İSTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

EVALUATION OF REPAIR MATERIALS FOR HIGH PERFORMANCE CONCRETE

M.Sc. Thesis by

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Programme: Structural Engineering

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YÜKSEK PERFORMANSLI BETONLARIN ONARIMINDA KULLANILAN MALZEMELERİN DEĞERLENDİRİLMESİ

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SUMMARY

Today, concrete has an intrinsic durability as a construction material and is normally expected to give trouble free service through out its intended design life, but its durability may change under some environmental conditions. For years of its service life, a concrete structure is exposed to several conditions. The result can be partially or generally deterioration of the concrete structure. Thus, most of the structures need renovation to meet its efficient requirements by suitable repair techniques. Consequently, with growing and developing concrete industry, repair of concrete has always been required and has become a main part of design and construction projects. However, the repair works, has traditionally known as an art, not science which causes endless repair failures.

This document includes a literature review of causes of concrete deterioration and how to repair the deteriorated structures. Planning and executing of a repair and methods of controlling the repair quality are presented below. In-situ and laboratory testing's performed and results are analyzed. The objective of the experimental program was to evaluate, under in-situ and laboratory conditions, a general performance criteria for selecting repair materials based on dimensional compatibility with substrate concrete.

In this research the compatibility between two repair materials and substrate concrete is investigated in two stages. First, specific properties of repair materials such as flow, shrinkage, compressive strength and permeability of the specimens are determined in the laboratory. Than trial castings are made on the field. Cores taken from the trail structures are investigated to predict the compatibility of the repair. The dimensional compatibility is also investigated on composite beam specimens prepared in the laboratory.

The interesting part of this research is the in-situ tests. The field studies are performed on RCC elements of the Marmaray Project TBM Tunnel. During the production phase of the elements, some defects such as holes, honeycombings, cracks and breaking of edges have occurred. Repair methods most commonly used are based on filling out of holes with mortar and injection of cracks. To increase the quality of those repairs some trial repairs were executed and cores are drilled out in the middle of the repairs. Permeability tests, adhesion tests and microanalyses are performed to determine the compatibility of the repair material with the substrate concrete. Rapid chloride permeability is determined and chloride diffusion coefficients are calculated the composite cores. Pull-off test method is used to determine the tensile strength on the repaired section. Using impact-echo testing

equipment, analyses are performed on the repaired sections to determine a correlation between material properties and compatibility and results are compared with adhesion test results.

Finally, fluorescent epoxy technique is used to determine the microstructure of the bonded area. Therefore, plane sections and thin sections are prepared for microstructural analysis.

YÜKSEK PERFORMANSLI BETONLARIN ONARIMINDA KULLANILAN MALZEMELERİN DEĞERLENDİRİLMESİ

ÖZET

Kendine özgü dayanıklılığı ile öne çıkan bir yapı malzemesi olan beton, normal şartlar altında tasarlandığı kullanım süresi boyunca işlevini yitirmeden kullanılabilir. Ancak bu süre zarfında birçok çevresel etkiye de maruz kalabilir. Bu etkiler ise kalıcı hasara neden olabilir. İşte bu yüzden birçok betonarme yapı uygun tamirat yöntemi kullanılarak restore edilir. Günümüzde, özellikle beton sektöründeki gelişme nedeniyle tamirat işleri inşaat projelerinin önemli bir parçası haline gelmiştir. Yalnız, piyasada beton tamiratı hala mühendislik işi olarak değil de ustalık işi olarak görüldüğünden bir çok tamirat hatası yapılıyor. Bunun sonucu olarak da defalarca tamiratın tamiratı yapılmak zorunda kalınıyor.

Bu çalışmada öncelikle beton hasarına neden olan etmenler ve bu hasarların tamir yöntemleri anlatıldı. Bununla birlikte, tamir yönteminin nasıl belirleneceğinden ve yapılan tamiratın kalitesinin nasıl kontrol edileceğinden bahsedildi. Laboratuvar ortamında ve şantiyede deneyler yapılarak sonuçlar karşılaştırmalı olarak sunuldu. Yapılan deneysel çalışmaların amacı boyutsal uyumu sağlayabilecek tamir malzemesinin seçilmesi için, laboratuvar ve sahada karşılaştırılmalı olarak performans kriteri belirlemekti.

Yapılan araştırmalarda genel olarak tamir malzemesi ve beton yüzeyi arasında kalan yapışma bölgesinin kalitesi incelendi. Çalışmalar iki aşamalı olarak gerçekleştirildi. Önce tamir malzemesi olarak kullanılacak harçların yayılma, birim ağırlık, priz süresi, basınç mukavemeti ve geçirimlilik gibi temel özelliklerinin belirlenmesi için laboratuvar deneyleri yapıldı. Daha sonra şantiyede betonarme bloklar üzerinde deneme tamiratları gerçekleştirildi. Tamiratlı bölgelerden alınan karotlar üzerinde de tamirat kalitesini belirlemek üzere bir takım deneyler yapıldı. Ayrıca son olarak laboratuvarda üretilen beton kirişler üzerinde yapılan tamiratların beton ile olan uyumu incelendi.

Bu araştırmanın en önemli bölümünü oluşturan şantiye çalışmaları Marmaray Projesi'ne ait TBM tünellerinde kullanılmak üzere imal edilen prekast betonarme segmanlar üzerinde yapıldı. Bu segmanlarda üretim sırasında küçük boşluklar, peteğimsi ayrışmalar, çatlaklar ve segman kenarlarında dış etkilerden oluşan kırıklar gibi yüzeysel hasarlar meydana geldiği görüldü. Kırıklar ve boşluklar tamir harcı kullanılarak, çatlaklar da enjeksiyon yöntemleriyle tamir edildi. Yapılan tamiratın kalitesinin kontrol edilmesi için tamiratlı bölgeden karotlar alınarak deneyler yapıldı. Tamiratlar üzerinde yerinde çekme deneyi, kompozit karotlardan hazırlanan numunelerde de hızlı klor deneyleri yapıldı. Bununla birlikte tamirat kalitesinin

hasarsız yöntemlerle tesbit edilebilmesi için de bazı çalışmalar yapıldı. Impact-echo adı verilen deney aleti kullanılarak yapılan analizler ile yerinde çekme deneyi arasında bir ilişki kurulmaya çalışıldı.

Ayrıca son olarak kompozit karotlardan hazırlanan ince kesit ve düzlem kesit numuneleri üzerinde mikroyapı incelemeleri yapılarak arayüzey kalitesi tesbit edildi.

1. INTRODUCTION

1.1 Background

Durability has an important role in designing of concrete structures. Mainly there are two disciplines to classify a durable structure. Engineers who are designing the building should guarantee the desired lifetime of the project, which is called durability by design. Secondly, the materials of which the building is made should meet the expected quality requirements in order to get a durable structure with adequate costs [41].

Today, concrete has an intrinsic durability as a construction material and is normally expected to give trouble free service through out its intended design life, but its durability under some different environmental conditions changes with the concrete design, mixed constituents, and the presence and positioning of reinforcement; and the detailing, placing, finishing, curing, and protection [13].

Deterioration can occur from a number of causes such as violation of the construction specifications or unexpected environment conditions than those calculated during the planning and design stages. For years of its service life, a concrete structure may be exposed to conditions of corrosion, freeze and thaw cycles, moisture cycles, temperature cycles, abrasion, and chemical attacks such as acid attack or sulphate attack. Physical damage can also arise from fire and explosion. The result can be partially or generally deterioration of the concrete which is the result of the possible reduction of the service life. Normally, most of the structures need renovation during the service life to meet its efficient requirements by suitable repair techniques [13].

This means, concrete structures require care in the form of usual maintenance. Water stagnation, paint pealing, plaster break-off, fungus growth, cracking of external rendering and cover concrete are common situations which may occur with time.

Penetration of moisture into concrete promotes corrosion process for reinforcement and further damages the concrete cover. But buildings remain for several years without getting due attention [3].

The recent growth of the construction industry in the past years has resulted increasing need for many improvements in materials, design practice, installation procedures, contracting processes, QA/QC procedures, education, and more. All of these are needed to improve service life, reduce costs and reduce conflicts. Consequently, with growing and developing concrete industry, repair of concrete has always been required and has become a main part of design and construction projects.

However, the repair works, has traditionally known as an art not science. Training repair techniques and performance of repair materials has not been necessary for engineers and contractors. Personal experience came always first, but gaining the sufficient experience takes long time and costly in terms of failed repairs. Most of all, repair failures have changed the public's image of concrete. Because of the premature repair failures and the endless "repair of repairs" the reputation of the concrete reduces. The incidence of premature failures results from a range of factors. These factors include inappropriate selection of repair materials, poor workmanship, and inadequate characterization of substrate concrete [17].

Although the situation is changing, there is still much few information available to estimate the performance of repair jobs. The repair business is greatly expanding with new materials and repair methods. At the same time, due to some changes and regulations, many existing, well-proven products are being redesigned into new products [12].

There are many competent repair materials available at the market and many unconfirmed claims for suitability and success. Even the highest-quality materials may fail if the application is incorrect. Poor repair works fail early or deteriorates the sound concrete material in a quite short period of time. As shown in the Figure 1.1 a good repair improves the function and performance of the concrete structure, whether the structure is a pavement, or a bridge, or a building [12].

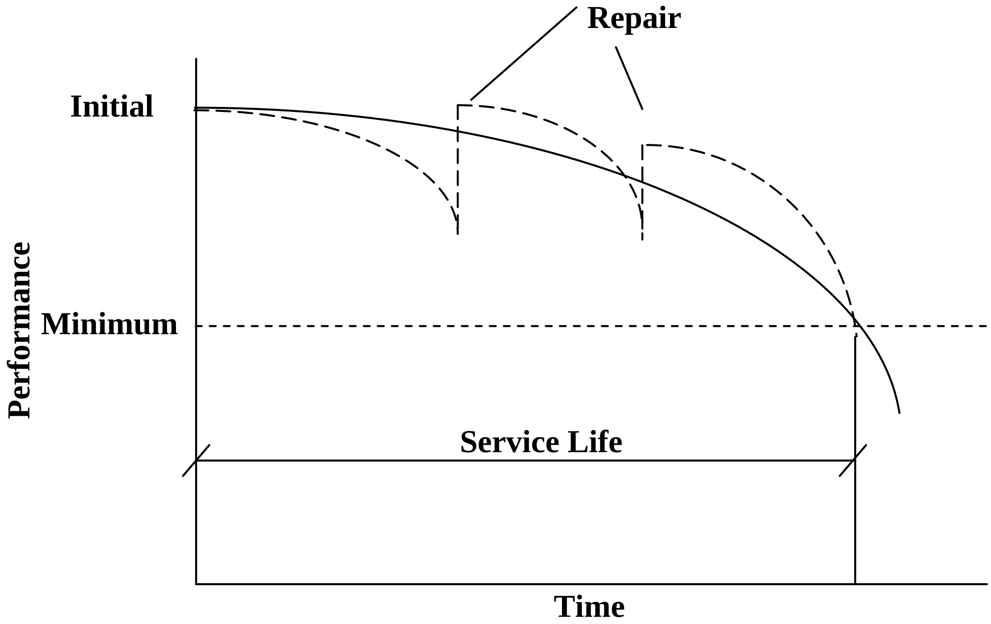


Figure 1.1: Performance and Service Life

Due to the availability of a wide variety of repair materials in the repair industry, with a wide range of economical, physical and mechanical properties, selection of repair material is an important task. According to the previous studies and the literature, the failure of concrete repairs is mainly due to wrong selection of repair material based on the behavior between repair material and substrate concrete [15].

To achieve a durable repair, it is essential that the properties of the repair materials and substrate concrete should match properly. The compatibility between repair material and substrate concrete exists when the composite section resists all stresses induced by applied load under different environmental conditions over the service life. Durability therefore, is a function not only of the properties of the repair materials, but also how such components and the system as a whole respond to load and to the exposure conditions of the structure [15].

1.2 Repair Management

For a good beginning it is necessary to have a planned approach to investigate the current conditions. Sometimes the cause of a situation may not be as it seems, or the cause and effect of the situation is not clear.

Both safety and environmental considerations are major factors in the management of a successful concrete repair project. Safety of workers, residents and visitors is a crucial objective for all projects. Care should also be taken with regard to the impact of the site on the environment. As local authorities become more environmentally aware, following the publication of ISO 14000, the conditions that sites enforce on their surrounding areas must be properly managed.

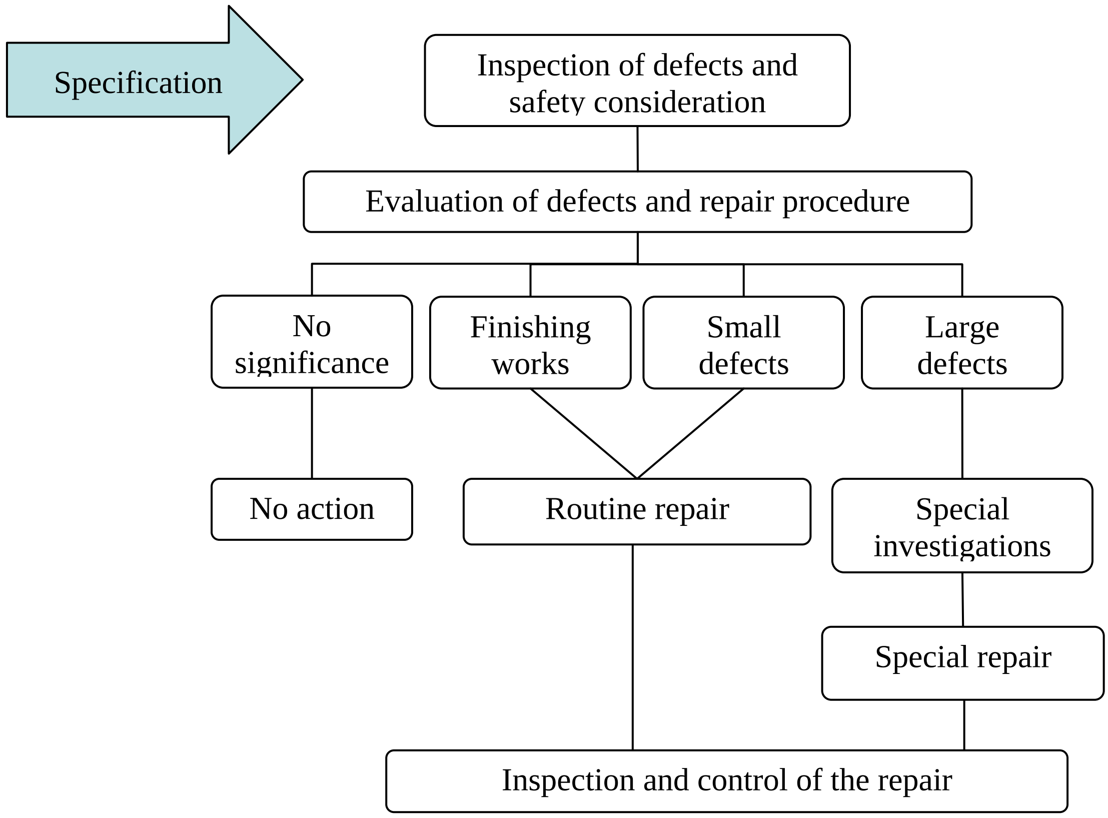


Figure 1.2: Procedure for Repair Management [43, 44]

There are different stages to recognize before starting a repair job. Preparation of detailed drawings, guidelines and specifications are required first. Specific requirements in terms of material specifications should be included. The specification should be clear and comprehensible. Since the full extent of concrete damage may not be completely known until concrete removal begins, plans and specifications for repair projects should be prepared with as much flexibility with regard to material quantities as possible [3]. The procedure for the repair management is shown in the Figure 1.2.

The first stage of a repair is the evaluation of the current condition of the structure after demoulding and the documentation of damage such as it is type and extent and plans of the structure. Conditions where the structure is located may be important for the execution of the work on site. It shall be decided if a visit to the structure is necessary before doing the planning. Information from the examination of the structure such as loads, environmental exposure and possible repair work shall be evaluated. The evaluation may also include a visual examination, non-destructive testing (NDT), crack size measurement, cover control and laboratory analysis of concrete core samples [3, 43, 44].

The second stage involves the evaluation of defects on basis of bearing capacity, aesthetic demands, durability and environmental impacts and economical consequences. On basis of these considerations, it is evaluated whether the defects are of little or great significance or of no importance at all. Normally, defects of no importance are left unrepaired. Large non-conformities require through investigations and evaluation of possible remedial actions [43, 44].

The third stage is the execution of repairs. This is a specialized job and those who have the essential expertise and equipment should be engaged. Because the success of a repair job will depend on the degree to which the work is executed in conformance with plans and specifications. The engineer should have a good knowledge of the procedures and give a considerate organization. In some cases it is required to monitor the efficiency of repairs by some tests before and after the repairs have been performed. Today, the work performed on repair projects requires much more attention to practice than for a new construction. The repair process consists of preparations such s removal of damaged concrete, cleaning and preparation of the surface before application. The second part of the execution is the application and finishing including curing [3, 43, 44].

Though, the work procedures can be divided into two categories. The defects arisen in the production and execution phase, which are only a little significance as regards economy, durability and aesthetics and which occur because of production belong to the routine repair procedures. Large repairs with significance as regards economy, time schedules, durability and aesthetics belong to the special repair procedures. [43, 44]

The last stage of the repair management is the inspection during repairs and after completion. It is necessary to carry out inspections during the execution of work to adjust the demands to the executions of repairs, including preparations. At the end of the work, the repairs are inspected to ensure that they are of the required quality. The final inspection includes testing of adhesion, visual inspections of the surface and of samples and cores [43, 44].

There are different techniques and repair materials available for repair jobs. To achieve durable, effective and economic repairs it is mostly important to select the appropriate material and repair methodology. Matching the repaired parts with the main structures is an important task. A durable construction requires understanding of structural engineering, material science, and environmental exposure conditions. Repair jobs also require the same level of attention in these areas [12].

In practice there is little information in this area. The engineer takes responsibility and should have good knowledge of new materials, repair methodologies, its control and the essentials of structural engineering to guarantee safety and serviceability of the structures during and after repair works.

2. CAUSES OF CONCRETE DETERIORATION

Concrete especially provides excellent protection for reinforcement. But during its service life it will be subject to chemical and physical changes and will be deteriorated.

2.1 General

After completing the inspection of the structure, causes of the deterioration mechanism should be determined. Reinforced concrete, a combination of concrete and steel, is a relatively inexpensive composite material which is widely used all over the world. Its performance is extremely advantageous compared to other construction materials. Concrete especially provides excellent protection for reinforcement. But during its service life it will be subject to chemical and physical changes. The most obvious is the change in appearance caused by natural weathering. A durable concrete differentiates here protecting its performance within its existence [3].

Concrete alone can remain for years durable. It is the reinforced concrete, which is utilized for variety of structural uses. However, reinforced concrete is less durable due to large number of factors, including variations in production, loading conditions and different environmental factors. Although, using a well constituted, properly compacted, and cured concrete may be significantly water tight and durable as long as capillary pores and micro-cracks in the interior do not become interconnected pathways leading to surface of the structure as shown in the Figure 2.1 [3].

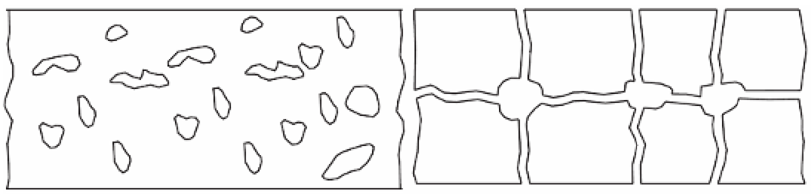


Figure 2.1 : Porous but Impermeable Structure (durable), Porous but Permeable Structure (not durable) [3]

Deterioration of concrete is an extremely complex matter. It is hardly possible to identify a specific, single cause of deterioration for every symptom detected during an evaluation of a structure. In most cases, the damage detected will be the result of more than one mechanism. In spite of the several causes, it should be mostly possible to determine the primary cause of the damage seen on a particular structure.

2.2 Determination of the Causes

It is hard to generalize the causes of the failures in reinforced concrete structures, because of the various physical and chemical factors. It is necessary to have an understanding of the basic causes of damage and deterioration. Here are some of the common causes of deterioration in concrete.

2.2.1 Early age deterioration

Early age deterioration of concrete is a persistent problem that arises from rapid volume changes such as plastic shrinkage, thermal deformation and drying shrinkage. These volume changes cause tensile stresses in the material when strength is relatively low.

In green concrete, the paste has a lower density than the particle density of aggregates so that gravity will tend to pull the heavier particles downwards and the water is displaced upward. This mechanism may cause voids under rebars or large aggregates, plastic settlement cracks, which may create routes for harmful compounds affecting the corrosion process (Figure 2.2). Working with low w/c ratio and better workmanship in vibration and finishing will improve plastic failures.

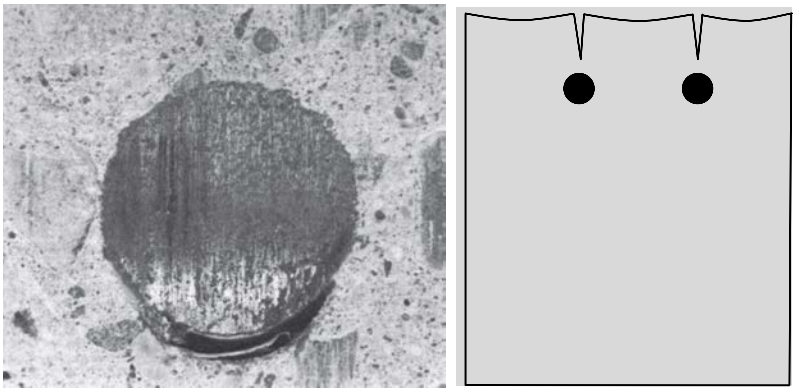


Figure 2.2: Void Under Rebar, Plastic Settlement Over Rebars [1]

The thermal expansion of concrete can be taken in the range 6-13 x10^{-6}/°C. If the concrete is able to expand on heating and contract in cooling without any restraint, there won't be any problems. Especially, thermal cracking may occur by massive concrete constructions because of the high heat production. This can be reduced by using slag cement or mineral admixtures like fly-ash [1].

2.2.2 Deterioration through chemical reactions

Concrete will perform satisfactorily when exposed to many kinds of chemical exposure. However, there are some chemical environments under which the service life of even the best concrete will be short, if there is no protection. That means it is always possible to prevent chemical deterioration or reduce the rate at which it takes place.

Generally harmful chemical reactions occur because of the external chemicals attack the concrete or because of the reactive aggregates used in the concrete. Penetration of chemical solutions through concrete causes the corrosion of the reinforced bars. Reactive aggregates may produce alkali silica gel, which has the property of sucking large amounts of water with a following increase in gel volume. In some cases the expanding gel fills pores and voids in neighboring locations but in some cases the expanding gel applies high pressure that cracking occurs. If the concrete dries, the gel shrinks and opens the cracks wider. In addition, chemical attack, including acids and sulfates may have a harmful effect on the concrete itself. When external sources of such chemicals are in contact with hardened concrete they can react with the outer surface, but if the concrete is porous they may be penetrate to react into the concrete. Barrier protection systems are used to minimize the effects of chemicals. Concrete which has been damaged by contact with chemicals can be repaired by removal of the damaged layers until sound concrete has been reached [2, 12].

2.2.2.1 Corrosion of rebars

Penetration of chemical solutions in to the concrete contributes to the corrosion of the embedded steel, resulting damage of concrete (Figure 2.3). The high alkalinity of the concrete, usually pH above 12, leads to an oxide coating passive layer forming on the rebars that reduces the possible rusting. The passivation zone can be destroyed by high levels of ions which form soluble iron compounds. Chloride ion in water is the most common cause of this depassivation and local corrosion of rebars with reduction of the cross section [42].

The other one is the carbonation of concrete, which leads to early cracking and spalling with comparatively little reduction of rebars. When the depth of carbonation has reached reinforcement, the paste in contact with the metal loses its alkalinity and the passivation zone will be destroyed by oxygen. Because it begins from the outer surface of the concrete, rebars near to the surface are in danger of carbonation and are not protected against corrosion. Barrier protection systems are commonly used to minimize the effects of corrosion [42].

Figure 2.3 : Spalling of Concrete due Carbonation and Chloride [42]

2.2.3 Freezing and thawing

As water turns to ice, there is an increase in volume of about 9%. When porous concrete is saturated with water this expansion on freezing may lead to damage (Figure 2.4). Use of de-icing salts containing chlorides increases the chance of frost damage. To prevent hardened concrete from frost damage, air is entrained into the fresh concrete using an admixture which creates about 1 mm small and evenly dispersed air bubbles. The water can expand freely without disrupting the concrete into the voids. However, concrete, with a 5% air entrainment may become a strength reduction of about 15% [2].

