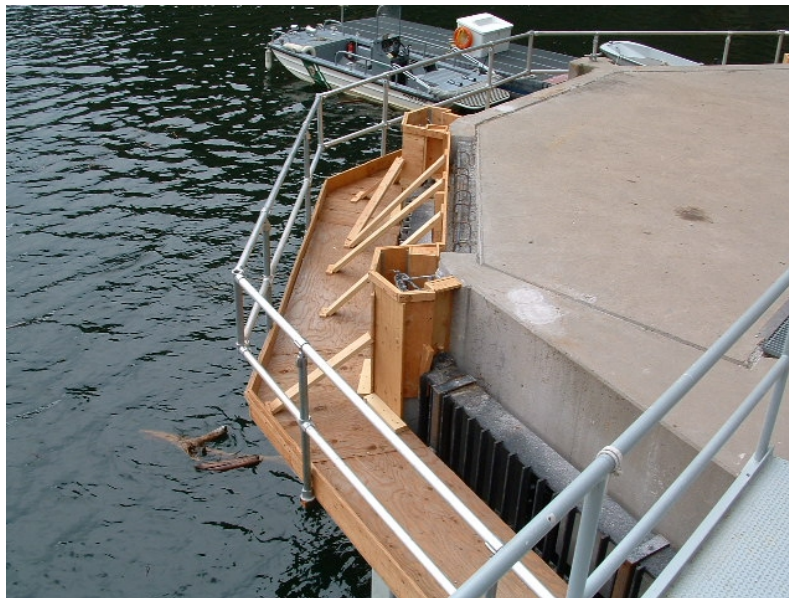# RECLAMATION
## *Managing Water in the West*

# Development of Specifications and Performance Criteria for Surface Preparation Based on Issues Related to Bond Strength

**Research and Development Office**
**Science and Technology Program**

**American Concrete Institute Foundation**
**Concrete Research Council**

**Final Report ST-2017-2886-1**

# Mission Statements

The U.S. Department of the Interior protects America's natural resources and heritage, honors our cultures and tribal communities, and supplies the energy to power our future.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public

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# Development of Specifications and Performance Criteria for Surface Preparation Based on Issues Related to Bond Strength

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# Executive Summary

Repair and strengthening of existing concrete structures are among the biggest challenges civil engineers face today and will face in the years to come. The present emphasis on sustainable development highlighting repair instead of new construction will only increase this challenge. Concerted efforts towards improving the durability of concrete repairs are still needed from scientists and engineers. One of the critical aspects of durability of concrete repairs and overlays is lasting and sufficient interfacial bond between the repair material and existing concrete substrate. This report contains the experimental data and findings from a long-haul international collaborative study sponsored by the ACI Concrete Research Council and the research partners devoted to the most significant factors influencing bond in repairs. Based on the test results collected in complementary test programs, guideline-type recommendations for concrete substrate surface preparation prior to repair were developed.

The primary objective of this research study was to identify the fundamental factors and characteristics of concrete substrates prepared for repair/overlay and to develop guideline specifications for surface preparation of existing concrete. The research activities, which included both laboratory and field testing and evaluation, were based upon the following specific objectives:

- To evaluate available methods for assessing the roughness parameters of a concrete surface prepared for repair;
- To evaluate the relationships between shear bond strength, pull-off tensile strength, and surface roughness;
- To estimate the effect of misalignment in a tensile pull-off test on the maximum recorded load;
- To develop a field test procedure for evaluating the optimum moisture conditioning of the particular concrete substrate;
- To evaluate the influence of concrete carbonation of the existing concrete on repair bond strength;
- To develop performance criteria and guide specifications for surface preparation of concrete prior to repair.

The experimental program was divided into the following six tasks:

- Task 1: Evaluation of existing techniques for the characterization of concrete surface roughness.
- Task 2: Evaluation of relationships between tensile bond, shear bond and surface roughness parameters.
- Task 3: Evaluation of the effect of misalignment upon pull-off testing results.
- Task 4: Appraisal of test procedures for evaluating the optimum moisture conditioning of a given concrete substrate.
- Task 5: Evaluation of the effect of substrate concrete carbonation upon repair bond strength.
- Task 6: Specification guidelines for surface preparation of concrete prior to repair.

Over the course of the project, the following surface preparation techniques were investigated:

- Jackhammering (handheld concrete breakers);
- Sandblasting;
- Scarifying;
- Shotblasting;
- Water jetting.

Each task is presented as a separate section in this report, with a corresponding table of contents, an in-depth discussion of testing and results, conclusions, and references.

# Conclusions

Among the techniques available today to measure surface roughness, the best suited method for field assessment appears to be the Concrete Surface Profile Chips (CSP) developed by the International Concrete Repair Institute (ICRI). They are quick and easy to use and yield reliable information, irrespective of the surface orientation. Its only shortcoming is that very rough surfaces are beyond the limits of the current set of chips.

Optical methods based on the moiré pattern to characterize surface roughness may be very effective after technological advances make the technology more suitable for field use. It offers significant advantages in terms of production rate and surface area treatment capability and yields a handful of reliable quantitative data.

Bond strength testing is a convenient and useful in-situ method for evaluating the mechanical integrity of the concrete surface prior to repair, as well as the repair bond strength. A reliable evaluation of bond properties can be obtained, provided that a minimum number of tests are performed, with adequate equipment and properly conducted testing.

No general correlation between the shear bond strength and tensile bond strength could be established due to the fact that the various surface preparation techniques result in different types of profiles and induced defects.

Increased surface roughness improves bond strength.

The use of impacting methods such as jackhammering leaves significant damage at the surface, which can easily outweigh the benefits of an increased roughness.

Assuming the pull-off testing process is followed correctly, and within a pull-off testing misalignment angle detectable by the average human eye (4° in the present study), load and coring misalignments were not found to yield significantly different stress fields and, for practical purposes, do not significantly alter the pull-off strength evaluation.

From a quality assurance/quality control standpoint, the results generated in the misalignment study indicate that pull-off test misalignments result in lower pull-off strength test results. This may cause the unnecessary rejection of repairs with adequate bond strength. If a sufficiently large number of tests are performed for statistical significance, a lower acceptance pull-off

strength limit in the field could be specified – with the mean acceptance value being decreased, for example by 5 to 10 %, to take into account the bias induced by testing misalignment. However, in most cases, the number of tests performed on site is usually limited and using the specified bond strength value as the field acceptance limit for quality control is conservative and is recommended.

The *Initial Surface Absorption Test* (ISAT) inspired by an experimental device initially developed at Queen's University in Belfast [16], and the modified capillary suction test (MCST) developed at the University of Liège in Belgium [17], provide good methods for testing concrete saturation levels. ISAT is quantitative test method to evaluate the saturation level of a concrete substrate: it is compact, cost-effective and rapid. The MCST test yielded clearer trends and less dispersed information than the ISAT test, as well as a better correlation with water content measurement.

There is a large range of saturation levels (50 to 90%) where bond strength remains high and constant, which seems to limit the influence of environmental conditions on adhesion of cement-based repair systems. The bond strength is relatively low for low saturation levels (≤ 50%), but it reaches higher values for saturation levels comprised grossly between 55 and 90% (this range likely varies depending on the nature of the overlaying material).

When an acrylic emulsion is used as a bonding agent, the highest saturation levels induce a water film at the interface, which is incompatible with most polymeric materials and reduces the effectiveness of adhesion.

These findings show the effect of water in the substrate concrete superficial zone and the difficulty encountered in evaluating reliably the actual saturation level. For the repair systems considered in this task, it seems that optimum saturation levels for repair bond strength would lie somewhere between 55 to 90%. Clearly, additional work is required to identify a methodology that could be used in field applications and, furthermore, to assess what are the optimum moisture ranges for cement-based repair materials. (A portion of this work was accomplished and is reported in *Concrete Substrate Moisture Requirements for Effective Concrete Repairs, ST-2016-2886-01*, Vaysburd, et.al, Science and Technology Program, Bureau of Reclamation, 2016)

For substrate surfaces prepared by sandblasting there was no difference in bond strength found between carbonated and non-carbonated concrete surfaces. However, for substrate surfaces prepared by chipping a significant reduction (16%) of bond strength was documented for carbonated surfaces compared to non-carbonated. The different effects of carbonation were attributed to the micro defects (bruising) of the surface prepared by chipping hammer, further aggravated by the carbonation process and inherent volume changes (carbonation shrinkage).

Jack hammers (concrete breakers) inevitably leave some bruising within the superficial layer of the concrete surface being treated and its detrimental effect upon repair bond can only be amplified if carbonation is allowed to occur prior to repair. However, it appears that carbonation may have little or no impact on bond strength for an otherwise sound, properly prepared concrete substrate surface.

Bonding agents are not recommended for repairs and overlays employing cement-based materials. Their use cannot compensate for inadequate surface preparation and may act as bond breakers when used improperly. Bonding agents provide an additional operation and material layer that can cause failure, e.g. a bonding agent that is allowed to cure prior to material placement, becoming a bond breaker. In addition, a bonding grout may have a high water-cement ratio leading to a low strength and risk of adhesive failure within the bonding agent layer itself.

The results of this large study were used to prepare a generalized guide specification to provide information on the surface preparation of concrete prior to repair and overlay. It can be used as a starting point by individuals involved in developing project specifications who are competent to analyze the significance and limitations of the guide specifications' content and who will accept responsibility for the application of the material and provisions it contains.

# Subject Areas for Further Study

Several fundamental aspects concerning concrete surface preparation prior to repair/overlay and bond strength development were addressed in this study. Although the results and analysis resulted in a better overall understanding of the problem, a number of questions remain unanswered. Studying the issues below would be the most effective way to provide more information to gain a better understanding to achieve optimum bond performance (strength and durability) in composite repair and overlay systems.

## Substrate roughness

As discussed in sections 2 and 3 of this report, the CSP replicate system proposed by ICRI is a useful tool for evaluating the roughness of a concrete surface prior to the application of a protective system, a coating or a layer of mortar. However, the actual CSP plates are rather narrow with respect to the spectrum of CSPs obtained with actual surface preparation techniques. In fact, with the existing 10-level scale, its use is limited at present and is thus confined to surface treatment applications where very little material is actually removed and is irrelevant in many concrete repair applications.

It is possible to extend the range of the CSP replicate system in order to cover rougher profiles, typical of those obtained with water jetting and jackhammering. Moreover, it can be improved through a real quantitative approach, by using optical profilometry. The identification of reference curves, but on a wider scale of surface roughness, will help broaden the range of application of this method to much coarser profiles and allow the identification of quantitative roughness criteria.

## Moisture conditioning of the concrete prior to repair

Despite the work accomplished in this project, some fundamental issues remain unresolved with regard to moisture conditioning of the concrete substrate prior to repair. In daily repair practice, inevitably loose specifications and the absence of measuring tools actually result in a wide range of moisture conditions.

In order to develop proper specifications, it is necessary to gain a better understanding of the transport mechanisms between repair materials and concrete substrates and the influence of the moisture state of the substrate upon bond development.

Both the issuing and implementation of such specifications will, in turn, require the development of a test method to evaluate quantitatively the actual moisture condition of concrete in the laboratory, as well as in the field. The envisioned method would allow the determination of optimum conditions for a given concrete substrate, as well as quality control testing. The method needs to be simple and applicable to both laboratory and in situ conditions. In that regard, further investigation should be directed towards measurement techniques already available, such as electrical impedance devices (flooring industry) or superficially encased relative humidity probes.

## Long-term bond

It must be emphasized that this study, as well as other reported work on the subject, is primarily dealing with "short-term" bond strength issues, not with the mechanisms and issues related to long-term bond behavior and durability. The short-term bond strength typically specified and evaluated can be used as an indication of the quality of workmanship (i.e., concrete surface preparation for repair, material selection, application, and curing). Long-term bond strength, however, is usually influenced by various other factors, among them environmental, loading, and fatigue conditions.

Therefore, it is desirable to pursue research efforts on those factors affecting long-term bond strength in concrete repair/overlay systems, notably the surface preparation parameters and characteristics.

## Compatibility issues in repair/overlay systems

When compatibility issues are properly addressed in repair systems, durability of the bond is achieved, as it ensures a lasting coexistence of the repair material and substrate concrete.

Incompatibility issues cause premature debonding and repair failures. Unfortunately, at the present time, much confusion, misconceptions, and misleading guidance exist concerning compatibility of repair materials and the substrate concrete. These issues negatively affect the design, specification, implementation, and, as a result, service life of concrete repairs and overlays.

Development of reliable guidelines addressing compatibility issues, with special emphasis on the factors related to dimensional compatibility issues, is needed for the repair industry to evolve as an engineering discipline.

As mentioned above, each task is discussed separately in sections, as essentially a series of stand-alone detailed reports.

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# Section 1 - General Introduction

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# Section 1 - General Introduction

## Background

The aim of a concrete repair or overlay[1] is to prolong the service life of the deteriorated/distressed concrete structure or a concrete element of that structure, to restore the load-carrying capacity and the stiffness or to strengthen the structure or its member. Consequently, monolithic action in the composite repair system is critical. A prerequisite for monolithic action is sufficient, long lasting bond between the existing substrate and the newly cast repair/overlay material.

The long-term performance of concrete repairs and overlays can be, to a large extent, linked to their resistance to debonding and cracking.

The mechanisms of cracking and debonding are complex and mainly depend on material characteristics, environmental influences, and degree of restraint. The most important repair/overlay material characteristics with respect to bond are tensile strength and extensibility properties of the viscoelastic repair materials and volume changes caused by shrinkage. Development and magnitude of interfacial bond strength and most importantly bond durability also largely depend on substrate surface preparation and workmanship during repair/overlay application.

The mechanisms and characteristics of bond between existing concrete and repair materials with respect to various factors and influences have been the subject of a many studies in recent years. The number of ongoing research projects in the field, the current state of knowledge, the codes of practice, and especially the poor performance of many repairs, however, show that many questions still remain unresolved. As a result, guidelines and project specifications for the design and implementation of repairs are often deficient in scope, performance criteria and detailing.

The scope of existing guidance and specifications concerning concrete repair bond issues is presently limited. Existing standards and specifications can basically be divided into design specifications for concrete repairs and structural design procedures for load-bearing concrete overlays.

This is mainly due to the need for further understanding of the factors and conditions affecting bond strength and durability as affected by the substrate concrete surface preparation for repair/overlay [1]. A number of research projects have been documented in the literature, discussing aspects of bond properties and characteristics in terms of material properties, concrete substrate surface texture and condition, curing procedures, as well as some environmental factors of influence. However, despite the relatively large pool of theoretical knowledge, the practical issues related to surface preparation of existing concrete for repair in order to systematically

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[1] In ICRI Concrete Repair Terminology (2010), an overlay is defined as *a bonded or unbonded layer of material placed on a concrete surface to either restore or improve the function of the previous surface*. From this point of view, an overlay can thus be considered as a repair, since its performance and durability are depending essentially on the same considerations.

achieve a lasting interfacial coexistence in a composite repair system are still inadequately addressed. Unfortunately, as a result, repair and overlay debonding is still frequently observed.

The need for more practical design and application recommendations has widely been recognized by designers, materials specialists and contractors [2, 3].

Regardless of the cost, complexity and quality of repair material or application method employed, the quality surface preparation of the substrate prior to the repair will often determine whether a repair project is a success or a failure and whether a repaired structure meets the design objectives [4].

The durability, in this context, may be defined as the lasting interfacial coexistence of two composite materials combined into a composite system. Although most of the specified requirements and engineers' considerations tend to focus on the achievement of the prescribed initial bond strength, it must be noted that this parameter can be only considered as a necessary condition, but certainly not a sufficient one. The most critical aspect is the durability of the bond, which is governed to a large extent by the service conditions of the repaired structure.

Concrete substrates are different. They differ in age, quality and service exposure: from relatively new concrete to very old and deteriorated concrete, exposed to various temperatures, relative humidities, chemically aggressive interior (inside the concrete substrate) and exterior environments, electrochemical condition and mechanical loads.

At the time this project was undertaken, published data and information allowed for the following characterization of the concrete substrate to be repaired/overlaid:

- It is physically and chemically very complex;
- Such complexity is variable from case to case;
- The complexity has to be considered on the basis of scale, which is relevant and dependent on the particular situation;
- Practical answers and guidance/performance criteria at the present time, as well as the problem of achieving optimum bond in the repair/overlay composite systems, depend more on broad judgment and experience rather than detailed knowledge.

An in-depth literature survey on concrete repair bond issues carried out at the onset of this project showed that many critical details and parameters are still ill-defined. Research is thus needed in order to develop or improve field test characterization methods, in order to enable the identification and field assessment of dependable performance criteria (QA & QC) for practical repair applications.

## Bond in repair and overlay composite systems

The characteristics of adhesion or "bond" can be studied from two opposite perspectives. On the one hand, it can be addressed based upon the conditions and nature of the contact between two materials, taking into account different bond mechanisms. On the other hand, it can be appraised through a quantitative measurement of the magnitude of adhesion, usually expressed in terms of stress or energy required to separate the two materials. In practice, available information on

repair adhesion commonly refers to the equivalent average tensile stress required to separate the concrete substrate and the repair material [5].

The term "adhesion" describes the condition in the boundary layer between two bonded materials with a common interface. Adhesion mechanisms can be divided basically into thermo-dynamic mechanisms, chemical bonding and mechanical interlocking.

Mechanical adhesion in repaired concrete members relies on the penetration and hardening of the repair material inside the open cavities and asperities at the surface of the concrete substrate and the physical anchorage resulting therefrom. Capillary absorption plays an important role in the anchorage effect, as it draws cement paste (or any other binding system being used) into small cavities of the substrate. The extent of this effect is dependent on the moisture condition of the substrate (mainly the surface moisture) and the viscosity of the repair material.

