity in comparison with control specimens, while 28% was observed for partially wrapped counterparts. However, only 5% enhancement obtained from wrapping with CFRP when the e/h equal to 0.86. In addition, it was found that full wrapping with CFRP reduced the loss in steel mass due to corrosion to about 28% in comparison with controls specimens, while a negligible effect on corrosion level was observed for partially raped samples.

Similarly, Spainhour and Wootton (2008) observed that the reduction of corrosion level of steel bars embedded in concrete depend on the amount and epoxy type of CFRP wrapping composites. Similarly, Gadve et al. (2009) reported that glass fibre reinforced polymers was more effective than carbon in reducing the rate of corrosion of cylinder specimens wrapped with CFRP and GFRP. The superiority of glass fibre was attributed to the high electrical resistivity or high thickness of glass fibre in comparison with carbon fibre. Masoud and Soudki (2006) observed a 16% reduction in corrosion of RC beams strengthened with U-shape glass fibre reinforced polymer (GFRP).

El-Maaddawy et al. (2014) observed a fluctuation in strength reduction of RC beams strengthened with a hybrid composite of carbon and glass polymer fibres in the presence of
different steel corrosion level. Tests results explained the reduction of strength were 10, 13 and 8% for specimens with corrosion level equal to 6, 11 and 18%, respectively. Also, it was found a slight effect of addition steel fasteners on the behaviour of strengthened specimens in the presence of corrosion.

In the field of strengthening RC members with TRM in the presence of corrosion, limited studies were recently conducted. Yin et al. (2017) investigated the flexural behaviour of RC beams strengthened with TRC layers and subjected to the combined action of loading and marine environment. The marine environment imposed in the form of chloride wet-dry cycles and freeze-thaw cycles. The specimens with corrosion rate more than 3.5% explained a separation of concrete cover because of the weakening of the cover due to corrosion. However, the study did not include a control beam (unstrengthened beam) to demonstrate the effect of corrosion on the performance of the strengthened beams. In another study, Jie et al. (2017) found that the corrosion of steel reinforcement led to weakening the bond between the TRC strengthening layer and the substrate concrete which in turns reduce the strength of the strengthened beams.

It can be stated that the strengthening layer of corroded members may reduce the degree of corrosion by hindering the penetration of chloride towards steel rebars which in turns could improve the durability of the strengthened member. In addition, it was evident from the available literature of members strengthened with TRM, and there is a lack of information about the effect of corrosion on the performance of the strengthened member under a different level of corrosion.

2.5.3 Acceleration and measuring of corrosion

Different methods have been used to accelerate corrosion including salt spray, chloride diffusion, alternate drying and wetting in salt water and impressing anodic current. In the salt spray technique, a fog with dissolved sodium chloride is created in an enclosed chamber. However, it was found that concrete specimens kept in a salt spray chamber for more than 100 days did not show any visible signs of corrosion (Gadve et al. 2009).

Adding chlorides to the concrete during casting and alternate immersion into NaCl are effective in accelerating corrosion, but impressed current (galvanostatic method) is more suitable considering the time constraint.

The galvanostatic method based on impressing current is applied on the embedded bars in concrete (Almusallam et al., 1996). The integrated system is used to apply direct current to the steel bars. This system includes a rectifier with a built-in ammeter to monitor the
current, and a potentiometer to control the current intensity. Specimens partially immersed
in NaCl solution in the plastic tank. The steel reinforcement is made to work as the anode
while an immersed stainless steel bar or plate counter electrode used to serve as the
cathode (Almusallam et al., 1996; Cabrera 1996; Ballim and Reid, 2003; Fang et al., 2004;
Masoud and Soudki, 2006; Shannag and Al-Ateeq, 2006; Belarbi and Bae, 2007; El-
Maaddawy, 2008; Malumbela et al., 2009; Abosrра et al., 2011; El-Maaddawy et al., 2014;
Yu et al., 2015).
Fang et al. (2004) reported that the initiation and propagation of the accelerated corrosion
depend on the following factors:

1. Permeability of the concrete matrix.
2. Cover thickness.
3. The electric current applied.
4. Density of the solution used.
5. The environmental temperature.

The degree of corrosion can be measured as a gravimetric loss in weight of the reinforcing
bars (Almusallam et al., 1996; Cabrera, 1996). The corrosion level was expressed using
the following equation (Fang et al., 2004):

\[
C_R = \frac{G_0 - G}{g_0 l} \times 100
\]

(2.6)

where \(G_0\) is the initial weight of the reinforcement before corrosion, \(G\) is the weight of
reinforcement after removal of the corrosion products, \(g_0\) is the weight per unit length of
the reinforcing bar, and \(l\) is the bond length.

Fang et al. (2004) pointed out that the corrosion, in terms of the mass loss of reinforcing
steel due to corrosion can be estimated by the following equation:

\[
m_t = \frac{t \times 7 \times 5.847}{2 \times 96487}
\]

(2.7)

where \(t\) is the duration of exposure and \(I\) is the average current over the duration.

To establish different levels of corrosion, a calibration curve of the relationship between
the duration of the impressed current and the corresponding degree of corrosion was
prepared (Almusallam et al., 1996).
The degree of corrosion can be calculated according to the following equation (Shannag and Al-Ateek, 2006):

\[
\frac{2RT}{D} \times 100 = 2312 \frac{T}{D} \quad (2.8)
\]

Where \( T \) is the time elapsed in years after corrosion initiation, \( R \) is the material loss per year (cm/year), \( i \) is the corrosion current density (A/cm\(^2\)), \( D \) is the diameter of reinforcing steel (mm).

Another form of Faraday’s law was applied by Belarbi and Bae (2007) to calculate the weight loss of steel due to corrosion. The formula is as follows:

\[
w_{(g)} = \frac{A_m}{z \times F} \sum \Delta t \times I_{ave} \quad (2.9)
\]

where \( w(g) \) is accumulated steel loss (g), \( A_m \) is atomic mass (for iron 55.85 g), \( I_{ave} \) is average current (A) applied over time increment \( \Delta t \) (s), \( z \) is valency (assuming that most of rust product is Fe(OH)\(_2\), it is taken as 2), and \( F \) is Faraday’s constant (96487 C/eq).

Val (2007) explained that the non-destructive monitoring of corrosion is based on electrochemical measurements. These measurements estimated the corrosion rates in terms of corrosion current density (\( i_{corr} \)). Faraday’s law of electrochemical equivalence is used to transform \( i_{corr} \) the equivalent loss of metal. Accordingly, a constant corrosion current density of 1 \( \mu \)A/cm\(^2\) corresponds to a uniform corrosion penetration of 11.6 \( \mu \)m per year. With assuming a constant corrosion rate, the reduction in the diameter of a corroded reinforcing bar \( \Delta D \) after \( t \) years since corrosion initiation can be estimated (in millimetres) as:

\[
\Delta D(t) = 0.0232i_{corr}.t \quad (2.10)
\]

For multi-number (n) of reinforcing bars with the same bar diameter (\( D_o \)), the change in cross sectional area (\( \Delta A_s \)) will be:

\[
\Delta A_s = A_{so} - A_s(t) \quad (2.11)
\]

Where, \( A_{so} \) is the uncorroded cross sectional area, and \( A_s(t) \) is the area after corrosion which calculated according to the following equation:
Another application to Faraday’s law used by Spainhour and Wootton (2008), Lachemi, Al-Bayati et al. (2014), Tang et al. (2014) which based on the relation between amount of corrosion and consumed electrical energy. The amount of corrosion can be theoretically calculated according to the following equation:

\[ \Delta m = \frac{t \cdot i \cdot M}{z \cdot F} \]  (2.13)

where \( \Delta m \) is the loss mass due to corrosion, \( t \) is the time (s), \( i \) the current (A), \( M \) is the atomic weight of iron (55.847 g/mol), \( z \) the ion charge (assumed 2) and \( F \) the Faraday’s constant (96487 C/eq).

It can be stated that there is an agreement about the validity of using the impressed current to accelerate the corrosion of RC members. In addition, Faraday’s law can be used to estimate the rate of corrosion. However, this law neglects the effect of steel and concrete properties; hence a calibration of estimated corrosion should be carried out.

### 2.6 Modelling for TRM composites

Based on the previous experimental investigation, textile fibres of TRM composites are exhibit a significant decrease in load, strain and stiffness in comparison with the behaviour of ideal fibres. It was suggested to use deficiency parameters (reduction parameters) to the ideal behaviour of textile fibres. Hegger and Voss (2008), for instance, proposed that the tensile strength of TRM composites can be calculated based on the strength of textile only with considering reduction factors as in the following equation:

\[ F_{ctu} = A_t \cdot f_t \cdot k_1 \cdot k_{0,\alpha} \cdot k_2 \]  (2.14)

Where: \( A_t \) is the cross sectional area of the textile reinforcement \( f_t \) is the tensile strength of the filament, \( k_1 \) is the coefficient for efficiency, \( k_{0,\alpha} \) coefficient of oblique-angled load: \( k_{0,\alpha} = 1 - \frac{\alpha}{90} \) and \( k_2 \) coefficient of biaxial load. Similarly, the flexural bearing capacity was calculated based on the tensile strength of the reinforcement and the internal lever arm, according to the following expression:
\[ M_u = k_{fl} \cdot F_{ctu} \cdot z \]  

(2.15)

Where: \( k_{fl} \) is the coefficient of bending load depending on the fibre material and \( z \) is the internal lever arm. However, there is no available specific values of these coefficients and relationship between these coefficients, and the properties of mortar and textile fibres were not specified.

Holler et al. (2004) pointed out that the textile reinforced concrete composites can be numerically modelled as a layered continuum of uniaxial textile fibres and plane stress layer of mortar with finite shell elements. The mortar matrix was modelled as nonlinear-elastic-plastic damage model with softening after cracking. Due to the absence of effective model for the bond-slip of the textile fibres within the matrix, they proposed a modified constitutive law of fibres, considering the bond effect. The model is based on modifying the stress-strain relationship of fibres to four points depending on the tensile properties of the mortar, as shown in Figure 2.20. The following equation expresses the first three points:

\[
\sigma_{Yr,\mu\%} = \frac{f_{ct,\mu\%}}{\rho} + f_{ct,5\%} \frac{E_Y}{E_c}
\]  

(2.16)

Where, \( \sigma_{Yr,\mu\%} \) is the tensile stress of fibre, \( f_{ct,\mu\%} \) is the tensile strength of mortar with fractal value \( \mu\% \), \( \rho \) fibre reinforcement ratio, \( E_Y \) and \( E_c \) are the modulus of elasticity of fibre and mortar, respectively. The fourth point is considered the tensile strength of the fibres. This model assumes a ridged bond between textile fibres and surrounding mortar.

![Figure 2.20: Modified stress strain relationship of textile fibres (Holler et al., 2004)](image-url)
Mumenya et al. (2010) proposed that the bond strength of textile fibres inside mortar can be determined using fibre pull out test of one fibre and the average bond strength ($\tau$) can be determined according to the following relationship:

$$\tau = \frac{P_{max}}{2\pi r l_e}$$  \hspace{1cm} (2.17)

Where: $P_{max}$ is the peak load from the pull-out test, $r$ and $l_e$ are fibre radius and embedded length respectively. However, this formula cannot be applied for TRM composite because it neglects the effect of reinforcement ratio and the adhesive properties of the matrix.

In the same field, Wang et al. (2016) reported that the bond strength between the textile fibres and matrix is mainly controlled by the surface roughness of fibres and the tensile strength of the matrix. It was suggested the following expression to determine the bond strength of FRCM composites:

$$\tau = \mu \frac{c + 0.5d}{1.664d} f_{cr}$$  \hspace{1cm} (2.18)

Where: $\mu$ is the friction coefficient, $c$ is the protective layer thickness; $d$ is the diameter of fibres and $f_{cr}$ is the cracking strength of mortar. Different studies investigate modelling the bond between the fibres and the mortar of the textile fibre cementitious composites.

Larrinaga et al. (2013) proposed a crack model to simulate the stages of the stress-strain relationship of TRM under direct tensile stresses. The proposed model was based on the Aveston-Cooper-Kelly theory to define the theoretical stress-strain behaviour of composites with a brittle matrix. According to their model, the first stage of stress-strain curve obeys the following formulas:

$$E_{c1} = E_f V_f + E_m V_m$$  \hspace{1cm} (2.19)

$$\sigma_{mc} = \frac{E_{c1} \sigma_{mu}}{E_m}$$  \hspace{1cm} (2.20)

Where: $E_{c1}$ is the composite stiffness, $E_f$ and $E_m$ are the tensile modulus of elasticity of fibres and mortar, respectively, $V_f$ and $V_m$ are the volumetric fraction of fibres and mortar, respectively, $\sigma_{mc}$ is the cracking stress and $\sigma_{mu}$ is matrix tensile failure. The stress strain at stage II is defined with $E_{c2} = 0$ and the strain in TRM can be determined from the following formula:
\[
\varepsilon_{mc} = \frac{\sigma_{mc} - k_t \sigma_{fu} (1 + \alpha_v)}{E_f} + \varepsilon_{mu}
\]  
(2.21)

\[
\alpha_v = \frac{E_m V_m}{E_f V_f}
\]  
(2.22)

Where: \(\varepsilon_{mc}\) is the strain in the TRM after cracking and \(k_t\) is the duration of the load which is equal to 0.6 for short term loading and 0.4 for long term loading. In stage III, the strain of TRM composites is controlled by the strain of textile fibres with taking in account a reduction factor (\(\beta\)). This factor is obtained empirically based on the results of the tested specimens. For basalt TRM, the failure strain is defined according to the following equation:

\[
\varepsilon_t = 0.65 \varepsilon_f
\]  
(2.23)

In another study, Larrinaga et al. (2014) simulated numerically the behaviour of basalt textile reinforced mortar under direct tensile load. They proposed a rigid bond between the textile fibre and the mortar. The fibres were modelled as discrete bar elements defined by a uniaxial stress-strain relationship obtained from the tensile test. Fracture –plastic model was used to simulate the behaviour of mortar. Within this model, the mortar cracks were modelled as an orthotropic smeared crack formation with rotated crack model up to 40% of the tensile strength followed by fixed crack model up to failure. Three-dimensional isotropic brick elements with eight nodes were used for modelling mortar.

Bilotta et al. (2017) proposed that the behaviour of the composite FRCM can be obtained by the summation of the single materials of fibres and mortar with taking in account of the cracking. Accordingly, the elastic stiffness of the FRCM composite is expressed as follows:

In the un-crack stage, both mortar and textile fibres are loaded and thus the elastic stiffness is equal to:

\[
E_{FRCM} = \frac{E_m A_m + E_F A_F}{A_{FRCM}}
\]  
(2.24)

- In the cracked stage, the load is only carried out by the fibres, therefore the elastic stiffness of the composite will equal to the stiffness of the fibres:

\[
E_{FRCM} = E_F
\]  
(2.25)
Where: $A_m$ is the area of mortar, $A_F$ is the area of fibres, $A_{FRCM}$ is the area if mortar minus the area of fibre. $E_m$ and $E_F$ are the modulus of elasticity of mortar and fibre, respectively. In addition, due to the unavoidable slip of fibres they suggested an efficiency factor ($\eta$) of the fibres within the FRCM composites. This factor represents the ratio between the ultimate achieved stresses in the fibres in bond test to the stress of fibres in tensile stress, which can be expressed according to the following equation:

$$\eta = \frac{\sigma_{\text{max}}}{\sigma_{\text{max,av}}} \quad (2.26)$$

It can be stated from the above models of TRM composites that the available models are still under investigation and there is no agreement on the validity of most of them. In addition, most of them neglect the effect of mortar properties, which it has a significant impact on the bond strength as reported by the previous studies.

### 2.7 Modelling of strengthened members with TRM composites

Different models have been proposed to simulate the behaviour of members strengthened with TRM. Some of these models adopted perfect bond between the substrate member and the strengthening layer and others used the same bond slip model proposed for FRP strengthening layers. Ortlepp et al. (2006), proposed a model to calculate the shear bond transfers between TRC strengthening layer and old concrete. In this model, they proposed that only the mortar between the textile fibres transfers the load and the dimension of this effective area control the load carrying capacity in the textile layer. The coefficient of the effective area ($k_{A,\text{eff}}$) is determined as the ratio between the areas of the mortar between the textile fibres to the total area. The maximum bond forces ($F_{L,\text{vu}}$) transferred from the strengthening layer to the old concrete can be determined according to the following equation:

$$F_{L,\text{vu}} = b_L \left( l_{v,\text{I}} \times \min \left\{ k_{A,\text{eff}} \times \tau_{V,J,\text{cu}}, \tau_{V,J,\text{cu}} \right\} + l_{v,\text{II}} \times k_{A,\text{eff}} \times \tau_{V,J,\text{fc}} \right) \quad (2.27)$$

Where, $b_L$ is the width of the strengthening layer, $l_{v,\text{I}}$ and $l_{v,\text{II}}$ are the bond length of un-cracked and cracked range, respectively, $\tau_{V,J,\text{cu}}$ and $\tau_{V,J,\text{fc}}$ are the shear bond stress in uncracked and cracked range, respectively. However, this formula is based on determining
the cracked and un-cracked length which is practically difficult to obtain due to multi cracking of the strengthening layer close to the failure of the strengthened member.

Ombres (2011) proposed that the behaviour of RC beams strengthened with FRCM can be modelled similarly to the adopted model of FRP by assuming perfect bond between the FRCM and concrete. The theoretical prediction is based on the equilibrium and compatibility of the cross-section of the strengthened RC beam. The adopted constitutive laws of the strengthened RC components are as following:

- Concrete: in compression, non-linear constitutive law based on Eurocode 2 was adopted

\[
\frac{\sigma_c}{f_c} = \frac{k\eta - \eta^2}{1 + (k-2)\eta}
\]  

(2.28)

Where: \( \eta = \frac{\varepsilon_c}{\varepsilon_{c1}} \), \( \varepsilon_{c1} = 0.0022 \), \( k = 1.1 \frac{E_c \varepsilon_{c1}}{f_c} \), \( f_c \) is the compressive strength of concrete cylinder, \( E_c \) is the modulus of elasticity of concrete which obtained from \( E_c = 22000 \left( \frac{f_c}{10} \right)^{0.3} \) (N/mm²).

In tension, linear elastic behaviour up to the cracking was adopted.

- Steel reinforcement rebar: elastic perfectly plastic, which is controlled by the yield strength and modulus of elasticity, was adopted.

- FRCM: was modelled as linear elastic material with brittle behaviour up to the ultimate tensile

Regarding the slip of the FRCM strengthening layer, Ombres (2011) explained because there is no available bond –slip law of FRCM –to –concrete, the analytical bond-slip relationship of FRP and concrete can be used. It was proposed two laws of bond slip; the first one is non-linear which was presented by Savoia et al. (2003) and bilinear law that proposed by Teng et al. (2006).

- The nonlinear bond slip law (Savoia et.al, 2003)

\[
\tau = \tau_{max} \frac{u}{u_o} \left[ \frac{2.86}{1.86 + \left( \frac{u}{u_o} \right)^{2.86}} \right]
\]  

(2.29)

\[
\tau_{max} = 3.5f_c^{0.19}
\]  

(2.30)

\( u_o = 0.051 \text{ mm} \)
- The bilinear bond slip law

\[
\tau = \tau_{\text{max}} \frac{u}{u_o} \quad \text{if } u \leq u_o \\
\tau = \tau_{\text{max}} \frac{u_{f,\text{max}} - u}{u_{f,\text{max}} - u_o} \quad \text{if } u \geq u_o
\]  

(2.31) (2.32)

\[
\tau_{\text{max}} = 1.5 \beta_w f_t
\]  

(2.33)

\[
u_o = 0.019 \beta_w f_t u_f = 2
\]  

(2.34)

\[
\frac{G_f}{\tau_{\text{max}}} \beta_w = \sqrt{\frac{2.25 - \frac{B_f}{B}}{1.25 + \frac{B_f}{B}}} G_f = 0.308 \beta_w \sqrt{f_t}
\]  

(2.35)

Where: \(u_o\) is the slip corresponding to the \(\tau_{\text{max}}\), \(B_f\) is the width of the FRCM layer, \(B\) is the width of the section and \(f_t\) is the tensile strength of concrete. It was indicated that nonlinear model is better than bilinear model in prediction the debonding strain while the bilinear model was better in prediction the ultimate moment capacity.

Similarly, Raoof et al. (2017) proposed to use the same equation that used for calculation the debonding stress of the FRP strengthening layer for TRM.

\[
f_{f,\text{bm}} = k_c k_m k_b \beta_i \sqrt{\frac{2E_f - 2/3}{t_f} f_{\text{cm}}}
\]  

(2.36)

Where: \(k_c\) is the intermediate crack factor and equal to 2, \(k_m\) is the matrix factor and equal to 0.25, \(k_b\) is the shape factor, \(\beta_i\) is the length factor which can be taken 1.0, \(E_f\) is the elastic modulus of elasticity of the composite material, \(t_f\) is the equivalent thickness of the textile and \(f_{\text{cm}}\) is the compressive strength of concrete. \(k_b\) can be calculated based on the following formula:

\[
k_b = \frac{\sqrt{2 - b_f/b}}{\sqrt{1 + b_f/b}} \geq 1.0
\]  

(2.37)

Where: \(b_f\) is the width of the composite and \(b\) is the width of the beam.

In the same field, D’Ambrisi et al. (2012) proposed bond effectiveness factor in evaluating the maximum debonding force and corresponding strains according to the following equations:
\[ k = k(n) \]  \hspace{1cm} (2.38)

\[ p_f = 2nb_fk(n) \]  \hspace{1cm} (2.39)

\[ P_{dbu} = 2nb_f\sqrt{k(n)E_f\varepsilon_{t1}G_f} \]  \hspace{1cm} (2.40)

\[ \varepsilon_{dbu} = 2\frac{k(n)G_f}{E_f\varepsilon_{t1f}} \]  \hspace{1cm} (2.41)

Where: \( k \) is the bond effectiveness factor, \( p_f \) is the perimeter of fibre, \( b_f \) is the width of fibre layer, \( n \) is the number of layers, \( P_{dbu} \) is the maximum debonding force, \( E_f, t_1 \) and \( G_f \) are the modulus of elasticity, thickness and interfacial fracture energy of fibres, respectively, \( \varepsilon_{dbu} \) is the strain at the debonding. However, this model is totally based on the properties of textile fibres and there is no considering to the properties of the matrix.

Liu et al. (2017) explained that TRC strengthening layer of columns could be modelled in ABAQUS as a rebar element to simulate the textile fibres by assuming a perfect bond with the surrounding matrix. The textile materials modelled as a hyperplastic material which is isotropic and nonlinear. The contact between the TRC layer and concrete can be modelled as breakable contact model. In this model, when the interface subjects to tensile and shear stresses, it allows the separation of the connected surfaces according to the following interface failure criterion:

\[ \left( \frac{\text{Max}(F^n,0)}{F^n_f} \right)^2 + \left( \frac{\sqrt{(F^1_f)^2 + (F^2_f)^2}}{F^2_f} \right)^2 \leq 1.0 \]  \hspace{1cm} (2.42)

Where: \( F^n \) is the normal force applied on the bonded nodes, \( F^1_f \) and \( F^2_f \) are the orthogonal shear forces tangent to the surface, \( F^n_f \) and \( F^2_f \) are the force required to cause failure in tension and shear, respectively.

However, Alabdhulhady et al. (2017) modelled the carbon and textile fibres as shell elements with 4-nodes to simulate the behaviour of RC beams strengthened with FRCM and subjected to torsion loading. The fibre material was modelled as elastic – orthotropic behaviour as obtained from the tensile test of fibres with considering only the fibres in the primary direction. The bond between the steel reinforcement and concrete was simulated as perfect bond and the same assumptions was adopted for fibres and mortar. The bond between the substrate concrete and FRCM strengthening layer was assumed as perfect
bond. That could be accepted because the RC members were fully wrapped with FRCM layers. Therefore, there is a low possibility of debonding the strengthening layer. It was obtained a good agreement with the experimental results where the validity of the numerical modelling is evaluated by comparing the ultimate capacity of the experimental and numerical results.

ACI 549.4R-13 (2013) guide for design and construction of structural members strengthened with FRCM assumes that bond between the substrate member and the strengthening layer is remain effective (perfect bond). However, the consequences of debonding is presumably accounted by limitation the proposed amount of enhancement the strengthening layer to not exceed 50% of the original capacity of the un-strengthened members. However, most of the previous studies regarding strengthening RC beams with TRM demonstrated a debonding of strengthening layer with improvement in capacity less than 50%. That demonstrates the limitation of using the adopted model by ACI 549.4R-13 (2013).

Overall, based on the critical review of the available theoretical models of RC structures strengthened with TRM, the following main points can be drawn:

• Appropriate design guidelines are not available for the RC members strengthened with TRM. The proposed guideline by ACI 549.4R-13 (2013) is based on the assumption of a perfect bond between the strengthening layer and the substrate member, which is not justifiable considering the previous experimental studies, observed debonding failure mode in most cases.

• Some researchers suggested the use of the same models proposed to predict the debonding of a strengthened structure with FRP, which include coefficients obtained based on the experimental studies of FRP materials. However, the difference between the epoxy and mortar as an adhesive is obvious, especially of multiple cracking of mortar that extends to the values of these coefficients.

• The available models neglect the effect of mortar properties of the TRM, which is significantly affecting the interface bond strength. However, the literature review indicated the effect of mortar strength on the bond. In addition, the previous studies have dealt with normal strength mortar and hence the data of models deal with the TRM with high strength mortar is non-existent.
2.8 Concluding remarks

The critical review of the previous studies of the strengthening methods indicated higher durability and compatibility of TRM composites compared with epoxy based adhesion strengthening methods. However, most of the literature reported a debonding of the TRM strengthening layer even though the attempts to improve the interface bond strength using different types of anchors. Moreover, there is no available data about the efficiency of TRM in repair corroded RC beam, and a lack of information about the durability of the strengthened beams with TRM in the presence of deteriorating agents such as corrosion was found. The bond between two cementitious materials cast at different ages was also reviewed and it was found high impacts of the properties of the strengthening layer on the bond effectiveness. Moreover, the corrosion concept by means of mechanism, measuring, acceleration and effects on the RC structures were reported. Finally, the available theoretical modelling of simulation the behaviour of TRM composites were presented and discussed. Accordingly, the following points can be concluded from the above literature review, for all strengthening techniques, interface bond strength between the substrate concrete and the strengthening layer is the key to successful strengthening technique and is the weakest part in the strengthened members. Therefore, improving the bond strength between substrate concrete and strengthening layer will improve significantly the performance of the strengthened RC members.

- The use of connectors (anchors) at the interface can significantly improve the performance of strengthening technique by delay or preventing the debonding failure of the strengthening layer. Taking account of using steel or polymer fibre connectors demonstrated a noticeable scatter, it is therefore of interest to investigate the effect of application cementitious connectors in improving the performance of TRM strengthening technique which has not been previously investigated.

- Although most of the previous studies of RC beams strengthened with TRM agree, that as the mechanical properties of the TRM matrix increases the performance of strengthening improves. However, the previous studies investigated normal strength mortar, and the effect of high strength mortar is not yet established.

- In addition to the reported debonding failure of TRM strengthening techniques, there are contradictory results about the effect the amount and mechanical properties of textile fibres on the enhancing of strengthening. Therefore, it is essential to investigate the impact of different mechanical properties and amount of textile fibres on the performance of strengthened RC beams with TRM.
- The amount of enhancement due to strengthening is influenced by the strength of the existing substrate RC members by means of concrete strength and amount and strength of steel reinforcement. These parameters are not well addressed, and therefore, it is interesting to examine their effect on the flexural strength of strengthened RC beams with TRM.

- The mechanical interlocking and chemical adhesive dominate the bond strength of the interface between the substrate and new cementitious layer. The chemical adhesion of the interface is controlled by C-S-H components which is a function of the cement content within the matrix (higher cement content produces higher C-S-H which in turns exhibit higher adhesive bond). In addition, SCMs increase the amount of C-S-H within the matrix structure by their pozzolanic reaction with calcium hydroxide. It is therefore of interest to investigate their effects on the performance of TRM composites.

- There is no information about the efficiency of using TRM strengthening technique in field of repair RC members subjected to deteriorated factors such as corrosion of steel reinforcement. This investigation is essential to determine whether the principles of predicting the ultimate strength of strengthening a member can be applicable for repair process.

- Insufficient studies investigated the effect of corrosion (low degree of corrosion were investigated, less than 5%) on the performance of strengthened RC beams with TRM. Therefore, it will be necessary to investigate the performance of TRM strengthening in the presence of different levels of corrosion.

- There is a lack of information about the numerical simulation of strengthened RC beams using TRM layers. In addition, most of the available models either neglected the bond slip between the substrate RC member and the strengthening layer or are not consider the effect of bond coefficients on the behaviour of the strengthened RC members. Therefore, a developed numerical model is developed and validated in this study.

- Due to the absence of a specific theoretical debonding for TRM strengthening technique, some researchers proposed to use the FRP models. However, the validation of these models is not well, and more examination of different properties of strengthening layer is required.
Chapter Three

Development of high performance Textile Reinforced Mortar

3.1 Introduction

This chapter details the experimental investigation for developing high-performance textile reinforced mortar (TRM). The outcomes of this investigation are essential in investigating the experimental and numerical behaviour of RC beams strengthening with TRM. Based on the critical review of the available literature, the behaviour of TRM composites significantly affects their performance as a strengthening layer of RC structures. Several parameters control the performance of TRM composites, some of them relate to the mortar such as workability and strength and others relevant to mechanical properties, amount and configuration of textile fibres. However, there is a lack of information about the effect of different mortar strength, particularly high strength mortar, on the performance of TRM as a composite material and as a strengthening layer.

Bond strength of both textile fibre - mortar and TRM - RC is the key to a successful strengthening RC system. Mortar has a crucial role in both bonds through providing the cohesion and adhesion properties of the hydrated cement. This chemical bond is directly related to the strength of mortar, which mainly depends on the amount of calcium silicate hydrate (C-S-H) and pores within the mortar structure (Neville, 2011). Also, it has been found that the addition of supplementary cementitious materials (SCMs) like GGBS, silica fume and fly ash increases the amount of C-S-H gel (Neville, 2011). Accordingly, the mechanical properties of mortar can be used as an indicator of the quality of the provided mortar bond strength.

Moreover, differential shrinkage between substrate concrete and TRM overlay can reduce the interface bond strength by means of initiation cracks in concrete-overlay interface (Rangaraju and Pattaik, 2008; Li Gengying, 2003). Study the shrinkage behaviour is important to understand the consequences of mortar properties on the volumetric change of TRM as a strengthening layer.
An extensive experimental investigation is implemented to characterise the properties of mortar and TRM with taking in account different amounts and types of textile fibres. As a part of this investigation, the properties of constituent materials of TRM and concrete were conducted. Workability of mortar and concrete were determined using flow table and slump test, respectively. The mechanical properties of concrete and mortar were characterised using compressive, tensile and flexural tests. X-ray diffraction test was carried out to demonstrate the change in hardened high strength mortar structure due to addition SCMs. Moreover, the durability of high strength mortar was quantified using chloride migration test.

The performance of TRM composites was investigated through direct tensile and flexural tests. In addition, free drying shrinkage test was conducted to investigate the shrinkage behaviour of mortar and TRM composites.

### 3.2 Properties of constituent material

This section presents the properties of constituent materials used for casting mortar, concrete and TRM composites. These materials include cement, fine aggregate, coarse aggregate, supplementary cementitious materials (SCMs), superplasticizer and textile fibres.

#### 3.2.1 Cement

Ordinary Portland cement type CEMII/A-LL 32.5R satisfying BS EN 197-1:2011 was used for casting normal strength mortar and concrete. Portland cement type CEM 52.5N was used for casting high strength mortar. The physical and chemical properties provided by the manufacturer (Hanson UK Company) of both types of cement are presented in Table 3.1.

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>CaO</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>MgO</th>
<th>SO₃</th>
<th>Na₂O</th>
<th>Na₂O</th>
<th>K₂O</th>
<th>L.O.I</th>
<th>Fineness (m²/kg)</th>
<th>Specific gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEMII/A-LL 32.5R</td>
<td>65</td>
<td>18.3</td>
<td>4.51</td>
<td>2.73</td>
<td>1.0</td>
<td>2.97</td>
<td>0.19</td>
<td>0.27</td>
<td>4.57</td>
<td>373</td>
<td>3.07</td>
</tr>
<tr>
<td>CEM52.5N</td>
<td>63.8</td>
<td>19.9</td>
<td>4.8</td>
<td>3.1</td>
<td>1.1</td>
<td>3.3</td>
<td>0.7</td>
<td>0.6</td>
<td>2.7</td>
<td>472</td>
<td>3.18</td>
</tr>
</tbody>
</table>
3.2.2 Fine and coarse aggregates

Uncrushed gravel with 10mm maximum aggregate size complying with BS EN 12620:2013 was used for all concrete samples. Sharp sand with a maximum aggregate size of 4mm satisfying BS EN 12620:2013 was used.

Fine silica sand satisfying BS EN 12620:2002+A: 2008 obtained from Sibelco UK was used to cast mortars for TRM. It contains a high proportion of silica in the form of quartz with yellow/brown colour and grain distribution ranges between 0.5 to 0.1 mm. Table 3.2 shows the chemical composition of silica sand (as supplied by the manufacturer).

![Table 3.2: Chemical properties of silica sand](image)

<table>
<thead>
<tr>
<th>Sample</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>CaO</th>
<th>MgO</th>
<th>SO₃</th>
<th>Na₂O</th>
<th>K₂O</th>
<th>L.O.I</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silica sand</td>
<td>99.73</td>
<td>0.1</td>
<td>0.05</td>
<td>--</td>
<td>1.0</td>
<td>--</td>
<td>&lt;0.05</td>
<td>0.01</td>
<td>0.09</td>
</tr>
</tbody>
</table>

3.2.3 Superplasticizer

Fosroc Auracast 200 high performance concrete superplasticizer based on polycarboxylate polymers (obtained from Resapoli UK company) was used to achieve the desired workability. Table 3.3 presents the chemical properties as received from the manufacturer.

![Table 3.3: Properties of Fosroc Auracast200 superplasticizer](image)

<table>
<thead>
<tr>
<th>Nature</th>
<th>Liquid</th>
</tr>
</thead>
<tbody>
<tr>
<td>Colour</td>
<td>Dark Straw</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>1.050 – 1.070</td>
</tr>
<tr>
<td>pH</td>
<td>4 +/- 1</td>
</tr>
<tr>
<td>Chloride content</td>
<td>&lt;0.1%</td>
</tr>
<tr>
<td>Na₂O equivalent</td>
<td>&lt;0.5%</td>
</tr>
<tr>
<td>Freezing point</td>
<td>Sensitive to freezing</td>
</tr>
<tr>
<td>Air entrainment</td>
<td>Typically, less than 2% additional air is entrained at normal dosage.</td>
</tr>
</tbody>
</table>
3.2.4 Supplementary cementitious materials (SCMs)

In this study, three types of supplementary cementitious materials (SCMs) were used in producing high strength mortar (HSM) as a volumetric replacement of cement. These materials include silica fume (SF), Ground-granulated blast-furnace slag (GGBS) and Fly ash (FA). X-ray diffraction test was carried out in the University of Brighton labs to study the crystal structures of the SCMs. Moreover, this test was also implemented, in Section 3.3.2.7, to study the effect of the SCMs in improving the mechanical properties of high strength mortar by means of the variation in the calcium hydroxide (Ca(OH)₂) as a result of the pozzolanic reaction of the SCMs to produce a supplementary C-S-H gel.

Densified (DSF) and un-densified silica fume (UDSF) were used in this study. The chemical composition and physical properties obtained from the manufacturer (Elkem A Bluestar Company) of both types of silica are presented in Table 3.4. The X-ray patterns of a powder of both silica fume show a peak at 2θ equal to 22°, which refers to a high characteristic of amorphous SiO₂ (Qing et al., 2007), as shown in Figure 3.1.

Ground-granulated blast-furnace slag (GGBS) satisfying BS EN 15167-1: 2006 obtained from Hanson Heidelberg Cement group was used. The chemical compositions and physical properties of GGBS are presented in Table 3.4 (as supplied by the manufacturer). X-ray analysis of the GGBS explains high peak of SiO₂ at 2θ equal to 21°. However, the peak had less intensity in comparison with silica fume peak, which refers to lower content of SiO₂ in GGBS mineral content, as shown in Figure 3.1. In addition, no frequent peaks were observed which demonstrate an amorphous phase (Divsholi et al., 2014).