Figure 2.4 : Freeze and Thaw at a Wall [2]

2.2.4 Weathering and fire damage

Weathering is the deterioration of the porous outer surface of concrete caused by the effects of sunlight rain, frost, and atmospheric pollution. The result is a change in appearance. This mechanism damages only the outer skin of concrete, underlying body remains protected.

Concrete provides the best fire resistance of any building material. However if it heated over 600°C dehydration begins which leads to loss of strength and concrete wont function at its full structural capacity. Even at 250°C some spalling take place and strength loss begins at the exposed surfaces. Using fibers prevents spalling and affected surfaces can be strengthened after.

2.2.5 Construction errors

Usually, most of the construction errors do not lead directly to deterioration. Errors made during construction such as adding improper amounts of water to the concrete mix, inadequate consolidation, inadequate formwork, improper location of rebars and improper curing may cause distress and deterioration which results cracking of the concrete. Cold joints, exposed reinforcing steel, irregular surface, honeycombing and bug holes can be observed at any concrete structure (Figure 2.5).

High water to cement ratio leads to high capillary porosity in cement paste which allows the aggressive chemicals to penetrate easily and allows reinforced concrete to get corroded. Lessened cover thickness allows concrete to get affected earlier against external environmental effects. Inadequate vibration produces many unexpected entrapped air voids and concrete gets porous [3].

Mostly seen construction errors occur because of the cover thickness faults, much or less vibration, improper finishing and premature removal of the formwork. Proper mix design, placement, and curing of the concrete, as well as an experienced contractor are necessary to prevent construction errors before occurring. Daily staff meetings during construction phase, repeated courses and training for workers may reduce many of the construction errors [6, 12].

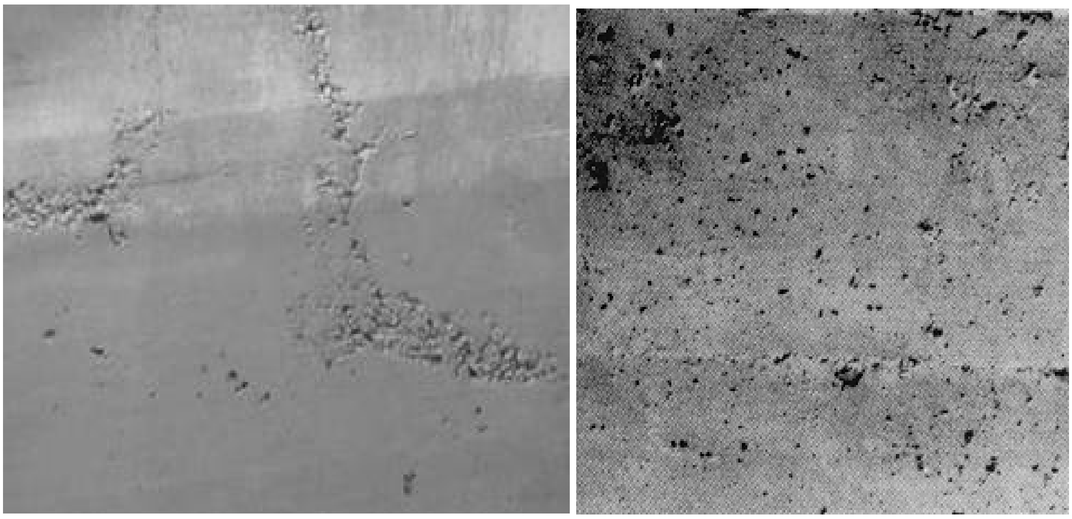


Figure 2.5 : Honeycombing and Bug Holes on the Surface [2]

2.2.6 Design errors

Because of the inadequate structural design concrete, exposed to greater stress than its capable of carrying it, will crack. Similarly high torsion or shear stress may result in spalling or cracking. Poor detailing is another reason for cracking through localized stress concentrations and cracking allows water or chemicals access to the reinforced concrete. Reduction in length, area, or volume of concrete due to creep, shrinkage, or both, affects the structures serviceability and durability. Insufficient joints in slabs are the most frequent causes of cracking. There are much more specific types of poor detailing and its possible effects on a structure. The design aspects should aim at minimizing the size and number of joints and cracks caused by thermal effects, creep and shrinkage. Generally, a careful review of all design calculations is the easiest way to prevent such errors [6, 12].

2.2.7 Accidental loadings

Accidental loadings are designated as short-duration, one-time events such as the impact of an earthquake, which may generate stresses higher than the strength of the concrete. All these bring many tragedies, bad economical consequences and human deaths, we saw in Erzincan 1939 or in Marmara 1999. Usually, damage caused by accidental loading will be easy to decide. Because of the wrong assessment of design loads deflections, crushing or cracking of structural members can occur, which allows the aggressive chemicals from its environment to penetrate in to the reinforced concrete. It is impossible to prevent accidental loading, only the effects can be minimized and the impacts can be reduced by proper design procedures [6, 12]

3. CONCRETE REMOVAL, CLEANING AND PREPARATION

The technique and the material used for the repair work are the most important factor to determine the repair life. But without the care during the removal and preparation stages of a repair work both of the factors are of no avail. This part of the work covers the removal techniques of the old concrete and cleaning and preparation of the surface for the repair materials.

3.1 Concrete Removal

It is essential that all of the deteriorated concrete be removed before repair materials are applied to provide sound concrete for the repair material to bond to. It is always false economy to attempt to save time or money by shortchanging the removal of deteriorated concrete. Whenever possible, the first choice of concrete removal technique should be high pressure hydro blasting or hydro demolition. These techniques have the advantage of removing the unsound concrete while leaving high quality concrete in place and they do not leave micro cracked surfaces on the old concrete. Impact removal techniques, such as bush hammering, scrabbling, or jack hammering, can leave surfaces containing a large amount of micro cracks which seriously reduce the bond of the repair material to the existing substrate (Figure 3.1).

Figure 3.1 : Impact Removal Technique

Subsequent removal of the micro cracked surface by hydro blasting, shot blasting, or by sandblasting may be required if impact removal techniques are used. A disadvantage of the high pressure water blasting techniques is that the waste water and trash must be removed in an environmentally acceptable way according to the regulations [6].

Impact concrete removal techniques, such as jack hammering for large jobs and chipping for smaller areas; have been used for many years. These removal procedures are quick and economical, but it should be kept in mind that the costs of subsequent removal of the micro cracked surfaces resulting from these techniques must be included when comparing the costs of these techniques to the costs of high pressure water blasting. The larger jackhammers remove concrete at a high rate but are more likely to damage surrounding sound concrete. The larger hammers can impact and loosen the bond of concrete to reinforcing steel for quite some distance away from the point of impact. Pointed hammer bits, which are more likely to break the concrete cleanly rather than to pulverize it, should be used to reduce the occurrence of surface micro cracking [6].

Shallow surface deterioration, usually less than 1.5 cm deep, is best removed with shot blasting or dry or wet sand-blasting. Shot blasting equipment is highly efficient and usually includes some type of vacuum pickup of the resulting dust and debris (Figure 3.2.). The use of such equipment is much more environmentally acceptable than dry sand blasting. Shallow deterioration to concrete surfaces can also be removed with tools known as scrabblers. These tools usually have multiple bits which hammer and pulverize the concrete surfaces in the removal process. Their use multiplies the micro fractures in the remaining concrete surfaces. Extensive high pressure water, sand, or shot blasting efforts are then needed to remove the resulting damaged surfaces. Such efforts are seldom attained under field conditions [4].

Figure 3.2 : Shot blasting Equipment to Remove Shallow Concrete Deterioration [4]

3.2 Surface Preparation

One of the most important steps in the repair of a concrete structure is the preparation of the surface to be repaired. The repair will only be as good as the surface preparation, regardless of the nature of the repair material. For reinforced concrete, repairs must include proper preparation of the reinforcing steel to develop bond with the replacement concrete to ensure desired behavior in the structure [4].

After the repair area has been prepared, it must be kept clean, protected and cured. In hot climates, this might be done by providing shade to keep the concrete cool, so reducing rapid hydration or hardening. In winter, steps need to be taken to provide sufficient insulation to prevent the repair area from being covered with snow, ice, or snowmelt water. It should be remembered that repair activities can also contaminate or damage an appropriately prepared region. Workmen placing repair materials in one area of a repair often track mud, debris, cement dust, or concrete into an adjacent repair area. This material will act as a bond breaker if not cleaned up before the new repair material is placed. The prepared concrete should be kept wet or dry, depending upon the repair material to be used. Surfaces that will receive polymer concrete or epoxy-bonded materials should be kept as dry as possible. Some epoxies will bond to wet concrete, but they always bond better to dry concrete. Surfaces that will be repaired with cementitious material should be in a saturated surface dry condition immediately before application. This condition is achieved by soaking the surfaces with water for 2 to 24 hours just before repair application. Immediately before material application, the repair surfaces should be blown free of water, using compressed air. The SSD condition prevents the old concrete from absorbing mix water from the repair material and promotes development of adequate bond strength in the repair material. The presence of free water on the repair surfaces during application of the repair material must be avoided [4].

3.3 Curing

All of the standard repair materials, with the exception of some of the resinous systems, require proper curing procedures. Curing is usually the final step of the repair process, followed only by cleanup and discharge, and it is fairly common to find that the curing has been shortened, performed unevenly, or eliminated entirely as a result of rushing to leave the job or for the sake of perceived economies. It should be understood that proper curing does not represent unnecessary costs. Rather, it represents a sound investment in long-term insurance. Inadequate or improper curing can result in significant loss of money. At best, improper curing will reduce the service life of the repairs. More likely, inadequate or improper curing will result in the necessity to remove and replace the repairs. The costs of the original repair are, thus, completely lost, and the costs of the replacement repair will be greater because the replacement repairs will be larger and must include the costs of removal of the failed repair material [4].

Failure to cure properly is the most common cause of failure of replacement mortar. It is essential that mortar repairs receive a moisture cure starting immediately after initial set and continuing for 14 days. In no event should the mortar be allowed to become dry during the 14-day period following placement. Following the 14-day water cure and while the mortar is still saturated, the surface of the mortar should be coated with curing compounds. If this curing procedure cannot be followed or if conditions at the job are such that this curing procedure will not be followed, money would be saved by using another repair material [6].

Epoxy mortar repairs should be cured immediately after completion at not less than the temperature range given by the class of the epoxy until the mortar is hard.

Posturing, if required by the specifications, can then be initiated at elevated temperatures by heating in depth the epoxy mortar and the concrete under the repair. Epoxy-bonded epoxy mortar should never be subjected to moisture until after the specified posturing has been completed. Even though an epoxy bond coat is used, it still remains essential to properly cure epoxy-bonded concrete. As soon as the epoxy-bonded concrete has hardened sufficiently to prevent damage, the surface should be cured by spraying lightly with water and then covering with an overlay or by coating with a curing compound [4].

Polymer concretes polymerize and harden very quickly under most ambient conditions and will develop nearly full strength within a 1-2 hour period. During this time, the fresh concrete must be protected from water.

The coated surfaces must be protected until the resin has completely cured to a hard finish. Such condition will be obtained within about 30 hours of application of the final topcoat. Low ambient temperatures or high relative humidity may change the hardening time [4].

Figure 3.3 : Curing of the Repair

4. PLANNING AND DESIGN OF CONCRETE REPAIRS

Concrete structures damaged by various mechanisms need to be repaired in order to maintain safety, appearance and durability to extend their service life. The main objective of any repair should be to maintain a durable repair. Planning and design of a repair is the major step for performing durable and reliable repairs.

4.1 General

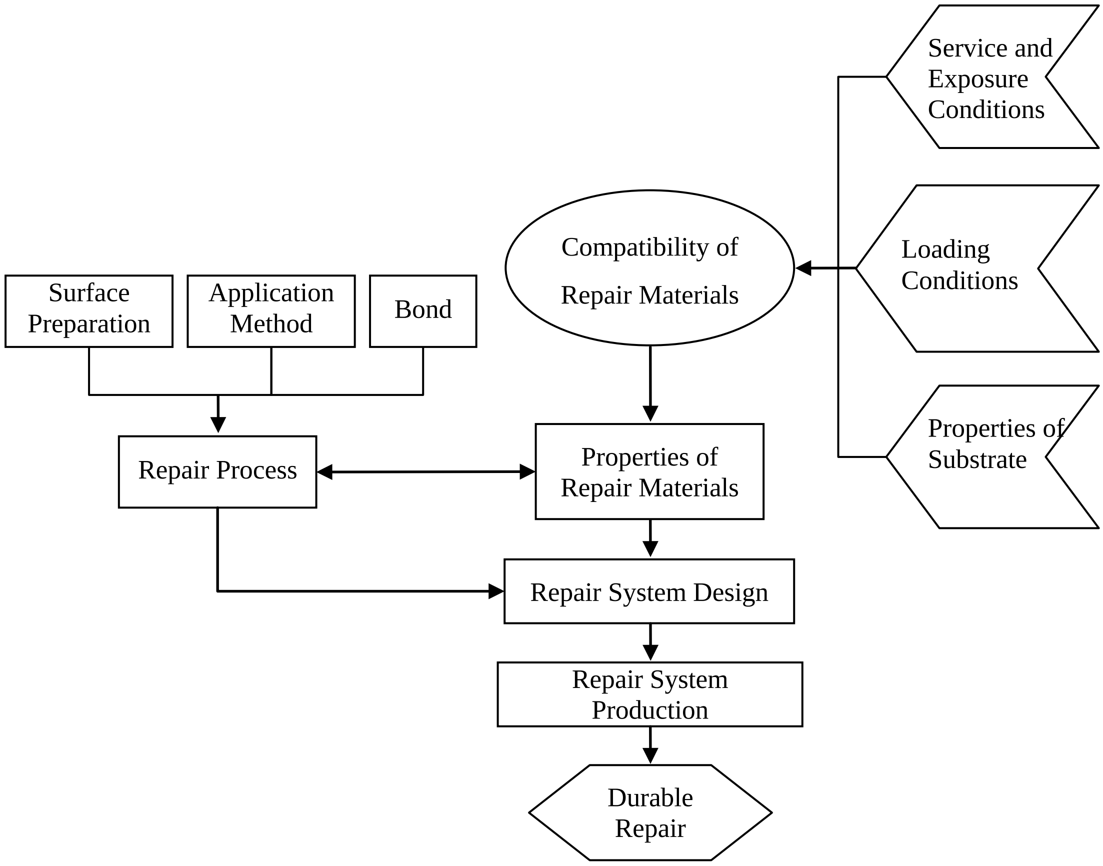


Figure 4.1 : Factors Affecting the Durability of Concrete Repair Systems [16]

Concrete structures damaged by various mechanisms need to be repaired in order to maintain safety, appearance and durability to extend their service life. The main objective of any repair should be to maintain a durable repair. As shown in the Figure 4.1 factors affecting the design and selection of repair systems are considered as components of one composite system. The proper repair depends on the evaluation of the causes of deterioration. Selection of a repair material is one of the many major steps for making durable and reliable repair; equally important properties are availability of materials, equipment, skilled labor, surface preparation, the method of application, construction practices, and inspection [16].

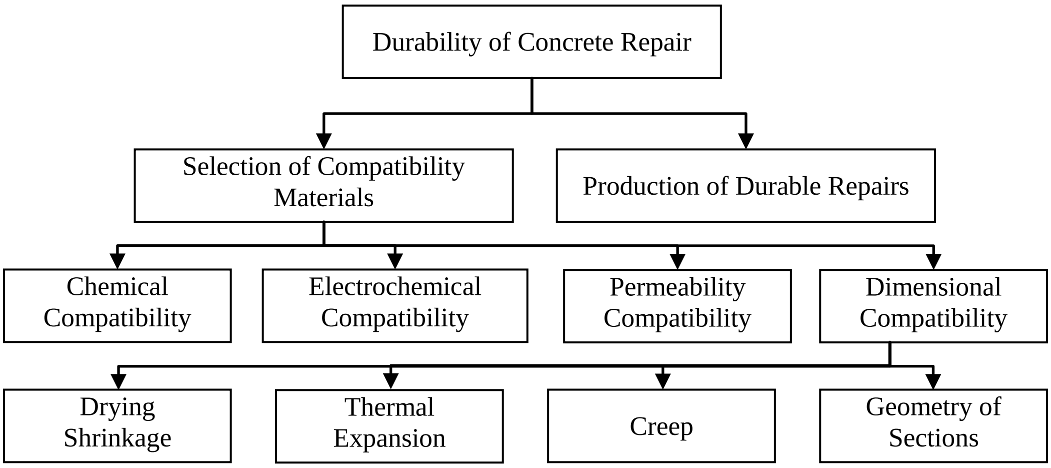


Figure 4.2 : Durability of Concrete Repairs due to Compatibility [15]

Factors affecting durability of repair system are shown in Figure 4.2. These factors must be considered in the design process to make the compatible repair material selection. Compatibility is defined as the balance of physical, chemical, and electrochemical properties and dimensions between the repair material and the old concrete without distress and deterioration over a designed service life. However dimensional compatibility, which is the phenomenon of volume changes, is one of the major problems affects the durability and strength of repairs. Restrained volume changes of the repair, the restraint being provided through bond, causes cracking and debonding of the repair work [15].

Good compatibility between the repair material and the substrate ensures a repair with a limited and predictable degree of change over time, where the repair material can withstand stresses resulting from volume changes and load for a specified environment over a designated period of time without experiencing distress and deterioration Consequently, the selected repair material should satisfy the dimensional compatibility with the old concrete. Properties which influence dimensional compatibility are drying shrinkage, thermal expansion, modulus of elasticity, geometry of sections and creep [15].

4.2 Selection of Repair Materials

Each damaged structure demands different application method and repair material. The repair material should meet these demands for a durable repair. Among these there are some practical problems with the execution of the work and environmental considerations such as noise and dust caused during removing old concrete. And of course different materials have different properties and limitations.

At this time, there are hundreds of prepackaged repair materials on the market. On one hand this is a great opportunity to make a correct choice for special application, on the other hand it increases the possibility of making a wrong selection. Even the highest-quality materials do not perform as expected if they are used inappropriately. Often it is difficult to make an evaluation of the needed repair material for a specified repair job, because test data are not available or, if they are, either they are not presented in appropriate terms or it is not possible to make a comparison with other competing materials through the use of nonstandard or modified test methods [13].

Consequently, repair work should be specified by an experienced person or company because the final choice of repair method and materials depends on many factors. The specialists should have a through understanding of how each method is executed and how the required material properly selected. Some properties, required of repair materials when compared with the concrete substrate to produce long-term structurally efficient repairs are listed on Table 4.1 [17].

Table 4.1 : Requirements of Patch Repair Materials for Structural Compatibility [17]

Property	Relationship of repair mortar (R) to concrete substrate (C)
Strength in compression, tension, and flexure	R≥C
Modulus in compression, tension, and flexure	R=C
Poisson's ratio	Dependent on modulus and type of repair
Coefficient of thermal expansion	R≈C
Adhesion in tension and shear	R≥C
Curing and long-term shrinkage	R≤C
Strain capacity	R≥C
Creep	Dependent on whether creep causes desirable or undesirable effects
Fatigue performance	R≥C
Chemical reactivity	Should not promote alkali-aggregate reaction, sulphate attack, or corrosion of reinforcement in the substrate
Electrochemical stability	Dependent on permeability of patch material and chloride ion content of substrate

Figure 4.3 shows an organized approach that is required in the selection of a repair material, which accounts for all applicable parameters and their impacts on the choice between alternatives [1].

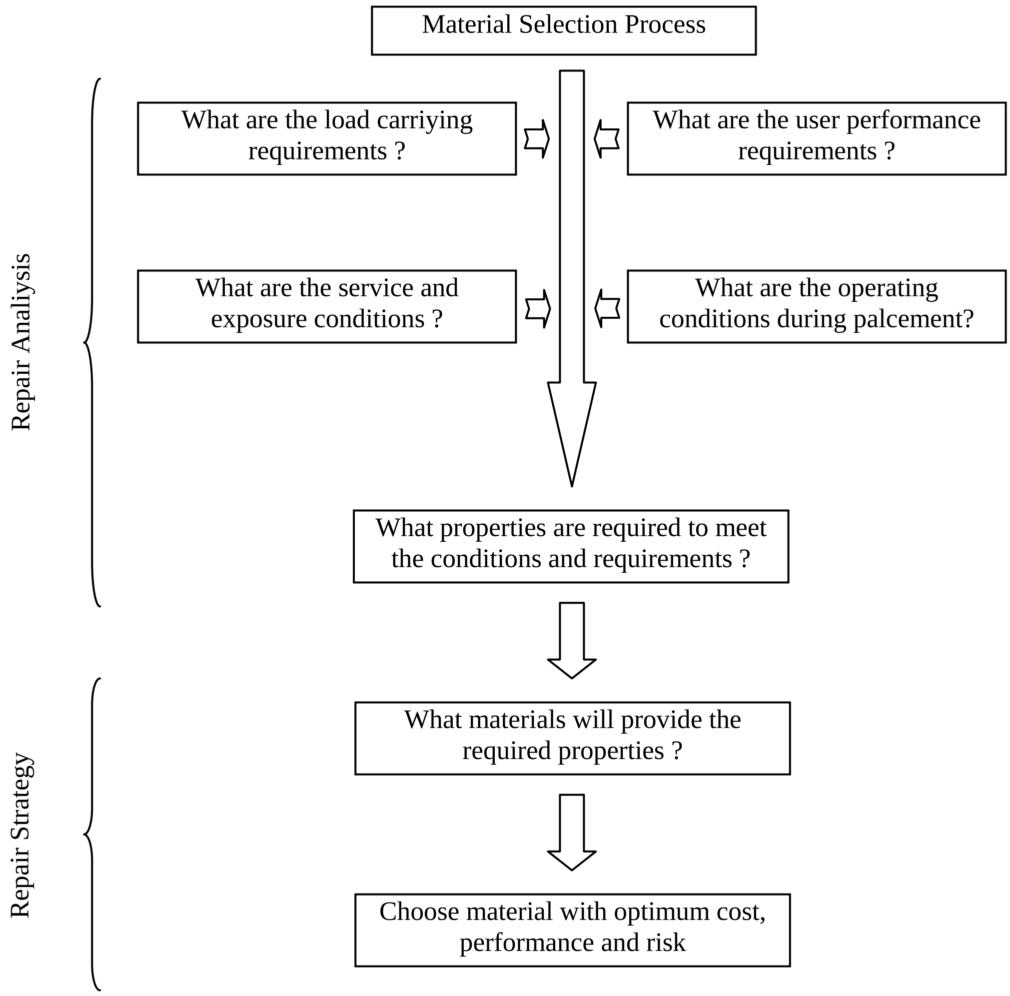


Figure 4.3 : The Selection Process for a Repair Material [1]

4.3 Types of Repair Materials

The repair types and relevant repair materials are classified in two categories. First one is for the crack injection and the other one is for spalling and disintegrations [6].

4.3.1 Crack repair

Cracks in concrete may affect appearance only but they are often a sign of a trouble that requires a solution before any serious failure occurs. They are usually classified based on width, stability. Cracks which are smaller than 0.05 mm in width are generally defined as ordinary cracks. Furthermore, very thin cracks may heal autogenously while hydration process of the cement. It is a natural process in presence of moisture. Larger cracks are impregnated with a resin of low viscosity under vacuum. There are several types of resins for impregnation, epoxy resins are the most known of them. Epoxy resins are always used with a hardener, well proportioned and mixed. Polyurethane chemical grouts are another common vacuum impregnation choice usually used to repair wet and active cracks [11].

Cracks larger than 2.5 mm are repaired with polyurethane, silicone sealants or polymer, polymer-cement and cementitious grouts. Properties and preparing procedures may differ but application procedures are similar. They easily mixed by hand or in a mixer until a homogeneous mixture are achieved. These materials can be hand applied without requiring any special equipment or skilled worker and poured in to the cracks [13].

4.3.2 Concrete replacements and overlays

Concrete replacements are required when spalling and disintegration occurs. There is no single method and material for concrete replacements. The most commonly used material for concrete replacement is good quality Portland cement concrete. It has many advantages when used as a repair material, because properties like modulus of elasticity and thermal expansion are parallel to those of the damaged concrete. Some other properties concerning durability can be improved with chemical and mineral admixtures such as silica-fume. Using another type of cement like polymer cement and magnesium-ammonium-phosphate cement (MAPC) may be a good solution for special applications when reduced permeability, rapid strength gain or volume stability is demanded. Preparing mortar mixes excluding coarse aggregates is another solution with some disadvantages, like high shrinkage behavior and varying hardening properties. But there are some prepackaged repair mortars commercially available. They offer more predictable performance through special admixtures and proprietary constituents. They can be easy prepared and applied on site in every condition and also performance test results are always available from manufacturer or from previous works. But they have a limited storage life [13].