It is important to note that mechanical adhesion in tension differs significantly from that in shear. For example, a high interface roughness may improve shear bond strength, whereas tensile mechanical bond strength primarily depends on vertical anchorage in pores and voids (Figure 1-1).
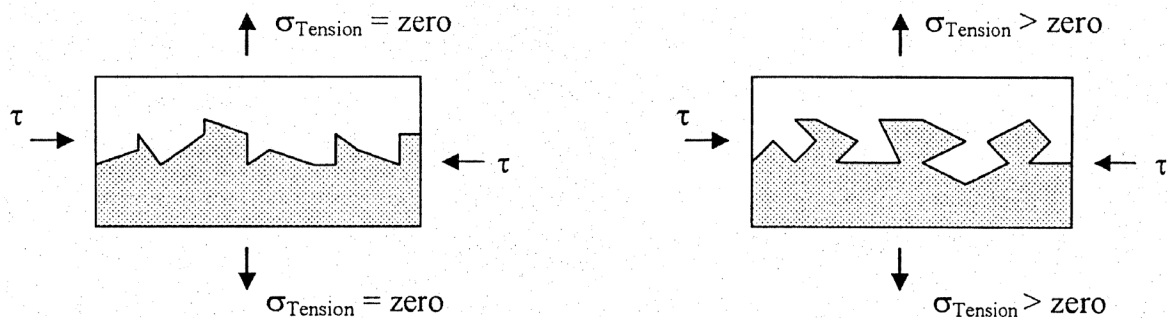


*Figure 1-1: Schematics of mechanical shear and tensile bond between substrate and repair, resulting from interlock mechanisms [6]*

The above is important for the correct choice of bond strength test methods and for appraising the relationship between the measured shear and pull-off (tensile) bond strengths. Usually, differential volume changes resulting from drying shrinkage or temperature gradients cause both shear and tensile stresses at the interface. In structural design, tensile stresses perpendicular to the interface are rare. By contrast, interface shear stresses occur frequently in composite systems such as repairs and overlays. Standards, specifications and established practice in the concrete repair field define bond strength commonly in relation to tensile strength (pull-off bond test) alone which, according to the above considerations, may be insufficient. Still today, the pull-off test is the only method commonly applied in the testing of bond strength on real structures.

Pigeon and Saucier [7] consider the interface between old and new concrete to be very similar to the well-known ITZ (interfacial transition zone) developing between the aggregates and the cementitious matrix in the bulk concrete. According to them, its formation is deeply influenced by the "wall effect", which leads to the presence of a weak layer within the resulting transition zone (Figure 1-2). Many will argue that the presence and extent of such a weak zone is dependent on the surface preparation performed prior to repair.
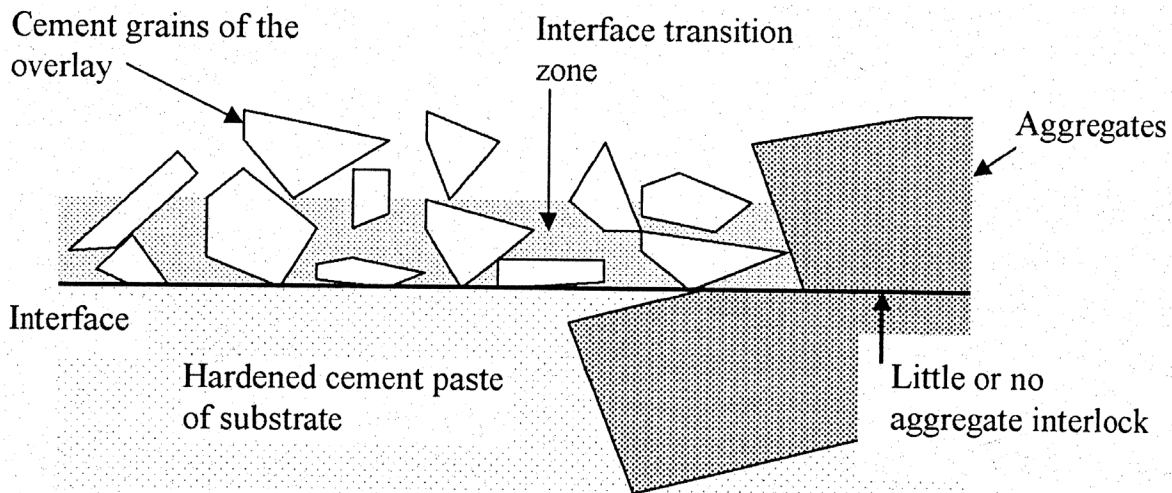
*Figure 1-2: Weak zone between substrate and repair system [7]*

Emmons and Vaysburd [8] presented an idealized model of a surface repair as a three-phase composite system, consisting of the existing concrete, the repair material and a transition zone between them (Figure 1-3). The authors stated that the characteristics of the transition zone are a function of the properties of the substrate (adherent), the properties of the repair material (adhesive) and the substrate surface preparation. Environmental factors, such as temperature or moisture, also play an important role on the properties of the interface region and on interfacial bond development.

A possible macroscopic characterization of the quality or degree of adhesion is obtained by the introduction of a transition zone along the geometrical interface between the adhesive and adherent phases. The thickness of the transition zone is the sum of the lengths in the adherent and the adhesive, where interactive forces of any nature change the mechanical nature of the original continuum [9]. This explains why the authors are referring to the so-called "*interphase zone*" [10].
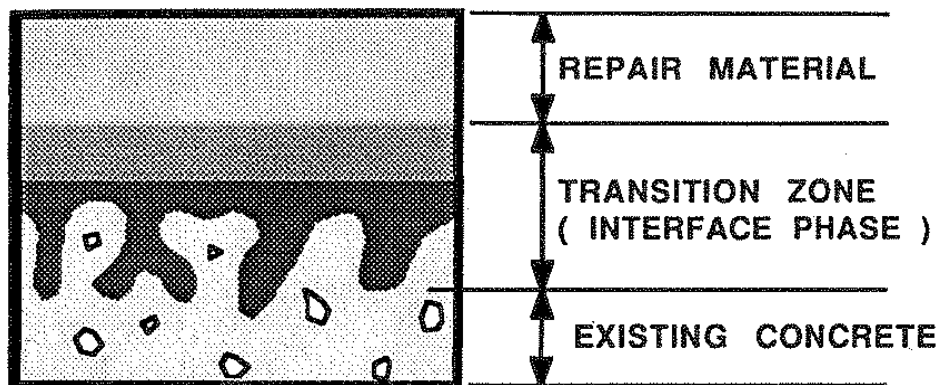


*Figure 1-3: Idealized model of a surface repair system [8]*

Adherence between a repair layer and the existing concrete is a case of adhesion between two solids, as a result of setting and hardening of an initially semi-liquid substance (repair material mixture) poured on a solid substrate (existing concrete).

The following factors have the greatest influence on the development of the transition zone and the resulting bond durability:

- Physical and chemical properties of the concrete substrate;
- Characteristics and condition of the prepared surface;
- Physical and chemical properties of the repair material;
- Environmental conditions (notably moisture and temperature).

The repair material and concrete substrate, similar to a variety of glued connections, can be considered as a contact connection where the constitutive binder in the repair material acts as the glue. In this case, the bond strength can be considered to be the result of mechanical interlocking, specific adhesion, and cohesion.

Mechanical anchorage of the repair develops within the roughness and the porosity of the substrate surface. When estimating the effect of the existing substrate surface, not only its roughness, but also the size and form of protrusions must be taken into account. In the case of extended and gentle unevenness, an increase of the bond strength only comes essentially from an increase in the effective contact area. The specified properties of the repair material (e.g. consistency, method of compaction, etc.) may have a considerable influence on mechanical anchorage and adhesion. The bond strength developing between existing concrete and a repair layer also depends to a great extent on cohesion of the repair material, which itself is determined by the actual strength of the constitutive binder, its mineralogical components and the conditions prevailing during the curing period.

Adhesion and cohesion are closely interconnected in the overall formation process of the contact zone. It is assumed that ultimately, adhesion is the most important aspect in the overall bond strength components.

## Objective and scope

Of critical importance to the long term useful performance of the composite repair/overlay system is the existing concrete surface preparation prior to application of the repair material. Proper surface conditioning is essential for the durability of the repaired structure. The repair material is often blamed for "*not sticking*", but the source of the trouble usually stems from inadequate surface preparation. Many factors have to be considered when addressing the influence of the concrete surface on bond in a composite repair/overlay system, notably the macro- and micro-roughness of the substrate surface, its mechanical integrity, its absorptivity and moisture content, its porosity characteristics governing the contact angle with the binding phase, the chemical and mineralogical makeup of the existing concrete, the condition of the substrate, and the exposure conditions.

The primary objective of this research study was to identify the fundamental factors and characteristics of concrete substrates prepared for repair/overlay and to develop guideline specifications for surface preparation of existing concrete.  The research activities, which included both laboratory and field testing and evaluation, were based upon the following specific objectives:

- To evaluate available methods for assessing the roughness parameters of a concrete surface prepared for repair;
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Over the course of the project, the following surface preparation techniques were investigated:

- Jackhammering (handheld concrete breakers);
- Sandblasting;
- Scarifying;
- Shotblasting;
- Water jetting.

Through the generation of a wide set of test data obtained in well controlled conditions, the research aimed at a better understanding of strength and long-term performance of repairs and bonded overlays.  Experimental procedures and test parameters were chosen based on literature review, theoretical background, and the related experience gained by the investigators in their respective activities (research, design and practice).

Even though this project had a wide scope, not all relevant issues were addressed in the research program.  In particular, the investigation was limited to normal weight concrete substrates and ordinary cement-based repair/overlay materials (for example, resin-based and lightweight concrete substrates and repair materials were beyond the scope of investigation).

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# Section 2 - Evaluation of Existing Techniques for the Characterization of Concrete Surface Roughness
# (Task 1)

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# Section 2 - Evaluation of Existing Techniques for the Characterization of Concrete Surface Roughness
# (Task 1)

This section of the report discuses work and findings for Task 1 of the project, which is devoted to evaluating existing techniques for characterizing concrete surface roughness. For the most part, the research operations were performed in the laboratories of the Department of Building Materials Engineering (DBME) at the Warsaw University of Technology (Poland), GeMMe Building Materials in the ArGEnCo Department at the University of Liège (Belgium), and Laval University, Quebec (Canada).

# Introduction

The roughness of the substrate is one of the parameters often considered to affect adhesion strength between repair material and existing concrete.

Nevertheless, this has been controversial for a number of years, because some reported bond test results have shown that surface roughness exerts only a minor influence on tensile bond. For example, in the tests performed by Silfwerbrand [1], adhesion to rough, water-jetted surfaces was compared with bond to smooth sandblasted surfaces. It was concluded that there could be a roughness "threshold value" beyond which further improvement of the substrate roughness would not enhance bond strength. According to these test results, the "threshold value" ought to be close to the surface roughness of typical sandblasted surfaces. However, it remains the opinion of many specialists in the industry that a rougher surface is beneficial to bond strength. Given that roughness depends directly on the surface preparation method, the investigations presented here are intended to shed new light on the subject and to ultimately resolve the controversy.

According to American National Standards Institute (ANSI) [2], the methods for measuring roughness and surface texture can be classified into three types: contacting methods, taper sectioning and optical (non-contacting) methods. Taper sectioning is used in metallurgy and involves cutting across a surface at a low angle $\alpha$ to physically amplify asperity heights by ctg $\alpha$ [3]. Among the contacting methods there are stylus-type profilometers, tactile tests, measurement of kinetic friction, measurement of static friction, rolling ball measurements, and measurement of the compliance of a metal sphere with a rough surface. Optical (non-contacting) methods include laser profilometry, interferometry and optical reflecting instruments. Light microscopy and scanning electron microscopy may be counted in this group of methods.

A state-of-the-art review on roughness quantification methods for concrete surfaces was recently presented by P. Santos et al. [4] (Table 2-1).

A variety of approaches have been used over the years to characterize the surface roughness of concrete: evaluation of the proportion of the surface occupied by aggregates, measurement of the maximum roughness amplitude, adhesion tests, and calculation of surface parameters based on

image analysis or on microscopy observations, etc. However, many of these methods are unable to provide a sufficiently detailed representation of the actual surface profile for the calculation of morphological and statistical parameters and are not user-friendly under field conditions. In order to achieve a reliable quantitative analysis of superficial concrete morphology after surface preparation, different profilometry and surfometry techniques can be used [4–10]. The data obtained through such techniques makes it possible to conduct a real quantitative assessment of the surface profile by means of statistical parameters calculated from the total superficial profile [11] and from the filtered waviness (low frequency/macroroughness) and roughness (high frequency/microroughness) profiles [12]. Some of these parameters, e.g., the arithmetic mean profile and the flatness coefficient, are particularly effective, both for the shape of valleys and peaks, as well as for their amplitude and frequency [13].

*Table 2-1: Comparison of various methods of concrete surface geometry characterization [4]*

| Roughness quantification method | Quantitative evaluation | Non-destructive | Cost | Portable | Work intensive | Contact with the surface |
|---|---|---|---|---|---|---|
| Concrete surface profile (CSP) | No | Yes | Low | Yes | No | No |
| Sand patch test | Yes | Yes | Low | Yes | No | Yes |
| Outflow meter | Yes | Yes | Low | Yes | No | Yes |
| Mechanical stylus | Yes | No | Moderate | No | Yes | Yes |
| Circular track meter | Yes | Yes | Moderate | Yes | No | No |
| Digital surface roughness meter | Yes | Yes | Moderate | Yes | No | No |
| Microscopy | Yes | No | High | No | Yes | No |
| Ultrasonic method | No | Yes | Moderate | Yes | No | No |
| Slit-island method | Yes | No | Low | No | Yes | Yes |
| Roughness gradient method | Yes | No | Low | No | Yes | Yes |
| Photogrammetric method | Yes | Yes | Moderate | Yes | Yes | No |
| Shadow profilometry | Yes | Yes | Low | Yes | Yes | Yes |
| Air leakage method | No | Yes | Low | Yes | No | Yes |
| PDI method | Yes | No | Low | No | Yes | Yes |
| 2D LRA method | Yes | Yes | Moderate | Yes | No | No |
| 3D laser scanning method | Yes | Yes | High | Yes | No | No |

The selected characterization techniques were compared for effectiveness, accuracy, consistency and field applicability. The following techniques were analyzed on a comparative basis:

- *Concrete surface profile* (CSP), in accordance with ICRI Guideline No. 310.2R-2013.
- *Sand patch test*, in accordance with ASTM E965 (similar to EN 13036-1:2010) and EN 1766.
- *Mechanical profilometry*, in which a high-precision extensometer is moved over the entire surface to obtain a 3D map (with $x$, $y$ and $z$ coordinates) from which morphological parameters are computed.
- *Laser technique*, in which the superficial elevation (distance from the laser beam source to the object) of each point is calculated on the basis of the laser beam transit time.

- *Interferometric profilometry*, based on observation and analysis of the shadow produced by the superficial roughness of the surface (moiré fringe pattern principle).
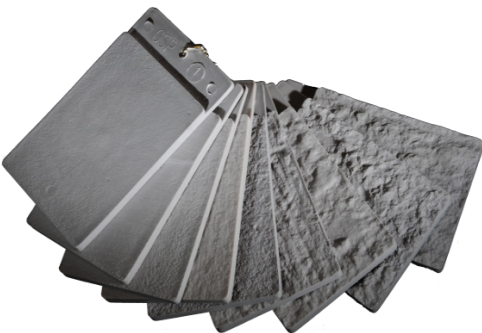
The aim of this task is to identify the most suitable techniques for both laboratory and field use, as well as the most relevant quantitative roughness characteristics [14].

# Surface profile characterization techniques

## Concrete surface profile (CSP)

The visual observation of surface roughness is the simplest evaluation method, but it is rather subjective. The systematic approach for visual surface qualification was proposed by ICRI (ACI 562 Repair Code). The reference replicates that make up the CSP (Table 2-2) represent concrete surfaces after typical surface treatments commonly used in the field: details are given in the ICRI Guideline No. 310.2R-2013. The range of evaluation is, however, limited to gentle surface treatments.

*Table 2-1: Concrete surface treatment methods and corresponding CSP's*

| Profile reference replicates | Surface preparation methods | CSP |
|---|---|---|
|  | Detergent scrubbing | 1 |
| | Low-pressure water cleaning | 1 |
| | Acid etching | 1–3 |
| | Grinding | 1–3 |
| | Abrasive blasting (sand) | 2–5 |
| | Steel shotblasting | 3–8 |
| | Scarifying | 4–9 |
| | Needle scaling | 5–8 |
| | Water jetting | 6–9 |
| | Scabbling | 7–9 |
| | Flame blasting | 8–9 |
| | Milling/rotomilling | 9 |

## Sand patch test

The sand patch tests described in ASTM E965 (very similar to EN 13036-1:2002) is one of the most commonly used methods for examining the macrotexture depth of concrete surfaces, mainly for road and airfield pavements. This method consists in careful application of a specific volume and grading of particles (glass spheres or sand) onto a surface and subsequent measurement of the total area covered (Figure 2-1).

*Figure 2-1: Measurement of surface macrotexture with the sand patch test procedure*

The surface roughness is characterized by the mean texture depth (MTD), calculated in accordance with Equation 2-1:

$$MTD = \frac{4\,V}{\pi\,D^2}\,[mm]$$
(2-1)

where:  $V$ = volume of granular material [mm$^3$]

$D$ = diameter of circle covered by granular material [mm]

A similar method for evaluating surface roughness is proposed in the European standard EN 1766:2000 in the case of concrete substrate preparation prior to repair. Silica sand with a 100/50 μm grading size is recommended for evaluation. The surface roughness index (SRI) is calculated in accordance with Equation 2-2:

$$SRI = \frac{V}{D^2}\cdot 1272$$
(2-2)

where the symbols are the same as in Equation 1-1; $V$ = 25 mL is recommended.