Fly ash type 450-S satisfying BS EN 450-1:2012 was used in the mixes of high strength mortar. This class of fly ash (class S) was used because of its high pozzolanic activity, and it has a constant fineness and carbon content. It was supplied from the Drax Power Station, North Yorkshire, UK under the CEMEX brand. The chemical and physical properties of the FA are shown in Table 3.4. The X-ray diffraction analysis of fly ash (Figure 3.1) shows that the major crystalline constituents of the fly ash are quartz (SiO₂) at 2θ equal to 26°. The amorphous SiO₂ appeared at 2θ of 21° at less intensity (Panias et al., 2007).
Table 3.4: Chemical and physical properties of SCMs

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Silica fume</th>
<th>GGBS</th>
<th>Fly ash</th>
</tr>
</thead>
<tbody>
<tr>
<td>CaO</td>
<td>0.1</td>
<td>40</td>
<td>2.38</td>
</tr>
<tr>
<td>SiO₂</td>
<td>92</td>
<td>35</td>
<td>59</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>0.7</td>
<td>12</td>
<td>23</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>0.8</td>
<td>0.2</td>
<td>8.8</td>
</tr>
<tr>
<td>MgO</td>
<td>--</td>
<td>10</td>
<td>1.39</td>
</tr>
<tr>
<td>SO₃</td>
<td>--</td>
<td>--</td>
<td>0.27</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.33</td>
<td>--</td>
<td>0.74</td>
</tr>
<tr>
<td>K₂O</td>
<td>1.45</td>
<td>--</td>
<td>2.81</td>
</tr>
<tr>
<td>L.O.I</td>
<td>4.6</td>
<td>2.8</td>
<td>1.61</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.20</td>
<td>2.90</td>
<td>2.20</td>
</tr>
<tr>
<td>Specific surface area (mm²)</td>
<td>15-30</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Mean particle size (μm)</td>
<td>0.15</td>
<td>9.2</td>
<td>--</td>
</tr>
</tbody>
</table>

Figure 3.1: X-ray patterns of SCMs
3.2.5 Textile fibres

Based on the literature review in Chapter 2, textile fibres with higher opening dimensions explain higher performance due to increase the interlocking between the fibres and surrounding mortar. In addition, the area of filaments between the openings also affects the performance of TRM composites.

In this study, textile basalt fibres with an opening dimension of 5×5 mm were investigated, as shown in Figure 3.2a. Basalt fibre mesh is made of continuous basalt filament (roving) in two orthogonal directions. Fibres are coated with Styrene-acrylic latex to improve their performance. This configuration is adopted to achieve a high contribution of textile reinforcement in cracks bridging after reaching the ultimate tensile strength of mortar. Small opening size (such as 5×5mm) of textile fibres can explain high tensile strength but require relatively higher workability to ensure full penetration of mortar within the textile fibres openings. Table 3.5 shows the properties of textile basalt fibres (as provided by the manufacturer).

<table>
<thead>
<tr>
<th>Table 3.5: Properties of textile basalt fibres</th>
</tr>
</thead>
<tbody>
<tr>
<td>Property</td>
</tr>
<tr>
<td>Density of un-sized filament material</td>
</tr>
<tr>
<td>Melting point</td>
</tr>
<tr>
<td>Mesh size</td>
</tr>
<tr>
<td>Thickness</td>
</tr>
<tr>
<td>Maximum load (N/5cm)</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Elongation at break (%)</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Breaking Elongation (mm)</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

Textile carbon fibres with opening mesh dimensions equal to 10x10mm were used, as shown in Figure 3.2b. This configuration was adapted to ensure the desired penetration between the carbon fibres and mortar. The higher opening mesh size of carbon fibres comparing to basalt fibres was adopted because of the area of carbon filaments is higher. Therefore, the mesh size should be higher than basalt fibre to achieve a similar performance regarding fibre penetration within mortar.
Figure 3.2: Textile fibre configuration of (a) basalt and (b) carbon

A direct tensile test has been carried out to determine the actual tensile strength and modulus of elasticity of basalt and carbon fibres. Samples with dimensions 25×200 mm with an effective length of 100 mm were prepared, as shown in Figure 3.3. Two steel plates were glued using epoxy Sikadur-30 at the ends of the samples to avoid the tearing of fibres at the grips of the testing machine. This configuration is applied similarly to the procedure used by De Santis and De Felice (2015).

Three specimens of each type of textile fibres were tested. Figure 3.4 shows the testing set up of basalt and carbon samples. A steel frame was fixed at the ends of the sample to hold Linear Variable Displacement Transducers (LVDT) to measure the elongation during loading. Universal Instron machine with a 0.002mm/sec displacement control-based loading was used for testing.

Figure 3.3: Dimensions and sample preparation of textile fibres
Figure 3.5 shows the stress-strain curves of three textile basalt fibre samples. All samples exhibited a linear behaviour under tensile loading until the sudden failure. The small variation in the stress-strain curves of the three samples could be attributed to the non-uniform loading of the individual textile fibre filaments. The average mechanical properties of the three samples are presented in Table 3.6.
The same test setup is adopted to establish the tensile properties of textile carbon fibres. Specimens exhibited linear stress-strain curves up to the ultimate tensile strength, as shown in Figure 3.6. One of the samples, Carbon-3, explained a partial slip at one side from steel plates, which led to reduce the tensile strength because of concentrating the load only on one part of textile carbon. Similar to textile basalt fibres, carbon samples exhibited linear behaviour up to rupture.

Accordingly, the results of sample Carbon-3 are neglected in determining the tensile mechanical properties of textile carbon fibres. The average tensile properties of two samples are presented in Table 3.6.

![Stress-strain curve of textile carbon fibres](image)

**Figure 3.6: Stress-strain curve of textile carbon fibres**

<table>
<thead>
<tr>
<th>Fibre type</th>
<th>$F_{tu}$ [MPa]</th>
<th>$\varepsilon_{fu}$</th>
<th>$E$ [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basalt</td>
<td>730</td>
<td>0.0112</td>
<td>70</td>
</tr>
<tr>
<td>Carbon</td>
<td>1200</td>
<td>0.0076</td>
<td>150</td>
</tr>
</tbody>
</table>

**Table 3.6: Mechanical properties of textile basalt and carbon fibres**
3.3 Composite materials properties

The composite materials used in this study include concrete, mortar and textile reinforced mortar (TRM).

3.3.1 Concrete

Several mixes were trailed to establish mix proportions of concrete with a nominal compressive strength of 30 MPa. Cement type CEMII/A-LL 32.5R was used. The initial mix proportions were selected based on the British standards for casting concrete (Neville, 2011). Standard slump test was carried out according to BS EN 12350-2:2009, to assess the workability of concrete. The average slump of three specimens ranged between 100 to 150 mm.

A Series of tests was conducted to determine the compressive strength of concrete at ages 7 and 28 days. Cube specimens with dimensions 100×100×100 mm\(^3\) satisfying BS EN 12390-3:2009 were used for this purpose. Splitting tensile test of cylinders (100 mm diameter and 200mm length) were carried out to assess the tensile strength of the concrete at 28 days according to BS EN 12390-6:2009. Avery Denison 7227 compression machine was used with a constant loading rate of 180kN/min and 90kN/min for compressive and tensile tests, respectively.

Figure 3.7 shows the compressive, tensile and slump test setup. The compressive and tensile tests results (average of three specimens) are presented in Table 3.7.

<table>
<thead>
<tr>
<th>Concrete mix</th>
<th>Mix proportions</th>
<th>(f_{cm}) (MPa)</th>
<th>(f_{ctm}) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C30</td>
<td>375 860 860 225</td>
<td>23.1 6 31.0 3</td>
<td>2.21 2</td>
</tr>
</tbody>
</table>

Table 3.7: Compressive and tensile strength of concrete
According to BS EN 12390-5:2009, specimens with dimensions $75 \times 75 \times 285 \text{ mm}^3$ were cast and tested under three-point flexural loading to determine the flexural strength of concrete. Figure 3.8 shows the test setup and the results for concrete C30. The deflection of the specimen was determined using LVDT at the mid span of the specimen. An Instron universal test machine with constant displacement-based control loading of 0.002mm/sec was used. Table 3.8 presents the flexural strength of the tested specimens, which represent the average of results of three samples.

### Table 3.8: Flexural strength of concrete C30

<table>
<thead>
<tr>
<th>Concrete mix</th>
<th>$P_u$ [kN]</th>
<th>Deflection [mm]</th>
<th>$f_r$ [MPa]</th>
<th>c.o.v. (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C30</td>
<td>3.59</td>
<td>0.36</td>
<td>3.76</td>
<td>1</td>
</tr>
</tbody>
</table>
3.3.2 Mortar

Three mix proportions of mortar were developed to provide three different grades of mortar strength. M35 and M70 represent the normal strength mortar (NSM) while M105 represents the high strength mortar (HSM). The effect of the volumetric replacement of cement with the SCMs was investigated on high strength mortar to achieve mortar with higher performance. These different mortar strengths were adopted to examine the effect of mortar properties on the behaviour of TRM composite besides their impacts on the bond strength with substrate concrete.

3.3.2.1 Mix proportions of normal strength mortar M35 and M70

Several mixes were trailed to achieve the desired compressive strengths of normal strength mortar. Absolute volume method was used to determine the proportions of the cement, aggregates, superplasticizer and water (Neville, 2011). The required quantities by mass of the constituent materials for the cubic meter were determined according to the following equation (Neville, 2011):

$$\frac{C}{1000 \times \rho_C} + \frac{SS}{1000 \times \rho_{SS}} + \frac{S.P}{1000 \times \rho_{S.P}} + \frac{W}{1000} = 1.0$$

(3.1)

Where: C, S.S, S.P and W are the amount of cement, silica sand, superplasticizer and water in Kg, respectively, \(\rho_C\), \(\rho_{SS}\) and \(\rho_{S.P}\) is the specific gravity of cement, silica sand and superplasticizer, respectively.

The differences between M35 and M70 are the amount of cement and the water-cement ratio. The workability of the mortar was assessed using flow table test according to the BS-EN 1015-3:1999. The adopted mix proportions and flow table test results of M35 and M70 are presented in Table 3.9. The flowability of mortar M70 was higher than M35 although the w/c ratio of M35 is higher than M70. That was due to increasing the amount of silica sand from 1350kg/m$^3$ of M70 to 1500kg/m$^3$ of M35. These findings agree with the observation of Aldahdooh et al. (2013), where it was found that the workability of cementitious matrix reduces with increasing the amount of fine aggregate.
Compressive tests were conducted to determine the compressive strength of concrete of cube specimens (50×50×50 mm³) that satisfies BS EN 12390-3:2009. The samples were cured in water at ages 7 and 28 days. The tensile strength of mortar has been assessed using direct tensile test of dog bone specimen with 78mm length and 25.5x25.5mm² cross-sectional area at the midspan, as shown in Figure 3.9. The average of three samples results is adopted to determine the tensile ($f_{ctm}$) and compressive ($f_{cm}$) strength. An Instron universal testing machine with a constant displacement-based loading rate of 0.01mm/sec and 0.002mm/sec were used to determine the compressive and tensile capacity of the specimen. The mechanical properties of M35 and M70 are presented in Table 3.10.

Table 3.9: Mix proportions of normal strength mortar

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement [kg/m³]</th>
<th>Silica sand [kg/m³]</th>
<th>Water [kg/m³]</th>
<th>Superplasticizer [kg/m³]</th>
<th>Flow [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>M35</td>
<td>500</td>
<td>1500</td>
<td>250</td>
<td>20</td>
<td>225</td>
</tr>
<tr>
<td>M70</td>
<td>750</td>
<td>1350</td>
<td>225</td>
<td>20</td>
<td>240</td>
</tr>
</tbody>
</table>

Table 3.10: Mechanical properties of normal strength mortar

<table>
<thead>
<tr>
<th>Mix</th>
<th>$f_{cm}$ at 7days [MPa]</th>
<th>c.o.v (%)</th>
<th>$f_{cm}$ at 28days [MPa]</th>
<th>c.o.v (%)</th>
<th>$f_{ctm}$ [MPa]</th>
<th>c.o.v (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M35</td>
<td>24.9</td>
<td>1</td>
<td>35.4</td>
<td>2</td>
<td>2.7</td>
<td>2</td>
</tr>
<tr>
<td>M70</td>
<td>58.8</td>
<td>1</td>
<td>69.4</td>
<td>2</td>
<td>3.7</td>
<td>5</td>
</tr>
</tbody>
</table>
3.3.2.2 Mix proportions of high strength mortar

Initially, three series were investigated to achieve mortar with compressive strength more than 100MPa. The primary variable between these series is the amount of water and superplasticizer. Absolute volume method according to equation 3.1 was used in determining the amount of constituent materials quantities. This method was adopted in production ultra-high-performance fibre reinforced concrete (Aldahdooh et al., 2013).

Four different doses of the superplasticizer were used within each series to control the workability of the mix. Series1 had w/c ratio varied between 0.12 and 0.15 did not produce satisfactory mortar samples, see Figure 3.10, hence was discarded. Similarly, Corinaldesi and Moriconi, (2012) found that UHPFRC mix with 0.16 water binder ratio was not flowable and the workability of the matrix was increased gradually with increasing the w/b ratio up to 0.26 with the same level of mechanical properties.
Flow table test was used to determine the workability of mortar, as shown Figure 3.11. Table 3.11 presents the results of the mixes proportions, workability and compressive strength at 7-day age. The flowability of series2 (w/c ratio ranging between 0.16 and 0.19) and series3 (w/c ratio ranging between 0.21 and 0.24) are presented in Figure 3.12. The results demonstrate that the superplasticizer was more effective than water in increasing the workability of high strength mortar. The best ratio of superplasticizer to water ratio (sp/w) that exhibited the highest workability of series 2 and 3 was 0.15. However, all mixes of series3 showed high workability and exceeded the base of flow table regardless of the sp/w.
Table 3.11: Mix proportions and flowability of high strength mortar

<table>
<thead>
<tr>
<th>Series</th>
<th>(W+SP)/C</th>
<th>Mix</th>
<th>W/C</th>
<th>Sp/w</th>
<th>Cement [kg/m³]</th>
<th>Silica sand [kg/m³]</th>
<th>Water [kg/m³]</th>
<th>S.P [kg/m³]</th>
<th>Flow [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>M2-1</td>
<td>0.16</td>
<td>0.28</td>
<td>1100</td>
<td>1100</td>
<td>180</td>
<td>50</td>
<td>225</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M2-2</td>
<td>0.17</td>
<td>0.21</td>
<td>1100</td>
<td>1100</td>
<td>190</td>
<td>40</td>
<td>240</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M2-3</td>
<td>0.18</td>
<td>0.15</td>
<td>1100</td>
<td>1100</td>
<td>200</td>
<td>30</td>
<td>260</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M2-4</td>
<td>0.19</td>
<td>0.10</td>
<td>1100</td>
<td>1100</td>
<td>210</td>
<td>20</td>
<td>200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M3-1</td>
<td>0.21</td>
<td>0.28</td>
<td>1045</td>
<td>1045</td>
<td>215</td>
<td>60</td>
<td>300</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M3-2</td>
<td>0.22</td>
<td>0.21</td>
<td>1045</td>
<td>1045</td>
<td>225</td>
<td>47</td>
<td>300</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M3-3</td>
<td>0.23</td>
<td>0.15</td>
<td>1045</td>
<td>1045</td>
<td>237</td>
<td>35</td>
<td>300</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M3-4</td>
<td>0.24</td>
<td>0.10</td>
<td>1045</td>
<td>1045</td>
<td>247</td>
<td>25</td>
<td>300</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The results of the compressive and tensile strength of series 2 and 3 are presented in Table 3.12. Figure 3.13 demonstrates the effect of the amount of water and superplasticizer on the mechanical properties of high strength mortar. The compressive and tensile strength results demonstrate that the specimens of series 3 have lower strength than series 2. In addition, it was found that mixes of series three have about 60% the tensile strength of mixes of series 2. That could be to increase the amount of water, which could lead to increase the voids in the hardened mortar. It can be noted from the mix proportion of series three that the number of solid particles (cement and silica sand) was 5% less than series two. According to Neville (2011), increasing 5% percent of voids volume can reduce the strength to about 30%. Moreover, increasing the w/c ratio can lead, in particular with a high amount of superplasticizer, to create cracks due to bleeding (Neville, 2011). These
cracks can significantly reduce the tensile strength especially with small dimensions of the dog bone tensile test specimen. The results also indicated w/c ratio of 0.18 presents high performance regarding of mechanical properties and workability for casting high strength mortar, and the best amount of superplasticizer ranges between 30 and 35 kg/m³.

The best mix proportion was M2-3 of series2 with a tensile and compressive strength of 5.8MPa and 103.5MPa, respectively. Hence, this mix was adopted in the investigation the effect of SCMs on the behaviour of high strength mortar.

![Figure 3.13: Effect of superplasticizer on the mechanical properties of high strength mortar](image)

<p>| Table 3.12: Mechanical properties of series 2 and 3 of high strength mortar |
|---|---|---|---|---|---|</p>
<table>
<thead>
<tr>
<th>Series</th>
<th>Mix</th>
<th>$f_{cm}$ at 7days [MPa]</th>
<th>c.o.v. (%)</th>
<th>$f_{cm}$ at 28days [MPa]</th>
<th>c.o.v. (%)</th>
<th>$f_{cm}$ [MPa]</th>
<th>c.o.v. (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Series2</td>
<td>M2-1</td>
<td>81.8</td>
<td>2</td>
<td>98</td>
<td>2</td>
<td>5.7</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>M2-2</td>
<td>84.7</td>
<td>4</td>
<td>100.1</td>
<td>3</td>
<td>5.7</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>M2-3</td>
<td>89.8</td>
<td>3</td>
<td>103.5</td>
<td>3</td>
<td>5.8</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>M2-4</td>
<td>74.7</td>
<td>4</td>
<td>95</td>
<td>2</td>
<td>5.8</td>
<td>2</td>
</tr>
<tr>
<td>Series3</td>
<td>M3-1</td>
<td>77.2</td>
<td>3</td>
<td>94.9</td>
<td>2</td>
<td>3.3</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>M3-2</td>
<td>85.6</td>
<td>4</td>
<td>99.4</td>
<td>3</td>
<td>3.2</td>
<td>4</td>
</tr>
<tr>
<td></td>
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<td>95.9</td>
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3.3.2.3 Supplementary cementitious materials (SCMs) for high strength mortar

Many researchers, as stated in chapter two, observed the enhancement of the performance of cementitious materials due to addition the SCMs as a replacement of cement. However, most of these studies investigate a single set of tests of one of SCM, which make it difficult to conclude their enhancement on the performance of mortar with different amount and type of SCMs. On another side, some studies explained that single replacement could exhibit higher improvement than binary or ternary replacements. Whereas, others found binary and ternary replacements are more effective than single replacement.

The enhancement of SCMs to the strength of mortar comes from their pozzolanic reaction with the Ca(OH)$_2$ that resulted from cement hydration to produce supplementary C-S-H gel (Neville, 2011). As a result of this reaction, mortar with SCMs explains higher chemical adhesion bond by reducing the pores (improve the durability) of the hardened structure of mortar. This improvement in bond and durability has significant impacts on the performance of TRM as a strengthening layer. Some of the previous researchers indicated that up to about 30% replacement of cement with SCMs, higher performance of concrete could be achieved. Three types of SCMs have been added to the high strength mortar mix M2-3 (which is coded as M105) as a volumetric replacement of cement. The SCMs included Ground-granulated blast-furnace slag (GGBS) densified silica fume (DSF), undensified silica fume (UDSF) and fly ash (FA). These materials were added to the high strength mix in the form of single, binary and ternary replacement of cement, as presented in Table 3.13. Absolute volume method (equation 3.1) was used in calculation the mix proportions of the mix. The mixing procedure included firstly dry mixing for about 5 minutes of cementitious materials (cement and SCMs) then the addition of silica sand. After that, during mixing, the water with superplasticiser was adding, and the mixture was continued for about 10 minutes.

The first group included single replacement of each SCMs with three replacement ratios, 10%, 20% and 30% of cement. However, another replacement ratio of 40% was investigated for GGBS based on the observed enhancement of the literature review. The investigated replacement for binary replacements were 5%GGBS+5%SCM, 10%GGBS+10%SCM and 15%GGBS+15%SCM. The ternary replacement included 5% and 10% each type of the SCMs. The effect of these materials was investigated on the workability, compressive strength, tensile strength, chemical structural components and chloride ingress resistance of HSM.
Table 3.13: Mix proportions for high strength mortar including SCMs

<table>
<thead>
<tr>
<th>Designed mix</th>
<th>Replacement ratio [%]</th>
<th>GGBS [kg/m³]</th>
<th>DSF [kg/m³]</th>
<th>UDSF [kg/m³]</th>
<th>FA [kg/m³]</th>
<th>C [kg/m³]</th>
<th>S.S [kg/m³]</th>
<th>S.P [kg/m³]</th>
<th>W [kg/m³]</th>
<th>Flow [mm]</th>
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<td>990</td>
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<td>--</td>
<td>--</td>
<td>880</td>
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</tr>
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</tr>
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<tr>
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<td>1100</td>
<td>30</td>
</tr>
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</table>
3.3.2.4 Workability of SCMs based high strength mortar

The workability of mortar is an essential characteristic for successful casting of the TRM layers. High workability is required to ensure adequate penetration of mortar inside the meshes of the textile fibres. The results of the flow test of high strength mortar were 260mm. This amount of workability was found experimentally to be sufficient for full and easy penetration of mortar within the textile fibres. The addition of SCMs altered the workability of high strength mortar, as presented in Table 3.13.

Figure 3.14 shows the effect of addition SCMs on the workability of HSM. For single replacements, all replacement ratios of GGBS and FA exhibited increasing in the workability as the replacement ratio increases. The addition of GGBS improves the workability because of the surface characteristics of the GGBS particles, which are smooth and absorb little water during mixing (Neville, 2011). Similarly, most of FA particles are spherical and solid which reduce the water demand for given workability (Neville, 2011). Johari et al. (2011) reported a similar observation of enhancing the workability with addition FA and GGBS to high strength concrete. Bounkendakdi et al. (2012) observed a similar enhancement of GGBS for self-compacting concrete with cement replacement up to 25%. Dave et al. (2017) found that addition of FA was significantly increasing the workability of concrete.

However, mixes with DSF and UDSF explained a reduction in workability for mortar mix without SCMs. That was because of particles of silica fume has a high surface area which absorb a high amount of water during mixing and that led to increasing the water demand of the mix (Neville, 2011). Sezer (2012), Bagheri et al. (2013) and Jalal et al. (2015) observed a similar reduction in workability due to addition silica fume.

However, in binary replacement, the addition of GGBS combined with DSF improved the workability of the mortar for all replacement ratios. Similarly, but to less extent, the workability of mortar with UDSF enhanced as the replacement ratio increased in the presence of GGBS. However, the addition of GGBS with FA reduced the workability in comparison with the single replacement of FA counterparts. For ternary replacement, the workability of the mortar increases as the replacement ratio increases. Mixes with SCMs contain UDSF explained higher workability of mixes with DSF counterparts. These results agree with the findings of Dave et al. (2017).

Overall, the results showed the effectiveness of addition GGBS in improving the workability of mortar in all forms of replacements (single, binary and ternary). In addition, regardless to the type and amount of SCMs replacement, binary and ternary replacement improves the workability of high strength mortar.
3.3.2.5 Compressive strength of high strength mortar with SCMs

Compressive strength test is used to determine the behaviour of materials under compressive stresses. This behaviour is essential to gain an overall picture of the quality of mortar because the strength of mortar is directly related to the structure of the hydrated cement (Neville 2011). Compressive strength tests at ages 7, 28 and 90 days were carried out on cube mortar samples with dimensions 50×50×50 mm according to BS EN 12390-3:2009. Three specimens were tested, and the average strength was reported. A universal testing machine was used with a constant rate of displacement-based loading of 0.01 mm/sec. Table 3.14 shows that inclusion SCMs exhibited different influences on the compressive strength depending on the type and amount of replacement ratio and that agrees with the previous studies (Johari et al., 2011).
Table 3.14: Compressive test results of high strength mortar with SCMs

<table>
<thead>
<tr>
<th>Designed mix</th>
<th>$f_{cm}$ at 7 days [MPa]</th>
<th>c.o.v (%)</th>
<th>$f_{cm}$ at 28 days [MPa]</th>
<th>c.o.v (%)</th>
<th>$f_{cm}$ at 90 days [MPa]</th>
<th>c.o.v (%)</th>
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</thead>
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<td>3</td>
<td>114.4</td>
<td>3</td>
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</tr>
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<td>112.2</td>
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<td>126.1</td>
<td>3</td>
<td>147.7</td>
<td>3</td>
</tr>
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<td>2</td>
<td>132.4</td>
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<td>4</td>
<td>129.5</td>
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<td>107.7</td>
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</table>

Figure 3.15 shows that up to 10% single replacement of all types of SCMs, the compressive strength at all ages increased with respect to control mortar. However, the improvement was continued for GGBS and UDSF up to 20% replacement ratio. Higher replacement ratio did not present comparable enhancement in strength, which could be because of lack of resulted calcium hydroxide from cement hydration.

At 28 days, the compressive strength was ranged between 94.2MPa to 132.4MPa. That means the increase in compressive strength was about 28%. The activity of the SCMs in improving the strength depends mainly on the amount of amorphous SiO$_2$ that reacts with calcium hydroxide (Ca(OH)$_2$) resulted from cement hydration (Neville, (2011)). Therefore,
the compressive strength increases as the amount of SiO₂ increases. Accordingly, the highest compressive strength was observed for 20% of un-densified silica, but the lowest was found for 30% replacement ratio of densified silica fume. That may be attributed to the low workability of mixes with densified silica fume, which lead to reduce the compaction of the mix during casting, see Figure 3.16. Compaction of cementitious materials (concrete and mortar) during casting significantly affects the strength through controlling the voids in the structure of hydrated cementitious materials (Neville, 2011).

The results also explained that mixes with a single replacement of GGBS explained higher compressive strength than mixes with FA in spite both pozzolanic materials improve the workability and FA has a higher content of silica. That could be because of most of the silica in mineral composition of FA has a crystalline structure as found in X-Ray diffraction test of FA.

Strength development of mixes with SCMs increased with time because of increasing the amount of calcium hydroxide due to cement hydration. However, the maximum contribution of SCMs was observed at 28 days, and this contribution decreased at 90 days. That could be attributed to reducing the permeability of hardened mortar structure, which restricted the penetration of water inside mortar. That adversely affects the chemical reaction of the pozzolanic materials to produce additional calcium silicate hydrate (C-S-H). Many researchers (Johari et al., 2011; Gesoglu et al., 2009) have reported the superiority of silica fume in increasing the compressive strength of concrete.

Results of binary replacement, Figure 3.16, revealed that the compressive strength at 28 days ranged between 92.2MPa and 135.2MPa. The highest compressive strength was observed of binary replacement of GGBS and UDSF. For this replacement, the compressive strength increased as the replacement ratio increased. However, the compressive strength of binary replacement of 5%GGBS and 5%DSF explained higher strength than binary replacement of UDS. That refers to higher reactivity of densified silica fume, which led to decrease the strength with increase the replacement ratio due to lack of Ca(OH)₂. At the age of 90 days, all replacement ratios of binary replacement of GGBS and DSF explained the same compressive strength due to the availability of Ca(OH)₂. Binary replacement containing FA did not contribute to compressive strength at the initial stage, and compressive strength was lower than control beams. However, at 90 days, binary replacement of GGBS and FA up to 20% explained higher compressive strength than control mix. That could be due to the availability of Ca(OH)₂ that required for the pozzolanic reaction which is supported by literature finding (Yazici et al.,2012; Dave et al., 2016).
Overall, the enhancement of strength of binary replacement of SCMs was higher than single replacement and that conformed by Gesoglu et al. (2009).

Figure 3.17 demonstrates that ternary replacement of GGBS, DSF and FA at initial stages exhibited negligible contribution to strength but at 90 days age the compressive strength increases as the replacement increases. In contrast, 5% ternary replacement with un-densified silica fume increase the compressive strength, but 10% reduced the strength for all ages. That could be due to wet segregation of cement and water from the mix as a result of high workability (flow excess 300mm). This segregation reduces the compactness of mix, which exhibited decreasing in strength (Neville, 2011). The compressive strength at 28days of binary replacement ranged between 93.5MPa and 119.6MPa, which indicate that binary replacement in more effective in enhancing the compressive strength and these results agree with the findings of Gesoglu et al. (2009).

Figure 3.15: Effect of single replacement of SCMs on compressive strength of HSM
Figure 3.16: Effect of binary replacement of SCMs on compressive strength of HSM

Figure 3.17: Effect of ternary replacement of SCMs on compressive strength of HSM
3.3.2.6 Tensile strength of high strength mortar with SCMs

The tensile strength of mortar is essential in determining the load that causing cracking of mortar. The chemical adhesion bond between the results of cement hydration and aggregates controls this strength (Neville, 2011). This effect can be extended to estimate the quality of adhesion bond between hydrated cement and textile fibres. Therefore, a mix of higher tensile strength can present higher bond strength for textile fibres. Direct tensile test of three dog bone specimens of each mix was used to determine the effect of SCMs on the tensile strength of HSM. The test was implemented in the same prescribed procedure of normal and high strength mortar.

Table 3.15 and Figure 3.18 show the results of the direct tensile test. The effects of single replacement explained that the addition of GGBS and fly ash decrease the tensile strength. While addition un-densified and densified silica fume improve the tensile strength. That could be attributed to less silica content of FA and GGBs. Dave et al. (2017) observe a similar reduction in tensile strength of concrete with 30% replacement of FA. The tensile strength of mixes with a single replacement of SCMs ranged between 4.3MPa and 6.7MPa. The un-densified silica fume exhibited a continuous increase in tensile strength. However, addition densified silica fume has an optimum replacement ratio of 10% and after that, the tensile strength decreases. That could be due to decrease the compactness of mix because of reducing the flowability of mixes with densified silica fume.

Binary replacement explained higher tensile strength, ranged between 5.3MPa and 6.7MPa than single replacements. The highest tensile strength was observed for addition 15% GGBS and 15%UDSF that yielded 6.7MPa. The lowest tensile strength was observed for binary replacement of GGBS with FA. That could be because of less silica (about 59%) in the chemical composition of FA in comparison with silica fume.

In ternary replacement, densified silica fume and fly ash explained negligible enhancing the tensile strength. While un-densified silica fume exhibited improving the tensile strength with replacement ratio equal to 15%. Above this replacement ratio, a reduction in tensile strength was observed. The tensile strength was ranged between 4.5MPa and 6.3MPa, which explains less contribution of tensile strength in comparison with binary replacements.
Figure 3.19 demonstrates a consistency of the general trend of the tensile and compressive strength of high strength mortar with SCMs. However, there is no direct proportionality between the two types of strength, and in some cases, particularly of high replacement ratio, the increase in compressive strength did not accompany with increasing in tensile strength, see Figure 3.20. That could be because of in case of high replacement ratio and the lack of calcium hydroxide, the SCM work as a filler rather than cementitious material. The figure also explains that binary replacements demonstrate higher strength enhancement than single and ternary replacements.
Figure 3.18: Effect of SCMs replacement on tensile strength of HSM

Figure 3.19: Variation of mechanical properties with single, binary and ternary replacement of SCMs
Figure 3.20: Relationship between compressive and tensile strength of HSM with SCMs

### 3.3.2.7 X-Ray diffraction analysis of high strength mortar with SCMs

X-ray diffraction test of powder samples was performed using X’Pert diffractometer with CuK radiation ($\lambda = 1.5418$ Å) operating at 40 kV, 30 mA (Figure 3.21). The purpose of this test is to identify the variation of the amount of calcium hydroxide (Ca(OH)$_2$) due to inclusion SCMs through their reaction with Ca(OH)$_2$ to produce supplementary C-S-H. The analysis of the X-ray diffraction pattern is based considering Ca(OH)$_2$ peaks appears at 2$\theta$ equal to 34° (Liu et al., 2017; Alhazaimy et al., 2012; Lothenbach et al., 2011; Seleem et al., 2010; Supit et al., 2014). The test of samples includes preparing a powder of the sample that tested under compression load at 28 days and then measures the reflection of the X-ray. Then the results of the test were analysed using the provided software with the X-ray machine.

Figure 3.22 presents the intensity peaks of Ca(OH)$_2$ obtained from the analysis of the X-Ray diffraction test of high strength mortar with SCMs. As shown in the figure, inclusion SCMs reduced the amount of calcium hydroxide for control mix (M105). This reduction increased as the replacement ratio increases. That indicates to the consumption of calcium hydroxide in the pozzolanic reaction of SCMs. The results demonstrate that SCMs with higher content of amorphous SiO$_2$, such as densified and un-densified silica fume, is more effective in contributing the pozzolanic reaction products which present
higher strength enhancement. These findings agree with the observation of Supit et al. (2014).

The results demonstrate high consistency between the obtained compressive strength and the intensity peaks of Ca(OH)$_2$. For the same replacement ratio, as the intensity peaks decrease, the compressive strength increases. However, the addition of densified silica fume with replacement ratio 20 and 30% exhibited a reduction in intensity peaks of Ca(OH)$_2$ but without a considerable contribution to strength enhancement. That could be attributed to the decrease in the flowability of the matrix, which in turns decrease the compaction of the mix during casting and increase the volume of voids (Neville, 2011).

The best performance in consumption Ca(OH)$_2$ was observed for mixes with un-densified silica fume and that because of its high content of SiO$_2$. In contrast, mixes contain fly ash present the highest intensity peaks of Ca(OH)$_2$ among all SCMs mixes. That demonstrates the low compressive strength that observed of mortar with FA in comparing with control mortar.

Overall, binary replacement of 15% GGBS and 15% UDSF explained the lowest intensity peak of Ca(OH)$_2$ which refers to its activity in the pozzolanic reaction that led to enhance the strength of mortar. This finding coincides with the highest compressive and tensile strength of this mix proportion.

![Figure 3.21: X-ray diffraction test machine](image)
3.3.2.8 Chloride penetration resistance of high strength mortar with SCMs

The resistance of cementitious materials to chloride penetration has a considerable influence on their durability. As part of this study to investigate the durability of strengthening layer, a chloride migration test was carried out on high strength mortar has SCMs. In this study, the strengthening layer will be added only to the bottom side of the beams since the investigating was devoted to the flexural behaviour of RC beams strengthened with TRM. Furthermore, this arrangement was considered sufficient to explore the ability of the TRM layer to protect the substrate concrete. However, it is recognised that the protection will perhaps be required at all sides that will be expose to the harmful environments.

In addition, mortar resistance to chloride penetration presents general indicator of the internal structure of mortar employing type and volume of pores (Neville, 2011). The presence of pores affects the strength of mortar, and this influence continues to the adhesion bond between two cementitious materials cast at a different age.

Chloride migration coefficient from non-steady-state migration experiments was carried out to quantify chloride penetration of high strength mortars. The experiments require cylindrical specimens with a diameter of 100mm and a depth of 50mm, sliced from casted
cylinders. The test was carried out based on the Nordtest satisfies NT build 492:2011. This method has been used by Elahi et al. (2010), Borosnyoi and (2016) and Karein et al. (2017) to determine the mortar resistance to chloride penetration.

The concept of this test is based on the resistance of hardened cementitious materials to the penetration of chloride ion when the sample is subjected to external potential to force the chloride ion to migrate from outside to inside the sample. Three samples of each mortar mix were placed in the vacuum container for vacuum treatment for three hours and then, the container was filled saturated Ca(OH)₂ for one hour. After that, the sample kept in the solution for 18 hours before applying the potential. The duration of the applied potential is recorded and then the sample is axially split, and a silver nitrate solution is sprayed on the freshly split sections. The colour of areas that contain chloride ion will change, and then the chloride penetration depth can be measured as the depth of changed colour areas. After that, the chloride migration coefficient can be calculated based on the penetration depth. Figures 3.23 and 3.24 show the arrangement of the migration set-up.

