Repair of submerged concrete piles with FRP composites

Master's Thesis in the Master's Programme Structural Engineering

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CHALMERS UNIVERSITY OF TECHNOLOGY
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Cover:
The first and third picture from the left are on specimens before subjected to accelerated long-term durability tests. The second and fourth pictures are the same specimens after the testing.

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ABSTRACT

Deterioration of concrete piles in marine structures due to harsh environmental conditions has highlighted the need of continuous maintenance and renewal of such structures. To repair these structures a relatively new and emerging repair method is to wrap the piles with fibre reinforced polymer (FRP) materials. The lightweight, high strength and flexibility of the FRP material provide for a quick and effective repair. The idea is that the FRP material with low permeability and good resistance to harsh environments will provide the pile with a protective layer and thus increase the durability of the pile.

The aim of this thesis is to investigate the feasibility of the FRP repair method for submerged concrete piles. The methodology used in the thesis is to perform: a literature review, interviews with experts in the field, accelerated long-term durability tests focusing on corrosion and a case study to compare costs between an FRP repair and a conventional pile jacket repair method.

From the interviews, conclusion could be made that there is a need and interest for new sound, quick and durable repair methods. The literature review showed that FRP reparations mostly have been made in the splash zone to increase the durability and thus enhance the service life of the pile. It also showed that wrapping concrete with glass fibre reinforced polymer (GFRP) or carbon fibre reinforce polymer (CFRP) reduces the corrosion rate equally.

The results from the experiment showed that FRP wrapping of reinforced concrete specimens reduces the corrosion rate, crack propagation and crack bandwidths. In the case study, the FRP repair method showed to be cost effective in comparison to a traditional concrete pile jacket reparation.

Key words: fibre reinforced polymer, FRP, repair, submerged, concrete, piles, wrapping, marine, corrosion, underwater, freeze and thaw

Reparation av undervattenspelare i betong med FRP komposit

Examensarbete inom masterprogrammet Konstruktionsteknik och Byggnadsteknik

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SAMMANFATTNING

Nedbrytning av betongpålar i marina konstruktioner som ett resultat av den hårda miljön har understrukit vikten av kontinuerligt underhåll av dessa konstruktioner. En ny metod för att reparera dessa konstruktioner är att linda pålar med fiberförstärkt plast (FRP). FRP materialets låga vikt, höga styrka och flexibilitet möjliggör för en snabb och effektiv reparation. Idén är att det täta FRP materialet tillsammans med dess goda egenskaper att motstå tuffa miljöer ska förse pålen med ett skyddande lager och därmed öka pålens beständighet.

Målet med detta examensarbete är att undersöka genomförbarheten av att reparera undervattenspålar med FRP. De använda metoderna för att undersöka detta är: litteraturstudie, intervjuer med experter inom området, accelererad långtidsprovning med fokus på korrosion och en fallstudie för att jämföra kostnader mellan en FRP- och en pågjutningsreparation**.**

Från intervjuerna kan slutsatsen dras att det finns ett behov och intresse för nya säkra, snabba och hållbara reparationsmetoder. Litteraturstudien visade att FRP reparationer mestadels har genomförts i skvalpzonen för att öka hållbarheten och därigenom förlänga pelarnas livslängd. Den visade också att lindning av betong med glasfiberförstärkt plast eller kolfiberförstärkt plast minskar korrosionshastigheten lika mycket.

Resultatet från experimentet har påvisat att FRP lindning av armerad betong minskar korrosionshastigheten, sprickspridningen och sprickbredden. I fallstudien så påvisades det att FRP lindningen är en kostnadseffektiv reparationsmetod i jämförelse med en traditionell betong pågjutning.

Nyckelord: fiberförstärkt plast, FRP, reparation, , betong, pålar, linda, marin, korrosion, undervatten

Contents

Preface

In this study interviews have been carried out with Stig Östfjord, Senior Adviser in Civil Harbour Construction at Port of Gothenburg, Robert Klein, Engineer Diver at ÅF Infrastructure AB, Martin Lindgren, Co-Owner at DAWAB Sweden AB and their inputs have been valuable for the work.

The experiment in the study was carried out in the laboratory of the Department of Structural Engineering at Chalmers University of Technology. The help from Carlos Berrocal in the planning of the accelerated corrosion test by an impressed current, Mohsen Heshmati in programming the climate chamber and Sebastian Almfeldt in the laboratory work were of great appreciation.

This master thesis had not been possible without the technical knowledge, enthusiasm, help and guidance from our supervisor Valbona Mara at ÅF infrastructure and examiner Reza Haghani.

Göteborg June 2017

Johan Röös

August Uddmyr

1 Introduction

1.1 Background

Deterioration of concrete piles in marine structures due to harsh environmental conditions has highlighted the need of continuous maintenance and renewal of such structures. To repair these structures a new and emerging repair method is to wrap the piles with fibre reinforced polymer (FRP) materials. The lightweight, high strength and durability of FRP materials make them suitable for quick and efficient structural repairs (Berver et al. 2001). This method has not been utilised in Sweden yet, but there is a keen interest in the introduction of such methods, especially from the Port of Gothenburg.

The modern part of the Port of Gothenburg is mainly built in concrete resting on piles constructed in the 50's, 60's and 70's[1]. In the 50's and 60's due to lack of knowledge about chloride ingress and its effect on reinforcement steel, concrete structures were built with the belief that they were everlasting. However, in the 70's concrete structures were designed for a lifespan of 50 years. Along with poor maintenance until 2002 this have created a significant need of repair of submerged concrete piles. The maintenance budget for the Port of Gothenburg has increased since 2002 from 15 million to 200-250 million a year[1] which has created the urge for new durable and efficient repair methods.

A submerged pile is divided into degradation zones dependent on the surrounding conditions along the height of the pile (Yao 2016). The different zones are shown in Figure 1 and starting from the top there is the atmospheric, splash/tidal, submerged and mud zone. Worst effects of degradation are in the splash zone[1] were continuous wetting and drying occurs resulting in, for example, increased rate of corrosion and salt scaling.

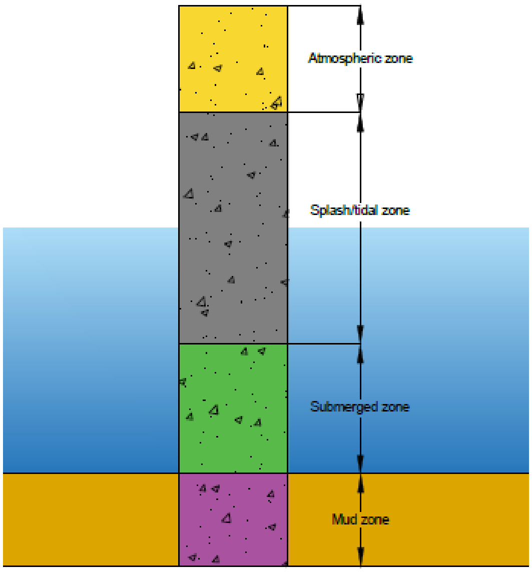


Figure 1: The different degradation zones of a submerged pile

[1] Stig Östfjord Port of Gothenburg, interview 30 Mars 2017

1.2 Aim and objectives

The aim of this master thesis is to increase the knowledge and awareness of the FRP repair method and to evaluate the effectiveness of this method for repairing submerged concrete piles. Within the aim, six objectives are defined:

- What are the degradation causes of submerged concrete piles?
- What are the existing rehabilitation/repair methods today?
- What are the advantages/disadvantages of FRP repair of piles?
- How is this method executed onsite and what is the state-of-practice today?
- What are the durability benefits of repairing piles with FRP?
- What are the cost and intangible benefits of repairing piles with FRP?
- Experimental work to investigate if FRP wrapping of reinforced concrete has a positive effect on corrosion rate.

1.3 Research approach

The aim is achieved by performing an extensive literature study and by interviews with experts within the field. The interviewees are performed with Stig Östfjord, Senior Adviser in Civil Harbor Construction at Port of Gothenburg, Martin Lindgren, Co-Owner at DAWAB Sweden AB, and Robert Klein, Engineer Diver at ÅF Infrastructure AB. The traditional techniques of repairing submerged piles are reviewed along with a review of the new method consisting of repairing submerged piles with FRP. The existing experience of this method is studied based on on-site applications. A case study, with material provided by the Port of Gothenburg, is carried out where the costs of a performed repair on a submerged concrete pile are compared with a hypothetical FRP repair. Experimental work to study the durability of FRP wrapped concrete specimens with a focus on corrosion resistance is carried out.

1.4 Limitations

The investigation is limited to the repair of existing submerged concrete piles with FRP composites bonded with adhesives directly to the concrete. The causes of degradation in Swedish climate and repair methods carried out underwater is studied.

In the experimental work to investigate the durability of FRP wrapped concrete specimens, limitations are made to a bi-directional glass fibre reinforced polymer (GFRP). The test is also limited to two layers of wrap applied by a wet layup with an epoxy resin. The experimental work of accelerated long-term durability tests was limited to two months. The evaluation of the durability tests is limited to measured values of the steel mass loss and visual examination.

2 Degradation Causes of Submerged Concrete Piles

There are several degradation causes of submerged concrete piles that ultimately requires the need of repair or retrofit of the piles. These can be damages resulting from mechanical actions on the piles or by natural effects on the concrete and the reinforcement steel. Mechanical actions are a physical impact on the piles, for example, damages caused by piling or abrasion by drift ice. Natural effects can be initially chemical resulting in loss of structural capacity or merely physical. The primary causes of degradation are accounted for below.

2.1 Chemical causes

2.1.1 Chloride ingress

In a marine environment, chloride ions from the seawater penetrate the concrete and disrupt the passive layer of the steel provided by the alkalinity of the concrete. The ingress is specially located in the tidal/splash zone, where the salt is water-borne and deposited to the surface by evaporation (Domone and Illston 2010). When the steel has been de-passivated, a local anode can form in that area. The remaining steel forms a cathode and electrons starts to flow from the anode to the cathode, resulting in corrosion at the anode. The corrosion product, rust, has a larger volume than the original steel, which creates an expansive force that leads to cracking, spalling and delamination of concrete (Suh 2006), see Figure 2.

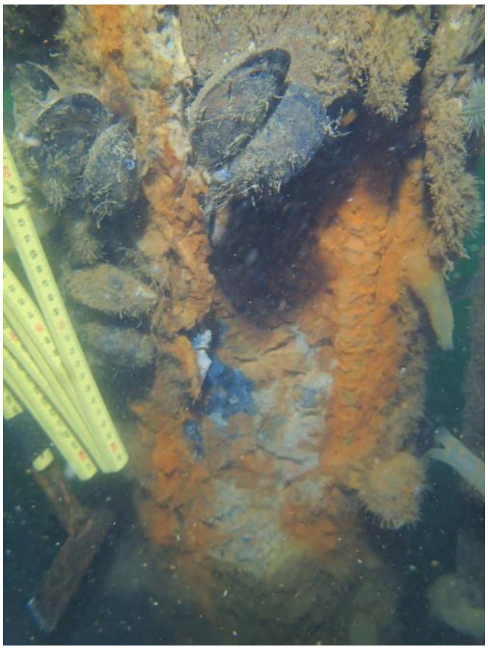


Figure 2: Spalled concrete and exposed corroded reinforcement (Robert Klein)

2.1.2 Sulphate attack

Sulphates in the seawater can react with the hydration products from the calcium aluminate phases of the hardened cement (Domone and Illston 2010). These reactions are expansive and form ettringite. Because the reaction is expansive it can cause cracks and spalling, see Figure 3. The damage starts at corners and edges and progresses inwards until failure. However, this process usually takes several years.

A delayed formation of ettringite can also occur from an internal sulphate attack during the heat hardening of concrete. Ettringite formed during the cement hydration at normal temperatures breaks down at temperatures higher than 70°C. The calcium silica hydrate then absorbs the sulphates, but when the concrete cools down and hardens the sulphates becomes active again and ettringite can re-form.

Figure 3: Vertical crack, ettringite (Robert Klein)

2.2 Mechanical causes

2.2.1 Freeze/thaw

Ice has 9% larger volume than water. Due to this, the volume of the water in the concrete pores expands when it freezes (Domone and Illston 2010). If there is insufficient space in the pores for the freezing water to expand an internal pressure is created. Lower capillary porosity and a higher degree of saturation will increase the pressure. The level of pressure also depends on if the pressure can be relieved through an escape boundary. As result of the internal pressure, cracking and spalling can occur and accumulate during freeze/thaw cycles. These phenomena occur in the splash zone (Thoresen 2014).

2.2.2 Abrasion

The piles are mainly subjected to abrasion in the splash zone, see Figure 4. The sources of the abrasion are waves and water-borne sediment and drift ice that wears on the surface of the pile. Of this two, drift ice is the most severe which in time can wear through the concrete cover and cause exposure of reinforcement (Bengtsson and Thornström 2011). Newly-produced submerged piles are provided with an ice protection to prevent this from occurring. However, in the Port of Gothenburg for the modern docks built in the 50's, 60's and 70's no ice protection was included when the piles were constructed[1].

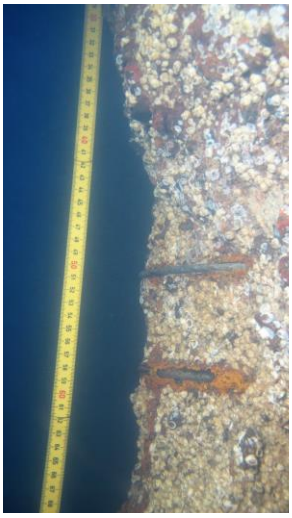


Figure 4: Abrasion in the splash zone (Robert Klein)

2.2.3 Accidental loads

The piles of a berth structure are protected from collisions with ships by the berth platform[2]. However, there is a risk of collision with the piles when a ship needs to back and turn out from the berth. The bulbous bow reaches out in front of the bow under the water and as the ship backs and turns the bulbous bow can hit the piles under the berth platform. This should usually be prevented by the bow thrust that pushes the bow out from the berth, but strong side wind and insufficient bow thrust can cause accidents like these to occur, see Figure 5.

Figure 5: Collision damage from ship (Robert Klein)

[2] Robert Klein ÅF infrastructure, interview 9 February 2017

There is also a risk that piles get damaged when the port is dredged. The lack of vision of where the underwater piles are located combined with the power of the excavator enables the bucket to hit and cause damage during dredging close by the piles[2].

In the port, ice layers can be pushed above and below each other by stream and ships. The ice layers settle upon each other and form hard pressed ice blocks. The thrust from ships can then cause these ice blocks to be slung into the piles causing damage upon collision[2].

2.2.4 Damage during production and installation

Damages on piles can occur already before installation. Defects can be missed at production, introduced during transportation and/or at site. During installation of driven piles horizontally aligned cracks can occur because of the applied stroke[1,2], see Figure 6. Piles installed with an angle can intersect and accidentally collide during piling resulting in cracks and spalling of concrete. Initial defects as these reduce the service life of the piles, because of the reduced concrete cover which enables increased rate of deterioration.

Figure 6: Horizontal cracks due to the applied stroke at installation

2.2.5 Salt scaling

In the tidal/splash zone of concrete piles, the salt will crystallise as the surface is subjected to wetting and drying. The salt crystals are formed when the water evaporates and the crystals will grow with continuous wetting and drying cycles (Domone and Illston 2010). As the crystals grow, the pressure at the concrete surface will increase which can cause disruption of the concrete. To reduce the deterioration due to salt scaling a low permeability concrete can be used.

2.2.6 Scour

Deepening of the sea bottom by the removal of material can cause reduction of bearing capacity for piles. Scour can influence bridge and berth piles as an effect of the massive water streams caused by ships and the removal of sea bottom material can also be an effect of active dredging[1]. As a result, the effective length of the piles increase and thus also the buckling length. This leads to a decrease in bearing capacity and can require strengthening or retrofit of the piles.

2.3 Concluding remarks

The first conclusion that can be made from studying degradation causes of submerged piles are that the environment is harsh and the causes of degradation are several. In Figure 7 the different degradation causes are summarised under the two groups, mechanical and chemical. One type of deterioration is seldom alone to cause the degradation in a submerged pile, for example, can physical degradations cause initial damages and reduction of the concrete cover which enables faster penetration of chlorides and sulphates.

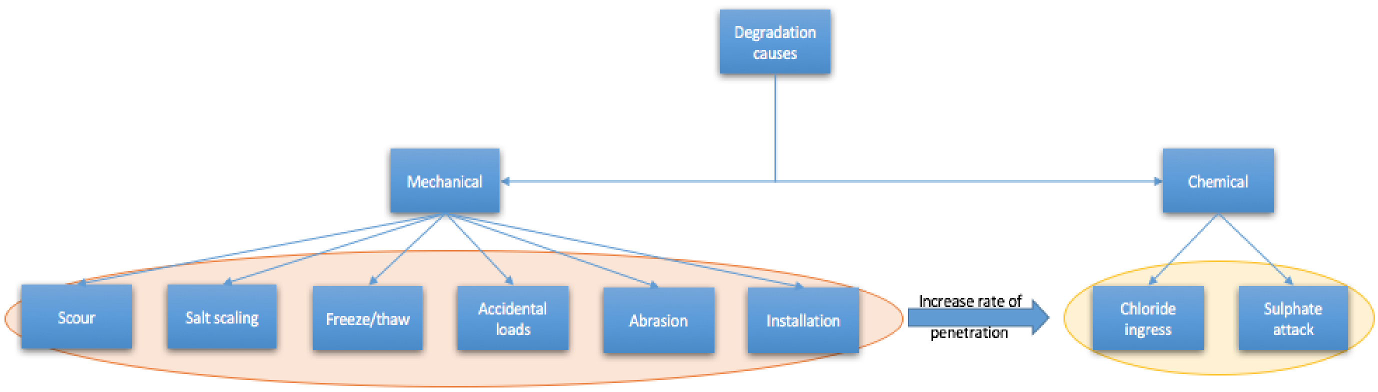


Figure 7: Overview of degradation causes

The degradation causes have also been categorised after the zones where their effects take place as shown in Figure 8. From this, the conclusion can be made that the splash/tidal zone is the most common area for damages and need of repair.

From an interview with Stig Östfjord (Port of Gothenburg, 30 Mars 2017) conclusions can be made that the most common degradation causes of submerged piles in the Port of Gothenburg are due to installation damages and ettringite (sulphate attack). Installation damages are due to the applied stroke when the piles are driven and ettringite are due to heat hardening during the production of the piles.

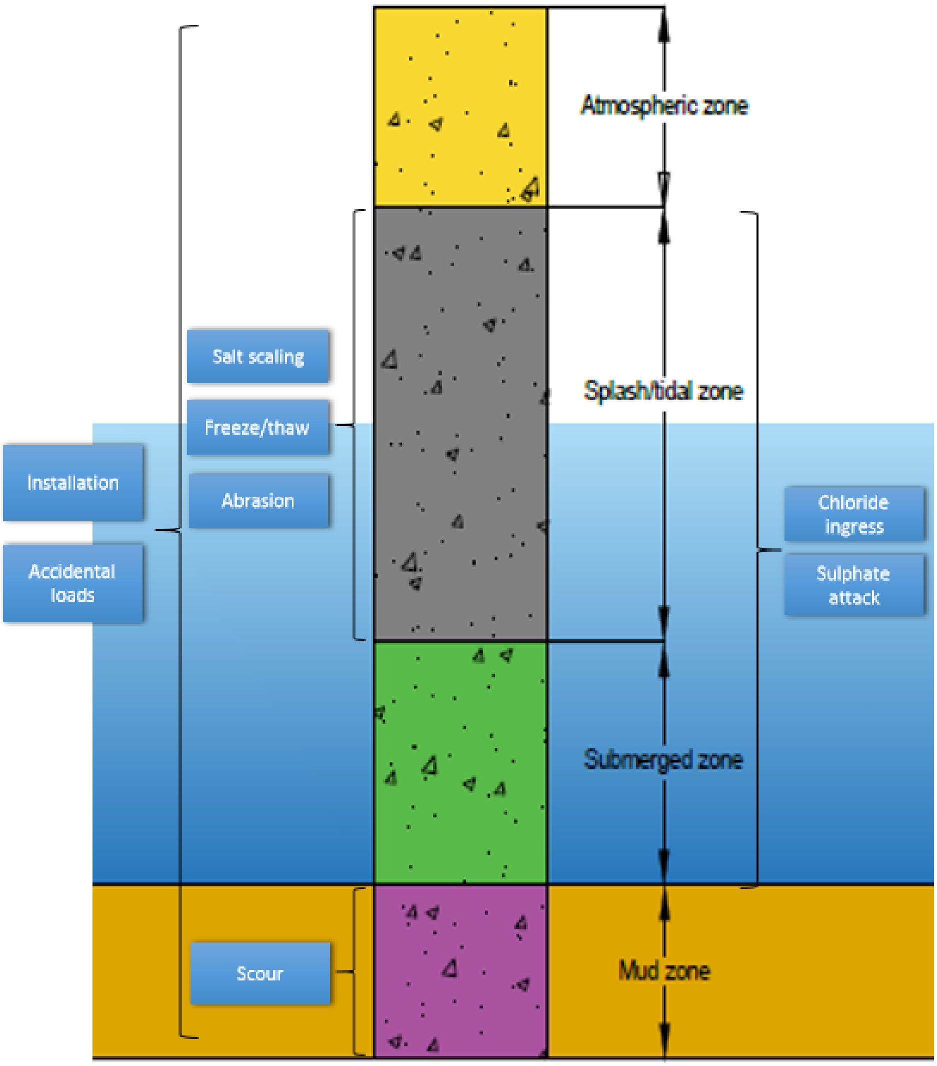


Figure 8: Degradation causes at different zones of a submerged pile

3 Traditional Repairing of Submerged Piles

3.1 Introduction

Due to the harsh environment and the degradation causes mentioned in chapter 2, there is a need for continuous repair of submerged piles with a focus on the splash/tidal zone. However, in the Port of Gothenburg, the maintenance work has not always been continuous. Almost no repair work had been done in the first 40-50 years[1] of the piles' service life. This has created an urge for repairs and large maintenance packages are carried out every fifteenth year in the present.

To detect damaged piles and decide if, how and when they should be repaired, main inspections are made every sixth year in the port[1]. The inspector evaluates the damage on the piles and an engineer decides how severe the damages is to the construction and if measures should be taken for repair. The result is analysed together with the project manager and the administrator and a ten-year maintenance strategy is carried out. For a repair to take place, the port of Gothenburg requires a minimum service life of 15-20 years of the repair. To achieve a reasonable solution the engineer designs the repair for a theoretical service life of at least 40 years.

Most methods for pile reparation that can be applied above water can also be used under water (Browne et al. 2010). However, the most efficient method above water may not be the most efficient in a marine environment. A detailed planning of the work is of great importance for an effective repair of submerged piles[3]. The marine environment and the berth platform prevents easy access to the area in need of reparation. For the repair work to take place, there are two alternatives, either the water must be excluded from the area in need of repair or the work must be performed underwater.

One method to get access to the damaged area of the pile is to exclude the water by installing cofferdams. There are cofferdams adjusted for pile repairs that consist of two steel box halves that are put together from each side of the pile. Soft rubber is used to seal the box and the water is pumped out. Thus, the repair can be performed under dry conditions which increase the work rate. However, the temporary work of installing the cofferdam and maintaining a dry environment is time-consuming and often results in increased repair costs (Browne et al. 2010).

To perform repair work at wet conditions commercial divers are needed. They can start the repair work more quickly as no installation of cofferdams is required. Divers also provides high flexibility and they can move between different areas of the substructure with ease. For the repair of berth and pier substructures, divers may provide the only access method (McLeish 1994). Divers can perform complex repair work with underwater adjusted equipment and they are most effective when the work is well planned with easy to use components[3] such as large bolts and pinholes.

[3] Robert Klein ÅF infrastructure, interview 17 January 2017

3.2 Traditional repair methods

The cause and extent of degradation of a submerged concrete pile determine the most suitable repair method. These can vary from providing a protective barrier from further degradation to increasing the structural capacity. In the following subchapters, different repairing methods are described.

3.2.1 Pile replacement

Replacements of complete piles can be necessary when for instance, the concrete is subjected to ettringite or at severe damages of the structural integrity (Browne et al. 2010). Instead of removing the damaged pile, supplemental pile/piles, which fulfil the same functions as the existing piles, are constructed in the vicinity of the damaged pile. The construction is made possible by cutting holes in the deck/platform where the piles are driven through. The pile cap and bracings are then modified to transfer the loads to the new supplemental piles, see Figure 9, and the holes are repaired.

Figure 9: Supplemental pile bents (Browne et al. 2010)

Pile replacement is a labour intensive repair method (Browne et al. 2010). Lack of space for supplemental piles may be a problem in a berth substructure with a high number of structural members, for example a high number of piles.

3.2.2 Pile jacket

One method to repair piles is to cast a jacket of concrete along the full length or at the damaged/deteriorated area of the pile (Browne et al. 2010). The encasement is performed by divers and can offer both a protective barrier and an increased structural capacity. For an increase in load bearing capacity, a reinforcement cage must be included in the jacket.

At material degradation from within the concrete, such as ettringite, reparation of the pile can be made by casting a new pile around the old[1]. The new pile is reinforced and designed to replace the entire bearing capacity of the old pile.

Pile jackets can also be installed before degradation has occurred in a preventative work. This is currently made in the port of Gothenburg where old piles without ice protection are equipped with pile jackets[1]. These jackets are applied in the splash zone and the concrete is cast in stay in place stainless steel forms which provide an extra layer of abrasion resistance from drift ice.

The installation of a pile jacket is performed by following the steps:

- Pressure washing the surface to remove marine growth and loose material (Browne et al. 2010).
- Applying a reinforcement cage around the pile if the structural capacity is in need of an increase.
- If necessary, dredging the mud at the bottom for reaching the clay. The reason to remove the mud is that it contains high amount of sulphates[1]. When the clay is reached a bottom plug is installed which is left in place after concreting.
- Installing the formwork which is made by temporary forms of steel or glass fibre reinforced composite material. These formworks consist of two half-cylindrical parts that are bolted together around the pile and centred with clamps, see Figure 10.
- Pumping the concrete into the form from the bottom up by pressure.
- Removing the formwork after the concrete is cured.