Selection of the material and execution of the repair changes with different repair thickness and repair location. Overlays thicker than 19 to 25 mm are known as deep concrete replacements repaired with any repair concretes. Shallow replacements are mortars about 1.6 to 3.2 mm thick and thin overlays used for surface defects are coatings less than 3.2 mm. With decreasing repair thickness workmanship procedures like mixing, placing and curing become significant. [13].

For deep concrete applications there are different solutions for horizontal and for vertical repairs. Concrete is mostly used material for horizontal repairs, or it can be modified with silica-fume, which is more expensive but more durable than conventional concrete. For vertical applications, workability and curing against gravitational forces and bonding ability to the old concrete should be considered. Therefore, several construction methods are available. Form-and-cast method, preplaced-aggregate method, shotcreting and application with trowel are some of them [6, 13]. There is a detailed repair material selection guide of ACI in Table A1.

4.3.3 Bonding agents

There are different types of bonding agents with different modes of action and different content of chemicals, characterized by thickness, material type, coating method and function. The most common bonding agent is high viscosity cement based mortar. In cases where a bonding agent is to be used, surface preparation should be done with care and should not be allowed to dry out before the repair is applied. The application should be done easily by spraying or booming. There are various epoxies and other polymer bonding agents available on market, if one of these products is used, the manufacturer’s guide must be followed [12].

4.4 Properties of Repair Materials and Evaluation of a Repair

Even how carefully a repair is done, use of wrong material will cause to premature repair failure. There are some properties during fresh, hardening and hardened condition of the repair materials which are essential for material selection and repair evaluation. Some of those properties and test methods to evaluate them and their relevance for a durable repair are expressed in the following text.

4.4.1 Workability

Workability of repair material is defined by constructability characteristics which may affect the ease of application of the repair material under several conditions. Cohesiveness, viscosity and repair environment are the main parameters for workability. Cohesiveness provides stability that prevents segregation and debonding during repair, especially repairs on vertical surfaces. Viscosity is defined as the resistance to flow and can be determined with flow tests. Materials with low viscosity are suitable for crack repair. Environmental conditions such as relative humidity, wind and sun, affect not only workability but also performance of the repair material negatively when they are neglected [3].

4.4.2 Setting and hardening

Since the repair materials set so rapidly, attention must be paid to how long it takes to mix and place the repair material, or else it will harden too fast and not bond appropriately. Setting time of the repair materials are usually measured with a Vicat apparatus according to a modified ASTM C 191, test method for time of setting of hydraulic cements [3].

4.4.3 Shrinkage

Drying shrinkage, after placing the repair material is a compatibility problem with the substrate concrete. It is well known that the cementitious repair materials shrink within the first few hours after placing which is the cause of debonding or cracking on the surface. These cracks are known as shrinkage cracks which allow an easy access for harmful components. This effect can be reduced by using mixtures with low w/c ratios and shrinkage reducing admixtures. Of course proper curing is vital.

There are various test methods to evaluate the shrinkage properties of repair materials in the laboratory and on the field. The modified ASTM C 157 - Standard test method for length change of hardened hydraulic-cement mortar and concrete, is used to determine the length changes that are produced by causes other than externally applied forces and temperature changes in hardened hydraulic cement mortar and concrete specimens made in the laboratory and exposed to controlled conditions of temperature and moisture. ASTM C 928 explains how to modify this test for repair materials [13, 28]. The classification of the shrinkage properties are shown in the Table 4.2.

Table 4.2 : Classification of Shrinkage Properties

Class	Strain [%]
Low Shrinkage	0-0.05
Moderate Shrinkage	0.05-0.1
High Shrinkage	0.1-0.3

Ring test (Figure 4.4) allows the determination of materials sensivity to cracking caused by restrained volume changes. The ring is monitored daily for evidence of cracking and the day that cracking is observed is recorded and the initial crack width is measured.

Figure 4.4 : Ring Restraint

The Structural Preservation System (SPS) plate test specimen was a nominal 51- by 102- by 1.321-mm beam (Figure 4.5). As the material expanded or contracted in response to moisture and temperature changes, deflection of the unrestrained end of the specimen is measured [9].

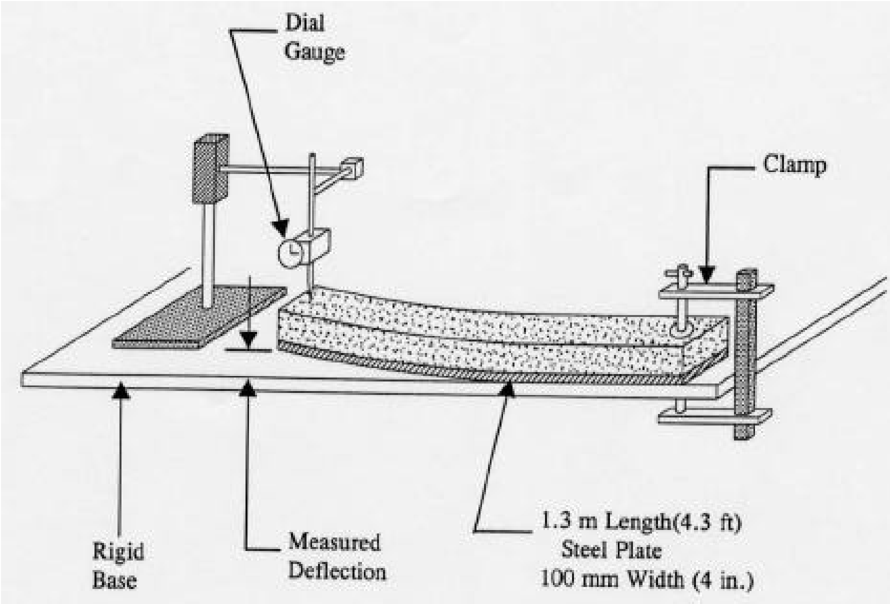


Figure 4.5 : SPS Plate Test **[9]**

The German angle test consists of 70- by 70-mm steel angles that are 1.0 m long (Figure 4.6) with a repair material. After casting, the test specimens are monitored for cracking under field-exposure conditions. Both, the SPS Plate and German Angle Tests can be used for a general assessment of a material's dimensional compatibility, or resistance to cracking [8].

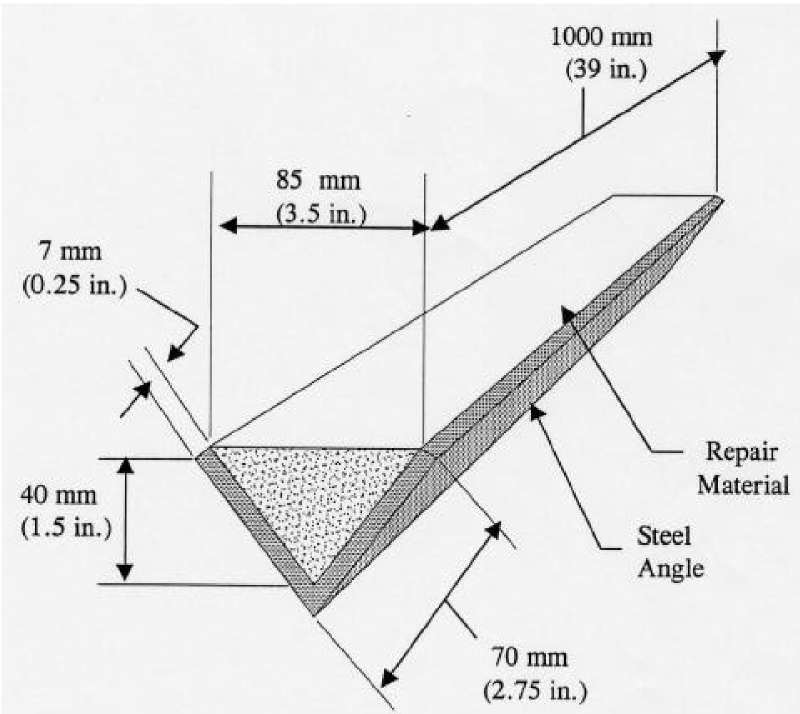


Figure 4.6 : German Angle Test [9]

4.4.4 Thermal expansion coefficient

Volume changes due to contraction or expansion of the materials because of the variations in temperature may cause cracking and debonding in repaired regions. The amount of the volume changes depends on the coefficient of thermal expansion. Non-cementitious materials like epoxy or polymeric binders with high thermal expansion coefficients are more sensitive than cementitious materials. Coefficient of thermal expansion can be determined according to ASTM C 531 - Standard test method for linear shrinkage and coefficient of thermal expansion of chemical-resistant mortars, grouts, and monolithic surfacing [13].

4.4.5 Mechanical properties

Repair materials should have compatible mechanical properties than the substrate to ensure uniform stress distribution and uniform strains under different loading conditions. There are some characteristics to determine mechanical properties of a repair material and repaired structure: Compressive strength, tensile strength, flexural strength, modulus of elasticity, creep and bond strength.

Compressive strength is the ultimate failure stress determined on 28 days under 20°C moisture cured specimens. Generally, it is not an important property in many repair applications. It is expected that the repair material have strength similar to or greater than the concrete substrate. ASTM C 39 and ASTM C109 are the test methods available for compressive testing (Figure 4.7) [13].

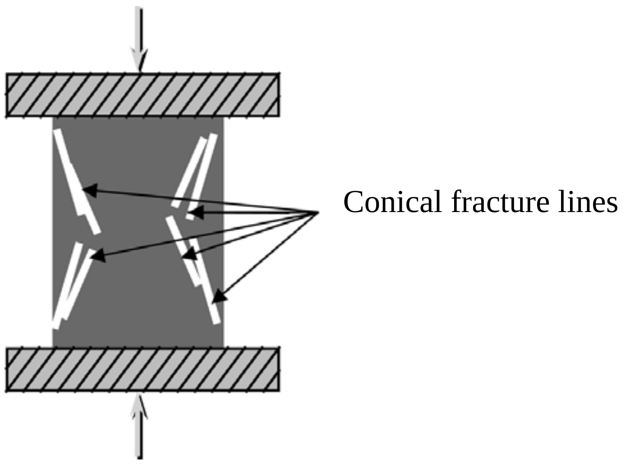


Figure 4.7 : Compression Test Setup [13]

Tensile strength is the ultimate stress under axial tension loading. A tensile force can be generated by a combination of external loading, volume changes and poor compatibility in the properties of the repair and the concrete. Exceeding the repair materials ultimate tensile capacity will cause of cracking, spalling or debonding.

It is generally observed that a repair section is mostly performed at the joints or in the tension area. Tension is created in the concrete by bending of the structure due to loading . Therefore, flexure test method would be an appropriate method to study the compatibility between repair and substrate material. Flexural strength is defined as the ultimate bending capacity of concrete. It is determined with three point bending test either with one or two loading points. ASTM C 78 - Standard test method for flexural strength of concrete is modified by Czarneck et al. 1999 to investigate the composite beam behavior with repair materials. The repair applied on the bottom of the concrete prism is evaluated compatible or incompatible with the substrate by the mode of failures (Figure 4.8) [10, 13].

It is well known that a stiffer material deflects less in the flexure test compared to a weaker material under the same loading. In the composite beam, if the compressive strength of the repair material is greater than the strength of substrate concrete, the stress-strain curve should have greater slope than the slope of the stress-strain curve of substrate concrete beam itself. If not, then the load transfer to repair material is not adequate and the repair material is not compatible with the substrate concrete [10].

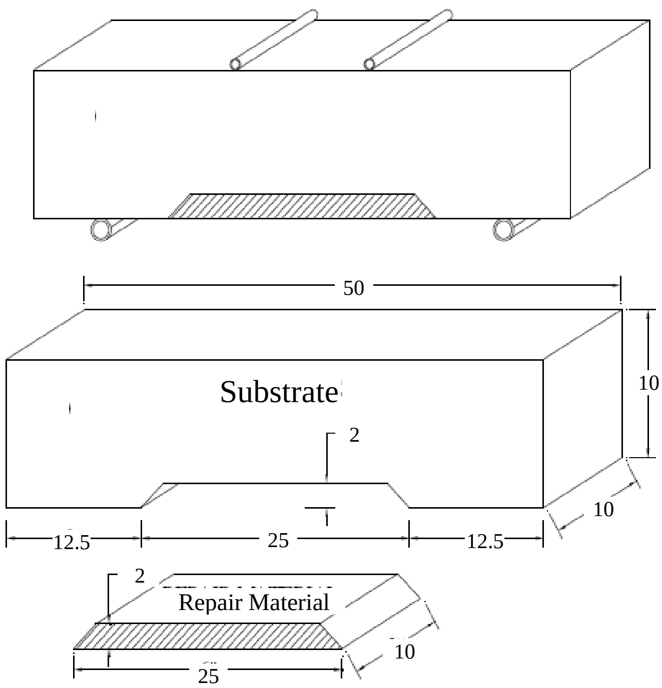


Figure 4.8 : Composite Beam Test Specimen (dimensions in cm) [7]

Modulus of elasticity of the repair material should be similar to the substrate concrete, especially for structural repairs. Variations between repair and the concrete can lead to uneven load distribution. If the repair material has a higher modulus of elasticity, it will attract more of the applied load; if it has a lower modulus of elasticity, deformation occurs and the load is transferred to the concrete. For nonstructural projects expectations changes, with low modulus elasticity repair material volume stability and related compatibility can be achieved easily, the potential for cracking and Delamination is reduced. ASTM C 469 is the standard test method to determine the modulus of elasticity under compression. (Figure 4.9) [13].

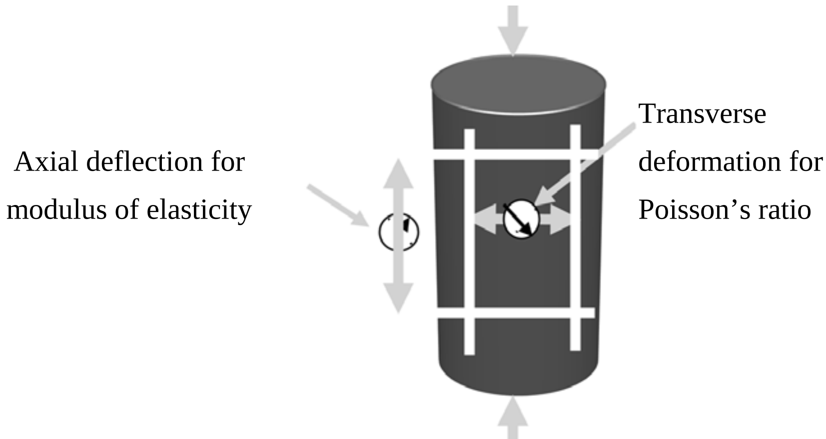


Figure 4.9 : Determining Modulus of Elasticity [13]

Bond strength is the resistance of the repair material to separation from the old concrete. Generally good bond quality of the repaired region is the primary requirement for a successful repair. There are many types of pull techniques to determine the adhesion of bonded toppings by tensile load. The pull-off test, CAN A23.2-6B setup shown in the Figure 4.10, is the mostly known test procedure to determine the bond between concrete substrate and repair materials. For this test a cylindrical semi-core sample is prepared and a tension force is applied to produce either a bond or nonbond failure. If the specimen fails away from the bonded area, bond strength is greater than the failure load in the test. If it fails at the bond area, the measured load is the bond strength. But this technique is sensitive to material mismatch, eccentricity of coring and coring depth into the substrate. Because of the improper preparation the pull-off load will reduce [19].

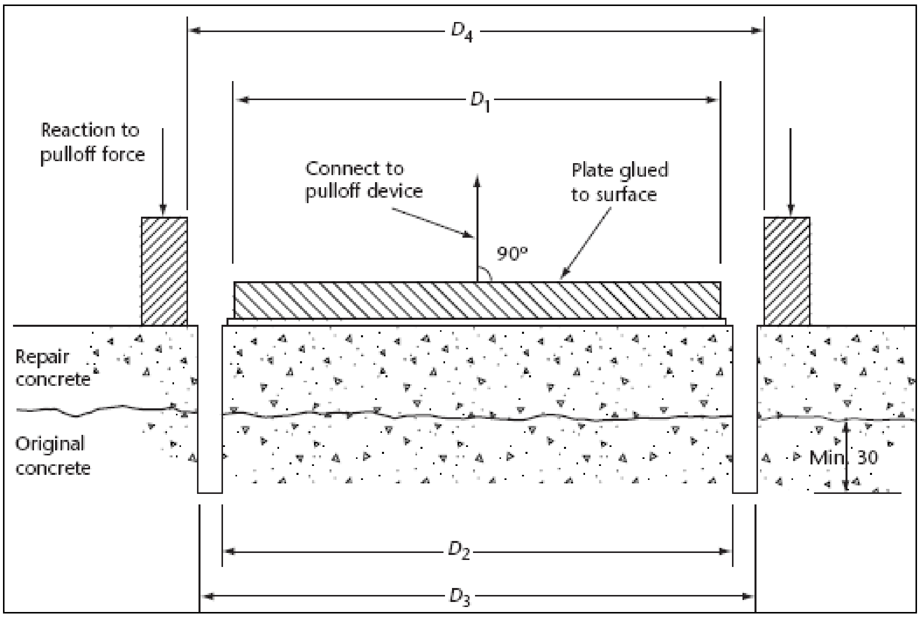


Figure 4.10 : Pull-Off Test (dimensions in mm) [24]

The second category measures the bond strength under a state of stress that combines shear and compression. The slant shear test ASTM C 882 to determine the bond strength by measuring the resistance to sliding between repair and the concrete along an inclined interface of the composite cylinder under compression, falls under this category. A square prism or a cylindrical sample made of two equal halves bonded at 30 degrees and tested under axial compression (Figure 4.11) [19].

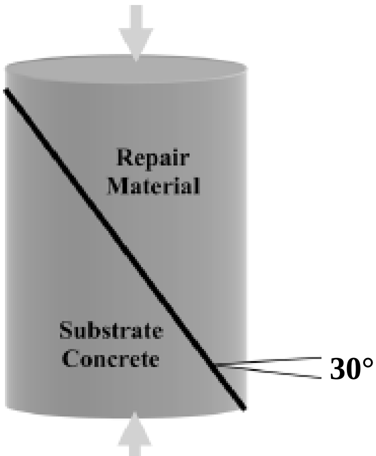


Figure 4.11 : Slant Shear Test Setup [13]

4.4.6 Permeability

Permeability is important when durability of the repair is concern. Penetration of water, water soluble chemicals and gases may cause or trigger incidents such as freeze and thaw, corrosion of rebars, alkali-silica reactions and sulphate attack. Thus, repair material should resist the penetration of harmful substances. Permeability generally changes with the water content, age of the material and size and content of the fine material.

Permeability of water into the repair mortars is measured through capillary water absorption based on weight recording. The increase in the mass of specimen resulting from absorption of water is measured as a function of time when only one surface of the specimen is exposed to water. The exposed surface of the specimen is immersed in water and water access of unsaturated mortar dominated by capillary suction during initial contact with water. The rate of absorption of water as a function of time is determined by measuring the increase in the mass of a specimen. The absorption, I, is the change in mass divided by the product of the cross-sectional area of the specimen and the density of water. The rate of absorption is defined as the slope of the line that is the best fit to absorption plotted against the square root of time in seconds. Normally there two different slopes defined as the initial rate of absorption and the secondary rate of absorption [25].

For chloride penetration there are two types of common testing. ASTM C1202 provides an approach to the resistance against chloride. The electrical conductance of the core samples are determined to provide a rapid indication of its resistance to the penetration of chloride ions. But this method is only applicable to types of samples where correlations have been established between this procedure and long term chloride ponding procedures, such as NT BUILD 443. In Table 4.3 there are values from standard to evaluate the test results [26].

Table 4.3 : Chloride Ion Penetrability Based on Charge Passed [26]

Charge Passed [coulombs]	Chloride Ion Penetrability
>4000	High
2000-4000	Moderate
1000-2000	Low
100-1000	Very Low
<100	Negligible

NT BUILD 443 specifies a procedure for the determination of penetration parameters for estimating the resistance against chloride penetration into the hardened samples [30].

4.4.7 Microstructure analysis

Microstructure allows engineers to identify concrete deterioration by controlling the properties and the performance of the concrete through its microstructure (cracking, loss of mass, loss of strength, appearance degradation, or changes in chemical makeup) and engineers to choose appropriate repair strategies. Therefore petrography takes an important role in the concrete repair industry. Micro analysis allows the investigator to identify the causes of deterioration, to determine the composition, texture, and current condition of the concrete, to determine the degree, location, and extent of the deterioration and to evaluate whether the deterioration will continue. It is also probable to predict a future damage and provides information on the three common causes of repair failure such as improper materials, poor workmanship, and poor design [22].

The most known method for microstructural analysis is the optical fluorescence microscopy. The method is established and has been used for many years in Denmark. It is based on vacuum impregnation of concrete using a yellow fluorescent epoxy. During impregnation the capillary porosity, cracks, voids and defects in the specimen are filled with epoxy. After impregnation specimens are prepared for the analysis [14].

4.4.8 Non-destructive testing

There are many different NDT methods that can be used to evaluate the extent of damage. Some of them are useful for diagnosing problems, specifying repairs, and measuring the deterioration. The Schmidt Rebound Hammer is perhaps the cheapest and simplest to use. Ultrasonic pulse velocity and acoustic pulse echo devices measure the time required for a generated sound wave to either travel through a concrete or to pass through the concrete and return. Damaged concrete deflects such waves and can be detected by comparison with sound concrete. Acoustic emission devices detect the elastic waves that are generated when materials are stressed or strained beyond their elastic limits [23].

To get information about bond strength using such a nondestructive method like Impact-echo test, the stresses in the waves generated by the elastic impact of a steel sphere on concrete must be greater in magnitude than the tensile strength of the bond at an interface, if the waves are going to be used. The P-waves generated by an elastic impact are compression waves. They change phase and become tensile only when they are reflected from the free boundaries of the structure or from internal cracks or voids. Thus, the initial P-wave is a compression wave, but the P-wave reflected from the opposite boundary of the structure, such as the bottom of a plate, is a tension wave. It is this tension wave that has the potential to break the bond at an interface as it propagates through the structure. That is, it has the potential to produce stresses that are larger than the tensile bond strength at the interfaces that exist within the concrete structure (Figure 4.12) [20, 21].

With such devices, it is possible to detect the impulses from development of microcracks in stressed concrete. With computer assistance, several acoustic emission devices have been used to discover the areas of deteriorated or damaged concrete. By these methods it is possible to provide information needed in calculations of the area and volume of concrete to be repaired and for preparation of repair specifications [23].

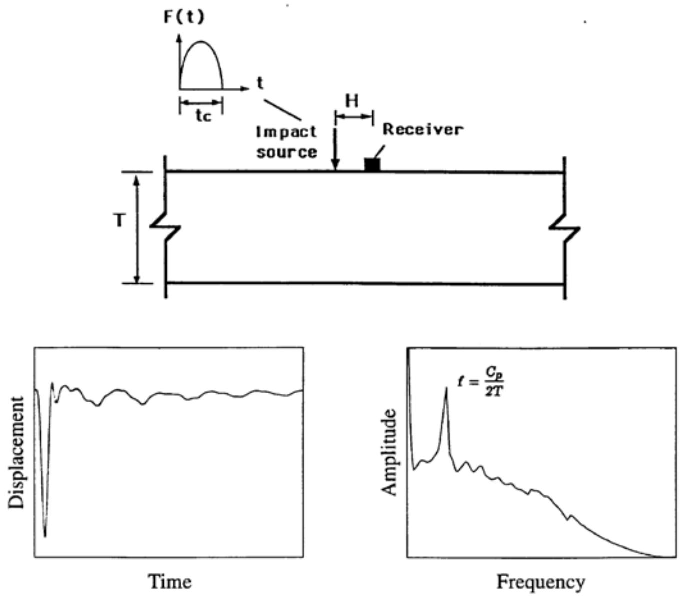


Figure 4.12 : Impact-echo Test, Displacement Waveform, Amplitude Spectrum [20]

5. EXPERIMENTAL WORK

The experimental program which is described in this part is undertaken to examine the bonding properties of the repair and the substrate. The first objective was to determine the properties of repair materials. Therefore some laboratory testing was performed. After that in-situ testings were performed on the trial structures. For the investigation, two different cement based repair mortars were chosen. One of them is a thixotropic rheoplastic repair mortar, ”B88”, the other one is a polymer modified, and fiber reinforced repair mortar, ”S612”. To obtain extra strong bonding, the substrate for B88 repairs was primed with a slurry coat of B88 and the substrate for S612 repairs was primed with a slurry coat of S610, one component cementitious, polymer modified bonding bridge. Alternatively both types of repairs were primed with two-component epoxy resins. The precise composition of these repair mortars and primer resins is proprietary and therefore unknown. However, these materials have high mechanical and durability properties.