The advantages of the sand patch method are its speed, non-destructive character and applicability in situ; a disadvantage is that the surface has to be protected from wind and rain. The main limitations are the range of validity (from 0.25–5 mm only) and the fact that it can be used only on horizontal surfaces.

## Mechanical profilometry

In this method, deviations of the surface geometry are detected by a sensor (stylus) that moves along the surface [3, 4]. The gauge turns vertical deflections of the stylus position into electrical signals which are recorded by the computer, thus creating a surface profile (Figure 2-1a). It is possible to regulate the distance between measurement points for better precision [4]. The geometry (round or conical) and size (radius) of the extremity of the stylus are of prime significance for the profile to record: some profiles characterized by small wavelengths will not detected if the diamond cone radius is too large (Figure 2-2b, c).

Roughness measurements usually yield images of the profile. To analyze the influence of the treatment on the surface, it is necessary to mathematically and statistically quantify the shape of the surface by means of several parameters (Tables 2-3 and 2-4). Another approach is surfometry, a surface metrology of the profile rendered in 3D: in this case, the profilometer is used to obtain several profiles in parallel. The results are analyzed in two orthogonal directions (*x*,*y*) to generate a 3D representation of the surface (Figure 2-3). This method yields a quantification of the surface geometry, irrespective of its anisotropy [15].
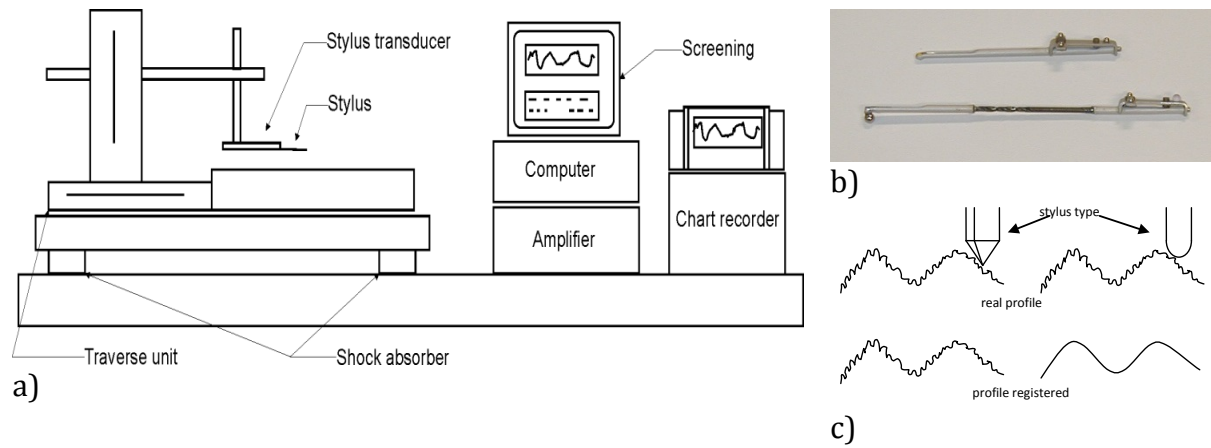
*Figure 2-2: Mechanical profilometer developed at the University of Liège – a) scheme of the measuring device; b) stylus used in concrete surface roughness evaluation; c) influence of the stylus geometry on the recorded profile*

*Table 2-2: Vertical amplitude parameters of surface profile as per EN ISO 4287*

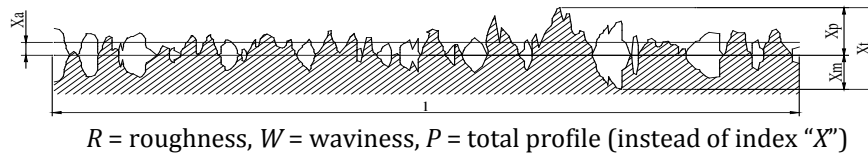| Symbol | Parameters | Definition |
|--------|-----------|------------|
| $m_x$ | Mean value and line | Line whose height (mean value) is determined by minimal sum square deviation of the profile defined as follows: $$X = \min \sum y^2(x)$$ |
| $X_p$ | Maximum peak height | Distance between the highest point of the profile and the mean line |
| $X_m$ | Minimum valley depth | Distance between the lowest point of the profile and the mean line |
| $X_t$ | Maximum height | Maximum distance between the lowest and the highest point of the profile and its equal: $$X_t = \max(X_p + X_m)$$ |
| $X_a$ $X_a'$ | Arithmetic mean deviation | Mean departure of the profile from the reference mean line as follows: $$X_a = \frac{1}{l}\int_0^l |y(x)|dx \qquad X_a' \approx \frac{1}{n}\sum_{i=1}^{n}|y_i|$$ , approximated by |
| $X_q$ | Root mean square deviation | Statistical nature parameter defined in the limits of the cut-off length as follows: $$X_q = \frac{1}{l}\int_0^l y^2(x)dx$$ |



$R$ = roughness, $W$ = waviness, $P$ = total profile (instead of index "$X$")

*Table 2-3: Horizontal amplitude parameters of surface profile as per EN ISO 4287*

| Symbol | Parameters | Definition |
|---|---|---|
| $S_k$ | Skewness of surface height distribution | A measure of asymmetry of profile deviations about the mean line, as follows: $S_k = \dfrac{1}{R_q^3} \dfrac{1}{n} \sum_{i=1}^{n} Y_i^3$ |
| $S_m$ | Mean period of profile roughness | Mean value of mean line consecutively including a peak and a valley $S_{mi}$, as follows: $S_m = \dfrac{1}{n} \sum_{i=1}^{n} S_{mi}$ |
| $n_p$ | Bearing length | Sum of partial lengths $n_i$ corresponding to the profile cut by a line parallel to the mean one for a given cutting level |
| $t_p$ | Bearing length ratio | Ratio between bearing length and cut-off length, expressed as a percentage: $t_p = n_p/l$. |



# Laser profilometry

The laser profilometry method essentially consists in laser travel measurement with an optical displacement sensor [10], as shown in Figure 2-4. The most recently developed laser profilometers are fast and accurate and allow the measurement of surface topography down to the sub-micrometer level over an area of 500 × 500 mm, in both 3D and 2D outputs. The technique is based on the principle of optical triangulation and requires a light source (commonly a diode laser), imaging optics and a photodetector. A diode laser is used for generating a collimated beam of light, which is then projected onto a target surface. A lens is focused on the spot of the laser light reflected onto a photodetector, which generates a signal that is proportional to the spot's position on the detector. As the target surface height changes (z), the image spot may shift due to the parallax. The sensor scans in two directions (x,y) to generate a 3D image of the surface of the element being characterized. Examples of concrete surface profiles generated through laser profilometry are presented in Figure 2-5.

| 3D representation | | |
|---|---|---|

a) Without treatment (NT)



b) Grinding (GR)



c) Sandblasting (SB)



d) Shotblasting 35s (SHB35)
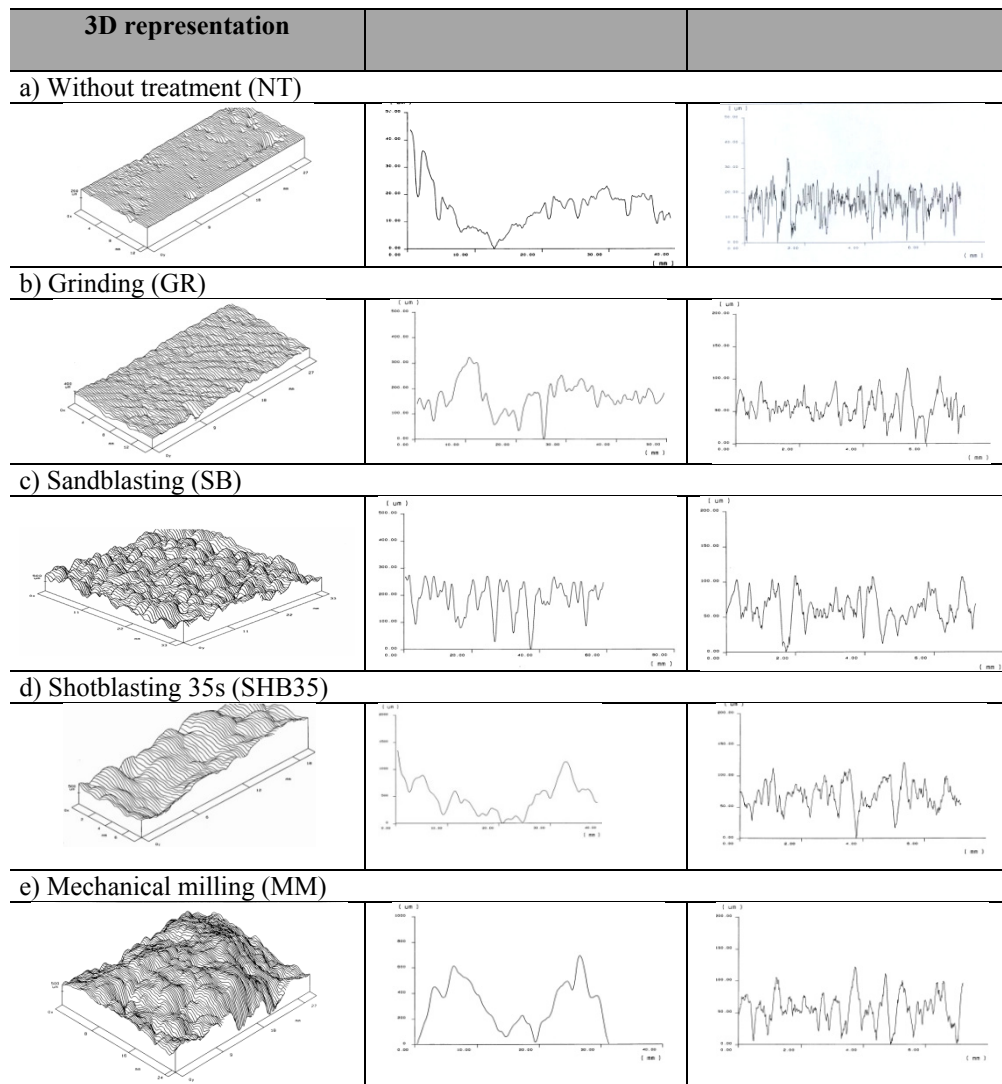


e) Mechanical milling (MM)



*Figure 2-3: Examples of 3D representation with surfometry, waviness and roughness profiles after different surface treatments, as determined through mechanical profilometry (University of Liège)*

The first applications of commercial laser profilometry were used to characterize surface geometry for tribology [11].  The technique has also been used to characterize concrete surfaces [8-11].

The recently developed circular track meter (CTM), in which a CCD laser displacement sensor is used, belongs to the same group of profilometers.  Eight individual segments are analyzed to investigate profile at different angles (0°, 45° and 90°) with respect to the traveling direction.  The CCD is mounted on an arm which is driven by a DC motor and rotates at 80 mm above the surface with a 142 mm radius.  The data are segmented into eight 111.5 mm arcs of 128 samples each.  The profile characterization data generated with CTM are the average profile depth and the root mean square (RMS).  More details can be found in the ASTM E2157-01 standard.
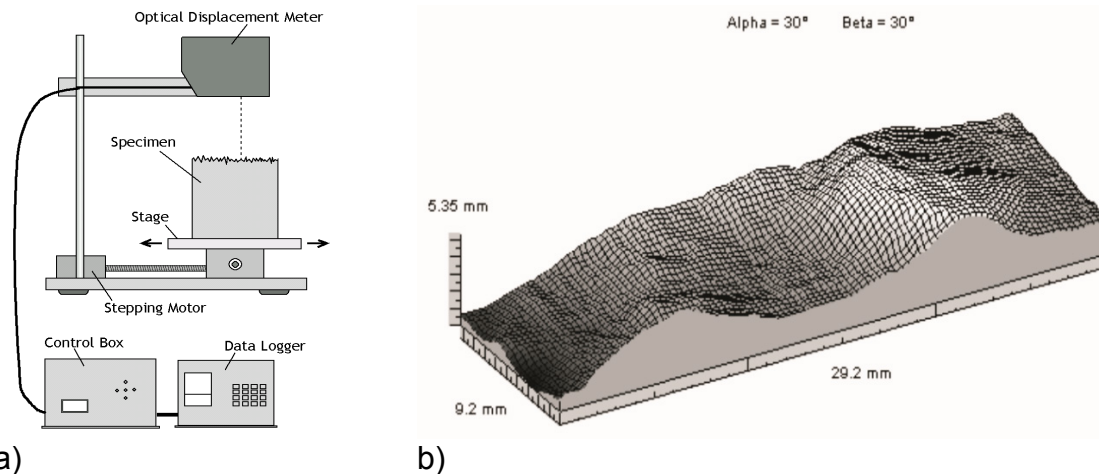
*Figure 2-4: Laser profilometry with optical displacement meter – a) experimental setup;*
*b) 3D representation of a shotblasted concrete substrate*
*(Warsaw University of Technology)*

## Interferometric surfometry

The various types of profilometers described above are mainly used in laboratory conditions. Recent studies have been devoted to the development and experimentation of optical devices (Figure 2-6a) which can be used in the field for civil engineering applications [6, 7, 14, 16]. Systems based on the moiré projection technique are exhibiting very interesting potential for that purpose. The moiré phenomenon appears when two networks of light rays, made of equidistant lines (alternately opaque and transparent) are superimposed. The technique of surface profile characterization is based on the measurement of a parallel fringe pattern from a deformed pattern projected on a non-plane surface (Figure 2-6b). The moiré fringes are similar to level lines representing the height variations of the object's surface. When a network of parallel fringes is projected onto a plane surface, it will not be deformed, but when projected onto an unspecified shape, this same network will be deformed. The main principle of the test is to compare two images with different moiré networks. The first image is the reference: it corresponds to the network of non-deformed parallel fringes. The second image contains the projected network deformed with respect to the non-plane shape. An algorithm analyzes the image and compares the grid of calibration and the deformed grid.

Maerz et al. [8] developed a portable concrete roughness testing device consisting of an optical laser-based imaging system that operates in accordance with the principles of the Schmaltz microscope and the shadow profilometry method. It uses a laser profiling line ("laser striping") that produces a non-Gaussian (i.e., uniform) distribution of light intensity along the line. The investigated concrete surface is illuminated with thin slits of red laser light at an angle of 45° while the observations are performed at 90° (Figure 2-1). A high-resolution (tiny) board CCD camera with a 7.5 mm lens is fixed vertically on the protection housing.

# Profile description

After treatment, concrete surfaces present fractal topography.  As with any fractal object, it is possible to break up this surface or this profile into a series of sub-profiles.  Each sub-profile can be differentiated in terms of wavelength: there are, however, no limits or precise criteria involved in validating the choice of decomposition method (Figure 2-8).  It is also possible to filter the results mathematically [5].  Since the two surfometry methods (mechanical and interferometry) have different resolution levels, it is possible to obtain complementary scales of topography.  The method using a mechanical stylus at high resolution yields roughness ($R$) and waviness ($W$) (Table 2-5) [4].  With the interferometry method at a resolution of 0.200-μm, it is possible to obtain two higher scales named meso-waviness ($M$) and shape ($F$).  In mechanical profilometry, filtering is often carried out through the use of stylus with different diameters.
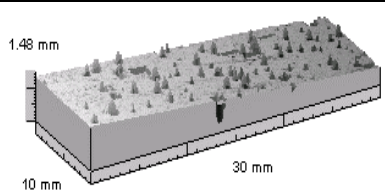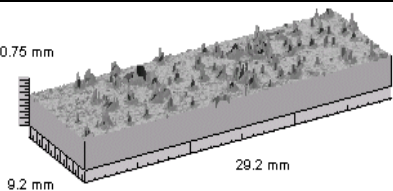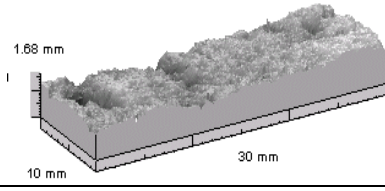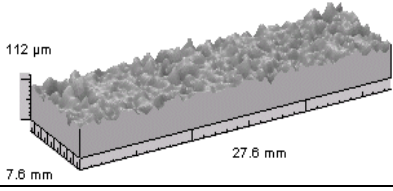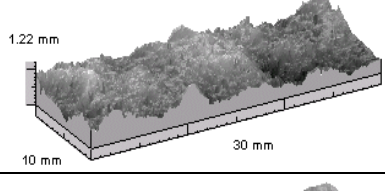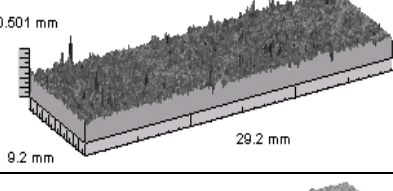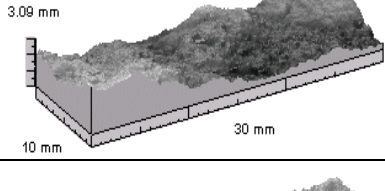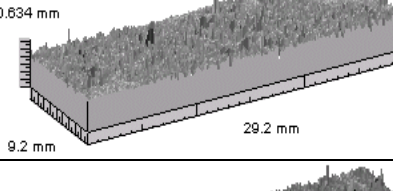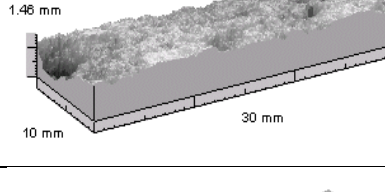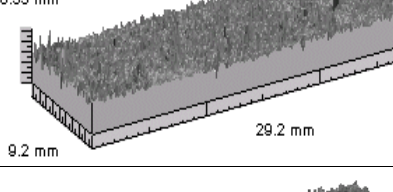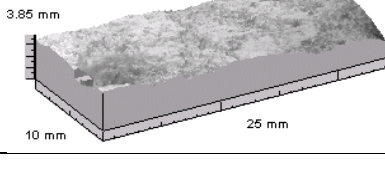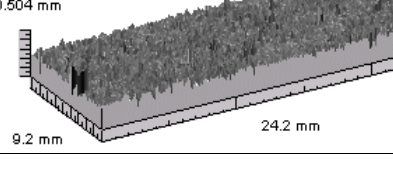
| Treatment | Waviness | Roughness |
|-----------|----------|-----------|
| As received, no treatment NT | 1.48 mm, 30 mm, 10 mm | 0.75 mm, 29.2 mm, 9.2 mm |
| Grinding GR | 1.68 mm, 30 mm, 10 mm | 112 µm, 27.6 mm, 7.6 mm |
| Sandblasting SB | 1.22 mm, 30 mm, 10 mm | 0.501 mm, 29.2 mm, 9.2 mm |
| Shot-blasting 35s SHB35 | 3.09 mm, 30 mm, 10 mm | 0.634 mm, 29.2 mm, 9.2 mm |
| Hand milling HMIL | 1.46 mm, 30 mm, 10 mm | 0.53 mm, 29.2 mm, 9.2 mm |
| Mechanical milling MMIL | 3.85 mm, 25 mm, 10 mm | 0.504 mm, 24.2 mm, 9.2 mm |