Figure 10: Cylindrical forms in steel for submerged concrete casting, (Robert Klein)

Pile jackets increase the size of the piles and thereby the self-weight and the effective area for current and wave loads also increases and the risk of buckling enhances. The main drawback with underwater concreting is to often fail in achieving the best result of the cast, especially in jacketing were thin layers of concrete are applied under pressure[1]. The jacketing requires a high experience and good workmanship from the divers and according to Thoresen (2014) lack of skill and expertise is one of the most common causes of damages during underwater casting of concrete. In the port of Gothenburg low downtime of the berths is of the essence, this creates a need for several diving crews working simultaneously on major reparations. Here the urge for short stoppage can compromise the quality of the repair due to lack of enough skilled and experienced diving teams. Installing the formwork around piles in a berth structure can be problematic where divers can't be assisted by cranes[4]. This creates a dangerous working environment where the form needs to be transported to the piles and lifted in place by the diver.

3.3 Common procedures in underwater repair

3.3.1 Surface preparation

Marine growth such as algae, barnacles, muscles and seaweed develops at the surface of submerged piles (McLeish 1994). This growth can both increase and decrease the integrity of the structure where algae growth can seal the concrete while seaweed has root systems that can grow into cracks and cause bursting actions. It is impossible to inspect and estimate damages in the piles without removing the marine growth and therefore it is removed before any inspection or reparation are performed (Browne et al. 2010), see Figure 11. A clean surface is essential to enhance the bond between the existing structure and the repair material in case of restoration. The cleaning is often made by pressure washing the concrete surface with high-pressure water jets[4]. Due to the surrounding water, the pressure decrease and the stream enlarge. Therefore, the jet tube must be held close to the concrete surface to remove marine growth and break out deteriorated concrete. For using high-pressure water jets efficiently, the divers must attach themselves to the pile. Otherwise, the force of the water stream pushes them away from the pile.

a) b)

Figure 11: a) Before surface preparation, b) after surface preparation

[4] Martin Lindgren, Dawab Sverige AB, interview 21 April 2017

3.3.2 Reinforcing

New reinforcement bars are applied around the piles to replace the corroded reinforcement or to increase the cross-sectional steel area so that the structural capacity of the pile can be restored or improved. When removing corroded reinforcement bars, hydraulically powered cutting tools can be used. It can be necessary to remove sound concrete to provide the new reinforcement with a sufficient splice or anchorage area (Browne et al. 2010). The removal of sound concrete can be reduced by using reinforcement bars with small diameters which require less splicing length and anchorage areas compared with bars with larger diameters. Reinforcement bars that are not replaced should be cleaned with a high-pressure water jet to remove rust. At smaller repair works, powered wire brushing can be used instead. For an increase of the service life of a concrete repair, a reinforcement mesh should be applied even if the present cross-sectional steel area is sufficient.

3.3.3 Casting concrete

The first method for underwater concreting was patented in 1910 by the Norwegian engineer August Gundersen (Thoresen 2014). The key was, and still is, to separate the fresh concrete and the water during the concreting and to prevent the cement to be washed out. This is a complicated process that should be performed by experienced divers to increase the quality of the work.

Divers carry out the casting of concrete underwater with the help of a hydraulic pump which creates a pressure to transport the concrete through a pipe into a form (Browne et al. 2010). The concrete used for underwater repairs is often low permeable and frost resistant. The form is filled from the bottom and upwards to prevent washout as the water floats upon the concrete and are pushed out from the form at the top.

3.4 Concluding remarks

To obtain and decide the need of repair for submerged piles, main inspections are carried out every sixth year. The process, from detecting the damage to performing the repair work is summarised in Figure 12. In the case of severe damages, the strategy and repair can be executed more quickly.

Figure 12: Main inspection every 6:th year. Inspection year one, strategy year two and repair year three.

The reparation of submerged piles can be made to restore the structural capacity and to restore the protective layer for degradation. Preventative work can also be undertaken to increase the service life of the submerged pile. These are summarised with the conventional repair methods in Figure 13.

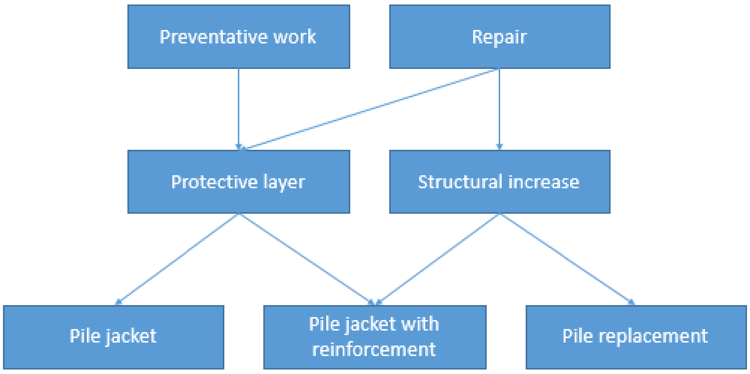


Figure 13: Map of repair methods and their purpose

The most common repair method is pile jacketing which is installed by a process that requires high experience and skilled workmanship. It is hard to guarantee a high quality of submerged concreting and the work is often carried out under tight time schedules to minimise the stoppage of the berth. In Figure 14 the advantages and disadvantages for pile jackets are shown.

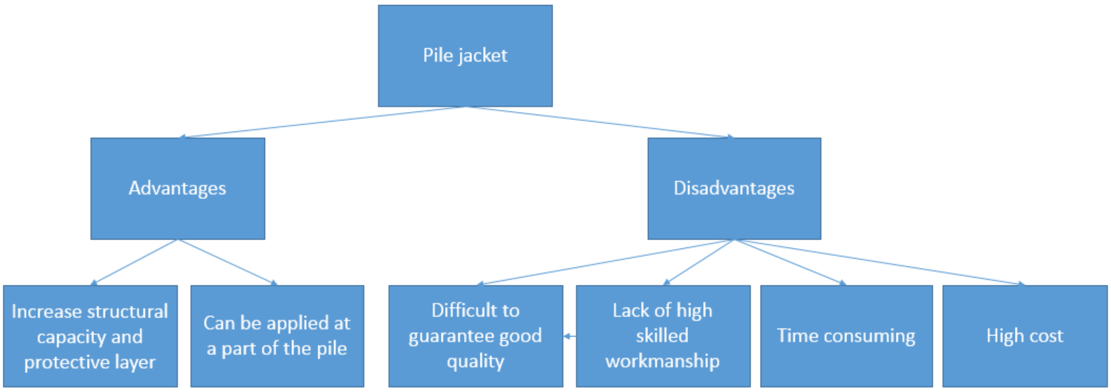


Figure 14: Pile jacket, advantages and disadvantages

The lack of skill for an underwater casting of concrete in thin layers has led to the urge of new repairing methods that are more efficient and easier to install. Restoring and strengthening piles by underwater concreting is a time-consuming technique and since the berths downtime is of essence the need of new repair methods that are sound, quick and durable has emerged.

4 FRP Repair of Submerged Piles

4.1 Introduction of the material

Fibre reinforced polymer (FRP) materials are composite materials made of fibre reinforcement embedded in a resin matrix. The fibres carry the load while the resin binds the fibres together, transfers loads between the fibres and provides protection to the fibres. FRP materials are generally anisotropic depending on the fibre orientation. By determining the arrangement of the fibres within the composite, specific material properties can be achieved which allows an optimal use of the FRP. The particular application of the FRP composite influences the choice of fibres and resin. In particular, the choice is affected by cost, surrounding conditions, required service life and needed strength (Au 2001).

4.1.1 Fibres

The properties of the FRP material are dependent on both the fibres and the resin. The fibres can either be in different forms, such as discontinuous or continuous fibres weaved randomly or in specific directions to achieve the desired properties. In structural applications continuous fibres are mostly used (Au 2001). The most common types of fibres used in the construction industry for FRP composites are glass, carbon and aramid fibres.

4.1.1.1 Glass fibres

There are three different types of glass fibres adapted for various applications: electrical, structural and corrosion glass. Electrical glass has great electrical insulation features and is the most used one in civil engineering because of its lower cost (Berver et al. 2001). Compared to electrical glass, structural glass fibres have higher strength and corrosion glass fibres have greater resistance to corrosive environments. Glass fibres are the most commonly used fibre type in civil engineering because it is more economical than carbon fibre and still can provide high strength.

4.1.1.2 Carbon fibres

In contrast to glass fibres, carbon fibres are electrically conductive. Carbon fibres have higher strength and are more brittle than glass fibres (Berver et al. 2001). Carbon fibres have high fatigue resistance but are five to ten times more expensive than glass fibre.

4.1.1.3 Aramid fibres

Aramid fibres, also known by the commercial name Kevlar fibres, are organic and have significant thermal resistance (Au 2001). Like carbon fibres, aramid fibres have high fatigue resistance and high strength. The impact resistance of aramid fibres is greater than for carbon fibres. However, the stiffness of aramid is considerably lower than the stiffness of glass. Drawbacks with aramid fibres are their tendency to poor bonding with most matrices and high absorption of moisture. Aramid is also sensitive to ultraviolet radiation. Thus, it is not preferable in harsh environments.

4.1.2 Resin Matrices

The matrix is considered as the secondary material in FRP and influences the transverse strength, shear and compression properties of the composite. Polymer matrices are classified as thermoplastics or thermoset depending on their structure and behaviour (Harichandran and Imad Baiyasi 2000). Thermoplastics soften or melt when heated and hardens when cooled. This is reversible and thermoplastics can be reshaped by reheating. For thermosets, the polymers cross-link and can therefore not be reshaped after curing and remain solid when heated. In contrast to thermosets, thermoplastics have higher impact strength and show better resistance to fracture and micro cracking. However, they are not commonly used in structural applications due to their ability to be reshaped.

Epoxy and polyester polymers are the most common used thermosetting resin matrices due to their good chemical resistance and good adhesion abilities. The polyester resin shows great mechanical properties and good environmental durability and is the most commonly used matrix because it is less expensive than epoxy (Berver et al. 2001). However, the adhesion for polyester to carbon and aramid is poor and the shrinkage during curing is significant. Thus polyester is not as widely used in the construction industry (Au 2001). Epoxy is superior to polyester regarding chemical and moisture resistance. Epoxy resins show excellent mechanical properties and provide better adhesion to more kinds of fibres than polyester and also good adhesion to substrates. Therefore, epoxy is the most commonly used resin for repairing civil structures, and the most expensive one.

Advancements in resin development have brought resins that can cure under water. These resins are refined in the sense that the amines, the hardener, are aromatic and water repellent, so that they react with the base and not water molecules[5] in contrastto conventional resins, where the hardener is based on aliphatic amines that are water-soluble. An example of such a resin is Fyfe's Tyfo® SW-1 underwater epoxy. There are also resins that are water-activated and cure upon contact with water, such as the proprietary urethane resin system in the Aquawrap® by Air Logistics which cures through a chemical reaction with water.

4.2 Reasons to use FRP

FRP composites have since the 80's been a promising material for repair and strengthening of concrete structures in civil engineering due to its advantageous properties compared to other traditional construction materials (Sen and Mullins 2007b). The development of resins that can be applied and cured in water has made the repair and strengthening with FRP composites a valid solution also for submerged structures.

Compared with concrete jacketing, which is the most common repair method for submerged concrete piles, there are several advantages of using FRP composites instead. The composites high strength to weight ratio, which is several times greater than for steel, and stiffness to weight ratio, lead to that repair and strengthening of structures can be made without any substantial addition of weight or sectional area

[5] Jonny Augustsson (jonny@nilsmalmgren.se) (8 May 2017) Fråga till NM Lab. Personal mail to Johan Röös (joroos@student.chalmers.se)

(Berver et al. 2001). Before curing, FRP materials are very flexible, which allows for fit to any shape of the existing structure. Since FRP materials are lightweight and easy to manage, they can provide a quick and easy installation without the need of heavy machinery or tools which are required to install the heavy formwork for concrete jacketing.

FRP is a dense and durable material and has excellent resistance to harsh environments such as in marine conditions. By wrapping concrete piles with FRP materials the migration of water, oxygen and ions into the concrete can be reduced and thus the corrosion process and the formation of ettringite can be impeded (Wootton, Spainhour, and Yazdani 2003). FRP wraps also provide confinement which will keep the concrete from spalling due to the expansive forces caused by corrosion or ettringite. Confinement of the piles also increases the axial strength and ductility due to increased lateral pressure.

FRP materials can cure rapidly, which makes them beneficial for emergency repairs where fast restoration is of the essence. Even though the initial costs of FRP materials are high, the quick and easy installation means lower construction costs and minimal time for the structure to be closed. Thus, the total cost of repairing a structure with FRP may be lower than for other methods.

4.3 Limitations of FRP composites

The long-term behaviour is not well known but what is understood is that the properties of FRP composite materials are affected by temperature and moisture content variations and ultraviolet radiation.

The resistance of FRP composites to weathering is highly reliant on the resin matrix. The most notable cause of degradation of resins is ultraviolet radiation as it causes crosslinking degradation of the resin (Au 2001). This results in chalking of the resin and making it more brittle.

High temperature variations increase the water absorption for FRP composites and moisture leads to bond failure between the fibres and the matrix (Harichandran and Imad Baiyasi 2000). Glass fibres are liable to moisture attack as they swell when absorbing moisture causing expansive internal forces in the composite and moisture also causes strength reduction of the fibres.

4.4 FRP repair method

The intention with an FRP repair method is to provide a quick and efficient repair with externally bonded FRP composites. It can be applied along the whole height of the pile or just over a particular area, such as the splash zone. The idea is that the low permeable FRP material with good resistance to harsh environments will provide the pile with a protective layer and thus increase the durability of the pile. Instead of replacing or installing new reinforcement the structural capacity can be restored or improved by the externally bonded FRP material. The usage of resins that can cure underwater allows the wrap to be applied by divers at the submerged area of the pile.

4.4.1 State of practice

The state of practice for FRP repair of submerged concrete structures has been reviewed. The focus is put on the method and the on-site application.

4.4.1.1 Friendship trails bridge

In (Sen and Mullins 2004) a field demonstration project in the application of FRP composites for underwater pile repair has been performed. The project was carried out on the Friendship Trails Bridge, which is part of a bridge network that spans Tampa Bay between St. Petersburg and Tampa. The Friendship Trails Bridge was built in 1956 for vehicular traffic and the original plan was to demolish it in 1997 due to the construction of the new Gandy Bridge but instead, it was converted to a pedestrian bridge. The environment in Tampa Bay is harsh which has led to that 77 percent of the piles in need for repair due to corrosion.

In this study, eight piles were selected, two for control and six for a demonstration of the FRP pile repair method. The selected piles were equipped with two probes each, see Figure 15, for comparing the corrosion rate by measuring the galvanic current between the two sensors.

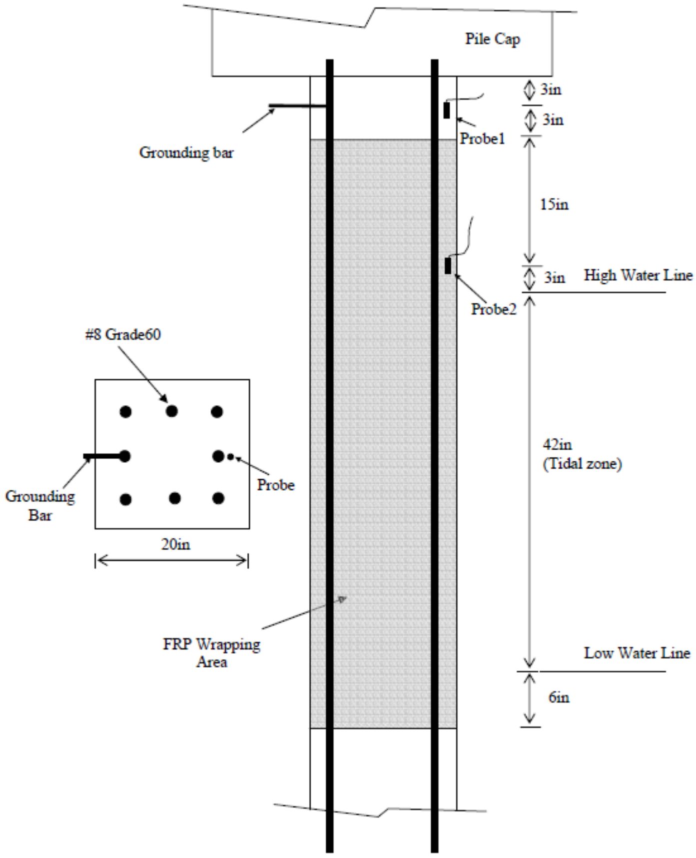


Figure 15: FRP wrapping area and probe placement (Sen and Mullins 2007a)

Two repair system were used, a pre-preg and a wet layup system. The pre-preg system, which is called Aquawrap® and developed by Airlogistic, uses a water-activated resin pre-impregnated in the FRP material, in this case both carbon fibre reinforced polymer (CFRP) and glass fibre reinforced polymer (GFRP). The wet layup system, developed by Fyfe, consists of Tyfo SEH-51A glass fibre weave and Tyfo SW-1 underwater epoxy. The design of the wrap was made from an estimated capacity loss of 20 % due to corrosion, see Figure 16. This resulted in the configuration of FRP showed in Table 1 with the material properties in Table 2.

Table 1: Repair system used for different piles.

Repair system	Specimen type	Longitudinal	Transverse
Pre-preg/Aquawrap	Carbon	1 layer	2 layers
	Carbon	1 layer	2 layers
	Glass	2 layers	4 layers
	Glass	2 layers	4 layers
Wet layup/Tyfo	Glass	2 layers	4 layers
	Glass	2 layers	4 layers

Table 2: Material properties Aquawrap and Tyfo SEH-51A

Fabrics	Composite	Tensile strength [MPa]	Tensile modulus [GPa]
Aquawrap	Uni-directional glass fibre	586	35.9
	Bi-directional glass fibre	324	20
	Uni-directional carbon fibre	827	75.8
	Bi-directional carbon fibre	586	22.0
Tyfo SEH-51	Uni-directional GFRP	460	20.9

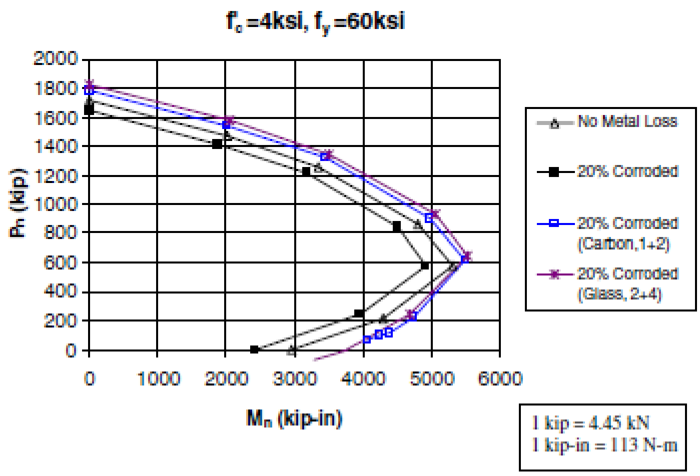


Figure 16: Interaction diagram for corrosion repair of piles (Sen and Mullins 2007a)

For accessing the piles, a lightweight four-parted scaffolding system with cut-outs for the piles was built and installed. The scaffolding was suspended from the pile cap, see Figure 17 at a height that allowed the wrap to be applied from the pile cap and 1,84 m down.

Figure 17: Scaffolding system suspended from the pile cap (Sen and Mullins 2007a)

With the access system in place, surface preparation was performed before wrapping. Marine growth was removed with a scrapper and projecting parts of the concrete were removed with a hammer and a chisel. Because the cross-sections of the piles were rectangular, all four corners were ground to a 19 mm radius with an underwater pneumatic grinder to avoid stress concentrations in the wrap. Voids in the surface of the piles were filled with a quick-setting hydraulic cement to provide a smooth surface. Just before wrapping the final preparation was to pressure wash the surface to remove dust, debris and marine growth.

The pre-preg system was pre-impregnated with resin at the factory and arrived at the work site in hermetically sealed pouches. Before the bags were opened Aquawrap® base primer 4 was applied by hand on the prepared surface. The carbon fibre was used in a longitudinal layer of uni-directional fibre, see Figure 18a, followed by two transverse layers of bi-directional fibres, see Figure 18b. The transverse layers were spirally wrapped around the pile without overlap. For the glass fibre wrap the same process as the carbon fibre was used and repeated because of the double number of layers. To consolidate the wrap and provide a better finish both the CFRP and GFRP system was applied with a glass fibre veil that was wrapped with a 50 mm overlap. For maintaining the wrap in place while curing, a plastic shrink film was used. The whole wrapping procedure was made in less than one hour a pile. This was within the working time of the Aquawrap® which according to (“Aquawrap®” 2017) is one hour. After a day of curing the plastic film were removed and the wrap was painted with base primer 4 for protection against UV radiation.

a) b)

Figure 18: a) Applying the longitudinal CFRP layer (Sen and Mullins 2004), b) Applying the second transverse CFRP layer (Sen and Mullins 2007a)

In the wet layup, the resin was impregnated by hand on-site which provided higher flexibility of wrap lengths but also more planning of the logistics. One of the two piles were wrapped using different epoxies for the dry and the submerged region. However, this pile was re-wrapped three months later because of poor bonding in the dry area. The weak bonding was a result of wave action and splashing, which made the dry region wet. For the re-wrap, the epoxy was impregnated in the fabric with a resin impregnator, see Figure 19, and the wrapping procedure was the same as for the pre-preg system.

Figure 19: On site saturation of fibreglass fabric (Sen and Mullins 2007a)

The galvanic current was measured during 160 days and due to that the currents were very small and inconclusive no conclusions could be made about the effect wrapping of piles have on corrosion rate (Sen and Mullins 2004).

Two years after the wrapping the bond strength was tested with a pull-out test (Sen and Mullins 2007a). The results showed that the bond was poor with inter-layer failure between the FRP layers for the pre-preg system and epoxy failure between the FRP and concrete for the wet-layup. It also showed that the pre-preg system performed better in the dry region and that the wet layup performed better in the wet region.

Rajan and Mullins conclude that the FRP repair system is viable for repairing of corroded piles. The pre-preg system was easier to use than the wet layup, but the wet layup provided greater flexibility than the pre-preg system. They also report that the utilised scaffolding system in the field demonstration was inexpensive and quickly fabricated.

4.4.1.2 Allen Creek Bridge

In Mullins et al. (2005) a field study is carried out to demonstrate underwater FPR repair of prestressed piles. The study was made on Allen Creek Bridge which was constructed in 1951 and widened to six lanes in 1982. The piles of the old part of the bridge are 500x500 mm reinforced concrete piles and those of the widened section are 350x350 mm prestressed concrete piles. The bridge is in the city of Clearwater and the water in the creek flows into Old Tampa Bay. Due to the harsh environment, the piles have been repaired several times.

Four of the 350x350 mm prestressed piles were selected for the study, two as control piles without wrap, one with glass fibre and one with carbon fibre with the Aquawrap system. As in (Sen and Mullins 2007a) the pre-preg system arrived at the work site pre-impregnated with resin in hermetically sealed pouches and with the same material properties as for the Aquawrap® in Table 2. The wrapping area of the pile was from 230 mm above the high-water line to 230 mm below the low water line, see Figure 20.

For measuring the corrosion rate, a probe system was installed before the application of the wrap. The system is shown in Figure 21, and it measures the change in corrosion potential due to the application of an incremental current or voltage. From the change in potential, the corrosion rate can be determined by linear polarisation.

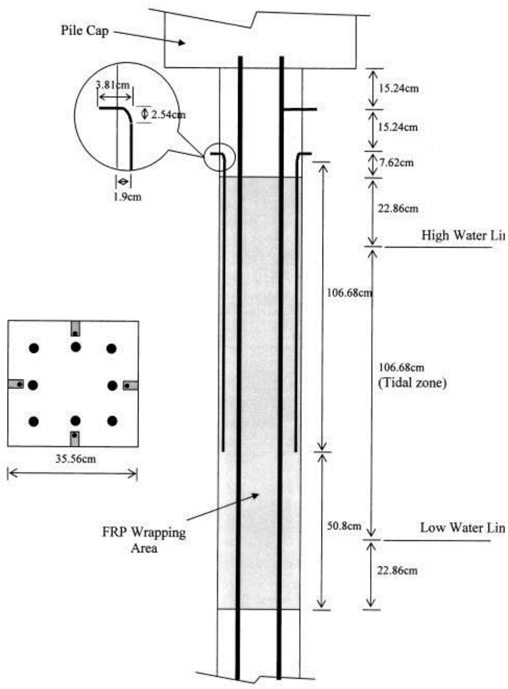


Figure 20: FRP wrapping are of pile

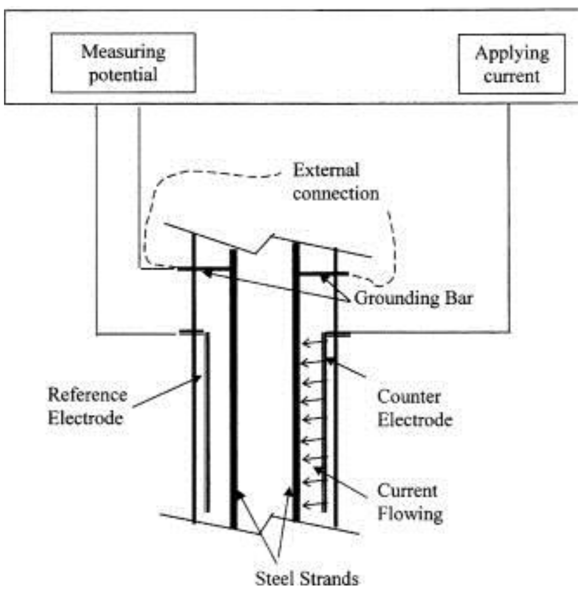


Figure 21: Setup for measuring the corrosion rate (Mullins et al. 2005)

The designing of the wrap was made in two steps. First, a strain compatibility analysis was carried out for restoring a 20% strain loss. Second, this layup was checked if it could withstand post repair expansion due to corrosion.

Before the application of the wrap, surface preparation was made by the same procedures as in (Sen and Mullins 2007a). But because the piles were in shallow waters, the work could be executed standing in the water without a scaffolding system or divers.