![Set-up of the chloride migration test (NT build 492:2011)](image)

The chloride non-steady-state migration coefficient is calculated using the following equations (NT build 492:2011):

\[ D_{nssm} = \frac{RT}{ZFE} \cdot \frac{x_d - \alpha \sqrt{x_d}}{t} \]  

(3.2)

Where:

\[ E = \frac{v-2}{l} \]  

(3.3)
\[
\alpha = 2 \frac{RT}{zFE} \cdot erf^{-1} \left( 1 - \frac{2c_d}{c_0} \right)
\]  
(3.4)

\(D_{nssm}\): non-steady-state migration coefficient, \(m^2/s\)

\(z\): absolute value of ion valence, for chloride, \(z = 1\)

\(F\): Faraday constant, \(F = 9.648 \times 10^4 \, \text{J/(V\cdot mol)}\)

\(U\): absolute value of the applied voltage, \(V\)

\(R\): gas constant, \(R = 8.314 \, \text{J/(K\cdot mol)}\)

\(T\): average value of the initial and final temperatures in the anolyte solution, \(K\)

\(L\): thickness of the specimen, \(m\); \(x_d\): average value of the penetration depths, \(m\)

\(t\): test duration, seconds

\(erf^{-1}\): inverse of error function

\(c_d\): chloride concentration at which the colour changes, \(c_d \approx 0.07 \, \text{N for OPC concrete}\)

\(c_0\): chloride concentration in the catholyte solution, \(c_0 \approx 2 \, \text{N}\).

Since the expression \(erf^{-1} \left( 1 - \frac{2c_d}{c_0} \right) = 1.28\), the following simplified equation can be used:

\[
D_{nssm} = \frac{0.0239(273+T)}{(U-Z)t} \left( x_d - 0.0238 \sqrt{\frac{(273+T)Lx_d}{(U-Z)}} \right)
\]  
(3.5)

where:

\(D_{nssm}\): non-steady-state migration coefficient, \(\times 10^{-12} \, \text{m}^2/\text{s}\);

\(T\): average value of the initial and final temperatures in the anolyte solution, \(\degree\text{C}\);

\(x_d\): average value of the penetration depths, \(\text{mm}\);

\(t\): test duration, \(\text{hour}\)

Figure 3.24 shows the experimental setup of the chloride migration test.
a. Preparation samples in the vacuum container

b. Applying electric potential

c. Measuring chloride penetration depth

Figure 3.24: Process of chloride migration test of HSM
Table 3.16 and Figure 3.25 present the results of the chloride migration test of the tested high strength mortars. The results demonstrate that all SCMs significantly improve the permeability of high strength mortar. The enhancement in chloride penetration resistance increased with increasing the replacement ratio of all types of SCMs. However, 10% singly replacement ratio of FA and GGBS showed marginal enhancement in resistance to chloride penetration. That may be because of the presence of continued pores inside the structure of the hardened sample.

The enhancement of single replacement of SCMs ranged between 2% and 83%. The highest improvement was observed for replacements with both types of silica fume. That could be attributed to the high pozzolanic reaction of silica fume because of the high content of amorphous SiO₂. That changes the microstructure by producing additional C-S-H, which fill the pores of the hardened structure and reduce the permeation of chloride. Karein et al. (2017) found that the addition of silica fume to normal strength mortar reduces the chloride migration coefficient to about 84% of control mortar. Similar enhancement but with lesser extent (maximum increase about 30% and 76% of GGBS and FA, respectively) was observed for mixes with replacement ratios of 20% and 30% of GGBS and FA. These results agree with the observed behaviour of single replacement of SF, FA and GGBS by Elahi et al. (2010) of high strength concrete.

Binary replacement explained higher resistance to the chloride penetration than single replacements and similar behaviour observed by Elahi et al. (2010) for binary replacement of SF, FA and GGBS. The enhancement of binary replacement ranged between 22% and 95% with respect to control mortar. Similar to the single replacement, silica fume exhibited the highest resistance to chloride diffusion of both densified and un-densified forms. However, un-densified silica fume explained 5% more than the enhancement of densified form. That could be due to higher compaction of the mix with un-densified silica fume because of higher workability. On other hands, binary replacement of GGBs and FA demonstrates same enhancement of single replacement of GGBS, which is less than the single replacement of FA. That means the combination of GGBS and FA reduces the activity of FA in enhancing the chloride resistance penetration. That may be due to both pozzolana materials increasing the flowability to level led to segregation the mix, which increase the pores in the hardened structure of mortar.
<table>
<thead>
<tr>
<th>Designed mix</th>
<th>$D_{assm} \times 10^{-12}$ [m$^2$/s]</th>
<th>c.o.v. [%]</th>
<th>R [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>M105</td>
<td>1.98</td>
<td>5</td>
<td>0</td>
</tr>
<tr>
<td>M10G</td>
<td>1.94</td>
<td>4</td>
<td>2</td>
</tr>
<tr>
<td>M20G</td>
<td>1.44</td>
<td>3</td>
<td>27</td>
</tr>
<tr>
<td>M30G</td>
<td>1.39</td>
<td>7</td>
<td>30</td>
</tr>
<tr>
<td>M40G</td>
<td>1.12</td>
<td>2</td>
<td>43</td>
</tr>
<tr>
<td>M10D</td>
<td>0.81</td>
<td>4</td>
<td>59</td>
</tr>
<tr>
<td>M20D</td>
<td>0.56</td>
<td>2</td>
<td>72</td>
</tr>
<tr>
<td>M30D</td>
<td>0.35</td>
<td>2</td>
<td>82</td>
</tr>
<tr>
<td>M10U</td>
<td>0.67</td>
<td>8</td>
<td>66</td>
</tr>
<tr>
<td>M20U</td>
<td>0.54</td>
<td>7</td>
<td>73</td>
</tr>
<tr>
<td>M30U</td>
<td>0.34</td>
<td>8</td>
<td>83</td>
</tr>
<tr>
<td>M10F</td>
<td>1.94</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>M20F</td>
<td>1.53</td>
<td>6</td>
<td>23</td>
</tr>
<tr>
<td>M30F</td>
<td>0.47</td>
<td>5</td>
<td>76</td>
</tr>
<tr>
<td>M5G5D</td>
<td>0.34</td>
<td>8</td>
<td>83</td>
</tr>
<tr>
<td>M10G10D</td>
<td>0.28</td>
<td>4</td>
<td>86</td>
</tr>
<tr>
<td>M15G15D</td>
<td>0.2</td>
<td>4</td>
<td>90</td>
</tr>
<tr>
<td>M5G5U</td>
<td>0.42</td>
<td>2</td>
<td>79</td>
</tr>
<tr>
<td>M10G10U</td>
<td>0.33</td>
<td>5</td>
<td>83</td>
</tr>
<tr>
<td>M15G15U</td>
<td>0.09</td>
<td>3</td>
<td>95</td>
</tr>
<tr>
<td>M5G5FA</td>
<td>1.54</td>
<td>3</td>
<td>22</td>
</tr>
<tr>
<td>M10G10F</td>
<td>1.36</td>
<td>2</td>
<td>31</td>
</tr>
<tr>
<td>M15G15F</td>
<td>1.27</td>
<td>5</td>
<td>36</td>
</tr>
<tr>
<td>M5G5D5F</td>
<td>1.7</td>
<td>4</td>
<td>14</td>
</tr>
<tr>
<td>M10G10D10F</td>
<td>0.46</td>
<td>8</td>
<td>77</td>
</tr>
<tr>
<td>M5G5U5F</td>
<td>1.04</td>
<td>8</td>
<td>47</td>
</tr>
<tr>
<td>M10G10U10F</td>
<td>0.18</td>
<td>8</td>
<td>91</td>
</tr>
</tbody>
</table>

R represents the percentage reduction of chloride migration coefficient in comparison with M105.

In ternary replacement, UDSF explained higher activity than DSF for all investigated replacement ratio. The highest enhancement (91%) was observed of 30% replacement ratio of UDSF, FA and GGBS. This enhancement is higher than the observed increase of the individual replacement of SCMs. However, binary replacement is still explained the best performance among single and ternary replacements.

The results also demonstrate that the compressive strength is not directly proportional to the chloride diffusion, as shown in Figure 3.25. The strength enhancement of 10% replacement ratio of all SCMs was accompanied by their improvement of resistance to
chloride penetration. However, higher replacements ratio (20% and 30%) explained non-proportional relationship depending on the type and amount of SCMs. That may be because of with high replacement ratio some of the SCM did not react to produce addition C-S-H and only serve as a filler, which enhances the structure density of mortar without considerable improvement in strength. Binary replacements of GGBS and UDSF explained high consistency between the strength and the chloride penetration resistance.

The overall results, demonstrate the efficiency of the SCMs in increasing the resistance of chloride penetration particularly with binary replacement of GGBS and un-densified silica fume.

![Figure 3.25: Effect of SCMs replacement on coefficient of migration ($D_{nm}$) of HSM](image)

### 3.3.2.9 Selection of high performance mortar mix

The main aspects of selection the mix proportions of HSM are workability, tensile strength and chloride migration coefficient. These aspects have been chosen because of in casting TRM layer, high workability is required to provide sufficient penetration of mortar within the mesh of fibres. The TRM strengthen layer will be subjected to tensile stress. Therefore, the tensile strength of mortar affects the performance of the TRM layer. Besides, the mortar should be durable against aggressive agents such as chloride penetration, which is identified in this study by means of the chloride migration coefficient ($D_{nm}$). This coefficient is considered a decisive factor in selecting the mortar for durability aspects. A lower value of $D_{nm}$ represents high resistance to chloride penetration, which reflects
higher durability. Table 3.17 shows the summarised results of the effect of SCMs on the performance of HSM.

Based on the analysis of the obtained data from flow table, compressive, tensile and permeability tests, the best mix proportions from the trailed mixes is the binary replacement of 15% of GGBS and 15% of un-densified silica fume. This mix proportions will be used as a cementitious matrix to cast high-performance TRM, and it will be coded as M135. For comparison purpose mortar strength of 35MPa, 70MPa and 105 MPa were also investigated in the performance of TRM composite.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Flow [mm]</th>
<th>$f_{cm}$ [MPa]</th>
<th>$f_{ctm}$ [MPa]</th>
<th>$D_{nsm} \times 10^{12}$ [m$^2$/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>M105</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10 GGBS</td>
<td>265</td>
<td>114.4</td>
<td>5.7</td>
<td>1.94</td>
</tr>
<tr>
<td>20 GGBS</td>
<td>275</td>
<td>115.3</td>
<td>4.7</td>
<td>1.44</td>
</tr>
<tr>
<td>30 GGBS</td>
<td>295</td>
<td>103.3</td>
<td>4.5</td>
<td>1.39</td>
</tr>
<tr>
<td>40 GGBS</td>
<td>300</td>
<td>100.1</td>
<td>4.3</td>
<td>1.12</td>
</tr>
<tr>
<td>10 DSF</td>
<td>200</td>
<td>113.2</td>
<td>6.7</td>
<td>0.81</td>
</tr>
<tr>
<td>20 DSF</td>
<td>175</td>
<td>112.2</td>
<td>6</td>
<td>0.56</td>
</tr>
<tr>
<td>30 DSF</td>
<td>165</td>
<td>94.2</td>
<td>5.4</td>
<td>0.35</td>
</tr>
<tr>
<td>10 USF</td>
<td>250</td>
<td>126.1</td>
<td>5.7</td>
<td>0.67</td>
</tr>
<tr>
<td>20 USF</td>
<td>240</td>
<td>132.4</td>
<td>6.2</td>
<td>0.54</td>
</tr>
<tr>
<td>30 USF</td>
<td>210</td>
<td>110.7</td>
<td>6.7</td>
<td>0.34</td>
</tr>
<tr>
<td>10 FA</td>
<td>300</td>
<td>114.3</td>
<td>5.4</td>
<td>1.94</td>
</tr>
<tr>
<td>20 FA</td>
<td>300</td>
<td>105.9</td>
<td>5</td>
<td>1.53</td>
</tr>
<tr>
<td>30 FA</td>
<td>300</td>
<td>105.7</td>
<td>4.5</td>
<td>0.47</td>
</tr>
<tr>
<td>5GGBS+5DSF</td>
<td>300</td>
<td>121.1</td>
<td>6.4</td>
<td>0.34</td>
</tr>
<tr>
<td>10GGBS+10DSF</td>
<td>300</td>
<td>108.2</td>
<td>6</td>
<td>0.28</td>
</tr>
<tr>
<td>15GGBS+15DSF</td>
<td>300</td>
<td>106.9</td>
<td>5.3</td>
<td>0.2</td>
</tr>
<tr>
<td>5GGBS+5UDSF</td>
<td>270</td>
<td>110.5</td>
<td>6.6</td>
<td>0.42</td>
</tr>
<tr>
<td>10GGBS+10UDSF</td>
<td>285</td>
<td>121</td>
<td>6.5</td>
<td>0.33</td>
</tr>
<tr>
<td>15GGBS+15UDSF</td>
<td>300</td>
<td>135.2</td>
<td>6.7</td>
<td>0.09</td>
</tr>
<tr>
<td>5GGBS+5FA</td>
<td>270</td>
<td>99</td>
<td>5.9</td>
<td>1.54</td>
</tr>
<tr>
<td>10GGBS+10FA</td>
<td>280</td>
<td>103.7</td>
<td>5.7</td>
<td>1.36</td>
</tr>
<tr>
<td>15GGBS+15 FA</td>
<td>300</td>
<td>92.2</td>
<td>5.6</td>
<td>1.27</td>
</tr>
<tr>
<td>5GGBS+5DSF+5FA</td>
<td>270</td>
<td>93.5</td>
<td>5.1</td>
<td>1.7</td>
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<tr>
<td>10GGBS+10DSF+10FA</td>
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<td>119.7</td>
<td>5.7</td>
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</tr>
<tr>
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<td>6.3</td>
<td>1.04</td>
</tr>
<tr>
<td>10GGBS+10UDSF+10FA</td>
<td>300</td>
<td>107.7</td>
<td>4.5</td>
<td>0.18</td>
</tr>
</tbody>
</table>
3.3.3 Textile reinforced mortar

As previously stated in chapter 2, the behaviour of TRM composites is characterised by the mortar properties, amount and mechanical properties of textile fibres. Accordingly, the effect of these parameters on the tensile behaviour of TRM was investigated.

The investigated parameters included four mortar compressive strengths of 35MPa, 70Mpa, 105Mpa and 135MPa, in addition to the different amount of textile basalt and carbon fibres. These aspects were investigated for TRM composites under direct tensile and flexural stresses. In addition, as a strengthening layer, the shrinkage of TRM affects the bond strength between TRM layer and the substrate RC member. Therefore, free drying shrinkage behaviour of TRM composite was investigated of TRM specimens and composites TRM – concrete samples.

3.3.3.1 Tensile strength of TRM

Based on the conducted literature review in Chapter 2, many researchers investigated the effect of variant properties of textile fibres on the tensile strength of TRM composites. However, there is a lack of information of the contribution of different mortar strength on the tensile strength. This section is devoted to investigating the effect of mortar strength on the behaviour of TRM composites subjected to direct tensile stresses. High strength mortar provides higher adhesion chemical bond, which could significantly improve the transferring stresses to the textile fibres that explains the higher tensile strength of TRM composites. The investigated parameters within this section include mortar strength, textile fibre strength and textile reinforcement ratio.

Normal strength mortar with a nominal compressive strength of 35MPa and 70MPa were investigated. Moreover, HSM with a nominal compressive strength of 105 MPa and 135MPa were used. Two types of textile fibres were used; basalt fibres with opening mesh size 5mm and carbon fibres with opening meshes size 10mm. The effect of the textile reinforcement ratio (number of layers within the cross-section of the sample) is investigated through applying 2, 3, 4, 8 and 16 layers of textile basalt fibres. For textile carbon fibres, 1, 3, 5, and ten layers were implemented.

Casting specimens included casting mortar up to designed level and putting the textile fibres, then adding mortar to another level and once again placing the textile fibres until reaching the required number of textile layer. Direct tensile test of dog bone specimens was used to assess the tensile behaviour of TRM composites. A universal testing machine with a rate of displacement control-based loading of 0.02 mm/sec was used.
The specimens are coded in the format of Mx-yz, where M refers to the mortar, x represents the nominal mortar compressive strength, y refers to the number of textile layers and z refers to the type of fibre (B for basalt and C for carbon). For example, M35-8B represented a mortar specimen of 35MPa compressive strength and reinforced with eight layers of textile basalt fibres.

The tensile strength of TRM composites was calculated by dividing the ultimate tensile load on the cross section of dog bone specimen. The tensile strength of textile fibres was calculated by dividing the ultimate tensile strength on the area of textile fibres within the TRM specimen. Many researchers used this procedure to determine the actual tensile stresses of textile fibres under direct tensile stresses (Donnini et al., 2016; Bilotta et al., 2017; Raoof et al., 2017; Younis et al., 2017; Larrinaga et al., 2014; Hegger and Voss, 2008). According to Hegger and Voss (2008), textile efficiency coefficient \( K_f \) can be determined according to the following equation:

\[
K_f = \frac{f_f}{f_{fu}}
\]

Where: \( f_f \) is the tensile stress in textile fibres within the TRM composite (MPa) and \( f_{fu} \) is the ultimate tensile strength of textile fibres (MPa). The area of textile was determined based on the specific gravity of textile fibres as demonstrated in Appendix A.

Table 3.18 and Figure 3.26 present the tensile strength of textile fibres and TRM composites. Three types of failure modes were observed depending on the mortar and fibre properties, as shown in Figure 3.27. These failure modes included textile rapturing (TR), Textile slippage (TS) and Mortar splitting (MS) which agree with the experimental observation of previous studies (Hegger and Voss, 2008; Bilotta et al., 2017).

TRM tensile strength was significantly improved by using high strength mortar. The ultimate tensile strength of TRM composite with 16 layers of textile basalt fibres increased from 6.53MPa of M35 to 14.77MPa of M135. Similarly, the tensile strength of carbon TRM of M35 and M135 were 7.42MPa and 13.47MPa, respectively. That could be attributed to the increase of the adhesion bond between mortar and textile fibres due to the higher content of C-S-H gel that resulted from hydrated cement.

The tensile strength of TRM composites of mortar M35 ranged between 3.4MPa and 6.53MPa of textile basalt fibres. Specimens with two layers of basalt fibres explained textile rapturing failure mode, see Figure 3.27 a. However, despite increasing the tensile strength, higher amount of textile fibres (4 and 8 layers) led to slippage of textile, as shown in figure 3.27.b. That could be due to increasing the transferring stresses, which exceeded
the textile-mortar bond strength. 16 basalt layers of M35 TRM composite reduced the
cross-section area between the textile layers, which led to double the transferring stresses.
These stresses were higher than the tensile strength of M35, which caused splitting of
mortar by means of cracks along textile layers, see Figure 3.27 c. Similar behaviour with
greater extent was observed of TRM carbon fibres of with mortar M35. That could be
because of the configuration of textile carbon fibres has higher filament area, which
concentrates the stresses in mortar in comparing with textile basalt fibres. Wu and Li
(2017) found similar low contribution (tensile strength of 4.2MPa) of PVA textile fibres
(tensile strength 1200MPa) impregnated with normal strength mortar.

Figure 3.26: Effect of fibre properties on tensile strength of TRM
Table 3.18: Direct tensile test results of TRM

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Fibre type</th>
<th>No. of layer</th>
<th>$f_i$ [MPa]</th>
<th>c.o.v. (%)</th>
<th>Failure mode</th>
<th>$f_{exp.}$ [MPa]</th>
<th>$f_{fu}$ [MPa]</th>
<th>$k_f$</th>
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</table>

$f_i$ is the tensile strength of the TRM composite.  
$f_{exp.}$ is the experimental tensile strength of the textile in the composite.  
$f_i$ is the ultimate tensile strength of the textile fibre.  
$k_f$ is the coefficient of the effectiveness of the textile fibres.  
TR is the textile rapture, MS is mortar splitting and TS is the textile slipping.
a. Textile rapture (TR)  
b. Textile slipping (TS)  

c. Mortar splitting (MS)  

Figure 3.27: Failure modes of TRM under direct tensile stresses  

TRM specimens with mortar strength 70MPa explained higher strength (11.53MPa and 9.95 MPa of the ultimate tensile strength of basalt and carbon TRM, respectively) than samples with mortar strength 35MPa. That was mainly due to increase the bond strength between the fibres and the surrounding mortar. Specimens reinforced with 2 and four textile basalt fibres explained fracturing failure mode while 8 and 16 layers slip from the mortar, see Figure 3.27 a and b, respectively. Despite the increase in tensile strength, 5 and ten carbon textile layers caused splitting of mortar while slippage of textile was observed for 1 and three layers.

The results explained that with the same amount and type of fibres, the tensile strength of specimens with mortar M105 exhibited higher tensile strength than NSM counterparts. As mentioned before, this was due to increase the bond between the mortar and the textile fibres because of increasing the C-S-H gel. The ultimate tensile strength of TRM composites was 14.28MPa and 12.52MPa of textile basalt and carbon fibres, respectively. The superiority of basalt samples may be due to the configuration of the basalt textile, which distributes the transferring stresses on a larger area than carbon fibres. In addition,
no mortar splitting failure mode was observed of samples with textile carbon fibres, and multiple cracks in mortar were observed before failure. TRM specimens with mortar strength of 135MPa explained the highest tensile strength among all other types of mortars. Basalt TRM composite of 2, 4 and eight layers failed due to reaching the ultimate tensile capacity of the textile reinforcement (Figure 3.27a). However, slippage of textile was observed of specimens with 16 textile basalt fibres, as shown in Figure 3.27b. All samples with textile carbon, except samples of one layer, explained slippage of textile fibres with lesser tensile strength than basalt counterparts. It is worth to mention that the enhancement in tensile strength was slight between samples of mortar M105 and M135 but samples of mortar M135 exhibited denser multiple cracking. That could be because of reaching, in most cases, the ultimate capacity of textile fibres. Otherwise, it indicates a less considerable contribution of mortar strength beyond mortar strength of 135MPa.

Figure 3.28 demonstrates the effect of mortar strength and textile properties on the efficiency of textile fibres in tensile strength of TRM composites. The results indicate that as the mortar strength increases, the contribution of textile fibres increases. Moreover, textile fibres with lesser filament area are more effective because of distributing the stresses over a more extensive area and avoiding stress concentration that could cause cracking of mortar. Appendix A presents the stress-strain curves of TRM composites.

Overall, the tensile strength of TRM composites is significantly influenced by the mortar properties and textile configuration, which control the stresses distribution within mortar structure. Implementation normal strength mortar (M35 and up to M70) in TRM composites limited the expected tensile strength due to splitting of mortar with high textile reinforcement ratio. On other hand, high strength mortar provides higher strength for the same type, amount and configuration of textile fibres and prevent splitting mortar even though with relatively high amount of textile fibres. However, the efficiency of textile contribution in tensile strength of TRM composites proportions adversely with the textile reinforcement ratio.
3.3.3.2 Flexural strength of TRM

In the field of strengthening of RC members, TRM composite is usually subjected to tensile stresses. These stresses are tensile stresses resulted from flexural loading and not direct tensile stresses. Although the apparent relationship between both types of stresses, but the behaviour of composites is different due to the stress distribution differences within specimen cross-section. The effect of these stresses on the ultimate capacity and failure mode was investigated using specimens with dimensions $25\times75\times285$ mm$^3$. All samples have the same thickness of the direct tensile test samples to investigate the correlation between the behaviour of textile fibres under direct tensile and flexural stresses state. Figure 3.29 shows the distribution of the textile fibres within the cross-section of samples.

The effect of the textile reinforcement ratio is investigated through applying 4, 8 and 16 layers of textile basalt fibres and 3, 5, and ten layers of textile carbon fibres. These amounts of fibres were investigated to obtain the similar textile reinforcement ratio of direct tensile test specimens. In addition, the same four mortar strengths that examined with direct tensile test were implemented in the flexural test. The samples are coded in the same format of dog bone samples of the direct tensile test.
The investigated samples (three samples of each case study) were cast in steel moulds and de-moulding after two days. After 28 days wet curing, the samples were tested under three point of loading using a universal machine with displacement control rate of loading of 0.02 mm/sec, as shown in Figure 3.30. The flexural stresses were calculated on the base of three-point bending arrangement that used in determining the flexural strength of concrete samples (Tsesarky et al., 2013).

Table 3.19 presents the flexural strength and the observed failure mode of the tested TRM samples. All samples without textile fibres exhibited flexural failure mode with a single crack at the mid-span (Figure 3.31). The flexural strength of mortars M35, M70, M105 and M135, were 3.6MPa, 5.4MPa, 12.2MPa and 13.2MPa, respectively. The flexural strength of mortar M135 was more than three times the strength of mortar M35. That was due to the higher mechanical properties of mortar M135 in comparison with M35.
The inclusion of textile fibres significantly increased the flexural strength of mortar depending on the type and amount of fibres, as shown in Figure 3.32. Four failure modes were observed during the flexural test of TRM. All samples with four layers of textile basalt fibres explained flexural failure mode with single cracks at the mid-span with tensile strength ranged between 10.4MPa and 12.6 Mpa. The contribution of textile basalt was clear for normal strength mortar (M35 and M70) by increasing the flexural strength to about 308% and 192% of mortar M35 and M70, respectively. However, a negligible contribution was observed for high strength mortar (M105 and M135). That was because the initial flexural strength of mortar was more significant than the expected flexural strength of TRM reinforced with four layers.

Increasing the amount of textile fibres (8 and 16 layers) of M35 mortar samples changed the failure mode to slippage of textile fibres, and that was noticed through the horizontal crack along the textile layer, as shown in Figure 3.31 d. Increasing the strength of mortar improves the flexural behaviour through increasing the ultimate flexural strength to about 100%, and change the failure mode to flexural with multiple cracking (Figure 3.31 c). That could be attributed to increasing the bond strength of the textile fibres and mortar due to increasing adhesion chemical bond that resulted from hydrated cement. The ultimate flexural strength was found for TRM specimen with mortar M135 reinforced with 16 layers (46.6Mpa).

The slippage between the textile fibres and mortar M35 was evident for all specimens of textile carbon fibre (Figure 3.31 c). Samples with three and five layers of carbon fibres exhibited horizontal carks along the textile layer, and the flexural strength was approximately the same. The later explained multiple vertical and horizontal cracks and higher stiffness than M35-3C. Specimens with ten carbon layers exhibited lower strength than samples with 3 and 5 layers. That could be due to reducing the amount of mortar between the textile layers, which led to a high concentration of stresses at the textile mortar interface and caused textile slippage.

TRM specimens of mortar M70 and M105 explained higher strength than mortar M35 counterparts, but the flexural strength ranged between 21.2 MPa and 30.6 MPa due to the textile slippage. However, M135 increased the flexural strength to 41.1 MPa with ten layers of textile carbon fibres. Inclined cracks with an angle equal to about 45° were observed (Figure 3.31 d) which refers to flexural shear failure. It worth to mention, that for all strengths mortar, textile basalt fibres explained higher performance than carbon fibres which could be attributed to the high area filaments which localise the stresses and destroying mortar.
Figure 3.33 demonstrates that regardless of the mortar and textile properties, the flexural strength of TRM composites is much higher than the tensile strength obtained from the direct tensile test. With taking in account, that some of the textile fibres, which is in the compression zone, did not contribute to resisting the tensile stresses. It may be stated that the behaviour of TRM composites as a strengthening differs from their behaviour under direct tensile strength because it will be obtained lower estimated values of the tensile strength. Appendix A presents the load-deflection curves of TRM composites under flexural loading.

The results demonstrate the effect of mortar strength on the behaviour of TRM composites through controlling the mode of failure. To achieve the higher performance of the TRM composites as a strengthening layer, a consistency between the amount of textile fibres and the strength of mortar should be considered. High strength mortar M135 exhibited the highest flexural strength among all other types of mortar, which refers to promising performance when implemented as the matrix of TRM composites in the field of strengthening.

Based on the obtained results, to avoid wasting textile fibres by means of slippage failure mode of negligible fibre contribution, the amount of textile fibres that will be investigated in next chapters should be ranged between 8 and 16 textile basalt layers and less than 10 layers of textile carbon fibres for mortar M135. On other hands, textile carbon fibre preferably does not implement with normal strength mortar, and the textile basalt fibres should be kept under 16 layers.
a. F-SC: flexural failure with single crack
b. F-MC: flexural failure with multiple cracks
c. T-MI: Textile–mortar interface failure
d. F-S: flexural–shear failure

Figure 3.31: Failure modes of TRM composites under flexural loading

Figure 3.32: Effect of fibre properties on flexural strength of TRM composites
Table 3.19: Results of flexural test of TRM composites

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Fibre type</th>
<th>No. of layer</th>
<th>$f_r$ [Mpa]</th>
<th>c.o.v. [%]</th>
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<tr>
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<tr>
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<tr>
<td>M105-5C</td>
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<td>25.1</td>
<td>9</td>
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<tr>
<td>M105-10C</td>
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<td>1</td>
<td>T-MI</td>
</tr>
<tr>
<td>M135</td>
<td>--</td>
<td>--</td>
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<td>F-SC</td>
</tr>
<tr>
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<td>12.6</td>
<td>6</td>
<td>F-SC</td>
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<tr>
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<tr>
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<td>46.6</td>
<td>5</td>
<td>F-MC</td>
</tr>
<tr>
<td>M135-3C</td>
<td>Carbon</td>
<td>3</td>
<td>29.3</td>
<td>1</td>
<td>F-SC</td>
</tr>
<tr>
<td>M135-5C</td>
<td>Carbon</td>
<td>5</td>
<td>32.0</td>
<td>9</td>
<td>F-MC</td>
</tr>
<tr>
<td>M135-10C</td>
<td>Carbon</td>
<td>10</td>
<td>41.1</td>
<td>1</td>
<td>F-S</td>
</tr>
</tbody>
</table>

3.3.3.3 Shrinkage behaviour of TRM

Shrinkage of cementitious materials causes a reduction in volume because of water evaporation. Shrinkage amount is directly linked to the permeability of hardened cementitious materials. In the initial stage of cement hydration, restrained cementitious member crack in dry conditions due to drying shrinkage that resulted from losing water (Neville, 2011). These cracks occur because of the tensile stresses resulted from shrinkage are higher than the tensile strength of hydrated cement which are low in the early age stages.

In the present study, such these cracks may lead to significantly reducing the bond strength between substrate concrete and new cementitious layers because of differential shrinkage. To understand the effect of shrinkage on the performance of TRM, two series of specimens were investigated. The first series includes measuring the free drying shrinkage of TRM composites in addition to concrete and mortar without textile fibres to serve as a control specimen. The second series consists of determination the drying shrinkage of composite concrete-mortar (with and without textile fibres) specimens. The configuration of this series is adopted to simulate the situation of the interface between substrate RC beams and TRM layers. These series aim to track the variation in the shrinkage of TRM and concrete when they are a composite and the consequences of shrinkage on the bond between the two materials through crack initiation in the interface or the new mortar.
ASTM C490 (2000) is adopted to assess the shrinkage in the form of determination length change in hardened specimens using the measurement of length change device, see Figure 3.34. The first measuring was taken at age two days to be a benchmark reading. After that, regular measurements were recorded, and the shrinkage percentage was calculated according to the following formula (ASTM-C490 2000):

\[
L = \left( \frac{L_x - L_i}{G} \right) \times 100
\]  

(3.11)

Where: \(L\) = change in length at x age (%), \(L_x\) = comparator reading of specimen at x age minus comparator reading of reference bar at x age, \(L_i\) = initial comparator reading of specimen minus comparator reading of reference bar at that same time and \(G\) = nominal gage length, 250mm.

Series one: Free shrinkage of TRM specimens
Four mixes of mortar were adopted; two mixes of normal strength mortar M35, M70, two mixes of high strength mortar M105, and M135. For each mix, three samples with dimensions of 75×75×285 mm³ were cast without addition textile fibres to serve as control specimens. TRM specimens were reinforced of 14 layers of textile basalt or carbon fibres to investigate the effect of addition fibres on the shrinkage behaviour. Casting specimens included casting a thin layer of mortar and then placing a layer of textile fibre then casting another thin layer of mortar then placing textile layers until reaching the designed fibre amount, as shown in Figure 3.35. The specimens were de-moulded after two days and stored at a relative humidity of 55% ± 5 and temperature 20°C ± 2.
Figures 3.36-41 present the shrinkage behaviour of normal, high strength mortar and TRM composites. The results of specimens without textile fibres (Figure 3.36) indicate that high strength mortar exhibited less shrinkage than normal strength mortar. That could be attributed to the low water-cement ratio \( (w/c = 0.18) \) in comparing with 0.5 and 0.3 of mortar M35 and M70, respectively. Neville (2011) explained that higher water-cement ratio of concrete and mortar increases the shrinkage because of evaporation of water from the surface of cement past. In addition, shrinkage of cementitious materials is directly proportional to the water-cement ratio of mixes with \( w/c \) ratio between 0.2 and 0.6.

The addition of textile fibres to the normal and high strength mortar exhibited different influence depending on the strength of mortar. Inconsiderable impact of both types of textile fibres (basalt and carbon) was observed on the drying shrinkage of specimens with mortar M35 (Figure 3.38). That may be due to a low bond between the mortar and textile fibres, which limited the contribution of textile fibres in restricting the volume change of hardened mortar.

The addition of textile fibres reduced the shrinkage of mortar M70, as shown in Figure 3.39. Carbon fibres were more effective in control the free shrinkage of the specimens than basalt fibres. That was because the reinforcement ratio of carbon fibres is more than the basalt fibres. However, Figure 3.40, specimens of mortar M105 explained lower shrinkage than specimens with textile fibres. That may be because of the effectiveness of textile fibres is limited within a specific range of volumetric range and since the shrinkage of high strength mortar is relatively low. Accordingly, a negligible contribution should be expected. From another side, the implemented of textile fibres could increase the porosity of hardened mortar structure and increase the shrinkage.
Similarly, the addition of textile fibres increases the free shrinkage of M135 mortar, as shown in Figure 3.41. Specimens reinforced with carbon fibres explained lower shrinkage than specimens reinforced with basalt fibres. However, the difference between shrinkage of both types of samples was small. Overall, the influence of textile fibres in control drying shrinkage of mortar is limited by the amount of textile fibres and the properties of mortar.
Series two: Free shrinkage of composite concrete-mortar specimens

The free shrinkage of composite specimens was carried out to investigate the behaviour of volumetric changes of two different cementitious layers cast at different ages. The composite samples include casting the first part, concrete, and after 28 dry curing, the second part of specimen, mortar, was cast. This procedure was adopted to minimise the shrinkage of concrete at the time of casting the additional layer, which led to maximising the differential shrinkage between the two cementitious materials. This form of specimens was adopted to investigate the behaviour of the mortar when it being used as a repair or strengthening layer of old concrete structures.

The investigated parameters of this test included strength of mortar (35, 70, 105 and 135MPa), textile reinforcement (0 and four layers), type of textile (basalt and carbon) and connectors ratio (0 and 5%). The specimens were coded as C-x-y-Mz, where C30 and M indicate to concrete and mortar, respectively, and x, y and z refer to the connector ratio, textile fibre type and mortar strength, respectively. The details of the investigated specimens are presented in Table 3.20.
Table 3.20: Characteristics of composite shrinkage specimens

<table>
<thead>
<tr>
<th>specimen</th>
<th>Connector ratio [%]</th>
<th>Type of fibre</th>
<th>No. of textile layers</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-M35</td>
<td>0</td>
<td>--</td>
<td>0</td>
</tr>
<tr>
<td>C-5-M35</td>
<td>5</td>
<td>--</td>
<td>0</td>
</tr>
<tr>
<td>C-M70</td>
<td>0</td>
<td>--</td>
<td>0</td>
</tr>
<tr>
<td>C-5-M70</td>
<td>5</td>
<td>--</td>
<td>0</td>
</tr>
<tr>
<td>C-M105</td>
<td>0</td>
<td>--</td>
<td>0</td>
</tr>
<tr>
<td>C-5-M105</td>
<td>5</td>
<td>--</td>
<td>0</td>
</tr>
<tr>
<td>C-5-B-M105</td>
<td>5</td>
<td>Basalt</td>
<td>4</td>
</tr>
<tr>
<td>C-5-C-M105</td>
<td>5</td>
<td>Carbon</td>
<td>4</td>
</tr>
<tr>
<td>C-M135</td>
<td>0</td>
<td>--</td>
<td>0</td>
</tr>
<tr>
<td>C-5-B-M135</td>
<td>5</td>
<td>Basalt</td>
<td>4</td>
</tr>
<tr>
<td>C-5-C-M135</td>
<td>5</td>
<td>Carbon</td>
<td>4</td>
</tr>
</tbody>
</table>

The composite specimens cast in two stages. The first stage included casting normal strength concrete, and after curing time, the surface of old concrete was prepared (roughing and drilling). Then, the second stage was carried out by casting the mortar layer. The thickness of concrete and mortar for all composite samples were 60±3 mm and 15±3 mm respectively. Figure 3.42 shows the process of preparing the tested specimens. Shrinkage of the composite specimen was determined using the same procedure that used in TRM specimens as shown in Figure 3.43a. In addition, a digital microscope was used to track the initiation of cracks in the composite samples, (Figure 3.43b).