*Figure 2-1: Examples of 3D representation of specimen surfaces obtained by laser profilometry after waviness and roughness filtering*
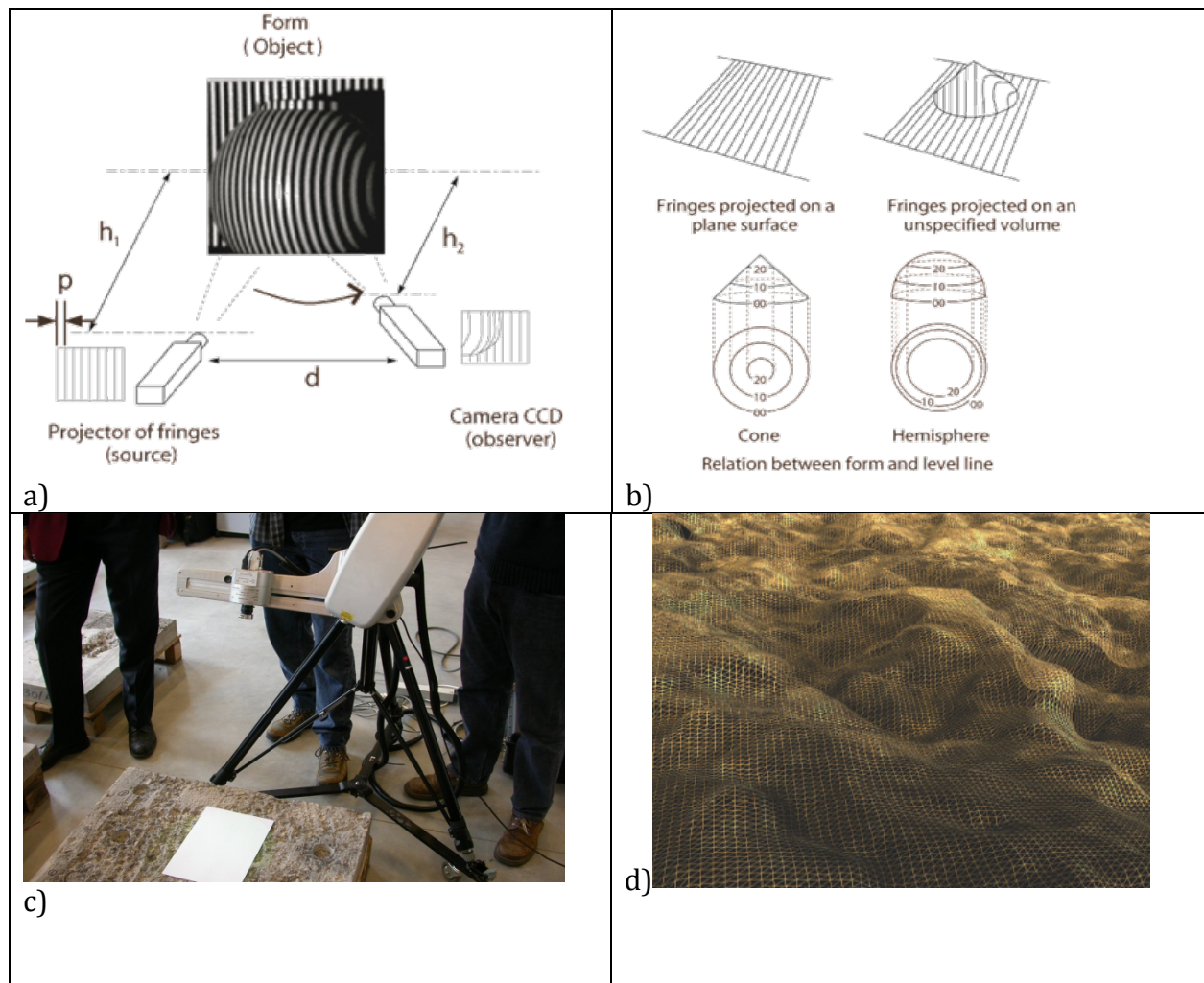
*Figure 2-2: Optomorphological profilometry – a) principle of measurement; b) relationship between form and level line; c) testing equipment; d) example of 3D representation of concrete surface (University of Liège)*
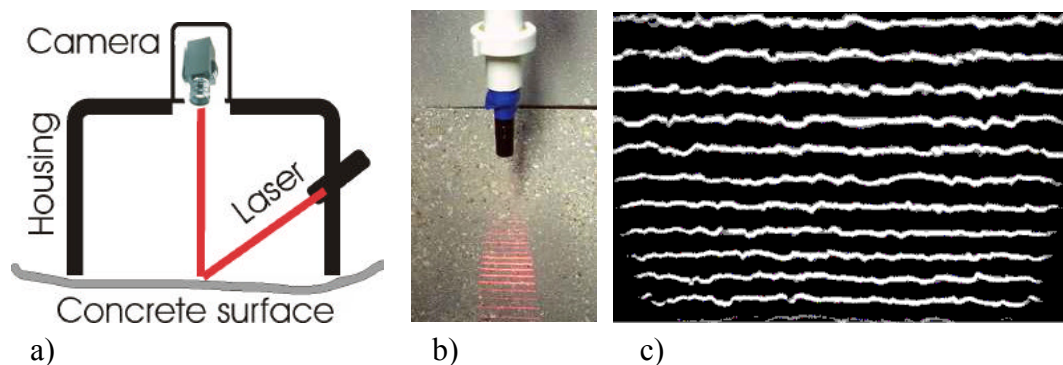


*Figure 2-3: Laser profilometry – a) schematic representation of the laser profiling testing device; b) line laser; c) laser image of a concrete surface*
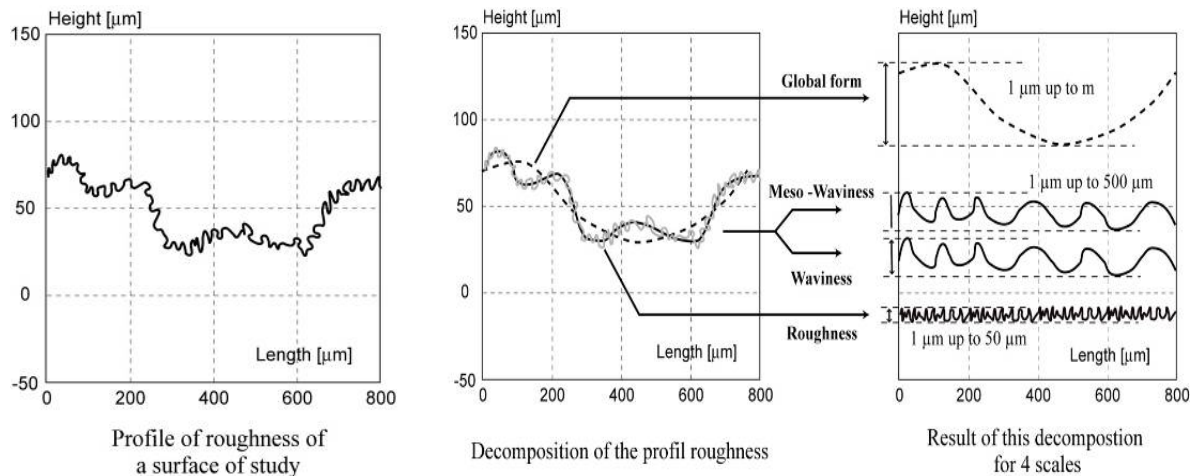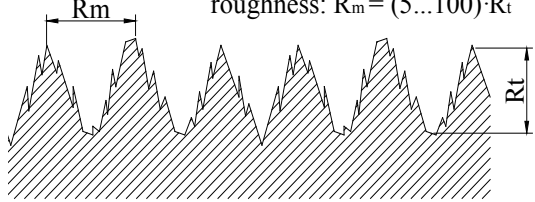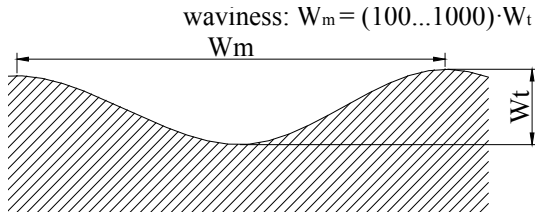
*Figure 2-4: Effect of scale on profile decomposition*

*Table 2-1: Surface profile make up*

| Roughness | High frequencies gap between grooves (amplitude) $R_m$ is 5–100 times the depth $R_t$ |  |
|---|---|---|
| Waviness | Mean frequencies amplitude $W_m$ is 100–1000 times the depth of holes $W_t$ |  |

In accordance with EN ISO 4287, the total (primary) profile, the waviness and roughness profiles can be characterized by several vertical (Table 2-3) and horizontal (Table 2-4) amplitude parameters. Surface parameters are determined on the basis of the mean line as a reference line: this reference is usually defined in such a way that, in the limits of the profile length, the sum of the squared values of the altitudes of the profile measured versus this reference line is minimal.

Using horizontal profile parameters, the Abbott curve, also referred to as the bearing curve [4], can be determined. This provides information about the surface profile: a gradual decrease in the curve suggests a surface with few holes, while a more steeply decreasing curve is characteristic of a surface with a lot of holes. Important parameters for analyzing the distribution of holes and peaks, as well as the shape of the profile can be graphically calculated from the Abbott curve (Table 2-6). These parameters are crucial when it comes to evaluating of the quantity of slurry, mortar, etc., needed for the interface area between the concrete substrate and the new layer (Figure 2-9).

Table 2-2: Abbott curve parameters

| Symbol | Parameters | Definition |
|--------|------------|------------|
| $C_R$ | Relative height of the peaks | Gives an idea of significance of the volume of very high peaks above the reference line |
| $C_F$ | Depth of the profile | Excluding high peaks and deep holes gives information on surface flatness; a lower value of $C_F$ indicates great surface flatness |
| $C_L$ | Relative depth of the holes | Gives an idea of the significance of the volume of voids under the reference line |
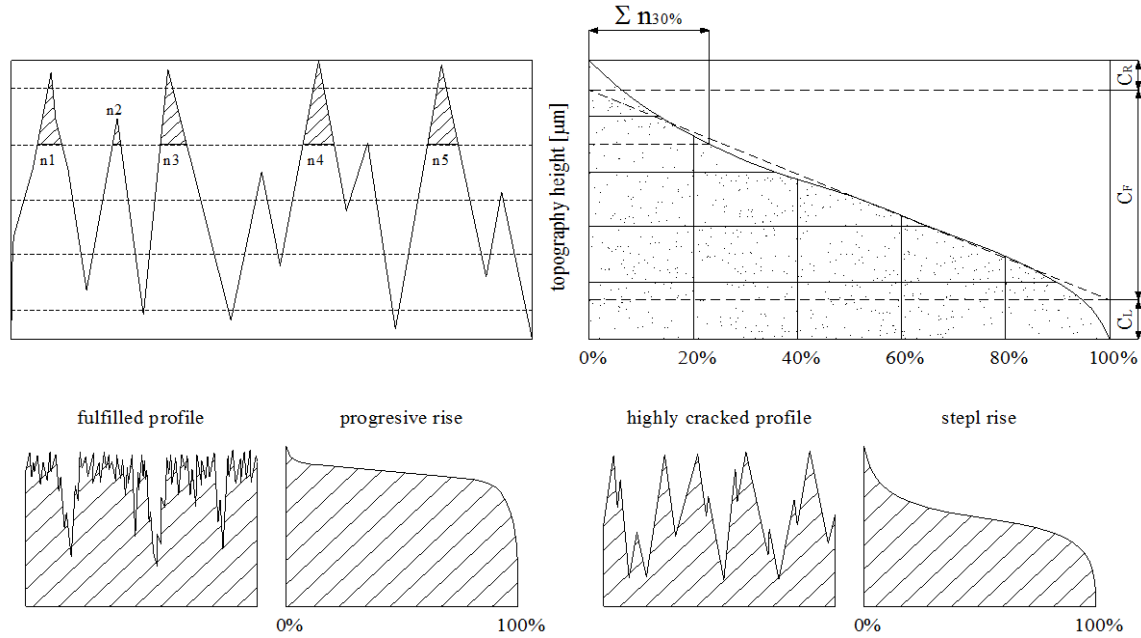


Figure 2-5: The Abbott curve and its interpretation

# Profile description with advanced methods

Concrete surface geometry can be characterized using a scientific approach called quantitative fractography, which is based on image analysis [15, 17]. This approach is well developed in the case of metals and ceramics in comparison with the situation prevailing for concrete-like composites [15–18]. However, geometrical and stereological parameters are also of significant importance in concrete-like composites [19–23]. Besides the profile parameters determined as per EN ISO 4287, three additional stereological parameters could be considered for characterization of concrete surfaces after surface treatment [15, 20]: the profile (linear) roughness ratio $R_L$, the surface roughness ratio $R_S$, and the fractal dimension $D$. The first two parameters are obtained with the following equations:

$$R_L = L/L_O \tag{2-3}$$
where:   $L$ = length of the profile line

$L_O$ = projected length of the profile line

$$R_S = S/S_O \tag{2-4}$$

where:     $S$ = true fracture surface area

                $S_O$ = apparent projected area

The fractal dimension $D$ was introduced in materials science by Mandelbrot as a characteristic allowing to define the irregularity of an object boundaries [22]. The basic requirement for the fractal boundary is that some structural feature(s) or unit(s) is (are) sequentially repeated at different levels. This implies that, from a statistical point of view, similar morphology can be observed in a wide fracture surface magnification range: a measure of this self-similarity is a fractal dimension.

These stereological parameters can either be determined from the image of the profile recorded on the specimen cross-section or the profile obtained by profilometry. The geometry of the fracture surface of concrete-like composites is related to the scale of observation. This implies that the self-similarity of the fracture surface may not extend over all magnification ranges.

Due to the inherent technical difficulties in evaluating $R_S$, examination of cross-section profiles is frequently opted for, and the profile roughness ratio $R_L$ is calculated using Equation 2-3 (Figure 2-11a). Recent developments in stereological methods allow users to estimate $R_S$ from fracture profile studies without simplifying assumptions concerning the relationship between $R_L$ and $R_S$. The surface roughness ratio $R_S$ can be effectively evaluated using a vertical sectioning method [15]. In this method, an arbitrary axis is chosen, and the specimen is saw cut parallel to this axis (Figure 2-10). It has been shown that sections sampled on three saw cut planes forming an angle of 120° around the axis are sufficient to characterize the surface profile and evaluate satisfactorily $R_S$ [24]. Wojnar [17] proposed a procedure to evaluate $R_S$ which consists of counting the intersection points of the fracture profile with a so-called grid of cycloids (Figure 2-11b). The cycloids allow users to relate the fracture area directly to the fracture profile, and the estimation of $R_S$ is independent of the magnification.