In the study, application of the carbon fibre composites was performed with one longitudinal and two transversal layers. The first longitudinal layer was applied to the pile in pre-cut 305x1524 mm long strips. These were placed on each side of the pile which was pre-saturated with an underwater epoxy adhesive. The bi-directional carbon fibre was then wrapped transverse and spirally from the top and down without overlap. When the bottom was reached the next layer of carbon fibre was wrapped in the same manner but starting from the bottom and up. For ease of wrapping rolls of 305 mm wide and 5-6 m were used to apply the transverse layers. Water was sprayed at the area located above the water to activate the resin. To provide a better finish and conceal the wrap a 254 mm wide pre-preg veil of glass fibre were applied from the bottom up with an overlap of 50 mm. Lastly, a plastic stretch film was used for securing the wrap during curing. After one day of curing the plastic stretch film was removed and the wrap was painted with an ultraviolet radiation protective paint. The entire wrapping was made within one hour per pile.

The application of the glass fibre was made with the same procedure as for carbon fibre except for that the number of layers was doubled.

After 413 days, the study showed that the corrosion rate of the wrapped specimens was lower than for the control samples. The results for carbon fibre and glass fibre were comparable. Mullins et al. (2005) conclude that underwater wrapping is a viable system. The study also showed that the bond between the wet concrete and the FRP was poor. However, laboratory tests showed that the bond strength was sufficient for the FRP system to restore the capacity of the pile to the original.

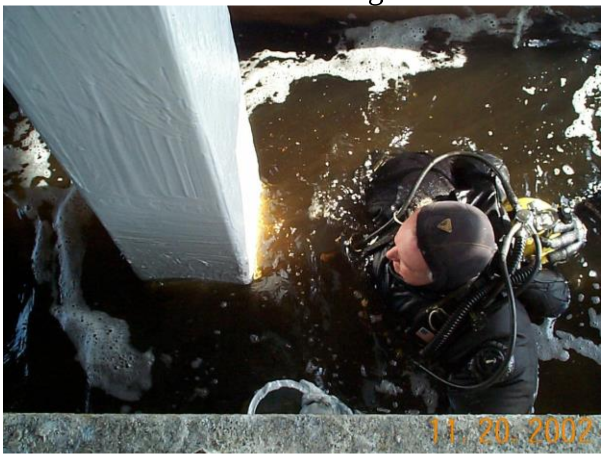
4.4.1.3 Innovative FRP piling repair without the use of cofferdams

Two 305 mm square piles of a bridge over North River in North Carolina were restored with an FRP system according to (Bazinet, Cerone, and Worth 2002). The bridge, located in tidal waters was in need of repair to reduce further degradation due to spalling from freeze/thaw cycles, splash zone water flow and corrosion of reinforcement.

The chosen repair system was a pre-preg system consisting of three layers of bi-directional glass fibre and one layer of a veil. In the case of rectangular piles as in this study, an epoxy layer must be applied between the concrete and the wrap. If the piles instead are circular the bond strength provided by the pre-impregnated resin is adequate on its own. The system had no volatile organic compound and was approved by NSF International, which is a public health and safety organisation, as environmentally friendly.

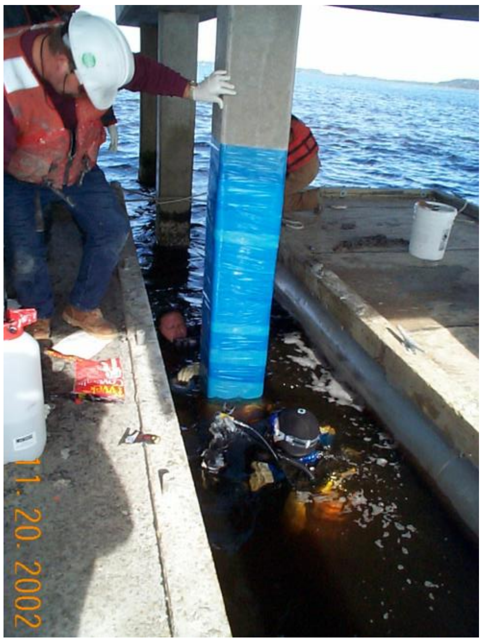
This system was applied from 350 mm below the low water line and up to the pile cap resulting in an area along 2.44 m and 1.83 m of the first respectively the second pile. The installation was made by first preparing the concrete surface. For the first pile, this was made by thoroughly cleaning and patching the surface while the second pile only was cleaned and marginally patched. When the surface preparation for the piles had been performed, an underwater epoxy was applied to the concrete surface, see Figure 22a. The wrap was then applied using 200 mm wide rolls starting from the top of the pile. One turn was wrapped around the top then the wrapping continued spirally downwards without overlap. At the bottom one turn of wrapping was made before proceeding back up. This was made until three layers were applied see Figure 22c. Above the waterline, water had to be sprayed on the surface before wrapping to activate the resin. To consolidate the wrap and provide a better finish a 250 mm wide veil was wrapped with a 50 mm overlap. During curing the pile was also wrapped with a temporary plastic stretch film, see Figure 22d. After removal of the stretch film, a layer of epoxy adhesive was applied to the wrap as a protective coating. The entire operation was made in less than a day by a crew that were new with this installation. The total man hours were eighteen for the contractor staff and six for the diving team.

a)



b)

c)



d)

Figure 22: Installation of Aquawrap repair system, a) epoxy applied by hand above water, b) epoxy applied by divers below water, c) divers wrapping bi-directional GFRP, d) divers wrapping the plastic stretch film[6]

[6] Franz Worth (fworth@airlog.com) (11 May 2017) Report of splash zone repair – Master thesis. Personal mail to August Uddmyr (uddmyr@student.chalmers.se)

4.4.1.4 54” OD bridge pile reinforcement & repair

In 2012 a barge collided and damaged several 54” OD concrete piles of Chesapeake Bay Bridge (Neptune Research Inc 2012). The damages were located in the splash zone, in which the concrete cracked and exposed the reinforcement, see Figure 23. Site conditions as tidal currents and rough waves increased the difficulties using traditional repair methods.

Figure 23: Damaged pile at Chesapeake Bay bridge (Neptune Research Inc 2012)

The piles were repaired with Neptune Research Incorporate (NRI) product Titan-218 carbon fibre with Titan saturant epoxy. The application was made by pressure washing the surface, resurface the concrete with a cementitious concrete patch, applying the epoxy to the surface and impregnating the carbon fibre, see Figure 24a. The carbon fibre was bi-directional and wrapped spirally in two layers from 1.8 m above to 1.8 m below the waterline, see Figure 24b. During curing the wrap was secured with a plastic shrink wrap, see Figure 24c.

a) b) c)

Figure 24: a) saturating carbon fibre with epoxy, b) application of the CFRP to the pile, c) applying plastic shrink wrap to keep the wrap in place during curing (Neptune Research Inc 2012)

Repairing with the Titan system was according to Neptune Research Inc (2012) an effective repair method adding structural integrity to the damaged piles. It was made with minimal resources without disturbing the traffic.

4.4.1.5 Carbon FRP repair of underwater bridge piers

In 2012 QuakeWrap repaired the piles of Bay View Bridge. The piles were partially submerged and subjected to several wet/dry cycles a day due to natural ocean tide (QuakeWrap 2012). This caused severe corrosion in the splash zone of the reinforced concrete piles.

QuakeWrap FRP repair system consisting of carbon fibre and their QuakeBond J333 epoxy resin was used to repair these piles. This system was chosen to achieve a long-term and maintenance-free solution. The system was applied from the pile cap to a couple of feet below the low water line. To provide a better finish the surface was painted.

The reparation was made in 2007 and five years later, 2012, QuakeWrap revisit the bridge. They observed marine growth and some paint loss in the submerged region, but no damages to the FRP system, see Figure 25.

a) b)

Figure 25: CFRP repair of piles, a) directly after installation, b) revisit five years later (QuakeWrap 2012)

4.4.1.6 Port of Gothenburg

In the port of Gothenburg, full scale tests have been carried out on piles with a repair system called pile cap[1]. This system consists of a glass fibre fabric with an epoxy grout and it was sealed with Syntho-glass. Syntho-glass is a pre-impregnated glass fibre that is water activated that originally have been used for sealing piles.

The application of the system was made by divers that had been certified on the system under training in a controlled environment[1]. It was carried out on three “out-of-action” piles that could be removed and sent for analysis at RISE which is a technical research institute in Sweden. Testing was made on strength, permeability and soaking leaching under accelerated long-term testing to determine the long-time behaviour. The result of the pile cap system with Syntho-glass was an increase of seven years in the service life of the piles. This was regarded as an insufficient increase of the service life and the Port of Gothenburg are currently investigating two other alternatives both involving Syntho-glass. The first option is to wrap the first layer with a thick stretch film followed by Syntho-glass and the second is to wrap the pile with Denso tape and Syntho-glass[1].

4.5 Limitations for repair with FRP

Wrapping concrete piles with FRP could lead to trapping moisture and chlorides inside the concrete and because of the wrap the concrete may not be able to dry out and thus, enhance the corrosion process. If the pile isn’t completely wrapped moisture may find its way behind the wrap, through the concrete, and deteriorate the bond between the FRP and concrete. Moisture trapped behind the wrap can upon freezing increase considerably in volume and also damage the wrap. Since the wrap surrounds the pile, it may be difficult to detect what is going on behind it by visual inspection.

Abrasion and physical damage can cause cracks and fracture in the wraps making them ineffective as it leads to easier access for moisture. Exposure of the FRP wrap to ultraviolet radiation acts as a synergy effect as it causes embrittlement of the resin and thus lower crack resistance upon impact.

The surface preparations of piles to be repaired with FRP wraps are of great importance to achieve an efficient repair with a good bond. Depending on the particular case this could be a limitation of the FRP repair method as proper grinding and smoothening of the surface can be a time-consuming procedure.

As the resin protects the fibres, it is important to provide the resin with protection. Therefore, to prohibit degradation of the composite, there is a need for protective coatings.

4.6 Literature review of experimental studies

Several other research projects have been made to evaluate the durability of FRP wrapped reinforced concrete. This section focuses on experimental studies that have investigated the durability by subjecting specimens to accelerated corrosion by an impressed current and by subjection specimens to wet/dry and freeze/thaw cycles. In addition, it includes studies on the bond between FRP wraps and concrete.

4.6.1 Corrosion potentials of lightweight concrete wrapped with fibre reinforced polymers (FRP)

Goucher (2013) studied how wrapping reinforced concrete with FRP would affect the corrosion for both lightweight and normal concrete. Forty-two concrete cylinders with a diameter of 50 mm and a height of 100 mm with one single reinforcement bar in the centre were subjected to an accelerated corrosion test by an impressed current. The samples were submerged in 5% saltwater and the test lasted for 50 days. As a sample failed, it was removed from the test and analysed considering reinforcement bar mass loss and time to failure. Failure incorporated current spike due to concrete cracking, FRP separation from the concrete and rupture of the FRP. Goucher used three different FRP systems. One glass FRP Tyfo SEH-51A system, one carbon FRP Tyfo SCH-41 system and one carbon FRP SikaWrap Hex 103C system. To declare for the legend in the figures in this section, the labelling of specimens was according to: LW or NW (lightweight or normalweight concrete), G or C (glass or carbon FRP), F or S (Fyfe or Sika) and 1L or 2L (1 or 2 layers of wrap).

The research showed that FRP wrapped specimens, both lightweight and normal concrete, had better protection against corrosion than unwrapped specimens, see Figure 26. Samples with two layer wraps performed better than samples with one layer wraps. Overall wrapped specimens had longer life span and less reinforcement bar mass loss than those who were unwrapped, see Figure 27 and Figure 28.

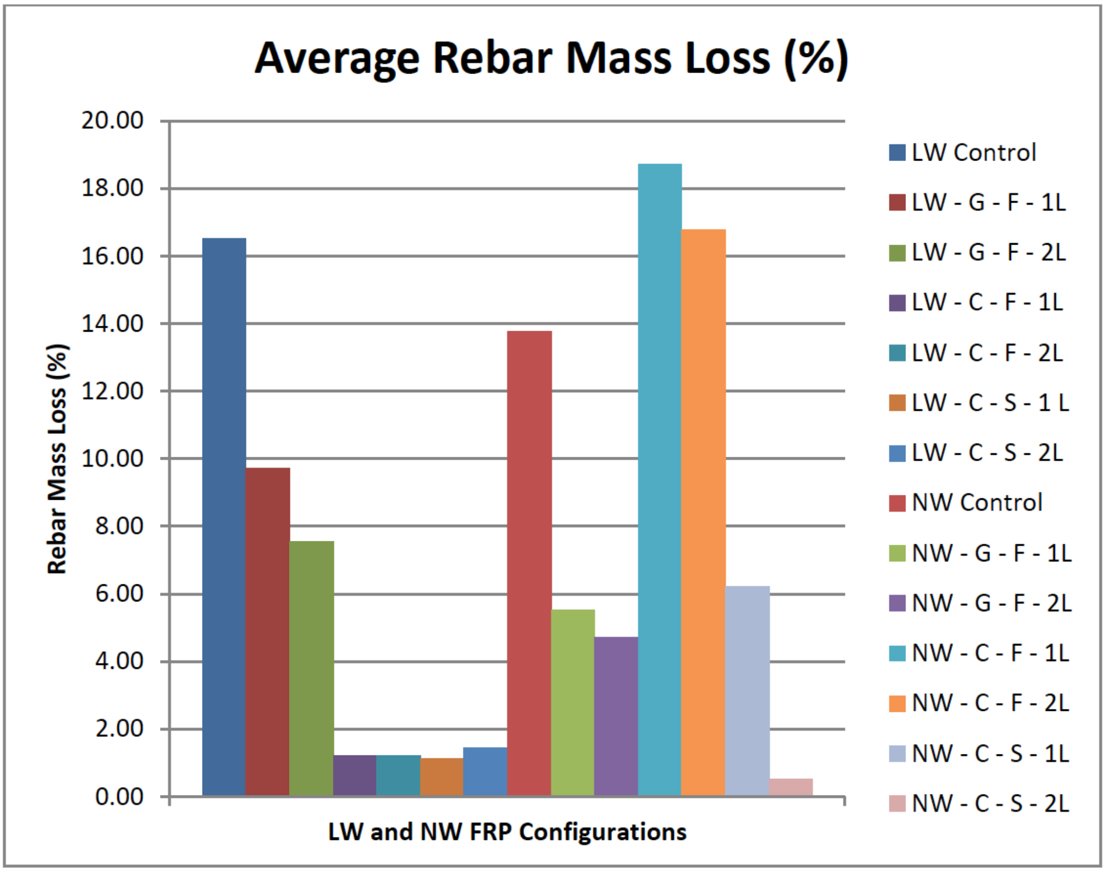


Figure 26: Average reinforcement bar mass loss (Goucher 2013)

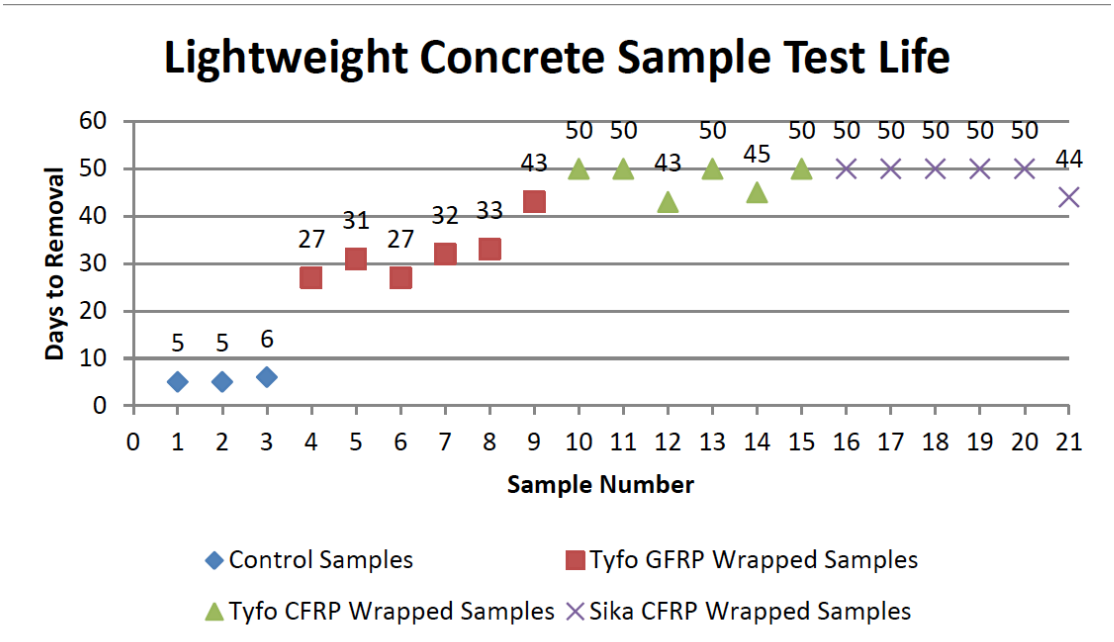


Figure 27: Lightweight concrete sample test life (Goucher 2013)

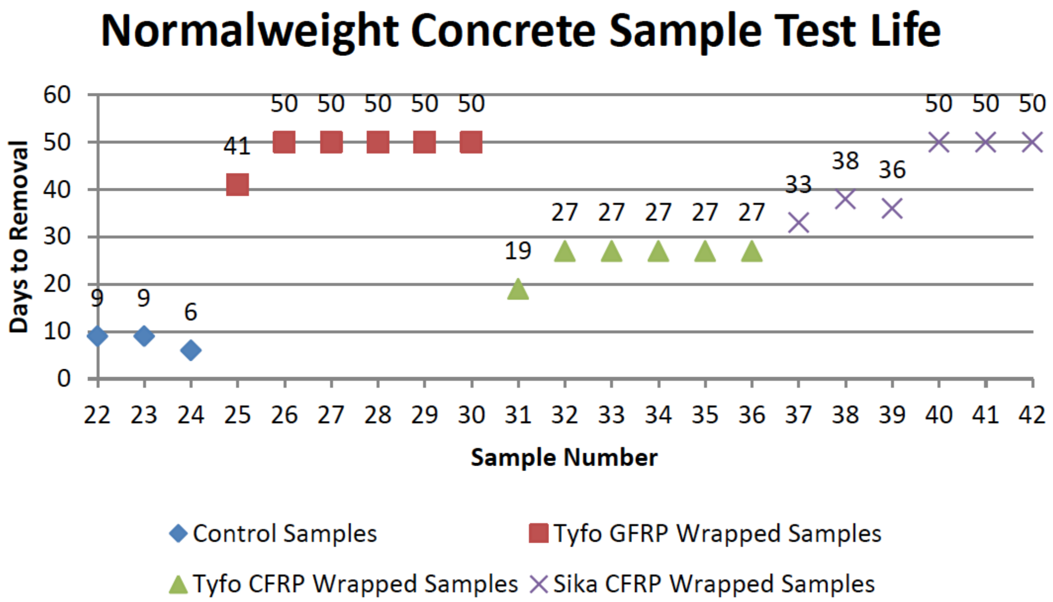


Figure 28: Normalweight concrete sample test life (Goucher 2013)

4.6.2 Repair of corrosion-damaged columns using FRP wraps

Baiyasi et. al. (2000) studied the effects of repairing corrosion damaged concrete with FRP wraps of both carbon and glass. The durability of FRP wrapped concrete specimens subjected to freeze/thaw and the corrosion rate for such specimens through accelerated corrosion test were examined. Cylindrical test specimens with dimensions of 152 mm diameter and 305 mm height were tested. The samples were provided with four reinforcing bars and the test was set up so that two of the bars were anodic and the other two cathodic. 24 concrete cylinders were submerged in 3% saltwater and exposed to accelerated corrosion for 13 days before being wrapped. After wrapping the accelerated corrosion was resumed and specimens were removed from the test at two different stages, after 130 days and 190 days, and analysed considering the reinforcement bar mass loss. The test showed that FRP wraps effectively reduced the corrosion rate by up to 59% compared to unwrapped specimens and that carbon and glass FRP impedes the corrosion equally. They also examined the effect of bonded and un-bonded FRP wraps and concluded that unbonded specimens had 20% higher corrosion rate than bonded specimens. The freeze/thaw test was made on 30 specimens similar to those in the accelerated corrosion test. The specimens were subjected to 300 freeze/thaw cycles and an expansive internal force, created by filling a longitudinal centric hole in the specimens with expanding cement, simulating the internal force by corroding steel. After the freeze/thaw cycles, the samples were subjected to compression strength tests. The study showed that the freeze/thaw environment did not cause any substantial damage to the FRP wrapped specimens as there was no significant difference in compression strength compared to the wrapped specimens which weren't subjected to freeze/thaw.

4.6.3 Corrosion of steel reinforcement in carbon fibre-reinforced polymer wrapped concrete cylinders

Spainhour et. al. (2003) studied how the application of carbon FRP wraps affects the corrosion of reinforcing steel in concrete. Different types of epoxy, the orientation of the fibre wrap and different numbers of layers were studied. A total of 42 cylindrical concrete specimens with diameter of 51 mm and a height of 102 mm reinforced with one steel bar in the centre of the specimen were used for the test. The test incorporated accelerated corrosion through an impressed anodic current and the concrete specimens together with the cathodes, consisting of steel bars, were placed in 5% saltwater. The accelerated corrosion test run continuously and as a sample failed it was removed. Failure included cracking of concrete, a spike in current flow or disruption of the wrap. The mass loss of the reinforcement bar and the chloride content of the failed specimens were then studied. The research showed that samples wrapped with FRP had beneficial measurements regarding service life, mass loss and chloride content compared to unwrapped samples. They conclude that corrosion of steel in concrete is impeded by the wrapping of carbon FRP composites. The research shows that wrapped specimens have up to 375% longer life span than unwrapped and that unwrapped specimens have over seven times higher corrosion rate than wrapped samples. They also conclude that samples with two layer wraps performed better than samples with one layer wraps, but increasing the wraps to three layers showed no considerable improvement compared with two layers.

4.6.4 Corrosion of steel reinforcements embedded in FRP wrapped concrete

Gadve et. al. (2009) studied the further development of corrosion in reinforced concrete after wrapping the concrete with both carbon and glass FRP. The research consisted of concrete cylinders subjected to accelerated corrosion by an impressed current. The specimens were cylinders with a diameter of 100 mm and a height of 230 mm with one reinforcement bar in the centre. The cathode consisted of a stainless-steel mesh. The cathode and the concrete cylinders were submerged in 3.5% saltwater and then exposed to anodic current. Before wrapping the specimens with FRP they were corroded in three different categories, pre-corrosion of specimens for 2, 4 and 8 days, respectively. This was done to model alternative corrosion damage before wrapping. After the specimens were wrapped, they were subjected to further accelerated corrosion for 24 days. At the end of the test, the samples were analysed by reinforcement bar mass loss and pull-out tests to study the effects of FRP wraps on the corrosion of steel. The pull-out force for specimens with FRP wraps was significantly higher than for unwrapped specimens, whereas the reinforcement bar mass loss was significantly lower for wrapped specimens compared with unwrapped specimens. Figure 29 shows the reinforcement bar mass loss versus pull-out strength. In the test, they also evaluated the resistance of the specimens as higher resistance indicates better protection of the reinforcement bar by subjecting the samples to a constant current of 100 mA and measuring the cell voltage. Figure 30, Figure 31 and Figure 32 shows the cell voltages for the specimens wrapped after 2, 4 and 8 days respectively. From the results, they concluded that applying FRP wraps on concrete with embedded steel impedes the rate of corrosion.