Figure 3.44 demonstrates that composite specimens of concrete - mortar M35 without cementitious connectors explained lower shrinkage than both M35 and concrete. That indicates the shrinkage of composite specimens is predominantly influenced by shrinkage of the concrete part that had lower shrinkage than concrete samples due to variation ages. However, specimens with connectors’ ratio equal to 5% exhibited higher shrinkage than concrete, especially at late ages. That was because of increasing the bond at the interface, which made the composite specimen influences by the high shrinkage of mortar M35. Not all composite specimens with mortar M35 exhibited apparent cracks.
Figure 3.42: Process of casting the composite concrete-mortar shrinkage specimens

(a) Casting concrete part  
(b) Preparing the surface  
(c) Drilling the connectors holes  
(d) Casting mortar  
(e) Composite concrete-mortar specimen

Figure 3.43: (a) measuring shrinkage and (b) tracking cracks of composite concrete-mortar specimen

(a)  
(b)
Results of composite specimens of concrete - mortar M70 (Figure 3.44) explained approximately the same shrinkage of concrete, which could be indicated to higher bond at the interface in comparison with mortar M35 counterparts. This proposed improvement in bond increases the contribution of mortar M70 of the drying shrinkage of the composites specimen. Similar to samples with M35, the inclusion of 5% connectors with ratio improve the bond, which led to increasing the shrinkage of the composite specimens because of increasing the influence of composite samples by the shrinkage of mortar M70. No evidence was observed about initiation of cracks in all samples of composite concrete and M70.
Figures 3.46 and 47 present the drying shrinkage of composite concrete and mortar M105. The results demonstrate that specimens of C-M105 explained lower shrinkage than composite samples which refers to the influencing the composite by the low volumetric change of the hardened structure of M105. This influence was higher in case of composite specimens of C-5-M105 due to inclusion 5% cementitious connector’s ratio. However, two horizontal cracks were observed in the M105 mortar layer, which can be attributed to the high restraint of the layer at the interface. Composite specimens with textile basalt and carbon fibres exhibited lower shrinkage than both concrete and M105. In addition, the results explained the efficiency of textile fibres in preventing the cracks that could be initiation in the new mortar layer.

![Shrinkage curves of composite concrete and M105 without textile fibres](image1)

**Figure 3.46:** Shrinkage curves of composite concrete and M105 without textile fibres

![Shrinkage curves of composite concrete and M105 with textile fibres](image2)

**Figure 3.47:** Shrinkage curves of composite concrete and M105 with textile fibres
Figure 3.48 demonstrates that the shrinkage of composite concrete-M135 was lower than the shrinkage of concrete due to high contribution of mortar M135 which could be due to adequate interface bond strength. Two horizontal cracks at the third of sample length were observed.

The composite specimens reinforced with textile carbon and basalt fibres control the cracks of mortar M135 and increase the efficiency of the mortar in reducing the free shrinkage. The shrinkage of these specimens was lower than the shrinkage of mortar M135.

![Image of shrinkage curves](image)

Figure 3.48: Free shrinkage curves of composite concrete and M135

By comparing the results of free shrinkage of both two series, it can be found that behaviour of mortar and TRM is more critical in case of composites specimens. In addition, using high strength mortar increase the influencing of TRM composites by the consequences of retrained condition when it being a strengthening layer of substrate concrete through the initiation of cracking. At this stage, the role of textile fibres is appeared by preventing these cracks. In addition, the shrinkage results of composite specimens indicate to the negative side of improving the bond strength between the old and new cementitious layers. Moreover, high strength mortar is more critical than normal strength mortar. In the field of strengthening, the drying shrinkage should be minimised by implemented wet curing of the new strengthening layer to reduce the consequences of differential shrinkage.
3.4 Concluding remarks
The results of the flow table test of mortar (M35 and M70), demonstrated that the amount of fine aggregate (silica sand) has a predominant influence on the flowability of mortar compared with the water-cement ratio. Besides, the properties of fresh and hardened high strength mortar (M105) are highly influenced by the amount of water and superplasticiser. Where, mixes with a water-cement ratio less than 0.16 failed to exhibit consistent mixture, while water-cement ratio higher than 0.21 showed high followability which was found a negligible effect on the compressive strength but reduced the tensile strength of mortar with respect to mixes with w/c ranged between 0.17 and 0.18. On other hands, superplasticiser exhibited higher impact on the properties of mortar and that effect increases with increasing the amount of water. The high amount of superplasticizer (40-60kg/m³) improved the workability of high strength mortar but reduced the tensile strength to about 60% of mortar compared with 30kg/m³ of superplasticizer.

Apart from silica fume, all replacement of the SCMs improved the workability of the high strength mortar depending on the type and ratio of replacement. In addition, the most senior improvement in workability observed for mixes with fly ash. However, fly ash demonstrated a relatively low improvement in the mechanical properties of HSM compared with other SCMs. Although both types of silica fume reduced the workability of mortar, un-densified form explained better properties of fresh mortar. Regarding the mechanical properties, both types of silica fume exhibited approximately similar behaviour up to 10% replacement. Higher replacement indicated to the superiority of un-densified silica fume in increasing the strength. Binary replacement of 15% un-densified silica fume and 15% GGBS presented the highest mechanical properties (135.2Mpa and 6.7Mpa of compressive and tensile strength, respectively). Moreover, the effect of enhancing the strength of mortar was evident from the low intensity of the Ca(OH)₂ that determined from the X-Ray diffraction analysis compared with mixes with lower strength.

Similarly, the addition of SCMs improved the resistance of mortar to the chloride penetration depending on the type and amount of SCMs replacement ratio. The best performance was observed for binary and ternary replacements. The enhancement of single replacement of SCMs ranged between 2% and 83%. The increase of binary replacement ranged between 22% and 95% with respect to control mortar. In ternary replacement, the highest enhancement (91%) was observed of 30% replacement ratio of UDSF, FA and GGBS. High strength mortar with 15% un-densified silica fume and 15% GGBS presented the lowest penetration of chloride with 0.09 coefficient of migration (Dnssm x10⁻¹² m²/s).
The implementation of high strength mortar significantly increases the effectiveness of textile fibres in resisting the direct tensile stresses. For instance, the effectiveness factor of 16 textile basalt fibres raised from 0.34 of mortar M35 to 0.79 of mortar M135. That means improving the efficiency of textile fibres to about 132%. Similar enhancement (about 100% improvement in the effectiveness factor) was observed for textile carbon fibres. Moreover, all types of mortar exhibited higher effectiveness of textile fibres than carbon fibres. TRM with normal strength mortar (M35 and M70) with 5 and 10 layers of carbon fibres exhibited a splitting failure mode, which referred to losing the bond between the textile fibres, and mortar (slipping) as a result of exceeding the stresses in mortar its tensile strength. However, no parental slippage of carbon fibres was observed for mortar M105, and M135 and multiple cracks were found for 8 and 16 layers of textile basalt fibres.

Under flexural loading, similar enhancement of the higher mortar strength was observed. The flexural strength of TRM composites reinforced with 16 layers of M135 was about twice the M35 counterpart. In addition, increasing the mortar strength from 35 to 135MPa altered the failure mode from horizontal cracks at the level of textile to a desired flexural with multiple cracks. Similarly, the results of direct tensile tests, TRM composites with textile basalt fibres presented higher flexural tensile strength than carbon fibres despite carbon fibres have approximately the same axial tensile strength (Af×fA). Besides, TRM composites with 10 layers of textile carbon fibres explained cracks at the level of textile of fibres, which refers to losing the effective bond with mortar. By comparing the tensile strength of TRM composite under direct tensile and flexural loading, it was found that flexural strength was much higher than tensile strength. For instance, the flexural strength of TRM with M135 reinforced with 16 textile basalt layers was about 3.14 times the tensile strength of the same TRM properties. That could lead to conclude the using of direct tensile strength in predicting the behaviour of TRM contribution as a strengthening layer will produce under estimated strength.

Based on the effectiveness and failure mode of TRM composites under direct and flexural, textile carbon fibres will be only investigated for high strength mortar (M135) with five layers in the strengthening RC beams. For textile basalt fibres, 8 eight layers will be studied for mortar M35 and M70 while 8 and 16 layers will be examined for mortar M135. Free shrinkage of mortar specimens was significantly reduced with using high strength mortar (the shrinkage of M135 was about 0.425 the shrinkage of M35). However, a limited impact of textile fibres was observed in the free shrinkage behaviour of TRM composites. For high strength mortar (M105 and M135), both types of textile fibres increased the shrinkage of TRM composite. On other side, textile fibres were found
effective in control the cracking and reduce the shrinkage of the overlay TRM. Moreover, in the presence of effective interface bond strength, between concrete and high strength TRM layer, cracks initiated in the TRM, which refers to higher bond with the substrate concrete, compared with normal strength mortar. That indicates the need of wet curing of the strengthening layer to reduce the shrinkage cracks.
Chapter Four

Experimental investigation of strengthening RC beams with TRM

4.1 Introduction

This chapter details the experimental investigation of strengthening RC beams using TRM composites. The investigation is divided into three main parts; strengthened RC beams, repair corroded RC beams and durability of strengthened RC beams by means of quantifying the reduction in strengthening performance due to corrosion.

The first part included examining the behaviour of RC beams strengthened with TRM under flexural loading. This part is devoted to exploring the effect of the following parameters: the strength of mortar, cementitious connectors’ ratio, amounts and types of textile fibres. Besides, the impact of mortar strength on the bond with substrate concrete was investigated through a pull-off test and slant shear compression test.

The second part consists of subjecting RC beams to corrosion conditions and then repaired the corroded beams using TRM layers. The purpose of this investigation is to address the effectiveness of TRM layer in presence of corrosion with respect to uncorroded beams by means of restoring the strength lost due to corrosion. The effect of cementitious connectors and degree of corrosion and type of textile fibres were investigated within this phase. All beams were tested under flexural loading. In addition, the corrosion of small-scale RC beams was initially investigated to determine the required impressed current to create the desired corrosion.

The durability of the strengthened RC beams under corrosion conditions were examined in the third part by means of subjecting the strengthened RC beams to corrosion. The investigation included subjecting the strengthened RC beams to corrosion conditions then tested under flexural stresses. The results were compared with respect to strengthened uncorroded strengthened beams to evaluate the reduction in performance of the strengthened corroded members. Finally, the experimental results of the three parts were presented and discussed.
4.2 Bond between substrate concrete and additional layer of mortar

According to European standards EN 1504-10 (2017), the bond strength between substrate concrete and new cementitious layers is defined as the adhesion between the new and substrate layer. The effectiveness of strengthening of RC structures is strongly influenced by the quality and behaviour of the interface bond strength. This section is devoted to investigating the interface bond strength between substrate concrete and mortar casting at different ages. Based on the conducted literature review in Chapter 2, different methods have been used to quantify the interface bond strength, and it is varied depending on the type of the applied stresses. In addition, the reported bond strength may significantly overestimate the true strength depending on the test method used.

Despite many researchers and international standards (ASTM and BS-EN) adopted slant shear compression test method to examine the quality of bond strength, but the validation of this method in the field of strengthening RC structures has two limitations. Firstly, the obtained bond coefficients characterise the bond strength when the interface under combined shear and compression, which means it may not address other possible cases such as pure shear, pure tension or combined shear and tension. Secondly, the bond strength is determined based on the assumption of interface failure mode, which is not always happened especially with high strength repair materials (Julio et al. (2006) and Tayeh et al. (2012)). Pull – off test method is adopted by EN 1542 (1999) and many researchers to identify the adhesion bond strength. Other researchers adopted a shear test to determine the adhesion interface strength.

Accordingly, in the absence of this study, two methods were adopted to assess the bond strength; bond under direct tension and slant shear-compression tests. The main objective of these tests is to characterise the bond when the interface subjected to the different state of stresses, to quantify the influence of mortar strength on bond and to examine the correlation of the cohesion interface bond strength under a different state of stresses.

4.2.1 Bond under direct tensile test

This test was carried out to determine the adhesion interface strength (adhesion coefficient) when the interface is under tensile stresses. Specimens of this test consisted two parts; concrete as a substrate and mortar with the same dimension as a repair material. The substrate part was designed to present 30MPa compressive strength at 28 days. Four strengths of mortar (35MPa, 70MPa, 105MPa and 135MPa) were adopted as repair material to study the behaviour of interface bond strength within different mechanical
properties of repair materials. The specimens’ preparations include casting the substrate concrete with dimension 100×100×150 mm³. Steel rebars were embedded at the centre of the sample to be used later in transferring the tensile stresses from the tensile machine to the sample, as shown in Figure 4.1. After 28 days wet curing period, the surface of the substrate concrete was roughened using air needle hammer to remove 2-3mm from the surface (exposing the aggregate). Then, following the same procedure of casting the substrate part, the second part of the specimen (mortar) was casted.

After another 28 days of wet curing, the specimens were tested under direct tensile test. Three samples of each mortar strength were cast, and the average was adopted to indicate the bond strength. Instron universal machine with displacement-based loading control of 0.002 mm/min was used. The bond strength is determined based on the ultimate load that can be applied to the composite specimen and the area of the interface, according to the following equation:

\[
\frac{f_{bt}}{A_c} = \frac{P}{A_c}
\]

Where: \(f_{bt}\) is the bond strength (MPa), \(P\) is the direct tensile applied load (kN) and \(A_c\) is the contact area (interface area) (m²).

a. Casting substrate concrete  c. Casting repair mortar  d. Test setup

b. Surface preparation

Figure 4.1: Procedures of bond test under direct tension
4.2.2 Slant shear-compression bond test

Slant shear test was carried out mainly to quantify the bond strength in shear, because of it has been adopted by international standards and many previous studies reported its effectiveness in determining the bond strength between two layers of a concrete cast at different ages. In addition, the results from this test can be used to determine the cohesion (c) and friction (\( \mu \)) coefficients that required in the simulation of interface bond under shear and compression stresses. Specimens with dimensions 100×100×300 mm\(^3\) prism with interface line at 30 to the vertical were adopted. Three grade of mortar strength were adopted; M35, M70 and M105. Three specimens were tested under compression as in a standard compression test for each mortar strength, and the average was taken.

In this test, the sample is subjected to compressive stresses at its ends. These stresses were analysed at the interface into compressive stresses normal to the interface, and shear stresses parallel to the interface. Figure 4.2 demonstrates the procedures and test setup of slant shear test. Specimen preparation included casting substrate concrete, and after 28 days of wet curing, the surface was roughened by removing 2-3 mm from the surface thickness. The mortar was cast to complete the prism and the prisms cured in wet condition for 28 days before testing.

A universal machine with a displacement-based loading rate of 0.002 mm/min was used for application compressive forces at the ends of the specimen. The ultimate capacity of the sample was analysed at the interface to become a vertical and parallel force to the interface. The obtained bond strength at the interface can be divided into shear bond (parallel to the interface) and compression bond (normal to the interface). The shear and normal strength can be defined by analysis the stresses at the inclined surface and the area of the interface, according to the following formulas (Austin Simon, 1999):

\[
\sigma_o = \frac{p}{A} \quad \text{(4.2)}
\]

\[
\sigma_n = \sigma_o \sin^2 \alpha \quad \text{(4.3)}
\]

\[
\tau_n = \frac{1}{2} \sigma_o \sin 2\alpha \quad \text{(4.4)}
\]

Where: \( \sigma_o \) is normal axial stress, \( A \) is the cross-section area of specimen, \( \sigma_n \) and \( \tau_n \) are normal and shear stresses at the interface, respectively, and \( \alpha \) is the inclination angle of the interface.
The relationship between the shear and normal stresses under compression can be determined using Coulomb theory (Austin Simon 1999) according to following formula:

$$\tau_n = C + \mu \sigma_n$$  \hspace{1cm} (4.5)

Where: $C$ is the cohesion coefficient (MPa) and $\mu$ is the friction coefficient.

**Figure 4.2: Procedures of slat shear compression test**

**4.3 Strengthening of RC beams using TRM**

Based on the literature review in chapter two, the behaviour of RC beams strengthened with TRM is strongly influenced by the properties of the interface and strengthening layer. The experimental investigation includes four parameters; cementitious connectors ratio, the strength of TRM mortar, type and amount of textile fibres. The effect of cementitious connectors was studied by considering three ratios; 0%, 5% and 7.5%. These values have been selected based on the experimental observation of small-scale specimens by means of providing the effective bond without damage the concrete cover of the substrate. Three mortar strengths were adopted; M35, M70 and M135 to study the effect of mortar properties on the interface bond strength. Five layers of textile carbon fibres, 8 and 16 layers of textile basalt fibres were used as a reinforcement of TRM layers. Two beams were adopted to investigate the behaviour of each case study.
4.3.1 Specimens design

Twenty RC beams with a total length of 1400 mm and 125×175 mm cross-section dimensions were carried out. The criterion of selection these dimensions as the previous studies with taking in account the available resources in the laboratory. Concrete with a nominal compressive strength of 30MPa at 28 days that designed in Section 3.4 was used for casting all beams. The beams were designed according to Eurocode2 (BS-EN 1992-1-1:2004) to have extra strength in shear to ensure flexural failure even after strengthening. The design details are presented in Appendix B. Figure 4.3 shows that all beams were reinforced with two longitudinal 10 mm diameter ribbed steel rebars and 8mm diameter ribbed for shear reinforcement.

Concrete cover of steel reinforcement is required to provide a sufficient transmission of bond forces and protection of steel reinforcement from fire and corrosion effects. A small concrete cover may lead to splitting tensile failure at the bottom side of the beams. However, high values of the thickness of concrete cover lead to increase the horizontal shear stresses at the level of steel reinforcement, which may lead to separation of concrete cover especially in presence of corrosion. For structural purposes, the minimum value of the thickness of the concrete cover is 10mm and 20mm according to Euro Code2 [BS-EN 1992-1-1:2004] and ACI-318 (2011), respectively. In this study, a clear concrete cover to the flexural reinforcement rebars was set as 20 mm by using plastic chairs. This value was adopted to speed of achieving the required data for of corrosion, ensure the consistency with the recommendation of international codes and previous studies that investigated RC beams strengthened with TRM.

A direct tensile test (Figure 4.3) of three specimens of each type of rebars was carried out according to BS-EN 10002:2001 using universal Instron machine with a 0.1mm/sec displacement-based control loading to determine the mechanical tensile properties. The results of the tests are presented in Table 4.1. Typical stress-strain curves of the steel rebars of 10 mm and 8mm diameter rebars are shown in Figure 4.4. The stress-strain curve of rebar 8mm did not explain specific yield strength, and hence, the yield strength was determined based on 0.2% proof stress.

<table>
<thead>
<tr>
<th>Bar diameter (mm)</th>
<th>Modulus of elasticity (GPa)</th>
<th>Yield stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>190</td>
<td>480</td>
<td>560</td>
</tr>
<tr>
<td>10</td>
<td>192</td>
<td>590</td>
<td>684</td>
</tr>
</tbody>
</table>

Table 4.1: Tensile mechanical properties of steel rebars
Table 4.2 presents the details of the investigated RC beams. Two beams were left without strengthening to serve as control beams. Ten beams were strengthened with a different type of mortar and amount of basalt and carbon fibres without application the cementitious connectors. However, eight beams were strengthened with high strength mortar TRM and different type of fibres and cementitious connectors ratios. To avoid damage of the steel reinforcement of the substrate beam during drilling the holes, the maximum depth of connector should be limited by the clear distance between the steel reinforcement and outer surface of the concrete. In addition, to ensure ease and sufficient penetration of mortar inside the holes and to avoid slipping of connectors during loading, the recommended depth of connectors is equal to the diameter.

Figure 4.5 shows the configuration of the strengthening layer and cementitious connectors of the investigated beams. All beams were tested under four-point flexural loading.
Table 4.2: Details of the investigated RC beams

<table>
<thead>
<tr>
<th>Specimen designation</th>
<th>Mortar of TRM</th>
<th>Type of fibre</th>
<th>No. of layers</th>
<th>Connector ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CON-1</td>
<td>--</td>
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<td>--</td>
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</tr>
<tr>
<td>CON-2</td>
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<td>--</td>
<td>--</td>
</tr>
<tr>
<td>8B0-M35-1</td>
<td>M35</td>
<td>Basalt</td>
<td>8</td>
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</tr>
<tr>
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<td>Basalt</td>
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<tr>
<td>8B0-M70-1</td>
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<td>M70</td>
<td>Basalt</td>
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<td>M135</td>
<td>Basalt</td>
<td>8</td>
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</tr>
<tr>
<td>8B0-M135-2</td>
<td>M135</td>
<td>Basalt</td>
<td>8</td>
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</tr>
<tr>
<td>8B5-M135-1</td>
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<td>Basalt</td>
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</tr>
<tr>
<td>8B5-M135-2</td>
<td>M135</td>
<td>Basalt</td>
<td>8</td>
<td>5</td>
</tr>
<tr>
<td>16B0-M135-1</td>
<td>M135</td>
<td>Basalt</td>
<td>16</td>
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<td>16B0-M135-2</td>
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<td>16B5-M135-1</td>
<td>M135</td>
<td>Basalt</td>
<td>16</td>
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</tr>
<tr>
<td>16B5-M135-2</td>
<td>M135</td>
<td>Basalt</td>
<td>16</td>
<td>5</td>
</tr>
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<td>Basalt</td>
<td>16</td>
<td>7.5</td>
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<tr>
<td>16B7.5-M135-2</td>
<td>M135</td>
<td>Basalt</td>
<td>16</td>
<td>7.5</td>
</tr>
<tr>
<td>5CB0-M135-1</td>
<td>M135</td>
<td>Carbon</td>
<td>5</td>
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<tr>
<td>5CB0-M135-2</td>
<td>M135</td>
<td>Carbon</td>
<td>5</td>
<td>0</td>
</tr>
<tr>
<td>5CB5-M135-1</td>
<td>M135</td>
<td>Carbon</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>5CB5-M135-2</td>
<td>M135</td>
<td>Carbon</td>
<td>5</td>
<td>5</td>
</tr>
</tbody>
</table>
Figure 4.6: Configuration of the TRM strengthening layer of the investigated RC beams
4.3.2 Casting and curing of specimens

Figure 4.7 shows the casting process of the investigated beams. After preparing the designed reinforcement, the beams were placed inside wooden moulds, and concrete was cast inside the moulds with using a handle vibrator. For each beam, three cubes with dimensions of $100 \times 100 \times 100$ mm$^3$ and three cylinders with a diameter of 100 mm and length of 200 mm were cast to assess the tensile and compressive strength of concrete. The beams were de-moulded after two days and left for 28 days dry curing.

![Figure 4.7: Casting the RC beams](image)

After 28 days dry curing period, the surface of the bottom side of beams was roughened using needle scaler hammer to remove 2-3 mm from the surface to increase the bond strength between the substrate concrete and the strengthening layer. For beams with cementitious connectors, hammer drill machine was used to make holes inside substrate concrete beams with holes diameter and depth equal to 10mm. The connector ratio was expressed in the form of the ratio between the contact areas (interface area) to the total area of the connectors. Figure 4.8 shows the details of the preparing process of the substrate surface of the RC beams.

After preparing the surface of the RC beams, beams were washed with high-pressure water to remove all dust inside the holes to ensure the clean surface before cast the TRM strengthening layer. Then, the textile fibres fixed on one edge of the beams using steel wires with separation the textile layers with plastic pieces (about 2 mm thickness) to ensure the designed location of the textile fibres. After that, a thin layer to the marked level was cast and then the textile layer was put using a hand-lay-up method and this procedure was repeated until reach the designed amount of fibres, as shown in Figure 4.9. The wooden moulds were used again to ensure achieve the same thickness and width of the strengthening layer of all strengthened beams. The thickness of the strengthening layer was kept equal to 25 mm for all strengthening RC beams. Three cubes with a dimension of
50×50×50mm³ and dog bones mortar specimens were cast for each TRM layer to quantify the mechanical properties of the mortar of TRM.

For all beams, after 24 hours from casting the strengthening layer, the moulds were released, and tissues covered the strengthening layer, then water was applied to the tissues and polyethylene sheets covered the strengthening layer to provide a wet curing condition for 27 days. The water was added regularly to ensure wet curing conditions for the specimens. This procedure was used based on the results of the free shrinkage, which explained the sensitivity of the strengthening layer to the shrinkage with dry conditions. Figure 4.10 demonstrates the wet curing procedure of the TRM strengthening layer.

Figure 4.8: Preparing the surface of the RC beams
Figure 4.9: Process of casting the strengthening layer of the RC beams

Figure 4.10: Wet curing of the TRM strengthening layer of the RC beams
4.3.3 Testing procedure of specimens

All beams were tested as simply supported beams using four-point loading at 28 days age, as shown in Figure 4.11. A steel distribution I beam was used to generate the two concentrated loads. A Zwick machine with maximum load capacity equal to 200 KN was used to apply the load. The load was applied in the form of displacement per time (0.25 mm/sec) to track the crack initiation. Two LVDT were used to measure the mid-span deflection of the tested beam. Two LVDT were used to measure the deformation of concrete at the middle top and bottom of the control beams.

![Test set up of control RC beams](image)

Figure 4.11: Test set up of control RC beams

For strengthened RC beams, four LVDT on each side were used to measure the relative movement (slip at the interface) between the substrate concrete and the strengthening layer. Due to the limitation of the number of LVDT that can directly be connected to the Zwick machine, data logger was used to tracking the deformation of three LVDTs. Figure 4.12 shows the instrumentation of the strengthened RC beams.
4.4 Repair of Corroded RC beams using TRM

Figure 4.13 demonstrate the process of the repair of corroded RC beams using TRM. The investigated parameters included the type of textile fibres (carbon and basalt) and the cementitious connector’s ratio (0 and 2.5%). The corrosion of longitudinal steel reinforcement was designed to lose about 10% of the original mass of steel rebars. To achieve that, besides the theoretical calculations that based on Faraday’s law, small-scale samples with the same properties of steel and concrete were investigated under corrosion to quantify the amount of impressed current required to create 10% corrosion level of longitudinal steel reinforcement.
4.4.1 Specimens design

Eight RC beams with the same dimensions, reinforcement and concrete properties of beams in Section 4.3.1 were cast. Tables 4.3 presents the details of investigated beams which includes two beams of each case study. All beams will be subjected to impressed current to create 10% corrosion level. Two beams were left without repair to serve as a control corroded beams. Four beams repaired with eight layers of basalt fibres, two with application 2.5% ratio of connectors and two without connectors. The last two beams repaired with five layers of carbon fibres with application connectors’ ratio of 2.5%. This connectors ratio was chosen to avoid possible destroying of substrate concrete cover due to corrosion cracks. Figure 4.14 shows the reinforcement details and the configuration of the connector of the investigated beams.

Table 4.3: Details of the investigated repaired corroded RC beams

<table>
<thead>
<tr>
<th>Specimen designation</th>
<th>Mortar of TRM</th>
<th>Type of fibre</th>
<th>No. of layers</th>
<th>Connector ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>COR-1</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>COR-2</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>C8B0-M135-1</td>
<td>M135</td>
<td>Basalt</td>
<td>8</td>
<td>0</td>
</tr>
<tr>
<td>C8B0-M135-2</td>
<td>M135</td>
<td>Basalt</td>
<td>8</td>
<td>0</td>
</tr>
<tr>
<td>C8B2.5-M135-1</td>
<td>M135</td>
<td>Basalt</td>
<td>8</td>
<td>2.5</td>
</tr>
<tr>
<td>C8B2.5-M135-2</td>
<td>M135</td>
<td>Basalt</td>
<td>8</td>
<td>2.5</td>
</tr>
<tr>
<td>C5CB2.5-M135-1</td>
<td>M135</td>
<td>Carbon</td>
<td>5</td>
<td>2.5</td>
</tr>
<tr>
<td>C5CB2.5-M135-2</td>
<td>M135</td>
<td>Carbon</td>
<td>5</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Figure 4.14: Loading setup and connectors distribution of the repaired corroded RC beams
4.4.2 Casting and curing of specimens

The same procedure of casting RC beams in Section 4.3.2 was used. The corrosion was designed to be induced on the longitudinal reinforcement. Therefore, longitudinal reinforcement was insulated using insulation tape at the contact points with the shear reinforcement, as shown in Figure 4.15. Electric wires were fixed on the longitudinal steel rebars to apply the current. In addition, the longitudinal steel reinforcement was electrically connected to ensure applying the current on both steel rebars. Similar to casting beams in Section 4.3.2, three cubes with dimension $100\times100\times100\text{mm}^3$ and three cylinders with diameter and height equal to 100mm and 200mm, respectively, were cast to assess the mechanical properties of the concrete. The same procedure of de-moulding and curing of beams in Section 4.3.2 were applied.

Figure 4.15: Details of reinforcement insulation and casting process of corroded RC beams
4.4.2 Inducing of corrosion

4.4.2.1 Inducing of corrosion of Small scale beams

Many researchers, as stated in the literature review, adopted an impressed current method to accelerate the corrosion of reinforcing steel. The required impressed current that causes a certain level of mass loss in steel rebars is variable. The variability is controlled by different parameters such as the properties of the concrete components, strength of concrete, type and size of steel rebars and the cover of steel rebars. Therefore, in this study, six concrete beams with dimensions 100×100×500 mm³ reinforced with single 10mm diameter steel rebars were cast and cured in dry condition for 28 days, as shown in Figure 4.16. The reinforcement rebars were weighted before casting to be used in calculations the mass loss after corrosion.

![Figure 4.16: Casting of the small-scale beams for corrosion inducing](image)

The same concrete, steel rebar and concrete cover that used in casting RC beams for strengthening investigation were used in casting the small-scale specimens. After 28 days curing period, the specimens were placed in plastic container with a saline solution of a 5% concentration of sodium chloride (NaCl) up to 30 mm depth of specimen. Then, external current was applied on the specimens using a power supply, as shown in Figure 4.17. The applied current was determined based on Faraday’s law by taking in account reaching a corrosion level equal to about 10% mass loss of steel rebars, the details of calculation are presented in Appendix B. Based the theoretical calculations, the specimens were subjected to external current equal to 0.3 mA/cm² between the steel reinforcement rebar and counter electrode.

The specimens were monitored regularly through the appearance of cracking in concrete due to initiation of corrosion. The first cracks were observed after 7 days of applying the current. After that, the specimens were got out from the salted water container and tested under flexural loading at 14, 21, 24 and 30 days.
The results of the corroded small-scale beams explained that the cracks in corroded specimens were increased with time as the corrosion of steel rebars increases and the crack was along the steel rebar, as shown in Figure 4.18.

The beams were tested under four-point flexural loading, and all beams exhibited shear failure due to the absence of shear reinforcement. The steel rebar was extracted and cleaned according to the ASTM G1-90: 1999 standards using hydrochloric acid, as shown in Figure 4.19. The extracted corroded rebars exhibited uniform corrosion in addition to localised pits. The degree of corrosion was calculated based on the following formula:

\[ Dc = \frac{w_1 - w_2}{w_1} \times 100 \]  

(4.6)

Where: \( w_1 \) is the weight of steel rebar before corrosion and \( w_2 \) is the weight of steel rebars after corrosion. The obtained results of corroded rebars showed that specimens explain cracks at a degree of corrosion equal to about 1.6%. That was due to concentration the corrosion on localised parts of the steel rebar, which led to produce stresses in concrete higher than the tensile stresses and crack the concrete. The crack length and width were increased with time as the corrosion ratio increases. However, the corrosion level was highly increased after 21 days of application the current due to increasing the inducing the salted water inside the cracked concrete. The corrosion level was doubled within 6 days from 4.56% at age 24 days to 8.83% at age 30 days. Table 4.4 presents the obtained corrosion level of the corroded beams. The relationship between the corrosion level and the duration of application the current is shown in Figure 4.20.
Figure 4.18: Crack patterns of the corroded small-scale beams

a. splitting the specimen

b. extracting the steel rebars

c. cleaning the steel rebars
d. weighting the cleaned corroded steel rebar

Figure 4.19: Process of determination of corrosion level of steel rebars

Table 4.4: Results of percentage of corrosion of steel rebars

<table>
<thead>
<tr>
<th>Beam</th>
<th>W₁ (gm)</th>
<th>W₂ (gm)</th>
<th>Degree of corrosion (D₀) %</th>
<th>Age (day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>255.2</td>
<td>251.2</td>
<td>1.57</td>
<td>7</td>
</tr>
<tr>
<td>B2</td>
<td>249.1</td>
<td>240.5</td>
<td>3.45</td>
<td>14</td>
</tr>
<tr>
<td>B3</td>
<td>253.8</td>
<td>243.5</td>
<td>4.06</td>
<td>21</td>
</tr>
<tr>
<td>B4</td>
<td>252.1</td>
<td>240.6</td>
<td>4.56</td>
<td>24</td>
</tr>
<tr>
<td>B5</td>
<td>253.7</td>
<td>231.3</td>
<td>8.83</td>
<td>30</td>
</tr>
</tbody>
</table>
Based on the obtained results, it can be stated that the application of current equal to 0.3 mA/cm² for 30 days can produce a corrosion level equal to about 9%, which is near to the calculated value based on the Faraday's law. Therefore, the applied current (0.3 mA/cm²) will be applied to large-scale beams for 30 days.

### 4.4.2.2 Inducing corrosion of RC beams

The samples were submerged in saline water with 5% concentration of NaCl up to 30 mm depth to provide a wet salted condition for the longitudinal reinforcement. Power suppliers with a maximum current capacity equal to 2.0 Am were used to induce the desired current (0.3 mA/cm²). Air pumps were used to produce air in the salted water to ensure sufficient dissolved oxygen that required for corrosion process. Figure 4.21 shows the setup of the adopted induced current of the investigated beams. After 30 days, the corroded samples were extracted and cleaned with high-pressure water to remove the remains of corrosion rust on the surface of the concrete.
4.4.3 Repaired corroded RC beams

The surface of the corroded RC beams was prepared in the same method that used in preparing beams for strengthening. Cementitious connector ratio of 2.5% was implemented on four beams before casting the strengthening layer. Figure 4.22 shows the process of casting the TRM strengthening layer of the repaired corroded RC beams. The procedure of casting the TRM strengthening layer was the same to the procedure that used for strengthening beams in Section 4.3.1. For each repair layer, three cubes of 50mm and three dog bones of mortar were cast to assess the mechanical properties of TRM mortar.

The beams were de-moulded after two days, and wet curing of the strengthening layer was implemented for 28 days. After that, the flexural test under four points of loading was carried out to investigate the performance of the repaired beams like Section 4.3.3.
4.5 Effect of corrosion on strengthened RC beams with TRM

To investigate the durability and longevity of the TRM strengthening technique under corrosion conditions, RC beams were strengthened with TRM strengthening layer and wet cured for 28 days. The beams were then subjected to impressed current to initiate about 10% corrosion level, as shown in Figure 4.23. The investigated parameters included type of textile fibres (carbon and basalt) and the cementitious connector’s ratio (0 and 5%). Table 4.5 presents the details of the investigated beams. These included four beams strengthened with eight layers of basalt fibres using mortar M135, two of them without cementitious connectors and the others with connector’s ratio of 5%. In addition, two beams strengthened with five layers of carbon fibres were investigating with application cementitious connectors’ ratio of 5%.

![Figure 4.23: Process of application corrosion on strengthened RC beams](image)

The results of these beams will be compared with beams that have the same strengthening properties without inducing corrosion to investigate the effect of corrosion on the strength reduction of the strengthened beams.

<table>
<thead>
<tr>
<th>Specimen designation</th>
<th>Mortar of TRM</th>
<th>Type of fibre</th>
<th>No. of layers</th>
<th>Connector ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S8B0-M135-1</td>
<td>M135</td>
<td>Basalt</td>
<td>8</td>
<td>0</td>
</tr>
<tr>
<td>S8B0-M135-2</td>
<td>M135</td>
<td>Basalt</td>
<td>8</td>
<td>0</td>
</tr>
<tr>
<td>S8B5-M135-1</td>
<td>M135</td>
<td>Basalt</td>
<td>8</td>
<td>5</td>
</tr>
<tr>
<td>S8B5-M135-2</td>
<td>M135</td>
<td>Basalt</td>
<td>8</td>
<td>5</td>
</tr>
<tr>
<td>S5CB5-M135-1</td>
<td>M135</td>
<td>Carbon</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>S5CB5-M135-2</td>
<td>M135</td>
<td>Carbon</td>
<td>5</td>
<td>5</td>
</tr>
</tbody>
</table>
All beams have the same dimensions, concrete and reinforcement properties of the RC beams in Section 4.3.1. In addition, the same procedure of casting, curing and strengthening that used in Section 4.3.2 was implemented to ensure the same properties of the strengthened RC beams.