# Experimental comparison and analysis of the techniques

## Concrete substrates and surface treatments

Several concrete substrates with different compressive strengths and prepared with a variety of surface treatment were characterized at Laval University, the University of Liège and the Warsaw University of Technology. The series of experiments may be divided into three groups covering a wide range of concrete strength values and types of surface preparation. The basic mixture design characteristics and average compressive strengths of the tested concretes are summarized in Table 2-7.
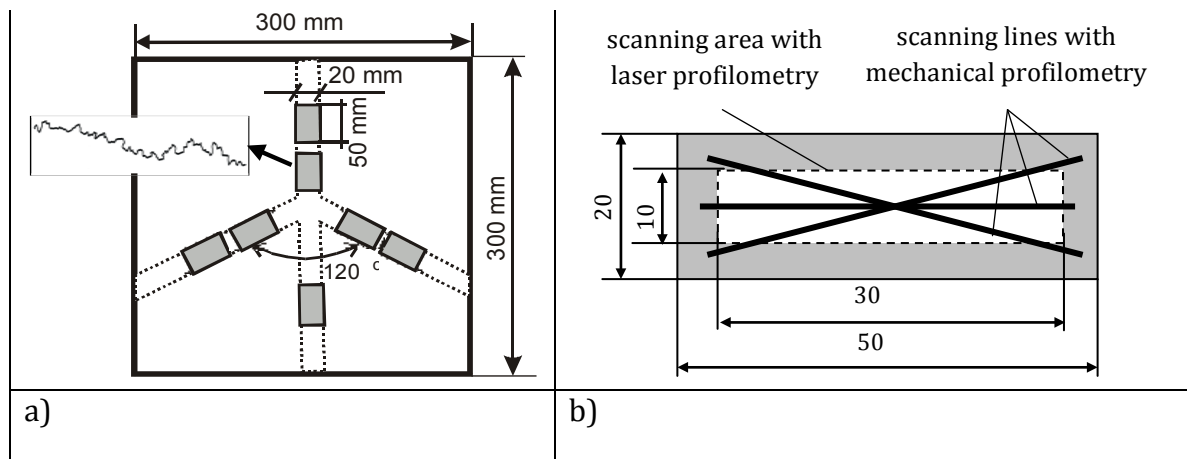
*Figure 2-1: Illustration of a) sampling for microscopic observation; and b) surface geometry characterization with laser and mechanical profilometry*



a)                                                                          b)
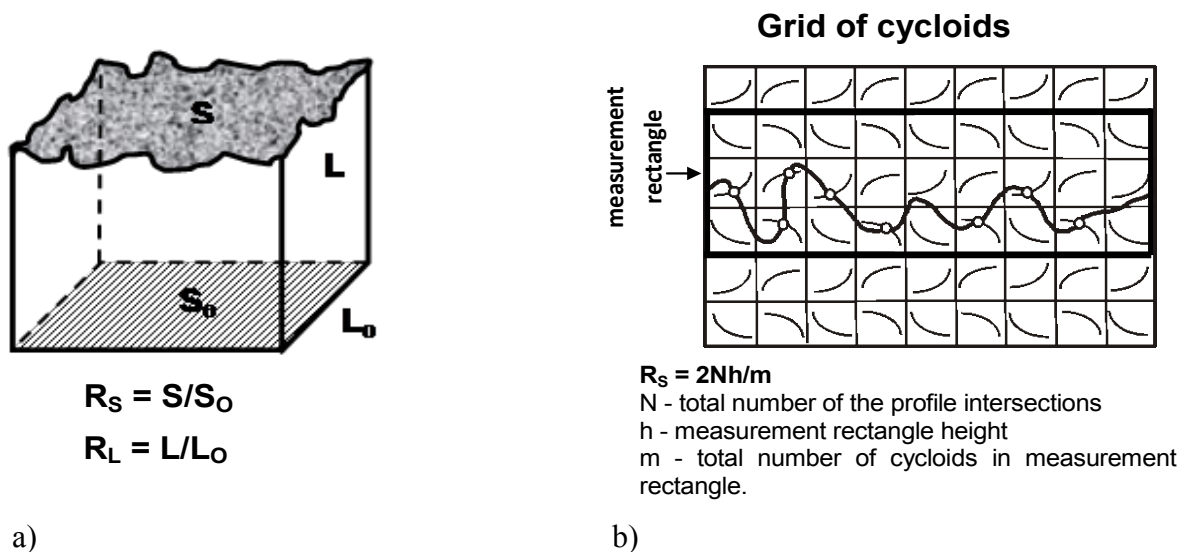
*Figure 2-2: a) Significance of the $R_S$ and $R_L$ parameters; b) evaluation of the $R_S$ value using grid of cycloids*

*Table 2-1: Composition and compressive strength of tested concrete substrates*

| Constituent / comp. strength | Group A | Group B | | | Group C | | |
|---|---|---|---|---|---|---|---|
| | C20A | C25B | C35B | C50B | C30C | C40C | C45C |
| Cement | CEM I 32.5 R | CEM I 32.5 R | | | CEM I 52.5 N | | |
| Sand | River sand (0/2) | River sand (0/2) | | | S-RRWSC-7 (0/2) | | |
| Aggregate | Gravel 2/8 | Gravel 2/8, 8/16 | | | Crushed limestone 2/8, 8/14, 14/20 | | |
| W/C | 0.60 | 0.54 | 0.42 | 0.31 | 0.72 | 0.59 | 0.50 |
| Plasticizer | no | yes | | | no | | |
| $f_{c\ 28-d}$ (MPa) | 22.0 | 31.5 | 45.7 | 62.1 | 35.0 | 41.3 | 48.8 |

For each series of concrete slabs, four different surface preparation methods were used.  In order to yield differences in profile roughness and the level of microcracking in the near-to-surface layer, the following surface treatment methods were selected in each group:

- Group A: Grinding (GR), sandblasting (SB), shotblasting (SHB20, SHB35 and SHB45, with treatment times of 20, 35, and 45 seconds, respectively), hand milling (HMIL) and mechanical milling (MMIL); untreated concrete samples (NT) were also tested as a control;
- Group B:  Polishing (PL), dry sandblasting (SB-D), jack hammering (JH) and water jetting at 250 MPa pressure (HD);
- Group C: Gentle surface preparation methods were used to obtain profiles of similar amplitude and low-level microcracking: brushing (NT), wet sandblasting (SB-W), scarifying (SC) and water jetting at 12 MPa pressure (LC).

# Evaluation of concrete surface texture with sand patch test

European standards EN 1766:2000 and EN 13036-2010 state that the validity of this measurement ranges from 0.25 to 5.0 mm.  The results for the three groups are presented in Table 2-8.
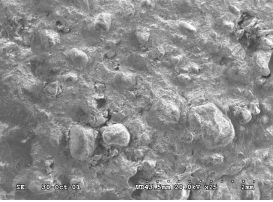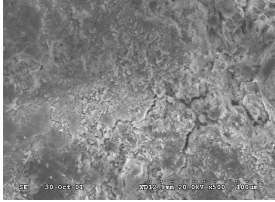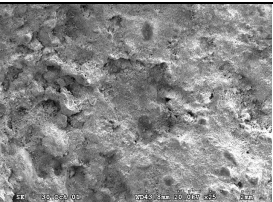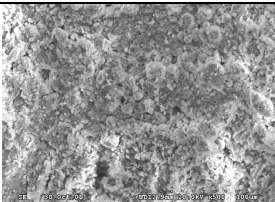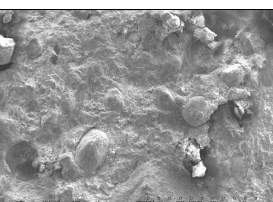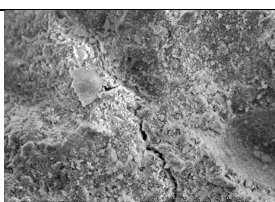
The results for Group A clearly show the significant effect of treatment aggressiveness on surface roughness of the concrete substrate [27].  The SEM images and qualitative descriptions are presented in Table 2-9.  The roughest surface treatments are shotblasting and sandblasting. Aggressiveness of the treatment strongly influences the quality of the near-to surface layer, as it governs the extent of induced microcracking [28].

*Table 2-2: Results of the sand patch test for the various surface treatment techniques*

| Group A slabs | | | | | |
|---|---|---|---|---|---|
| **ID** | **Surface treatment** | **SRI (mm)** | **ID** | **Surface treatment** | **SRI (mm)** |
| C20-NT | No treatment | 0.55 | C20-SHB35 | Shotblasting 35 s | 1.59 |
| C20-GR | Grinding | 0.72 | C20-SHB45 | Shotblasting 45 s | 1.85 |
| C20-SB | Sandblasting | 1.40 | C20-HMIL | Hand milling | 0.79 |
| C20-SHB20 | Shotblasting 20 s | 1.01 | C20-MMIL | Mechanical milling | 1.05 |
| **Group B slabs** | | | **Group C slabs** | | |
| **ID** | **Surface treatment** | **SRI (mm)** | **ID** | **Surface treatment** | **SRI (mm)** |
| C30-PL | | 0.25 | C25-LC | | 0.37 |
| C40-PL | Grinding | 0.20 | C35-LC | Water jetting 12 MPa | 0.39 |
| C45-PL | | 0.14 | C50-LC | | 0.16 |
| C30-SB-D | | 0.29 | C25-BR | | 0.39 |
| C40-SB-D | Dry sandblasting | 0.28 | C35-BR | Brushing | 0.39 |
| C45-SB-D | | 0.31 | C50-BR | | 0.41 |
| C30-SCA | | 0.89 | C25-SB-W | | 0.50 |
| C40-SCA | Scabbling | 0.89 | C35-SB-W | Wet sandblasting | 0.61 |
| C45-SCA | | 0.80 | C50-SB-W | | 0.41 |
| C30-HD | | 2.22 | C25-SC | | 0.66 |
| C40-HD | Water jetting 250 MPa | 5.00 | C35-SC | Scarifying | 0.88 |
| C45-HD | | 3.20 | C50-SC | | 0.50 |

In general, it is found that a given treatment induces lower roughness as the strength of the substrate concrete increases.  Besides, the results for the polished slabs C40-P and C50-P are beyond the range of validity of the SPT, while the results for slabs C30-P and C40-HD (water jetting) fall just within.  The surface obtained with water jetting is also very irregular, and the results are characterized by high coefficients of variation.  Overall, water jetting yielded the roughest profiles, followed by scabbling, sandblasting and polishing.  In Group C, it can be seen that the test specimen surfaces exhibit little differences in profile roughness, essentially because treatments were overall less aggressive than in the two other groups.

*Table 2-3: SEM observations of concrete surfaces after various treatments*

| | |
|---|---|
| **Grinding** | |
|  | Surface without sharp edges with few and non-uniformly located valleys at the surface; narrow cracks observed at higher magnifications. |
| **Sandblasting** | |
|  | Surface similar to that after grinding; shallow irregularities of surface (peak-to-valley height did not exceed 1 mm); sharp edges of aggregate particles and microcracks observed at higher magnifications, very often forming non-uniform networks. |
| **Shotblasting** | |
|  | Highest roughness of surface increasing with the treatment time; high irregularities of surface (peak-to-valley height increased locally to 7 mm for 45 seconds); formation of a dense network of microcracks and cracks, often along aggregate particles, as well as presence of deteriorated or removed particles were observed with increased treatment time. |
| **Milling** | |
|  | Surfaces after milling similar and close to the concrete surface after shotblasting; very high irregularity of the surface, but lower than that after shotblasting; at higher magnifications deep and wide cracks, signs of particle removal and loosed concrete fragments were observed. |

Further, the repeatability of the sand patch test and the effect of the volume of sand were analyzed. The tests were performed on four specimens of each concrete mixture from Group C. Comparison of the SRI values yielded when using 10 and 25 mL of sand, respectively, shows a very strong correlation (r = 0.95). However, the SRI values determined with a volume of sand of 25 mL were systematically 6% higher (Figure 2-12).
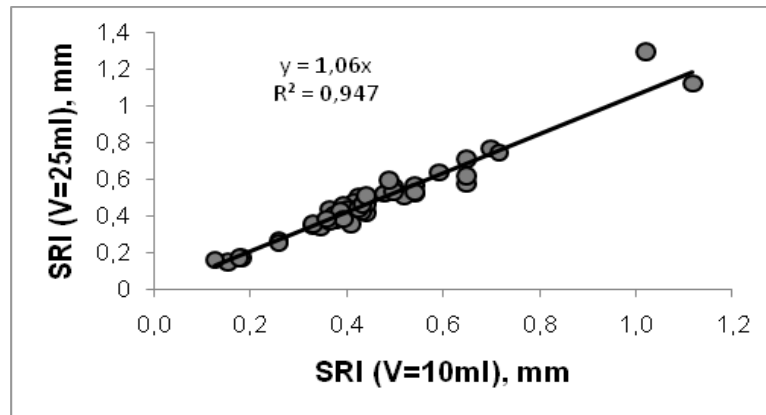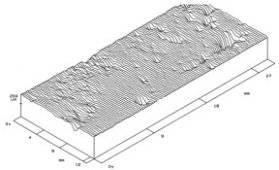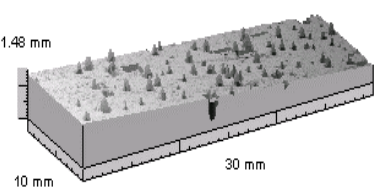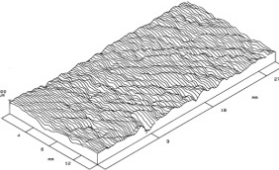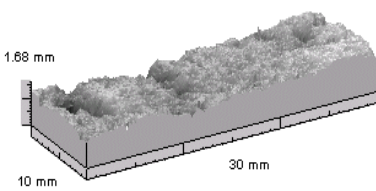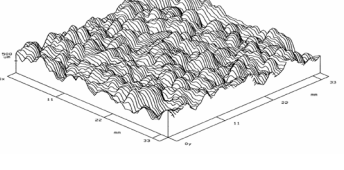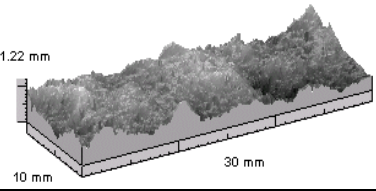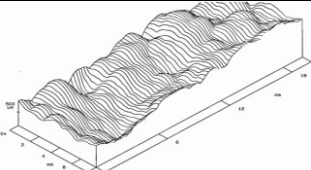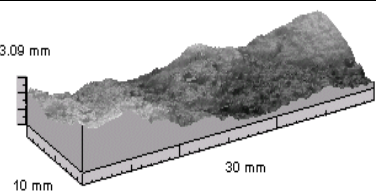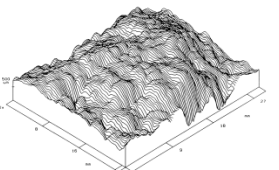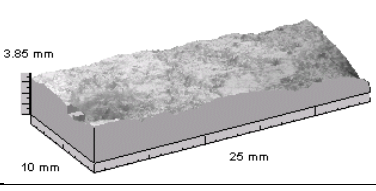


*Figure 2-3: Influence of the sand volume used in the sand patch test*

## Mechanical vs. laser profilometry

A comparative study of surface roughness characterization with laser and mechanical profilometers was conducted using concrete specimens from the Group A series. Sampling was performed in accordance with the aforementioned vertical sectioning method (Figure 2-10). In the case of mechanical profilometry, the surface was scanned along three lines with a length of 30–40 mm; for laser profilometry, an area of $10 \times 30$ mm was scanned along parallel lines with a distance of 50 μm between the subsequent lines [12, 14]. Examples of surface roughness profiles are presented in Table 2-10.

The recorded profile was first transformed to remove the effect of the profile orientation ("shape" filtering) [6]. The total profile obtained was then filtered and decomposed into low and high frequencies to separate parameters of waviness and roughness, respectively. The filter used to separate waviness from the total profile was selected to be 0.8 mm for both methods. The total height of the profile $X_t$, the arithmetic mean of the deviations of the profile from the mean line $X_a$, and the maximum depth of valleys $X_v$, were selected for the surface geometry characterization for all levels of filtering [6], i.e., for the total ($X = P$), waviness ($X = W$) and roughness ($X = R$) profiles. The Abbott curve parameters were also calculated. Further in the text, indexes $p$ and $s$ denote parameters measured by mechanical and laser profilometry respectively.

*Table 2-4: The examples for concrete surface roughness representation with mechanical (left) and laser (right) profilometry*

| Treatment | Waviness | Roughness |
|---|---|---|
| As received, no treatment NT |  |  |
| Grinding GR |  |  |
| Sandblasting SB |  |  |
| Shotblasting 35 s SHB35 |  |  |
| Mechanical milling MMIL |  |  |

The results of surface geometry characterization with the four methods can be summarized as illustrated in Figure 2-13. The geometrical parameters determined for the microscopic level (profile amplitude parameters) generally indicate that the highest roughness was obtained for shotblasting after 45 seconds and the lowest for grinding (Figure 2-13). With respect to profilometry methods, the waviness parameters are lower than those of the total profile by about 5% for mechanical profilometry and by 9% for laser profilometry. This confirms that the overall shape of the profile has been preserved through the waviness filtering process.

a)



b)



c)



d)
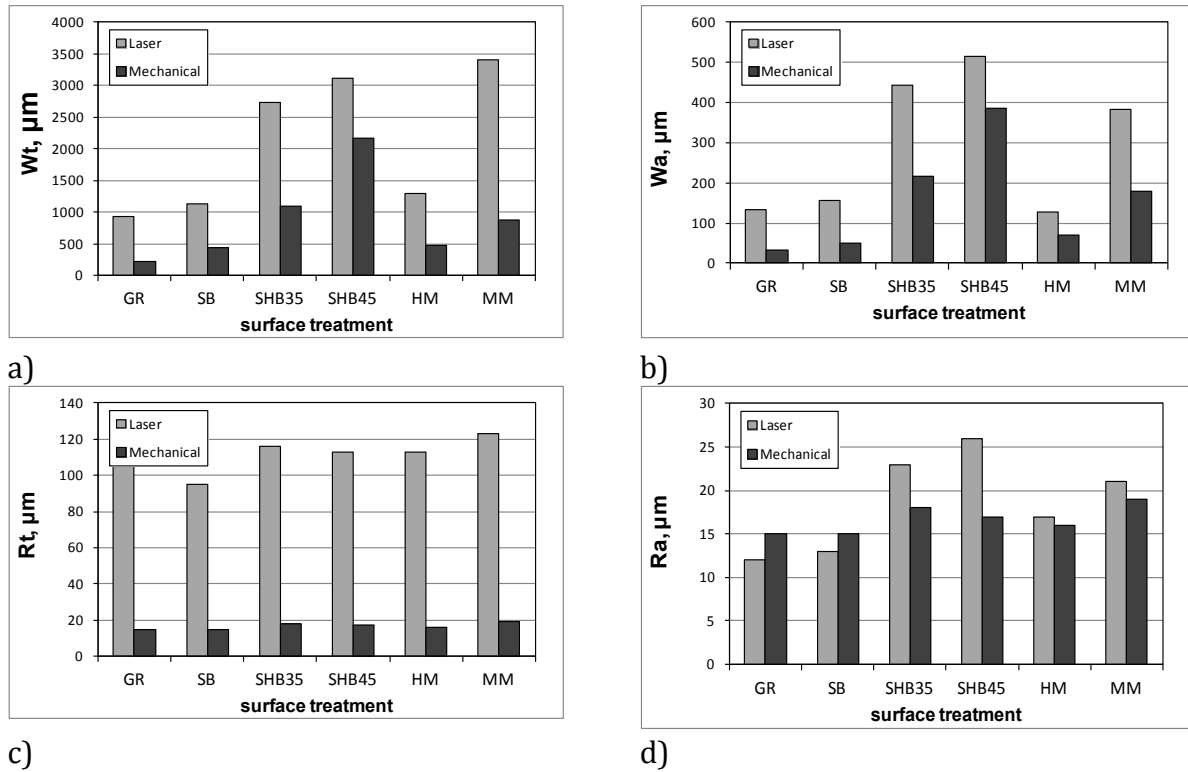
*Figure 2-4: Waviness parameters a) $W_t$ and b) $W_a$; microroughness parameters c) $R_t$ and d) $R_a$; as determined through laser and mechanical profilometry*

The mean roughness values are relatively close to each other, irrespective of the treatment type and the profilometry method being used ($R_{ap} = 17 \pm 2$ and $R_{as} = 19 \pm 7$). However, with the same filtering technique, the total height of the roughness profile determined through laser profilometry was 2.8–5.5 times higher than that obtained with mechanical profilometry. Hence, roughness parameters cannot strictly be used as surface quality indicators after treatment (Figure 2-13c, d).