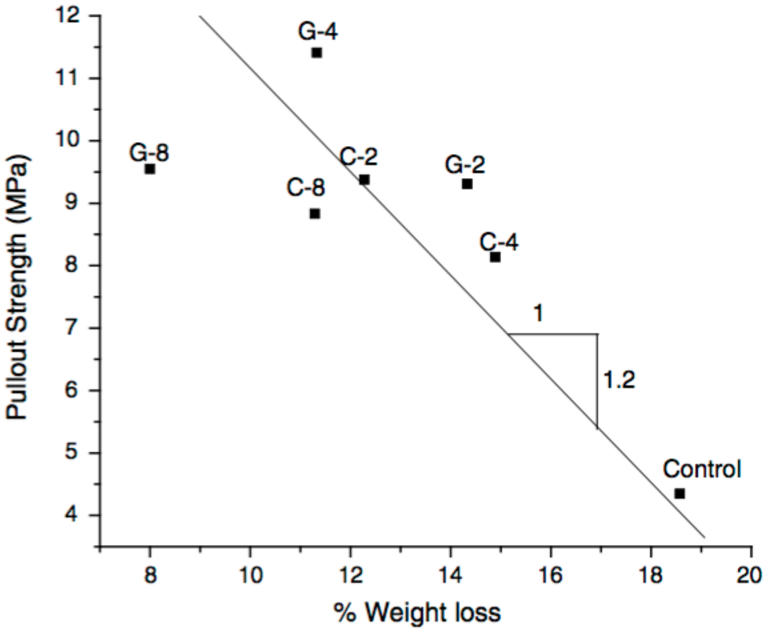


Figure 29: Variation of pull-out strength with % reinforcement bar mass loss (Gadve et al. 2009)

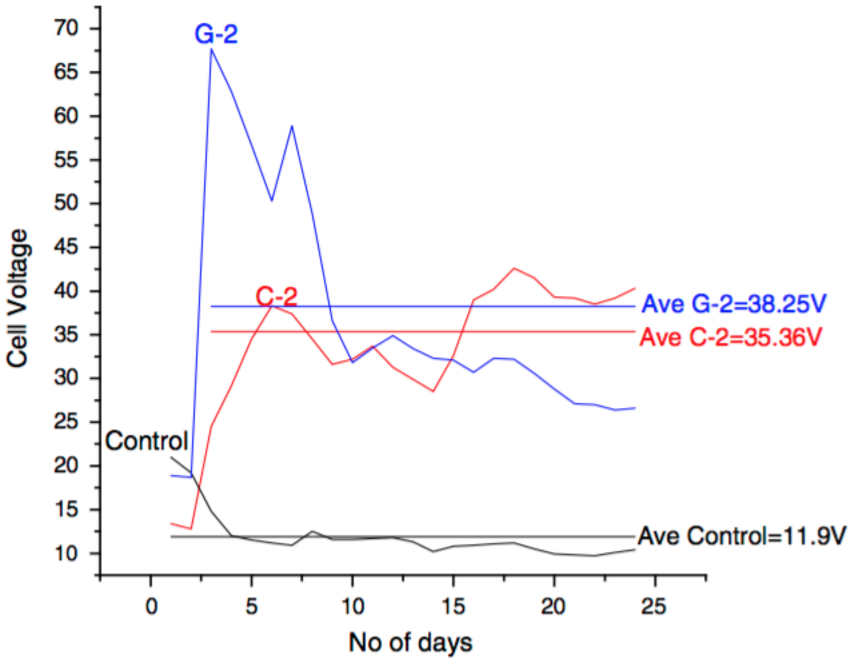


Figure 30: Cell voltage for specimens wrapped after 2 days with glass (G-2) and carbon (C-2) FRP (Gadve et al. 2009)

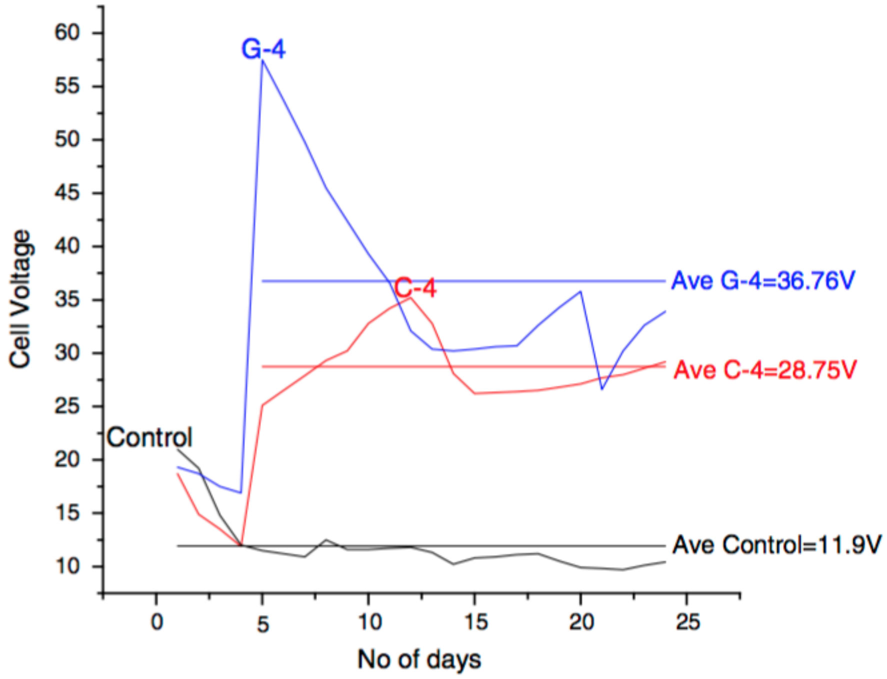


Figure 31: Cell voltage for specimens wrapped after 4 days with glass (G-4) and carbon (C-4) FRP (Gadve et al. 2009)

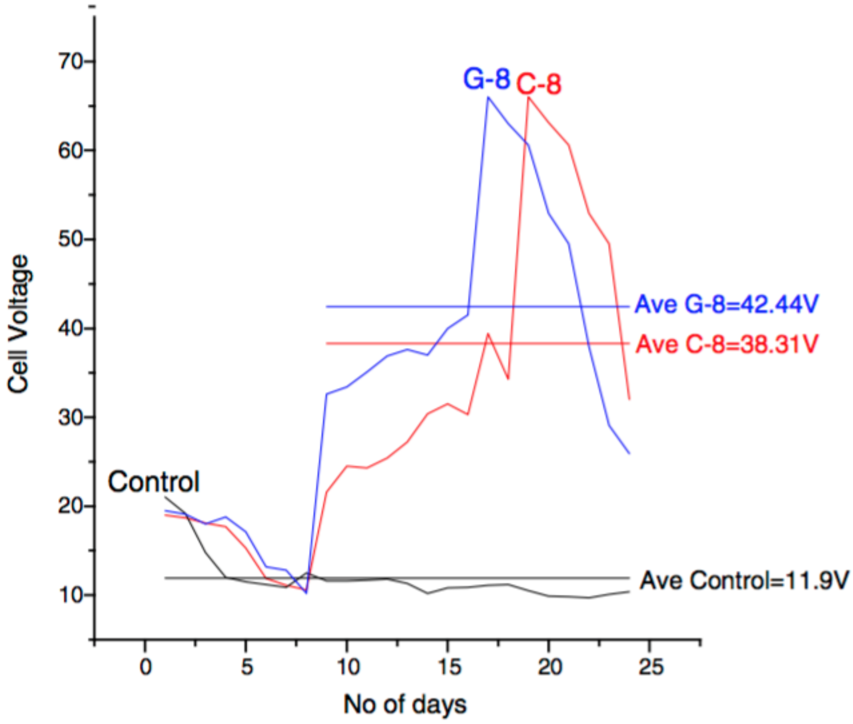


Figure 32: Cell voltage for specimens wrapped after 8 days with glass (G-8) and carbon (C-8) FRP (Gadve et al. 2009)

4.6.5 Effectiveness of fibre-reinforced polymer in reducing corrosion in marine environment

Suh et al. (2007) investigated the effectiveness of FRP in reducing corrosion in a marine environment. This was made by accelerated long-term durability testing on twenty-two pre-stressed specimens with the cross-section of 150x150 mm and length of 1520 mm, see Figure 33. The splash zone was fabricated along 550 mm of the centre of the pile with a 3% chloride content by weight. To allow potential measurements inside the wrapped region, the specimens were made with embedded reference electrodes which can be seen in Figure 33.

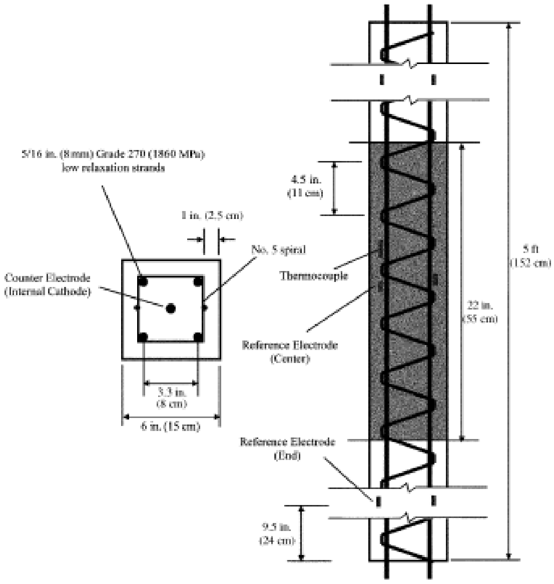


Figure 33: Geometry of specimens (Suh et al. 2007)

The wrapping was made over a length of 910 mm at the centre of the specimen, extending 175 mm above and below the chloride contaminated area. Sixteen specimens were wrapped with 1-4 layers of FRP, eight with CFRP and eight with GFRP. The accelerated long-term durability testing was made during three years of tidal simulations in a 3.5% salt water solution. Twenty specimens were placed in an outdoor tank and two of the control specimens were placed in an indoor tank to provide a controlled environment. The tide in the tanks were changed every 6 hours where the water level, from the bottom, changed between 800 mm at high tide to 350 mm at low tide. The corrosion rate was measured with linear polarisation during the test. The result showed that the average corrosion rate of the control specimens was 0.018 mm/year while the corrosion rate of the wrapped specimens was 0.0055 mm/year. Both the linear polarisation and gravimetric testing showed that CFRP and GFRP were equally effective in reducing corrosion. The result of the gravimetric testing, see Table 3, also showed that two layers of FRP were optimal. Another discovery made was that 30 wires in the six unwrapped specimens had breakage due to localised corrosion while only one wire had breakage among the sixteen wrapped specimens.

Table 3: Average metal loss, gravimetric testing (Suh et al. 2007)

Specimen type	Layers	Metal loss [%]	
		Strand	Tie
Outdoor	0	6.6	10.1
Indoor	0	6.6	8.9
CFRP	1	3.5	7.1
	2	3.1	5.7
	3	3.4	6.9
	4	3.3	6.9
	Average	3.3	6.9
GFRP	1	3.6	6.7
	2	3.3	6.2
	3	3.5	5.9
	4	3.3	6.5
	Average	3.4	6.3

The bond between the FRP and the concrete was tested after the tidal simulations. This was made with pull-out tests on four CFRP specimens and four GFRP specimens with one, two, three and four FRP layers. The test was performed in three different zones, the dry zone, tidal zone and the submerged zone. At the dry and tidal zones, most bond failures occurred in the concrete. In the submerged zone, most bond failure occurred in the epoxy. However, the average bond for the CFRP, 1.8-2.0 MPa, and the GFRP, 1.8-2.1 MPa were similar and largely unaffected by exposure. The number of layers did not show any difference in bond strength indicating that the interlayer bond between the FRP layers was good.

4.6.6 Bond enhancement for FRP pile repair in tidal waters

Winters et al. (2008) studied if pressure bagging and vacuum bagging could enhance the bond for FRP pile repair in tidal waters. The idea is to provide a pressure that ensures contact between the FRP and concrete during curing and thus enhance the bond. The schematic of the two systems can be seen in Figure 34.

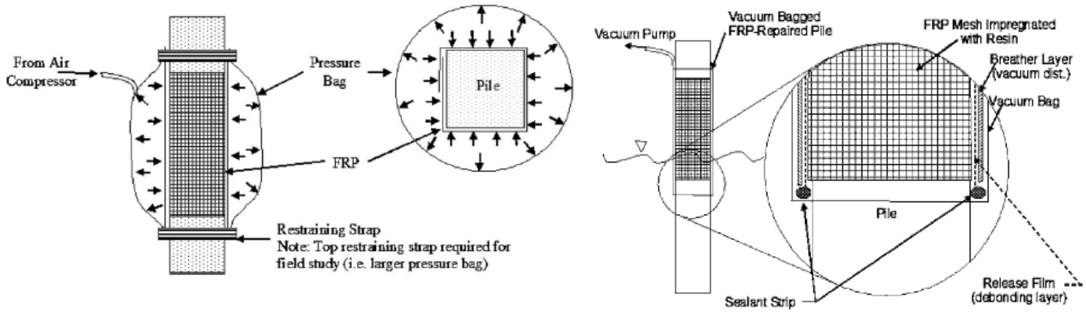


Figure 34: Schematics of the two systems, a) pressure bagging, b) Vacuum bagging (Winters et al. 2008)

Testing was made on eight pre-stressed concrete specimens with a cross-section of 305x205 mm and a height of 1520 mm. These samples were placed in a fresh water tank with the water level of 934 mm from the bottom. To encourage marine growth the piles was left in the tank for three months before wrapping. The wrapping was made from 457 mm above the water level to 457 mm below with two different systems. A pre-preg system developed by Air Logistic Corporation and a wet lay-up system designed by Fyfe. These systems were identical to the ones used in (Sen and Mullins 2007a). A summary of the wrapping systems is provided in Table 4.

Table 4: Wrap systems (Winters et al. 2008)

Test pile	Wrapping system	Confinement system	Applied pressure (kPa)
A1	Air logistic One longitudinal layer,	Control (stretch wrap)	0
A2	Two transverse layers	N/A	N/A
A3		Vacuum bag	68.9
A4		Vacuum bag	68.9
A5		Pressure bag	68.9
F1	Tyfo SHE-51A	Pressure bag	34.5
F2	One longitudinal layer,	Pressure bag	14.5
F3	Two transverse layers	Control (stretch wrap)	0

For increasing the FRP-concrete bond, a base resin coating was applied before wrapping the pre-preg system. Two different resins were tested in this experiment, Aquawrap Base Primer 4 and Bio-dur 563. Both were applied at two surfaces each of the four piles The first pile (A4) to be tested with the vacuum system encountered problems to seal the vacuum bag due to cracks below and above the wrapping area. This was solved by filling the cracks with epoxy and vacuum was obtained 45 minutes after wrapping For pile A2 and A3, the base resin was applied beyond the wrapping area for sealing of the cracks. Pile A3 was wrapped before the applied base resin had cured and the uncured resin had difficulties in sealing the cracks and providing an airtight layer. The base resin on pile A2 was allowed to cure for 24 hours before wrapping. However, before installation, the bond between the base primer and the concrete was inspected and it was found that the bond was inadequate. Thus, no wrapping was made on pile A2. The difficulties to achieve an airtight layer for the vacuum bagging in the experiment shows that this approach can be problematic for field repairs. However, the testing with the pressure bagging was made without any significant difficulties.

The bond was evaluated with the results from pull-out tests with and Elcometer 106 adhesion tester by ASTM D 4541. For the pre-preg system, the result showed that pressure bagging increased the bond strength, from 0.26 MPa to 1.17 MPa, in the wet region and, from 0.94 MPa to 1.57 MPa, in the dry region. The result from the wet layup was mixed with a marginal reduction, from 2.01 MPa to 1.95 MPa, in bond strength in the dry region and an increase, from 1.65 MPa to 1.77 MPa, in bond strength in the wet region.

To distinguish between satisfactory and unsatisfactory bond strengths a cut of limit was used. The chosen limit, 1.38 MPa, is specified in ACI 441 as the minimum pull-out strength for bond critical applications. The percentage of satisfactory pull-out tests is shown in Figure 35

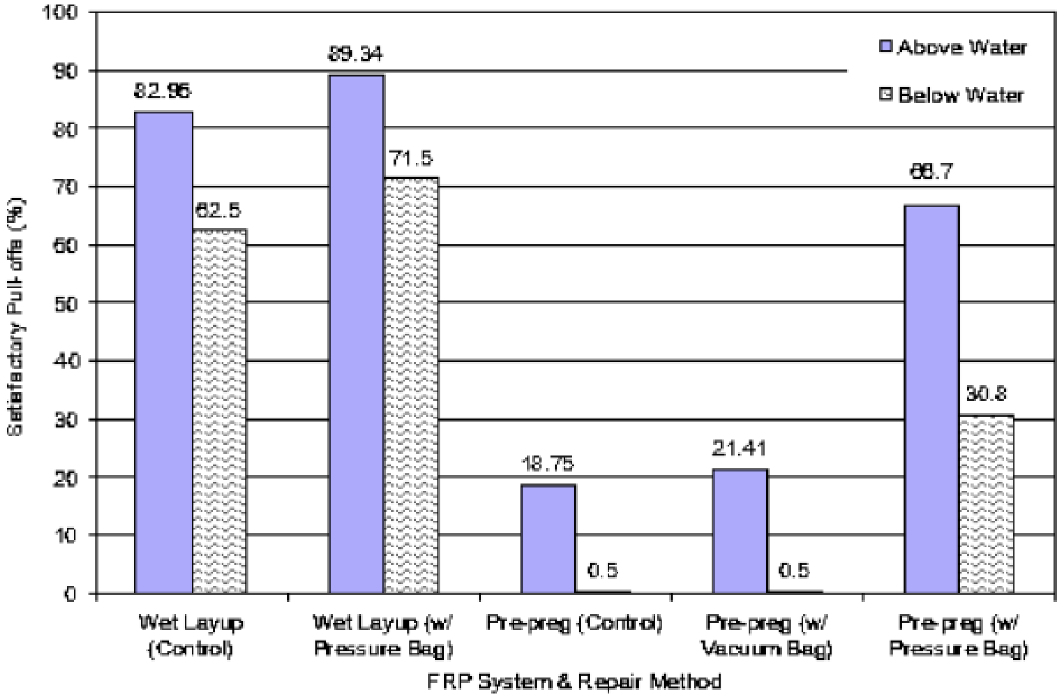


Figure 35: Percentage of satisfactory pull-out results using 1.38 MPa cut off (Winters et al. 2008)

4.6.7 Effective repair for corrosion control using FRP wraps

Suh et al. (2010) investigated the influence on corrosion by two different repairing strategies using FRP wraps. The first strategy, "full repair", includes removing of chloride-contaminated concrete, cleaning of reinforcement and reshaping the cross-section while the second repair, "resin injection repair", consisted of injecting cracks with epoxy and smoothening of the surface. Both strategies were then wrapped with bi-directional CFRP.

The testing was made on seventeen pre-stressed piles with the dimensions and splash zone set-up as in (Suh et al. 2007), see Figure 33. Before repair, the specimens were subjected to a constant current of current 110 mA during 125-days to achieve a 20% metal loss. For confining the corrosion to the splash zone, a soaker hose-sponge was used to keep this area moist.

After 125-days the specimens were repaired, three with the "full repair", eleven with the "epoxy injection repair" and the three remaining was left for control. The wrapping was made in one to three layers of the splash zone or the whole pile, see Table 5 and Figure 36. For some of the specimens, the top side of the piles was sealed, see Table 5, to prevent chloride ingress from above.

Table 5: Summary of results (Suh et al. 2010)

				Metal loss					Number of broken wires	
				Single strand						
Repair type	Wrap height	Wrap layers	Seal	Max [%]	Min [%]	Pile average [%]	Increase due to exposure [%]	Average increase [%]	Total per pile	Average per repair type
Baseline control[a]	N/A	N/A	N/A	22.3	16.9	20.5				
Unrepaired control	N/A	N/A	N/A	86.0	77.8	82.3	61.8	59.6	28	28
				86.5	69.3	77.9	57.4		28	
Full repair	0.9 m	2	Yes	23.2	20.4	22.0	1.5	1.1	17	10
				23.4	18.9	22.0	1.5		6	
			No	21.5	19.4	20.7	0.2		7	
Resin injection repair	0.9 m	1	Yes	26.5	22.1	24.2	3.7	4.3	26	21.5
				27.3	21.4	25.3	4.8		17	
		2	Yes	24.1	19.9	22.3	1.8	0.6	10	13.4
				21.9	19.9	20.8	0.3		13	
			No	21.0	19.4	20.3	-0.2[b]		17	
		3	Yes	24.3	19.7	22.4	1.9	3.2	12	17.4
				34.3	22.4	27.6	7.1		24	
			No	23.2	17.5	21.0	0.5		16	
	1.5 m	2	Yes	24.0	20.7	22.1	1.6	0.9	16	13.7
				22.1	20.1	20.7	0.2		12	
			No	24.9	19.4	21.4	0.9		13	

[a]This specimen were gravimetric tested after the 125 days of 110 mA to control that 20% corrosion was achieved.
[b]Average steel loss in this specimen was less than the baseline control of 20.5%.

For testing of the effectiveness of the repair, the specimens were placed in a tank with a 3% chloride solution see Figure 36. The water temperature was maintained at 60°C and the tide was changed every six hours. During the incoming tide, hot salt water was sprayed on the piles to simulating water splash above the wrap.

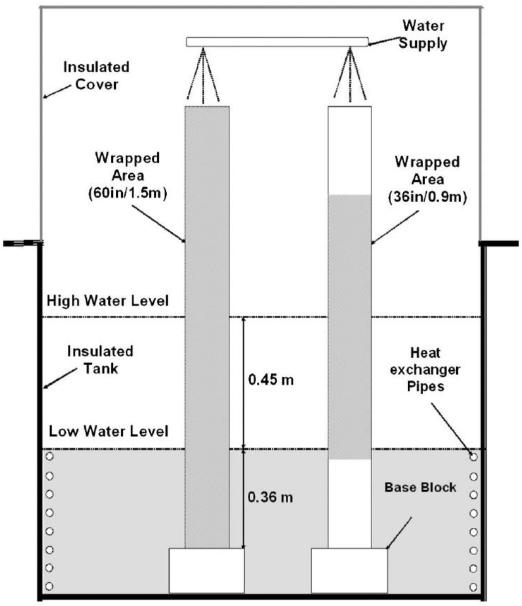


Figure 36: Tidal simulation (Suh et al. 2010)

After 2.3 years and 1,700 tidal cycles, the metal loss was measured by gravimetric testing and the result is shown in Table 5. The average increase in metal loss for the unwrapped specimens were 60% compared to 1.8% for the wrap. From this, they concluded that FRP wraps effectively slows down the corrosion rate. The research also showed that the results of the “full repair” and “epoxy injection repair” were comparable and that two layers of wrap are the most efficient number for slowing down corrosion.

4.7 Concluding remarks

As can be concluded from the state of practice review, repair of submerged reinforced concrete piles with FRP composites is made primarily to achieve durable protection from degradation. The FRP wraps are mainly applied in the splash zone where the deterioration is most substantial.

Since FRP composites are lightweight and flexible, they provide a quick and easy installation and a more beneficial work environment for the workers. This may offset the high initial cost of the material and lead to a more cost-effective repair method compared to the ones used today. Because FRP composites don't corrode and are a low permeable material, it acts as a protective layer impeding chemical degradation and may also lead to a more durable solution.

To achieve an effective result with FRP repair the surface preparation is important to get an adequate bond between the FRP and substrate. It is also important to provide the FRP wrap with a protective coating to prevent degradation of the composite. Because of FRP's high strength to weight ratio repair can be made slender and with the proper choice of protective coating, it can be made indistinguishable from unrepaired piles. Upon review, epoxy is the favourable resin to use in the composite since it is superior to polyester in the considered application together with either glass or carbon fibres. Advantages and disadvantages of the FRP repair method are summarised Figure 37.

From the review of the performed experimental tests in the literature, it can be concluded that CFRP and GFRP wraps are equally effective in reducing the corrosion rate of reinforced concrete. It is also found that two layers of wrap are the most effective number of layers for reducing the corrosion rate. Wraps with a better bond between the FRP and concrete proved to be more efficient in reducing the corrosion rate which further indicates the importance of achieving a good bond to get an effective repair. The review shows that the bond can be enhanced by pressure bagging the wrap while it cures. It can also be concluded that the bond between the FRP and the concrete is deteriorated by moisture as the failure, according to performed pull-out test, is in the epoxy for the submerged zone while in the concrete for the dry and tidal zones.

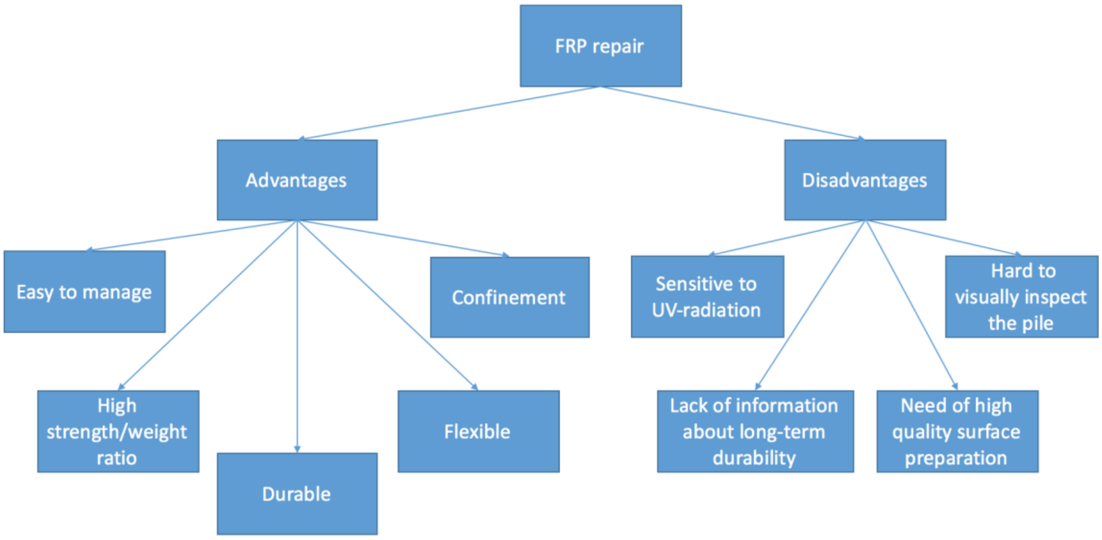


Figure 37: Advantages/disadvantages of FRP repair

5 Durability test of Repairing Concrete with FRP

5.1 Introduction

Accelerated long-term tests have been carried out to check the durability of FRP wrapped reinforced concrete. The experiment is made in a simulated marine environment and includes one accelerated corrosion test series through impressed current and one freeze/thaw test series.

The objectives of the experiment are to investigate the effect of wrapping reinforced concrete with FRP on the corrosion rate, to study what effect confinement of the wrap has on the crack propagation, as well as to investigate the durability of the FRP/concrete bond in a marine environment.

Henceforth, the preparation of specimens is described and the test scheme with set up for the different tests are explained followed by results and conclusions.

5.2 Specimens

For the experiment 15 specimens were prepared, nine for the accelerated corrosion test and six for the freeze/thaw test. The specimens were cylindrical “lollipop” specimens with the reinforcement placed in the centre and protruding out from the top surface. The performed literature review showed that this is a common type of sample for accelerated corrosion testing. The specimens had a diameter of 100 mm and a height of 180 mm with a reinforcement bar of 16 mm in diameter.

5.2.1 Concrete

Two 25 kg premixed bags of Weber fine concrete C30/37 were mixed with 5 kg of water resulting in a water/cement ratio of 0.51. When the concrete and the water had been mixed, a superplasticizer was added until the concrete achieved a satisfying workability. Properties of the concrete can be seen in Table 6.

Table 6: Concrete specifications

Compressive strength	C30/37
Water/cement ratio	0,51
Aggregate	Natural sand 0-4 mm
Pot time	2 hours
Air	5 %
Cement content	C450 kg/m^3
Exposure class	XC4 XF3 (SS EN 206-1)
Frost-resistant	Yes (SS 13 72 44)

5.2.2 Reinforcement bars

Reinforcement bars of 16 mm diameter and 200 mm length were chosen for the tests. A small area around the bar was sandblasted in order to provide a better electrical connection between the reinforcement bars and the wires during the accelerated corrosion test, see Figure 38.

Figure 38: First three centimetres to the left is the sandblasted area.

After the sandblasting, the reinforcement bars were weighed and the individual weight of the bars was recorded. In preparation for the casting, the sandblasted side of the reinforcement bars was attached to wooden boards of 10 mm thickness through predrilled holes, see Figure 39. The free length of the bars was 170 mm.

Figure 39: Reinforcement bars attached to wooden boards prior to casting

5.2.3 Casting

Cylindrical steel forms, see Figure 40, with a diameter of 100 mm and a height of 200 mm were used for the casting.