The edges of the beams were coated with bituminous paint to minimize the penetration of saline water from the edges of the beams and to ensure that the penetration of salted water will be through the strengthening layer, as shown in Figure 4.24.

The impressed current intensity of 0.3 mA/cm² was applied for 30 days. The same setup of repaired beams in the application corrosion was applied. Then the beams were tested under four-point flexural loading in the same procedure of Section 4.3.3.

![Figure 4.24: Coating the edges of the strengthened beams with bituminous material](image)

To address the effect of corrosion on the strength of the RC beams, all corroded RC beams were demolished, and the reinforcement was extracted. The amount of corrosion that affects the flexural strength of the RC beams was determined only for the effective length (length of reinforcement between supports). The same procedures that used in cleaning the reinforcement of small-scale specimens was used to remove the rust from the corroded steel rebars. Due to the difficulties of cleaning and weighing the whole effective length of the corroded rebars, the rebars were cut into three parts and then cleaned.
4.6 Results and discussion

4.6.1 Bond test results

4.6.1.1 Bond under direct tensile test

Table 4.6 shows the average results of three specimens of concrete and mortar mechanical properties and the adhesive bond under direct tensile stresses. As shown in Figure 4.25, all samples explained adhesive failure mode (failure at the interface). However, high strength mortar (M135) overlays showed that some parts of the substrate concrete still adhered on the overlay part. That may be due to increasing the adhesion between the mortar and the substrate concrete with taking in account the expected weakness of the layer near to the surface due to roughening producers. The results of adhesive bond test ranged between 0.74MPa of M35 and 2.08 MPa of M135. This high improvement in adhesion (about 2.81 of M35) could be attributed to the higher chemical adhesion of M135 as a result of the higher amount of C-S-H gels that resulted from cement hydration and the pozzolanic reaction of SCMs. It is worth to mention, that the tensile strength of M135 equal to about 2.66 of M35, which is very close to the adhesion enhancement. However, the ratio between the compressive strength of mortar M135 and M35 is about 3.96, which could refer to closer relationship between the tensile strength and the adhesive bond than compressive strength.

According to EN 1504-3 (2005), the threshold of the adhesive bond is 1.5MPa and 2.0 MPa for non-structural and structural repair, respectively. These values are considered relatively higher than the obtained adhesive bond (apart from mortar M135) which could be due to the differences in method of determining the adhesive bond strength. EN 1504-3 (2005) adopted pull-off test in quantifying the adhesive bond and this test method, as reported by many researchers, exhibited adhesive bond strength ranged between 2.54MPa and 2.65MPa for rough surface and overlay compressive strength of 45MPa (Courard et al. 2014 and Julio et al. 2004). However, Courard et al. (2014) found the pull-off adhesive bond between smooth concrete and overlay was 2.04MPa, which is considered relatively high for smooth surface. While, for similar properties of concrete and overlay, Julio et al. (2004) observed an inconsiderable pull-off bond strength of smooth surface. It is worth to mention that Al-Salloum et at. (2012) found the pull-off bond strength between concrete and cementitious mortar with a compressive strength of 23.9MPa and 56.4MPa is equal to 0.39 MPa and 0.7MPa, respectively. That indicates the proportional relationship between the adhesive bond and mechanical properties of mortar.
Table 4.6: Properties of mortar and concrete and results of direct tensile bond test

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete</th>
<th>Mortar</th>
<th>Bond</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_{cm}$ (MPa)</td>
<td>$f_{cim}$ (MPa)</td>
<td>$f_{cm}$ (MPa)</td>
</tr>
<tr>
<td>C37-M35</td>
<td>36.2</td>
<td>2.71</td>
<td>34.1</td>
</tr>
<tr>
<td>C37-M70</td>
<td>36.2</td>
<td>2.71</td>
<td>69</td>
</tr>
<tr>
<td>C37-M105</td>
<td>36.2</td>
<td>2.71</td>
<td>105.1</td>
</tr>
<tr>
<td>C37-M135</td>
<td>36.2</td>
<td>2.71</td>
<td>135.2</td>
</tr>
</tbody>
</table>

Figure 4.25: Failure modes of normal and high strength tensile bond specimens

Figure 4.26 demonstrates the relationship between the adhesive bond and the tensile strength of the overlay. The results of the tensile bond strength were proportioned linearly with the tensile mortar strength. Based on the relationship between the tensile strength of mortar and strength and failure mode of tensile bond strength, the following formula is proposed to determine the adhesion bond strength between old concrete and new mortar for rough surface (exposing aggregate).

$$f_{bt} = 0.28 \times f_{tm}^{1.05}$$  \hspace{1cm} (4.15)

Where: $f_{bt}$ is the tensile bond strength (MPa), $f_{tm}$ and $f_{tc}$ are the tensile strength of mortar and concrete in MPa, respectively.

Czarnecki (2009) reported that the adhesion bond strength can be determined using the following equation depending on the properties of overlay.

$$f_A = k f_{tk}$$  \hspace{1cm} (4.16)
Where: $k$ is equal to 1.2, 1.25 and 1.3 for modified cement mortar, polymer cement concrete and polymer concrete, respectively and $f_{tk}$ is the tensile strength of overlay. Based on this equation, the adhesive bond of M35 would be about 3.04MPa. For M135, the adhesive bond would be about 8.04MPa, which is impossible because of cementitious materials cannot explain adhesion bond greater than its cohesion bond (tensile strength). That could be referred to a realistic application of the proposed equation in this study in comparing with Czarnecki’s equation.

Overall, implementing high strength mortar significantly improves the adhesive bond strength and the bond is proportional to the tensile strength of the overlay. Higher mortar strength exhibited higher adhesion bond strength.

![Figure 4.26: Relationship between tensile strength of mortar and tensile bond strength](image)

### 4.6.1.2 Results of combined shear and compression bond test

The mean values of three results of bond strength in shear are presented in Table 4.7. All specimens with overlay mortar strength M35 explained adhesive failure at the interface (Figure 4.27a). However, samples with overlay mortar M70 and M105 exhibited monolithic failure by means of contentious cracks passing both parts of the prism, as shown in Figure 4.27b. Therefore, the calculated shear bond of specimens with overlay mortar M70 and M105 represented a lower estimate of the bond strength of shear in the interface. These results agree with the findings of Julio et al. (2006) of the bond between two concrete layers cast at different ages.
It was evident that monolithic failure of composite specimens is the major limitation of this test method in quantifying the bond under shear stresses. In such this case, the bond cohesion and friction coefficients cannot be determined. Nevertheless, it can be determined for specimens with mortar strength equal to 35MPa.

Table 4.7: Results of shear and normal stresses at the interface

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Shear stress [MPa]</th>
<th>Normal stress [MPa]</th>
<th>C.O.V [%]</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>C37-M35</td>
<td>8.5</td>
<td>4.9</td>
<td>11.9</td>
<td>Adhesive</td>
</tr>
<tr>
<td>C37-M70</td>
<td>13.3</td>
<td>7.7</td>
<td>2.5</td>
<td>Monolithic</td>
</tr>
<tr>
<td>C37-M105</td>
<td>14.4</td>
<td>8.3</td>
<td>4.7</td>
<td>Monolithic</td>
</tr>
</tbody>
</table>

a. Adhesive failure mode  
b. Monolithic failure mode

According to BS EN 1992-1-1:2004, when the interface is under combined shear and compression stresses, the cohesion and friction coefficients are controlled by the roughness of the substrate concrete and the bond properties of the new layer. Shear at the interface between concrete cast at different times without steel reinforcement crossing the interface can be expressed by the following equation (BS EN 1992-1-1:2004):

\[ V_{Rdi} = c f_{cta} + \mu \sigma_n \]  

(4.15)
Where: $c$ is a factor depends on the roughness of the interface and $f_{ctd}$ is tensile strength of the new layer in MPa. The cohesion bond ($c f_{ctd}$) is related to the tensile strength of the new layer and the roughness of the substrate concrete ($c$). The substrate roughness also controls the internal friction coefficient ($\mu$). Therefore, the coefficient $c$ and $\mu$ are considered constant for the same substrate surface condition. Equation 4.15 can be applied to C37-M35 to determine the required coefficients of bond stresses, which are the subject of this test.

For C37-M35 samples, the obtained results of $\tau_u$ and $\sigma_n$ are 8.5MPa and 4.9 MPa, respectively. Santos and Julio (2014) suggested that the cohesion bond could be represented by the average value of the bond strength under pure shear and bond under direct tensile stresses. Hence, by assuming of the cohesion bond equal to the adhesion bond under tensile stresses (0.74 MPa), the value of the internal friction coefficient ($\mu$) would be 1.58. This value is very close from the recommended friction coefficient (1.5) by fib Model Code 2010 for rough surfaces. Moreover, Climaco and Regan (2001) explained that the Coulomb criterion can be used to predict the shear and normal stresses at the interface between old and new concrete. It was proposed the following equation to determine the shear stresses at the interface:

$$\tau = 0.25f_{cc}^{2/3} + 1.4\sigma \quad for \quad \sigma \geq 0.1f_{cc}^{2/3} \quad (4.16)$$

$$\tau = 0.1f_{cc}^{2/3} + 3\sigma \quad for \quad 0 \leq \sigma \leq 0.1f_{cc}^{2/3} \quad (4.17)$$

Where: $\tau$ is the shear stress at the interface (N/mm²), $f_{cc}$ is the cylindrical compressive strength of the new layer (N/mm²) and is the normal stress at the interface (N/mm²). By substituting the experimental results of normal strength mortar samples (C37-M35), the shear stresses will be equal to 9.1MPa, which is very close from the obtained shear stresses (8.5MPa). In addition, the first equation demonstrate that the friction coefficient can be taken equal to 1.4 with is close to the proposed value in this study (1.58).
4.6.2 Results of RC beams strengthened with TRM

This section describes the results of the tested beams by means of flexural strength, load deflection response, crack patterns, ductility and slippage of the strengthening layer. Additionally, the effect of these aspects on the behaviour of strengthened RC beams was discussed with respect to the investigated parameters (strength of TRM mortar, connectors ratio, type and amount of textile fibres). The experimental results were used to validate the proposed numerical modelling of RC beams strengthened with TRM in Chapter five.

4.6.2.1 Flexural strength of strengthened RC beams with TRM

The flexural strength was compared with the strength of control (un-strengthened) beams to assess the contribution of the TRM layer in the flexural strength of strengthened RC beams. Table 4.8 presents the cracking, yielding, and ultimate load capacity of the tested beams. The cracking load represents the load at the onset of the first crack, which determined through the experimental observation. The load at yielding of steel reinforcement (yielding load) was quantified from the load-deflection curves as the load where the load-deflection curve was no longer linear.

Control beams failed, as expected, in a typical flexural failure mode through yielding of longitudinal steel reinforcement followed by concrete crushing. The average ultimate load of the control beams was 51kN. These results were confirmed by the observed strain using LVDT in compression zones at the ultimate load, which was 0.002 mm/mm. According to international codes such as Euro Code2 [BS-EN 1992-1-1:2004] and ACI-318 (2011), the concrete reaches its ultimate compressive strength at a strain ranged between 0.003 and 0.0035 mm/mm. Hence, the beam reached its ultimate load due to yielding of longitudinal steel reinforcement before reaching the ultimate compressive strength of concrete.

Despite the debonding of the strengthening layer, all strengthened beams without application cementitious connectors explained higher cracking, yielding and ultimate load than control. Ombres (2011), Contamine et al. (2012), El-Rgaby et al. (2017), Wu, and Lie (2017) observed similar debonding failure mode. Beams strengthened with eight layers of textile basalt fibres explained different ultimate flexural loads ranged between 57.1 kN and 64.7 kN depending on the mortar strength of the TRM. However, beams strengthened with 16 textile basalt layers failed at ultimate load of 67.9kN. The highest ultimate flexural load was observed of beams strengthened with five layers of carbon fibres with an ultimate load of 73.3kN.
Cracks in strengthened RC beams were initiated in higher loads than control. The ratio between the load when the first cracks appeared to the ultimate load ranged between 31% and 39% of beams strengthened with 8 layers of textile fibres. Higher ratio (46%) was observed for beams strengthened with 16 textile basalt layers. For all strengthened beams, except strengthening with textile carbon fibres, longitudinal steel reinforcements were yielded at higher loading level than control beams due to contribution of the TRM strengthening layer.

The application of cementitious connectors increases the contribution of the TRM strengthening layer in the resisting the applied load as a result of improving the interface bond strength. Three failure modes were observed; rupture of textile fibres followed by concrete crushing of beams strengthened with 8 and 5 layers of basalt and carbon fibres, respectively, debonding of strengthening layer of beams strengthened with 16 layers of basalt fibres (with cementitious ratio of 5%), and concrete cover separation of beams strengthened with 16 basalt layers and 7.5% connectors ratio.

Figure 4.28 presents the enhancement in the flexural strength due to TRM strengthening layer. Carbon fibres explained the highest improvement in all stages of loading. The enhancement in cracking load of specimens with high strength mortar was higher than enhancement of yielding and ultimate load. In contrast, the lower increase in cracking load was observed of RC beam strengthened with normal strength mortar (M35 and M70). At the load stage of first crack, the strengthening layer of M35 and M70 improving the load to about 45% and 55%, respectively, compared with control beam. For the same level of loading, the enhancement of high strength mortar (M135) ranged between 105% and 157% depending on the axial strength of the TRM. Moreover, the application connectors raised the enhancement to 172% of beams strengthened with 5 layers of textile carbon fibres.

The enhancement in ultimate load was ranged between 12% and 19% of strengthening layer with normal strength mortar and increased to be varied between 27% and 44% with using high strength mortar (M135). Besides, the application of cementitious connectors increases the enhancement in ultimate load to about 61%.
Table 4.8: Results of the tested control and strengthened RC beams

<table>
<thead>
<tr>
<th>Beam</th>
<th>Load (kN)</th>
<th>Load ratio</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>First crack</td>
<td>Yield</td>
<td>Ultimate</td>
</tr>
<tr>
<td>CON-1</td>
<td>11.5</td>
<td>12.2</td>
<td>44.2</td>
</tr>
<tr>
<td>CON-2</td>
<td>12.8</td>
<td>17.8</td>
<td>55.7</td>
</tr>
<tr>
<td>8B0-M35-1</td>
<td>18.3</td>
<td>25.1</td>
<td>58.2</td>
</tr>
<tr>
<td>8B0-M35-2</td>
<td>17.2</td>
<td>18.9</td>
<td>56.5</td>
</tr>
<tr>
<td>8B0-M70-1</td>
<td>19.2</td>
<td>27.3</td>
<td>63.9</td>
</tr>
<tr>
<td>8B0-M70-2</td>
<td>18.5</td>
<td>27.1</td>
<td>60.1</td>
</tr>
<tr>
<td>8B5-M135-1</td>
<td>25.4</td>
<td>31.4</td>
<td>65.0</td>
</tr>
<tr>
<td>8B5-M135-2</td>
<td>24.7</td>
<td>30.8</td>
<td>63.3</td>
</tr>
<tr>
<td>8B5-M135-1</td>
<td>30.5</td>
<td>31.4</td>
<td>68.1</td>
</tr>
<tr>
<td>8B5-M135-2</td>
<td>32.2</td>
<td>30.1</td>
<td>66.8</td>
</tr>
<tr>
<td>16B0-M135-1</td>
<td>32.0</td>
<td>31.1</td>
<td>67.1</td>
</tr>
<tr>
<td>16B0-M135-2</td>
<td>30.8</td>
<td>30.1</td>
<td>66.6</td>
</tr>
<tr>
<td>16B5-M135-1</td>
<td>30.5</td>
<td>31.4</td>
<td>68.1</td>
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<tr>
<td>16B5-M135-2</td>
<td>32.2</td>
<td>30.1</td>
<td>66.8</td>
</tr>
<tr>
<td>16B7.5-M135-1</td>
<td>32.1</td>
<td>31.1</td>
<td>67.1</td>
</tr>
<tr>
<td>16B7.5-M135-2</td>
<td>30.8</td>
<td>30.1</td>
<td>66.6</td>
</tr>
<tr>
<td>5C0-M135-1</td>
<td>30.8</td>
<td>30.1</td>
<td>63.5</td>
</tr>
<tr>
<td>5C0-M135-2</td>
<td>30.7</td>
<td>30.1</td>
<td>62.8</td>
</tr>
<tr>
<td>5C5-M135-1</td>
<td>33.4</td>
<td>33.2</td>
<td>72.8</td>
</tr>
<tr>
<td>5C5-M135-2</td>
<td>33.0</td>
<td>33.0</td>
<td>72.3</td>
</tr>
</tbody>
</table>

Ave. refers to the average value of two identical beams results

$P_c/P_u$ = cracking load of a beam / ultimate load of a beam

$P_y/P_u$ = yielding load of a beam / ultimate load of a beam

SY-CC is a steel yielding followed by concrete crushing failure mode

DS-CC is the debonding of the strengthening layer followed by concrete crushing failure mode

DS-TR is the debonding of the strengthening layer followed by the textile rupture failure mode

SY-CS is the steel yielding followed by concrete cover separation failure mode

TR-CC is the textile rupture followed by concrete crushing failure mode
To relate the results of the experimental investigation of interface bond test with the maximum load can be applied before slip, the load is calculated based on the shear stresses of the interface bond test, which will be calculated at the interface between the substrate concrete and the strengthening layer, as shown in Figure 29. The maximum load before slip can be determined according to the following equations:

\[
\tau = \frac{VQ}{bl} \quad (4.18)
\]

\[
Q = A \cdot Y \quad (4.19)
\]

\[
I = \frac{bh^3}{12} \quad (4.20)
\]

\[
P = 2V \quad \text{(For simply supported beam)} \quad (4.21)
\]

Where: \(\tau\) is the shear stress (N/mm\(^2\)), \(V\) is the shear force (N), \(Q\) is the first moment of inertia (mm\(^3\)), \(b\) is width of the section (mm), \(I\) is the second moment of inertia of the cross section (mm\(^4\)) and \(P\) is the maximum flexural load (N).
Table 4.9 present the results of beams strengthened with TRM that has mortar strength M35, M70 and M135 and exhibited TRM debonding failure mode. The results demonstrated a good agreement with the experimental load of beam 8B0-M35. However, the variation in results was obvious in the rest of the beams, where the predicted load was much higher than the experimental. That could be due to neglecting the effect of cracking on the stress distribution. In addition, it is worth mentioning that debonding of the strengthening layer of beam 8B0-M35 started from the ends where the shear is ultimate. All other beams exhibited debonding started from the mid span, zero shear region of the investigated beams, and continued towards the ends. That may explain the variation between the experimental and predicting loads based on the above formulation. These calculations can be reinforce the reasons for adopting numerical analysis in this study to predict the behaviour of RC beams strengthened with TRM.

Table 4.9: Results of maximum load based on shear stresses

<table>
<thead>
<tr>
<th>Beam</th>
<th>$F_b = \tau$ (N/mm$^2$)</th>
<th>$Q$ (mm$^3$)</th>
<th>$I$ (mm$^4$)</th>
<th>$V$ (N)</th>
<th>$P$ (KN)</th>
<th>$P_{exp}$ (KN)</th>
<th>$P/P_{exp}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>8B0-M35</td>
<td>0.74</td>
<td>273437.5</td>
<td>833333333.3</td>
<td>28.15</td>
<td>56.3</td>
<td>57.1</td>
<td>0.99</td>
</tr>
<tr>
<td>8B0-M70</td>
<td>1.15</td>
<td>273437.5</td>
<td>833333333.3</td>
<td>43.8</td>
<td>87.6</td>
<td>60.5</td>
<td>1.45</td>
</tr>
<tr>
<td>8B0-M135</td>
<td>1.9</td>
<td>273437.5</td>
<td>833333333.3</td>
<td>72.35</td>
<td>144.7</td>
<td>64.7</td>
<td>2.23</td>
</tr>
<tr>
<td>16B0-M135</td>
<td>1.9</td>
<td>273437.5</td>
<td>833333333.3</td>
<td>72.35</td>
<td>144.7</td>
<td>67.9</td>
<td>2.13</td>
</tr>
<tr>
<td>5C0-M135</td>
<td>1.9</td>
<td>273437.5</td>
<td>833333333.3</td>
<td>72.35</td>
<td>144.7</td>
<td>73.3</td>
<td>1.97</td>
</tr>
</tbody>
</table>
4.6.2.2 Load-deflection response of strengthened RC beams with TRM

The load-deflection curves (Figure 4.30) displayed a considerable enhancement in the stiffness of the strengthened RC beams with respect to the control beam. The load-deflection curve of the control beams was nearly bilinear response characteristics, which is similar to the general behaviour of under-reinforced concrete beams under flexural loading. In addition, a gradual descending branch was observed of both control beams after reaching the ultimate load. However, all strengthened beams displayed a sudden descending branch in the load-deflection response after achieving the ultimate capacity. That response due to either reaching the ultimate capacity of the textile fibres, which have brittle behaviour, or due to a considerable slippage between the strengthening layer and substrate concrete. In addition, it is interesting to note that the descending part of all strengthened beams was dropped beyond the descending part of controls, which could be indicating to reaching the steel reinforcement the yield stresses.

Bending stiffness beams strengthened with carbon fibres was higher than basalt fibres counterparts, due to increasing the axial stiffness \((E_fA_f)\) of the textile reinforcement. Similarly, beams strengthened with 16 textile basalt layers presented higher stiffness than eight layers counterparts.
Figure 4.30: Load deflection curves of the control and strengthened RC beams
4.6.2.3 Crack patterns of strengthened RC beams with TRM

Figures 4.31 to 4.40 demonstrates the crack patterns of the tested RC beams at failure. Control beams (Figure 4.31) exhibited flexural cracks initiated from the tension face of the beam at the region between the applied load. As the load increases, these cracks were propagated vertically towards the compression face. At loading stage close to the ultimate, cracks under the applied load point became wider, and with increasing the load, cracks propagated quickly towards the compression face. In addition, the crushing of concrete was observed in the compression face after reaching the stress in the longitudinal steel reinforcement the yield limit.

The addition of TRM layers to the RC beams increases the load that required for initiation the first crack with respect to the control beams. The delay in the initiation cracks differs with the properties of the strengthening layer as presented in Table 4.10. Beams 8B0-M35 displayed flexural cracks in the substrate concrete in the region between the applied loading points. These cracks started from the interface between the concrete and strengthening layer and propagated vertically towards the compression face of the beam. No evidence of initiation cracks in the strengthening layer was observed. With the progress of loading, horizontal cracks at the interface were initiated from the mid-span of the beam and propagated towards the ends which led to peeling the TRM layer from the substrate concrete. Similar to control beams, concrete crushing was observed at the compression face of the beam. The same cracks configuration was observed of beams 8B0-M70 and 8B-M135, as shown in Figures 4.33 and 4.34. However, beams strengthened with high strength mortar (8B-M135) displayed flexural cracks in the strengthening layer before exhibiting a local separation (depending) of the TRM layer at the mid-span of the beams. The same behaviour was observed of beams strengthened with carbon fibres (5C0-M135) as shown in Figure 4.39.

Despite the improvement that achieved with using high strength mortar (M135), beams strengthening with 16 layers of textile basalt fibres presented a peeling of TRM and crack patterns similar to 8B0-M35 beams (Figure 4.36). Strengthened beams with application 5% cementitious connectors ratio exhibited flexural cracks (Figures 4.35 and 4.37) initiated from the strengthening layer and propagated vertically towards the compression face of the beams with 8 and 16 textile basalt fibres. However, 7.5% connectors ratio led to initiation horizontal cracks at the concrete cover which led, with increasing the loading, to concrete cover separation failure (Figure 4.38). One of the two beams that strengthened with 5 carbon layers in presence of 5% cementitious connectors explained flexural cracks similarly to the beams with textile basalt fibres. While the other beam (5C5-M135)
exhibited concrete cover separation in the same manner of 16B7.5-M135, as shown in Figure 4.40. It is also observed for all beams exhibited concrete cover separation, no evidence of concrete crushing in compression face and flexural cracks in the strengthening layer.

Figure 4.31: Crack patterns of the control beam

Figure 4.32: Crack patterns of beams 8B0-M35
Figure 4.33: Crack patterns of beams 8B0-M70

Figure 4.34: Crack patterns of beams 8B0-M135

Figure 4.35: Crack patterns of beams 8B5-M135
Figure 4.36: Crack patterns of beams 16B0-M135

Figure 4.37: Crack patterns of beams 16B5-M135

Figure 4.38: Crack patterns of beams 16B7.5-M135
4.6.2.4 Ductility of strengthened RC beams with TRM

Ductility is an important parameter in the design of a concrete structure, which can be defined as the ability to sustain large deformations without losing strength capacity. Ductility can be expressed in terms of deformations or energy (Attari et al., 2012). In this study, ductility based on deflection (deflection ductility) was adopted to assess the ductility of control and strengthening RC beams. Deflection ductility index is defined as the ratio between the deflection at ultimate load to the deflection at the yielding of steel reinforcement, and can be determined based on the following equation (Attari et al., 2012):

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y}$$

(4.22)
Table 4.10 presents the details of deflections at yielding and ultimate load and the deflection ductility index of control and strengthening RC beams. Figure 4.41 demonstrates that all strengthened RC beams, except beams strengthening with 16 layers of textile basalt fibres, either have the same ductility of the control beam or higher. The deflection ductility of the control beams was 1.35. Beams strengthened with eight layers of textile basalt fibres using normal strength mortar (M35 and M70) exhibited the same ductility (1.35) of the control beams. However, despite using high strength mortar (M135) which should reduce the ductility of the strengthened beams in comparison with normal strength mortar counterparts, it exhibited higher ductility. That may be attributed to the failure mode of the beams 8B0-M135, which exhibited gradual slippage of the strengthening layer at the mid-span of the beam. That can be confirmed by the less ductility of 8B5-M135 (1.34) which did not display slipping of TRM strengthening layer. In addition, the same behaviour was observed of beams strengthened with textile carbon fibres, whereas, beams 5C0-M135 yielded higher ductility than beams with cementitious connectors (5C5-M135).

The relationship between the deflection ductility index and the failure mode is also evident on beams strengthened with 16 layers of textile basalt fibres. Beams with sudden failure mode either peeling of TRM or concrete cover separation had less ductility than control beams. However, non-considerable reduction in ductility was observed of beams 16B5-M135 due to the exhibiting the same failure mode of beams 8B5-M135.
### Table 4.10: Deflections and ductility index of control and strengthened RC beams

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Mid span deflection (mm)</th>
<th>Ductility index</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Yield Ave.</td>
<td>Ultimate Ave.</td>
</tr>
<tr>
<td>CON-1</td>
<td>7.2</td>
<td>10.7</td>
</tr>
<tr>
<td>CON-2</td>
<td>7.4</td>
<td>8.9</td>
</tr>
<tr>
<td>8B0-M35-1</td>
<td>8.4</td>
<td>10.7</td>
</tr>
<tr>
<td>8B0-M35-2</td>
<td>8.4</td>
<td>11.9</td>
</tr>
<tr>
<td>8B0-M70-1</td>
<td>8.4</td>
<td>12</td>
</tr>
<tr>
<td>8B0-M70-2</td>
<td>7.7</td>
<td>9.8</td>
</tr>
<tr>
<td>8B0-M135-1</td>
<td>8.1</td>
<td>12.1</td>
</tr>
<tr>
<td>8B0-M135-2</td>
<td>7.8</td>
<td>10.7</td>
</tr>
<tr>
<td>8B5-M135-1</td>
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<td>10.3</td>
</tr>
<tr>
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<td>7</td>
<td>10</td>
</tr>
<tr>
<td>16B0-M135-1</td>
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</tr>
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<td>16B0-M135-2</td>
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</tr>
<tr>
<td>5C5-M135-2</td>
<td>9.8</td>
<td>11.8</td>
</tr>
</tbody>
</table>

#### 4.6.2.5 Slipping of strengthening layer of strengthened RC beams with TRM

For all strengthened RC beams, the slip of the strengthening layer with respect to the substrate concrete had different values even within the same properties of the strengthening layer. That may be due to initiation cracks in the concrete within the measured slip region, which led to display overestimated values of slips. The load slip curves at the maximum bending moment region of the tested beams are illustrated in Appendix C.

Therefore, to study the slip of the strengthening layer, the maximum slip that recorded within the maximum bending moment region was adopted. In addition, to demonstrate the effect of the cementitious connectors and the strengthening layer properties and the slip, the load and slip of beam 8B0-M35 will be considered as a reference for comparison with other investigated case studies. Accordingly, the slip corresponding to the cracking load of 8B0-M35 was determined for all strengthened beams, and it was coded as Se (equivalent
slip). Table 4.11 and Figure 4.42 demonstrate the slip of the strengthening layer of the strengthened RC beams. The equivalent slip of the tested beams was varied between 0.044 mm and 0.02 mm. Beam 8B0-M35 displayed the highest slip (0.04 mm) among all other strengthened RC beams. Beams 8B0-M70 and 8B0-M135 exhibited less slip (0.034 mm and 0.029 mm, respectively) with respect to reference strengthened beam (8B0-M35). A similar reduction in slip was observed of beams 16B0-M135 and 5C0-M135. The application of 5% cementitious connectors’ ratio reduced the slip from 0.029 mm to 0.02 mm of beams strengthened with eight layers of textile fibres (beams 8B5-M135). Non-considerable difference was observed between the slip of beams 16B5-M135 (Se = 0.026 mm) and 16B7.5-M135 (Se = 0.027 mm).

Table 4.11: Slip results of strengthened RC beams

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max. slip measurement</th>
<th>Equivalent slip measurement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Load (kN)</td>
<td>Slip (mm)</td>
</tr>
<tr>
<td>8B0-M35-1</td>
<td>18.3</td>
<td>0.046</td>
</tr>
<tr>
<td>8B0-M35-2</td>
<td>17.2</td>
<td>0.042</td>
</tr>
<tr>
<td>8B0-M70-1</td>
<td>19.2</td>
<td>0.053</td>
</tr>
<tr>
<td>8B0-M70-2</td>
<td>18.5</td>
<td>0.037</td>
</tr>
<tr>
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<td>0.059</td>
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<td>24.7</td>
<td>0.09</td>
</tr>
<tr>
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<td>27.4</td>
<td>0.058</td>
</tr>
<tr>
<td>8B5-M135-2</td>
<td>27.1</td>
<td>0.056</td>
</tr>
<tr>
<td>16B0-M135-1</td>
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<td>0.026</td>
</tr>
<tr>
<td>16B0-M135-2</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>16B5-M135-1</td>
<td>30.5</td>
<td>0.07</td>
</tr>
<tr>
<td>16B5-M135-2</td>
<td>32.2</td>
<td>0.089</td>
</tr>
<tr>
<td>16B7.5-M135-1</td>
<td>32.1</td>
<td>0.078</td>
</tr>
<tr>
<td>16B7.5-M135-2</td>
<td>30.1</td>
<td>0.09</td>
</tr>
<tr>
<td>5C0-M135-1</td>
<td>30.8</td>
<td>0.091</td>
</tr>
<tr>
<td>5C0-M135-2</td>
<td>30.7</td>
<td>0.1</td>
</tr>
<tr>
<td>5C5-M135-1</td>
<td>33.4</td>
<td>0.088</td>
</tr>
<tr>
<td>5C5-M135-2</td>
<td>33.0</td>
<td>0.13</td>
</tr>
</tbody>
</table>
4.6.2.6 Effect of mortar strength on the behaviour of strengthened RC beams

The results of beams 8B0-M35, 8B0-M70 and 8B0-M135 were analysed and discussed to address the effect of mortar strength of the TRM strengthening layer on the behaviour of strengthened RC beams.

As shown in Figures 4.28 and 4.43, the enhancement of the cracking, yielding and ultimate capacity due to the strengthening is proportional to the mortar strength of the TRM. That could be attributed to the enhancement of the interface bond strength or the improvements in the behaviour of TRM composite or both. All these reasons are accepted according to the experimental results of bond strength between concrete and mortar and the direct tensile and flexural behaviour of TRM composites, which demonstrated high improvement in performance with increasing the mechanical properties of mortar. However, the crack patterns of beams strengthened with normal strength mortar (M35 and M70) (Figures 4.32 and 4.33) displayed few cracks in strengthening layer and the concentration of cracks was in the interface and substrate concrete. That led to beam failure before reaching a sufficient loading level in the strengthening layer. Moreover, with the same level of loading (cracking load of beam 8B0-M135), high strength mortar (M135) reduced the slip to about 34% of beam 8B0-M135. That could refer to the higher influence of the mortar strength on the interface bond rather than improving the strength of TRM.
composites. The achieved enhancement at the level of cracking load, which were 54% of M135 and 10% of M35, can be considered another evidence of the effectiveness of mortar strength in enhancing the interface bond strength. Despite the higher stiffness of the strengthened beams with mortar M135 than M35 and M70 (Figure 4.44), beams 8B0-M135 presented higher deflection ductility than 8B0-M35 and 8B0-M70. That was due to reach higher ultimate load, which correspond with increasing the deflection as a result of more effective interface bond strength.

It can be seen that the effectiveness of high strength mortar in improving the behaviour of the strengthening RC beams with TRM by means of increasing the strength and stiffness without reducing the ductility of the strengthened member. In addition, the interface bond strength has a predominant impact on the behaviour of strengthened RC beams, which means limiting the proposed enhancement of a structural member with the available interface bond strength.

![Figure 4.43: Effect of mortar strength on the improvement of strengthening](image1)

![Figure 4.44: Load-deflection curves of RC beams strengthening with M35,M70 and M135](image2)
4.6.2.7 Effect of fibres amounts on the behaviour of strengthened RC beams

The results of two categories were compared to understand the effect of the effect of amounts of fibres on the behaviour of RC beams. The first category included beams 8B0-M135 and 16B0-M135, which did not have cementitious connectors. The same textile and mortar properties but with the inclusion of 5% cementitious connectors ratio were examined in the second category (8B5-M135 and 16B5-M135).

Figure 4.45 demonstrates the superiority of the strengthened RC beams with cementitious connectors in increasing the ultimate load capacity. The figure also indicates to increase the contribution of textile fibres in carrying the applied load with increasing the amount of textile fibres. However, this improvement does not linear because it is restricted by the interface bond strength.

For specimens without connectors, increasing the amount of textile fibres from 8 to 16 layers increase the resistance of strengthened beam to initiation cracking from 105% to 157% with respect to the cracking load of control specimens. That means the enhancement of 16 textile basalt layers was about 150% of 8 layers. However, at ultimate load stage, the enhancement due to the strengthening was increased to about 22% (from 27% to 33%) when the amount of textile fibres was doubled. That was due to the debonding of the strengthening layer of beams with 16 textile basalt layers (Figure 4.37). The debonding was due to initiation flexural cracks in the substrate RC beams, which led to initiation and propagation of horizontal cracks along the interface. In addition, Figure 4.43 shows increasing in the stiffness of strengthened beams as the amount of textile fibres increases. That was due to increase the axial stiffness of the strengthening layer because of the modulus of elasticity of fibres is much higher than mortar. The effect of TRM stiffness was evident through reducing the deflection ductility from 1.43 of 8 layers to 1.08 of 16 layers. That effect is also observed on the slip of the strengthening layers, where TRM with 16 layers exhibited less slip at the same level of loading.

Specimens with cementitious connectors demonstrate a higher effect of the amount of textile fibres on the behaviour of strengthened RC beams at the ultimate load stage. The ratio between the enhancement of 16 and 8 layers at initial stages (up to cracking load) and final stage (ultimate load) was 127% and 126%, respectively. That demonstrates the effect of connectors in enhancing the interface bond strength, which led to minimise the consequences of debonding on the enhancement of strengthening. Despite the flexural cracks that observed in strengthening layer of beams 16B5-M135, a debonding of the strengthening layer was noted at the mid-span of the beam close to the ultimate load stage.
(Figure 4.37). That demonstrates the reason behind low enhancement of strengthening layer despite increasing the amount of fibres. A negligible effect of the amount of textile fibres was observed on the ductility of the investigated beams with cementitious connectors. Moreover, it interesting to notice from Figure 4.46 that beams strengthened with 8 layers of basalt fibres explained higher stiffness than beams strengthened with 16 layers. That could be attributed to the slip of textile reinforcement within the TRM mortar due to reducing the amount of mortar that supports each textile filament. The effect of the bond between textile fibres and mortar became effective in the presence of improving the interface bond strength, which led to increasing the transferred stresses from the substrate beam to the strengthening layer.