Both the total height and the mean value of the waviness profile measured with laser profilometry are of the order of 1.3 to 4.3 times higher than those calculated through the mechanical method and 7 times higher in terms of Abbott parameters.

The relationships between parameters determined with laser and mechanical profilometry show different levels of statistical significance. Nevertheless, the results indicate that the surface profile characteristics determined with both methods are comparable, irrespective of the observation level. This observation is confirmed by the high correlation coefficient ($r > 0.94$) of the relationship between the corresponding mean values of waviness profile $W_a$ (Figure 2-14a), and Abbott parameters $C_R$ and $C_F$ (Figure 2-14c). A higher scatter in the results for both profilometry methods was observed for other amplitude parameters. Lower statistical significance (Figure 2-14b) was obtained for the total heights of the waviness profile ($W_{ts}$ vs. $W_{tp}$) and the maximum depth of the valleys ($W_{vs}$ vs. $W_{vp}$) as well as the relative depth of holes, $C_L$ (Figure 2-14c). This could be because of differences in the surface area scanned with laser and mechanical profilometry. However, Figures 2-14b and c indicate that the low correlation is due to the low values of amplitude parameters obtained though mechanical profilometry for the surface obtained with mechanical milling. This surface was too irregular and had a significant

number of deep and wide cracks (Table 2-9), which are better detected by the laser profilometer than by the stylus.

The relationship between $W_a$ and SRI exhibits relatively high correlation coefficients ($r$), with values of 0.77 and 0.94 for laser and mechanical profilometry, respectively (Figure 2-14d). This confirms that SRI provides a satisfactory estimate of the mean deviation of a concrete surface profile and that it can be used for field evaluation of surface roughness.



a)

b)

c)

d)
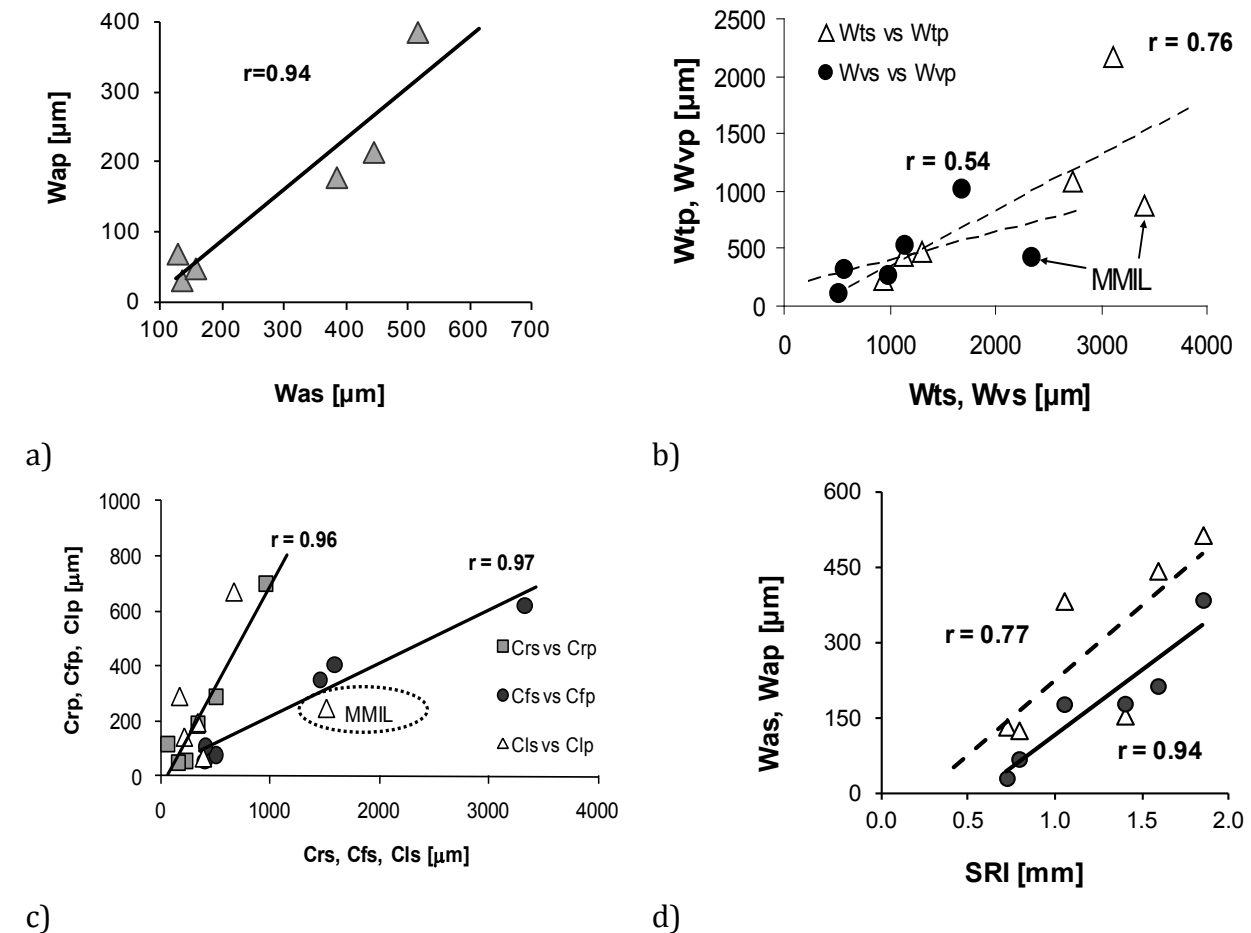
*Figure 2-5: Relationships between waviness parameters a) $W_a$, b) $W_t$ and $W_v$ and c) Abbott parameters, determined through laser ($\triangle$) and mechanical ($\bullet$) profilometry; d) relationship of $W_{as}$ and $W_{ap}$ vs. SRI*

# Surface roughness characterization with the microscopic observation method

Microscopic observations were performed on 20 × 50 mm samples that were sawn from 300 × 300 mm Group A test specimens (Figure 2-10a). The geometric length of the profile was 350 mm for each substrate type. The values of the stereological parameters $R_L$ and $R_S$ were calculated using the computer program *Profile 1.1* [24] for the profile images recorded with a light microscope at 10× magnification. The fractal dimension $D_b$ was calculated with the same program using the box-counting method [25]. The results are summarized in Table 2-11. It can be seen that among the different types of surface preparation methods investigated, hand milling resulted in the highest profile variability. Besides, the values of $D_b$ are not found to vary

significantly from one method to the other and are comparable to those reported elsewhere for a wide range of concrete surfaces ($D_b$ = 1.03–1.25) [20–22, 26, 27].  Unfortunately, it means that the fractal dimension is not sensitive enough for evaluating concrete surface profile characteristics in a discriminating fashion.

*Table 2-5: Stereological parameters evaluated with the microscopic method for various types of surface preparation*

| Surface preparation | Statistical parameter | Stereological parameters | | |
|---|---|---|---|---|
| | | $R_L$ | $R_S$ | $D_b$ |
| NT | Mean (mm) | 1.477 | 1.739 | 1.089 |
| | STD (mm) | 0.077 | 0.116 | 0.038 |
| | CV (%) | 0.052 | 0.067 | 0.035 |
| GR | Mean (mm) | 1.451 | 1.703 | 1.110 |
| | STD (mm) | 0.082 | 0.125 | 0.071 |
| | CV (%) | 0.057 | 0.073 | 0.064 |
| SB | Mean (mm) | 1.554 | 1.837 | 1.139 |
| | STD (mm) | 0.127 | 0.262 | 0.041 |
| | CV (%) | 0.082 | 0.142 | 0.036 |
| SHB20 | Mean (mm) | 1.563 | 1.870 | 1.104 |
| | STD (mm) | 0.116 | 0.171 | 0.035 |
| | CV (%) | 0.074 | 0.091 | 0.032 |
| SHB45 | Mean (mm) | 1.578 | 1.892 | 1.084 |
| | STD (mm) | 0.180 | 0.262 | 0.038 |
| | CV (%) | 0.114 | 0.138 | 0.035 |
| HMIL | Mean (mm) | 1.475 | 1.682 | 1.085 |
| | STD (mm) | 0.099 | 0.345 | 0.043 |
| | CV (%) | 0.067 | 0.205 | 0.040 |
| MMIL | Mean (mm) | 1.503 | 1.779 | 1.094 |
| | STD (mm) | 0.099 | 0.148 | 0.042 |
| | CV (%) | 0.066 | 0.083 | 0.039 |

(all data in the table represents the average results for three specimens)

The analysis of the relationship between $R_s$ and other parameters of profile characterization showed that $R_s$ correlates strongly with SRI (high correlation coefficient $r$ = 0.97; Figure 2-15a). The relationships between $R_s$ and $W_{as}$ and $W_{ap}$ exhibit a weaker correlation ($r \sim 0.8$; Figure 2-15b).
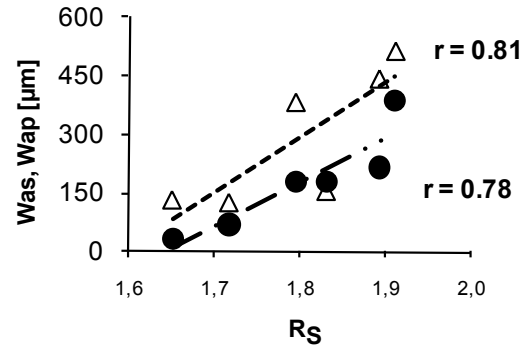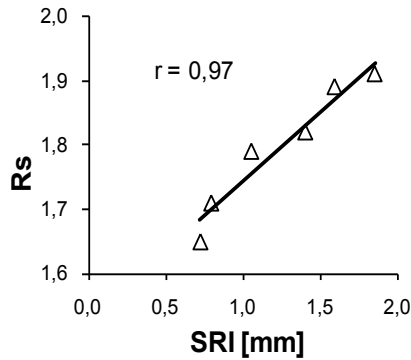
*Figure 2-6: Relationships a) $R_S$ vs. SRI and b) $R_S$ vs. arithmetic mean deviation of waviness profile, as determined through laser ($W_{as}$) and mechanical ($W_{ap}$) profilometry*

# Concrete surface texture evaluated with mechanical and interferometric methods

In a test comparison between mechanical and interferometric surface characterization approaches, three types of surface preparation techniques were investigated (Table 2-12): scarifying, high pressure water jetting (18,000 psi pressure and 6 gal./h water flow) and polishing. The latter was carried out using two abrasive rotating plates until the surface became smooth to the touch.

Figure 2-16 shows photographs of the respective profiles obtained with the surface preparation techniques investigated. A careful visual examination of all prepared surfaces leads to the following observations with respect to their macroscopic and visible effects:

- Polishing produces a very smooth surface with brightness close to that of a mirror;
- The high-pressure water jetting technique induces a particular texture characterized by large waves mostly parallel to the water flow;
- Scarifying generally induces some oriented macroroughness (grooved surface); in this study it was, however, intentionally eliminated by the operator by means of successive transverse and perpendicular operations.

*Table 2-6: Surface preparation of specimens tested through interferometric and mechanical profilometry*

| Reference | | |
|-----------|---|---|
| PTW | 150 L (no. 4) | Polished troweled surface |
| HPW | 150 L (no. 5) | High pressure water jetting |
| SC2 | 150 L (no. 6) | Scarifying |



a) polishing          b) water jetting          c) scarifying

*Figure 2-7: Concrete surface profiles obtained with selected surface preparation methods*

A first series of mechanical profilometry measurements was performed using a stylus with a diamond sphere radius of 6 µm (Figure 2-2).  The length of measurement was 8 mm and the filter used to separate roughness from the profile was set at 0.8 mm.  Three profiles were recorded on one sample of each type of preparation, in different directions.  A second series was carried out using a stylus with a length of 79 mm and a diamond with a radius of 1.5 mm in order to evaluate the waviness.  The length of the measurement was increased to 30 mm or more.  The filter was again set at 0.8 mm, and a 16-mm filter (twice the nominal size of the aggregates) was used to extract the shape from the profile.  The data summarized in Table 2-13 show that the $R_a$, $R_q$ and $R_t$ values are 1.5–3 times smaller for the polished concrete profile than those obtained with water jetting and scarifying, while the amplitude and statistical roughness value are relatively close for water jetting and scarifying.

These findings confirm that the surface treatment technique (Figure 2-17) has no major influence on the microroughness ("high-frequency waves") of the profile.  Furthermore, it demonstrates once again that waviness parameters are sufficient to define concrete surface roughness.

As described previously, the optical method based on the moiré pattern is an interferometric technique used to obtain 3D profile information based on the interference of light and shade stripes [5].  The measurement accuracy is directly related to the density of the fringe network and the capacity of differentiation of the network by image analysis.  Theoretically, with a light beam projection angle of 45° and a 512 × 512 pixel CCD camera, a resolution of approximately 1/5,000 of the size of the object can be obtained.  In the present application, for a 350 × 350 mm surface area, the 3D resolution reached 200 µm, with a measurable maximum vertical amplitude of the order of 100 mm.

Three interferometric topography evaluations were carried out. Figure 2-18 shows the equipment used for optical measurement, which can be performed on the actual surface of the specimen or member, irrespective of its size, without the need for sampling.  At this scale, water jetting seems to induce the roughest profile, while polishing and scarifying yielded smoother and rather similar profiles.  This is due to the bubble effect at the surface, which increases roughness.  It can be seen in Table 2-14 that the roughness amplitude value ($M_a$) yielded with water jetting is 20 times that for scarifying and polishing.  At this scale, the other treatments left rather smooth surfaces, polishing generating the flattest profile.  Most of the apparent roughness of polished surfaces comes from the bubble holes.