Figure 40: Cylindrical steel forms used for casting

Before casting, the forms were greased with form oil to ease the demoulding. The forms were then partially filled with concrete and the prepared reinforcement bars were placed in the centre of the cylinder, held in place by the wooden boards resting on the top edges of the form, see Figure 41.

Figure 41: Casting of the specimens

This setup resulted in concrete specimens with the height of 180 mm with a concrete cover for the reinforcement bars of 30 mm at the bottom and 42 mm on the side, see Figure 42.

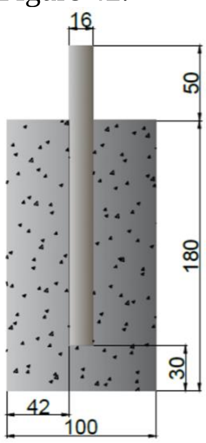


Figure 42: Dimensions of specimen

5.2.4 Curing

The concrete had cured for one and a half day at room temperature before the cylindrical forms were demounted and the specimens were placed to cure in a water bath for 12 days, see Figure 43. The tests started before the concrete was completely cured due to time limitations in the project. This was judged as reasonable considering that tests regarding structural capacity were not an objective of the experiment.

Figure 43: Water curing

5.2.5 FRP configuration

With the results of the literature review presented in section 4.7, it was decided to wrap the concrete specimens with two layers of GFRP since it showed that this was the favourable number of layers.

5.2.5.1 Fibres

The used material in the tests was a 0°/90° bi-directional glass fibre weave with a density of 1273 g/m^2. It was delivered in a 10 m long and 1.270 m wide roll and the first step was to cut the glass fibre in sheets with dimensions of 180x350 mm and 180x380 mm for the first respectively second layer, Figure 44.

Figure 44: Cutting of glass fibre mats

5.2.5.2 Resin

A two-component epoxy resin consisting of 0.5 kg NM 275A laminate and 0.275 kg NM 275B hardening agent was used in the test. This epoxy was chosen because its ability to be applied to a moist surface and that it is very water resistant. By mixing the two-components the epoxy started to cure. The work had then to be completed during the pot time of forty minutes.

5.2.5.3 Wet layup

Wet layup method was used for applying the GFRP to the specimens. The pre-cut glass fibre sheets were saturated with epoxy in a tray before being applied to the concrete specimens. The first layer was wrapped around the specimen with the second layer of wrap applied directly after the first, which can be seen in Figure 45. The second layer was applied with a one-quarter lap shift to ensure that the joints of the different layers were separated. After wrapping the epoxy was allowed to cure for a day.

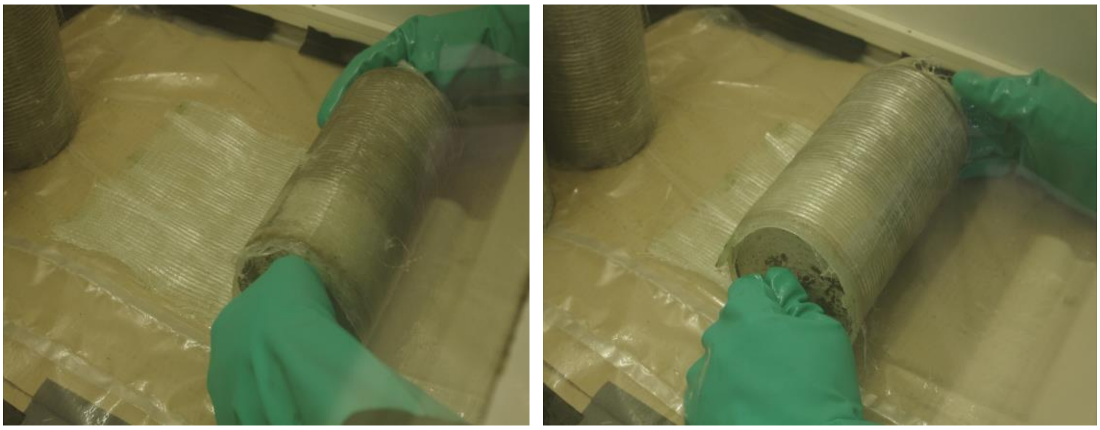


Figure 45: Wrapping of concrete specimens with saturated GFRP sheets

As a security measure, the work involving epoxy and curing was made in a fume hood, see Figure 46, to lead away by-products from the curing of the epoxy which can cause allergic reactions.

Figure 46: Fume hood, work station for epoxy

5.3 Test scheme

The specimens were subjected to accelerated corrosion and freeze/thaw cycles for two months. Different groups of specimens were tested under different configurations regarding the time of wrapping and the conditions they were subjected to. The test scheme is shown in Figure 47.

Group A-C were subjected to accelerated corrosion for the entire test period of two months. The specimens in group A were wrapped from the very beginning, the specimens in group C were wrapped after 1 month and group B were control specimens without any wrap.

Group D and E were subjected to accelerated corrosion until the reinforced concrete specimens had developed cracks due to corrosion, which occurred after a month. Then group D was wrapped and inserted in a climate chamber together with the specimens without wrap in control group E. These two groups were subjected to freeze/thaw cycles during the remaining time of the test period, see Figure 47.

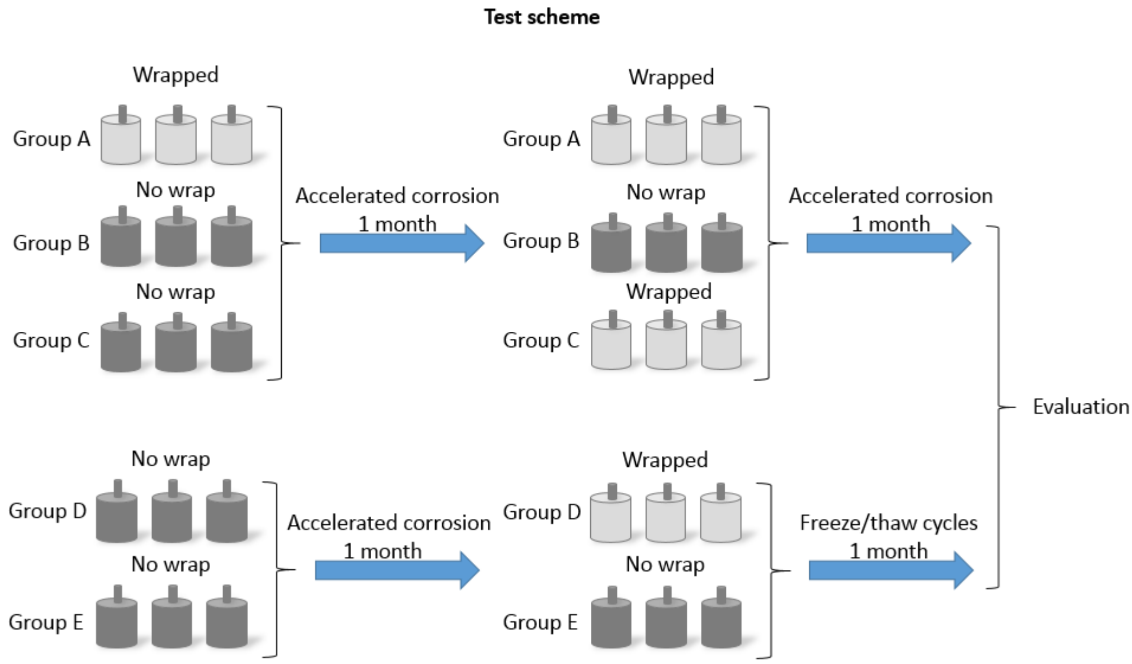


Figure 47: Test scheme of experiment

5.4 Accelerate corrosion test

The natural corrosion processes can take several years before causing damage to the reinforced concrete. It is not viable to make use of natural corrosion for experimental testing. Instead, the rate of corrosion is increased by performing an accelerated corrosion test with an impressed current. The electrochemical reactions can be simulated and enhanced by a constant voltage or a constant current, see Figure 48.

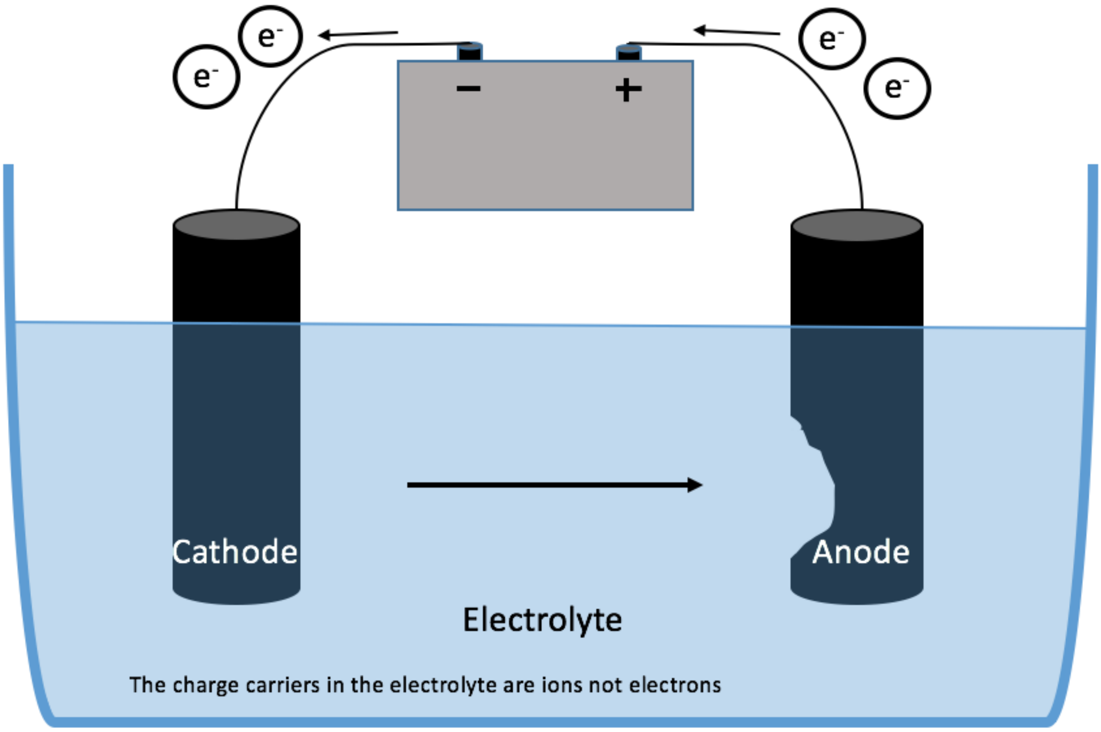


Figure 48: Electrochemical reaction with impressed current

For the accelerated corrosion test, it was decided to subject the specimens to a constant current so that the corrosion rate for the different specimens would be the same to evaluate if the confinement provided by the wraps had any effect on the cracking of the concrete. For maintaining the natural corrosion process, the applied current density should be below 350 $\mu A/cm^2$ (Berrocal et al. 2017). The constant current of 17 mA was applied and was calculated according to Appendix A to provide a corrosion rate of 10% of the diameter of the reinforcement bar. However, after two weeks of accelerated corrosion there was no evidence of crack initiation in the specimens and therefore it was decided to increase the current to 19.5 mA to increase the rate of corrosion.

5.4.1 Set up

All the specimens were placed in one bucket each and sealed to the bottom with a layer of silicone to prevent chloride penetration from the bottom, see Figure 49. To achieve the electrochemical cell a copper mesh was wrapped around each specimen and kept in place with rubber bands. Wires were attached to the reinforcement bar, acting as the anode, and to the copper mesh, acting as the cathode, for connecting the specimens in series. The cell was then completed by filling the bucket with 3.5% salt solution, acting as the electrolyte, so that the water level covered two-thirds of the specimen, see Figure 50.

Figure 49: Sealing of bottom surface

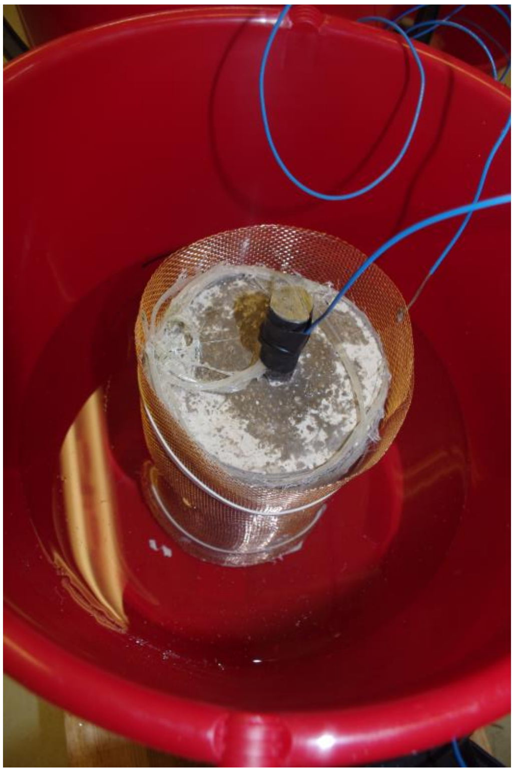


Figure 50: Wiring to and from one specimen

The electric wiring was then made in a series circuit, one for each specimen group A, B and C, to ensure the same amount of current through each specimen. Figure 51 shows a schematic drawing of one circuit.

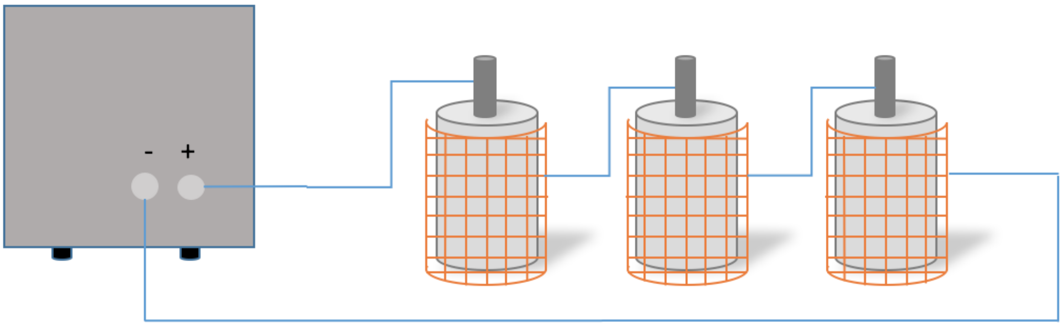


Figure 51: Specimens connected in series

5.5 Freeze/thaw test

The worst deterioration of submerged concrete piles occurs in the splash zone of the pile where it is subjected to wet and dry cycles. Corrosion of the reinforcement is dependent on access to moisture and oxygen and is accelerated by high temperatures and the presence of chlorides. To maximise the effect of these factors and to simulate a Swedish marine environment the specimens were subjected to cycles of high and low temperature combined with high and low relative humidity inside a climate chamber.

5.5.1 Set up

After one month of accelerated corrosion through impressed current, the specimens in group D and E were inserted in the climate chamber. The chamber was programmed to provide cycles that start at a temperature of 20°C and an RH of 95 % for two hours. Then the RH changes to 0% and the temperature decrease linearly to -20°C during one hour. The temperature and the RH remain constant for two hours before both the temperature and the RH increase linearly during one hour back to the starting values of 20°C and 95% RH, see Figure 52. With this configuration one cycle is made during 6 hours, resulting in 4 cycles a day and a total amount of 110 cycles during the test period.

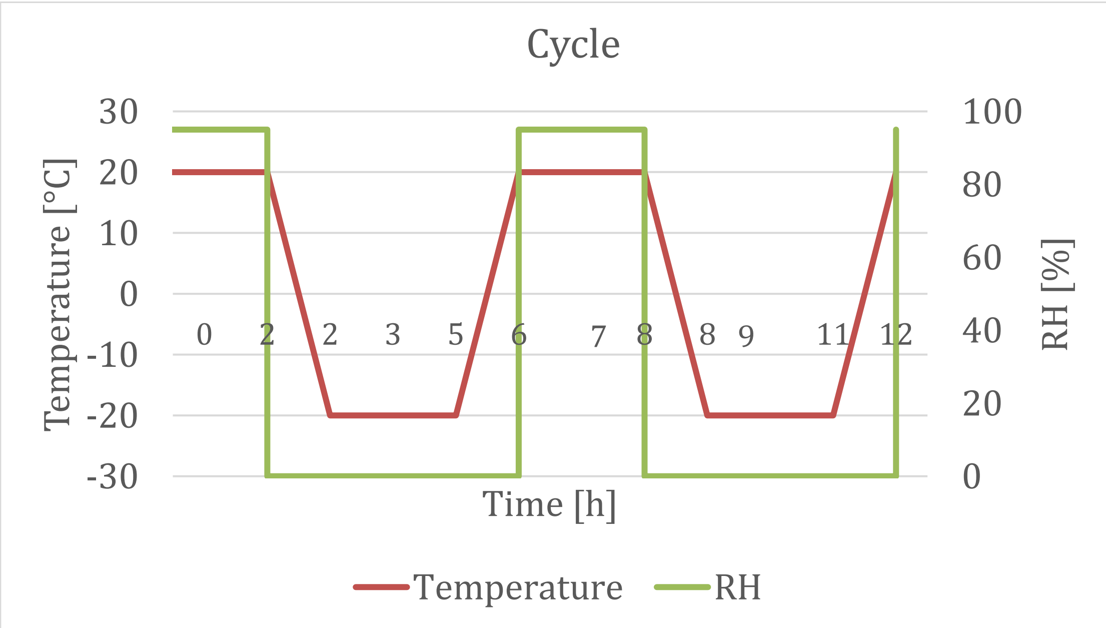


Figure 52: Climate cycle inside the climate chamber

5.6 Data collection

To evaluate if the FRP wraps had a positive effect on the corrosion of the reinforcement, voltage measurements were made to determine the current resistance of the specimens. Higher resistance indicates that the FRP wraps provide protection of the reinforcement bars.

During the experiment the cracks were measured continuously with the card shown in *Figure* 53 for comparison of the crack propagations.

Figure 53: Measure card for cracks

After the experiment had been completed, the FRP wraps were carefully removed so that the crack propagation in the concrete could be recorded and compared between the different test groups.

To further investigate the effect of wrapping concrete with FRP on the corrosion rate the reinforcement bars from all specimens in the experiment were extracted from the concrete, sandblasted to remove all rust and weighed to record the reinforcement bar mass loss.

For investigation of the FRP/concrete bond durability, no measurements were made, but continuous ocular inspection of the wraps was made. The bond durability was also evaluated based on the difficulty to remove the wraps and the state of the interface between the concrete and the FRP.

5.7 Results

5.7.1 Reinforcement bar mass loss

Figure 55 displays the initial weight compared to the final weight of the reinforcement bars for all specimens included in the experimental study. It is from this figure clear that groups B and C suffer the most steel mass loss.

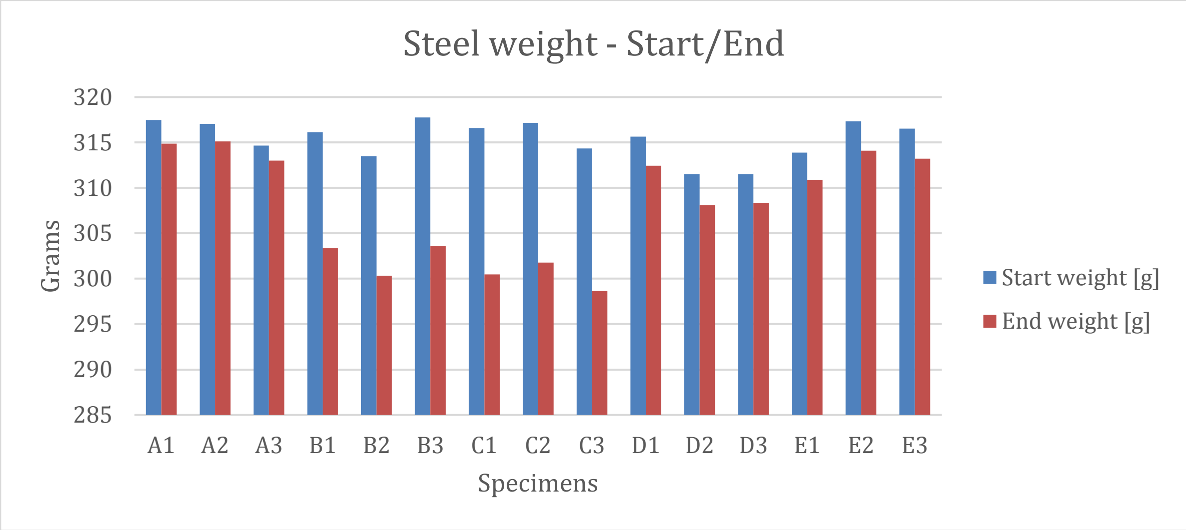


Figure 54: Comparison of the start and end weight of reinforcement bars for the different specimens

5.7.1.1 Accelerated corrosion test

The mass loss of reinforcement bars for the specimens subjected to accelerated corrosion by an impressed current is compiled in Table 7. The average mass loss per group is displayed in Figure 55. Groups B and C has significantly higher mass loss compared to group A. Between groups B and C there is only a marginal difference where group A has an average mass loss of 0.65%, group B of 4.23% and group C of 4.98%.

Table 7: Reinforcement bar mass loss for specimens in the accelerated corrosion test

Reinforcement bar	Start weight [g]	End weight [g]	Mass loss [%]
A1	317.47	314.87	0.82
A2	317.04	315.1	0.61
A3	314.65	313	0.52
B1	316.14	303.35	4.05
B2	313.5	300.32	4.20
B3	317.75	303.62	4.45
C1	316.59	300.47	5.09
C2	317.17	301.78	4.85
C3	314.35	298.65	4.99

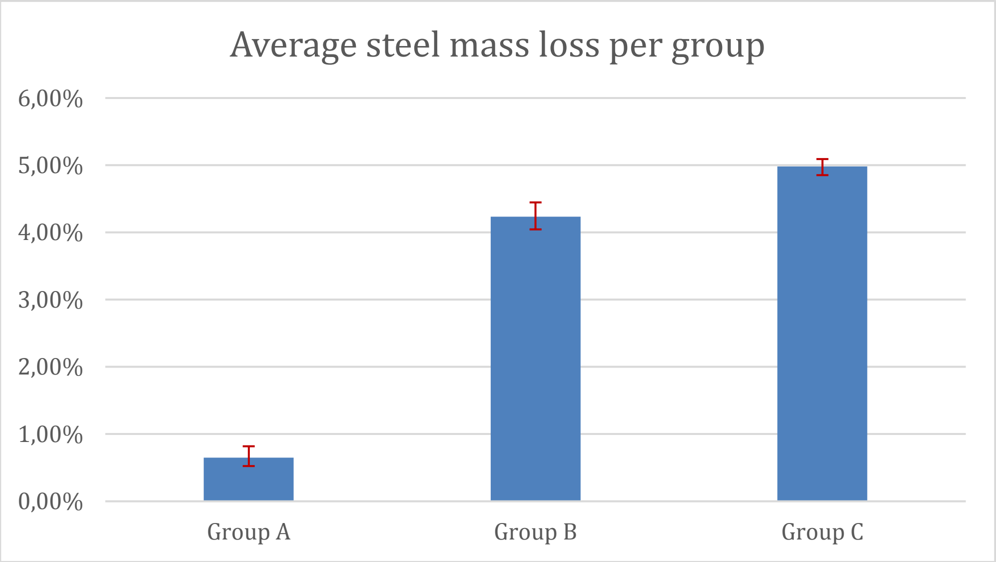


Figure 55: Average reinforcement bar mass loss and spread per group in the accelerated corrosion test

5.7.1.2 Freeze/thaw test

The mass loss of reinforcement bars for the specimens subjected to freeze/thaw cycles is compiled in Table 8. The average mass loss per group is displayed in Figure 56. There is a negligible difference between the two groups where the average mass loss is 1.04% for group D and 1.00% for group E.

Table 8: Reinforcement bar mass loss for specimens in the freeze/thaw test

Reinforcement bar	Start weight [g]	End weight [g]	Mass loss [%]
D1	315.63	312.44	1.01
D2	311.54	308.11	1.10
D3	311.52	308.35	1.02
E1	313.88	310.9	0.95
E2	317.33	314.11	1.01
E3	316.54	313.23	1.05

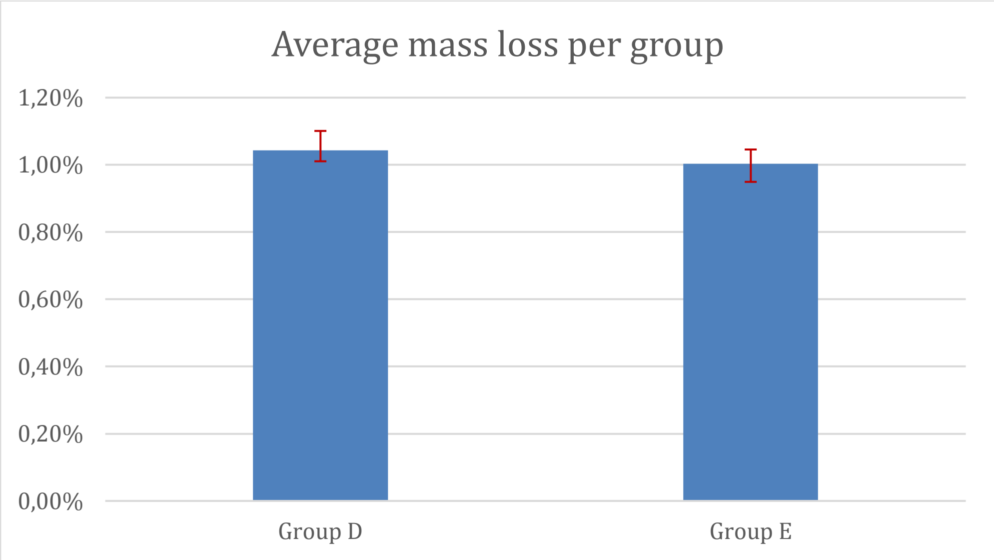


Figure 56: Average reinforcement bar mass loss and spread per group in the freeze/thaw test

5.7.2 Crack propagation

5.7.2.1 Accelerated corrosion test

The crack propagation is shown in Table 9 and Figure 57. The specimens in group A didn't have any cracks with a bandwidth above 0.1 mm while the specimens in group B and C had cracks with a bandwidth of 0.1 mm or above at all surfaces. The specimens in group B had longitudinal cracks reaching the full height of the specimens while the specimens in group C had shorter longitudinal cracks reaching partly along the height of the specimens. The samples in group B also had transverse cracks which couldn't be seen for the specimens in group C.