Figure 4.45: Effect of amount of textile basalt fibres on the improvement of strengthening

Figure 4.46: Load-deflection curves of RC beams strengthening with 8 and 16 textile basalt
It can be stated that increasing the amount of textile fibres improve the capacity of the RC beams at all load stages. However, it was observed the improvement was effective in the initial stages of loading rather than ultimate load. That explained the effect of interface bond strength on the contribution of the strengthening layer. In addition, in the presence of sufficient bond strength, increasing the textile reinforcement ratio in the strengthening layer could increase the slippage of the textile within the mortar. Hence, the ratio of textile fibres should be limited to the available bond strength of mortar, and that indicates to the need of a mortar with high adhesion properties (high strength mortar).

4.6.2.8 Effect of fibres type on the behaviour of strengthened RC beams

The axial strength of the textile fibres was calculated based on the material properties of fibres to understand the effect of fibre properties on the behaviour of strengthened RC beams. For the investigated case studies (5 layers of carbon and 8 and 16 layers of basalt), the axial strength (ultimate tensile strength × the area of textile fibres) are 37kN, 74kN and 76 kN of beams 8B5-M135, 16B5-M135 and 5C5-M135, respectively. Therefore, the effect of textile fibre type was studied by comparing the experimental behaviour of two groups; strengthened beams without connectors (16B0-M135 and 5C0-M135) and beams with cementitious connectors (16B5-M135 and 5C5-M135).

For the first group, the enhancement of 5 carbon layers was the same of 16 basalt layers at the initial stage of loading (up to cracking load). However, at ultimate load stages, carbon fibres became more effective than basalt fibres through enhancing the load to about 44% compared with 33% of 16 textile basalt fibres. That may be attributed to either the effect of modulus of elasticity of textile fibres or the nature of stress distribution inside the TRM composite, which depends on the textile configuration. It is worth to mention that under direct tensile test, the ultimate axial tensile stresses in 5 carbon and 16 basalt layers were found 533MPa and 574Mpa, respectively. That could indicate better stress distribution of textile basalt fibres comparing with carbon fibres. Accordingly, the higher performance of carbon fibres may relate to the higher properties of the modulus of elasticity. Figure 4.48 (load-deflection responses) demonstrates that strengthened beams of carbon fibres have a higher stiffness than basalt fibres which may confirm the above claim. However, the ductility of beams strengthened with carbon fibres (1.44) was higher than basalt fibres (1.32) counterpart. That was due to the lower deflection that observed of beams strengthened with carbon (7.7mm) at yielding load, compared with 9.8mm deflection of beams strengthened with basalt fibres.
Similar behaviour of textile carbon fibres was observed in the presence of 5% cementitious connector’s ratio. The increase in the ultimate capacity of beams 5C5-M135 was about 61% compared with 43% of beams 16B5-M135. That means the enhancement of carbon specimens is about 1.4 times the enhancement of basalt fibres. These values are similar to the obtained values of strengthened RC beams without cementitious connectors. In addition, the presence of higher interface bond strength (due to application connectors) increases the differences of stiffness between the beams strengthened with textile carbon and basalt fibres (Figure 4.47). That was because of the high differences in the modulus of elasticity, which became obvious in presence of effective interface bond strength. On other hands, both types of textile fibres explained a non-considerable difference in the ductility (1.32 and 1.34) which was attributed to the high deflections corresponding to the high ultimate load, which compensated the effect of higher stiffness of carbon fibres that could reduce reduction ductility.

Based on the obtained results, it can be stated that the textile fibres properties have a considerable influence on the behaviour of strengthening RC beams with TRM through controlling the amount of the enhancement of the ultimate load and failure mode. Textile fibres with high mechanical properties can present higher improvement in the ultimate capacity in comparison with textile fibres with low mechanical properties.
4.6.2.9 Effect of cementitious connectors on the behaviour of strengthened RC beams

This section demonstrates the effect of the cementitious connectors on the behaviour of strengthened RC beams by comparing the results of three groups; beams strengthened with 8 basalt layers (8B5-M135 and 8B5-M135), beams strengthened with 16 basalt layers (16B0-M135, 16B5-M135 and 16B5-M135) and beams strengthened with 5 carbon layers(5C5-M135 and 5C5-M135).

For all groups, the application of cementitious connectors increasing the contribution of strengthening layer in resisting the applied load and changing the failure mode to behave close to the control beams (flexural failure mode). For the first group, the application of 5% of cementitious connectors’ ratio increased the enhancement to about 26% (from 27% of beams without connectors to 34% of beams with connectors) at ultimate load. In addition, the connectors changed the failure mode to rapturing of textile basalt fibres followed by crushing of concrete. At initial load stage, the contribution of connectors had less impact in enhancing the load through increasing the enhancement of load from 105% to 123% (17% improvement). On other hands, the cementitious connectors reduced the ductility of the strengthened beams from 1.43 to 1.34 because of increase the contribution of the TRM composite in resisting the bending of the strengthened beams and reaching the ultimate capacity of the textile fibres. That was obvious through the higher stiffness that observed of beams with connectors from the load-deflection responses (Figure 4.30).

For the second group (beams strengthened with 16 basalt fibres), a negligible effect of connectors was observed in the initial stages of loading. However, at ultimate load stage, connector’s ratio of 5% and 7.5% explained 1.3 and 1.55 times the enhancement of strengthening layer without connectors. In addition, the application of connectors changed the failure mode from peeling off the strengthening layer of beams 16B0-M135 to localised debonding at the mid-span for beams 16B5-M135, see Figure 4.36 and 4.37. That may be due to insufficient interface bond strength and more cementitious connectors are required. On other hands, increasing the connector ratio to 7.5% enhanced the interface bond strength (the enhancement in ultimate load to about 51%), but led to the separation of concrete cover (Figure 4.39) before reaching the ultimate capacity of the textile fibres. That may be due to exceeding the stresses in concrete its tensile strength or due to the microcracks that may be due to initiation in the concrete during drilling the holes. Another reason may be applicable, for the same interface area, when the ratio of connectors increases the area between the connectors decreases, which lead to destroying this area when the stress at the connectors exceeds the capacity of concrete.
The impacts of connectors on the failure mode of the strengthened RC beam led to improving the ductility by means of achieving a higher value of deflection at ultimate load. For instance, the ductility of beams 16B0-M135 and 16B5-M135 was 1.08 and 1.32, respectively. That was because of beams 16B0-M135 exhibited peeling of the strengthening layer, which limited the ultimate deflection that corresponding the load.

For beams strengthened with textile carbon fibres, the application of 5% connectors’ ratio increased the enhancement of the ultimate capacity from 44% of beams without connectors to about 61% of beams with connectors. That means strengthened beams with connectors explained 1.4 times the enhancement of specimens without connectors. In addition, the application of connectors changed the failure mode from local debond to flexural failure mode with textile carbon rupturing. In addition, higher stiffness was observed of beams with connectors (Figure 4.30).

Strengthened beams with connectors explained a flexural failure mode by means of rupturing of textile fibres followed by crushing of concrete (Figure 4.41). That demonstrates a monolithic work of the composites beam and strengthening layer in resisting the applied load. However, one beam of 5C5-M135 explained a concrete cover separation at a load very close to the other beam with the same strengthening properties. That may be due to initiation cracks during drilling of the substrate concrete, which led to weakening the concrete.

It can be stated that the application of sufficient connectors improves the interface bond through changing the failure mode from debonding of strengthening layer to the flexural failure mode. The results explained that the effectiveness of connectors ranged between 26% and 55% in increasing the enhancement of the strengthened beams. However, the effectiveness is bounded by the strength of the substrate concrete and the modulus of elasticity of the textile fibres.
4.6.3 Results of repaired corroded RC beams using TRM

Figure 4.48 demonstrates the tension face appearance of the corroded RC beams after subjecting to corrosion for 30 days. The visual inspection of the corroded RC beams displayed different corrosion consequences by means of cracks configuration and amount of rust on the concrete surface. The crack direction of all beams was longitudinal along the steel rebars. That indicates to a different degree of corrosion was created inside the beams, despite all beams were subjected to the same corrosion conditions.

![Corroded RC beams](image)

Figure 4.48: Corroded RC beams

After repairing the corroded beams using TRM, all beams were tested under four-point flexural loading as described in Section 4.3.3. The corroded rebars were extracted from the RC beams (Figure 4.49), and the rebars were cleaned in the same procedure that used for small-scale beams, as shown in Figure 4.51. The degree of corrosion was determined for each beam using the same process that used for small-scale beams in Section 4.4.2.1.
Figure 4.49: Extraction of the corroded steel rebars from RC beams

Figure 4.50: Cleaning corroded rebars and corrosion configurations
4.6.3.1 Flexural strength of repaired RC beams with TRM

This section presents the effectiveness of TRM in repair corroded RC beams through comparing the behaviour of the repaired beams with corroded reference beams (control beams). In addition, the results were also compared with the behaviour of beams strengthened with TRM without subjecting to corrosion to demonstrate the possible variation in the TRM performance in the presence of corrosion.

Table 4.12 presents the summary of the flexural and the corrosion test results of the investigated beams. The results demonstrated a different degree of corrosion of the tested beams despite all beams was subjected to the same corrosion conditions. The degree of corrosion was ranged between 4% and 7.3%. However, according to theoretical calculations and the results of small-scale beams, the degree of corrosion should be about 9-10%. That difference was due to the effect of shear reinforcement in sharing the applied current with the longitudinal reinforcement. That also could be a reason for the variation of the degree of corrosion among the tested beams by means of exhibited different level of insulation between the shear and longitudinal reinforcement. In addition, the difference in the degree of corrosion was accompanied with variation in the maximum crack width of concrete due to corrosion. The maximum crack width was ranged between 0.207 mm and 0.647mm of a degree of corrosion 4% and 7.3%, respectively.

The flexural results of the corroded control beams (beams without repaired layers) explained a reduction in the flexural capacity with respect to the non-corroded beams. The decrease was proportional to the degree of corrosion. The ultimate capacity of beam COR-1 and COR-2 were 45.3kN and 47.8kN, respectively, which refers to a reduction in ultimate load of 11% and 8% in comparison with uncorroded beams. The difference in the ultimate capacities of the corroded control beam is attributed to the variation of the degree of corrosion (7.3% and 5.6%). Both beams displayed flexural failure mode through yielding of steel reinforcement followed by crushing of concrete at the compression face of the beam.

All repaired beams exhibited greater strength than control beams (non-corroded beams). That indicates the efficiency of the TRM layer in restoring the strength of the corroded RC beams. Figure 4.51 displays a variation in the effectiveness of TRM layer between repair and strengthening procedure. This variation appeared by the reduction in the strength of repaired corroded beams with respect to strengthening RC beams have the same properties of the TRM layer. The highest reduction was observed at the initial stage of loading (up to cracking load) which may be due to crack initiation in the tension face of concrete, which led to reduce concrete strength.
Beams C8B0-M135 explained a negligible reduction in ultimate strength compared with strengthening beams despite corrosion, which ranged between 4% and 4.7%. In addition, both beams explained flexural failure mode in contrast with strengthening beams, which displayed debonding of TRM at the mid-span of the beams. Similar non-considerable effect of corrosion was found of beam C8B2.5-M135-1 that has 5% degree of corrosion. However, beam C8B2.5-M135 presented 9% reduction in ultimate load in presence of 6.1% degree of corrosion.

Both corroded beams repaired with 5 layers of textile carbon fibres exhibited a reduction in the strength with respect to strengthening beams counterpart. The reduction was ranged between 4% and 12% of the degree of corrosion 4% and 4.9%, respectively. In addition, beam C5CB2.5-M135-2 (4.9% corrosion) presented a concrete cover separation close to the ultimate load stage.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Load (kN)</th>
<th>Corrosion</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>First crack</td>
<td>Yield</td>
<td>Ultimate</td>
</tr>
<tr>
<td>COR-1</td>
<td>10.4</td>
<td>41</td>
<td>45.3</td>
</tr>
<tr>
<td>COR-2</td>
<td>11.7</td>
<td>42.6</td>
<td>47.1</td>
</tr>
<tr>
<td>C8B0-M135-1</td>
<td>21.4</td>
<td>56.3</td>
<td>62.7</td>
</tr>
<tr>
<td>C8B0-M135-2</td>
<td>20.0</td>
<td>57.6</td>
<td>65</td>
</tr>
<tr>
<td>C8B2.5-M135-1</td>
<td>21.2</td>
<td>58.5</td>
<td>68</td>
</tr>
<tr>
<td>C8B2.5-M135-2</td>
<td>22.8</td>
<td>54.8</td>
<td>62.3</td>
</tr>
<tr>
<td>C5CB2.5-M135-1</td>
<td>28.1</td>
<td>70.0</td>
<td>79.1</td>
</tr>
<tr>
<td>C5CB2.5-M135-2</td>
<td>26.2</td>
<td>65.6</td>
<td>72.6</td>
</tr>
</tbody>
</table>

C_w is the maximum width of crack (mm)

D_C is the degree of corrosion (%)

SY-CC is steel yielding followed by concrete crushing failure mode

SY-CS is steel yielding followed by concrete cover separation
It can be stated that high strength TRM composite is a powerful material in repairing corroded RC beams. In addition, the effectiveness of TRM as a repair/strengthening does not identify in the presence of corrosion, and the reduction in its contribution should be considered, especially when the degree of corrosion is higher than 4%.

4.6.3.2 Load deflection curves of repaired RC beams with TRM

Figure 4.52 presents the load-deflection curves of the control corroded and repaired RC beams. As can be seen from the figure, the load-deflection curves of the corroded control beams show that the corrosion affects the performance of RC beams at ultimate load stage only. Both beams displayed negligible differences in stiffness up to reaching the yielding of steel reinforcement load stage. All repaired beams presented higher stiffness (Figure 4.52) than control beams with and without corrosion. That was due to increasing the stiffness of the repaired beam due to the TRM.
4.6.3.3 Crack patterns of repaired RC beams with TRM

As shown in Figure 4.53, flexural cracks were initiated at the mid-span of beams COR-1 and COR-2 and propagated vertically towards the tension face of the beams in a typical flexural failure mode.

Similarly, the crack patterns of beams C8B0-M135-1 and C8B0-M135-2 (Figure 4.54) explained that flexural cracks were initiated at the mid-span of the beams and propagated vertically towards the compression face of the beams. In addition, both beams explained horizontal cracks at the level of longitudinal steel reinforcement at mid-span of the beams. These cracks dissipated the transferred stresses from the substrate RC beams to the strengthening layer, which led to reduce the possibility of initiation crack at the interface level, which can lead to debonding of the strengthening layer. Similar horizontal crack at
the mid-span was observed of beams C8B2.5-M135 (Figure 4.55), but denser crack
distribution was also observed. That may indicate higher interface bond strength due to the
presence of the cementitious connectors, which led to increasing the transferred stresses
between the substrate concrete and TRM layer. The effectiveness of improving the
interface bond strength was evident from the higher ultimate capacity of beams with
connectors.

Figure 4.56 presents that beam C5CB2.5-M135-1 displayed flexural cracks initiated
from the TRM similarly to beams strengthened with textile basalt fibres. Horizontal cracks
at the mid-span of the beam at the level of longitudinal reinforcement were also observed.
However, Figure 4.55, beam C5CB2.5-M135-2 that had a higher degree of corrosion
(4.9%) than beam C5CB2.5-M135-1 displayed beside the flexural cracks horizontal shear
cracks in the concrete cover. These cracks were joined as increased the applied load to be
wider and caused destroying of concrete cover.

Figure 4.53: Crack patterns of control corroded beams COR-1 and COR-2

Figure 4.54: crack patterns of beams C8B0-M135

Figure 4.55: Crack patterns of beams C8B0-M135-1 and C8B0-M135-2
4.6.3.4 Ductility of repaired RC beams with TRM

Table 4.13 presents the deflections and ductility index of the corroded control and repaired RC beams. The corroded control beams exhibited ductility ranged between 1.57 and 1.73, which is higher than non-corroded control beams. That may be attributed to the reducing in reinforcement area, because of corrosion, which in turns decrease the deflection accompanied to the yielding of steel reinforcement.

All repaired beams displayed less ductility, as expected than control-corroded beams due to increasing the stiffness (deflection) at the initial stage in the presence of the TRM repair layer. The ductility varied between 1.24 and 1.5 depending mainly on the failure mode of the repaired beam, which relates with the degree of corrosion and properties of the
interface bond strength. The corrosion influenced ductility of repaired beams in terms of its consequences on the interface bond strength rather than in terms the variation of the longitudinal steel reinforcement properties. Beams C8B0-M135-1 and C8B2.5-M135-1, for example, have approximately the same degree of corrosion and deflection at yielding load, but different ductility (1.24 and 1.37). That may be due to the debonding of the strengthening layer of beam C8B0-M135-1 because of insufficient interface bond strength, which limited the ultimate deflection accompanied with load. Similarly, beams C5CB2.5-M135-1 explained higher ductility (1.5) than beam C5CB2.5-M135-2 (1.28) due to the separation of the concrete cover of the later beam because of the higher degree of corrosion.

Table 4.13: Deflections and ductility index of corroded control and repaired RC beams

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Dc (%)</th>
<th>Mid span deflection (mm)</th>
<th>Ductility index ($\mu_\Delta$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Yield</td>
<td>Ultimate</td>
</tr>
<tr>
<td>COR-1</td>
<td>7.3</td>
<td>6.0</td>
<td>9.5</td>
</tr>
<tr>
<td>COR-2</td>
<td>5.6</td>
<td>6.3</td>
<td>10.9</td>
</tr>
<tr>
<td>C8B0-M135-1</td>
<td>4.7</td>
<td>6.7</td>
<td>8.3</td>
</tr>
<tr>
<td>C8B0-M135-2</td>
<td>4</td>
<td>7.3</td>
<td>9.5</td>
</tr>
<tr>
<td>C8B2.5-M135-1</td>
<td>5</td>
<td>6.8</td>
<td>9.3</td>
</tr>
<tr>
<td>C8B2.5-M135-2</td>
<td>6.1</td>
<td>7.2</td>
<td>8.9</td>
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<td>C5C2.5-M135-1</td>
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<td>6.8</td>
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<tr>
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<td>4.9</td>
<td>7.4</td>
<td>9.5</td>
</tr>
</tbody>
</table>

4.6.3.7 Effect of cementitious connectors of repaired RC beams with TRM

The effect of application cementitious connectors on the performance of repaired corroded RC beams was studied by comparing the behaviour of beams C8B0-M135 and C8B0-M135. These beams have a close degree of corrosion (4.7% and 5%) and different connector’s ratio (0 and 5%).

The results showed improvement in the flexural strength of the repaired RC beams in comparison with specimens without connectors’ counterparts. Beam C8B2.5-M135-1, which has corrosion level of 5%, explained flexural capacity equal to 68 kN, which is higher than the ultimate capacity of beam C8B2.5-M135-1 (62.3) kN. In addition, Figure 4.57 demonstrates that beams with connectors explained lower stiffness than beams.
without connectors. That may be due to dissipation the vertical deformation (deflection) into relative slipping between the substrate concrete and TRM layer. The crack patterns of both beams (Figure 4.56) were similar except beam C8B2.5-M135 demonstrates a denser crack distribution in the mid-span of beams. That indicates to higher contribution of the TRM layer because of improving the interface bond strength. On other hands, higher ductility was observed of a repaired beam with connectors due to reaching higher ultimate deflection. It can be concluded that the application of connectors in repair corroded RC beams has similar effectiveness of strengthening RC beams when the level of corrosion is less than 5%.

4.6.3.8 Effect of textile fibre type of repaired RC beams with TRM

To address the effect of fibre type on the behaviour of repaired corroded RC beams, the behaviour of C8B2.5-M135-1 and C5C2.5-M135-2 were compared. Both beams had approximately the same degree of corrosion (5% and 4.9%). Despite that beam, C5C2.5-M135-2 has about twice axial tensile strength of C8B2.5-M135-1, the increase in ultimate strength was about 7% compared with beams strengthened with basalt fibres. That was due to the concrete cover separation that observed of beam C5C2.5-M135-2. In addition, higher stiffness (Figure 4.58) was observed of beam C5C2.5-M135-2 especially at the initial stage of loading. That may be attributed to dissipation of the energy from the applied load to create horizontal cracks in

Figure 4.57: Load deflection curves of beams C8B0-M135-1 and C8B2.5-M135-1
the concrete cover. Accordingly, beam C5C 2.5-M135-2 displayed less ductility than C8B2.5-M135-1.

The crack patterns of beam C5C2.5-M135-2 (Figure 4.54) shows, besides the flexural cracks near the mid-span of beam, horizontal cracks at the level of longitudinal reinforcement due to the corrosion effects. These cracks were joined with the inclined cracks at the end of strengthening layer and led to destroy the concrete cover. However, beam C8B2.5-M135-1 displayed flexural cracks at the mid-span of the beams and with load progression, one of the cracks under the applied load point became wider until rupturing of textile basalt fibres. On other hands, with a degree of corrosion of 4%, beam C5C2.5-M135-1 exhibited 79.1kN ultimate load, which demonstrates higher efficiency of the carbon fibres in the presence of a lower degree of corrosion.

It can be stated that textile fibres with high mechanical properties present high enhancement in repair corroded RC beams. However, high mechanical properties can increase the possibility of separation of the concrete cover especially when the corrosion level more than 4%.

![Figure 4.58: Load deflection curves of beams C8B2.5-M135-1 and C5C2.5-M135-2](image)

Figure 4.58: Load deflection curves of beams C8B2.5-M135-1 and C5C2.5-M135-2
4.6.4 Results of strengthened RC beams after corrosion

This section demonstrates the durability of strengthened RC beams under corrosion. The durability was assessed by comparing the performance of strengthened beams subjected to corrosion with strengthened beams without corrosion, to quantify the reduction in strength, stiffness and ductility. In addition, effect of corrosion level, fibre type and connectors ratio were also compared.

Similar to the to the RC beams that subjected to corrosion of repaired beams, strengthened beams exhibited a different level of corrosion despite all beams were subjected to the same intensity of the impressed current. That may be attributed to the contribution of shear reinforcement in corrosion. Figure 4.59 displays high rust of corrosion at the positions of shear reinforcement. That indicates initiation and propagation of corrosion in shear reinforcement.

![Figure 4.59: Typical corrosion of strengthened RC beams](image)

4.6.4.1 Flexural strength of strengthened RC beams after corrosion

Table 4.14 presents the flexural load, failure mode and the degree of corrosion of the strengthened RC beams after subjecting to corrosion. The corrosion results demonstrate a variation in the degree of corrosion among the tested beams ranged between 5.4% and 9.5%. However, these corrosion results were higher than RC beams subjected to the same corrosion conditions. That may be because of addition the TRM layer restricted the penetration of chloride ions to the longitudinal reinforcement from the bottom face of the beam. That led to increasing the penetration from the sides (where the shear reinforcement is closer to the external surface) and led to corrosion shear reinforcement. In addition, with
insufficient insulation between longitudinal and shear reinforcement, shear reinforcement corrosion increases the corrosion of longitudinal reinforcement.

Figure 4.60 demonstrate a reduction of the strength at all stage of loading for all strengthened beams under corrosion with respect to strengthened beams without corrosion. In addition, strength of corroded strengthened beams was less than repaired beams due to increasing the degree of corrosion. Both beams S8B0-M135 exhibited very close ultimate flexural load (55.2 kN and 53.5 kN, respectively) which could be attributed to the non-considerable difference in the degree of corrosion (5.5% and 5.8%). This amount of corrosion reduced the effectiveness of strengthening layer at ultimate load to about 15% and 16%. In addition, both beams failed due to yielding of steel reinforcement followed by debonding of the strengthening layer.

However, despite beam S8B0-M135-1 has the same corrosion (5.4%), its ultimate strength was 61.1kN which was due to increasing the interface bond strength (presence of the cementitious connectors) up to change the failure mode to textile fracture followed by concrete crushing at the compression face of the beam. Despite that improvement in interface bond strength, 10% reduction in ultimate strength was observed. On other hands, connectors demonstrate less effectiveness in the presence of 6.7% degree of corrosion of beam S8B0-M135-2. That was because of the separation of concrete cover at the level of longitudinal reinforcement due to the corrosion consequences. Accordingly, high reduction (34%) in ultimate strength was observed. Similar failure mode was observed of beams S5CB5-M135 due to the high degree of corrosion (6.5% and 9.5%). The reduction in ultimate strength was 24% and 32% of beams S5CB5-M135-1 (6.5% corrosion) and S5CB5-M135-2 (9.5% corrosion), respectively.

It is interesting to mention that for the same degree of corrosion (about 5.5%) the reduction in ultimate strength of strengthened RC beams is about twice the reduction of non-strengthened beams (control beams). However, it is worth to mention that the strengthened beams still have higher flexural resistance than the un-strengthened corroded beams. For higher degree of corrosion, additional procedures may need to be considered such as remove the damaged substrate concrete cover before strengthening. Accordingly, investigation of removing concrete cover and application the TRM on the sides of beams in addition to the bottom is recommended as an interesting future work.
<table>
<thead>
<tr>
<th>Beam</th>
<th>Load (kN)</th>
<th>Corrosion</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>First crack</td>
<td>Yield</td>
<td>Ultimate</td>
</tr>
<tr>
<td>COR-1</td>
<td>10.4</td>
<td>41</td>
<td>45.3</td>
</tr>
<tr>
<td>COR-2</td>
<td>11.7</td>
<td>42.6</td>
<td>47.1</td>
</tr>
<tr>
<td>S8B0-M135-1</td>
<td>19.0</td>
<td>50.1</td>
<td>55.2</td>
</tr>
<tr>
<td>S8B0-M135-2</td>
<td>18.1</td>
<td>48.1</td>
<td>53.5</td>
</tr>
<tr>
<td>S8B5-M135-1</td>
<td>21.8</td>
<td>53.0</td>
<td>61.1</td>
</tr>
<tr>
<td>S8B5-M135-2</td>
<td>15.6</td>
<td>40.0</td>
<td>44.9</td>
</tr>
<tr>
<td>S5C5-M135-1</td>
<td>21.6</td>
<td>52.1</td>
<td>62.8</td>
</tr>
<tr>
<td>S5C5-M135-2</td>
<td>18.2</td>
<td>48.7</td>
<td>56.3</td>
</tr>
</tbody>
</table>

DC is the degree of corrosion (%)

SY-CC is steel yielding followed by concrete crushing failure mode
TR-CC is textile rupture followed by concrete crushing failure mode
SY-DS is steel yielding followed by debonding of strengthening layer failure mode
SY-CS is steel yielding followed by concrete cover separation

Figure 4.60: Reduction in strength of strengthened RC beams after corrosion
4.6.4.2 Load deflection curves of strengthened RC beams after corrosion

Figure 4.61 (load deflection curves) demonstrates that, apart from beams S8B5-M135, beams with the same strengthening properties have identical load deflection response and the effect of degree of corrosion only obvious close to the ultimate load stage. However, beams S8B5-M135 displayed different load deflection behaviour due to the difference in the failure mode. S8B5-M135-2 had a concrete cover separation failure mode while beam S8B5-M135-1 presented a rupture of textile fibres followed by concrete crushing failure mode.

In addition, the figure also indicates to reduction in stiffness of the strengthened beams after subjecting to the corrosion comparing with strengthened beams without corrosion. The reduction in stiffness was proportional with the degree of corrosion. The highest reduction was observed of beams S5CB5-M135 that have 6.5% and 9.5% degree of corrosion.

![Figure 4.61: Load deflection curves of beams strengthened with and without corrosion](image)
4.6.4.3 Crack patterns of strengthened RC beams after corrosion

Figures 4.62-4.64 present the cracks patterns of strengthened RC beams after subjecting corrosion at the ultimate load stage. Besides the flexural cracks that differ in configuration depending on the degree of corrosion, all beams displaced horizontal crack at the level of longitudinal reinforcement. These horizontal cracks were initiated at the end of TRM layers during the test and propagated horizontally to join with the horizontal cracks at the mid-span of the beams.

Beam S8B0-M135-1 (Figure 4.62) exhibited a fewer number of flexural cracks and greater crack spacing than beam S8B0-M135-2. That was due to the peeling the strengthening layer of beam S8B0-M135-1 which reduced the contribution of TRM in gaining higher load which correspond to increase the cracks and decrease the space between them. In addition, it can be noted that most of the cracks were either initiated from the TRM layer and propagated vertically to stop at the horizontal crack or started from the level of horizontal crack and propagated vertically towards the compression zone of the beam. That may indicate losing the effective bond between the components of the substrate concrete at the level of longitudinal reinforcement due to consequences of corrosion.

However, Figure 4.63, the flexural cracks were initiated from the TRM layer and propagated vertically (crossing the horizontal crack) towards the compression face of beams S8B5-M135-1. With the progression of loading, one of flexural crack in the TRM layer became wider until rupture of textile fibre. Due to a higher degree of corrosion (6.7%), beam S8B5-M135-2 explained similar crack patterns to beams S8B0-M135 because of reducing the bond of concrete components at the level of longitudinal reinforcement.

Similar to beams S8B0-M135, the cracks of beams S5C5-M135-2 was initiated from the level of the longitudinal steel reinforcement and propagated towards the compression face of the beam. However, beam S8B0-M135-1 displayed wider horizontal crack at the level of longitudinal reinforcement and it was joined with a horizontal crack at the end of the TRM layer at the level of TRM/concrete.
Figure 4.62: Crack patterns of beam S8B0-M135

Figure 4.63: Crack patterns of beams S8B5-M135

Figure 4.64: Crack patterns of beam S5C5-M135
4.6.4.4 Ductility of strengthened RC beams after corrosion

Table 4.15 and Figure 4.65 demonstrate that the corrosion reduces the ductility of the strengthened RC beams with respect to strengthened RC beams without corrosion. The reduction ranged between 1% of beam S8B5-M135-2 and 17% S8B0-M135-2.

The highest reduction was observed of beams S8B0-M135 (16% and 17%) due to the debonding of the strengthening layer, which reduced the deflection at the ultimate load stage. Similarly, with less extent, the concrete cover separation of beams S5C5-M135 reduced the deflection ductility to 3% of beam with 6.5 % degree of corrosion and 6% of beam with 9.5% degree of corrosion. A negligible effect (less than 2%) was observed of the degree of corrosion ranged between 5.4% and 6.7% of beams S8B5-M135.

Table 4.15: Deflections and ductility index of strengthened RC beams after corrosion

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Dc (%)</th>
<th>Mid span deflection (mm)</th>
<th>Ductility index ($\mu_\Delta$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Yield</td>
<td>Ultimate</td>
</tr>
<tr>
<td>COR-1</td>
<td>7.3</td>
<td>6</td>
<td>9.5</td>
</tr>
<tr>
<td>COR-2</td>
<td>5.6</td>
<td>6.3</td>
<td>10.9</td>
</tr>
<tr>
<td>S8B0-M135-1</td>
<td>5.5</td>
<td>7.2</td>
<td>8.7</td>
</tr>
<tr>
<td>S8B0-M135-2</td>
<td>5.8</td>
<td>6.9</td>
<td>8.2</td>
</tr>
<tr>
<td>S8B5-M135-1</td>
<td>5.4</td>
<td>8.2</td>
<td>10.8</td>
</tr>
<tr>
<td>S8B5-M135-2</td>
<td>6.7</td>
<td>5.4</td>
<td>7.2</td>
</tr>
<tr>
<td>S5C5-M135-1</td>
<td>6.5</td>
<td>7.5</td>
<td>9.7</td>
</tr>
<tr>
<td>S5C5-M135-2</td>
<td>9.5</td>
<td>6.6</td>
<td>8.3</td>
</tr>
</tbody>
</table>

Figure 4.65: Reduction in ductility of strengthened RC beams after corrosion
4.6.4.5 Effect of connectors on strengthened RC beams after corrosion

By comparing the ultimate capacity of beam S8B0-M135-1 and S8B5-M135-1, which have the very close degree of corrosion (5.5% and 5.4%), it can be found that 5% connectors increase the capacity to about 11% compared with beams without connectors. That was due to increasing the interface bond strength, which led to change the failure mode from debonding of TRM (beam S8B0-M135-1) to textile rupture (S8B5-M135-1), (Figures 4.62 and 4.63). Figure 4.66 explains a reduction in stiffness in the presence of connectors, which may be attributed to dissipation the applied vertical deformation, resulted from the application of flexural loads, in the propagation of horizontal cracks at the interface, which reduce the vertical deflection of beam S8B5-M135-0. That was obvious from the higher ductility of beam S8B5-M135-1 (1.32) compared with beam S8B0-M135-1 (1.21).

The effect of interface bond strength was extended to the crack patterns, where beam S8B5-M135-1 displayed flexural cracks initiated from the TRM layer and propagated vertically towards the compression face of the beam. However, horizontal cracks were commenced at the end of the interface and propagated to join with the horizontal cracks at the level of longitudinal reinforcement of beam S8B0-M135-1. However, the effectiveness of connectors was seemed adversely affect the capacity of beam S8B5-M135-2 that had 6.7% degree of corrosion. That was because of the corrosion led to weakening the concrete at the level of longitudinal steel reinforcement, which in turns caused a concrete cover separation due to increase the tensile and shear stresses resulted from loading the TRM layer. On other hands, beam S8B0-M135-2 exhibited the same stiffness (load deflection curve of Figure 4.61) and ductility of beam S8B0-M135-1 that had less degree of corrosion (5.4%). That indicates the degree of corrosion does not affect, but it limited their contribution to the properties of the substrate concrete. Similar negative impact of the application connectors was evident of beams strengthened with 5 textile carbon fibres that have 6.5% and 9.5% degree of corrosion. In addition, the influence of connectors became more effective in reducing the strength of corroded beams as the corrosion increased.

Based on the obtained results it can be stated that the debonding and the relative slipping of the strengthening layer (less effective interface bond strength) may present higher stiffness by dissipation the vertical deformation to a horizontal direction. In addition, the application of connectors improves the behaviour of corroded strengthening RC beams with corrosion level less than 5.5%. However, above this level, the use of connectors may work adversely by reducing the ultimate capacity due to destroying the concrete cover.
Figure 4.66: Load deflection curves of beams S8B0-M135-1 and S8B5-M135-1

4.6.4.6 Effect of textile fibre type on strengthened RC beams after corrosion

To address the effect of textile fibre type on the behaviour of strengthened RC beams subjected to corrosion, the behaviour of beams S8B5-M135-2 and S5C5-M135-1 which have approximately the same degree of corrosion (6.7% and 6.5%) were compared. In addition, the results of the ultimate strength of beams S8B5-M135-2 and S5C5-M135-2 were compared to demonstrate the variation in effectiveness in the higher degree of corrosion (9.5%).

Despite both beams displayed concrete cover separation failure mode, beams strengthened with textile carbon fibres exhibited 1.4 times the ultimate load of beams strengthened with 8 layers of textile basalt fibres. That may be either due to the higher axial tensile strength \( (A_t f_t) \), which is about twice basalt fibres or due to the higher modulus of elasticity of carbon fibres. Since both beams displayed concrete cover separation at different level loading, hence the concrete separation may be more influenced by the stiffness of the TRM, which is controlled by the modulus of elasticity of textile fibres rather than the axial strength. To demonstrate that, due to the lower modulus of elasticity of textile fibres, it needs higher curvature (deformations) to explain their contribution in resisting loading which interns accelerate the concrete cover separation due to the brittleness properties of concrete.

However, Figure 4.67 demonstrates higher stiffness of beams strengthened with textile fibres than carbon fibres, which should be based on the above claim less than carbon fibres. That may be, as previously stated, due to the dissipation of the vertical deformation
(deflection) in the horizontal direction. On other hands, beam S8B5-M135-2 explained higher ductility than S5C5-M135-1, which may confirm the above assumption about the effect of modulus of elasticity on the strength.

Moreover, the superiority of textile carbon fibres can be noted from the ultimate strength of beam S5C5-M135-2 which was about 1.25 the ultimate strength of beam S8B5-M135-2, despite the degree of corrosion of beam S5C5-M135-2 was 1.42 the corrosion of beam S8B5-M135-2.

![Figure 4.67: Load deflection curves of beams S8B0-M135-1 and S8B5-M135-1](image)

4.7 Concluding remarks
The results of pull-off test demonstrated a linear relationship between the strength of the addition mortar and the adhesion bond strength with a rough concrete substrate. High strength mortar (M135), for example, explained about 2.81 times the adhesion of normal strength mortar (M35). In addition, the proposed equation to determine the adhesion bond strength explained relatively realistic values compared with other previous studies. Similar high bond properties of high strength mortar were observed through slant shear compression test by means of exhibiting monolithic failure passing through both parts of the sample. Based on the pull-off and slant shear test results, the adhesion bond can be considered as a property controlled by the characteristics of the overlay mortar and roughness of the substrate.