As with most repair materials, specific instructions provided by the manufacturer were followed in preparation of a batch of the repair material for casting the test specimens. The laboratory investigation was performed to determine technically important properties of these mortars. Composite beam specimens, concrete-repair material, were prepared and subjected to four point bending test, to determine compatibility of repair types and effect of curing conditions.

A secondary objective of this work is to determine whether the workmanship affects the repair quality. Therefore, trial castings were prepared by different trained workmen at the same time with the same type of materials.

The substrate concrete specimens for bending test, produced at the laboratory were cast with high performance concrete, with maximum aggregate size of 10 mm. The mix proportion of the concrete is shown in Table 5.1.

Table 5.1 : Substrate Concrete Proportions for Laboratory Specimens, per m^3

Items	Quantity
w/cm	0.38
Water	135
Portland Cement, GU	350
Fly Ash, C Type*	60
Coarse Aggregate No.1	830
Crushed Sand	460
Natural Sand	530
HRWR	1.4% by mass of cm
Air Entrainer	1.5‰ by mass of cm
* k= 0.3	

Trial castings on the site were prepared on precast concrete panels of a TBM Tunnel Construction. The mix proportions of the trial panels are shown in Table 5.2.

Table 5.2 : Substrate Concrete Proportions for Trial Panels, per m^3

Items	Quantity
w/c	0.315
Water	140
Portland Cement, GU	450
Coarse Aggregate No.2	385
Coarse Aggregate No.1	475
Crushed Sand	475
Natural Sand	555
HRWR	0.85% by mass of cement

5.1 Tests Performed in the Laboratory

During the laboratory trials, flow and unit weight of the fresh mortars were measured and specimens for compressive strength, shrinkage, permeability, and microstructural analysis were cast. Specific instructions provided by the manufacturer were followed in preparation of a batch of the repair material. According to the instructions the water to repair material ratios for B88 and S612 were chosen 0.15 and 0.12 to obtain similar workability.

5.1.1 Fresh properties

The flow of the repair mortars was determined according to ASTM C 1437 standard practice using flow table by dropping the table 25 times in 15 seconds. Flow was measured immediately after mixing, within 5 minutes from the time of addition of water into the mix. The flow diameters of both repair mortars were obtained between 150-200 mm.

The unit weight was calculated by weighing the mortar with a calibrated container. The unit weights of B88 and S612 were 2100 kg/m^3 and 2230 kg/m^3.

Setting time of the repair mortars were measured using Vicat needle. The initial setting time was determined as the elapsed time required to achieve a penetration of 2.5 cm, the final setting as the total elapsed time when the needle does not sink visibly into the paste. The frequency of penetration of the needle was every half an hour from the repair material poured inside the container. The final setting time of the repair materials are between 4 to 5 hours.

5.1.2 Shrinkage

Specimens were prepared in accordance with ASTM C 531 standard practice, except the dimensions of the prisms were modified as 40x40x160 mm. Immediately after final setting, specimens were removed from the molds, were sealed with polyethylene sheeting to prevent rapid evaporation and the initial lengths were measured. Measurements were taken daily for two weeks at 21±2°C. Then the sealing was removed and specimens were set at 100°C oven for three days. After cooling in the desicattor for one day final measurements were taken [29].

5.1.3 Compressive strength

The compressive strength of the repair materials was determined with different specimen types. The firs one was according to the TS EN 196-1, using 4x4 cm cubes cutted from 4x4x16cm mortar prisms. Additionally, 20x20 cm cubes were cast in the laboratory. Φ100 mm cores were taken after 28 days moisture curing and tested according to the TS EN 12540-1. And also 150 mm x 300 mm cylinders were cast and tested as per TS EN 12390-3. The compressive strength of substrate concrete was determined using 150x300 mm cylinder as per TS EN 12390-3. Cylindrical specimens were capped with sulphur mixture. The cubes of the repair materials were tested in compression at 7 days and 28 days. The cylinders of the substrate concrete were tested at 7 days and 28 days [31-33].

5.1.4 Permeability

The capillary water permeability was determined by measuring the increase in the mass of a 7x7x7 cm cube specimen resulting from absorption of water as a function of time when only one surface of the specimen is exposed to water. The exposed surface of the specimen is immersed in water and water ingress of unsaturated mortar dominated by capillary suction during initial contact with water [25]. Two specimens for each repair material were tested at 23 days curing in 21±2°C curing room and the rate of absorption of water is determined.

The chloride permeability of concrete was determined in accordance with ASTM C1202 using a 50 mm thick, 100 mm mortar disc cut from the 100 mm x 200 mm cylinder prepared in the laboratory. The disc specimens were fixed between two cells containing ionic solutions (Figure 5.1). One of the cells was filled with 0.3 M NaOH solution and the other with 3.0% NaCl solution whilst a 60V DC was applied between the two cells. The resistance of concrete to chloride ion penetration is represented by the total charge passed in coulombs during a test period of 6 h. The chloride ion permeability test was carried out on the core specimens after 28 days.

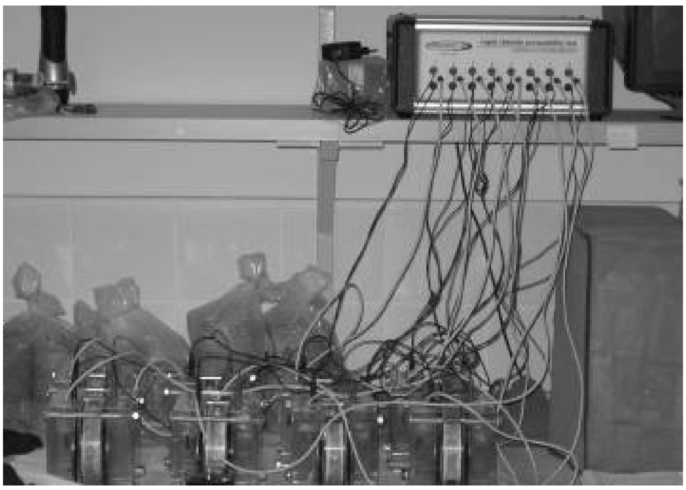


Figure 5.1 : ASTM C1202 Test Setup

5.1.5 Chloride diffusion

A water-saturated, 60 mm thick, 100 mm mortar disc cut from the 100 mm x 200 mm cylinder casted in the laboratory was, on one plane surface, exposed to water containing sodium chloride. After 35 days of exposure time, thin layers are ground off parallel to the exposed face of the specimen and the chloride content of the layers, C_x, is measured by potentiometric titration (Figure 5.2). The original (initial) chloride content of the mortar, C_i, is measured at a suitable depth below the exposed surface. The effective chloride transport coefficient, De, and the boundary condition of the chloride profile at the exposed surface, Cs, are calculated by non-linear regression. The penetration parameter, K_{Cr}, is calculated for a selected chloride concentration, C_r [30].

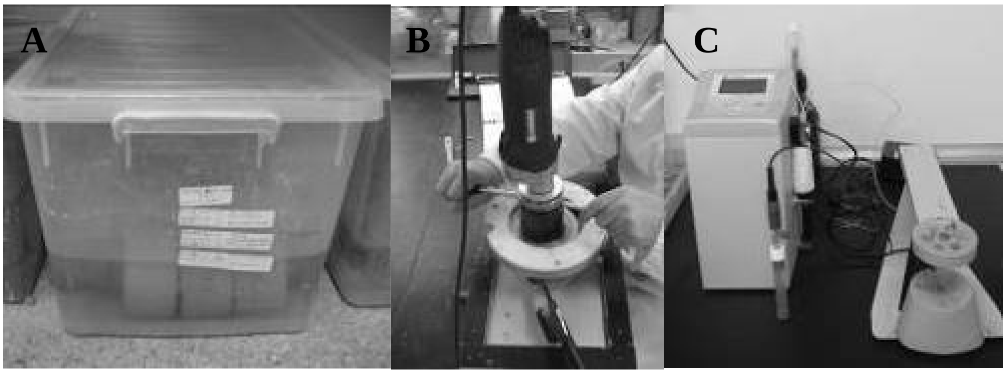


Figure 5.2 : Chloride Diffusion Test Procedure. A- NaCl Exposure, B- Powder Grinding, C- Potentiometric Titration

5.1.6 Composite beam test

This test was conducted for compatibility between repair and substrate concrete. The prism for evaluating the compatibility of repair material with substrate concrete was fabricated to the dimensions of 10x10x50 cm with a wide-mouthed notch 20x10x2 cm. After demoulding, the prisms were moist cured for minimum 28 days, and the notch areas were textured using a handheld breaker and primer was applied on the roughened surface. Two types of primers were used for each repair material, to investigate the effects of primer types on the failure characteristics. One of them was cementitious the other one was epoxy based. During the test, the repair material was placed on the bottom (tension side of the specimen) of the specimen as shown in the Figure 5.3.

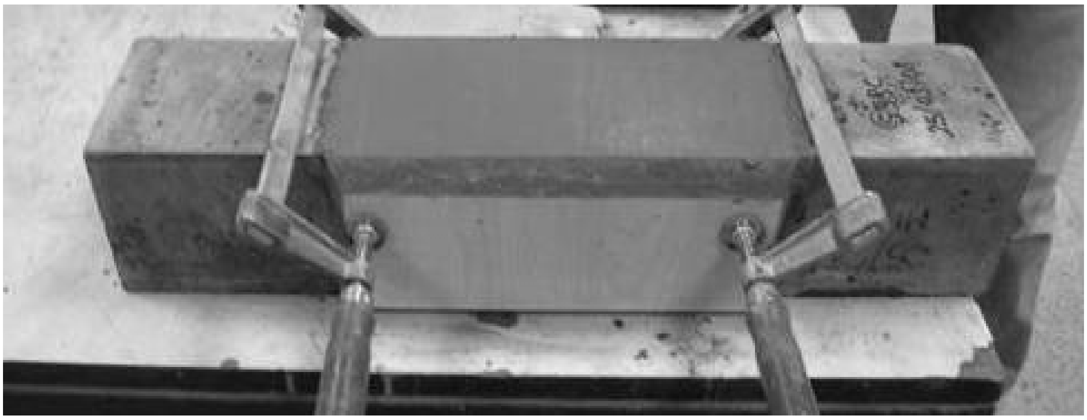


Figure 5.3 : Composite Beam Test Specimen

Also in this study, two different curing methods were used to investigate the effects of curing on the failure characteristics of the composite sections. After patching the notched area with the repair materials some of the composite sections were cured in air-dry curing and some of them were cured in moist curing for minimum 14 days and than subjected to a bending strength test, similar to ASTM C 78.

The repair materials were assessed compatible or incompatible with the substrate concrete by the mode of failures. In this study weaker repair materials are used to investigate the compatibility between repair materials and substrate. In addition load-deflection behaviors of the composite beams are evaluated. Since the repair materials are weaker in compressive strength and the load transfer at four point bending test is adequate, composite beam is forced to fail in the middle third portion of the beam or in the middle of the beam through the repair material due to maximum stress caused. If the failure mode is on the edge of the notch or if the repair material is popped out, instead of failing in the middle-third of the composite beam, then the repair material is not compatible with the substrate beam as shown in the Figure 5.4. [10].

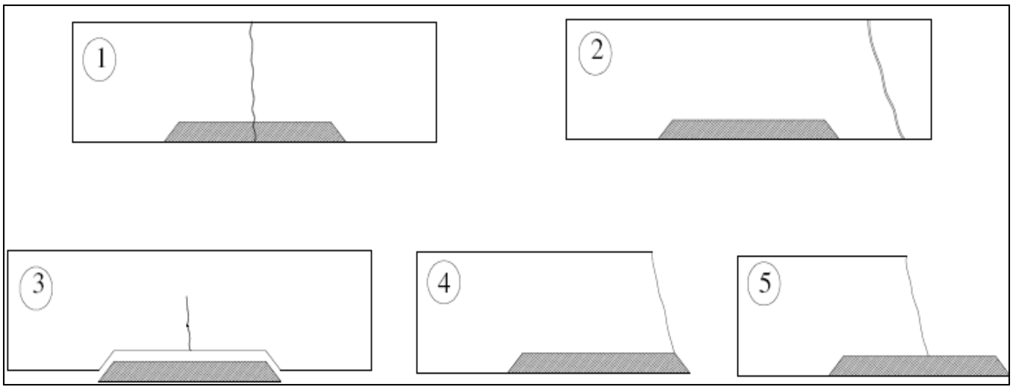


Figure 5.4 : Test Evaluation. 1,2-Compatibility; 3,4,5-Incompatibility [10]

5.2 Tests Performed on the Field

Tests were performed on the trial panel structures one of the TBM Tunnel construction in Istanbul. As shown in the Figure 5.5, 30 by 30 cm, nearly 3 cm deep patches were prepared with handheld breakers on the concrete structure. For the application with cementitious binder the existing surface was moistened and the binder was applied with brush. There is no need for moistening when epoxy binder is used. Specific instructions provided by the manufacturer were followed in preparation of a batch. All batches were mixed with a mixer on the field.

Figure 5.5 : Preparation of the Repair Surface and Application of the Primer

Each repair was applied with trowel by hand and immediately covered for curing as shown in the Figure 5.6. The cover was watered periodically for about one week.

Figure 5.6 : Application and Curing of the Repair

5.2.1 Rapid chloride

The chloride permeability of repaired specimens was determined in accordance with ASTM C1202 using Ø100 mm cores taken from repaired structure as mentioned in chapter 5.1.5 above.

5.2.2 Pull-off testing

The in-situ pull-off testing was performed according to NT BUILD 365. A cylindrical Φ75 mm semi core sample is prepared on the repaired structure and a roundel was glued on centrically above the repaired area. When the glue has hardened the roundel was pulled by the equipment centered above as shown in the Figure 5.7 and the bond strength was calculated.

Figure 5.7 : Pull-off Preparation

The location of rupture for each specimen was observed and the eccentricity was controlled by taking pictures (Figure 5.8).

Figure 5.8 : Specimen Control after Rupture

5.2.3 Microstructural analysis

For microstructural analysis, specimens were prepared from the 100 mm x 200 mm cylinders casted in the laboratory and Ø100 mm cores from repaired structures according to the Danish standards, DS 423.39 and DS 423.40 2002. As shown in the Figure 5.9 an optical stereomicroscope and a polarizing stereomicroscope were used to perform the analysis.

Figure 5.9 : Optical Microscope on the Left, Polarizing Microscope on the Right

In DS 423.39 the test methods and procedures for production of fluorescence impregnated plane sections, which can be used to analyze the internal stability of concrete in accordance with DS 423.41, was described. With a water-cooled diamond saw, a slice of 100 x 200 mm and at least 25 mm thick concrete sample is cutted and glued on a level ground iron plate. After the glue is hardened the specimen is impregnated with fluorescent epoxy under 95% vacuums for minimum one hour. When epoxy has stiffened, the sample is polished to a plane surface 0.25 mm, 1.0 mm and 2.0 mm under the impregnated surface, and photos of each polished samples are taken under UV-light.

In DS 423.40 the procedures for producing a fluorescence impregnated thin section for use in analysing the microstructure of concrete in accordance with DS 423.36 and DS 423.42~44, was described. To prepare a thin section, a concrete sample is cut into small pieces of 35x45x20 mm and is glued onto a piece of reference glass and impregnated with fluorescent epoxy. After hardening the excess epoxy is ground away and the re-ground surface is glued onto an object glass. Finally the thin section is ground and polished to a thickness of 30 μm and covered with a thin glass. The final thin section is ready to analyse under the polarizing stereomicroscope. Photos of each sample are taken under plain polar, crossed polars and UV-light.

UV photography reveals three main elements. The granular aggregates normally show up black, correlating to 0% porosity. Porous material shows up with varying fluorescence intensity, because the porosity varies. Air filled pores normally appear as green circles, correlating with 100% porosity. Air entrainment and large crack formations also stand out in yellow. Additionally, the bond region between concrete substrate and the repair material was investigated [34-40].

5.2.4 Impact-Echo testing

The impact-echo method was used to determine interfacial bond quality, specifically unbonded fraction of area and bond tensile strength, in concrete precast panels (Figure 5.10). Specimens on trial panels on the construction yard were designed to study the effects on the impact-echo response caused by variations in bond strength. Therefore pull-off testing was performed after obtaining the impact-echo response from the portion of the repaired structure. The objectives were to determine how bond strength affects the impact-echo response and whether the impact-echo-method could be used to quantify the bond strength [20-21].

Figure 5.10 : Impact-Echo Testing on the Construction Site

6. TEST RESULTS AND DISCUSSION

6.1 Mechanical Properties of the Materials

In this chapter, test results from the mechanical properties such as compressive strength and shrinkage of the repair materials and the substarate concrete are given.

6.1.1 Compressive strength

Compressive strength test results of the repair mortars and the substrate concretes are shown in Table 6.1, Table 6.2, and Table 6.3. These values are the average of the compressive strengths of specimens as shown in the appendix from Table A2 to Table A5. All the compressive strengths found increasing from 1 day to 28 days in moist curing.

Table 6.1 : Average Compressive Strength Test Results for B88

Age [Days]	Type	Dimensions [mm]	Strength [MPa]
7	Cube	40x40x40	53.5
28	Cube	40x40x40	65.0
7	Core	Φ100	49.5
28	Core	Φ100	53.5

Table 6.2 : Average Compressive Strength Test Results for S612

Age [Days]	Type	Dimensions [mm]	Strength [MPa]
1	Cube	40x40x40	13.5
7	Cube	40x40x40	45.0
28	Cube	40x40x40	59.5
7	Cylinder	150x300	33.0
28	Cylinder	150x300	43.0

Table 6.3 : Average Compressive Strength Test Results for Substrate Concrete

Age [Days]	Compressive Strength [MPa]	
	Structure	Laboratory
1	35.5	13.5
7	65.5	52,5
28	74.0	72.5

Further investigations revealed that different size repair mortar specimens show different compressive strength values at same ages. In Figure 6.1 and Figure 6.2 shows the size effect on mortar specimens at 7 and 28 days.

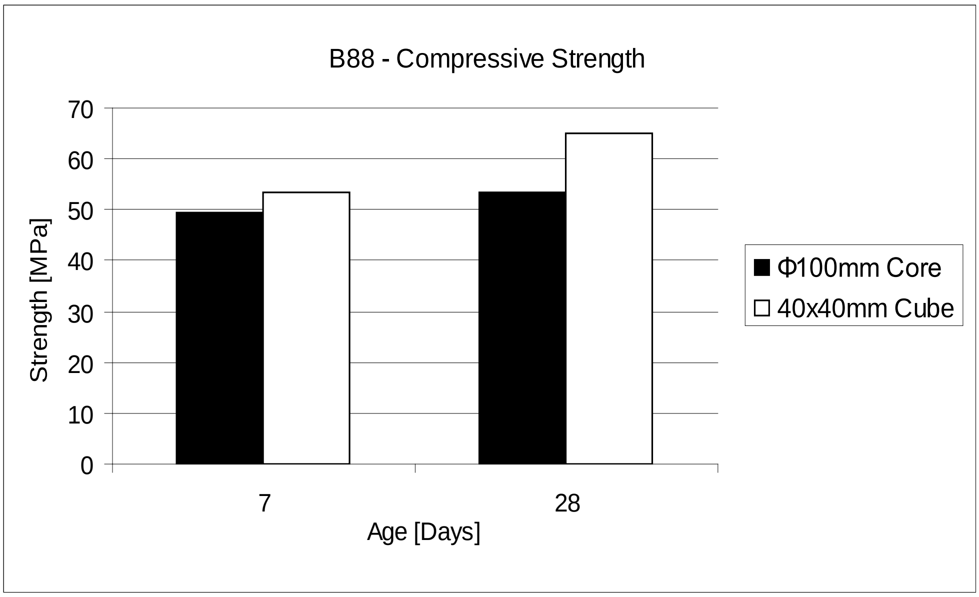


Figure 6.1 : Size Effect on B88 Repair Mortar

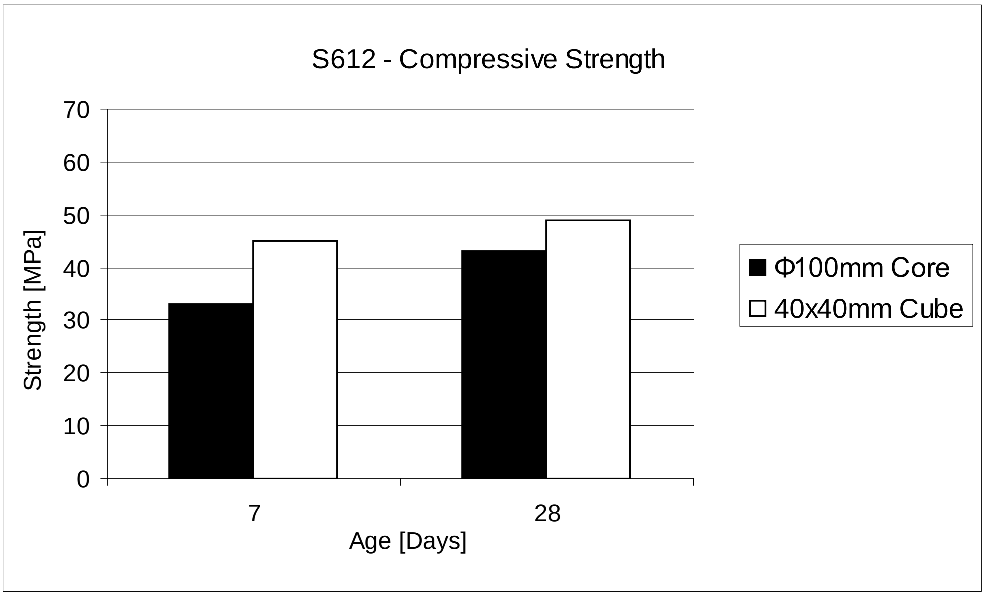


Figure 6.2 : Size Effect on S612 Repair Mortar

Figure 6.3 shows the difference in compressive strength between repair mortars and the substrate concrete at 7 and 28 days. It can be observed from the Figure 6.3 that the substrate concrete specimens have more than 35% compressive strength at 28 days.

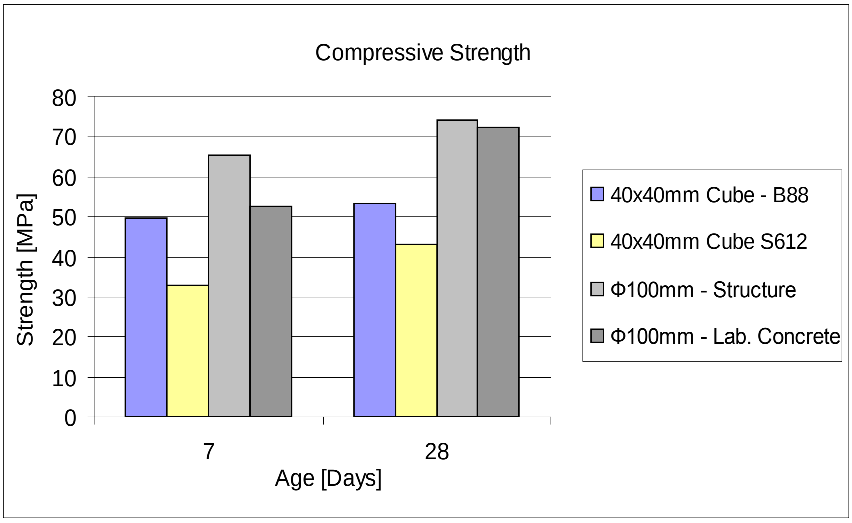


Figure 6.3 : Difference in Compressive Strength Results

6.1.2 Shrinkage

To obtain shrinkage paramaters such as autogeneous shrinkage, linear shrinkage and restraint shrinkage, various tests were performed on repair materials.

6.1.2.1 Autogeneous shrinkage

Autogeneous shrinkage test results of the repair mortars are shown in Figure 6.4 and Figure 6.4. There are three specimens for each mortar and the values of the each specimen are shown in the appendix Table A6 and Table A7. Measurements were taken daily until 14 days. According to the test results shrinkage values are over 0.05 %, which is corresponding to moderate shrinkage.