Table 9: Crack propagation for specimens in the accelerated corrosion test

Specimens	**Crack band width >= 0.1 [mm]**			
	Top surface	**Bottom surface**	**Side surface**	
			Longitudinal Partial/full height	Transverse
A1	No	No	No	No
A2	No	No	No	No
A3	No	No	No	No
B1	Yes	Yes	Full	Yes
B2	Yes	Yes	Full	Yes
B3	Yes	Yes	Full	Yes
C1	Yes	Yes	Partial	No
C2	No	Yes	Partial	No
C3	No	Yes	Partial	No

The bandwidth of the largest cracks at the different surfaces is shown in Table 10. It can be seen that the specimens in group B had the largest crack bandwidths and that the specimens in group A had the smallest crack bandwidths.

Table 10: Crack bandwidth for specimens in the accelerated corrosion test

Specimens	**Crack bandwidth [mm]**		
	Top surface	**Bottom surface**	**Side surface**
A1	< 0.1	< 0.1	< 0.1
A2	< 0.1	< 0.1	< 0.1
A3	< 0.1	< 0.1	< 0.1
B1	0.2	0.9	0.9
B2	0.1	0.8	0.6
B3	0.1	0.8	0.7
C1	0.1	0.3	0.1
C2	< 0.1	0.1	0.1
C3	< 0.1	0.3	0.1

Figure 57: Crack propagation, a) specimen A2, b) Crack along the full height of specimen B3, c) Crack partly along the height of specimen C2, d) No cracks at the bottom of specimen A1, e) Crack at bottom of specimen B2, f) Crack at bottom of specimen C3

5.7.2.2 Freeze/thaw test

The crack propagation is shown in Table 11 and Figure 58. No cracks with a bandwidth of 0.1 mm or above could be seen at the top surface for any of the specimens in group D or E. Cracks at the bottom surface could only be seen at two of the specimens in group E. These two specimens also had longitudinal cracks reaching the full height of the specimens while the other specimens had shorter longitudinal cracks reaching partly along the height of the specimens. Transverse cracks could be seen on all specimens in group E but not on any of the specimens in group D.

Table 11: Crack propagation for specimens in the freeze/thaw test

Specimens	Crack bandwidth >= 0.1 [mm]			
	Top surface	**Bottom surface**	**Side surface**	
			Longitudinal Partial/full height	Transverse
D1	No	No	Partial	No
D2	No	No	Partial	No
D3	No	No	Partial	No
E1	No	Yes	Full	Yes
E2	No	Yes	Full	Yes
E3	No	No	Partial	Yes

The bandwidth of the largest cracks at the different surfaces is shown in Table 12. The crack bandwidths were in general small for all the specimens. It can be seen that the specimens in group E had larger crack bandwidths than the specimens in group D.

Table 12: Crack bandwidth for specimens in the freeze/thaw test

Specimens	Crack band width [mm]		
	Top surface	**Bottom surface**	**Side surface**
D1	< 0.1	< 0.1	0.1
D2	< 0.1	< 0.1	0.1
D3	< 0.1	< 0.1	0.1
E1	< 0.1	0.1	0.1
E2	< 0.1	0.2	0.3
E3	< 0.1	< 0.1	0.2

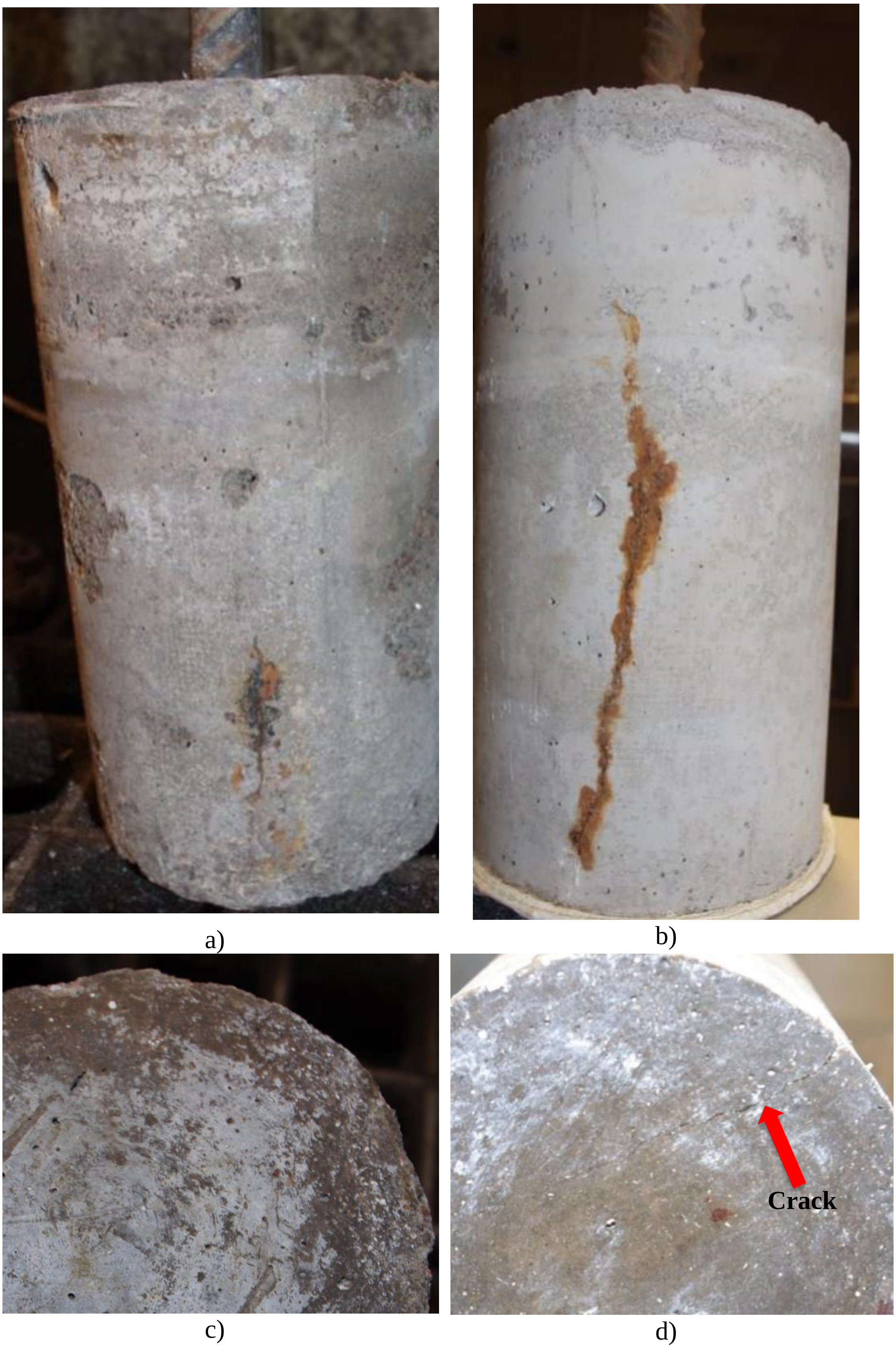


Figure 58: Crack propagation, a) Crack partly along the height of specimen D1, b) Crack along the full height of specimen E2, c) No crack at the bottom of specimen D1, d) Crack at the bottom of specimen E2

5.7.3 Current resistance

The current resistance of the specimens in groups A, B and C is shown in Figure 59 and is displayed over the last month of testing, in other words after group C were wrapped. For the specimens in group A, which were wrapped from the very beginning of the experiment, it took 45 days before any current flowed through the specimens. The resistance of the specimens in group A was then around 1700 ohms. The specimens in group B had a constant resistance around 160 ohms. The specimens in group C showed a resistance of 1500 ohms directly after wrapping but declined with time and converged to around 700 ohms.

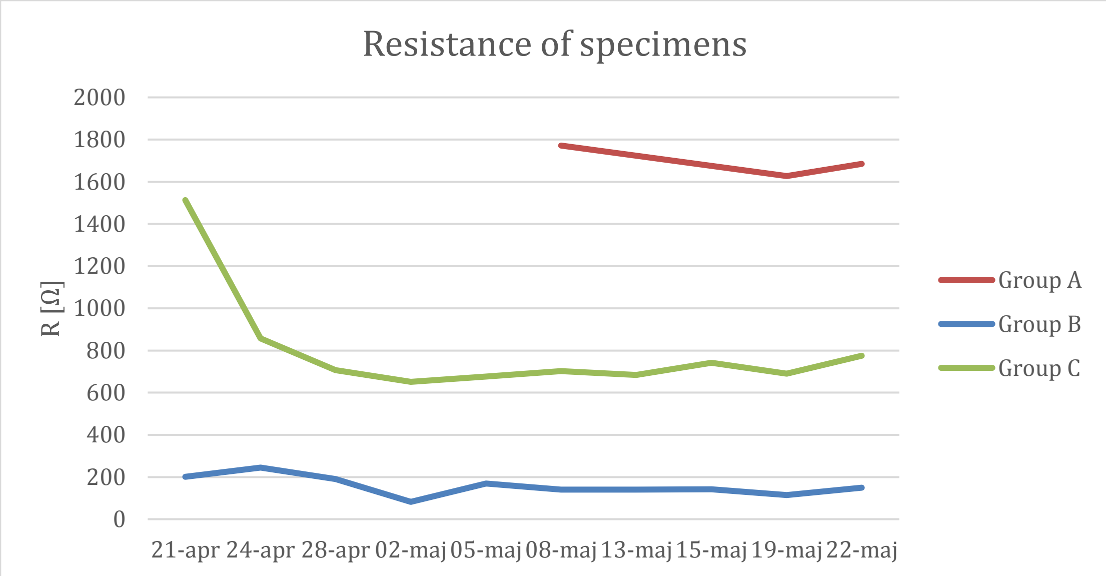


Figure 59: Current resistance o for groups A, B and C

5.7.4 Bond

The inter-laminar bond between the GFRP layers was stronger than the bond between the GFRP and the concrete. The specimens in group A had the strongest concrete/GFRP bond followed by the specimens in group D and the specimens in group C had the weakest concrete/GFRP bond. For the specimens in group A the bond failure occurred in the concrete, see Figure 60, in group D failure could be seen in both concrete and epoxy, see Figure 61, while the failure in group C mostly occurred in the epoxy, see Figure 62. It was also noted that the concrete/GFRP bond was weak at rust deposits as can be seen in Figure 62. Bubbles that could be seen in the GFRP were a result of poor wrapping and no new or enlargement of the existing bubbles could be observed during the experiment.

a) b)

Figure 60: Bond failure in concrete, a) specimen A3, b) inside of wrap A3

a) b)

Figure 61: Bond failure in concrete and epoxy, a) specimen D1, b) inside of wrap D1

a) b)

Figure 62: Bond failure mostly in epoxy, a) specimen C2, b) inside wrap C2

5.8 Discussion & Conclusion

From the results of the experiment, it can be concluded that wrapping reinforced concrete with GFRP has a positive effect on reducing the corrosion rate. The current resistance of the wrapped specimens in group C is well above four times higher than the current resistance of the unwrapped specimens in group B, which proves this. As well as it took 45 days before any current flowed through the specimens in group A and when it did, the resistance of these specimens was about 10 times higher than for the unwrapped specimens in group B. Why current flowed through the specimens in group C directly after wrapping but not for the specimens in group A could be because group A was wrapped when dry and before any degradation of the specimens. So, the specimens were sealed by the wrap with no access to moisture and therefore no corrosion. The specimens in group C were wrapped while moist and after being subjected to one month of accelerated corrosion which caused cracks and contaminated the concrete with chlorides. Via inspection one could also detect better seal of the bottom of the specimens in group A than for group C, which creates easier access for the electrolyte in the specimens of group C. However, why current started to flow through specimens in group A is assumed to be because the electrolyte penetrated near the bottom of the specimens. This is assumed since it was detected when extracting the reinforcement bars, that the reinforcement was only corroded at the end of the bar with internal cracks in the concrete near the bottom as a consequence. Studying the different curves in Figure 59 one can see that they are fluctuating. This is assumed to depend on that during the inspection of the specimens they were removed from the buckets and as a result, rust deposit may detach from the cracks and increase the permeability. In combination with that, the electrical path in the circuit may end up with a higher or lower resistance when the copper mesh was reattached on the specimens with a different position. Another observation made was that the water in which the wrapped specimens were placed was coloured blue while the water for the unwrapped specimens didn't have any discoloration. So, it can be assumed that this is a deposit from the FRP. However, the water was not analysed so it can't be stated if this has any significant environmental impact.

By reviewing the reinforcement bar mass loss no obvious conclusion can be made about the effect of wrapping the reinforced concrete with GFRP. For the specimens in the accelerated corrosion test there is only a marginal difference between the different groups except for group A. This can be explained by that the specimens were subjected to the same amount of current and therefore the same corrosion rate and that the specimens in group A were subjected to current flow for a much shorter time than the other groups. However, there is a slight difference between group B and C where group C has a higher corrosion rate even though it has been concluded that wrapping with GFRP should reduce the corrosion rate. This is a result of that the power sources used in the test had different setting accuracy leading to a slightly higher current for the specimens in group C. For the specimens in the freeze/thaw test, there is no difference in corrosion rate. This could be a result of the limited time for the experiment as corrosion is a slow process and may show a different result if the test was run for a longer period. The corrosion may also have been accelerated if the specimens were subjected to higher temperatures during the freeze/thaw test but it was decided to simulate Swedish climate and therefore the upper limited was set to 20°C.

From studying the cracks in the specimens, it can be concluded that wrapping the concrete with GFRP creates confinement of the concrete and reduces the crack propagation. Considering the accelerated corrosion test, there were only indications of micro cracks in the specimens of group A because of the rust deposits on the concrete surface, but no cracks with a bandwidth over 0.1 mm were found. This is reasonable concerning their corrosion rate. The specimens in group B had more and greater cracks than the specimens in group C. For group B, the cracks larger than 0.1 mm bandwidth ran through the whole specimens from the reinforcement bar to the edge of the concrete at the top, down and through the bottom. There were also transverse cracks in these specimens. While for group C, the cracks larger than 0.1 mm bandwidth ran through the bottom and progressed partly and longitudinally up along the sides of the specimens. For these specimens, there were no cracks at the top surface except for specimen C1, this crack was however not connected with any crack along the side of the specimen, and no transverse cracks were found. The cracks were overall larger for the specimens in group B than for the specimens in group C when comparing the different sides of the specimens. The largest crack bandwidth for group B was 0.9 mm while the largest for group C was 0.3 mm. However, the cracks are not measured with precise instruments so these numbers should be taken more as an indication of the crack propagation. Even though the specimens in group C had somewhat higher corrosion rate than the specimens in group B, the cracks are fewer and smaller for the specimens in group C which indicate that the confinement of the concrete provided by GFRP wraps reduces the crack propagation. Similar comparisons can be made for the specimens in the freeze/thaw test where the wrapped specimens of group D both had less and smaller cracks compared to the unwrapped specimens in group E. Neither of the specimens had cracks with a bandwidth larger than 0.1 mm on the top surface but all the specimens in group E had transverse cracks and specimens E1 and E2 had longitudinal cracks along the full height of the specimens which progressed through the bottom. While the specimens in group D only had shorter longitudinal cracks at the side surfaces. The largest crack bandwidth for group D was 0.1 mm while the largest for group E was 0.3 mm. Since there were both less and smaller cracks for the wrapped specimens of group D than for the unwrapped specimens of group E, although the corrosion rate was the same for both groups, further strengthens the conclusion that GFRP wraps impede the crack propagation of the concrete.

The results of the bond strength were achieved by comparing the difficulties to remove the GFRP wrap with manual labour. This gives no exact result, but an indication of the effects a marine environment has on the bond strength. The bond was strongest for the specimens in group A and the failure mode was discovered in the concrete and thus no extensive deterioration of the adhesive layer was found. The bond was weakest for the specimens in group C and the failure mode for the specimens was discovered in the epoxy. The adhesive layer for the specimens in group C was the most deteriorated as a result of being submerged, leading to access of more moisture behind the wrap, and a higher corrosion rate, resulting in more rust deposits between the wrap and the concrete. The failure mode for the specimens in group D was discovered in both the epoxy and the concrete. The bond for the specimens in group D was stronger than for group C but weaker than for group A. It can therefore be concluded that the concrete/GFRP bond was better when the GFRP was applied to a dry and intact surface as it was harder to remove the wrap for group A, which was wrapped from the very beginning, than for groups C and D, which were wrapped after one month of accelerated corrosion. As well as the concrete/GFRP bond is deteriorated by moisture and rust deposits as group C had weaker bond than group D.

Finally, it can be concluded from the results of the experiment that GFRP wrapping of reinforced concrete reduce the corrosion rate due to that wrapped specimens had higher current resistance than unwrapped and that it is more effective when wrapped on intact concrete. The GFRP wraps provide confinement which impedes the crack propagation of the concrete. The concrete/GFRP bond is stronger when wrapped on a clean and intact concrete surface and that the bond is deteriorated by moisture ingress and rust deposits.

6 Case study

6.1 Introduction

To investigate if the FRP repair method is a competitive alternative to traditional repair methods of submerged concrete piles the costs of an FRP and a traditional repair system is compared. The Port of Gothenburg has provided material from a performed repair of a pile. The cost of this repair is then compared to the cost of repairing the same pile with an FRP system.

6.2 Conditions

The submerged concrete pile selected for the case study was part of a larger repair project of 70 piles located in Skarvikshamnen berth site 510-511. This is part of Gothenburg's energy harbour where ships with a maximum length of 250 m and a maximum depth of 12.5 m can dock ("Port of Gothenburg" 2017). The cross-section of the piles supporting these berth structures are 450x450 mm and the repair should be made along 10 m of the selected pile[7], see Figure 64 and Appendix C, to decrease the ingress of sulphates and chlorides.

6.3 Cost analysis

6.3.1 Performed repair

The completed repair was made by casting a circular concrete pile jacket with a diameter of 670 mm, see Figure 63. The pile jacket was applied from just below the ice-protection down to solid clay, see Figure 64 and Appendix C. This repair had a construction cost of 200-250 kkr[7]. The procedures for installing the pile jacket were as follows:

1. Remove mud until the clay is reached
2. Pressure wash the pile surface to remove marine growth and loose concrete.
3. Installation of the bottom plug
4. Installation of the formwork
5. Cast concrete in the formwork
6. Demounting of the formwork

Table 13 shows required times for different working moments estimated with inquiries from Robert Klein, Engineer Diver at ÅF Infrastructure AB.

Table 13: Time requirement estimation of installing the pile jacket

Estimated time required	
Surface cleaning	8 [h]
Installation of formwork, including bottom plug	16 [h]
Apply concrete	4 [h]
Demounting formwork	8 [h]

[7] Stig Östfjord (stig.ostfjord@portgot.se) (8 May 2017) Examensarbete Personal mail to August Uddmyr (august.uddmyr@afconsult.com)

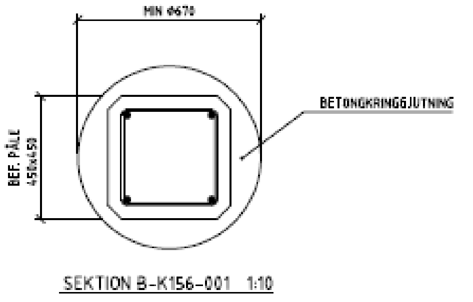


Figure 63: Cross-section of the original pile and the pile jacket[7]

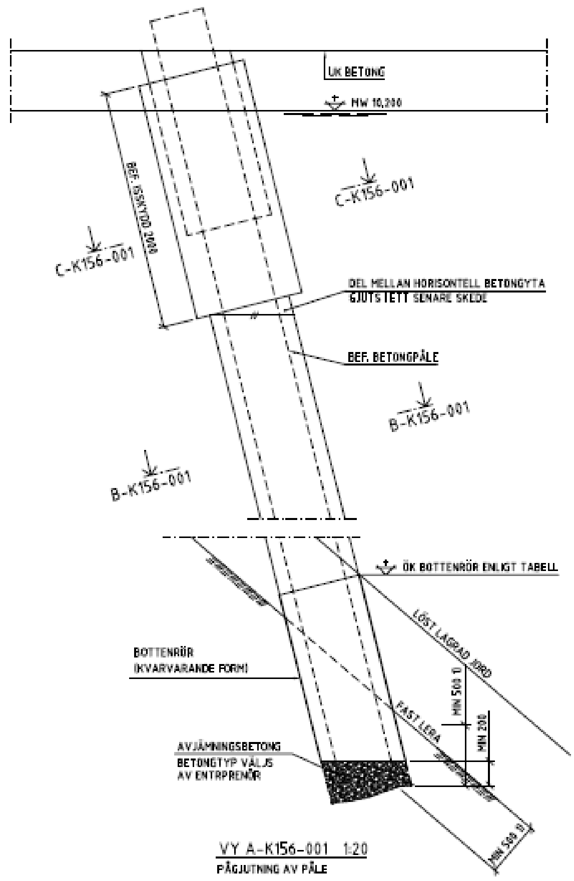


Figure 64: Section of pile jacket repair[7]

6.3.2 FRP repair

Since the repair in this comparative case study only has been made with concrete jacketing the FRP repair is designed with the help of the findings in the literature review above and by consulting experts within the field.

The FRP repair is made with the Aquawrap® pre-preg system by Air Logistics where both glass and carbon fibres are considered. For repair, the idea is to wrap the pile with two layers of bi-directional fibre along the whole height of 10 m, as for the concrete jacket. The work is performed by hand by divers. The procedures for installing the FRP system are the same for both glass and carbon and are as follows:

1. Pressure wash the pile surface to remove marine growth and loose concrete.
2. Apply cement filler to fill voids and to create a smooth surface of the pile.
3. Apply base primer to the pile surface simultaneously as wrapping the pile spirally with two layers of bi-directional fibre with overlap.
4. Apply a plastic stretch film over the wrap to hold it in place while curing.
5. After one day of curing, remove the plastic stretch film and paint the wrap with the base primer.

Table 14 shows detailed information about the amount of time required for the different working moments. The numbers are based on the literature review and inquiries via Robert Klein, Engineer Diver at ÅF Infrastructure AB.

Table 14: Time requirement estimation of installing FRP system

Estimated time required	
Surface cleaning	8h
Surface smoothening	8h
Wrapping, include applying base primer and plastic stretch film	8h
Coating of wrap with base primer	4h

Table 15 shows detailed information about the costs for material and required work for installing the intended FRP system. The information is provided by Franz Worth[6] and by Robert Klein, Engineer Diver at ÅF Infrastructure AB.

Table 15: Cost estimation of Aquawrap® FRP repair system

Estimated labour cost	
One diving crew	2000 [kr/h]
Surface cleaning (8h)	16000 [kr]
Surface smoothening (8h)	16000 [kr]
Wrapping (8h)	16000 [kr]
Coating of wrap (4h)	8000 [kr]
Material cost	
Cement filler, Lampocem	109 [kr/m^2]
Base primer, BP-4	422 [kr/m^2]
Carbon fibre	1651 [kr/m^2]
Glass fibre, G-05	887 [kr/m^2]
Material cost per pile	
Cement filler, Lampocem	1960 [kr]
Base primer, BP-4	15178 [kr]
Carbon fibre	59429 [kr]
Glass fibre, G-05	31926 [kr]
Total cost per pile	
Carbon wrap	132567 [kr]
Glass wrap	105063 [kr]

6.3.3 Sensitivity analysis

In the FRP repair, appreciations and assumption are made to calculate the cost of a repair with 40 years of service life. This provides a degree of uncertainty in the results and therefore a sensitivity analysis is carried out to regard the most sensitive and uncertain parameters.

Five different cases are regarded in the sensitivity analysis for both the CFRP and GFRP system. These cases are:

1. The described FRP repair
2. FRP repair but with twice as long wrapping time
3. FRP repair which is remade after 20 years
4. FRP repair with one layer extra wrap after 30 years
5. FRP repair re-coated every 10:th year

In these five cases costs emerge at different years, because money is worth more as asset today than in the future (Hjelm and Karlsson 2014) the net present value (NPV) is calculated. This is made according to equation 1 with a 2 % discount rate.

$$NPV = \sum_{n=0}^{L} \frac{C_n}{(1+r)^n} \qquad \text{(Eq. 1)}$$

there

NPV	life-cycle cost expressed as net present value
n	the year the cost occurs
C_n	sum of cost year n
r	discount rate
L	service life

6.4 Result

Costs of the different cases of FRP repair is calculated in Appendix D and the result is shown in Table 16. All assumed FRP repair cases, except for case 3 with the CFRP repair, are less expensive than the traditional pile jacket repair. The CFRP repair ranges from 11% more expensive to 34% less expensive than the pile jacket repair. The GFRP repair are 12% to 47% less expensive than the pile jacket repair. In Figure 65 and Figure 66 the NPV is shown with the year the cost emerges. The time estimations of the FRP repair resulted in an installation time of 4 days while the time installing the pile jacket were made during 5 days.