The results of strengthened RC beams explained that high strength mortar was more effective in enhancing the capacity of RC beams at all stages of loading. For beams strengthened with 8 basalt layer, the improvement in the ultimate load of mortar M35, M70
and M135 was 12%, 19% and 27%. In addition, the application of 5% of cementitious connector’s ratio increased the improvement in ultimate load to about 26% compared with beams without connectors of beams strengthened with eight layers of textile basalt fibres. Besides, these connectors changed the failure mode of the strengthened beams from debonding of TRM to rupturing of the textile fibres. The effectiveness of connectors was higher for beams strengthened with 16 basalt layers (30% higher enhancement with respect to beams without connectors). Similarly, with higher enhancement ratio (39%) was observed for beams strengthened with 5 layers of textile carbon fibres. However, increasing the connector’s ratio to 7.5% increased the improvement of beams with 5% connectors by 19%. That reduction in the effectiveness of connectors was due to the separation of the concrete cover. Hence, it can be stated that the substrate concrete strength significantly influences the effectiveness of connectors in improving the capacity of RC beams.

On other side, carbon fibres exhibited higher effectiveness than basalt fibres through enhancing the load to about 44% of 5 layers compared with 33% of 16 textile basalt fibres for beams without connectors. Similarly, in presence of 5% connectors’ ratio, the achieved enhancement of in ultimate load of 5 layers of carbon and 16 layers of basalt was 43% and 61%, respectively. Up to 6.1% degree of corrosion, all repaired corroded RC beams demonstrated higher ultimate strength than non-corroded RC beams. In addition, all repaired beams exhibited higher stiffness than corroded and non-corroded RC beams. That demonstrates the efficiency of the TRM layers in repair corroded RC beams. Moreover, the repaired corroded RC beams with basalt fibres indicated a negligible reduction in the strength compared with strengthened RC beams (without corrosion) when the degree of corrosion less than 5%. However, for the same degree of corrosion, beams repaired with 5 layers of textile carbon fibres explained a 12% reduction in ultimate strength compared with strengthened beams without corrosion. That reduction was due to the concrete cover separation because of the tensile stresses resulted from TRM layer.

For corroded strengthened RC beams with 5.5% degree of corrosion, the reduction in ultimate strength of strengthened RC beams is about twice the reduction of non-strengthened beams (control beams) compared with beams without corrosion. That was because of the separation of concrete cover at the level of longitudinal reinforcement due to tensile stresses resulted from the strengthening layer. For instance, strengthened beams with 8 layers of textile basalt fibres exhibited 34% reduction in ultimate strength compared with non-corrode strengthened beam when the degree of corrosion was 6.7%.
Chapter Five

Numerical Investigation of RC Beams Strengthened with TRM

5.1 Introduction

This chapter details the development of a numerical model to simulate the flexural behaviour of RC beams strengthened with TRM layer. Due to the absence of a valid analytical model that can be used to predict the behaviour of RC beams strengthened with TRM, a Finite element method (FEM) using ATENA software (Cervenka et al., 2014) was carried out. The main purpose of this investigation was to further study of the performance of strengthened beams with emphases on the aspects were difficult to achieve experimentally, such as the stresses and strains distribution of concrete, steel, textile fibres and interface of the strengthening layer. In addition, parametric studies were carried out to address the effect of additional vital parameters has not been investigated experimentally due to the time, cost and equipment aspects.

ATENA software was adopted in the numerical investigation because of previous researchers indicated its efficiency (i.e. good agreement with the experimental behaviour was achieved) in the numerical modelling of TRM and strengthened RC structures. For instance, Larrinaga et al., (2014) used ATENA to simulate the behaviour of TRM layers under tensile stresses. In the field of strengthening RC beams, ATENA used by Hashemi and Al-Mahaidi (2012) (strengthened beams with TRM) and Lampropoulos et al., (2016) (strengthening of RC beams with UHPFRC).

In this study, the numerical investigation is divided into three stages; modelling, validation and parametric studies. Modelling stage included modelling of finite elements, materials, boundary conditions, loading, and solutions of nonlinear equations. The comparison between the experimental and numerical behaviour of control and strengthened RC beams was implemented in the validation stage. After proving the validation of the proposed model, parametric studies that include the effect of concrete strength, reinforcement ratio of longitudinal steel rebars, amount of textile fibres and the ratio of cementitious connectors were studied.
5.2 Numerical Modelling

5.2.1 Finite elements

Three types of elements were adopted in the numerical investigation; 3D solid element, Truss element and interface element. A 3D solid element was used to simulate the cementitious materials (concrete and mortar). The truss element was used to model the steel reinforcement and textile fibres, and the interface between the old concrete and new strengthening layer was modelled using the interface element.

5.2.1.1 3D solid element

Three-dimension brick elements with 8 nodes were adopted to model the concrete and the mortar. The element has three degrees of freedom at each node and translation in x, y and z directions. It can simulate the nonlinear properties of concrete such as cracking, crushing and plastic deformation. This element adopted by many researchers in FEA of concrete (Lampropoulos et al. 2016, Jawdhari and Harik 2018, Banjara and Ramanjaneyulu 2017, Hashemi and Al-Mahaidi 2012 and Mohamed et al. 2017). The element is isoperimetric element integrated by using Gauss integration at the integration points. Figure 5.1 shows the geometry of the adopted 3D brick element.

Figure 5.1: Geometry of 8-nodes brick element (Cervenka et al., 2014)
5.2.1.2 Truss element

Two-dimensional linear isoperimetric truss element (Figure 5.2) was used to model the steel reinforcement and textile fibres. This element was defined using two nodes and integrated by Gauss integration at 1 or 2 integration points. The element has three translational degrees of freedom at each node. Many previous studies adopted truss element to simulate the steel reinforcement (Lampropoulos et al. 2016, Jawdhari and Harik 2018, Banjara and Ramanjaneyulu 2017, Hashemi and Al-Mahaidi 2012 and Mohamed et al. 2017).

![Figure 5.2: Geometry of 2-nodes truss element (Cervenka et al. 2014)](image)

5.2.3.3 Interface element

Interface elements were used to simulate the contact between two different surfaces, which represent in this study the interface between the substrate concrete and the mortar of the TRM strengthening layer. Three-dimension gap nonlinear geometry elements were used. The interface is defined surfaces located on the opposite side of the interface and before the initiation of slip, the interface surfaces can share the same position. Figure 5.3 shows the geometry of the adopted interface gap element.

![Figure 5.3: Geometry of the interface gap element (Cervenka et al. 2014)](image)
5.2.2 Material modelling

The reliability of FEA in predicting the mechanical characteristics of strengthened RC member highly depends on the adoption of models of the material behaviour. Therefore, it is essential to establish suitable models to simulate the behaviour of composite member materials. This section presents the adopted numerical behaviour of concrete, mortar, steel and textile reinforcement under different states of stresses.

5.2.2.1 Cementitious materials (Concrete and Mortar)

ATENA offers a broad range of constitutive behaviours for quasi-brittle materials. In this study, the nonlinear response of concrete and mortar was simulated as CC3DNOnLinCementitious2, which is based on a fracture-plastic model. This model represents a combination of the constitutive model for tensile (fracturing) and compressive plastic behaviour. The fracture model is based on the orthotropic smeared crack formulation and cracks band model. It employs Rankine failure criterion, exponential softening, and it can be used as a rotate and fixed crack model (Cervenka et al. 2014). The details of the adopted model are presented in the next subsections.

The mechanical properties of concrete and mortar required for numerical modelling were obtained from the experimental investigation. Table 5.1 presents the adopted mechanical properties of concrete and mortar. Poisson’s ratio was taken equal to 0.2 which is recommended by CEB-FIP Model Code 90.

<table>
<thead>
<tr>
<th>Cementitious material</th>
<th>$f_{cm}$ (MPa)</th>
<th>$f_{ctm}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>31</td>
<td>2.22</td>
</tr>
<tr>
<td>M35</td>
<td>34.4</td>
<td>2.6</td>
</tr>
<tr>
<td>M70</td>
<td>69.1</td>
<td>3.71</td>
</tr>
<tr>
<td>M135</td>
<td>135.2</td>
<td>6.2</td>
</tr>
</tbody>
</table>

Table 5.1: Cementitious materials properties used in the numerical analysis
5.2.2.1.1 Modelling of cementitious material in tension

The behaviour of the non-cracked cementitious materials in tension was assumed linear elastic up to reaching the ultimate tensile strength of the cementitious materials. The effective tensile stress ($\sigma_t$) can be determined based on the following equation (Cervenka et al., 2014):

$$\sigma_t = E_c \times \varepsilon \quad 0 \leq \sigma_c \leq f_{ctm} \quad (5.1)$$

Where: $\varepsilon$ is the tensile strain of the cementitious material, $f_{ctm}$ is the tensile strength of the cementitious material obtained from the experimental tests (Table 5.1) and $E_o$ is the initial modulus of elasticity of the cementitious materials, which determined according to the ACI-318 (2011) by the following equation:

$$E_o = 4730\sqrt{f'_c} \quad (5.2)$$

Where $f'_c$ is the compressive strength in MPa.

The crack initiation can be divided into three stages. Figure 5.4 shows the adopted modelling of the crack stages. The first stage represents the uncracked stage before reaching the tensile strength of the cementitious material. Then, the process zone (stage two) of a potential cracks with decreasing tensile stress on the crack face due to bridging effects. Third stage represents the cracked stage when the crack opening continues without stress.

![Figure 5.4: Stages of crack opening of concrete and mortar (Cervenka et al., 2014)](image)

After cracking, a fictitious crack model based on a crack-opening law, and fracture energy was adopted to model the crack propagation in the cementitious materials. An exponential
crack opening law was carried out to describe the function of the crack opening regarding tensile strength and fracture energy of the cementitious materials. Zhang and Teng (2014) presented the validity of using the exponential model of crack opening. Figure 5.5 shows the curve of the adopted crack opening law.

![Exponential crack opening law of concrete and mortar (Zhang and Teng 2014)](image)

Hordijk (1991) experimentally derives the experimental expression of this law; it can be presented by the following equations;

\[
\frac{\sigma_t}{f_{ctm}} = \left\{1 + \left(3 \frac{w}{w_c}\right)^3\right\} \exp\left(-6.93 \frac{w}{w_c}\right) - 10 \frac{w}{w_c} \exp(-6.93) \tag{5.3}
\]

\[
w_c = 5.14 \frac{G_f}{f_{ctm}} \tag{5.4}
\]

Where \(w\) is the crack opening, \(w_c\) is the crack opening at the complete release of stress, \(\sigma\) is the normal stress in the crack and \(G_f\) is the fracture energy needed to create a unit area of the stress-free crack, which can be calculated according to the following equation (proposed by CEB-FIP Model Code90).

\[
G_f = (0.0469D_a^2 - 0.5D_a + 26) \left(\frac{f_s}{10}\right)^{0.7} \tag{5.5}
\]

Where \(D_a\) is the maximum aggregate size in mm.

Rotated crack model was adopted to model the cracks in the cementitious materials. In this model, the crack is formed when the principal stress exceeds the tensile strength and it is assumed that the cracks are uniformly distributed within the material volume.
5.2.2.3 Modelling of cementitious material in compression

The behavior of the cementitious materials was modelled base on the equation that recommended by CEB-FIP Model Code 90, which enables wide range of curve forms and is appropriate for most of concrete strengths (shown in Figure 5.6). Hashemi and Al-Mahaidi (2012) adopted this model and good agreement was achieved with the experimental behaviour.

\[
\sigma_c = f_c \times \frac{kx-x^2}{1+(k-2)x} \tag{5.6}
\]

\[
x = \frac{\varepsilon}{\varepsilon_c} \tag{5.7}
\]

\[
k = \frac{\varepsilon_0}{\varepsilon_c} \tag{5.8}
\]

Where: \(\sigma_c\) is the concrete compressive stress, \(f_c\) is the concrete compressive strength, \(x\) is the normalized strain, \(\varepsilon\) is the strain, \(\varepsilon_c\) is the strain at peak stress, \(k\) is plasticity parameter and \(E_c\) is the secant elastic modulus at the peak stress \((E_c = \frac{\sigma_0}{\varepsilon_0})\). According to CEB-FIP Model Code 90, the ultimate strain of the cementitious materials was assumed equal to 0.003. After that strain, the concrete fails by compressive crushing and the stress drops gradually to zero.

Figure 5.6: Compressive stress strain diagram (Cervenka et al. 2014 and CEB-FIP Model Code 90)
After cracking, the softening part in compression is linearly descending, as shown in Figure 5.7. Fictitious compression plane model was adopted to simulate the behaviour of compression after cracking. This model assumes that compression failure is localized in a plane normal to the direction of compressive principal stress. In addition, it is assumed that the compressive displacement and energy dissipation are localized which means it is independent of the size of the structure. That gives advantages of this model through reducing the dependency of results on the finite element mesh. Figure 5.7 shows the softening displacement law in compression.

![Figure 5.7: Softening displacement law in compression (Cervenka et al. 2014)](image)

The energy required to generate a unite area of the failure plane is indirectly defined by means of the plastic displacement $w_d$. From experiments of Van Mier (1986), $w_d$ is consider equal to 0.5 mm for normal strength concrete. In addition, CEB-FIP Model Code 90 recommends this value (0.5 mm) regardless to the concrete strength. In this study, 0.5 mm was adopted for all strengthens of cementitious materials because it was found a negligible effect on the behaviour with different values. The softening law is transformed from a fictitious failure plane (Figure 5.5) to the stress strain relation valid for the corresponding volume of continuous material, Figure 5.6. The slop of the softening part is defined by the maximum stress and the compressive strain $\varepsilon_d$ at the zero stress. This strain can be determined by using the following expression:

$$\varepsilon_d = \varepsilon_c + \frac{w_d}{l_d} \quad \text{(5.9)}$$

Where: $l_d$ is the band size of cracking

After cracking, the compressive strength was reduced in the direction parallel to the cracks. The reduced compressive strength is governed by the following equation (Cervenka et al. 2014):

$$f_{c}^{\text{ef}} = f_{c} - \frac{f_{c}}{2} \quad \text{(5.10)}$$
\[ f_c = c + (1 - c)e^{-(128\varepsilon_o)^2} \]  \hspace{1cm} (5.10)

There is strength reduction for zero normal strain and for large strains, the strength is approaching the minimum value \( f_c = c f'_c \). The constant \( c \) represents the maximal strength reduction under the large transverse strain. The value \( c = 0.45 \) was derived for the concrete reinforced with the fine mesh. The other researchers found the reductions not less than \( c = 0.8 \). The value of \( c \) can be adjusted by input data according to the actual type of reinforcing. After some trials, the value that can present the nearest behaviour to the experimental was 0.5.

### 5.2.1.2 Steel reinforcement rebars

Longitudinal and shear steel reinforcement rebars were modelled as discrete reinforcement using truss elements. In this model, a state of uniaxial stress is assumed. Bilinear law with hardening is adopted to simulate the stress strain diagram of steel reinforcement, as shown in Figure 5.8. The linear part of the stress strain diagram is controlled by the yield strength of the steel rebars and the elastic modulus of steel \( E_s \). The second linear part represents the plasticity of steel rebars with hardening modulus \( E_{sh} \). For perfect plasticity, \( E_{sh} \) is equal to zero and the limit strain \( (\varepsilon_{lim}) \) represents the limited ductility of steel. The ultimate stress in case of hardening is controlled by the ultimate tensile strength of steel reinforcement \( (\sigma_t) \). The adopted material properties of steel reinforcement were presented in Table 4.1 Section 4.3.1. The reinforcement rebars was modelled to exhibit the same behavior in tension as well in compression.

![Figure 5.8: The stress-strain law of steel reinforcement (Cervenka et al., 2014)](image-url)
The bond between the concrete and steel rebars was assumed perfect. Many researchers (Jawdari and Harik 2018, Banjara and Ramanjaneyulu 2017, Hashemi and Al-Mahaidi, 2012, Mohamed et al., 2017) were proved the validity of this assumption in modelling steel reinforcement of RC beams.

### 5.2.1.3 Textile fibres

Textile basalt and carbon fibres were modelled as discrete reinforcement, similarly to the steel reinforcement. The stress-strain relationships of fibres were modelled as linear elastic based on the obtained results of the direct tensile test of textile fibres (Section 3.2.5, Figures 3.7 and 3.8). The stress-strain curves were characterised by the ultimate tensile strength, modulus of elasticity and ultimate strain at failure.

The bond between fibres and surrounding mortar was assumed as perfect bond. This assumption was adopted because of the absence of valid bond-slip model of textile fibres. In addition, most of the available literature adopted perfect bond simulation of the TRM (Ombre 2011, Larrinaga et al. 2014, Huang et al. 2017 and Sneed et al. 2017).

Moreover, based on the literature review, the debonding of the strengthening layer may exhibit a higher bond between the textile fibres and mortar than concrete and TRM composite. That means the debonding of the strengthening layer occurs before reaching the ultimate bond strength between textile fibres and mortar which make validation of assumption of the perfect bond.

### 5.2.2 Interface between concrete and TRM layer

Interface model, proposed by Cervenka et al. 2014, was adopted to simulate the behaviour of the interface between the substrate concrete and the strengthening TRM layers. This model is based on Mohr-Coulomb criterion with tension cut off. The constitutive relation for a general three-dimensional case is given in terms of tractions on interface planes and relative sliding and opening displacements. Lampropoulos et al. (2016) demonstrated the effectiveness of this model in simulating the interface between RC and UHPFRC.

The model is defined by the following equations (Cervenka et al. 2014):

\[ |\tau| \leq c - \sigma \cdot \mu \quad , \quad \sigma \leq 0 \]  

(5.12)
\[ \tau = \tau_0 \sqrt{1 - \frac{(\sigma - \sigma_c)^2}{(f_t - \sigma_c)^2}} \quad 0 \leq \sigma \leq f_t \quad (5.13) \]

\[ \tau_0 = \frac{c}{\sqrt{1 - \frac{\sigma_c^2}{(f_t - \sigma_c)^2}}} \quad (5.14) \]

\[ \sigma_c = \frac{f_t^2 \mu}{c - 2f_t \mu} \quad (5.15) \]

The coefficients of the interface model \((C, f_t, \mu)\) were determined from the bond tests under direct tension and under combined shear and compression (Section 4.6.1). Table 5.2 presents the adopted values for each mortar strength. Lampropoulos et al. (2016) found that the friction coefficient \(C\) and \(\mu\) equal to 1.9MPa and 1.5 were adequate in simulation the interface bond for rough substrate concrete surface strengthened with UHPFRC. These values are nearly the same for mortar M135 of this study.

<table>
<thead>
<tr>
<th>Mortar</th>
<th>(C) (MPa)</th>
<th>(\mu)</th>
<th>(f_t) (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M35</td>
<td>0.74</td>
<td>1.58</td>
<td>0.74</td>
</tr>
<tr>
<td>M70</td>
<td>1.15</td>
<td>1.58</td>
<td>1.15</td>
</tr>
<tr>
<td>M135</td>
<td>1.9</td>
<td>1.58</td>
<td>1.9</td>
</tr>
</tbody>
</table>

5.2.4 Boundary conditions

Displacement boundary conditions are required to constrain the member to ensure obtain the experimental behaviour of the tested beams. Due to geometrical and loading symmetry, only quarter of the specimens were considered for the analysis. To satisfy the symmetry requirements, the movement in the direction perpendicular to the plane of symmetry was restrained.

The roller support near the end of the beam was modelled as a single line of nodes with restriction the displacement in a vertical direction. Steel plates were used at support to avoid stress concentration in concrete. Figure 5.9 shows the adopted boundary condition of the investigated RC beams.
5.2.4 Loading

The load was applied using a displacement-based control approach. This approach can track the drops in loading magnitudes that occur due to local damage such as crushing or cracking of concrete and debonding of the strengthening layer. The displacement was applied at the position of the point load in a vertical direction (z-direction). Similar to supports, steel plates were used at applied load to avoid stress concentration. Figure 5.10 shows the application of the prescribed deformation. Two different values of displacement were carried out 0.1mm and 0.05mm and the results (Figure 5.11) demonstrated identical behaviour up to ultimate load, then a negligible difference was observed. Therefore, 0.1 mm displacement was adopted for the numerical analysis.
5.2.5 Cementitious connectors

The circular cross-section of the cementitious connectors was converted to the equivalent square section to avoid instability of the finite element analysis due to the difficulty of connecting the element nodes between the circular lines and straight lines. Therefore, the circular connectors replaced by an equivalent area of square connectors as shown in Figure 5.12.

![Figure 5.12: Numerical simulation of cementitious connectors](image)
5.2.6 Solutions of nonlinear equations

Three methods can be used to solve the nonlinear structural equations. These are iterative technique, incremental technique and incremental iterative technique. The incremental iterative method is considered the most common technique used for nonlinear finite element solution. Based on this method, the applied displacement represents the incremental part, and within each increment, some iterations are implemented to satisfy the equilibrium equation. In this study, regular and modified Newton-Raphson method were examined for the nonlinear solution. Figure 5.13 demonstrated identical behaviour of both methods, hence regular Newton-Raphson method was adopted.

![Figure 5.13: Effect of solution method on the load deflection response](image)

5.2.7 Element mesh sensitivity

Element mesh size affects the accuracy of the numerical results and the required computing time. Finite element model with fine mesh size can present highly accurate results but may take long computing time and, in some cases, very fine mesh exhibited unstable analysis due to localized the stresses at the applied loading region. However, coarse mesh size explains less accurate results and reduced computing time. That depends on the type of structure and loading and boundary condition. The effect of element meshes size was examined to establish the best simulation of the geometry, which can exhibit the nearest behaviour to the experimental.

Figure 5.14 illustrates five trail mesh sizes (20, 25, 30, 40 and 50 mm) that conducted to specify the best element mesh size. Based on the results, Figure 5.15, the differences in prediction the ultimate load among the conducted meshes were about 1%. However, the mesh size of 25 mm explained the nearest deflection to the experimental. Therefore, this
mesh is adopted for the numerical analysis of all RC beams. Banjara and Ramanjaneyulu (2017) found that 25 mm element mesh size is efficient in the numerical simulation of strengthened RC beams with GFRP.

Figure 5.14: Investigated element mesh sizes

Figure 5.15: a. load deflection curves and b. variation in load and deflection of control RC beam with different element mesh sizes
5.3 Validation of the proposed numerical model

5.3.1 Control RC beam

RC beam without strengthening layer that serves as control beam was analysed numerically and the results were compared by means of load and deflection at ultimate capacity and the load deflection response of experimental data. In addition, the crack patterns, stresses in concrete steel reinforcement at ultimate load were also compared with the mechanical properties of the concrete and steel rebars.

5.3.1.1 Load deflection curve of control beam

Figure 5.16 demonstrates the adopted numerical load-deflection curve (mesh size 25mm) compared with the experimental behaviour of control beam. As shown from the figure, the numerical load-deflection response explained higher stiffness than the experimental. That may be due to the cracks that created in the RC beams because of shrinkage especially all RC beams were cured in dry conditions.

The average amount of experimental deflection at the ultimate load was 9.8 mm while the numerical value was 9.6 mm. The obtained numerical ultimate capacity was 49.7kN. Both values are considered very close with the average ultimate capacity of the experimental investigation (ultimate load 51kN).

![Figure 5.16: Experimental and numerical load deflection curve of control beam](image-url)
5.3.1.2 Crack patterns of control beams

Figure 5.17 demonstrates the cracks distribution of the RC control beam at the ultimate load capacity. The crack patterns explained similar configuration to the experimental counterpart. However, higher intensity was observed in the numerical analysis and that was due to the sensitivity of the analysis in prediction the cracks when the tensile stresses in concrete exceed the tensile strength. In addition, most of these cracks are difficult to observed experimentally. Flexural cracks were initially started at tension face at the mid-span of the beam and propagated towards the compression zone of the RC beam. In addition, the width of cracks at the mid-span is decreased as the cracks remote from the mid-span. Moreover, continuous vertical cracks at the mid-span explain higher width at tension face than near compression region, which is identical with the experimental. In addition, it can be observed that there are horizontal cracks near the compression zone, which refers to the crushing of concrete when the ultimate compressive strength is reached at that region. The inclined (shear cracks) refer to exceeding the shear stresses to the shear strength of concrete and shear is transmitted by shear steel reinforcement.

5.3.1.3 Stresses in concrete and steel reinforcement

Figures 5.18 demonstrates the stress distribution in concrete and steel at ultimate load stage. The compressive stresses, at the compression face, were 26.8MPa, which is close to the compressive strength of concrete (31MPa).

Since the RC beam is under reinforced, the behaviour of beam is significantly controlled by the mechanical properties of the steel reinforcement. The failure of the beam was due to reaching the yielding stress of steel reinforcement (597MPa). This value is identical with the experimental yield strength of steel reinforcement (590MPa). However,
the ultimate stresses in shear reinforcement were about 60 MPa, which are less than the yield strength of shear reinforcement (480MPa).

![Stresses in concrete (MPa)](image1)

![Stresses in steel reinforcement rebars (MPa)](image2)

Figure 5.18: Stresses in concrete and steel reinforcement at ultimate load

### 5.3.2 Strengthened RC beams

Finite element analysis has been conducted to demonstrate the validation of the proposed models of simulation textile fibres and the proposed coefficients for the bond model between substrate concrete and TRM layer. The validation was examined through comparing the experimental ultimate load, deflection and load-deflection curve with the obtained numerical results. In addition, within this comparison, the stress distribution of steel reinforcement and textile fibres were studied for a deeper understanding of the influence of the TRM layer properties on the behaviour of RC beams strengthened with TRM layer. In addition, to study the efficiency of the proposed improvement in the strengthening technique by comparing the stresses in the textile fibres with the ultimate tensile strength.

Table 5.3 demonstrates a good agreement between the numerical and the experimental results. The maximum error in ultimate load capacity was about 9% for beams strengthened with a high amount of fibres in the presence of cementitious connectors. That indicated the validity of the proposed model in prediction the flexural behaviour of RC beams strengthened with TRM in presence of cementitious connectors.
Table 5.3: Experimental and numerical results of load and deflection of strengthened RC beams

<table>
<thead>
<tr>
<th>Beam</th>
<th>$\Delta u_{exp.}$ mm</th>
<th>$P_{exp.}$ kN</th>
<th>$\Delta u_{num.}$ mm</th>
<th>$P_{num.}$ kN</th>
<th>$\frac{\Delta u_{num.}}{\Delta u_{exp.}}$</th>
<th>$\frac{P_{num.}}{P_{exp.}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>8B0-M35</td>
<td>11.3</td>
<td>57.1</td>
<td>10.7</td>
<td>59.3</td>
<td>0.94</td>
<td>1.04</td>
</tr>
<tr>
<td>8B0-M70</td>
<td>10.9</td>
<td>60.5</td>
<td>11.3</td>
<td>60.6</td>
<td>1.04</td>
<td>1.00</td>
</tr>
<tr>
<td>8B0-M135</td>
<td>11.4</td>
<td>64.7</td>
<td>12.0</td>
<td>64.8</td>
<td>1.05</td>
<td>1.00</td>
</tr>
<tr>
<td>16B0-M135</td>
<td>9.7</td>
<td>67.9</td>
<td>9.9</td>
<td>68.9</td>
<td>1.02</td>
<td>1.01</td>
</tr>
<tr>
<td>8B5-M135</td>
<td>10.2</td>
<td>68.2</td>
<td>9.9</td>
<td>68.9</td>
<td>0.97</td>
<td>1.01</td>
</tr>
<tr>
<td>16B5-M135</td>
<td>13.0</td>
<td>72.8</td>
<td>12.6</td>
<td>79.7</td>
<td>0.97</td>
<td>1.09</td>
</tr>
<tr>
<td>16B7.5-M135</td>
<td>12.2</td>
<td>77.0</td>
<td>11.9</td>
<td>79.6</td>
<td>0.98</td>
<td>1.03</td>
</tr>
<tr>
<td>5C0-M135</td>
<td>11.1</td>
<td>73.3</td>
<td>10.5</td>
<td>72.1</td>
<td>0.95</td>
<td>0.98</td>
</tr>
<tr>
<td>5C5-M135</td>
<td>12.6</td>
<td>82.3</td>
<td>11.9</td>
<td>89.1</td>
<td>0.94</td>
<td>1.08</td>
</tr>
</tbody>
</table>

$\Delta u_{exp.}$ : is the experimental deflection at the ultimate load.

$P_{exp.}$ : is the experimental ultimate load.

$\Delta u_{num.}$ : is the numerical deflection at the ultimate load.

$P_{num.}$ : is the numerical ultimate load.

5.3.2.1 Load – deflection behaviour

Figures 5.19 and 5.20 demonstrate a good agreement between the predicted and experimental load deflection response. However, higher stiffness was observed of all analysed beams compared with the experimental.

The numerical load-deflection curves of all beams without connectors demonstrated a gradual descending response after reaching the ultimate capacity. That refers to the debonding or partial debonding of the strengthening layer and the failure is controlled by the interface bond strength. However, only beam 8B0-M135 explained a sudden descending in the deflection response after reaching the ultimate capacity.

The load-deflection curves of strengthening RC beams with cementitious connectors explained higher stiffness than experimental curves (see Figure 5.20). Apart from beam 8B5-M135, all beams explained a higher difference in numerical and experimental load-deflection response. That may be due to the effect of the application of connectors, where in practice, drilling the substrate RC beams produce unavoidable cracks which may reduce the strength of the concrete cover, and lead to decrease the stiffness of the strengthened beams. In addition, all beams explained sharply descending in load-deflection curves after reaching the ultimate capacity which refer to rupturing of the textile fibres.
Figure 5.19: Load - deflection curves of strengthened RC beams without connectors
5.3.2.2 Stresses of textile fibres and steel reinforcement

For strengthened beams without connectors, textile fibres and longitudinal steel reinforcement exhibited different values of the tensile stresses depending on the type of mortar and amount and type of textile fibres, as shown in Figures 5.21 and 5.22.

At ultimate load, for all strengthened RC beams, the longitudinal reinforcement reached the yield strength (590MPa) while the maximum stresses in the shear reinforcement were about 240MPa. For beam 8B0-M35 the maximum stresses in textile basalt fibres were about 420MPa while longitudinal steel reinforcement reached the yield point which indicates to the debonding of the TRM layer and the tensile stresses only carried out by steel reinforcement. However, the enhancement in the capacity of the strengthened beam was due to the contribution of the TRM in the initial stages of loading. Similar behaviour
was observed of beam 8B0-M70, but higher tensile stresses in the textile basalt fibres were observed due to enhancing the interface bond strength.

Using high strength mortar (M135) improves the contribution of textile basalt fibres in increasing the ultimate capacity by increasing the transferred stresses from the substrate concrete to the strengthening layer. The stresses in textile fibres of beam 8B0-M135 exceeded the yield strength of steel to reach about 637MPa. However, this level of stress is lower than the tensile capacity of the textile basalt fibres (730MPa). That demonstrates despite the enhancement of the interface bond strength due to using high strength mortar, that bond insufficient to reach the tensile capacity of the textile fibres which indicated to partial debonding. Similarly, the tensile stresses of carbon fibres of beam 5C0-M135 were lower than the ultimate tensile capacity. That refers to slip between the TRM strengthening layer and substrate RC beam and more improvement to the interface bond strength is required.

Despite using HSM as a matrix of TRM layer, the tensile stresses of textile fibres of beam 16B0-M135 were less than the yield strength of steel reinforcement. That was due to the debonding of the strengthening layer from the substrate RC beam.

The effect of connectors on the stresses of textile and steel reinforcement at the ultimate load was evident. The textile fibres reached the ultimate tensile capacity of beam 8B5-M135, which refers to effective bond strength up to rupturing the textile fibres. However, the same connector ratio failed to make the textile fibres reaching the ultimate tensile capacity of beams with 16 textile basalt fibres. In addition, even with increasing the connectors’ ratio to about 50% (from 5% to 7.5%), the tensile stresses in textile fibres at ultimate loading was about 648MPa. That was due to separation of the concrete cover of the substrate RC beams. Similarly, the tensile stresses of textile carbon fibres of beam 5C5-M135 were about 1130MPa while the ultimate carbon capacity is 1200MPa.

Generally, the numerical analysis of the strengthened beams demonstrates the effectiveness of the using high strength mortar and cementitious connectors in increasing the interface bond strength, which leads to higher contribution of the textile fibres in resisting the applied loads.
• All stresses values are in MPa

Figure 5.21: Stresses distribution in steel rebars and textile fibres

Figure 5.22: Maximum stresses in textile fibres at ultimate load
5.3.2.3 Normal stresses in concrete at the level of steel reinforcement

The normal stresses (tensile stresses) in concrete at the level of steel reinforcement were specified to demonstrate the observed experimental separation of concrete cover. Figure 5.23 illustrates the distribution of the normal stresses in concrete at the level of steel reinforcement. The numerical behaviour agrees with the experimental results of beams exhibited concrete cover separation. The stress distribution of the strengthened beams explained that as the interface bond strength increases the tensile stresses in concrete increases. For instance, for the same amount and type of textile fibres, the stress in concrete of beams 8B0-M35, 8B0-M70 and 8B0-M135 was about 0.53, 0.77 and 0.95MPa. However, in case of beam 16B0-M135, the stresses were about 1.12 MPa because of debonding the TRM strengthening layer. The same was right of beam 5C0-M135; the tensile stresses were about 1.76 MPa. These values are lower than the tensile strength of substrate concrete (2.22MPa) and therefore no concrete cover separation was observed.

The application of connectors increases the stresses significantly in concrete at the level of reinforcement because of increasing the interface bond strength which in turns increase the transferred stresses from the strengthening layer. The tensile stresses of concrete were increased to about 2.5 MPa of beam 8B5-M135 in localised areas near the cementitious connectors. However, RC beams strengthened with 16 textile basalt layers exhibited tensile stresses in concrete more than 6.0 MPa. This value was more than the tensile strength of substrate concrete, which led to the separation of the concrete cover. Similarly, with less extent was found for beam 5C5-M135. It can be found that as the transferred stresses from the strengthening layer increases, the tensile stresses in substrate concrete increases, which indicate the effect of the substrate concrete strength on the performance of the strengthened member.
5.3.2.4 Interface behaviour

Figure 5.24 demonstrates that strengthened beams without cementitious connectors exhibited horizontal displacements (slip) higher than vertical displacements (delamination). However, the opposite was observed for strengthened beams with cementitious connectors. This indicates the contribution of the connectors in reducing the horizontal displacements of the strengthening TRM layer. For both cases the maximum horizontal and vertical displacements were found at the end of the strengthening layer.

Figure 5.25 explained that the amount of vertical and horizontal displacements of the interface depends on the ultimate capacity of the strengthened beams which in turns controlled by the effectiveness of bond. For instance, despite that the higher interface bond strength of beam 16B0-M135 than 8B0-M35, the vertical and horizontal displacements of the later was lesser. On other hand, the ultimate load of beam 16B0-M135 was about 1.16 the ultimate load of beam 8B0-M35. Moreover, both beams exhibited the same
delamination of the strengthening layer at ultimate load. The inclusion of cementitious connectors significantly reduced the horizontal displacements of the strengthening layer. The maximum effect was observed for beams 16B5-M135 and 5C5-M135 by reducing the slip to about 83% and 71%, respectively.

Based on the numerical results, it can be stated that the physical (using connectors) and chemical (using high strength mortar) improvements increase the enhancement of the TRM strengthening layer up to reaching close stress to the ultimate capacity of the textile fibres, but it cannot prevent the relative displacements of the strengthening layer.
Figure 5.24: Normal and shear displacements in mm of the interface of RC beams

Figure 5.25: Maximum displacements of the interface of strengthened RC beams
5.4 Parametric study

After proving the validation of the proposed numerical simulation, the effect of different parameters that have not been investigated experimentally due to time, cost and space limitations were numerically studied. These parameters included; concrete strength, reinforcement ratio of longitudinal steel rebars, amounts of textile fibres and the ratio of cementitious connectors.

5.4.1 Effect of substrate concrete strength

Based on the experimental and numerical observations, the substrate concrete strength has a significant effect on the behaviour of strengthened RC beams with high textile reinforcement ratio such as 0.032. Beams strengthened with this ratio explained a separation of concrete cover in the presence of cementitious connectors. Therefore, beam 16B7.5-M135 was investigated numerically with using concrete compressive strength of 60MPa (C60) to study the effect of concrete cover strength on the behaviour of the TRM strengthening layer. In addition, the same textile basalt reinforcement ratio but with 0 and 5% connectors ratios were analysed to demonstrate the effect of concrete strength with different connectors ratios.