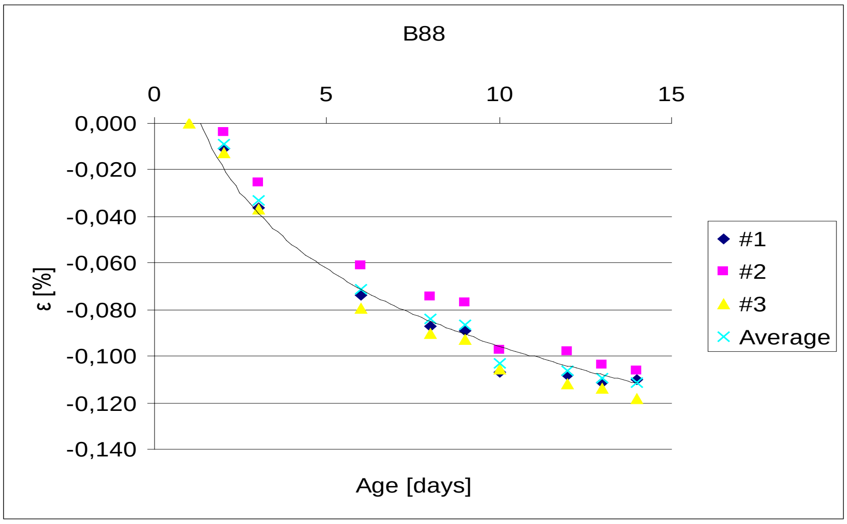


Figure 6.4 : Autogeneous Shrinkage Test Results of B88

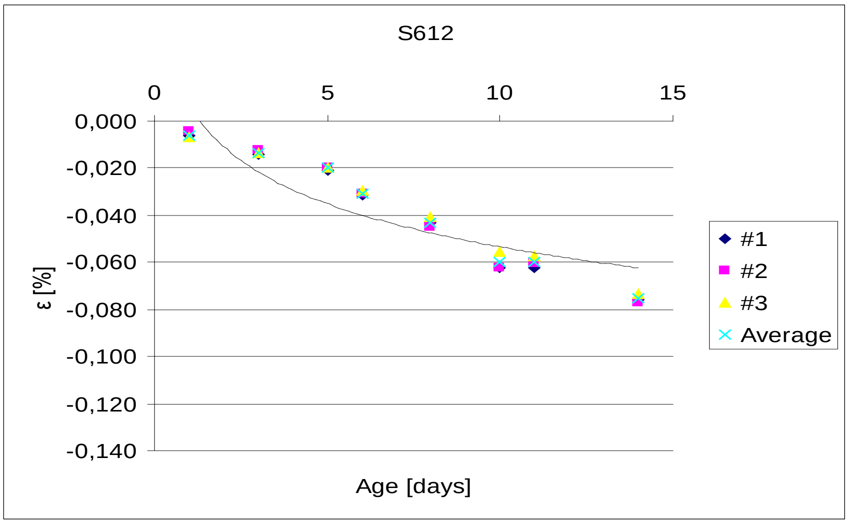


Figure 6.5 : Autogeneous Shrinkage Test Results of S612

6.1.2.2 Linear shrinkage

After two weeks specimens are placed for about 3 to 4 days at 100°C ovens and measurements are taken. The values of the each specimen are shown in the appendix Table A6 and Table A7 and calculated linear shrinkage test results are shown in Table 6.4.

Table 6.4 : Linear Shrinkage Test results for Repair Materials

Strain, ε [%]	1	2	3	Avg.
B88	-0,202	-0,197	-0,204	-0,201
S612	-0,164	-0,162	-0,160	-0,162

According to the Table 4.1 both repair materials have moderate shrinkage properties.

6.1.2.3 Restrained ring test

One ring specimen is cast for each repair material and rings are monitored daily for evidence of cracking. The day that cracking is observed is recorded and the initial crack width is measured with a crack comparator. Both specimens were cracked on the tenth day after casting and initial crack width was 0.15 mm. After three weeks there were two more cracks and the width of the initial crack was increased up to 0.50 mm. The type of cracking is shown in the Figure 6.6.

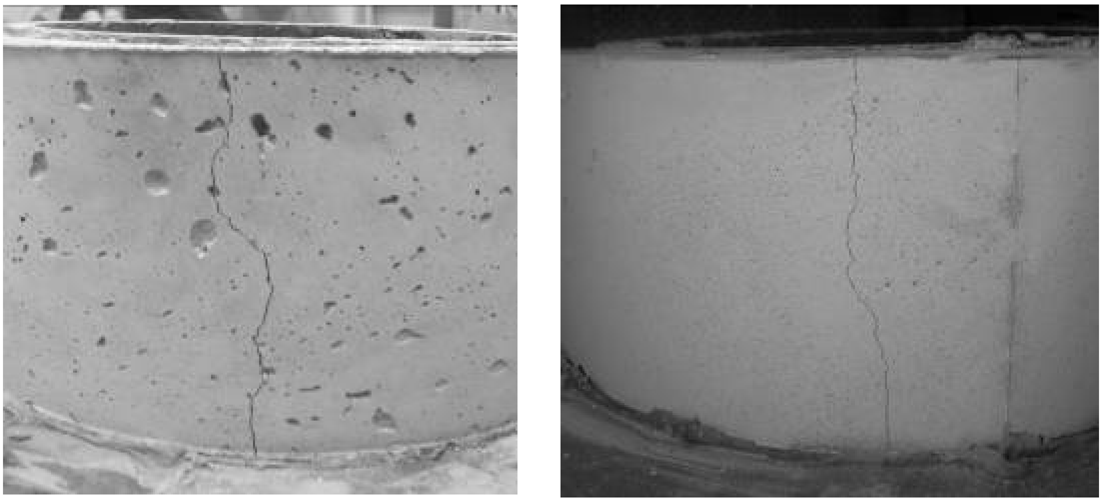


Figure 6.6 : Ring Test, B88 and S612

6.2 Durability Properties

To obtain durability properties of the repair materials various permeability tests were performed as shown in the following chapter.

6.2.1 Capillary water absorption

The capillary water absorption test results for repair materials are shown in the Figure 6.7. The initial rate of absorption and the secondary rate of absorption values are calculated as shown in the Table 6.5.

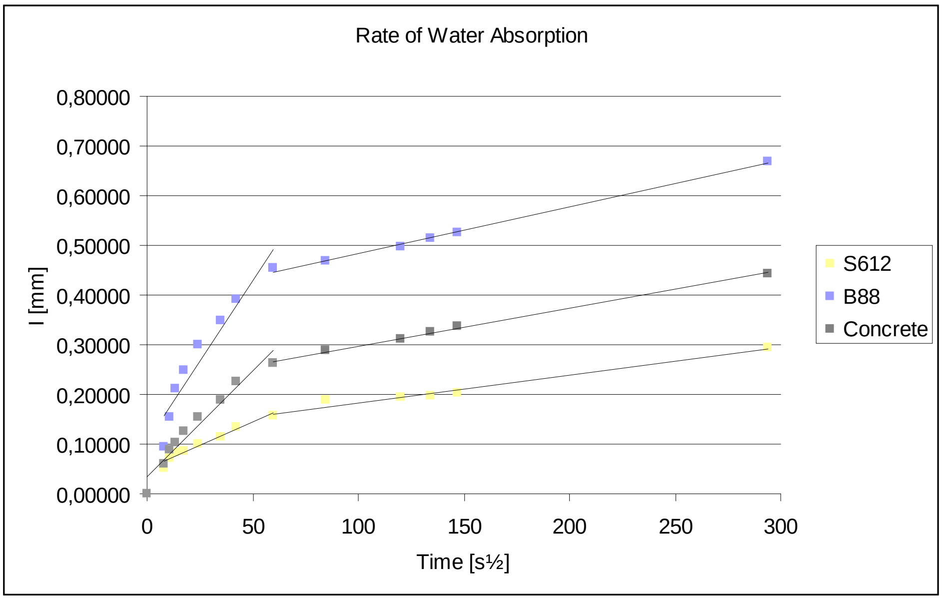


Figure 6.7 : The Average Capillary Water Absorption

Table 6.5 : The Average Rate of Water Absorption, ($x10^{-3}$)

B88	The initial rate of absorption, S_i [mm/s^{½}]	6.4 (R^2=0,91)
	The secondary rate of absorption, S_s [mm/s^{½}]	0.9 (R^2=0,99)
S612	The initial rate of absorption, S_i [mm/s^{½}]	1.9 (R^2=0,96)
	The secondary rate of absorption, S_s [mm/s^{½}]	0.6 (R_2=0,97)
Concrete	The initial rate of absorption, S_i [mm/s^{½}]	4.2 (R^2=0,95)
	The secondary rate of absorption, S_s [mm/s^{½}]	0.8 (R_2=0,99)

The data between 1 min and 1 hour is used for the regression analysis, which does not follow a linear relationship (a correlation coefficient of less than 0.98) and therefore the initial rate of absorption can not be determined. For a correlation coefficient less than 0.90 it can be observed that the rapier material B88 has a higher initial rate of absorption. The data between 1 hour and 24 hours is used to determine the secondary rate of absorption and it is correlation coefficient (0.97) shows a linear relationship. The values calculated for the secondary rate of water absorption of approximately $0.1x10^{-2}$ mm/s^{½} can be judged as rather low (Approximately 80 years penetration of a covercrete of 50 mm).

6.2.2 Rapid chloride permeability

The average rapid chloride test results for specimens are shown in the Table 6.6 and Table 6.7. These values are the average of specimens as shown in the Appendix from Table A8 to Table A9.

It is well known that very low permeability is desirable for a repair material. Actually repair materials are proprietary, material ingredients are unknown. Therefore, rapid chloride permeability may not be appropriate to measure the permeability of repair materials. However, this gives a relative measure of permeability of chloride ions, which may cause corrosion.

This test is normally performed after 28 maturity days. The result for an early test is shown on Table 6.6. The result is high as well as the standard deviation.

Table 6.6 : Average Rapid Chloride Test Results for Repair Mortars

	Age [days]	Average Charge Passed [coulombs]	Standard Deviation	Class
B88	11	5876	776	HIGH
B88	36	2125	228	MODERATE
S612	42	272	46	VERY LOW

Table 6.7 : Average Rapid Chloride Test Results for Substrate Concrete

	Age [days]	Average Charge Passed [coulombs]	Standard Deviation	Class
Structure	45	1672	59	LOW
Laboratory	37	2074	140	MODERATE

The classification of permeability according to the ASTM C1202 is shown in the Figure 6.8. B88 is moderately permeable with the substrate concrete specimens, they are nearly low permeable. Nonetheless, S612 is low permeable, which means there are less connected capillary voids and cracks, thus its permeability can be neglected.

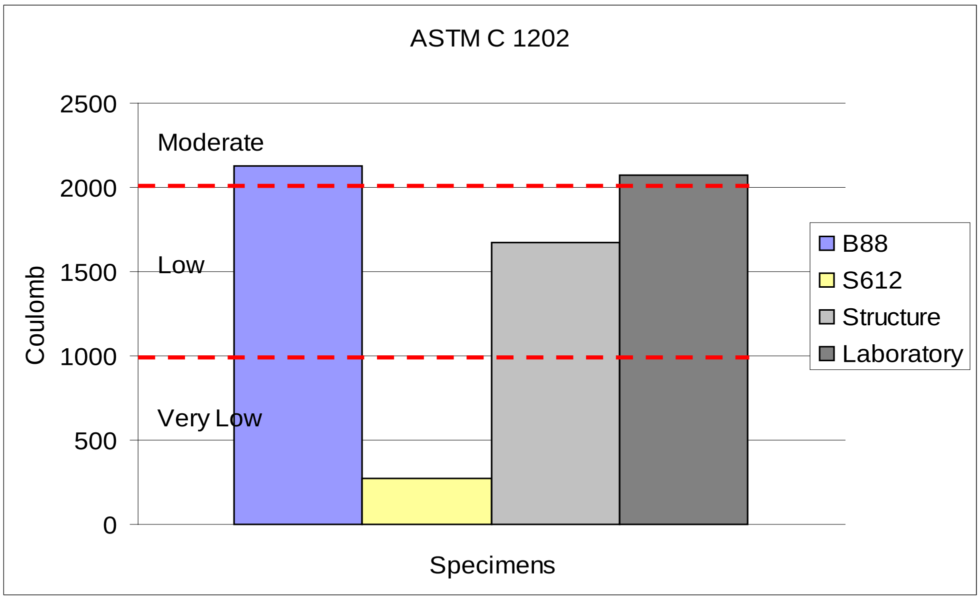


Figure 6.8 : Rapid Chloride Test Results

6.2.3 Chloride diffusion

The average chloride diffusion test results for specimens are shown in the Table 6.8. These values are the average of specimens as shown in the Appendix Table A10.

Table 6.8: Average Coefficients for Specimens

	$D_E[m^2/s]$	K_{Cr} [mm/√year]
B88	$1.44x10^{-12}$	17
S612	$5.68x10^{-13}$	11
Structure	$5.70x10^{-12}$	32

From the chloride diffusion test results obtained from the non-linear regression analysis on hardened specimens, the effective chloride transport coefficient, De, and the penetration parameter, K_{Cr} are proportional as shown in the Figure 6.9 and Figure 6.10. As a result the desired chloride permeability is achieved, thus the results of repair materials are less than the substrate concrete.

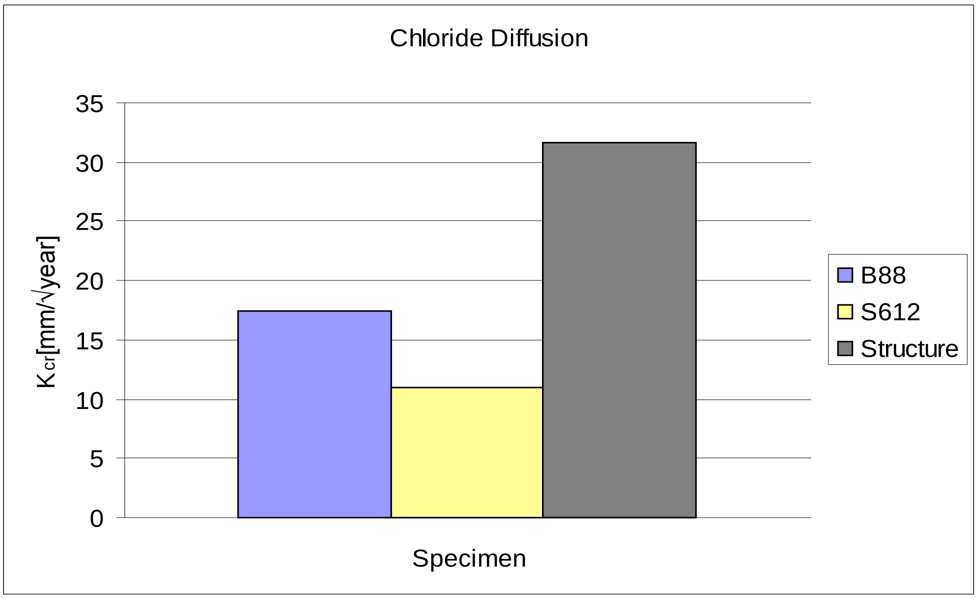


Figure 6.9 : Penetration Parameter, K_{Cr}

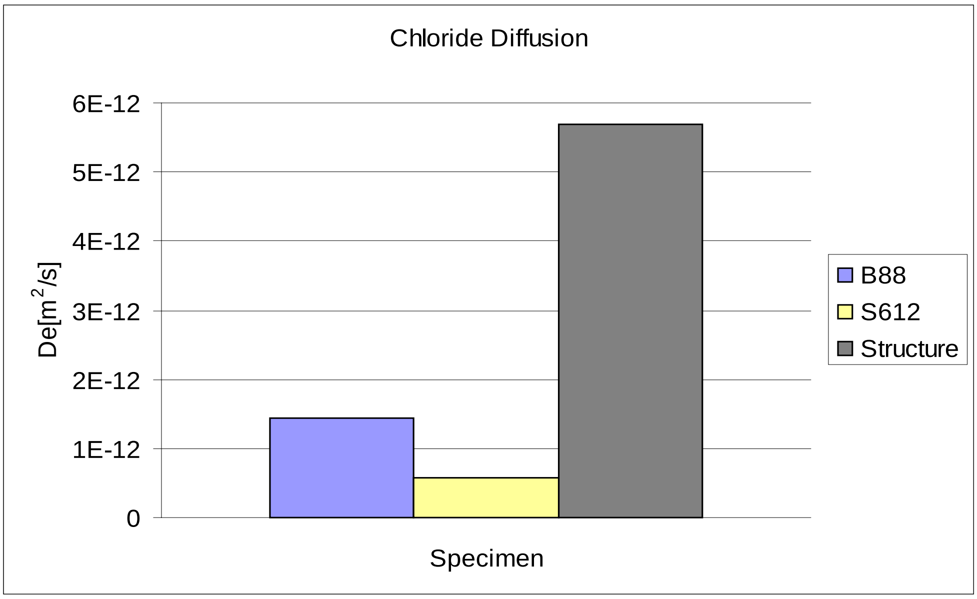


Figure 6.10 : Transport Coefficient, D_e

6.3 Summary of the Material Properties

The summary of the tests which are performed on the specimens produced at the laboratory are shown in Table 6.9. Both repair materials have similar mechanical properties but water and chloride permeability of B88 is higher than S612, which reveals higher porosity. Moreover, shrinkage results of B88 are higher than S612, which can increase the tendency of the specimens to crack. A connected crack system through the material can be the reason for the high permeability.

Table 6.9: Summary of Laboratory Results

Test \ Specimen	B88	S612	Concrete
Compressive Strength [Mpa]	65.0	59.5	74.0
Autogeneous Shrinkage [%]	-0.110	-0,075	-
Lineer Shrinkage [%]	-0,201	-0,162	-
Restrained Shrinkage [days]	10	10	-
1. Rate of Absorption	$6.4x10^{-3}$	$1.9x10^{-3}$	$4.2x10^{-3}$
2. Rate of Absorption	$0.9x10^{-3}$	$0.6x10^{-3}$	$0.8x10^{-3}$
Rapid Chloride [coulomb]	2125	272	1672
Penetration Parameter [mm/√year]	$1.44x10^{-12}$	$5.65x10^{-12}$	$5.70x10^{-12}$
Transport Coefficient [m^2/s]	17	11	32

6.4 Rapid Chloride on Repaired Specimens

The average rapid chloride test results for specimens cored from the repaired structure are shown in the Table 6.10. These values are the average of specimens as shown in the Appendix Table A11.

Table 6.10 : Average Rapid Chloride Test Results for Repaired Cores

Specimen	Age [days]	Repair Thickness [mm]	Primer Type	Average Charge Passed [coulombs]	Standard Deviation	Class
B88	61	-	B88	3053	176	MODERATE
	68	-	Epoxy	403	100	VERY LOW
	86	18	B88	2134	113	MODERATE
	85	14	Epoxy	850	118	VERY LOW
S612	72	16	S610	525	133	VERY LOW
	72	20	Epoxy	237	64	VERY LOW
	78	19	S610	378	67	VERY LOW
	78	23	Epoxy	223	20	VERY LOW

Obviously, using primer between repair material and substrate affects the results. As shown in the Figure 6.11 there are two trials for both repair material and each trial is performed once with cementitious primer, and once with epoxy primer. It can be observed from the results in Figure 6.11 that in all repair materials using epoxy primer reduces the chloride permeability. Permeability class of the cores repaired with B88 using cementitious primer is, moderate and the cores repaired with S612 using cementitious primer, very low. It can be observed that the chloride diffusion of the repairs B88 is much higher than the repairs S612. Even though the permeability class for all specimens repaired with epoxy primer is very low, it can be observed that results of the specimens with B88 are higher than the specimens with S612. The results for S612 are as low as negligible.

Consequently, rapid chloride permeability of the repairs is affected by repair material quality and mostly by primer type.

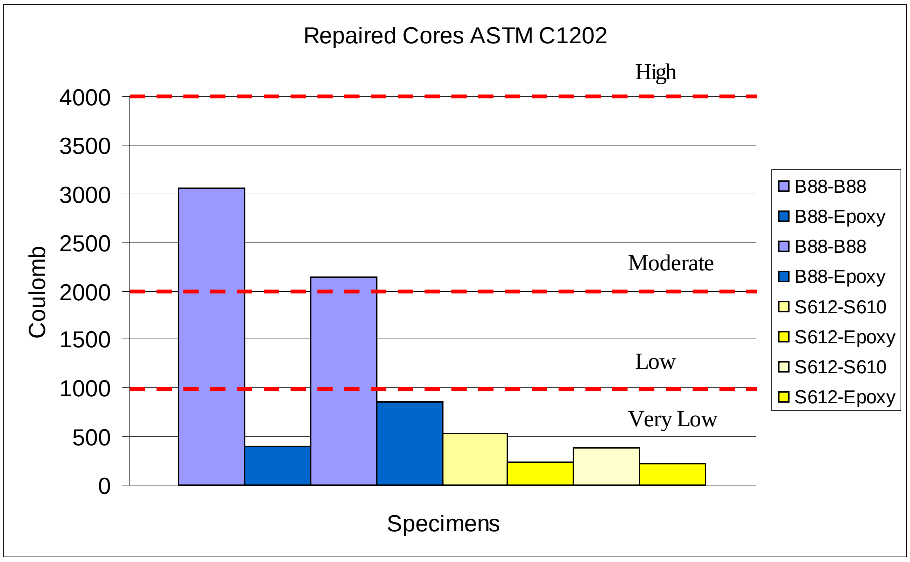


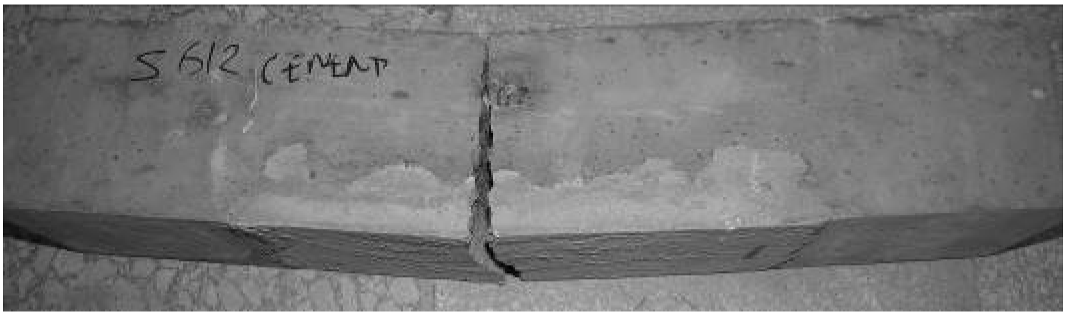
Figure 6.11 : Composite Cores Test Results

6.5 Compatibility

In this chapter, the author investigates the compatibility between repair materials and substrate concrete using a various test methods.

6.5.1 Composite beam test

Figure 6.12 shows the failure patterns for the composite beam specimens subjected to flexural loading and Table 6.11 shows the results of the failure types observed in the composite sections prepared with cement based primer and epoxy based primer, for moist cured and air cured specimens.



(a) Failure at the center

(b) Failure at the edge

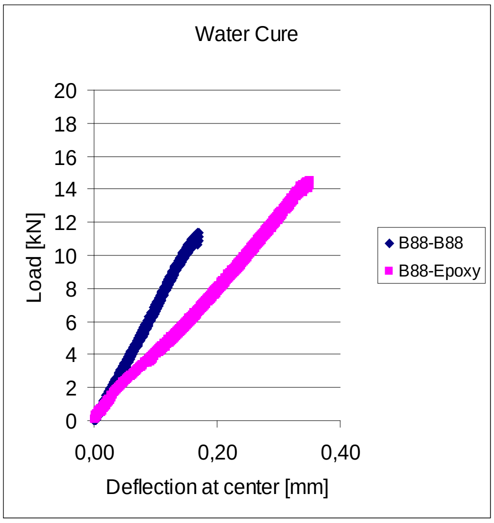
Figure 6.12 : Failure Patterns

It can be observed from the Table 6.11 that there is no significant difference in failure types between samples cured in water. The repair materials are deforming adequately, these materials can be compatible with the substrate concrete. But repair material B88 primed with cementitious primer shows in air-dry cured condition incompatible with the substrate concrete. This indicates that the curing influences the compatibility between repair and substrate concrete.

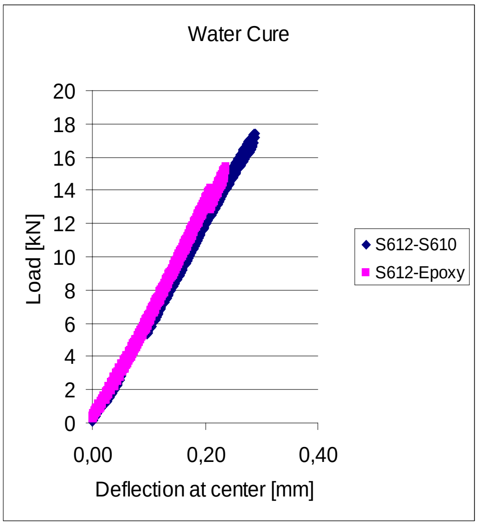
Table 6.11 : Failure Results

Specimen	Water Cure	Air-Dry Cure
B88-B88	CENTER	EDGE
B88-Epoxy	CENTER	CENTER
S612-S610	CENTER	CENTER
S612-S610	CENTER	CENTER

In addition, the corresponding load-deflection curves of these specimens in the flexure test are shown in the Figure 6.13 and Figure 6.14.