*Table 2-7: Surface profile characteristics determined through mechanical profilometry – waviness (W), roughness (R) and Abbot parameters (C)*

| Surface profile parameter | | Polishing | Water jetting | Scarifying |
|---|---|---|---|---|
| $W_a$ | (mm) | 6 | 420 | 127 |
| $W_p$ | (mm) | 13 | 1003 | 346 |
| $W_q$ | (mm) | 9 | 501 | 158 |
| $W_v$ | (mm) | 47 | 923 | 445 |
| $W_t$ | (mm) | 60 | 1926 | 791 |
| $R_a$ | (mm) | 5 | 14 | 15 |
| $R_q$ | (mm) | 7 | 17 | 19 |
| $R_t$ | (mm) | 70 | 96 | 102 |
| $C_R$ | (mm) | 4 | 152 | 412 |
| $C_F$ | (mm) | 10 | 228 | 827 |
| $C_L$ | (mm) | 14 | 231 | 537 |

| a) Polishing | b) Water jetting | c) Grinding |
| --- | --- | --- |

**Surface scan**
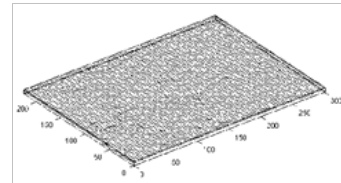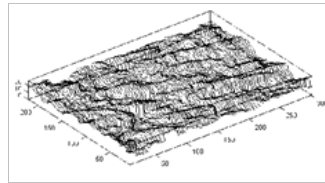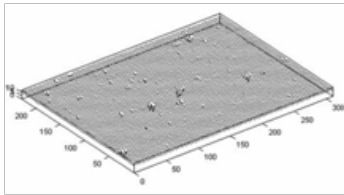


**Roughness profile**
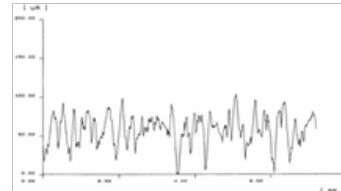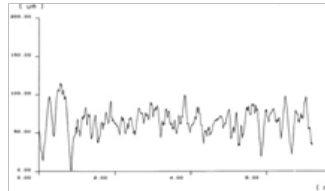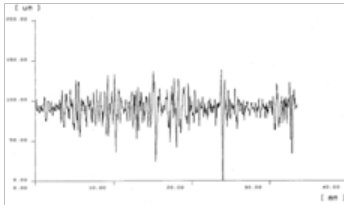


**Waviness profile**
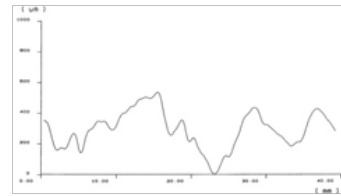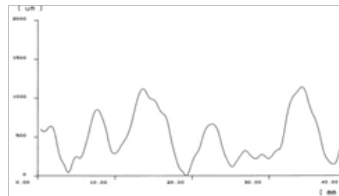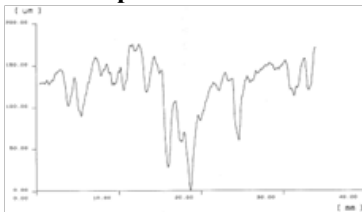


*Figure 2-8: 3D representation of surfaces and corresponding roughness and waviness profiles for three different types of treatment*



*Figure 2-9: Use of an interferometric measuring device for concrete surface characterization*

Table 2-8: Surface profile characteristics determined through interferometric profilometry – overall shape (F), meso-waviness (M) and Abbot parameters (C)

| Surface profile parameter | | Polishing | Water jetting | Scarifying |
|---|---|---|---|---|
| $F_a$ | (mm) | 0.137 | 0.358 | 0.326 |
| $F_t$ | (mm) | 4.1 | 10.8 | 12.6 |
| $F\,S_m$ | (mm) | 129 | 85.3 | 102.3 |
| $M_a$ | (mm) | 0.169 | 2.85 | 0.315 |
| $M_t$ | (mm) | 19.7 | 27.8 | 10.2 |
| $M\,S_m$ | (mm) | 15.3 | 36.5 | 22.5 |
| $C_R$ | (mm) | 0.30 | 4.65 | 0.41 |
| $C_F$ | (mm) | 0.29 | 5.76 | 0.55 |
| $C_L$ | (mm) | 0.35 | 5.71 | 0.81 |

## CSP profiles vs. interferometric measurements

The aim of this part of the the investigation was to compare the surface geometry characteristics obtained with optical profilometry and visual method (CSP profiles). The nine CSP plates (Figure 2-19a) have been characterized at Laval University with the optometric method [5] using a 512 × 512 pixel CCD camera, a vertical resolution of 200 µm and a surface area of 350 × 350 mm. The measurement length was approximately 500 µm. Because of the vertical resolution of the test device, it is impossible to separate roughness from waviness in this case. A profile obtained through this approach consequently yields a description of the meso-waviness and overall shape. Figure 2-19b shows that the optometric device does not allow to detect any significant variations in terms of roughness level under a threshold CSP value (no. 5) corresponding to the vertical resolution of the optometric device. Nevertheless, above this value, the optometric method accurately reproduces the surface roughness level in accordance with the CSP scale. Similar investigations were performed by Maerz et al. [7].

It can be concluded that it is possible to significantly improve the CSP replicate system through a real quantitative approach. The actual CSP plates are rather narrow with respect to the spectrum of CSPs obtained with actual surface preparation techniques. The identification of reference curves, similar to those presented by Perez et al. but on a wider scale of surface roughness, will help broaden the range of application of this method to much coarser profiles such as those obtained with jack hammering and water jetting, for example.
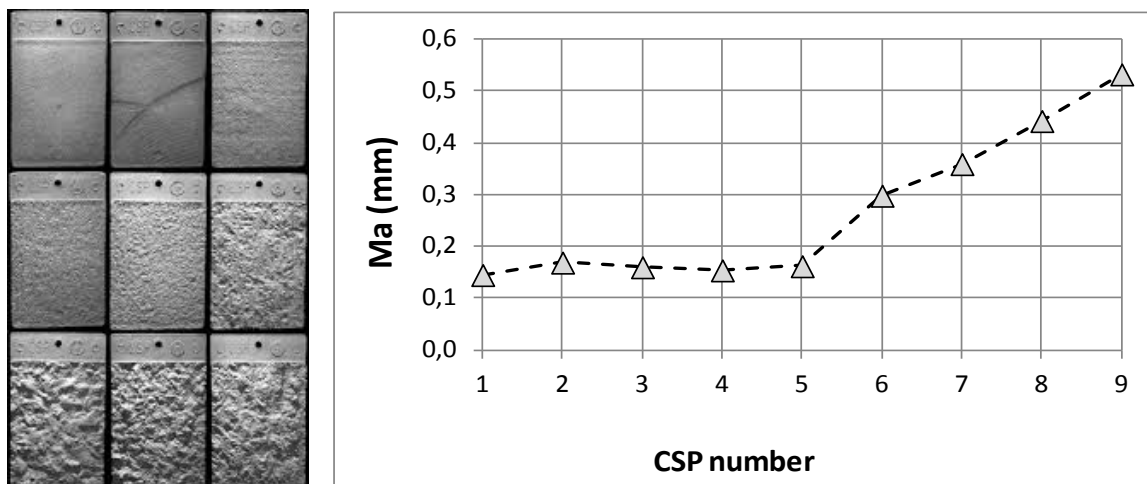
*Figure 2-10: ICRI CSP evaluation (Guideline No. 310.2R–2013) – a) photographs of the nine replicates ordered from 1 to 9; b) characterization of the CSP replicates performed by Perez et al. using an interferometric method [5]*

# Conclusions

Characterization of surface roughness is an important aspect in assessing a concrete substrate prior to repair. Various techniques have recently been available for CSP characterization. The combination of different methods can yield a very accurate description of roughness at various scale levels, depending on the roughness range to be analyzed. For instance, mechanical and laser (laboratory-type) profilometers allow for a more accurate microroughness characterization, while the interferometric (optical) method provides a better description of the shape of the profile. Nevertheless, investigations of a range of concrete surface treatments with very precise laser and mechanical profilometers clearly indicated that the surface treatment technique does not have much influence on microroughness (high-frequency waves). This indicates that only the waviness parameters actually need to be determined in order to assess surface roughness prior to repair.

Among the techniques available today, the best suited method for field assessment appears to be the CSP developed by ICRI: it is quick and easy to use and yields reliable information, irrespective of the surface orientation. However, its use is limited at present to surfaces of up to 6 mm in profile height, for which it was actually designed; it is clearly not suitable for water jetting or jack hammering evaluations.

The advantages of the sand patch test method are its speed and applicability in situ on a surface that must be protected from wind and rain. Its main limitations are the range of validity (0.25–5 mm), which usually excludes surfaces with holes and high peaks, and the ability to apply it only on horizontal surfaces. It does, however, have good correlations with statistical parameters such as $W_a$.

Since the operation of surface preparation essentially influences waviness, the optical method based on the moiré pattern, which offers significant advantages in terms of production rate and surface area treatment capability, could in fact be used alone to perform the whole surface roughness characterization.  This method directly yields a handful of reliable quantitative data, but the equipment available today is not adapted to daily field applications.  Nevertheless, with the rapid technological development in that field, the availability of suitable optical devices may be in the near future.  This would allow for even more rapid and objective assessments.

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## Standards and test methods

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ICRI Guideline No. 310.2R-2013 Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, Polymer Overlays, and Concrete Repair International Concrete Repair Institute, St. Paul (MN), USA, 2013, www.icri.org"

# Section 3 - Evaluation of Relationships between Tensile Bond, Shear Bond and Substrate Roughness Parameters (Task 2)

# Contents

# Figures

# Tables

# Section 3 - Evaluation of relationships between tensile bond, shear bond and substrate roughness parameters (Task 2)

For Task 2, experimental work was carried out both at the US Bureau of Reclamation in Denver (CO), USA, and the Research Center on Concrete Infrastructure (CRIB), Laval University, Quebec City (QC), Canada.

## Introduction

In addition to adhesion and cohesion, another parameter often considered to affect the tensile bond between a repair material and existing concrete is the substrate roughness.  In fact, this subject has been controversial for years.

In some studies, the reported bond test results have shown that surface roughness has only a minor influence on the tensile bond.  For instance, in the tests performed by Silfwerbrand [1], bond to rough water jetted surface was compared with bond to smooth sandblasted surface.  It was concluded that there could be a roughness "threshold value" beyond which further improvement on the roughness would not enhance bond strength.  According to these test results, the "threshold value" ought to be close to the surface roughness of the typical sandblasted surfaces.

Still, it remains the opinion of a number of other specialists in the industry that a rougher surface is beneficial to bond strength.  Talbot et al. [2] investigated the influence of surface preparation and concluded that concrete substrates with smoother surface profiles, produced for instance by grinding or simple sandblasting, experienced significant loss of bond strength with time.  On the contrary, surfaces that were roughened mechanically and subsequently sandblasted exhibited good bond durability.  The reason for this may lie in the fact that high interface roughness, as it is commonly achieved in field repairs in practice, improves the resistance against interface shear stress resulting from repair material drying shrinkage.

Also, as raised in the previous explanation for the potentially beneficial influence of roughness upon bond strength, the differential volume changes between the repair material and the substrate can induce potentially critical shear stresses in some areas of the interface [3].  However, in practice, bond strength of concrete repairs and overlays is generally defined as the tensile strength in the direction perpendicular to the interface plane and measured essentially in direct tension through pull-off testing.

It is necessary to realize that adhesion mechanisms in tension and in shear may differ significantly.  For example, a high interface roughness may improve shear bond strength, whereas tensile mechanical bond strength primarily depends on vertical anchorage in pores and voids. Under service conditions, the repair interface is subjected to both tensile and shear stresses. When specifying and/or evaluating bond strength values, it might thus be important to address explicitly the dominant interface stress condition encountered in the actual structure.

To this day, relatively little data have been reported in relation with shear bond strength. Most published studies are in agreement that shear bond strength is higher than tensile bond strength. However, there is no agreement on the magnitude of the correlation. In the studies that were reviewed, the reported average shear bond strength to tensile bond strength ratio varies from 1.2 to 2.0. That range is obviously too wide for converting satisfactorily the pull-off test results to shear bond strength. However, it is easier to measure the tensile bond strength, and it can be used reliably as a definition of bond if a decent relationship between the two bond strength parameters is established.

Hence, the main objectives of this task were to establish the relationship between both tensile and shear bond strengths and the substrate roughness. Since roughness directly depends on the surface preparation method, this task is intended to shed new light on the subject and help resolve the controversy regarding the extent of its influence.

## Methodology

Two experimental programs were conducted complementarily at the U.S. Bureau of Reclamation (USBR), Denver (CO), USA and at Laval University (CRIB), Quebec City (QC), Canada.

   1.1. Test program conducted at USBR

The test program conducted at USBR is summarized in Table 3-1. Details pertaining to the test variables, the test specimens, the surface preparation techniques and the test methods are provided in the following subsections.

Investigated variables and specimen preparation

A series of 12 concrete slabs 46 in. by 22 in. by 6 in. (1170×560×150 mm) were manufactured for the test program (Figure 3-1). The slabs were by cast using a 6000 psi (40-MPa) ready-mix concrete. The basic properties of both mixtures are displayed in Table 3-2. The slabs were exposed to drying at least six months to achieve relative dimensional stability, after what surface preparation was performed.

Three of the most common surface preparation techniques were selected for investigation: sandblasting (SA), water jetting (WJ), and jackhammering (JH), with the characteristics provided in Table 3-1. Sets of four base slabs were prepared with each of these techniques, as shown in Figure 3-2.

*Table 3-1: Test program conducted at USBR*

| Item | Details |
|---|---|
| **Test specimens**<br><br>• Base slabs | - 1170×560×150 mm base concrete slabs<br>- 1 slab series: 12 slabs prepared with 40-MPa OPC concrete<br>- base slabs moist cured for 3 days after casting and exposed to drying for more than 6 months prior to repair |
| • Repaired test slabs | - repairs performed on the slab series submitted to 3 different surface preparation methods and pre-wetted to *SSD*: 75-mm thick overlays with a 40-MPa OPC concrete mixture |
| **Investigated surface preparation techniques** (surface prep. prior to repair) | - sandblasting (SA)              (4 slab per series)<br>- 15-ksi (100 MPa) handheld water jetting (WJ)    (4 slab per series)<br>- 15-lb handheld jackhammering (JH)   (4 slab per series) |
| **Characterization test methods for**:<br><br>• Surface roughness | - ICRI *Concrete Surface Profile* (CSP) index<br>- ASTM *Macrotexture Depth* test (*Sand Patch* test)<br>- Optical profilometry |
| • Surface integrity | - Schmidt hammer |
| • Bond strength (28-d) | - Pull-off test<br>- Torque test |

After surface preparation, evaluation of surface integrity and characterization of surface roughness were performed. The slabs were then repaired (3 in. (75-mm) overlay) with the same 40-MPa concrete mixture. The repair concrete mixtures properties are also summarized in Table 3-2. The repaired specimens were moist-cured for 3 days, after which they were air-dried for at least 28 days, until the bond strength tests were carried out.

*Figure 3-1: Preparation of test slabs at USBR*

*Table 3-2: Concrete mixture characteristics and mechanical properties (USBR)*

| Material | Test slab concrete mixtures | Repair concrete mixture |
|---|---|---|
| Nominal strength | 40 MPa (6000 psi) | 40 MPa (6000 psi) |
| Mixture characteristics | ASTM Type I cement ½ in (14-mm) coarse aggregates | ASTM Type I cement ½ in (14)-mm coarse aggregates |
| Fresh concrete properties<br>    Slump (in/mm)<br>    Air content (%) | 3 (75)<br>3.2 | 3 (75)<br>4.5 |
| Compressive strength[1] (MPa)<br>    7 d<br>    28 d | -<br>38.7 | 34.1<br>39.2 |
| Splitting tensile strength[2] (MPa)<br>    7 d<br>    28 d | -<br>3.1 | 3.8<br>4.4 |
| Elastic modulus[3] (GPa)<br>    7 d<br>    28 d | -<br>- | 24.2<br>26.1 |

[1]ASTM C39; [2]ASTM C496; [3]ASTM C469.

Surface roughness characterization

The roughness of the surface profiles achieved with the various investigated surface preparation techniques was evaluated with three different methods.  All slab profiles were characterized in accordance with ICRI *Concrete Surface Preparation* index (CSP; ICRI No. 310.2R-2013), with the *Macrotexture depth* method (EN 13036-1 / ASTM E965-06) – often referred to as the sand patch test method – and a *Moiré*-type optical profilometry method to evaluate the average half-amplitude parameter ($R_a$).
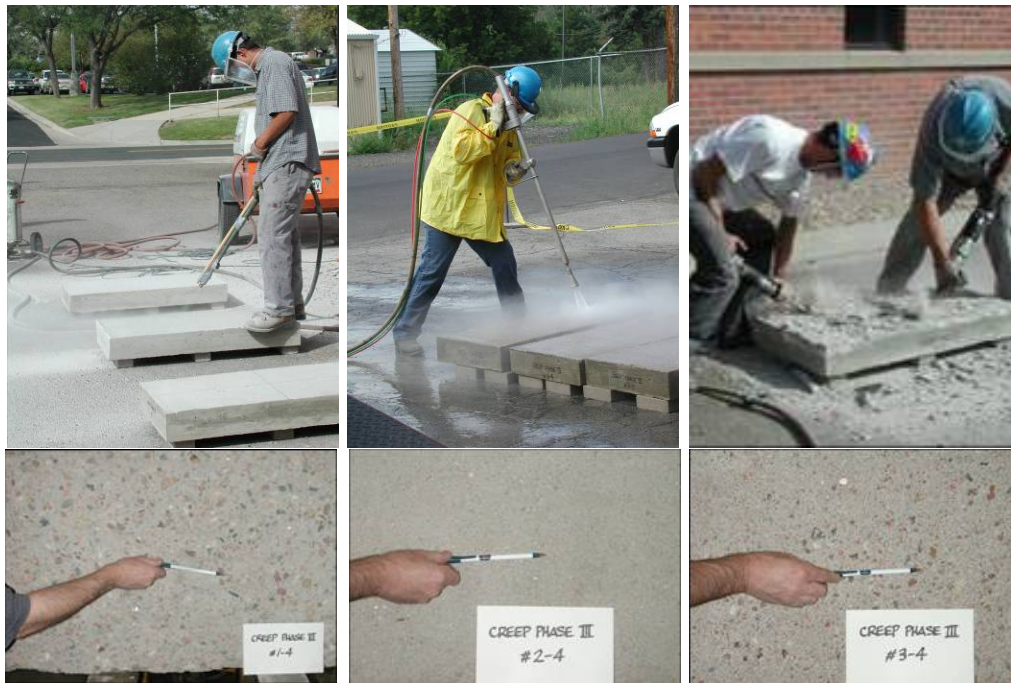
|a) Sandblasting | b) Water jetting (15 ksi) | c) Jackhammering (15 lb) |

*Figure 3-2: Surface preparation of the test slabs at USBR*

Evaluation of surface integrity

Surface integrity of the prepared test slab was evaluated on an exploratory basis through Schmidt hammer soundings. It was reported by Courard et al. [4] to be a potentially interesting mean for detecting the presence of surface damage, provided that the number of tests is sufficient. Seeking a simple and field-friendly way to assess surface integrity prior to repair, Schmidt hammer soundings were thus performed in a systematic fashion on all prepared test slabs, using a template grid with regularly-spaced data points collected along the X- and Y- directions over the whole surface.

Bond strength evaluation

All 12 repaired test slabs were characterized for bond strength with a combination of pull-off tests and torsional shear tests.