Moreover, for comparison with textile carbon fibres, beams 5C0-M135 and 5C5-M135 were numerically analysed with using concrete strength C60. Beam without strengthening layer of 60MPa concrete compressive strength was analysed to demonstrate the enhancement in the ultimate capacity of the proposed improvements of TRM strengthening technique for high strength RC beams. The ultimate capacity of RC beam with concrete compressive strength of 60 MPa was 53.9 kN that means the increase in the ultimate capacity was about 8% compared with control beam of 31MPa compressive strength.

5.4.1.1 Ultimate capacity and load – deflection behaviour

Table 5.4 presents the ultimate deflections and loads of beams with concrete strength C60. All strengthened beams with concrete compressive strength equal to 60 MPa exhibited increasing the ultimate capacity in comparison with C30 counterparts. The increase in the ultimate capacity of beams with cementitious connectors was about 19% in comparison with C30 counterparts. However, beams without connectors explained 12% and 4% of beams 16B0-M135 and 5C0-M135, respectively. That was because of both beams explained debonding of the strengthening layer. The results explain that the efficiency of
cementitious connectors increases as the strength of substrate concrete increases due to
preventing the separation of the concrete cover. By comparing the ultimate capacity of the
strengthened beams with C60 RC control beam, it can be noted that the enhancement due
to strengthening with TRM is increased compared with the obtained improvement in case
of beams with concrete compressive strength of 30MPa. For example, strengthened beams
without connectors explained enhancement ranged from 34% to 44% of beams
strengthened with 5 layers carbon and 16 layers basalt textile fibres. The application of
connectors improves the enhancement significantly to about 76% and 99% in comparison
with control beam of textile carbon and basalt fibres, respectively.

In addition, all strengthened RC beams explained higher stiffness than C30 counterparts, as shown in Figures 5.26 and 5.27. The effect of concrete strength in the
presence of connectors was higher than specimens without connectors. That was due to the
debonding the strengthening layer of beams without connectors before reaching the
ultimate capacity of textile fibres and concrete.

Table 5.4: Ultimate deflections and loads of beams with concrete C60

<table>
<thead>
<tr>
<th>Beam</th>
<th>$\Delta_{unum}$ (mm)</th>
<th>$P_{unum}$ (kN)</th>
<th>$\Delta_{num}/\Delta_{num,c30}$</th>
<th>$P_{num}/P_{num,c30}$</th>
<th>$P_{num}/P_{control,c60}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>16B0-M135-C60</td>
<td>12.4</td>
<td>77.5</td>
<td>1.25</td>
<td>1.12</td>
<td>1.44</td>
</tr>
<tr>
<td>16B5-M135-C60</td>
<td>12.1</td>
<td>94.6</td>
<td>0.96</td>
<td>1.19</td>
<td>1.76</td>
</tr>
<tr>
<td>16B7.5-M135-C60</td>
<td>11.2</td>
<td>91.5</td>
<td>0.94</td>
<td>1.15</td>
<td>1.70</td>
</tr>
<tr>
<td>5C0-M135-C60</td>
<td>11.3</td>
<td>75.0</td>
<td>1.08</td>
<td>1.04</td>
<td>1.39</td>
</tr>
<tr>
<td>5C5-M135-C60</td>
<td>11.0</td>
<td>107.1</td>
<td>0.92</td>
<td>1.2</td>
<td>1.99</td>
</tr>
</tbody>
</table>
Figure 5.26: Load-deflection curves of textile basalt strengthened RC beams C60

Figure 5.27: Load-deflection curves of textile carbon strengthened RC beams C60
5.4.1.2 Stresses of textile fibres and steel reinforcement

Figure 5.28 illustrates the stresses distribution in steel reinforcement and textile fibres at the ultimate load. For all strengthened beams, the stresses in the longitudinal reinforcement reached the yield strength (590MPa) regardless of the failure mode whatever rupturing of textile fibres or debonding of the strengthening layer.

The stresses in the textile fibres without connectors at the ultimate load were less than their ultimate tensile capacity due to the debonding of the strengthening layer, see Figure 5.29. Specimens without connectors failed at about 56% of the ultimate tensile strength of the textile fibres. That was due to the debonding of the strengthening layer. However, all beams with connectors failed due to rupturing the textile fibres with reaching tensile stresses about 98% of the ultimate capacity of the textile fibres. That demonstrates the efficiency of the cementitious connectors in enhancing the contribution of the textile fibres in resisting the applied loads.

As shown in Figure 5.29, apart from beam 5C0-M135, all strengthened beams with 60MPa concrete strength demonstrated higher tensile stresses in the textile fibres at the ultimate load. However, beam 5C0-M135 exhibited lower tensile stresses in carbon fibres due to the debonding of the strengthening layer.

- All stresses values are in MPa

Figure 5.28: Stresses distribution in steel and textile reinforcement of C60 strengthened beams
5.4.2 Effect of longitudinal steel reinforcement ratio

Two longitudinal reinforcement ratios were numerically investigated to study the effect of TRM strengthening layer on the behaviour of RC beams in presence of different reinforcement ratios. Instead of 10 mm diameter rebar, 8 and 12 mm diameter rebars were used as a longitudinal reinforcement for RC beams which equivalent to longitudinal reinforcement ratio of 0.0084 and 0.012, respectively. The same mechanical properties were adopted to ensure that the variation in the behaviour is due to the amount of steel reinforcement. In addition, control beams with 8 and 12 mm longitudinal reinforcement were analysed to study the enhancement due to strengthening. The ultimate numerical capacities of control beams of 8 and 12 mm steel rebars were 33.6kN and 67.3kN, respectively. These beams will work as a control beam to the strengthened RC beams.

Beams 16B5-M135 and 5C5-M135 were selected to investigate the effect of longitudinal steel reinforcement ratio. Theses beams have been chosen because of both beams presented the highest enhancement of basalt and carbon textile fibres.

5.4.2.1 Ultimate capacity and load – deflection behaviour

Table 5.5 presents the numerical results of beams with 0.0084 and 0.012 steel reinforcement ratio. As expected, the numerical results of the strengthened beams with 8mm diameter rebars demonstrated higher enhancement than beams with 10 and 12 diameter steel rebars. That demonstrate the efficiency of the textile strengthening layer in sufficient compensation the reduction in the longitudinal reinforcement ratio. Beams
strengthened with textile basalt fibres explained increasing in ultimate capacity up to about 113% in comparison with 8mm control beams. However, this enhancement descended to about 31% of 12 mm longitudinal steel reinforcement beams.

The same behaviour with greater extent can be noted for beams strengthened with textile carbon fibres. Strengthened beams with 0.0084 longitudinal steel reinforcement ratio exhibited increasing in ultimate capacity up to about 150% in comparison with non-strengthened beams. While only about 46% enhancement in ultimate capacity in case of beams with steel reinforcement ratio equal to 0.012. The low achieved improvement of high steel reinforcement ratio was because the strengthened RC beams changed from under reinforced concrete beam to over reinforced concrete beam. That means the strength of substrate concrete significantly controls the failure of strengthened beams by means of crushing of concrete.

Table 5.5: Results of strengthened beams with steel rebars diameters 8 and 12 mm

<table>
<thead>
<tr>
<th>Beam</th>
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<th>$P_{num}$, kN</th>
<th>$\Delta_{num}/\Delta_{con}$</th>
<th>$P_{num}/P_{con}$</th>
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<td>10.7</td>
<td>98.1</td>
<td>1.43</td>
<td>1.46</td>
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</table>

The load-deflection response (Figure 5.30) of strengthened 0.0084 steel reinforcement beams in both of textile carbon and basalt fibres explained lower stiffness of strengthened beams with a 10mm diameter of steel rebars. That was due to the reduced in stiffness of the substrate RC beams due to reducing the amount of longitudinal of steel reinforcement. At ultimate load, strengthened beams with textile carbon and basalt fibres exhibited a sharp descending in the load-deflection curve. That refers to the rupturing of the textile fibres due to reaching their ultimate tensile capacity. In contrast, strengthened beams of 0.012 steel reinforcement ratio (Figure 5.30) explained gradual descending in the load-deflection curve after achieving the ultimate capacity. That was because the failure of the beams was due to the crushing of the concrete in the compressive zone before reaching the ultimate capacity of the textile fibres.
5.4.2.2 Stresses of textile fibres and steel reinforcement

Figure 5.31 illustrates the stress distribution textile fibres and steel reinforcement of at the ultimate load. Similar to previous investigated studies, the tensile stresses in longitudinal steel reinforcement reached the yield strength of all strengthened beams. The stresses in textile fibres of strengthened beams with steel reinforcement ratio of 0.0084 were about 98% to the ultimate tensile capacity of textile fibres. That indicated the rupture of the textile fibres at the ultimate load.

However, the textile fibres were stressed to about 82% of their ultimate tensile strength of beams with reinforcement ratio of 0.012. That means the strengthened beams reached the ultimate load before reaching the tensile capacity of the textile fibres and the failure was due to concrete crushing where the strengthened beams became over reinforced beam.

Figure 5.32 shows that the contribution of the textile fibres in resisting the applied load decreases as the steel reinforcement of the substrate beam increases.
5.4.3 Effect of textile fibres amounts

The amount of textile fibres has a decisive influence on the behaviour of the TRM strengthening layer. Therefore, to address the behaviour of strengthened beams with three different reinforcement ratios of textile fibres, beams strengthened with 10 layers of textile basalt fibres were numerically studied. This amount of textile fibres is equivalent to the textile reinforcement ratio equal to five layers of textile carbon fibre.

In addition, to compare the strengthened beams that have the same textile reinforcement ratio but with a different type of textile fibres, RC beams strengthened with 4 and 8 layers of textile carbon fibres were also analysed. The reinforcement ratio of four and eight layers of carbon fibres are equivalent to the TRM strengthening layer with 8 and 16 layers of
textile fibres, respectively. Moreover, the investigation includes strengthened RC beams without connectors and with 5% of cementitious connector ratio to demonstrate the effect of connectors on the performance of the TRM strengthening layer.

5.4.3.1 Ultimate capacity and load – deflection behaviour

Table 5.6 presents the numerical ultimate deflections and loads of the strengthened beams. RC beam strengthened with 10 layers of textile basalt fibres explained enhancement in ultimate capacity about 32% in comparison with control beam. The enhancement increased to about 45% because of application 5% cementitious connectors ratio. Strengthened beams with four carbon textile layers explained enhancement to the ultimate capacity to about 42% without connectors and 70% of specimens with connectors.

The application of connectors increased the enhancement in the ultimate capacity of the strengthened beams with 8 layers of textile carbon fibres from 53% to 102% in comparison with control beams. The difference between the enhancement of strengthening layer between strengthened beams with and without cementitious connectors is increased as the textile reinforcement ratio increased. That may be because of reaching the ultimate capacity of the textile fibres with low textile reinforcement ratio and omit the contribution on connectors in enhancing the interface bond strength.

<table>
<thead>
<tr>
<th>Beam</th>
<th>$\Delta_{\text{num}}$, mm</th>
<th>$P_{\text{num}}$, kN</th>
<th>$\Delta_{\text{num}}/\Delta_{\text{onum}}$</th>
<th>$P_{\text{num}}/P_{\text{onum}}$</th>
</tr>
</thead>
<tbody>
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<td>65.7</td>
<td>1.15</td>
<td>1.32</td>
</tr>
<tr>
<td>10B5-M135</td>
<td>11.7</td>
<td>72.3</td>
<td>1.22</td>
<td>1.45</td>
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<tr>
<td>4C0-M135</td>
<td>11.7</td>
<td>70.4</td>
<td>1.22</td>
<td>1.42</td>
</tr>
<tr>
<td>4C5-M135</td>
<td>11.2</td>
<td>83.5</td>
<td>1.17</td>
<td>1.68</td>
</tr>
<tr>
<td>8C0-M135</td>
<td>11.1</td>
<td>75.8</td>
<td>1.16</td>
<td>1.53</td>
</tr>
<tr>
<td>8C5-M135</td>
<td>11.1</td>
<td>100.2</td>
<td>1.16</td>
<td>2.02</td>
</tr>
</tbody>
</table>

By comparing the enhancement of 10 layers textile basalt fibres with 5 layers of carbon which has the same textile reinforcement ratio, it can be found that textile carbon fibres explained higher enhancement to the ultimate capacity in both cases of strengthened beams with and without cementitious connectors. The same advantage was found of textile carbon fibres for strengthened RC beams with 8 layers. Figure 5.33 demonstrates that for the same
interface properties and equivalent textile fibre reinforcement, textile carbon fibres exhibited higher enhancement to the flexural capacity. That could be attributed to the higher mechanical properties of carbon fibres compared with basalt.

Strengthened RC beams with TRM textile fibres without connectors explained a slight improvement in the enhancement of the ultimate capacity despite increasing of the textile fibre amounts. That was due to the insufficient of interface bond strength which leading to debonding the strengthening layer. That can be noted from the response of the load-deflection curve, Figure 5.34, of the strengthened beams without connectors which explained gradual descending part. However, Figures 5.34 and 5.35, in the presence of connectors, the enhancement in the ultimate capacity increases as the textile reinforcement ratio increases up to reaching the ultimate compressive strength of concrete. Gradual descending of the load-deflection curve was also observed for these beams with eight layers of carbon fibres.

It can be noted, after ultimate load stage, the load-deflection curve of strengthened beams with connectors of all beams explained a sharp descending in the load-deflection curve which indicates rupturing of textile fibres. Except beam 8C5-M135 exhibited gradual descending of the load-deflection curve which refers to the crushing of concrete failure mode.

![Figure 5.33: Improvement in ultimate capacity of strengthened beams with different amounts of textile basalt and carbon fibres](image)

Figure 5.33: Improvement in ultimate capacity of strengthened beams with different amounts of textile basalt and carbon fibres
5.4.3.3 Stresses of textile fibres and steel reinforcement

At ultimate capacity, all longitudinal steel reinforcement reached the yield strength for all strengthened RC beams, as shown in Figure 5.36. As a result of the insufficient interface bond strength of strengthened RC beams without connectors, the tensile stresses of textile basalt fibres at ultimate load reached about 62% at ultimate capacity. Similarly, strengthened RC beams with 4 and 8 layers of carbon textile fibres failed at about 77% and 53% of the ultimate tensile strength of textile fibres, respectively. It can be noted that the loss of the textile fibre contribution to the bearing capacity of the strengthened RC beams increases with increasing the amount of fibres.
The application of cementitious connectors increases the contribution of the textile fibres up to reaching about 98% of ultimate tensile strength of beams strengthened with 10 layers of textile basalt and 4 layers of textile carbon fibres. However, the tensile stresses in carbon textile fibres of beam 8C5-M135 were about 82% of the ultimate tensile strength of textile fibres. That was due to reaching the ultimate compressive strength of substrate concrete.

In addition, the stress distribution of textile fibres of all strengthened beams at ultimate capacity demonstrates that high intensity of stresses is concentrated at the region of the maximum bending moment. However, stresses at the ends of textile fibres are less than 20% of the ultimate tensile strength of textile fibres.

Figure 5.36: Stresses in MPa in steel reinforcement and textile fibres at cracking and ultimate load
5.4.4 Effect of cementitious connectors ratio

Based on the experimental investigation, it was found that the maximum connector’s ratio could be applied without causing a severe damage to the concrete cover is 7.5%. Therefore, this connector ratio was numerically investigated on 8 and 10 layers of textile basalt fibres and 4, 5 and 8 layers of textile carbon fibres to study the effect of increasing connectors ratio from 5 to 7.5% on the behaviour of the strengthened beams. The numerical results of strengthened beams with 7.5% connector’s ratio were compared with the results of 5%connector’s ratio counterparts.

Table 5.7 presents the numerical results of strengthened beams with 7.5% connector’s ratio. The results demonstrated that increasing the connector’s ratio from 5% to 7.5% has an adverse effect on the ultimate capacity of the strengthened RC beam with 8 and 10 layers of textile basalt fibres, as shown in Figure 5.38. Similar observation with lesser extent was experimentally noticed of beams strengthened with 16 layers of textile basalt fibres. That leads to negligible effect on the ultimate deflections of the strengthened beams. That may because, in case of 8 and 10 layers of textile basalt fibres, the interface bond strength that provided by 5% connectors ratio is enough to reach the ultimate tensile strength of the textile fibres. However, a reduction in ultimate capacity was observed of the numerical results of beams strengthened with 16 layers of textile basalt fibres. Figure 5.37 shows the load-deflection curves of strengthened RC beams with textile basalt fibres.

Beams strengthened with textile carbon fibres exhibited a reduction in ultimate capacity and deflection when the connector’s ratio increased from 5% to 7.5%, Figure 5.38 and Figure 5.39. The decrease in the ultimate capacity was increased as the amount of textile carbon fibres increased. The reduction in the capacity may be due to increasing the interface bond strength, which led to increasing the intensity of the stresses at the concrete cover, which in turns weaken the cover through the initiation of cracks. That effect is enhanced as the modulus of elasticity and amount of fibres increases. These numerical results agree the obtained experimental results of RC beams strengthened with 16 layers of textile basalt fibre. These beams explained a separation of the concrete cover when the connector’s ratio increased from 5% to 7.5%. Therefore, 5% connectors ratio is considered adequate to provide the required interface bond strength with textile fibres ratio for concrete strength of 30MPa to avoid concrete separation.
Table 5.7: Ultimate deflections and loads of strengthened beams with 7.5% connectors ratio

<table>
<thead>
<tr>
<th>Beam</th>
<th>$\Delta_{unum}$ (mm)</th>
<th>$P_{unum}$ (kN)</th>
<th>$\Delta_{unum}/\Delta_{n-5%}$</th>
<th>$P_{num}/P_{n-5%}$</th>
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</thead>
<tbody>
<tr>
<td>8B7.5-M135</td>
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<td>67.8</td>
<td>1.02</td>
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<tr>
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<td>0.88</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Figure 5.37: Load-deflection curves of strengthened RC beams with textile basalt fibres
Figure 5.37: Load-deflection curves of strengthened RC beams with textile carbon fibres.

Figure 5.39: Improvement in capacity of beams with 5 and 7.5 % connectors ratio.
5.5. Summary and concluding remarks

Table 5.8 presents the numerical results of the ultimate load and deflections of the studied RC beams. The proposed model demonstrated good agreement with the experimental results with the overall error in ultimate load of 9%. However, all numerical analysis explained higher stiffness than experimental results, which could be due to ignoring the effect of shrinkage and the assumptions of the perfect bond of the steel reinforcement and textile fibres.

By comparing the crack patterns and level of stresses at ultimate load in addition to the deflection and load, it can be found the validity of proposed coefficients of the interface bond model; especially the assumption of considering the adhesion bond in tension and shear is identical. The results also indicated that the TRM strengthening layer increases the tensile strength in concrete at the level of longitudinal reinforcement from 0.6 MPa to about 3.0 MPa in presences of sufficient interface bond strength due to the application the cementitious connectors. In addition, the predicted slip of the strengthening layer was less than the observed from the experimental. That may be because of the accuracy of the method that used in measuring the slip which was based on the relative displacement that recorded from LVDTs with ignoring the crack initiation and horizontal deformation in concrete and TRM.

From the investigation of the parametric studies, it was found that increasing the compressive strength of concrete from 30 to 60 MPa exhibited 17% increase in ultimate capacity. Moreover, a negligible effect of concrete strength on the improvement strengthened RC beams without connectors due to the debonding of the TRM layer (e.g. the improvement ranged between 44 and 48%, which are close of the enhancement with C30). However, for beams with 5% cementitious connector’s ratio, the increase of concrete strength significantly increased the ultimate load to about 80% and 100% of beams strengthened with 16 basalt and 5 carbon textile fibres, respectively.

The effectiveness of connectors is also influenced by the axial strength of the strengthening layer (e.g. the amount and mechanical properties of the textile fibres). For instance, beams strengthened with 8 and 10 textile basalt fibres exhibited a negligible difference in the enhancement of the TRM when the ratio of connectors increased from 5% to 7.5%. Moreover, with a higher amount and or strength of textile fibres, the increase in the connectors (more than 5%) reduces the ultimate strength. That was due to increasing the stress concentration in the concrete, and with 30 Mpa concrete strength, these stresses lead to separating the concrete cover.
Table 5.8: Numerical results of strengthened beams

<table>
<thead>
<tr>
<th>Beam</th>
<th>$\rho_f$ (%)</th>
<th>$\Delta_{num}$ (mm)</th>
<th>$P_{num}$ (kN)</th>
<th>$\Delta_{num}/\Delta_{con}$</th>
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<th>$f_{te}$ (MPa)</th>
<th>$k_f$</th>
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<td>1.97</td>
<td>996</td>
<td>0.83</td>
</tr>
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</table>

$\rho_f$ is the textile fibre reinforcement

$\Delta_{num}$ is the mid span deflection at ultimate load.

$P_{num}$ is the ultimate load

$f_{te}$ is the effective tensile stress in textile at ultimate load

$k_f$ is the effectiveness textile fibres ($k_f = f_{te}/f_{tu}$)

$f_{tu}$ is the ultimate tensile of textile fibres.
Chapter Six

Conclusions and recommendations

6.1 Summary
The proposed improvements in the existing TRM strengthening technique has provided new insights to the strengthening of RC beams through preventing the debonding of the strengthening layer to achieve higher enhancement. The first part of this study was devoted to developing high-performance textile reinforced mortar with emphasising in increasing the adhesion properties of the matrix by using high cement content and SCMs. Different replacement ratios of GGBS, silica fume and fly ash were utilised. The effect of the high-performance matrix was investigated and compared with normal matrices through flexural and direct tensile tests of TRM composites.

The second part of this study involved comprehensive experimental evaluation of the effect of matrix properties and textile fibres, basalt and carbon, with highlights on the inclusion of cementitious connectors on the behaviour of RC beams strengthened with TRM. The numerical simulation, the third part, was carried out to study the stress distribution of the concrete, steel and textile reinforcement. Besides, case studies were conducted to explore the effect of substrate concrete strength, steel reinforcement properties, amount of cementitious connectors and textile fibres on the structural behaviour of strengthened RC beams.

Finally, the adverse effect of corrosion on the performance of the strengthening was examined utilising repair corroded RC beams with different degree of corrosion. Moreover, address the reduction in the structural performance of strengthened RC beams in the process of corrosion was also investigated. Based on the experimental and theoretical investigations of this study, the following conclusions can be drawn.
6.2 Overall conclusions

6.2.1 Development of high performance textile reinforced mortar

- Overall, it was found, in production of high strength mortar (mortar with compressive strength more than 100MPa), the best amount of superplasticizer to achieve high performance in terms of workability, tensile and compressive strength ranges between 30 and 35 kg/m³. However, higher amounts of superplasticizer improve the workability and compressive strength but reduce the tensile strength of the mortar.

- The addition of SCMs improves significantly the durability of high strength mortar through reducing the permeability by means of restricting access chloride to hardened mortar. Binary replacement of 15% GGBS and 15% un-densified silica fume explains the highest performance among single, binary and ternary replacements through reducing the permeability of high strength mortar up to 95%. Single, binary and ternary up to 30% replacements of SCMs (GGBS, SF and FA) by volume of cement improve the mechanical properties of high strength mortar. Binary replacement of 15% GGBS and 15% un-densified silica fume exhibits the best behaviour among the replacements in terms of compressive and tensile strength.

- The bond strength between substrate rough concrete and mortar is highly influenced by the strength of mortar up to reaching the tensile strength of substrate concrete. Mortar with compressive strength of 135MPa exhibits 2.8 times the bond strength of 35MPa compressive strength.

- The behaviour of TRM is significantly influenced by the bond strength that provided from mortar, which depends on the strength of mortar. For the same properties and amount of textile fibres, TRM with high strength mortar (135MPa compressive strength) explains double tensile and flexural strength of TRM with normal strength mortar (35MPa compressive strength).

- Using high strength mortar in TRM allows increasing the amount and strength of textile fibres that can be used before slipping of textile fibres, which in turns increase the strength of TRM. However, normal strength mortar limits the available strength of TRM due to exceeding the transferred stresses by textile fibres the tensile strength of mortar.

- High strength mortar with 135MPa compressive strength explains a significant reduction in free shrinkage to about 62% and 57% in comparison with 35MPa and 70MPa compressive strength, respectively. However, composite specimens of
normal strength concrete and high strength mortar explains cracks in the new mortar due to high bond strength of the interface and these cracks increase as the bond strength increases. However, the addition of textile fibre to normal strength mortar reduces the free shrinkage while it increases the free shrinkage of high strength mortar due to increase its porosity. However, the textile fibres improve the performance of the high strength mortar TRM against shrinkage of restrained specimens by controlling the cracking of the mortar.

6.2.2 Structural performance of RC beams strengthened with high performance textile reinforced mortar

- Mortar strength of TRM has a significant effect on the behaviour of RC beams strengthened with TRM through controlling the interface bond strength and textile-mortar bond strength. Beams strengthened with high strength mortar (135MPa mortar compressive strength) TRM exhibit 27% increasing in ultimate capacity of control beam while only 12% enhancement is achieved with mortar compressive strength 35MPa counterparts due to debonding of the strengthening layer.

- Application of high strength cementitious connectors improves the interface bond strength of strengthened RC beams to provide effective bond strength until textile fracturing and changes the failure mode from debonding of the strengthening layer to flexural failure. However, the effectiveness of the connectors is limited by the tensile strength of concrete substrate to avoid concrete cover separation.

- Higher mechanical properties of textile fibres are more efficient in improving the structural performance of the strengthened members. Strengthened RC beams with textile carbon fibres exhibited higher enhancement to the ultimate capacity of control beam up to about 61% and 44% with and without connectors, respectively in comparison with 43% and 33% of RC strengthened with textile fibres. Even though, the basalt reinforcement ratio is higher than carbon textile fibres are due to the higher modulus of elasticity of textile carbon in comparison with basalt fibres.
6.2.3 Numerical simulation of RC beams strengthened with high performance textile reinforced mortar

- The proposed numerical model explains good agreement with the experimental results. Although, the effect of assumption perfect bond of textile fibres with mortar appears on the results of strengthened beams in presence of connectors due to the effective interface bond strength. That effect presents by means of error in predicting the ultimate load capacity reach to about 9% of the experimental results.
- The strength of substrate concrete significantly affects the performance of the strengthened beams in particular with presence of cementitious connectors. For RC beams with 60 MPa compressive strength, the enhancement of the ultimate capacity of control can be reached to about 100% and 80% of textile carbon and basalt fibres respectively. In addition, for RC beams with 31 concrete compressive strength, increasing the connectors’ ratio from 5% to 7.5% leads to concrete cover separation due to increase the stress intensity in concrete.
- The amount of textile reinforcement should be determined with considering the available steel reinforcement ratio to avoid changing the member from under reinforced to over reinforced. However, the failure of over reinforced beams is controlled by the compressive strength of the concrete which omit the expected contribution of the textile fibres in resisting the applied loads.
- The efficiency of increasing the amount of textile fibres and cementitious connectors is limited by the substrate concrete strength and to achieve higher enhancement in the capacity of normal strength concrete, strengthening in both tension and compression faces are required.

6.2.4 Repair of corroded RC beams using high performance textile reinforced mortar

- Corrosion of steel reinforcement reduces the flexural strength of RC beams depending on the level of corrosion. The relationship between the reduction in flexural strength and corrosion level is not linear because it depends on the type of corrosion whether its pits or continuous corrosion. The reduction in flexural strength is about 8% and 11% for 5.6% and 7.3% corrosion level, respectively.
- For degree of corrosion up to 4%, the flexural behaviour of the repaired beams was identical to the RC beams with TRM without corrosion. Up to 6.1% corrosion level, using TRM improves the flexural strength of the corroded beams to exceed the flexural strength of the un-corroded RC beams.
- The effect of connectors was found identical in both corroded and non-corroded repaired RC beams for degree of corrosion up to 5%. However, higher corrosion level of RC beams with application of cementitious connectors may need further investigation. Moreover, textile fibres with high mechanical properties can present higher enhancement to the flexural strength compared with control beams despite the separation of concrete cover.

6.2.5 Corrosion impacts on the structural performance of RC beams strengthened with high performance textile reinforced mortar
- For RC beam that strengthened with TRM before subjecting to corrosion, corrosion has more critical effect on strengthened RC beams than non-strengthened RC beams. The reduction in flexural strength of strengthened RC beams due to corrosion is about twice the reduction of non-strengthened RC beams for the same amount of corrosion. However, the strengthened beams exhibited higher flexural strength than control beams (un-strengthened). Corrosion in strengthened RC beams changes the failure mode from flexural to separation of concrete cover depending on the amount of corrosion and textile fibre ratio of the strengthening layer.
- The effect of corrosion on RC beams is higher after strengthening than before strengthening beams due to confinement of corrosion products in case of strengthening, which leaded to reduce the interface bond strength. However, in repair, some of these products and weak concrete cover are removed before casting the TRM layer. For degree of corrosion higher than 5.5%, it is recommended to extend the TRM layers to the sides of the beams to reduce the consequences of cover separation.
- For RC beams strengthened before corrosion, the corrosion level and textile fibre type control the reduction of the flexural strength. Basalt TRM of strengthened beams with 6.7% corrosion level lose about 34% of ultimate capacity which means explains negligible enhancement in ultimate capacity of corroded beams. However, carbon fibres explain 32% reduction in ultimate strength with corrosion level up to 9.5%.
6.3 Recommendations for future work

- Experimental and theoretical investigations of the bond strength between the textile fibres and surrounding mortar for varied properties of mortar and textile fibres are required to address the bond slip of textile fibres on the behaviour of TRM. Then incorporate the bond slip model in the numerical and analytical modelling of strengthened RC beams with TRM.

- Experimental and numerical investigations of flexural strengthening RC beam in both tension and compression zones. The main parameters that can be address are the steel reinforcement ratio, properties of TRM and ratio of connectors.

- Experimental and numerical investigations of strengthened RC beams in shear with TRM of different properties and amounts of shear reinforcement, textile fibres and cementitious connectors.

- Theoretical investigation of repaired and or strengthening corroded RC beams in presence varied amounts of longitudinal steel reinforcement corrosion level. With highlighting on the interface bond strength.

- Experimental and numerical investigations of flexural strengthening RC beam in both tension and compression zones. The main parameters that can be address are the steel, thickness of concrete cover, reinforcement ratio, properties of TRM and ratio of connectors.

- Experimental and numerical investigations of strengthened RC beams in shear with TRM of different properties and amounts of shear reinforcement, textile fibres and cementitious connectors. The main interesting key research would be different configuration of the TRM layer such as U confinement of TRM layer at different spaces along the beams.

- Investigate the behaviour of TRM composites as a strengthening layer under different exposer of thermal and environmental factors for short and long-term periods to demonstrate the service life of the strengthened RC beams with high performance TRM.

- Investigate the effect of the load history on the performance of strengthened RC beams with TRM by means of strengthened preloading members up to service load before strengthening. In addition, the effect of creep of the TRM as a composite and as a strengthening layer needs to be investigated.

- Investigate the efficiency of the high performance TRM in strengthening and protection of RC beams by means of strengthening the bottom and sides of the beam with taking in account high degree of corrosion.
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Appendix A

A.1 Calculations of the area of textile fibres.

The area of fibres was calculated based on the density of the textile fibres according to the following procedure:

1. Measure the length of sample (L).
2. Measure the volume of sample (V).
3. Measure the weight of sample (W).
4. Calculate the density of sample \( \rho = \frac{W}{V} \)
5. Determine the density of the sample per length \( \lambda = \frac{W}{L} \)
6. Determine the area of filament \( A_f = \frac{\lambda}{\rho} \)

For textile basalt and carbon fibres, the area of one filament was 0.25mm\(^2\) and 1.6mm\(^2\), respectively.

A.2 Stress strain curves of TRM composites under direct tensile load.
Tensile stress-strain curve of TRM composites under direct tensile load

A.3 The load deflection curves of the TRM composites under 3-point flexural loading
Load deflection curves of TRM composites under flexural loading
Appendix B

B-1 Design of RC beams according to Euro Code 2.

The analysis is based on the following equations:

Equilibrium equations:

\[ \sum F_x = 0 \] (\( F_x \) represents the horizontal force).

\[ T = C \]

\[ T = A_s \cdot F_y \]

\[ C = F_{cd} \cdot x \cdot b \]

\[ x = \frac{A_s \cdot F_y}{F_{cd} \cdot b} \]

\[ d = h - d_b/2 - \text{cover} \]

\[ z = d - x/2 \]

\[ M = T \cdot z \]

\[ P = \frac{2M}{0.525} \]

- \( h = 175 \text{ mm}, b = 125 \text{ mm}, \text{cover} = 20\text{ mm} \)
- \( d = 150 \text{ mm} \)
- \( F_y = 590 \text{ MPa}, D_b = 10.2 \text{ mm}, \text{As of two bars} = 163.4\text{mm}^2 \)
- \( A_s \cdot F_y = 96.4 \text{ kN} \)
- \( x = 24.88 \text{ mm} \)
- \( z = 137.56 \text{ mm} \)
- \( M = 13.26 \text{ kN.m} \)
- \( P = 50.51 \text{ kN} \)

Check yielding of longitudinal reinforcement:

\[ \varepsilon_s = \varepsilon_{cu} \left( \frac{d-x}{x} \right) \]
'\( \varepsilon_{cu} = 0.003 \)
\( \varepsilon_s = 0.015 \)
\( E_s = 190\,000\, MPa \)
\( F_y = 2850\, MPa > \text{yield strength of steel rebars} \)

Shear capacity of concrete:
\[ V_{Rd,c} = \left[ C_{Rd,c} \times K(100 \times \rho_1 \times f_{ck})^{1/3} + K_1 \times \sigma_{cp} \right] b_w \times d \]
\[ K = 1 + \frac{200}{\sqrt{d}} \leq 2.0 \]
\[ \rho_1 = \frac{A_{s1}}{b_w \times d} \leq 0.02 \]
\[ \sigma_{cp} = \frac{N_{Ed}}{A_c} < 0.2 f_{cd} \]
\[ C_{Rd,c} = \frac{0.18}{Y_c} \]
\[ K_1 = 0.15 \]

Shear force provided by steel reinforcement is the smaller of the following values:
\[ V_{Rd,s} = \frac{A_{sw}}{S} z \times f_{ywd} \times \cot \theta \quad \text{And} \quad V_{Rd,s} = \frac{a_{cw} \times b_w \times Z \times v_1 \times f_{cd}}{(\cot \theta + \tan \theta)} \]
\[ v_1 = 0.6 \quad \text{for} \quad f_{ck} \leq 60\text{MPa} \]
\[ v_1 = 0.9 - \frac{f_{ck}}{200} \quad \text{for} \quad f_{ck} \geq 60\text{MPa} \]

Where, \( V_{Rd,c} \) is the shear capacities of concrete, \( A_{s1} \) is the tensile reinforcement, \( b_w \) is the smallest width of the cross-section in tensile area (mm), \( N_{Ed} \) is the axial force in the cross section due to loading or prestressing (N), \( A_c \) is the area of concrete section, \( V_{Rd,s} \) is the shear strength provided by steel shear reinforcement, \( A_{sw} \) is the area of shear reinforcement, \( S \) is the spacing between shear reinforcement, \( f_{ywd} \) is the yield stress of shear reinforcement, \( v_1 \) is strength reduction factor and \( a_{cw} \) is a coefficient represents the state of stress in compression chord. For non-prestressed members \( a_{cw} \) is taken equal to 1.0.

Applied load according to shear design is 147.8 KN > flexural load (50.51kN)
B-2 Calculations of corrosion of small scale beams

Based on Faraday’s law, the required time to create 10% degree of corrosion in steel reinforcement is determined as following (Fang et al., 2004)

\[ t = \frac{m \times z \times F}{M \times I} \]

Where:
- \( t \) = time (second)
- \( m \) = mass loss due to corrosion (gm)
- \( z \) = valance (2)
- \( F \) = Faraday’s constant (96480 A.sec.)
- \( M \) = atomic weight of metal (55.85)
- \( I \) = imposed current (A)

Total length of steel rebar = 42 cm
Density of steel rebar = 7.85 gm/cm³
Mass of the rebar = \( 7.85 \times \pi \times (0.5)^2 \times 42 = 259 \) gm
10% degree of corrosion = \( 0.1 \times 259 = 25.9 \) gm

Apply corrosion current density \( (i_{corr}) = 0.3 \) mA/cm²
\[ I = 0.3 \times 2 \times \pi \times 0.5 \times 42 = 39.6 \text{ mA} \]

Thus, the required time is 26 days.
Appendix C
Load–slip curves of strengthened RC beams with TRM