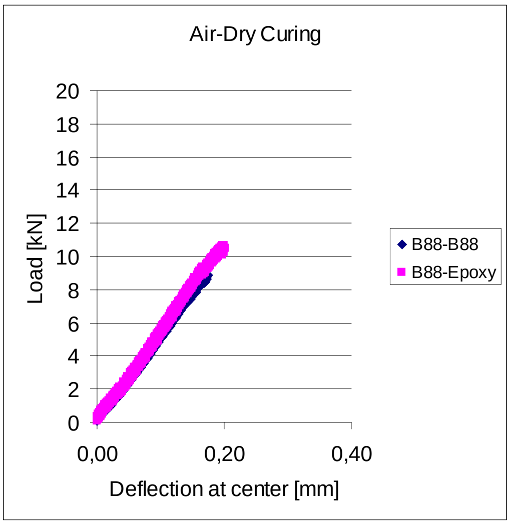


(a)

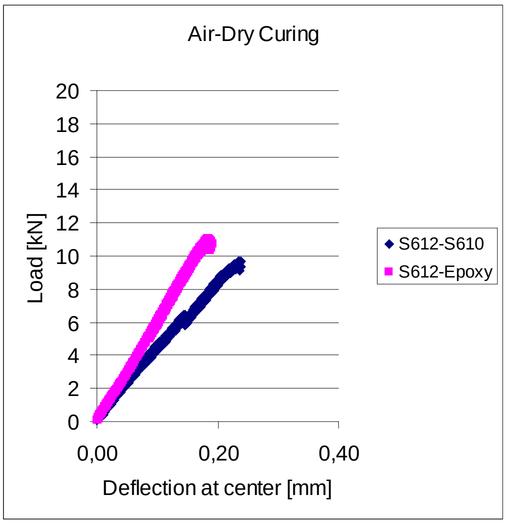


(b)

Figure 6.13 : Load Deflection of B88 Composite Beam in Water Curing



(a)



(b)

Figure 6.14 : Load Deflection of S612 Composite Beam in Water Curing

It can be observed that water curing improves the deflection ability and the load carrying capacity of the beam specimens. Specimens cured in air-dry condition have less flexural strength and less deflection at center.

6.5.2 Pull-Off test

The average pull-off test results for specimens from the repaired structure are shown in the Table 6.12. These values are the average of specimens as shown in the Appendix from Table A12 to Table A13. It can be observed that the bond strength is not related to the age of the repair mortar after a specific maturity.

Table 6.12 : Average Pull-Off Test Results

Specimen	Repair Age [days]	Primer Type	Average Bond Strength [MPa]
B88	26	B88	1.77
	34	B88	1.60
	34	E	2.33
	53	B88	1.99
	53	E	2.47
S612	29	E	2.51
	36	S610	1.74
	46	S610	2.41
	58	S610	2.15
	58	E	3.18

As shown in the Figure 6.15 the average bond strength results range between 2.0 to 2.5 MPa. The obtained average test results of S612 are higher than the test results of B88. Obviously, for both mortars, specimens prepared with epoxy primer provide better bonding strength.

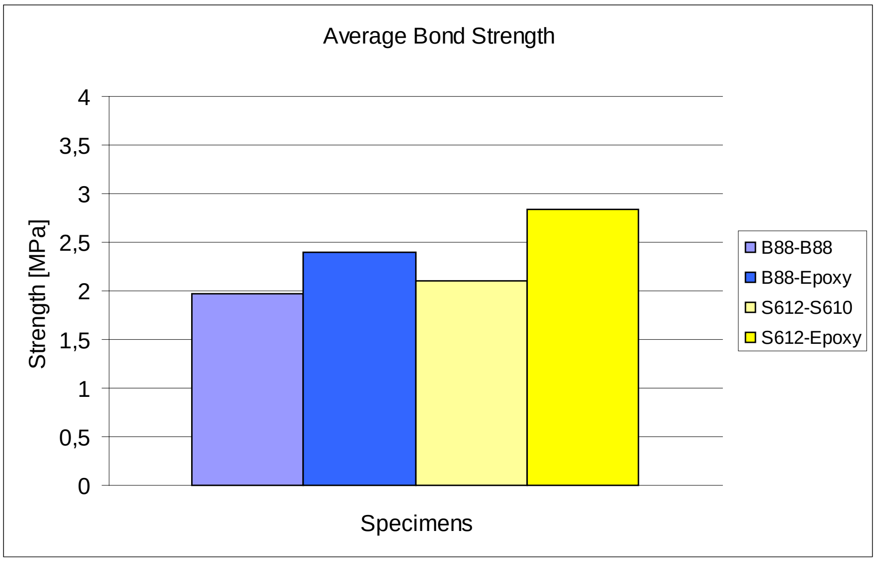


Figure 6.15 : Average Bond Strengths

In Table 6.9 and Table 6.10, there are pull-off test results from repaired sections, which are prepared simultaneously by different workers, to obtain the effect of workmanship on the repair quality. Two different repair mortars with cement based and epoxy primers were used. The environmental condition and the curing method was the same for all prepared repairs and the pull-off test was performed by the same technician on the same day. As shown in the Table 6.13 the average bond strength results are between 2.0 to 2.7 MPa and using epoxy primer increases the rupture loads. Interestingly there is no obvious relation between the location of rupture and the pull-off results.

Table 6.13 : B88 Pull-Off Test Results by Workmanship

Primer Type	Worker	Bond Strength [MPa]	Average [MPa]	Location of Rupture
B88	MC	2.18	2.0	Cons. Joint
		1.71		Cons. Joint
		2.18		N/A
	YG	2.18	2.0	Cons. Joint
		1.48		Cons. Joint
		2.29		Cons. Joint
		1.95		Cons. Joint
		1.95		Cons. Joint
Epoxy	MC	1.95	2.7	N/A
		3.06		N/A
		2.40		Substrate
		3.29		Cons. Joint
		3.29		Substrate
		1.95		N/A
	YG	1.95	2.3	N/A
		1.95		Repair Mat.
		2.40		Repair Mat.
		2.40		N/A
		2.18		N/A
		2.84		Cons. Joint

It can be observed from the Figure 6.16 that the pull-off strengths are not strongly affected by the workmanship if cementitious primer is used. But the application of the epoxy primer has resulted differences between two workers (Figure 6.17).

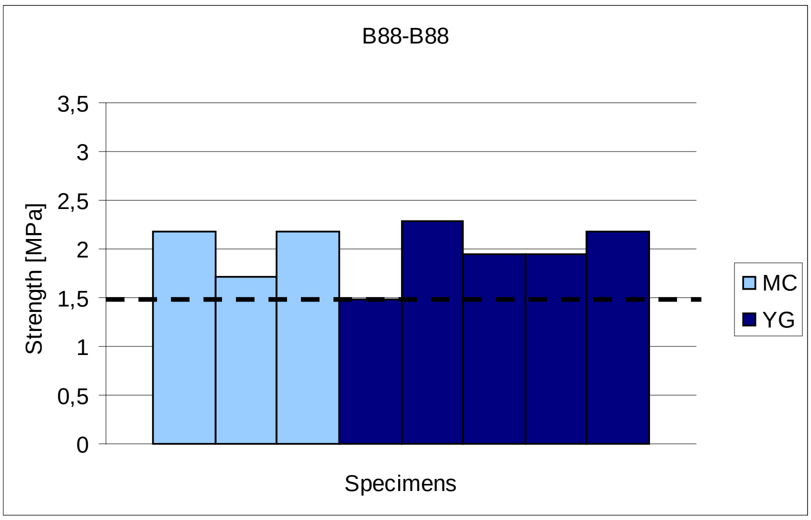


Figure 6.16 : Effect of Workmanship on B88 Repair with Cementitious Primer

Though both workers are well trained and experienced at repair works, MC has higher results when using epoxy primer.

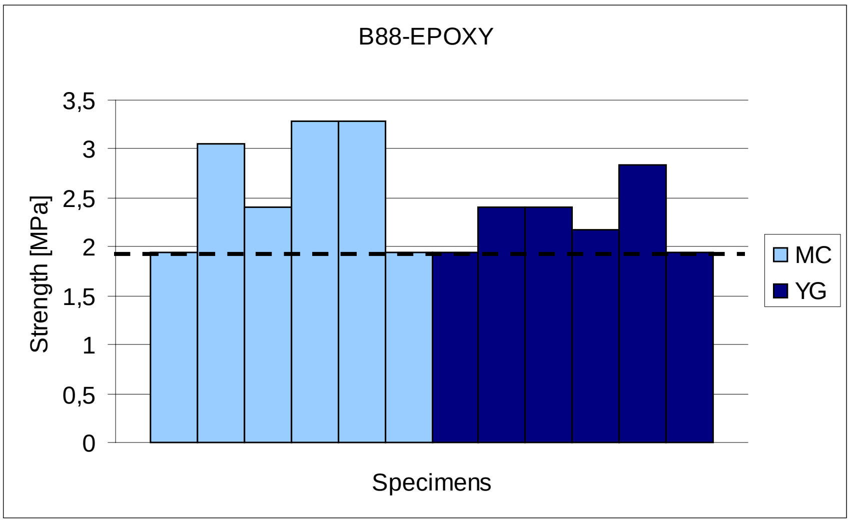


Figure 6.17 : Effect of Workmanship on B88 Repair with Epoxy Primer

On the Table 6.14, there are pull-off test results obtained from the S612 repairs prepared with both cementitious and epoxy primer. Obviously, specimens prepared with epoxy primer provide better bonding strength. It can be observed from the Figure 6.18 that the pull-off strengths are not strongly affected by the workmanship if cementitious primer is used. But application of the epoxy primer has resulted differences between two workers again as shown in the Figure 6.19.

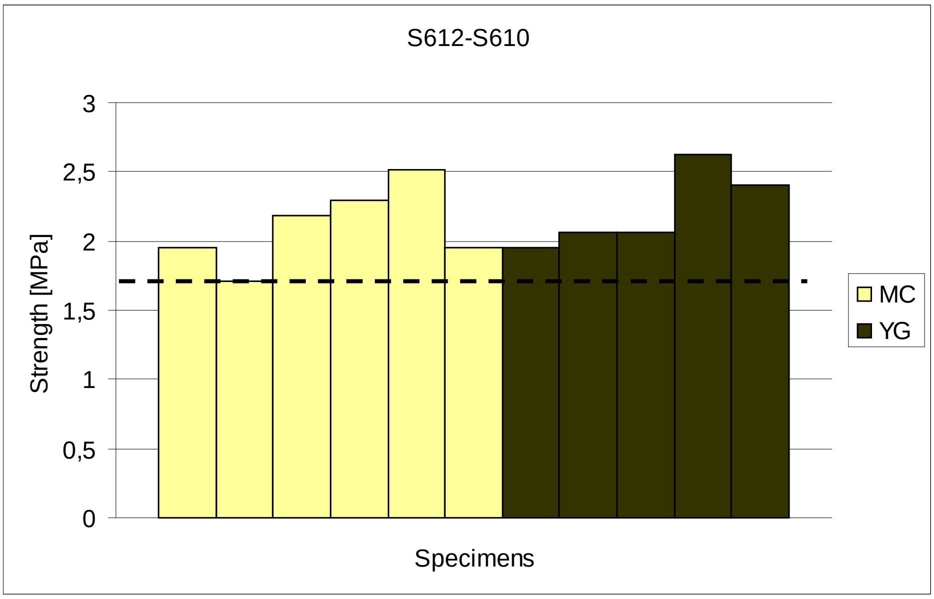


Figure 6.18 : Effect of Workmanship on S612 Repair with Cementitious Primer

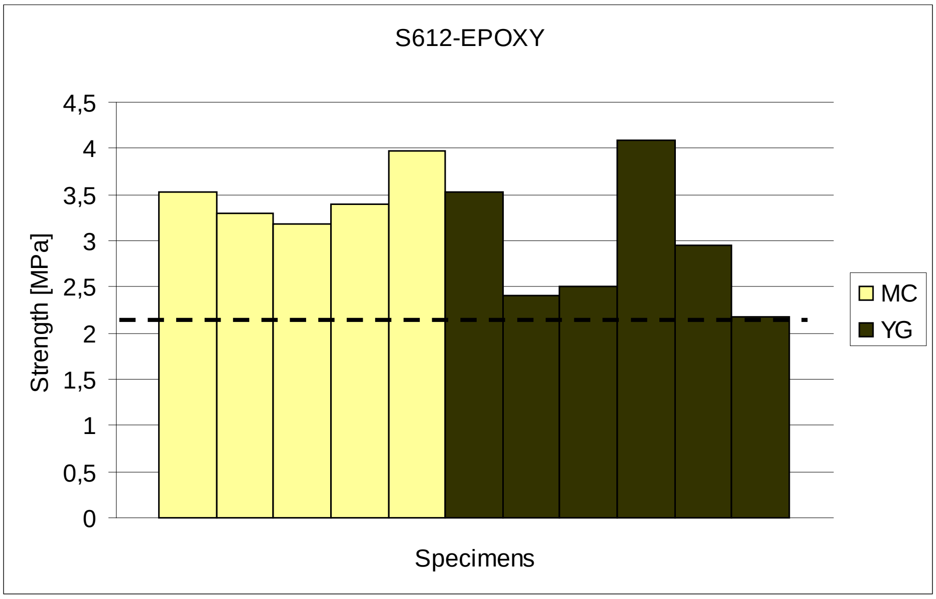


Figure 6.19 : Effect of Workmanship on S612 Repair with Epoxy Primer

Table 6.14 : S612 Pull-off Test Results by Workmanship

Primer Type	Worker	Bond Strength [MPa]	Average [MPa]	Location of Rupture
S610	MC	1.95	2.1	Repair Mat.
		1.71		Repair Mat.
		2.18		Repair Mat.
		2.29		Substrate
		2.51		Cons. Joint
		1.95		Cons. Joint
	YG	2.40	2.2	Cons. Joint
		1.95		Cons. Joint
		2.06		Cons. Joint
		2.06		Cons. Joint
		2.62		Repair Mat.
Epoxy	MC	3.52	3.47	Repair Mat.
		3.29		Cons. Joint
		3.18		Cons. Joint
		3.40		Repair Mat.
		3.98		Substrate
	YG	3.52	2.94	Substrate
		2.40		Substrate
		2.51		Substrate
		4.09		Repair Mat.
		2.95		Cons. Joint
		2.18		Substrate

It can be observed that MC has a better result than the other worker again with an epoxy primer between concrete and substrate. The deviation of the results provided by YG is apparently high, which is the measure of the quality of the repair work. The relation between the location of rupture and the pull-off results is inconsistent and there is no obvious difference between three types of rupture. Consequently, pull-out results are influenced by repair material, bonding agent and workmanship at adequate curing conditions.

6.6 Summary of Composite Specimens

Test results obtained from composite specimens are summarized in **Table 6.15.**

Table 6.15 : Summary of Results

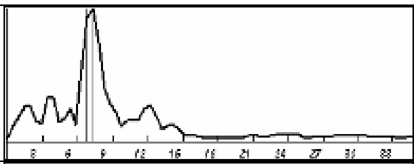
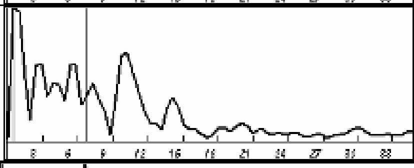
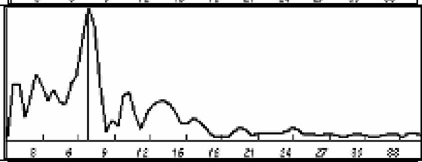
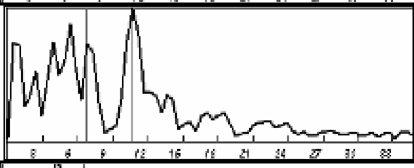
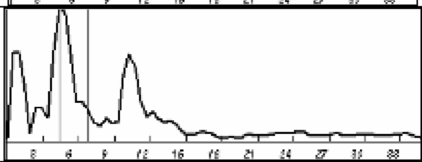
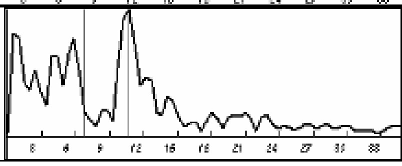
Repair Material	B88		S612	
Primer	Cement	Epoxy	Cement	Epoxy
RC	Moderate	Very Low	Very Low	Very Low
Pull-Off [MPa]	1.79	2.4	2.1	2.9
Beam	Center/Edge	Center	Center	Center

6.7 Impact-Echo Response

Table 6.16 shows the impact-echo response to determine the quality of bond at the interface. Therefore the impact-echo response and bond strength results are compared. The result obtained from the reference surface is a typical impact-echo result from sound concrete. The other results are obtained from repaired sections with different type of repair materials.

In all cases P-wave velocity in the concrete was 4000 m/s. Displacement waveforms contain 1024 points recorded at a sampling interval of 2 μs. The resulting digital amplitude spectra have a resolution of 0.488 kHz. Using the wave speed, the expected plate thickness frequency calculated as 7.33 kHz, which is corresponding to the thickness of the structure 300 mm.

Table 6.16 : Impact-Echo Response

Location	Amplitude-Frequency Relation	Bond Strength [MPa]	Location of Rupture
Reference		-	-
B88 with B88 Primer		1.85 MPa 1.63 MPa 2.44 MPa	Joint Joint Joint
B88 with Epoxy Primer		3.46 MPa 3.46 MPa	Substrate Substrate
B88 with Epoxy Primer		3.11 MPa 1.51 MPa 1.41 MPa	Repair Joint Joint
S612 with S610 Primer		2.88 MPa 3.00 MPa 2.54 MPa	Joint Joint Joint
S612 with S610 Primer		2.44 MPa 2.30 MPa	Joint Repair

The frequency-amplitude graph of B88 with epoxy primer in the Table 6.16 is the response obtained from the strongly bonded region, where the pull-off strength is >3.0 MPa. The frequency of the P-wave reflections through the full thickness, 6.84 kHz dominates the frequency response. It is the closest digital point to the expected frequency of 7.33 kHz. The other frequency-amplitude graphs include large-amplitude peaks at low frequency values and smaller amplitude peaks at higher frequency values. The large amplitude peaks are caused by flexural vibrations of the delaminated section, and the low amplitude peaks correspond to P-wave reflections between the delaminated interface and the surface, which means the interfacial bond in this transition region, is considered to be very weak or partially debonded. Especially on the second and fourth graphs, these peaks are distinctive. It can be observed that the location of rupture is near the construction joint and rupture loads are lower than usual.

7. MICROSTRUCTURAL ANALYSIS

7.1 Laboratory Specimens

The Figure 7.1 and Figure 7.2 show the images of the fluorescent impregnated and polished 100x200 mm plane sections and Figure 7.3 and Figure 7.4 show the images of the fluorescent impregnated 35x45 mm thin sections from the cylindrical specimens of the repair materials B88 and S612 batched in the laboratory.

As shown in the Figure 7.1 the first image is taken under the normal light and the other images are taken under the UV-light. Fluorescent epoxy appears green under UV-light. Normally the stability of the specimens is evaluated on a finished 1.0 and 2.0 mm under the impregnated layer. In this research 0.25 mm sections are also evaluated. As seen in the image 0.25 mm, there is higher capillary porosity and some connected porous zones in paste areas. The maximum size of entrapped air voids is 5 mm and there are many air-filled pores with a diameter 2~5mm. Fine cracks are available through the section. Under 1.0 mm and under 2.0 mm sections the capillary porosity, air voids and cracks disappear, because they are unconnected.

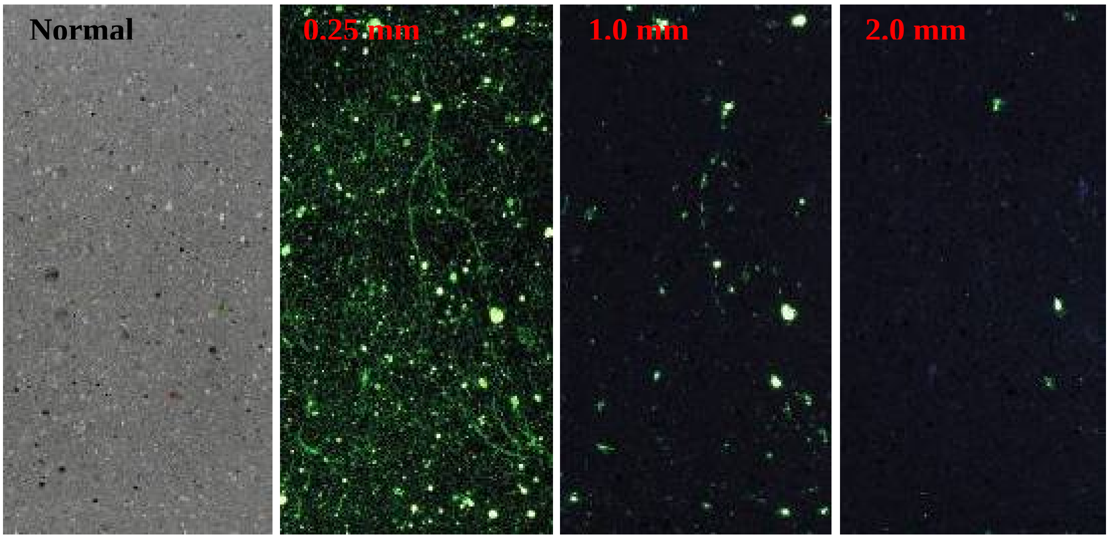


Figure 7.1 : Plane Section Pictures B88

In the Figure 7.2 the first image is taken under the normal light and the other images are taken under the UV-light. As seen in the image 0.25 mm capillary porosity is greatly lacking in uniformity and there are many connected porous zones in paste areas. There are some air-filled pores with a diameter 5~8mm. Under 1.0 mm it is possible to see connected porous zones but under 2.0 mm sections the capillary porosity disappears. Between these two samples there is no significant difference in capillary porosity and there are also no visible cracks and connected voids. Specimens prepared in the laboratory show capillary characteristics under 2 mm impregnated plane sections.

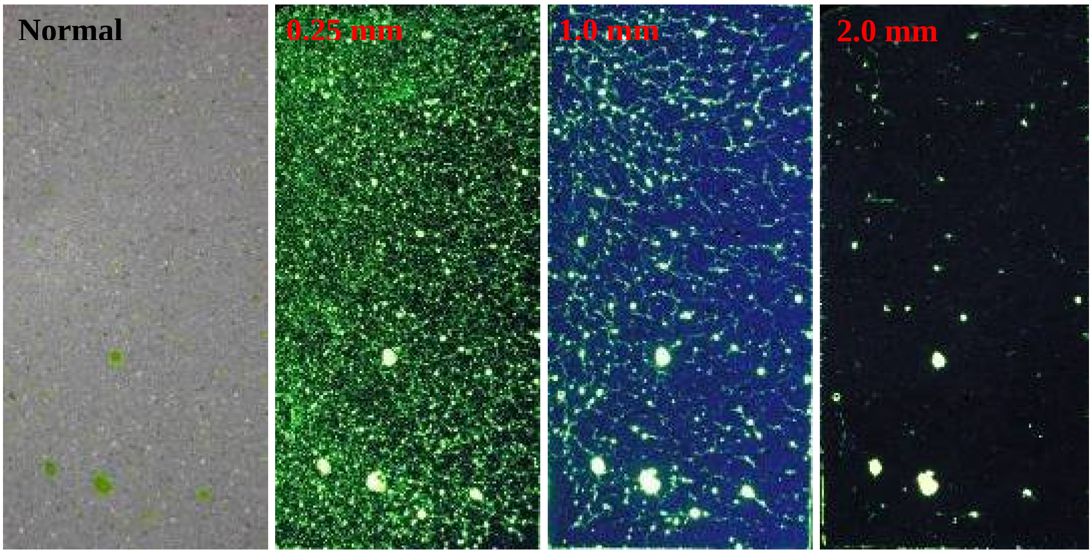


Figure 7.2 : Plane Section Pictures S612

Using a polarizing microscope with UV-Light and band stop filter, magnifying 50x, ten random fields across the thin sections are analyzed to document the quantity of microcracks in repair materials. Microcracks are categorized as paste cracks and bond cracks. The width of these cracks is <0.010 mm. Cracks with a width of 0.01-0.1 mm are defined as fine cracks and with a width of >0.1 mm are defined as large cracks. There are no fine cracks and macro cracks on both thin sections. The result of the microcrack analysis is shown in the Table 7.1.