Table 16: Result, cost calculations according to net present value

	Installation Cost, year 0 [kr]	NPV over 40 years [kr]
Pile jacket	200 000	200 000
CFRP – case 1	133 000	133 000
CFRP – case 2	149 000	149 000
CFRP – case 3	133 000	222 000
CFRP – case 4	133 000	179 000
CFRP – case 5	133 000	197 000
GFRP – case 1	105 000	105 000
GFRP – case 2	121 000	121 000
GFRP – case 3	105 000	176 000
GFRP – case 4	105 000	144 000
GFRP – case 5	105 000	170 000

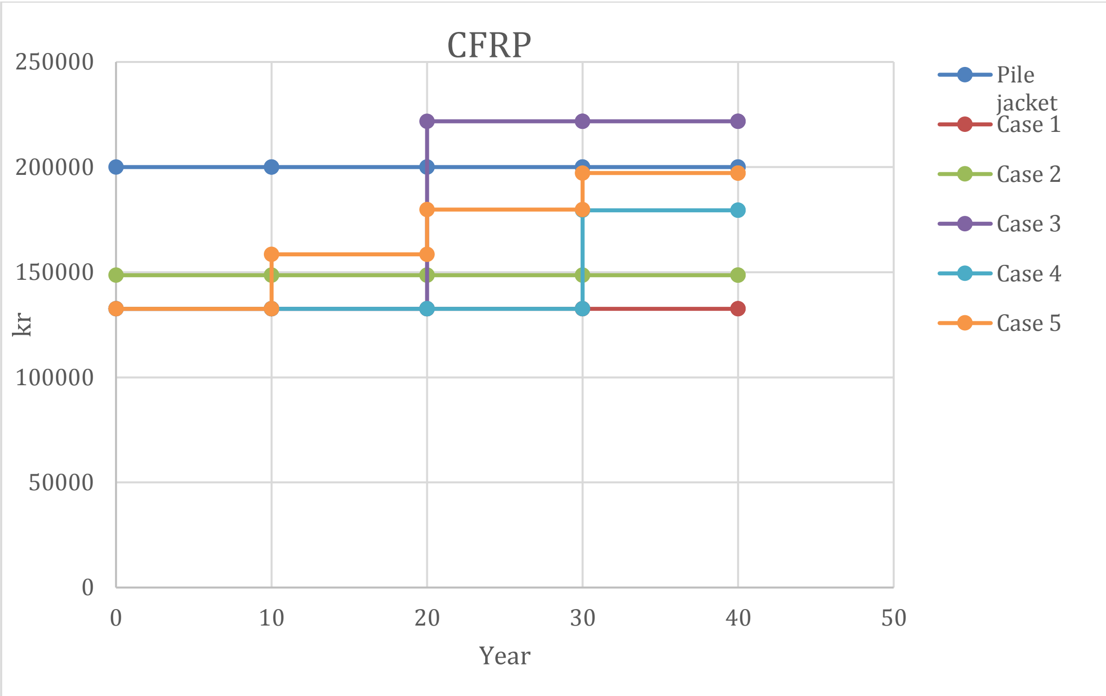


Figure 65: NPV of CFRP repairs at different years

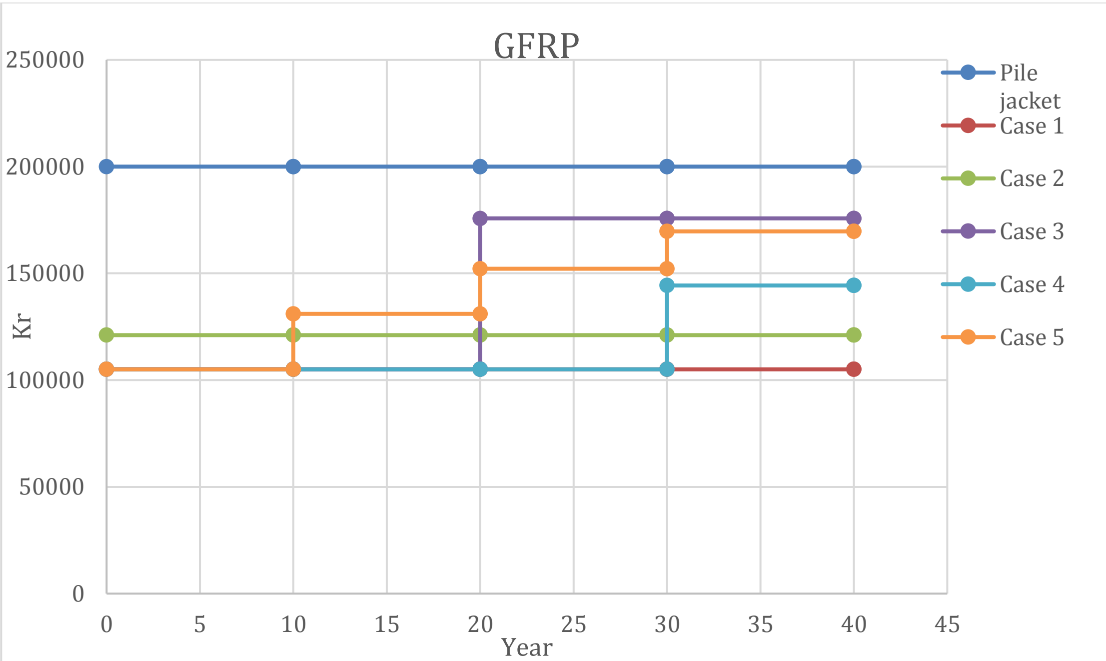


Figure 66: NPV of GFRP repairs at different years

6.5 Concluding remarks

The conclusion can be made that the FRP repair system is a competitive alternative to repairs made with pile jackets in concrete. The results of the case study show that all cases, except for the CFRP – case 3, were less expensive than the performed pile jacket. It also showed, from an economical view at repairs, that GFRP is favourable to CFRP for increasing the durability of the structure.

The costs of the performed pile jacket are from a repair project regarding seventy piles, a repair of only one pile would lead to a higher cost for the pile jacket than used in the studied case. This further shows that the FRP repair method is a competitive alternative. A faster repair method, like FRP repair compared to concrete pile jacket, is not only economically beneficial with regard to fewer man hours but also in the sense of attendant costs in the view of stoppage of the berth.

Considering a larger repair project, like the one in this case study, with a lot of piles it can be hard and time-consuming to reach the piles and install the unwieldy formwork. Here, the FRP system may offer a better solution because of its lightweight and that it is easier to handle.

Analysis of the costs for the systems is made for cases were the FRP repair method is assumed to have the same or shorter service life than the pile jacket. This assumption could be seen as conservative since the reason for investigating the FRP system is the belief that it is more efficient in increasing the durability of submerged piles.

To get the FRP system to correspond with the pile jacket which is installed down to solid clay there are some uncertainties of how to solve the detail at the bottom for the FRP system. One can either dredge and wrap the whole way down and then fill back up with clay or, install a bottom plug similar to the one for the pile jacket. Depending on how to solve this it may amount in extra costs and longer installation time. However, this is not considered in the cost analysis.

7 Summary and recommendations for further studies

The most common area for damages is in the splash/tidal zone of a submerged concrete pile and the degradation are mostly due to chloride ingress and sulphate attack. The most common repair method today is pile jacketing which is a difficult, time consuming and expensive method. This has led to the urge of new repair methods that are sound, quick and durable. To achieve this, the method of wrapping piles with FRP composites has emerged on account of the FRP's advantageous properties compared to traditional construction materials. The FRP wraps are mainly applied in the splash/tidal zone to achieve durable protection from degradation and thus enhance the service life of the pile. Both GFRP and CFRP wraps are equally effective in reducing the corrosion rate of reinforced concrete and two layers of wrap is the most effective number of layers for impeding the corrosion rate.

It is concluded from the results of the experiment that GFRP wrapping of reinforced concrete reduce the corrosion rate due to that wrapped specimens had higher current resistance than unwrapped and that it is more effective when wrapped on healthy concrete. The GFRP wraps provide confinement which impedes the crack propagation of the concrete. The concrete/GFRP bond is stronger when wrapped on a clean and intact concrete surface and that the bond is deteriorated by moisture ingress and rust deposits.

According to the performed case study in this report it can be concluded that the FRP repair system is a competitive alternative to repairs made with pile jackets in concrete when considering costs.

Based on the result and experience gained during this project a summary of why to use, challenges with and recommendations for further studies of the FRP repair method is summarised below.

Reasons to use:

- Durable material.
- Lightweight material.
- Flexible material that can fit any existing shape of a pile.
- Provides for quick and easy installation without the use of heavy machinery.
- Offers confinement of the concrete and reduces crack propagation and prevents spalling.
- Reduces corrosion rate of reinforced concrete.
- Economically viable in comparison to repair with concrete jacket.

Challenges:

- Development/introduction of underwater epoxy that is environmentally approved in Sweden.
- Develop installation procedure to achieve an efficient repair with a good bond.
- How to provide the FRP wrap with ice/abrasion protection.
- How to inspect and detect degradation behind the FRP wrap

Recommendations for further studies:

- Investigate how the FRP withstand abrasion and physical damages that occur in a marine environment.

- Investigate how ice protection should be implemented in a FRP repair system.
- Further studies on long-term durability of externally bonded FRP to investigate how the bond strength is affected by a marine environment.
- Investigate what strengthening effects that can be achieved by wrapping submerged concrete piles with FRP.
- Testing of the FRP repair method in the field to gain knowledge of the ease of installation and to optimize the installation procedure.
- Investigations on externally bonded FRP of new piles before driving. This may enhance the service life of the piles and benefit the life cycle cost compared with repairing them with FRP or traditional methods.

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Appendix A

Calculation of current for accelerated corrosion test

Corrosion rate:

$$CR = K \cdot \frac{i}{\rho} \cdot \frac{M}{n}$$

M/n Considered dimensionless

$M := 55.847$ [g/mol] Atomic weight

$n := 2$ the ionic charge for iron

$$K := 3.27 \cdot 10^{-3} \cdot \frac{mm \cdot g}{\mu A \cdot cm \cdot yr}$$

$\rho := 7.87 \frac{g}{cm^3}$ density of steel

$CR := \frac{mm}{yr}$ corrosion rate

$i := \frac{\mu A}{cm^2}$ current density

Decrease in cross-secion area of 10% during 60 days:

$\phi := 16mm$ Reinforcement diameter

$$r := \frac{\phi}{2} = 8 \cdot mm$$

$$0.9 \cdot \frac{\pi \cdot \phi^2}{4} = (r - x)^2 \cdot \pi$$

$$x := 0.01mm$$

$x := root\left[0.9 \cdot \frac{\pi \cdot \phi^2}{4} - (r - x)^2 \cdot \pi, x\right] = 4.103 \times 10^{-4} \cdot m$ decrease in 60 days

$$CR := x \cdot \frac{365}{60} \cdot \frac{1}{yr} = 2.496 \frac{mm}{yr}$$

Calculate current densiy with obtained corrosion rate:

$$i := \frac{CR \cdot \rho \cdot n}{K \cdot M} = 215.117 \frac{\mu A}{cm^2}$$

Surface area of reinforcement:

$$l := 155mm$$

$$A_s := l \cdot 2\pi \cdot r = 77.911 \cdot cm^2$$

Current:

$I := i \cdot A_s = 16.76 \cdot mA$

Voltage:

$n_{15} := 15$ $\quad n_9 := 9$ $\quad n_3 := 3$ number of speceimens in serie

$R := 700\Omega$ Assume one specimen gives 700Ω resistance

$U_{15} := n_{15} \cdot R \cdot I = 175.981\,V$

$U_9 := n_9 \cdot R \cdot I = 105.589\,V$

$U_3 := n_3 \cdot R \cdot I = 35.196\,V$

Adjustment of current:

$i := 250 \frac{\mu A}{cm^2}$ 10/4, increased current density to speed up the corrosion

$Inc := \frac{250 - 215.117}{215.117} = 0.162$ An increase with 16.2%

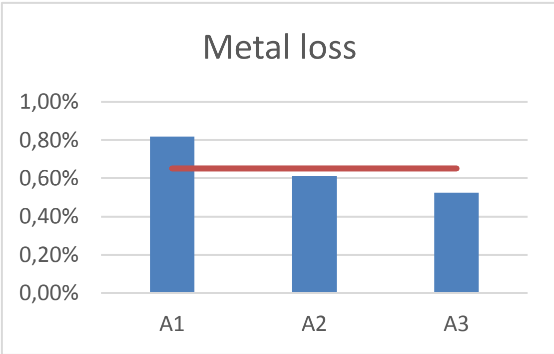
$I := i \cdot A_s = 19.478\,mA$ New current

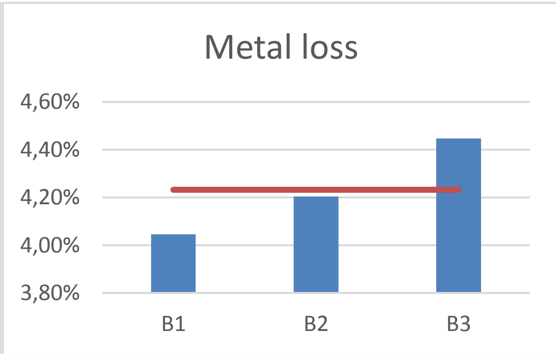
Appendix B

Reinforcement bar mass loss

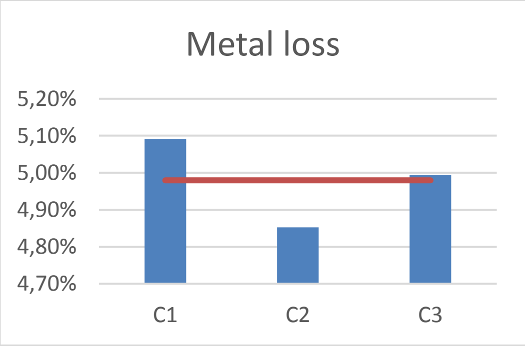
This excel sheet shows the reinforcement bar weight before the starting of the testing, 6/3-2017, and the weight after the testing, 22/5-2017. The metal loss have been calculated and the results are displayed in different diagrams.

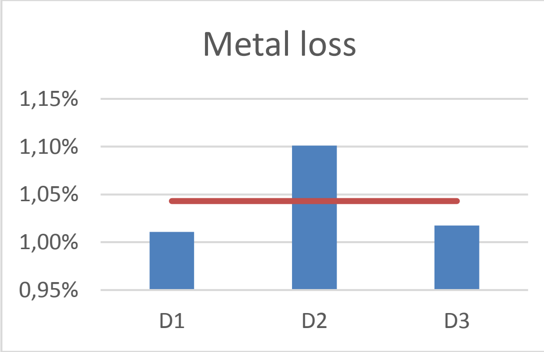
	Reinforcement bar	**A1**	**A2**	**A3**	**B1**	**B2**	**B3**
6/3-2017	**Start weight [g]**	317,47	317,04	314,65	316,14	313,5	317,75
22/5-2017	**End weight [g]**	314,87	315,1	313	303,35	300,32	303,62
	Metal loss [g]	2,6	1,94	1,65	12,79	13,18	14,13
	Metal loss [%]	0,82%	0,61%	0,52%	4,05%	4,20%	4,45%
	Average metal loss [%]	0,65%			4,23%		

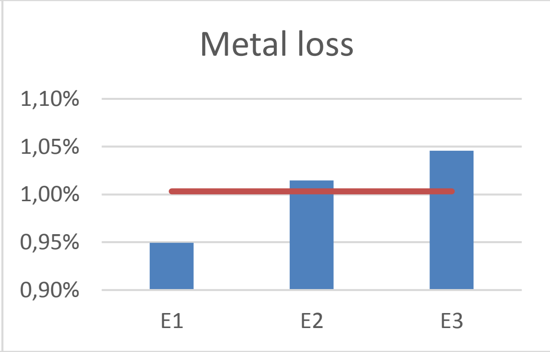


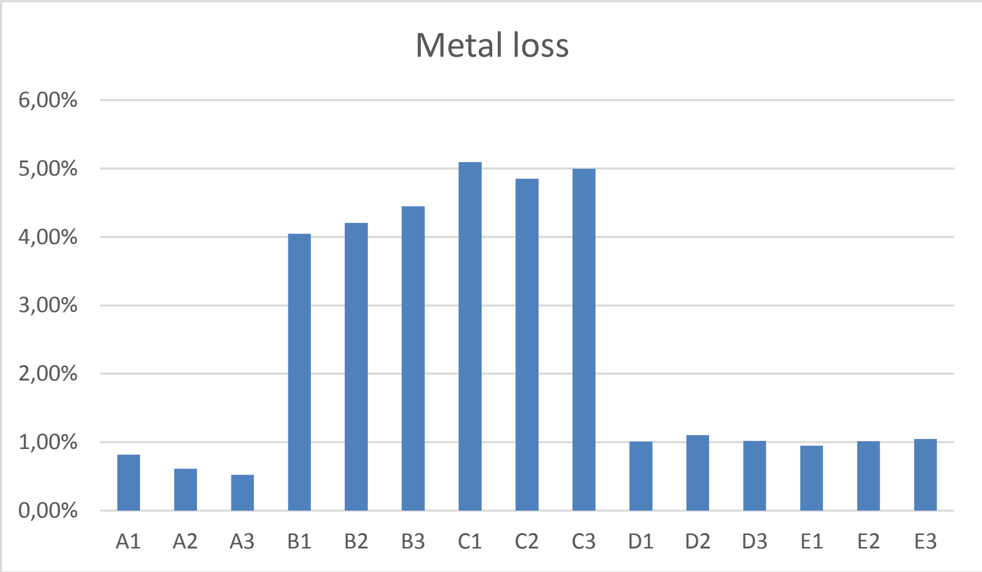


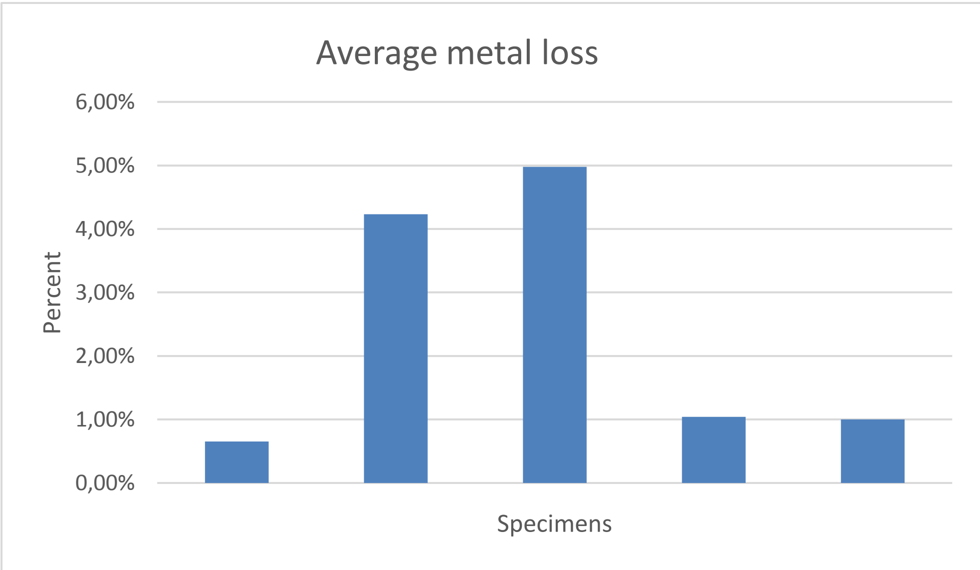
Reinforcement bar	C1	C2	C3	D1	D2	D3	E1	E2	E3
Start weight [g]	316,59	317,17	314,35	315,63	311,54	311,52	313,88	317,33	316,54
End weight [g]	300,47	301,78	298,65	312,44	308,11	308,35	310,9	314,11	313,23
Metal loss [g]	16,12	15,39	15,7	3,19	3,43	3,17	2,98	3,22	3,31
Metal loss [%]	5,09%	4,85%	4,99%	1,01%	1,10%	1,02%	0,95%	1,01%	1,05%
Average metal loss [%]	4,98%			1,04%			1,00%		

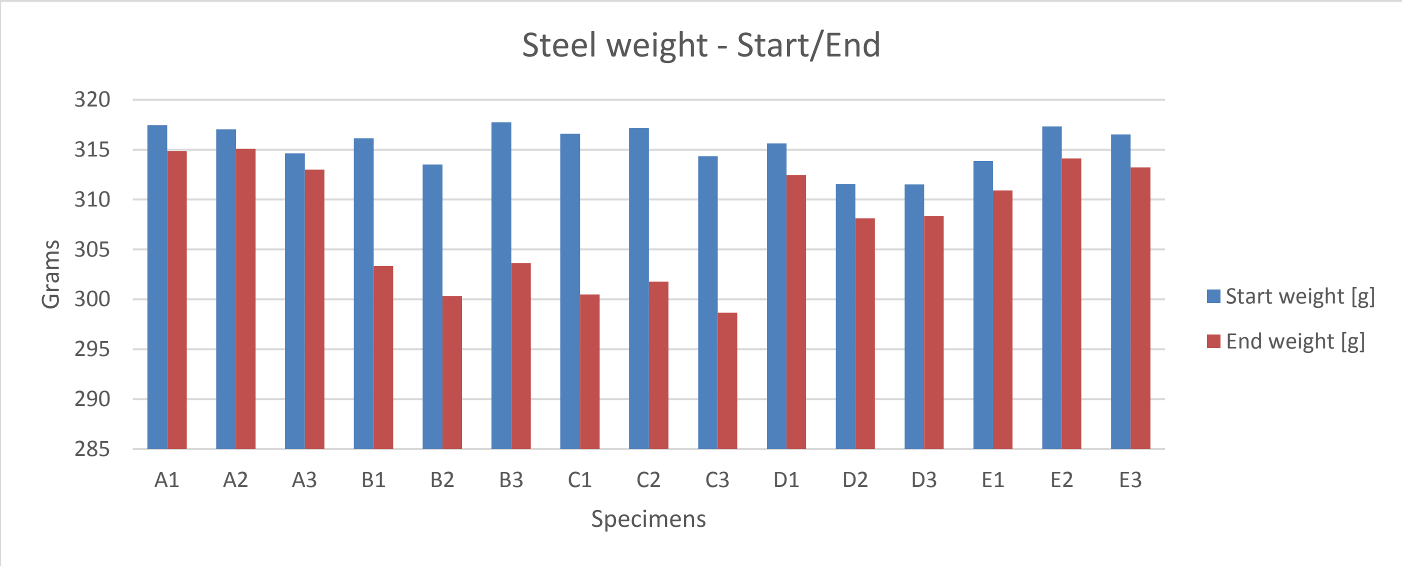




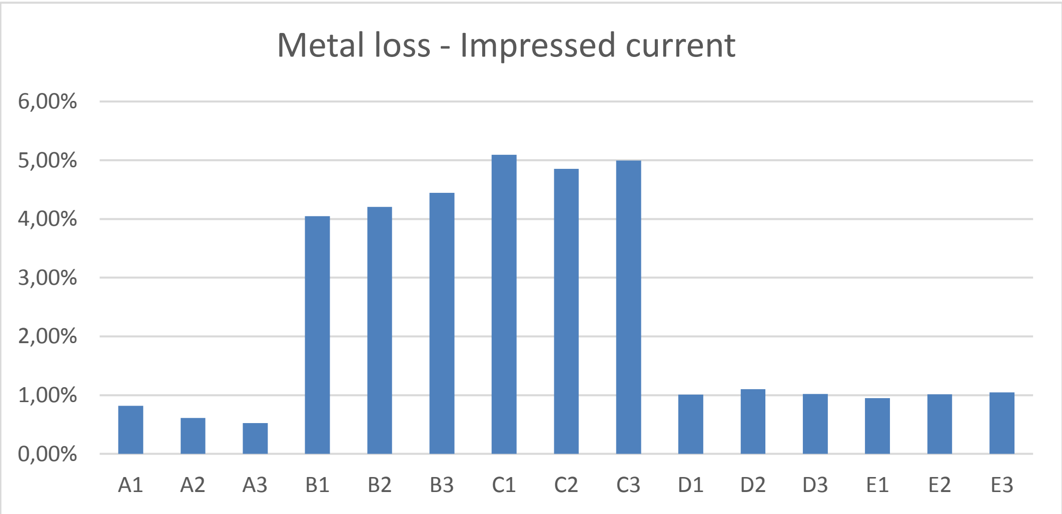



Metal loss
6,00%
5,00%
4,00%
3,00%
2,00%
1,00%
0,00%
A1
A2
A3
B1
B2
B3
C1
C2
C3
D1
D2
D3
E1
E2
E3

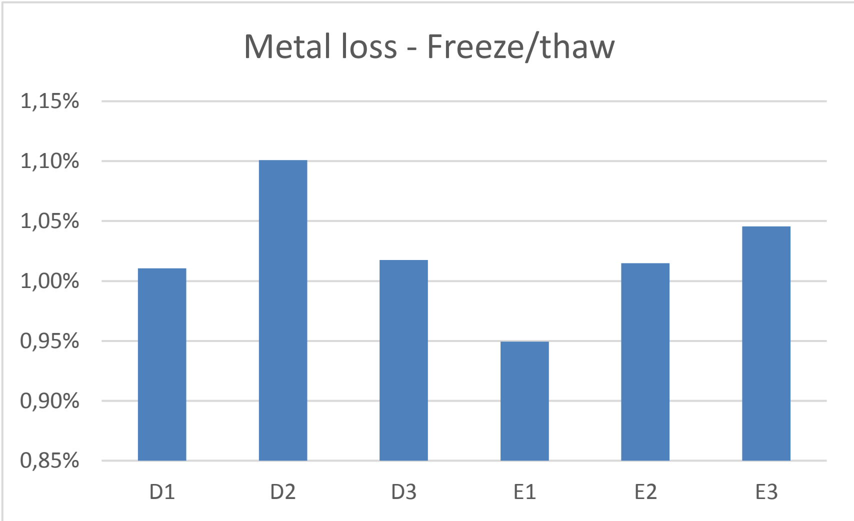

Average metal loss
6,00%
5,00%
4,00%
3,00%
2,00%
1,00%
0,00%
Percent
Specimens



Steel weight - Start/End
Grams
320
315
310
305
300
295
290
285
A1
A2
A3
B1
B2
B3
C1
C2
C3
D1
D2
D3
E1
E2
E3
Specimens
Start weight [g]
End weight [g]



Metal loss - Impressed current
6,00%
5,00%
4,00%
3,00%
2,00%
1,00%
0,00%
A1
A2
A3
B1
B2
B3
C1
C2
C3
D1
D2
D3
E1
E2
E3



Metal loss - Freeze/thaw
1,15%
1,10%
1,05%
1,00%
0,95%
0,90%
0,85%
D1
D2
D3
E1
E2
E3