Table 7.1 : Microcracks per mm^2

-	Micro Paste Cracks [number/mm^2]	Micro Bond Cracks [number/mm^2]
B88	0,60	0,32
S612	0.00	0.00

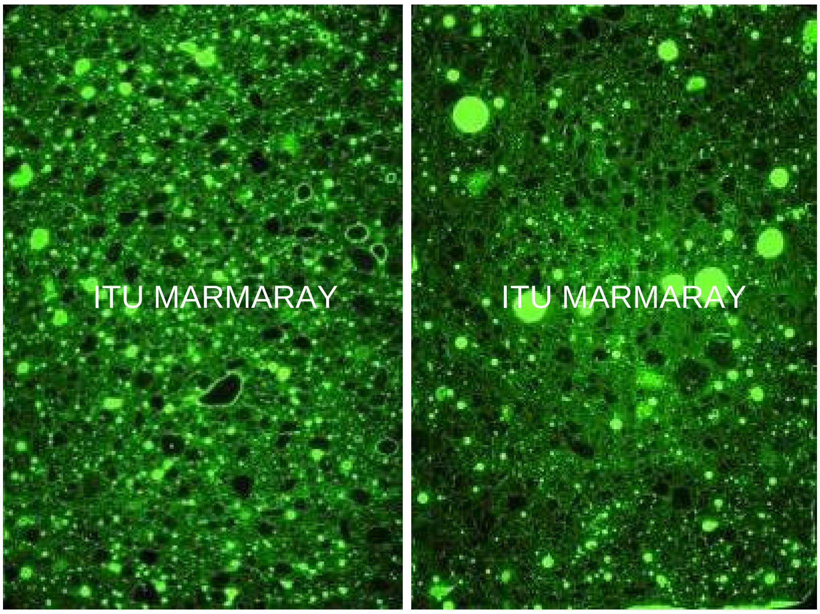


Figure 7.3 : B88 Thin Section (left), S612 Thin Section (right), 35x45 mm

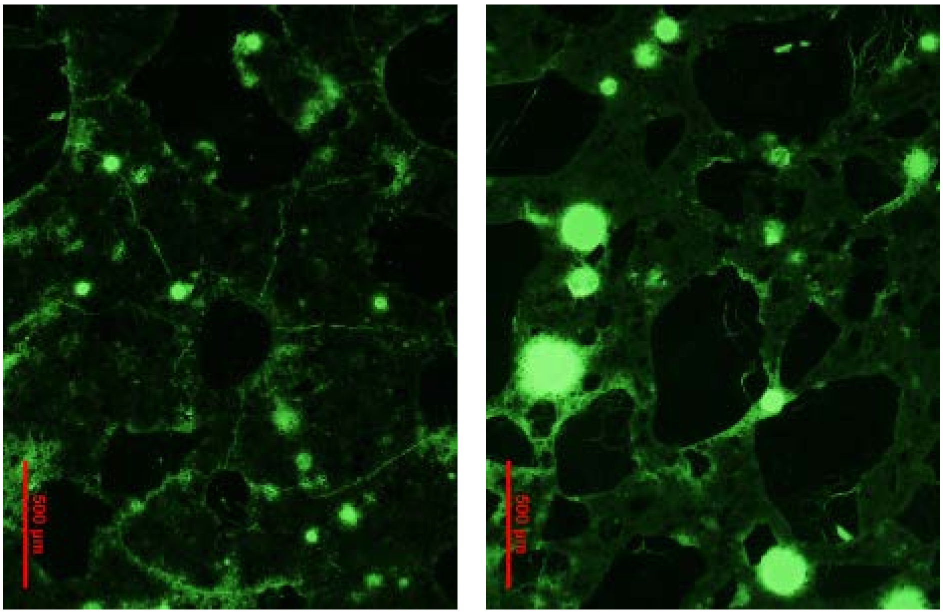


Figure 7.4 : Micro-cracking in the specimens

7.2 Composite Cores

Plane sections and thin sections from the horizontal casting joint were prepared from Φ100 mm cores drilled from different repairs in different structures.

The Figure 7.5 and Figure 7.6 shows plane sections with cementitious primer and Figure 7.7 and Figure 7.8 shows plane sections with epoxy primer. The repaired part is on the right side of the images. It is not possible to detect the cementitious primer from these images. But epoxy primer appears blues under UV-light. Observations are made generally about the interface between repair and substrate and about the repair material itself.

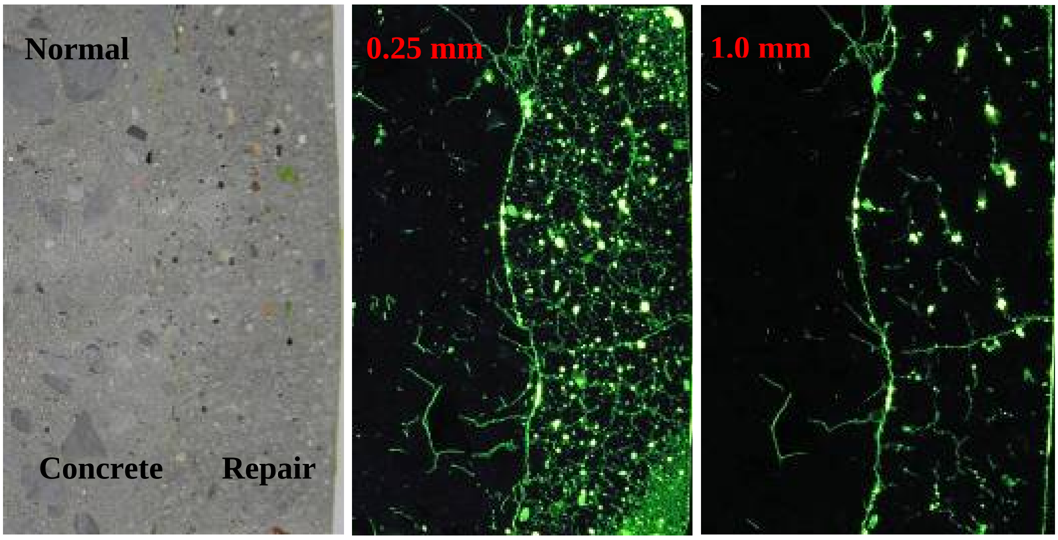


Figure 7.5 : Plane Section Pictures B88 with B88 Primer

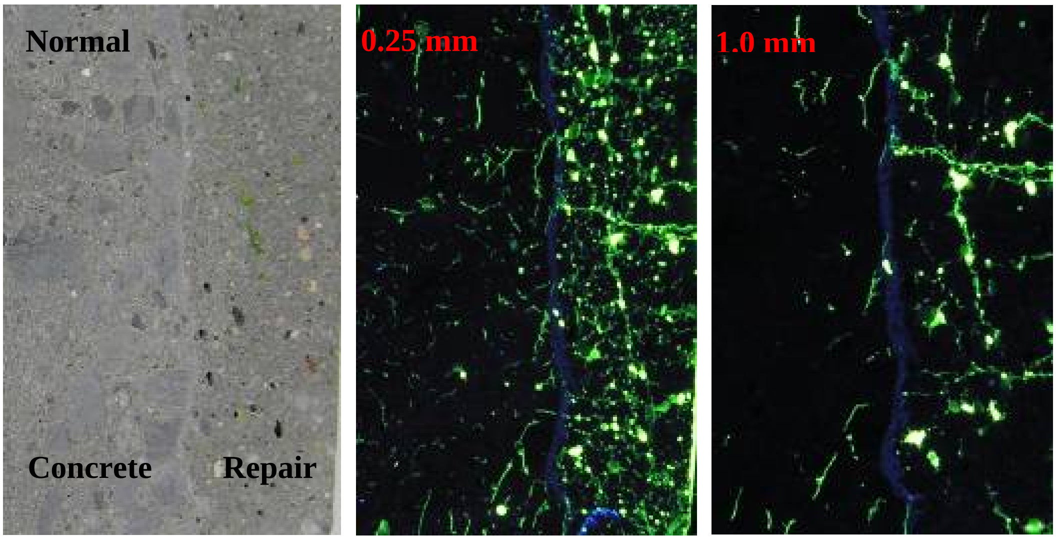


Figure 7.6 : Plane Section Pictures B88 with Epoxy Primer

There are fine cracks in the substrate concrete parallel to the surface, due to mechanical preparations. Many entrapped air voids were located in the repair mortar along the interface due to insufficient compaction of the repair material. There are microcracks vertical to the surface, running through the thickness of the repair material, which may be the result of drying shrinkage.

Repairs made with cementitious material have a continuous interface filled with fluorescent epoxy, which looks like a crack. The interface of the specimens prepared with epoxy primer has uniform homogeneity and there is no sign of fluorescent epoxy in-between.

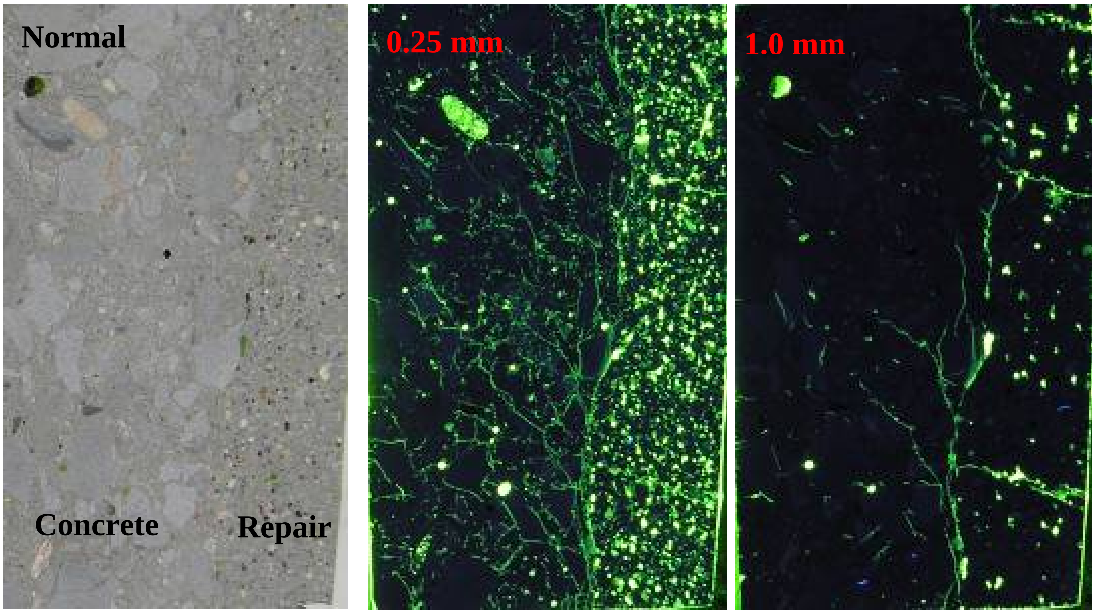


Figure 7.7 : Plane Section Pictures S612 with S610 Primer

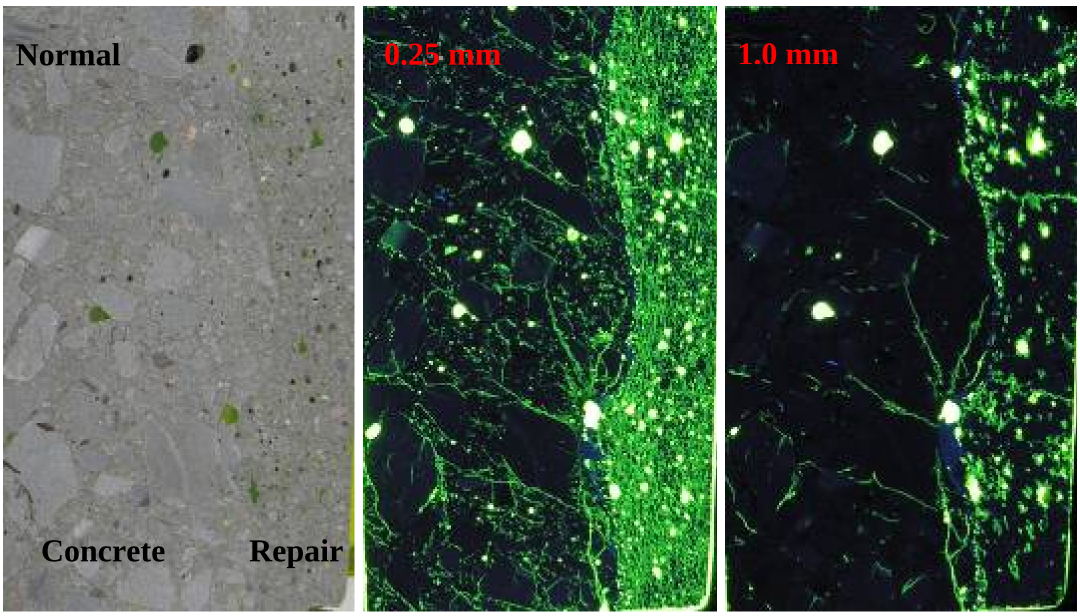


Figure 7.8 : Plane Section Pictures S612 with Epoxy Primer

From Figure 7.9 to Figure 7.12 displayed images belong to the thin sections prepared from repairs. The images were taken under UV-light. The repair part and the substrate concrete are shown in the images. It is not possible to differentiate the primer type from these images. But the interface is visible as a crack through the section.

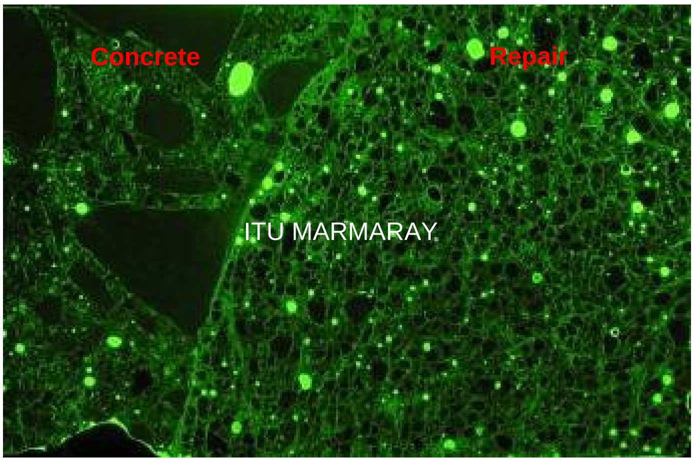


Figure 7.9 : B88 with B88 Primer Thin Section, 30x45 mm

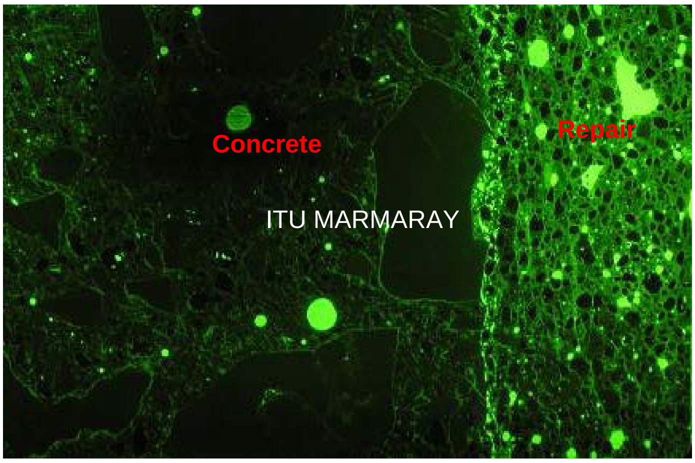


Figure 7.10 : B88 with Epoxy Primer Thin Section, 35x45 mm

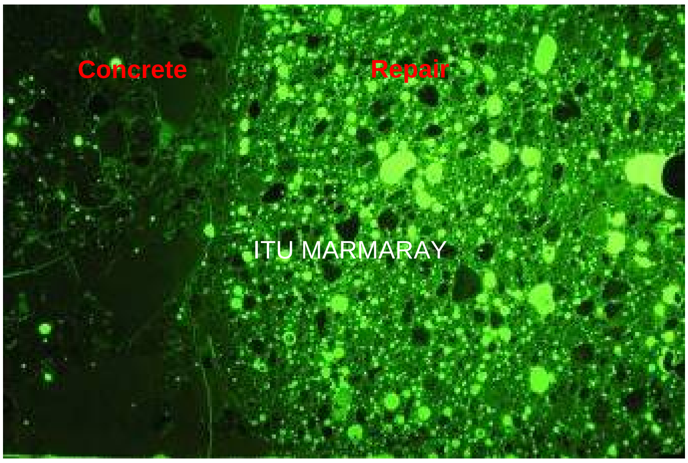


Figure 7.11 : S612 with S610 Primer Thin Section, 35x45 mm

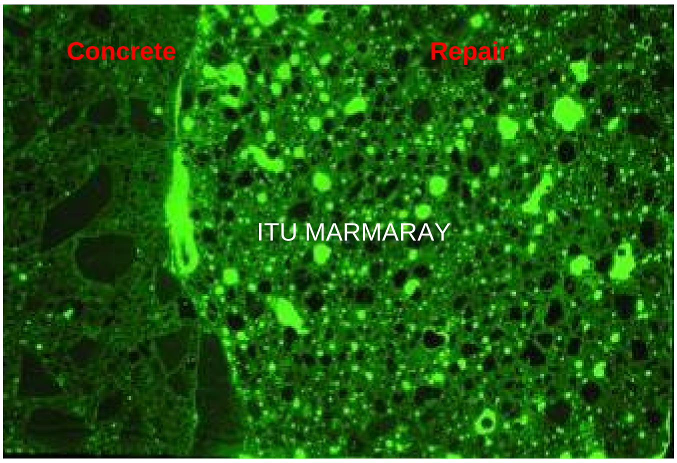


Figure 7.12 : S612 with Epoxy Primer Thin Section, 35x45 mm

The entrapped air voids in the repair mortar and along the interface mentioned above on the plane section images are more evident on the thin section images below. But there are no visible cracks in the repair mortar. Inhomogeneous entrapped air content in S612 is more than in B88.

From Figure 7.13 to Figure 7.16 displayed images from the thin sections are taken under optical polarized microscope with 50x magnification. The images on the left are taken under UV-light and the images on the right are taken under normal light. The repair part is always on top, the interface appears in the middle section of the images and the bottom section is the substrate concrete.

In Table 7.2 there are some observations of the repair materials under microscope. These values are the average of three thin sections from the same batch. The thickness of the repair was measured and under optical-stereo microscope at 63x magnification the microstructure of the interface and the repair mortar was analyzed. Vertical microcracking on the repair surface caused by shrinkage is the result of poor curing. With increased shrinkage cracks propagate and they are getting wider.

Table 7.2 : Observations from the Thin Sections

-		Thickness [mm]	Vertical Microcracks [#]	Fine Cracks [#]	Coarse Cracks [#]	Entrapped Voids	Unbonded Length [%]
S612	1	28	1,7	1,0	0,0	many	14,6
	2	23	0,7	0,0	0,7	many	5,8
	3	18	2,3	0	0	many	2,3
B88	1	25	0,0	0,0	0,0	few	0
	2	19,3	3,3	0,3	0,0	some	4,9
	3	18	1	0,3	0,3	many	6,8
	4	17	2,3	0,7	0	many	5,2

The interface between the repair mortar and the substrate concrete was investigated and the unbonded areas were determined. The unbonded areas were observed under polarized microscope with 100x magnification. Under UV-Light areas filled with fluorescent epoxy appear green and areas filled with primer appear dark.

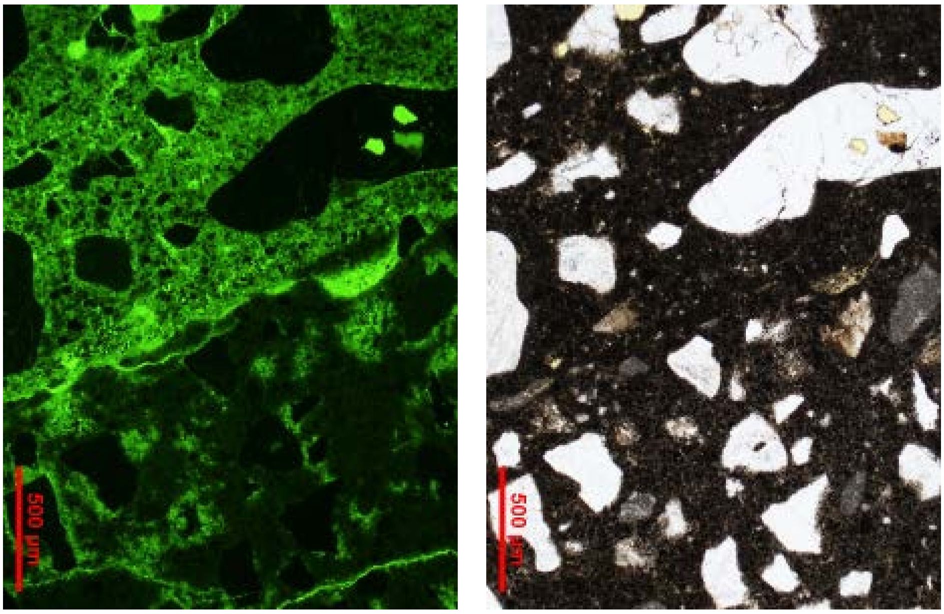


Figure 7.13 : Interface of Repair B88 with B88 Primer

If there is dust or dirt on the surface of the interface, it also appears green. Areas filled with dust and dirt because of poor cleaning is calculated as unbonded areas.

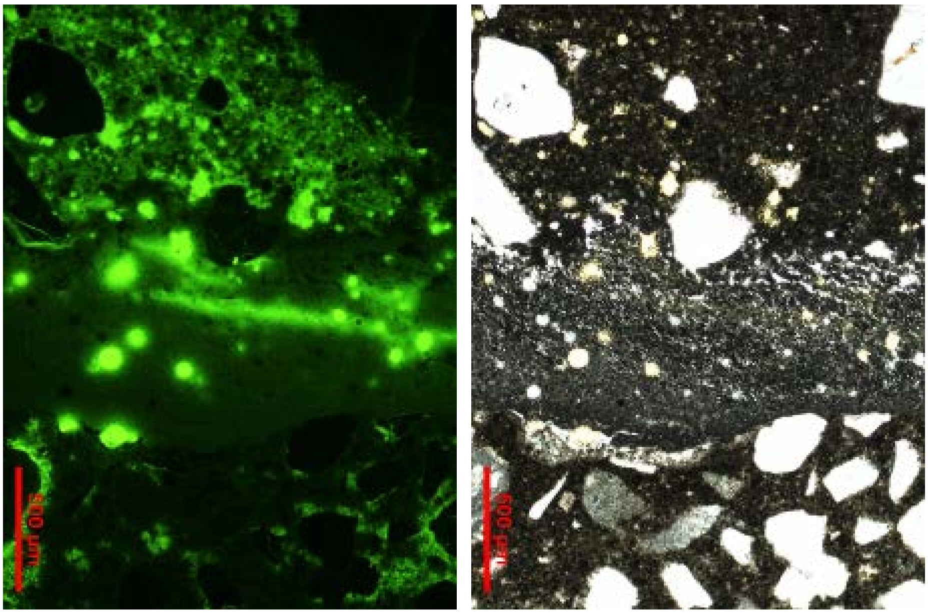


Figure 7.14 : Interface of Repair B88 with Epoxy Primer

If air bubbles were located in the interface, thus indicating insufficient application of the primer, the interface appears green again. Increased number of air voids is the result of poor compaction.

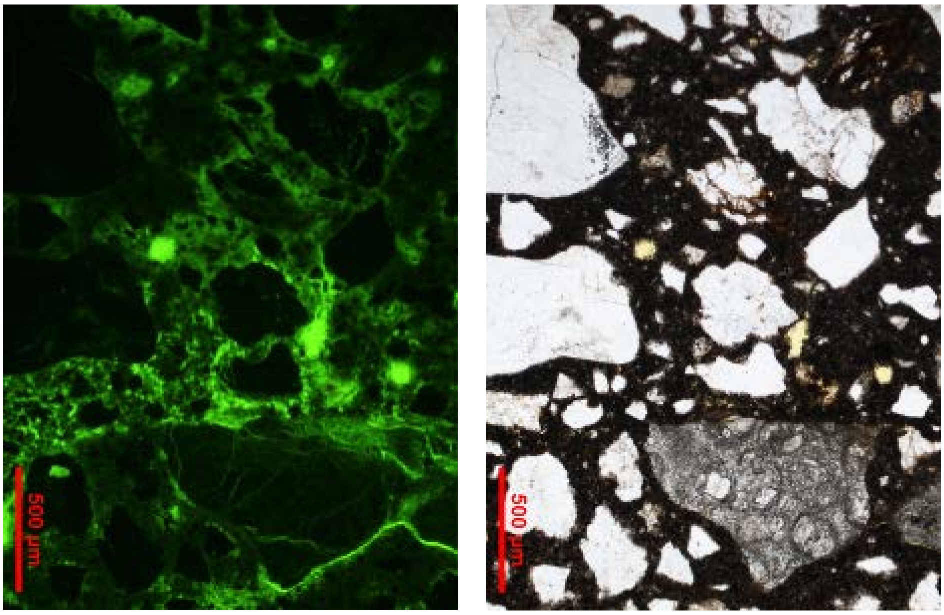


Figure 7.15 : Interface of Repair S612 with S610 Primer

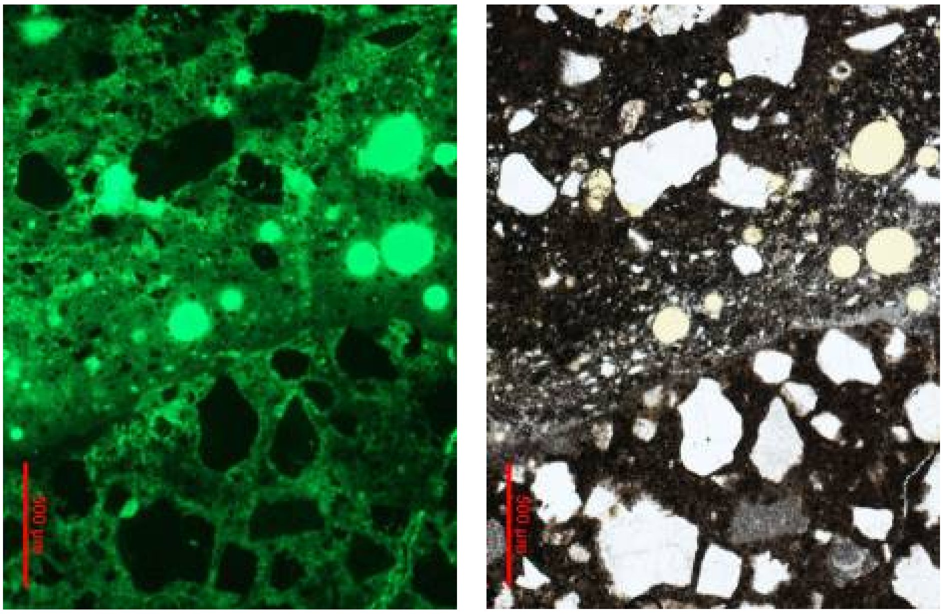


Figure 7.16 : Interface of Repair S612 with Epoxy Primer

7.3 Effect of Workmanship and Mixing

In Figure 7.17 there is a clump of silica-fume undispersed in the cement paste of the repair material designated in the dotted area. It can be identified at plain polarized light by its angular form, its brown color and most of the times there is a crack through the middle of the silica fume particle, filled with fluorescent epoxy. At the crossed polar it's opaque and appears dark as shown on the right image. Silica fume is a special product used in cementitious materials, but it is very hard to mix it. It is observed that all thin sections prepared from the repaired cores have nondispersed silica fume particles.

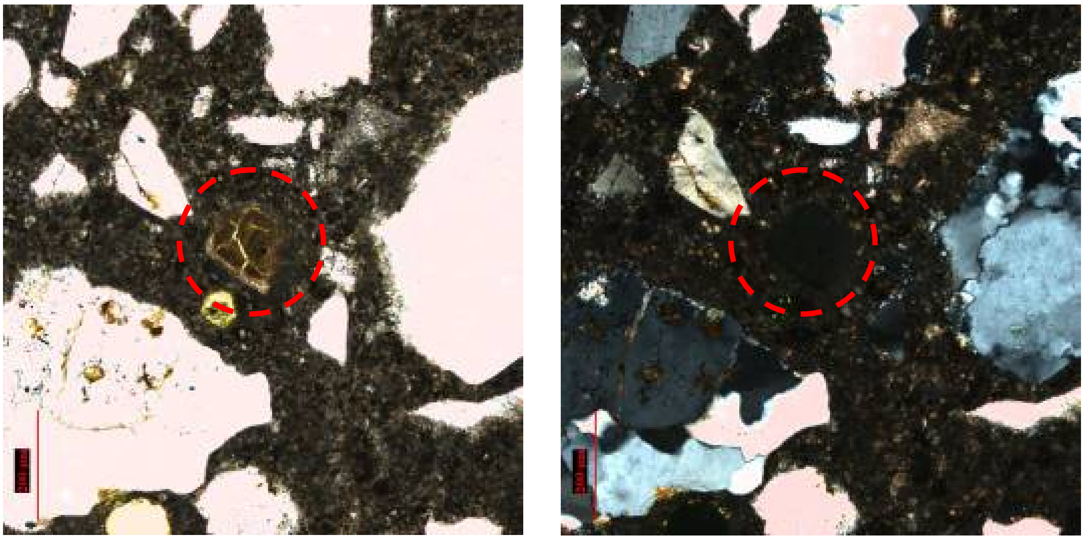


Figure 7.17 : Undispersed Silica-Fume Particle

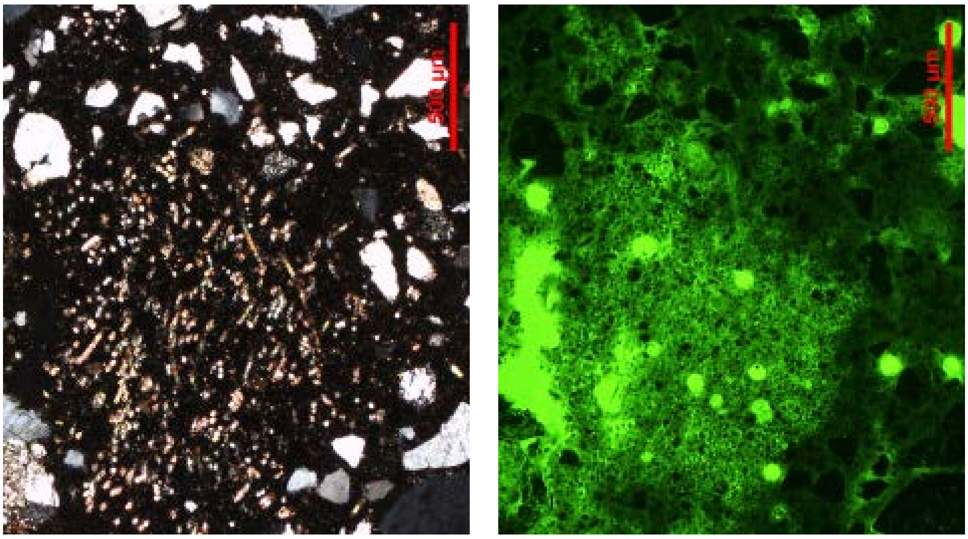


Figure 7.18 : Clump of Fibers in the Matrix

In the Figure 7.18 there is a clump of fiber in the cement paste. It is because of the poor mixing procedure and workmanship. Using fibers in repair materials improves its resistance against crack formation due to volume changes, but if it is not well dispersed, homogeneity lacks in uniformity shown in the Figure 7.18 right image.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

Based on the results of the performed tests it can be concluded that not only the compressive strength and permeability are the essential parameters in selecting repair material but also drying shrinkage and length change of the rapair materials and load-deflection curve of the composite beam, which are the most important factors influencing dimensional compatibility, are important to consider before selecting a repair material. Compatibility between repair materials and the existing concrete is essential for concrete repair durability. The results of this research show that the prepacked repair materials on the market have the desired permeability parameters such as chloride permeability and rate of water absorption, but their compressive strengths are insufficient for high strength precast concretes. As capillary porosity is one of the main aspects of the durability of reinforced concrete structures, the results described above show that using such prepacked repair materials for patch repairs can increase the durability of reinforced concrete structures. Also individual repair material properties do not appear to offer an adequate measure of compatibility with a substrate concrete. Therefore, to make a right choice based on repair material properties is not adequate for a expected durability of the concrete repair.

Based on the results from the laboratıry testing it is found out that for repair materials of lower strength than the substrate concrete, the failure of composite beam should be in the middle-third instead of the edge. Repair materials having high shrinkage values (>0.1%) should be moist cured to avoid incompatible failure. Also using epoxy-primer decreases the incompotible failure modes.

Based on the results from composite cores taken from the structure it is found out using an epoxy-primer is more successful by increasing mechanical and permeability properties. The charge passing through the cores is reduced in rapid chloride test and pull-off tensile strength is increased while using epoxy primer between repair materials and concrete substrate.

Pull-off test is a complex testing process to obtain the bond strength because of high variation. There are many factors influencing the pull-off strength. In this research, it is found out that using epoxy primer generally increases the load of rupture. It is not possible to define a correlation between the rupture loads from the pull-off test and compressive strength of the repair materials. Rupture load from the pull-off test not only depends on the compressive strength of the repair mortar but also compatibility between repair material and the substrate concrete.

One of the most known non-destructive testing methods, Impact-echo response was used to understand the quality of the bond strength at the interface in the repaired concrete structures. When the bond strength between the repair material and the substrate concrete is as good as the pristine concrete, impact-echo stress waves travel along the whole cross-section of the repaired concrete, presenting similar full section response. When the measured bond strength is low impact-echo response presents many peaks in the spectrum. This technique can be qualitatively used to detect areas where interfaces are partially or fully debonded. But measuring the bond strength with impact-echo by correlating with pull-off-test is not possible.

Microstructural analysis is used to determine the defects which are caused generally by the workmanship procedures. Insufficient preparation of concrete surfaces before repairing and insufficient compaction of the repair material can implement further mechanical damages, thus creating weak zones and entrapped air voids at the interface. Fiber clumps and undispersed silica fume particles are observed depending on the quality of mixing of the repair mortar. When the curing is not applied efficiently, shrinkage cracks can be observed in the micro analysis. It is obvious that the performance of a repair material depends on such factors as mixture proportioning and construction operations, finishing and curing. Therefore inspection and quality control according to the working procedures are vital for better performance.

It is found out that there is no indication of differences of the conditions or quality of the samples due different ages of the repairs. But workmanship, especially mixing of the repair material and compacting to the substrate is vital for a good quality repair. Most of the prepacked repair materials include silica fume and fibers, which should be well dispersed in the cement paste to achieve the desired properties. Because of the poor mixing procedure, these components are not only losing their effectiveness but also disservice the material itself.

In conclusion, it was observed, that there was a lack of significant correlation between individual property of the repair materials and field performance. However, these results are limited with top-selling two repair materials and further information is needed to classify the materials more exactly.

8.2 Recommendations

Tests were made for two repair materials and with four material properties. In the future, more repair materials can be used with more mechanical and durability properties.

In this study cores were drilled in the repaired part of the structure and the permeability was measured on these composite cores. In the future, cores can be taken on the repair border to measure the permeability of the interface of the exposed surface.

Beneath dimensional compatibility, for each reinforced concrete repair, electrochemical compatibility must be considered and therefore reliable standard test methods needed to predict the electrochemical behavior in a repaired structure and to select an effective protection system.

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APPENDIX A

Table A1 : Repair Material Selection Guide of ACI

	Favorable properties	Unfavorable properties	Remarks
1. Deep concrete replacements and overlays			
A. Top surface applications			
Concrete	2,3,7,8	1,9,10,11	Most commonly used
Low-w/cm concrete	2,3,7,8	1,11	Improved durability
Silica-fume concrete	2,3,4,5,8,9,10,11	1,7	Significantly improved durability
Polymer-cement concrete	2,3,4,5,8,9,10,11	1,7	Significantly improved durability
Polymer concrete	1,2,[†]4,5,8,9,10,11	1,2,[†]3,7,12	Significantly improved durability; used in special situations
MAPCC	1,2,3,4,5,6,9,10	11	Good durability, good dimensional stability, rapid setting. Used where quick application is desired. Not commonly used for overlays.
B. Vertical and overhead applications			
Concrete	2,3,8	1,4,5	Form-and-cast, preplaced aggregate, and shotcrete applications
Silica-fume concrete	2,3,4,5,6,8,9,10, 11	1,7	Form-and-cast and shotcrete applications
Polymer-cement concrete	2,3,4,5,6,8,9,10, 11	1	Form-and-cast and shotcrete applications
Polymer concrete	4,5,6,8,9,10,11	1,3,7,12	Form-and-cast and preplaced aggregate applications
Cement mortar	2,3	1	Shotcrete and occasionally trowel-applied applications
Silica-fume mortar	2,3,4,5,6,9,10,11	1,7	Shotcrete and occasionally trowel-applied applications

Polymer-cement mortar	2,3,4,5,6,7,9,10, 11	1	Trowel-applied applications and shotcerete
Polymer mortar	4,5,6,8,9,10,11	1,3,12	Trowel-applied applications
2. Shallow concrete replacements and overlays			
Cement mortar	2	1,3,8	Poor durability; used in relatively benign applications
Silica-fume mortar	2,4,5,6,8,9,10,11	1,3	Improved durability; commonly used
Polymer-cement mortar	2,4,5,6,8,9,10,11	1,3	Improved durability; commonly used
Polymer mortar	4,5,6,8,9,10,11	1,2,3,12	Good durability; used in special situations or as the applications gets thinner
MACPM	1,2,34,5,6,8,10	11	Good durability, good dimensional stability, rapid setting
3. Thin overlays			
Cement mortar	-	1,3,8	Sometimes used
Silica-fume mortar	4,6,8,9,11	1,3	Good durability
Polymer-cement mortar	4,6,8,9,11	1,3	Good durability
Polymer mortar	4,5,6,8,9,11	1,2,3,12	Good durability

† For polymer concrete, a lower modulus of elasticity than the substrate concrete is beneficial in relieving differential stresses between the repair material and the substrate concrete.

Important material properties:
1-Volume stability
Mechanical Properties:
2-Modulus of elasticity
3-Coeffcient of thermal expansion
4-Bond strength
5-Tensile strength
Construction characteristics:
6-Cohesiveness
7-Ease of construction
External and chemical environment factors:
8-Freezing-and-thawing durability
9-Permeability
10-Electrical resistivity
11-Resistance to chemical attack
12-Low heat deflection or glass transition temperature

Table A2 : Compressive Strength Test Results for B88

#	Specimen Code	Type	Age [Days]	Results	Average
1	B88	40X40X40	7	53.0	53.7
2	B88	40X40X40	7	53.5	
3	B88	40X40X40	7	53.5	
4	B88	40X40X40	7	53.0	
5	B88	40X40X40	7	54.5	
6	B88	40X40X40	7	54.5	
7	B88	40X40X40	7	53.5	
8	B88	40X40X40	7	54.5	
9	B88	40X40X40	7	53.0	
1	B88	40X40X40	28	61.5	64.8
2	B88	40X40X40	28	64.5	
3	B88	40X40X40	28	65.0	
4	B88	40X40X40	28	65.0	
5	B88	40X40X40	28	66.5	
6	B88	40X40X40	28	65.0	
7	B88	40X40X40	28	61.0	
8	B88	40X40X40	28	66.5	
9	B88	40X40X40	28	68.5	
#	Specimen Code	Type	Age [Days]	Results	Average
1	B88	Φ100mm	7	43.5	49.5
2	B88	Φ100mm	7	52.5	
3	B88	Φ100mm	7	52.5	
1	B88	Φ100mm	28	56.0	53.5
2	B88	Φ100mm	28	51.5	
3	B88	Φ100mm	28	53.0	

Table A3 : Compressive Strength Test Results for S612

#	Specimen Code	Type	Age [Days]	Results	Average
1	S612	40X40X40	1	14.5	13.5
2	S612	40X40X40		13.5	
3	S612	40X40X40		12.5	
1	S612	40X40X40	7	48.5	45.0
2	S612	40X40X40		41.0	
3	S612	40X40X40		45.5	
1	S612	40X40X40	28	58.0	59.5
2	S612	40X40X40		61.0	
3	S612	40X40X40		59.5	
#	Specimen Code	Type	Age [Days]	Results [MPa]	Average [MPa]
1	S612	CY150	7	33.5	33.0
2	S612	CY150		32.0	
1	S612	CY150	28	44.0	43.0
2	S612	CY150		41.5	

Table A4 : Compressive Strength Test Results for Structure Concrete

	Compressive Strength [MPa]				Tensile Strength [MPa]				Modulus of Elasticity, E_0 [GPa]			
Age [Days]	1	2	3	Av.	1	2	3	Av.	1	2	3	Av.
1	32.4	38.1	34.7	35.0	3.45	3.01	3.29	3.25	32.9	28.9	31.7	31.2
7	70.8	62.1	63.9	65.5	5.71	5.83	6.07	5.85	38.0	37.2	37.9	37.7
28	73.9	74.5	73.9	74.0	6.84	6.54	6.47	6.60	46.8	42.8	42.3	44.0

Table A5 : Compressive Strength Test Results for Laboratory Concrete

	Compressive Strength [MPa]				Tensile Strength [MPa]				Modulus of Elasticity, E_0 [GPa]			
Age [Days]	1	2	3	Av.	1	2	3	Av.	1	2	3	Av.
1	13.0	13.5	14.5	13.5	1.35	1.30	1.55	1.40	26.5	29.5	26.5	27.5
7	53.5	51.5	52.5	52.5	4.60	4.30	4.50	4.45	41.0	39.0	38.5	39.5
28	74.0	71.0	72.0	72.5	6.20	6.50	6.95	6.55	44.0	42.5	43.5	43.5

Table A6 : Shrinkage Test Results of B88

L(i-i) [mm]		1	2	3	Avg.
		112,56	110,15	110,92	
days at 23 ± 2 °C		ε [%]	ε [%]	ε [%]	
26.03.2008	1	0,002	0,007	0,000	0,003
27.03.2008	2	-0,011	-0,004	-0,013	-0,009
28.03.2008	3	-0,036	-0,025	-0,037	-0,033
31.03.2008	6	-0,074	-0,061	-0,079	-0,071
02.04.2008	8	-0,087	-0,074	-0,090	-0,084
03.04.2008	9	-0,089	-0,077	-0,093	-0,086
04.04.2008	10	-0,107	-0,097	-0,105	-0,103
06.04.2008	12	-0,108	-0,098	-0,112	-0,106
07.04.2008	13	-0,111	-0,103	-0,114	-0,109
08.04.2008	14	-0,110	-0,106	-0,118	-0,111
After 3 days 100°C oven 34		-0,202	-0,197	-0,204	-0,201

Table A7 : Shrinkage Test Results of S612

L(i-i) [mm]		1	2	3	Avg.
		111,08	112,80	115,52	
days at 23 ± 2 °C		ε [%]	ε [%]	ε [%]	
11.04.2008	1	-0,006	-0,004	-0,007	-0,006
13.04.2008	3	-0,014	-0,012	-0,014	-0,014
15.04.2008	5	-0,021	-0,020	-0,020	-0,020
16.04.2008	6	-0,032	-0,031	-0,029	-0,031
18.04.2008	8	-0,043	-0,045	-0,041	-0,043
20.04.2008	10	-0,062	-0,062	-0,055	-0,060
21.04.2008	11	-0,062	-0,060	-0,057	-0,060
24.04.2008	14	-0,076	-0,077	-0,074	-0,075
After 3 days 100°C oven		-0,164	-0,162	-0,160	-0,162

Table A8 : Rapid Chloride Test Results for Repair Materials

#	Specimen Code	Age [Days]	Results [Coulomb]	Average [Coulomb]	St. Dev [Coulomb]
1	S612	42	230	272	46
2	S612	42	286		
3	S612	42	352		
4	S612	42	266		
5	S612	42	272		
6	S612	42	225		
#	Specimen Code	Age [Days]	Results [Coulomb]	Average [Coulomb]	St. Dev [Coulomb]
1	B88	11	5174	5876	776
2	B88	11	5857		
3	B88	11	6912		
4	B88	11	6165		
5	B88	11	6340		
6	B88	11	4806		
1	B88	36	2385	2125	228
2	B88	36	2031		
3	B88	36	1958		

Table A9 : Rapid Chloride Test Results for Substrate

	Age [days]	Average Charge Passed [coulombs]	Class	Average
Laboratory	37	1993	Low	2075
	37	2236	Moderate	
	37	1995	Low	
Structure	45	1635	Low	1672
	45	1642	Low	
	45	1740	Low	

Table A10 : Chloride Diffusion Test Results for Specimens

	#	$D_E [m^2/s]$	K_{Cr} [mm/√year]
B88	1	$1.32x10^{-12}$	17
	2	$1.7x10^{-12}$	19
	3	$1.31x10^{-12}$	17
S612	1	$6.47x10^{-13}$	12
	2	$4.46x10^{-13}$	10
	3	$6.05x10^{-13}$	11
	4	$5.5x10^{-13}$	11
	5	$4.2x10^{-13}$	9
	6	$7.37x10^{-13}$	13
Structure	1	$5,59x10^{-12}$	31
	2	$4,95x10^{-12}$	30
	3	$6,55x10^{-12}$	34

Table A11 : Rapid Chloride Test Results for Φ100mm Repaired Cores

#	Specimen Code	Primer Type	Age [Days]	Results [Coulomb]	Average [Coulomb]	St. Dev [Coulomb]	Repair Thickness [mm]
1	S612	S610	76	541	525	133	20.2
2	S612		76	384			8.0
3	S612		76	649			21.0
4	S612	S610	78	304	378	67	20.0
5	S612		78	395			17.4
6	S612		78	435			19.7
1	S612	Epoxy	72	297	237	64	17.9
2	S612		72	298			26.5
3	S612		72	253			19.2
4	S612		72	161			17.3
1	S612	Epoxy	78	205	223	20	23.8
2	S612		78	220			18.9
3	S612		78	244			27.5
1	B88	B88	61	2949	3053	129	21.0
2	B88		61	3198			14.3
3	B88		61	3013			19.1
1	B88	Epoxy	68	518	403	100	15.7
2	B88		68	354			14.1
3	B88		68	338			11.4

Table A12 : Pull-off Test Results for S612

#	Primer	Repair Age [Days]	Pull-Off Strength [MPa]	Average [MPa]	Concrete Age [Days]	Location of Rupture [%]	Operator
1	Epoxy	58	3.52	3.47	136	Repair	MÇ
2			3.29			CJ	
3			3.18			CJ	
4			3.40			Repair	
5			3.98			Substructure	
6			3.52	2.94		Substructure	YG
7			2.40			Substructure	
8			2.51			Substructure	
9			4.09			Repair	
10			2.95			CJ	
11			2.18			Substructure	
1	S610	58	1.95	2.13	136	Repair	YG
2			1.71			Repair	
3			2.18			Repair	
4			2.29			Substructure	
5			2.51			CJ	
6			1.95	2.17		CJ	MÇ
7			2.40			CJ	
8			1.95			CJ	
9			2.06			CJ	
10			2.06			CJ	
11			2.62			Repair	
1	Epoxy	29	2.62	2.51	338	Disk	-
2			2.73			Disk	
3			2.18			Substructure	
1	S610	31	1.18	1.28	124	Repair	
2			0.96			Repair	
3			1.41			Repair	
4			1.29			Repair	
5			0.74			Repair	
6			1.18			Repair	
7			0.61			Repair	
8			0.96			Repair	
9			1.41			Repair	
10			0.96			Repair	
11			1.96			Repair	
12			1.73			Repair	
13			1.85			Repair	
14			1.85			Repair	
15			1.96			Repair	
16			1.61			Repair	
17			0.85			Repair	
18			0.53			Substructure	
1	S610	36	1.41	1.74	129	CJ	
2			1.96			CJ	
3			1.61			CJ	
4			1.61			CJ	
5			2.09			CJ	
1	S610	42	2.77	2.11	135	CJ	
2			1.96			CJ	
3			1.61			Substructure	

Table A13 : Pull-off Test Results for B88

	Primer	Repair Age [Days]	Pull-Off Strength [MPa]	Average [MPa]	Concrete Age [Days]	Location of Rupture [%]	Operator
1			1.95	2.66		N.A.	
2			3.06			N.A.	
3			2.40			Substructure	YG
4			3.29			CJ	
5			3.29			Substructure	
6			1.95			N.A.	
7	Epoxy	53	1.95	2.29	122	N.A.	
8			1.95			Repair Material	
9			2.40			Repair Material	MÇ
10			2.40			N.A.	
11			2.18			N.A.	
12			2.84			90CJ-10Substructure	
1			2.18			CJ	
2			1.71	2.02		CJ	MÇ
3			2.18			N.A.	
4	B88	53	2.18		122	CJ	
5			1.48			CJ	
6			2.29	1.97		CJ	
7			1.95			CJ	
8			1.95			CJ	
2			2.30			Repair Material	
3			2.65			CJ	
4	Epoxy	34	2.77	2.33	129	CJ	
5			1.85			Disk	
6			2.09			CJ	
1			1.41			CJ	
2			1.61			CJ	
3			1.61			CJ	
4	B88	40	1.85	1.99	135	Substructure	
5			3.00			CJ	
6			1.73			CJ	
7			2.19			Substructure	YG
8			2.54			CJ	
1	Epoxy	40	2.54	-	135	Repair Material	
1			1.85			80Repair Material-20CJ	
2			1.85			Repair Material	
3			0.96			20CJ-80 Substructure	
4			1.61			65Repair-15CJ-30Sub	
5	B88	26	1.41	1.77	75	30RM-70CJ	
6			2.54			Repair Material	
7			2.30			10CJ-90Sub	
8			2.77			40RM-60CJ	
9			1.18			70CJ-30Sub	
10			1.18			15RM-75CJ-10Sub	

CURRICULUM VITAE

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