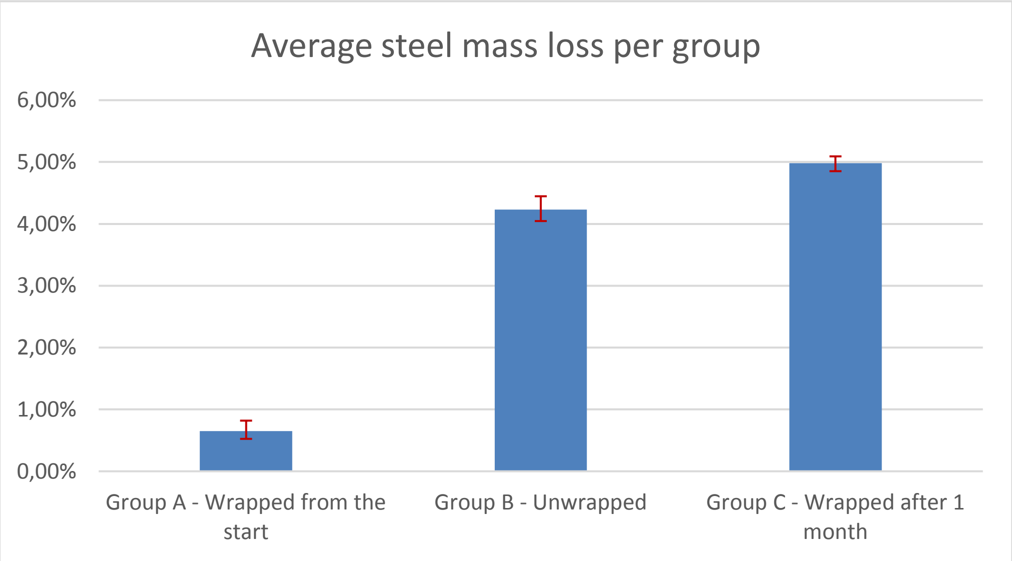
Accelerated corrosion

Reinforcemei	Start weight	End weight [g]	Mass loss [%]
A1	317,47	314,87	0,82%
A2	317,04	315,1	0,61%
A3	314,65	313	0,52%
B1	316,14	303,35	4,05%
B2	313,5	300,32	4,20%
B3	317,75	303,62	4,45%
C1	316,59	300,47	5,09%
C2	317,17	301,78	4,85%
C3	314,35	298,65	4,99%

Average mass loss per group	
Group A - Wrapped from the start	0,65%
Group B - Unwrapped	4,23%
Group C - Wrapped after 1 month	4,98%

Spread		
	plus	minus
Group A	0,17%	0,13%
Group B	0,21%	0,19%
Group C	0,11%	0,13%

Standard error	
Group A	0,087%
Group B	0,117%
Group C	0,070%



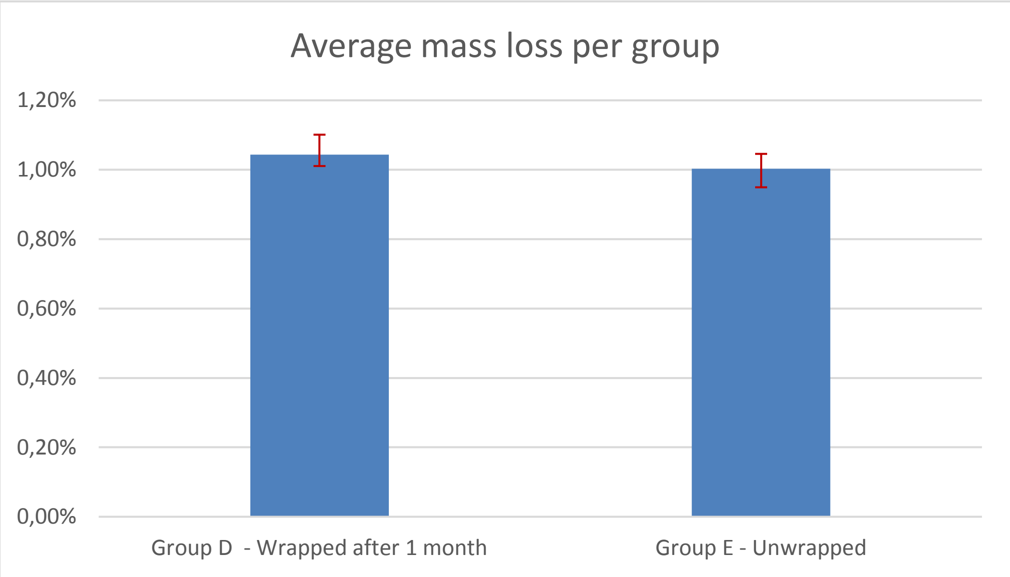
Freeze/thaw test

Reinforcemei	Start weight	End weight [g]	Mass loss[%]
D1	315,63	312,44	1,01%
D2	311,54	308,11	1,10%
D3	311,52	308,35	1,02%
E1	313,88	310,9	0,95%
E2	317,33	314,11	1,01%
E3	316,54	313,23	1,05%

Average mass loss per group	
Group D - Wrapped after 1 month	1,04%
Group E - Unwrapped	1,00%

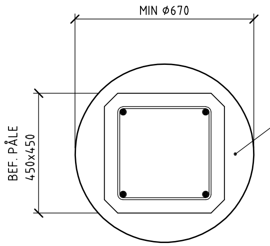
Spread		
	plus	minus
Group D	0,06%	0,03%
Group E	0,04%	0,05%

Standard error	
Group D	0,0003
Group E	0,0003

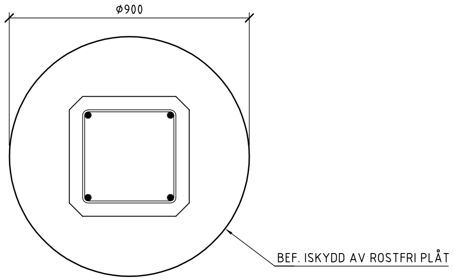


Appendix C

Case study – Drawings of performed pile jacket repair



SEKTION B-K156-001 1:10



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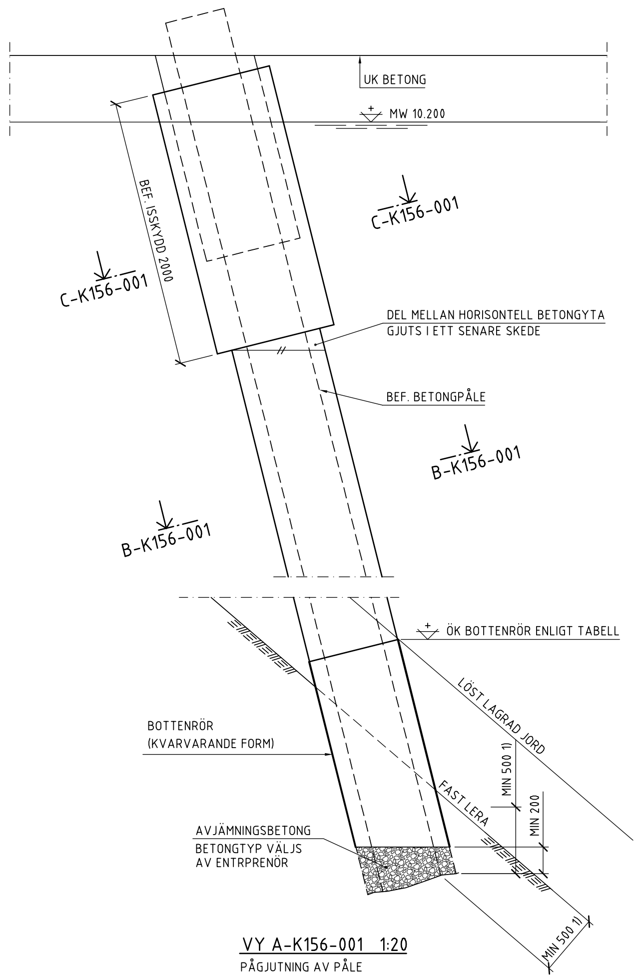

VY A-K156-001 1:20
PÅGJUTNING AV PÅLE

FÖRESKRIFTER

ALLMÄNNA ANVISNINGAR SE RITNING NR KP51-K011-001.

FORMSÄTTNING SKA UTFÖRAS ENLIGT EBB.183. 2)

BETONG ENLIGT EBE.121. 2)

KONTROLL AV VIDHÄFTNING UTFÖRS ENLIGT EBE.121 2)
AVFORMNING ENLIGT EBB.1

FÖRKLARINGAR:
1) AVSER MINSTA MÅTT FRÅN OSKYDDAD PÅLE.
2) AMA-KODER ENLIGT DENNA RITNING HÄNVISAR TILL BESKRIVNING TB-1001 UPPRÄTTAD 2013-05-17
3) EJ UTFÖRD PGA DÅLIG ÅTKOMLIGHET.

TABELL ÖK BOTTENRÖR

PÅLE:	PLUSHÖJD:
157	-
158	-
159	-
159A	-
160	-
161	-
162	-
163	-
164	-
173	-
174	-
174A	3)
175	3)
175A	-
176	-
177	-
178	-
179	-
180	-
189	-
189A	-
190	-
191	-
192	-
193	-
194	-
195	-
196	-

ARBETSORDNING

1. KONTROLLERA ATT PÅLAR ÄR FÖRSEDDA MED BEFINTLIGA ROSTFRIA ISSKYDD.
2. AVLÄGSNA LÖSA JORDMASSOR RUNT PÅLARNA TILLS MAN KOMMER TILL FAST LERA.
3. PÅLENS BETONGYTOR FRÅN UNDERKANT ROSTFRITT ISSKYDD NER TILL ÖVERKANT FAST LERA VATTENBLÄSTRAS/VATTENBILAS (MAX 500 BAR) SÅ ATT ALL LÖS OCH PORÖS BETONG AVLÄGSNAS.
4. DISTANSER (KORROSIONSBESTÄNDIG) PLACERAS PÅ PÅLE SÅ ATT FORMEN CENTRERAS KRING PÅLEN.
MONTAGE AV BOTTENRÖR OCH INVÄNDIG SCHAKT/URSPOLNING AV LERA RUNT PÅLEN.
GJUTNING AV AVJÄMNINGSBETONG.
5. DISTANSER (KORROSIONSBESTÄNDIG) PLACERAS PÅ PÅLE SÅ ATT FORMEN CENTRERAS KRING PÅLEN.
FORM UPP TILL UNDERKANT ISSKYDD MONTERAS.
AVJÄMNINGSBETONGENS ÖVERYTA RENGÖRS INNAN FORMEN MONTERAS SÅ ATT INTE JORD OCH SLAM BLANDAS MED BETONGEN VID KRINGGJUTNINGEN AV PÅLEN.
6. KRINGGJUTNING UPP TILL ÖVERKANT RÖRFORM (HORISONTELL YTA) UTFÖRS PÅ ETT SÅDANT SÄTT ATT INGA LUFTICKOR KVARSTÅR. VID GJUTNING KONTROLLERAS ATT LÄCKAGE EJ UPPSTÅR.
7. PÅLEN AVFORMAS NÄR BETONGEN NÅTT ERFORDERLIG HÅLLFASTHET. BOTTENRÖRET LÄMNAS KVAR.
8. KVARVARANDE DEL MELLAN UNDERKANT ISSKYDD OCH HORISONTELL ÖVERYTA PÅ GJUTNING KRINGGJUTS I ETT SENARE SKEDE.

0 1 2 3 4 5 10 m
1:100

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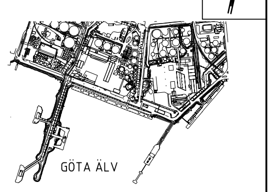
GÖTEBORGS HAMN

INFORMATIONSHANDLING

SITUATIONSPLAN | NORRPIL



PROJEKTKOD	OBJEKT	SEKTION/TYPYTA
U0005		
AUTOCADVERSION	APPLIKATIONSPROGRAM	VERSION
BaTMan NUMMER	LAGERSTRUKTUR	KOORDINATSYSTEM

UPPDRAG
SKARVIKSHAMNEN, REPARATION 2016, KAJPLATS 510-511

ELU VÄSTRA HAMNGATAN 14
411 17 GÖTEBORG
TEL: 031-339 32 00
FAX: 031-13 08 08

UPPDRAGSNUMMER	RITAD AV	HANDLÄGGARE/KONSTRUKTÖR
70419	NH JONSSON	P. GRANSTRÖM
DATUM	ANSVARIG/GODKÄND AV	TEKNOMR
2016-04-15	PER GRANSTRÖM	K

INNEHÅLL
SKARVIKSHAMNEN KP 510-511
TILLÄGGSBYRGGA

PÅGJUTNING AV BETONGPÅLE A1

SKALA	RITNINGSNUMMER	BET
A1 =1:10, 1:20 A3 =1:20, 1:40	KP51-K156-001	

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Appendix D

Case study - Cost calculation of FRP repair

Case study - Cost of FRP repair

Dimension of the repair area of the pile

Pile	
Depth [m]	0,45
Width [m]	0,45
Height [m]	10
Surface area [m2]	18

Calculation of the amount of filler needed for smoothening of the surface

Filler		
Thickness [m]	0,005	*Assumed thickness of filler*
Volume [m3]	0,09	
Lampocem		*The used filler, recommended by Robert Klein*
kg/m3	1800	
kr/kg	12,1	
kr/m3	21780	

Time estimations

Time estimations of different procedurs that need to be made for underwater repair of piles. The estimations are made with help of Robert Klein, engineer diver at ÅF infrastructure and the report “Underwater FRP Pile Wrap of the Friendship Trails Bridge.” Tampa, FL: Departement of Civil and Environmental Engineering, University of South Florida by Sen, Rajan, and Gray Mullins. 2004.

Procedur	**time [h]**	
Cleaning surface	8	*Pressure washing the surface to remove marine growth and loose concrete*
Smoothen surface	8	*Filling voids and smoothening of the surface with filler*
Wrapping	8	*Applying 1 layers BP-4 and 2 layers carbon/G-05 fabric*
Coating	4	*Applying 1 layer BP-4*

Costs

The cost of the carbonfibre, glassfibre(G-05) and Base primer 4 (BP-4) were provided by mail from Franz Worth at airlog.

Franz Worth (fworth@airlog.com) (11 May 2017) Report of splash zone repair – Master thesis. Personal mail to August Uddmyr (uddmyr@student.chalmers.se)

The setup of the repair was made out from the litterature review and resuktet in two layers of wrap and two layers of BP-4, one as primer and one as protective coating.

The cost of the divers was assumed with the help of Robert Klein, engineer diver at ÅF infrastructure

Costs		
Divers [kr/h]	2000	*Price from Robert Klein (1 diving crew incluing 3 divers and equipment)*
Filler [kr/m3]	21780	
Airlog		*Costs provided by Franz Worth at airlog for a carbon and a glasfiber (G-05) aquawrap*
Carbon [$/pile]	6236	*2 layers of wrap*
G-05 [$/pile]	3350	*2 layer of wrap*
BP-4 [$/sqft]	4,11	*Base primer 4, primer and protective coating*
Bp-4 [$/m^2]	44,24	

Currency	2017-05-15	*Currency collected at forex.se at the 2017-05-15*
kr/USD	9,53	

Cost per pile

Cost/pile	**[kr/pile]**	
Material		
Filler	1960	
G-05	31926	*2 layers of wrap*
Carbon	59429	*2 layers of wrap*
BP-4	15177,75	*2 layers, first layer to increase the bond between Concrete/FRP and one as protective coating*
Labour		
Cleaning surface	16000	
Smoothen surface	16000	
Wrapping 8 h	16000	
Coating 4 h	8000	

Total installation cost per pile of the Carbon/G-05 aquawrap system.

Includes material: Filler, Carbon/G-05 fabric, BP-4.
Includes labour: Cleaning surface, smoothen surface, wrapping, coating.

Tot installation cost/pile	**[kr/pile]**
Carbon	**132567**
G-05	**105063**

The maintenance cost are calculated for different cases in the sensitivity analysis

Maintenance	**[kr/pile]**	
Re-wrap 1 layer carbon	84892	*Includes: 1 layer Carbon wrap, 2 layers BP-4, cleaning surface, wrapping*
Re-wrap 1 layer glas	71140	*Includes: 1 layer G-05 wrap, 2 layers BP-4, cleaning surface, wrapping*
Re-coarting 1 layer	31589	*Includes: 1 layer BP-4 primer, cleaning surface, re-coating*

Comparsion between FRP system and pile jacket

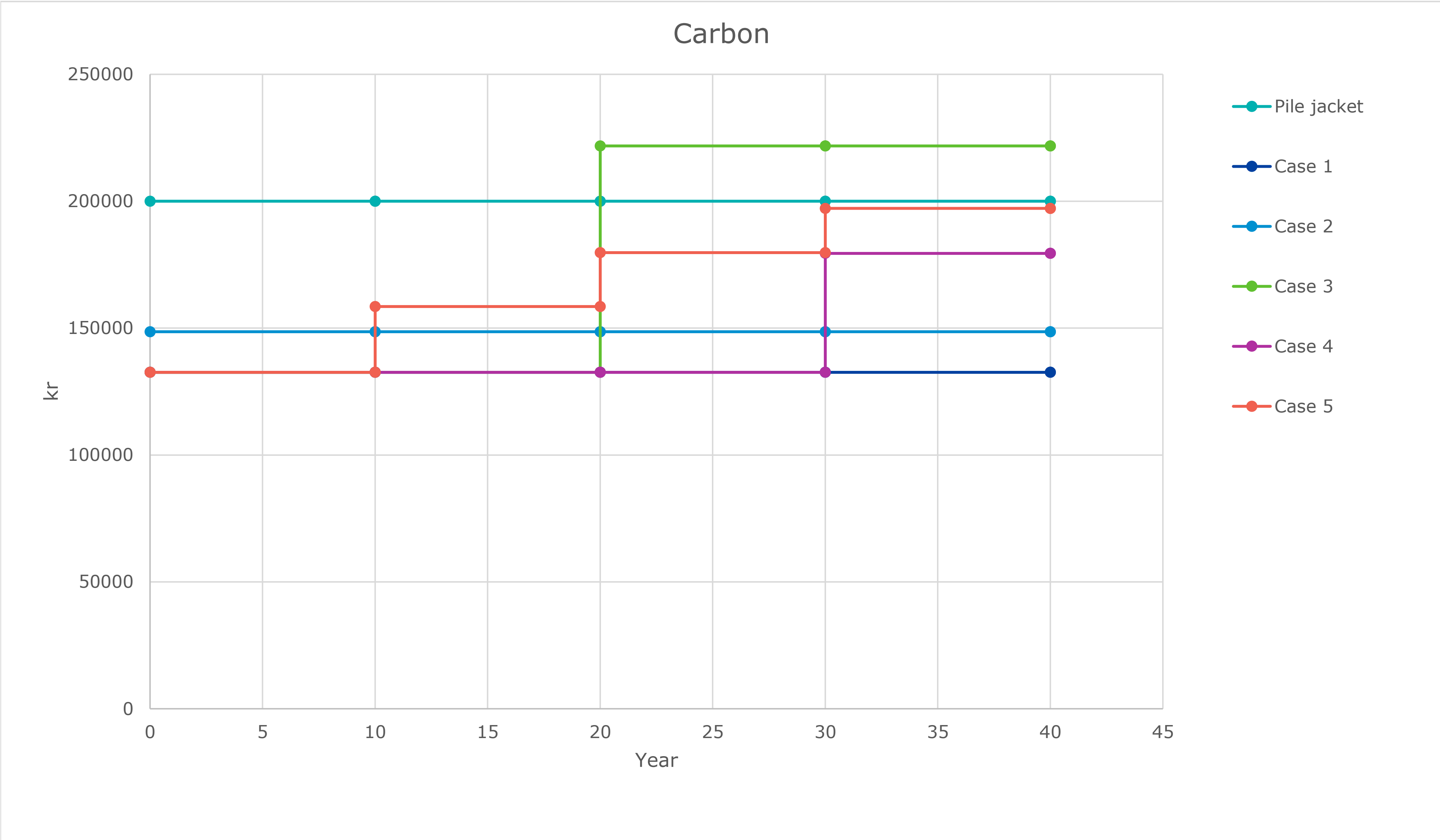
The FRP repair is compared with a pile jacket repair with the design life of 40 years. To compare the costs today with future costs, net present value are used.

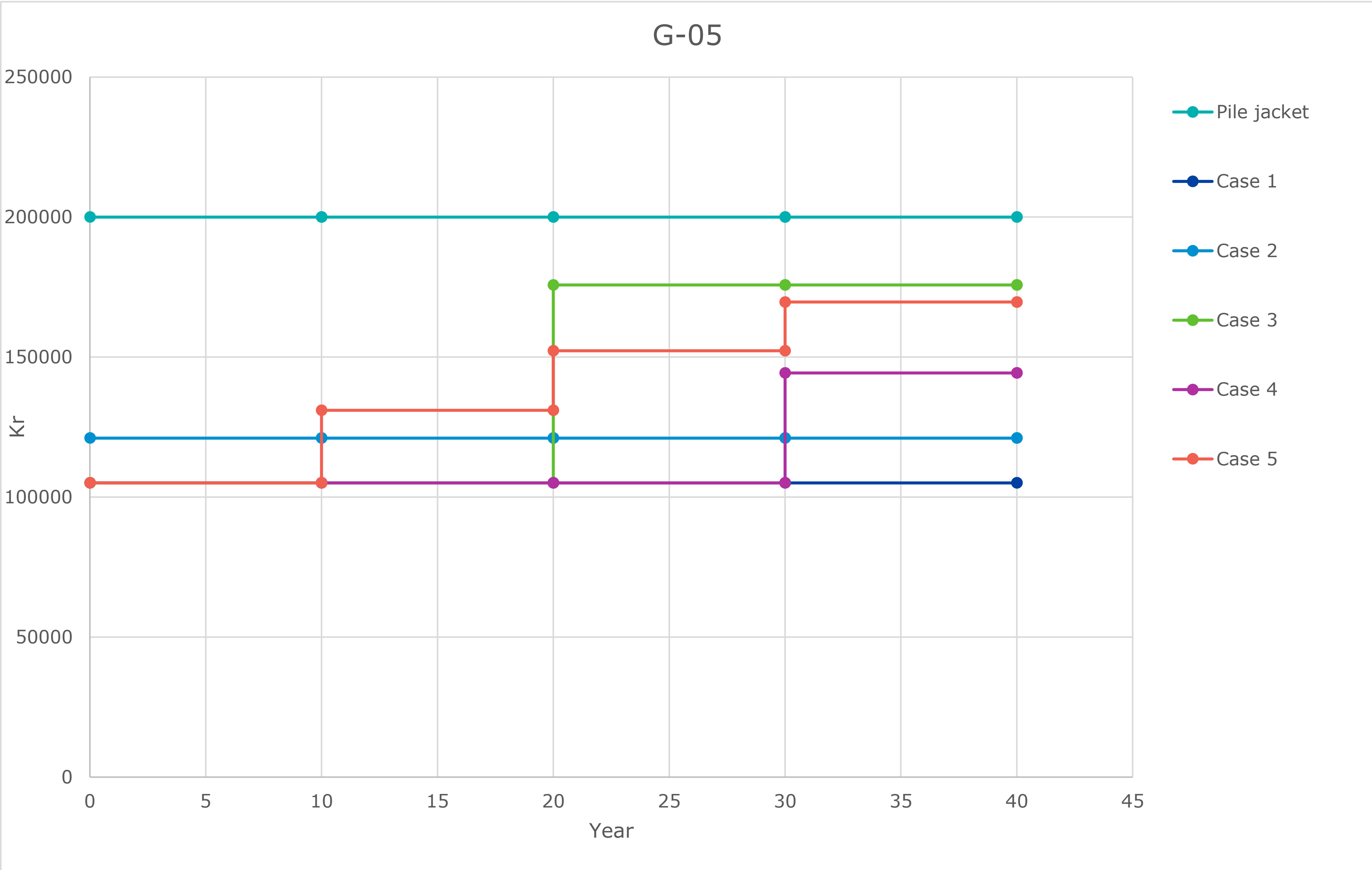
$$NPV = \sum_{n=0}^{L} \frac{C_n}{(1+r)^n}$$

NPV=Net present value
L=Service life, 40 years
n=the year the cost develop
Cn=sum of the cost at year n
r=discount rate, assumed 2%

Five different cases with the FRP repair have also been carried out and the cases are as follows:
Case 1: Original repair with service life of 40 years.
Case 2: FRP repair with dubble wrapping time, service life 40 years.
Case 3: Two FRP repairs with a service life of 20 years each.
Case 4: FRP repair with a service life of 30 years, re-wrap 1 layer carbon/G-05 to increase service life with 10 years.
Case 5: FRP repair with re-coating every 10:th year to fufill 40 years of service life

	NPV [kr/pile]									
Year:	**0**	**10**	**10**	**20**	**20**	**30**	**30**	**40**	**40**	
Pile jacket	200000	200000	200000	200000	200000	200000	200000	200000	200000	
Carbon	132567	132567	132567	132567	132567	132567	132567	132567	132567	*Case 1*
	148567	148567	148567	148567	148567	148567	148567	148567	148567	*Case 2*
	132567	132567	132567	132567	221781	221781	221781	221781	221781	*Case 3*
	132567	132567	132567	132567	132567	132567	179434	179434	179434	*Case 4*
	132567	132567	158480,9	158480,9	179739	179739	197179	197179	197179	*Case 5*
G-05	105063	105063	105063	105063	105063	105063	105063	105063	105063	*Case 1*
	121063	121063	121063	121063	121063	121063	121063	121063	121063	*Case 2*
	105063	105063	105063	105063	175768	175768	175768	175768	175768	*Case 3*
	105063	105063	105063	105063	105063	105063	144338	144338	144338	*Case 4*
	105063	105063	130977	130977	152236	152236	169675	169675	169675	*Case 5*


Carbon
250000
200000
150000
100000
50000
0
kr
0
5
10
15
20
25
30
35
40
45
Year
Pile jacket
Case 1
Case 2
Case 3
Case 4
Case 5


G-05
250000
200000
150000
100000
50000
0
Kr
0
5
10
15
20
25
30
35
40
45
Year
Pile jacket
Case 1
Case 2
Case 3
Case 4
Case 5

Calculate how much cheaper the most and least expensive FRP systems are compared to the pile jacket

	Case 1 (Cheapest)	Case 3 (most expensive)
Carbon	34%	11%
G-05	47%	12%

The carbon fibre ranges between 11% more expensive to 34% less expensive than the pile jacket.
The glass fibre (G-05) ranges betenn 12% to 47% less expensive.