For the evaluation of tensile bond strength, the most widely used method is the pull-off test (ASTM C1583; CAN/CSA A23.2-6B; EN 1542; BS 1881). This test method consists of drilling a core through the repair material down to a minimum depth within the substrate, gluing a steel dolly onto the top of the core with epoxy, and then pulling on the steel dolly using a special loading rig. The tensile bond strength is equal to the maximum recorded stress when failure occurs in the interfacial zone, whereas a lower boundary value of bond strength is obtained when failure occurs elsewhere. In this part of the study, the pull-off strength tests were performed in accordance with the CAN/CSA A23.2-6B procedure.

Torsional shear tests have been included on the experimental program to evaluate the bond shear response and sensitivity with respect to the tensile behavior. In this test procedure, a ring glued to the surface is twisted off using a torque housing with eccentric loading. The housing is anchored to the surface and the loading is performed with the same pulling unit as in the pull-off test procedure (different adapters). There is no standard procedure for this test. More information can be found on the manufacturer's website (germann.org) and in a paper by Petersen and Poulsen [5].

The pull-off and shear tests were performed at least 28 days after pouring of the overlays. All 12 repaired test slabs were characterized for bond strength with a combination of nine pull-off tests and nine torsional shear tests on each of them, resulting in a total 36 pull-off tests and 36 shear bond strength tests per slab series. The template for the different tests is presented in Figure 3-3. The pull-off and shear bond tests were performed at specific locations from one test slab to the other, selected in such a way to assure better reproducibility of the results.
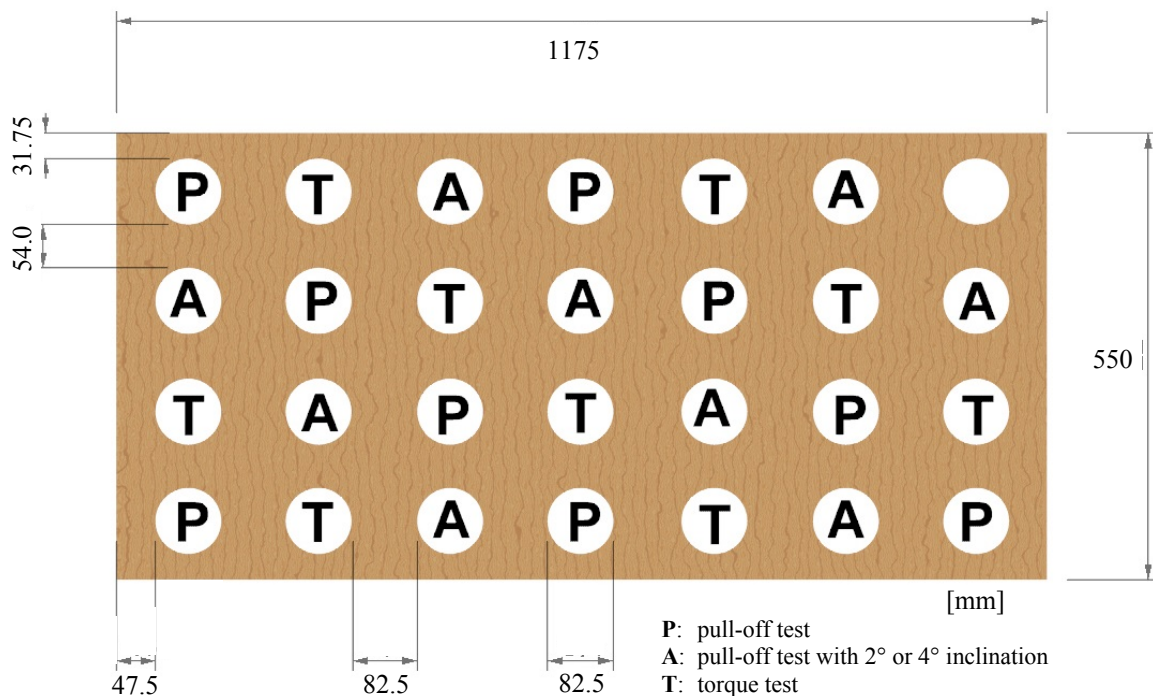


*Figure 3-3: Core-drilling template for mechanical bond testing (USBR program)*
*(note: the pull-off tests with an inclination are addressed in the next section of the report)*

### 1.2. Test program conducted at Laval University

The test program conducted at Laval University (UL) is summarized in Table 3-3. Details pertaining to the test variables, the test specimens, the surface preparation techniques and the test methods are provided in the following subsections.

*Table 3-3: Test program conducted at UL*

| Item | Details |
|---|---|
| **Test specimens**<br><br>&bull; Base slabs | - 1250×625×150 mm base concrete slabs<br>- 2 slab series:  16 slabs prepared with 20-MPa OPC concrete<br>                15 slabs prepared with 30-MPa OPC concrete<br>- base slabs moist cured for 3 days after casting and exposed to<br>  drying for more than 3 months prior to repair |
| &bull; Repaired  test slabs | - repairs performed on the two slab series submitted to various surface preparation methods and pre-wetted to *SSD*: 75-mm thick overlays with a 40-MPa OPC concrete mixture |
| **Investigated surface preparation techniques**<br>(surface prep. prior to repair) | - scarifying (**SC**)             (3 slabs per series)<br>- shotblasting (**SH**)          (3 slabs per series)<br>- sandblasting (**SA**)          (3 slabs per series)<br>- water jetting (**WJ**)         (3 slabs per series)<br>- jackhammering (**JH**)       (3 slabs per series)<br><br>- artificially cast roughness profiles (1 slab / 20-MPa series) |
| **Characterization test methods for**:<br><br>&bull; Surface roughness<br><br><br>&bull; Surface integrity<br><br><br>&bull; Bond strength (28-d) | <br><br>- ICRI *Concrete Surface Profile* (CSP) index<br>- Optical profilometry<br><br>- Schmidt hammer<br>- Pull-off test<br><br>- Pull-off test<br>- Torque test |

Investigated variables and specimen preparation

Two series of 16 concrete slabs (1250×625×150 mm) were manufactured for the test program (Figure 3-4).  The first series was made with a 30-MPa ready-mix concrete, while the second series was prepared using a 20-MPa concrete.  The basic properties of both mixtures are displayed in Table 3-4.  The slabs were exposed to drying for at least three months to achieve relative dimensional stability, after what surface preparation was performed.

*Figure 3-4: Test slabs prepared at UL*

*Table 3-4: Concrete mixture characteristics and mechanical properties (UL program)*

| Material | Test slab concrete mixtures | | Repair concrete mixture |
|---|---|---|---|
| Nominal strength | 20 MPa (3000 psi) | 30 MPa (4500 psi) | 40 MPa (6000 psi) |
| Mixture characteristics | *w/cm* = 0.65 | *w/cm* = 0.57 | *w/cm* = 0.40 |
| | CSA Type 10 cement ¾ in (20-mm) coarse aggregates | | CSA Type 10 cement ½ in (14-mm) coarse aggregates Black pigments (6% wgt of C) |
| Fresh concrete properties[1] Slump (mm) Air content (%) | 80 6 | 80 6 | 80 6 |
| Compressive strength[2] (MPa) 7 d 28 d 56 d | 17.1 20.8 21.5 | 27.4 33.1 35.4 | 37.8 43.6 46.1 |
| Splitting tensile strength[3] (MPa) 7 d 28 d 56 d | 1.7 1.9 2.0 | 2.9 3.2 3.3 | 2.6 2.9 3.0 |
| Elastic modulus[4] (GPa) 7 d 28 d 56 d | 18.2 19.7 - | 24.2 27.1 29.5 | 25.3 27.5 - |

[1]Specified values; [2]ASTM C39, [3]ASTM C496, [4]ASTM C469.

In order to cover a sufficiently large spectrum in terms of roughness and, at the same time, to address most usual surface preparation techniques, the following methods were selected for investigation: sandblasting (SA), shotblasting (SH), scarifying (SC), 15,000-psi handheld water jetting (WJ), and 15-lb handheld jackhammering (JH) (Figure 3-5).  In both test slab series, sets of three base slabs were prepared with each of these techniques.  In addition, to prevent the potential influence of any induced damage and isolate the effect of roughness upon bond strength, one artificially profiled test specimen was cast in each slab series.  V-shape rippled acrylic dies

were installed at the bottom of the test slab forms to obtain wave amplitude values of 2, 4, 6 and 8 mm respectively in four adjacent areas along the specimen length, the wavelength being of 30 mm in all of them (Figure 3-6).



*Figure 3-5: Scarification and shotblasting surface preparation at CRIB (UL program)*
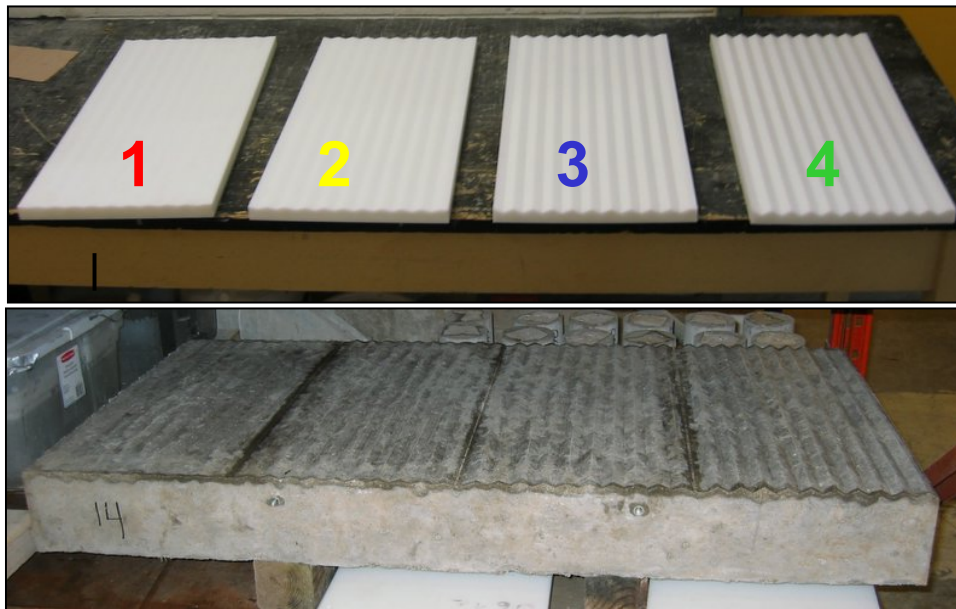


*Figure 3-6: V-shape rippled acrylic dies and resulting profiled slabs (UL program)*

After surface preparation, evaluation of surface integrity and characterization of surface roughness were performed. The slabs were then repaired (75-mm overlay) with a 40-MPa repair concrete. The artificially profiled slabs (one per slab series) were very lightly sandblasted to remove laitance. In order to easily locate the interface location on the cores and establish more precisely the failure mode and path, a black pigment was added to the repair material at the time of mixing (rate of addition: 6 % by weight of cement). The repair concrete mixtures properties are also summarized in Table 3-4. The repaired specimens were moist-cured for 7 days, after what they were air-dried for at least 28 days, until the bond strength tests were carried out.

Surface roughness characterization

The roughness of the surface profiles achieved with the various investigated surface preparation techniques was evaluated with two different methods. All slab profiles were first characterized in accordance with ICRI *Concrete Surface Preparation* index (CSP; ICRI No. 310.2R-2013), and then using *Moiré*-type optical profilometry to yield the average half-amplitude parameter ($R_a$).

Evaluation of surface integrity

Surface integrity of the prepared test slab was evaluated through pull-off experiments and, still on an exploratory basis, Schmidt hammer soundings.

Seeking a simple and field-friendly way to assess surface integrity prior to repair, Schmidt hammer soundings were performed in a systematic fashion on all prepared slabs, using a template grid with regularly-spaced data points collected in the X- and Y- directions over the whole surface.

Pull-off tests were performed immediately after the Schmidt soundings. Surface integrity of the prepared test slab was evaluated through pull-off experiments performed on the prepared slabs, in accordance with a procedure proposed by Courard and Bissonnette [6]. It has proven to be reliable for detecting the presence of surface damage, provided that the number of tests is sufficient. On each tested slab, eight substrate pull-off tests were performed (4 on 75-mm cores; 4 on 100-mm cores), in accordance with the pattern shown on Figure 3-7.



**P**: pull-off test (repair)
**T**: torque test (repair)
**S**: substrate pull-off test (**S***: 100-mm core)
**D**: full-depth core extracted for direct tensile testing (repair)
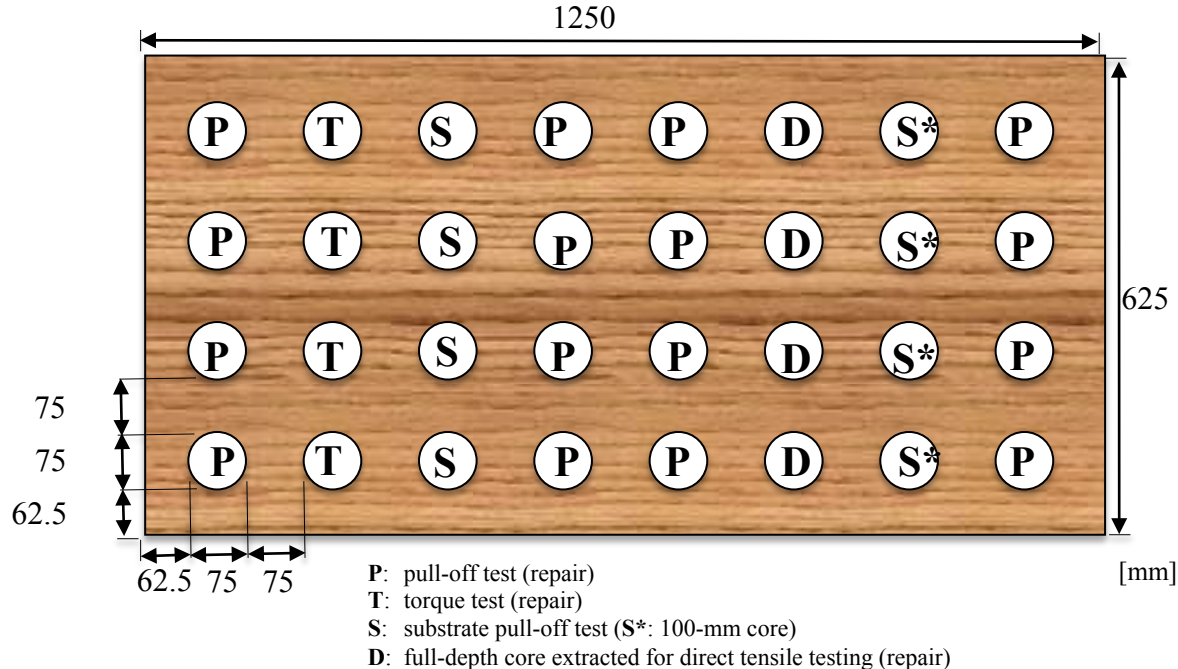
*Figure 3-7: Core-drilling template for mechanical bond testing (UL program)*

Bond strength evaluation

All 32 repaired test slabs were characterized exhaustively for bond strength with a combination of pull-off tests, direct tensile tests and torsional shear tests.

For the evaluation of tensile bond strength, pull-off tests were performed in accordance with CAN/CSA A23.2-6B.  In addition to the pull-off tests, direct tensile bond strength tests were performed on cores taken from the slabs.  As part of the specimen preparation procedure, a five-millimeter deep circumferential saw cut was performed on the cores at the interface level in order to reduce their cross-sectional area and promote interfacial failure.  Core preparation was completed by gluing steel dollies at both ends.  The specimens were then tested in a universal testing frame in accordance with the U.S. Army Corps of Engineers test method for determining direct tensile strength (CRD-C 164).

Torsional shear tests have again been carried out to evaluate the bond shear response and sensitivity with respect to the tensile behavior, using the same test procedure used at USBR.

The pull-off, direct tension and shear bond tests were performed at least 28 days after pouring of the overlays.  All 32 repaired test slabs were characterized for bond strength with a combination of 16 pull-off tests, four direct tensile tests and four torsional shear tests on each of them, resulting in a total 48 pull-off tests, 12 direct tensile tests, and 12 shear bond strength tests per slab series.  The template for the different tests is presented in Figure 3-7.  The different tests were performed at specific locations from one test slab to the other to assure better reproducibility of the results.

## Results and analysis

1.3. Test program conducted at USBR

Surface roughness

The surface roughness characteristics corresponding to the various surface preparation profiles, as obtained using the ICRI CSP index (ICRI No. 310.2R-2013), the ASTM E965-06 Macrotexture depth test method, and optical profilometry respectively, are summarized in Table 3-6.

Overall, the roughness parameters determined with the three different characterization methods are consistent: sandblasting and jackhammering yielded the lowest and highest roughness values respectively, while water jetting yielded an intermediate profile, somewhat on the low side.

| Test | Parameter | Surface preparation | | |
|---|---|---|---|---|
| | | **Sandblasting** | **Water jetting** | **Jackhammering** |
| ICRI CSP profiles (ICRI No. 310.2R-2013) | Avg. CSP index (1-9) | 4.5 | 6 | > 9 |
| Macrotexture depth (ASTM E965-06) | Avg. depth (mm) | 0.5 | 1.4 | 5.8 |
| Optical profilometry | Half-amplitude $R_a$ (mm) | 0.4 | 0.5 | 2.6 |

It is interesting to compare the quantitative evaluations generated with the ASTM procedure and optical profilometry. The macrotexture depth test method allows for evaluating the average depth of the surface profile roughness, while the optical method yields a digital 3-D representation of the profile, which is treated to extract roughness parameters such as $R_a$, as shown in Figure 3-8. Since the $R_a$ parameter corresponds to half of the surface profile wave amplitude, it should then be expected to amount approximately to half the macrotexture depth yielded in accordance with ASTM E965-06. On the graph of Figure 3-9, the experimental data actually show a satisfactory correlation.
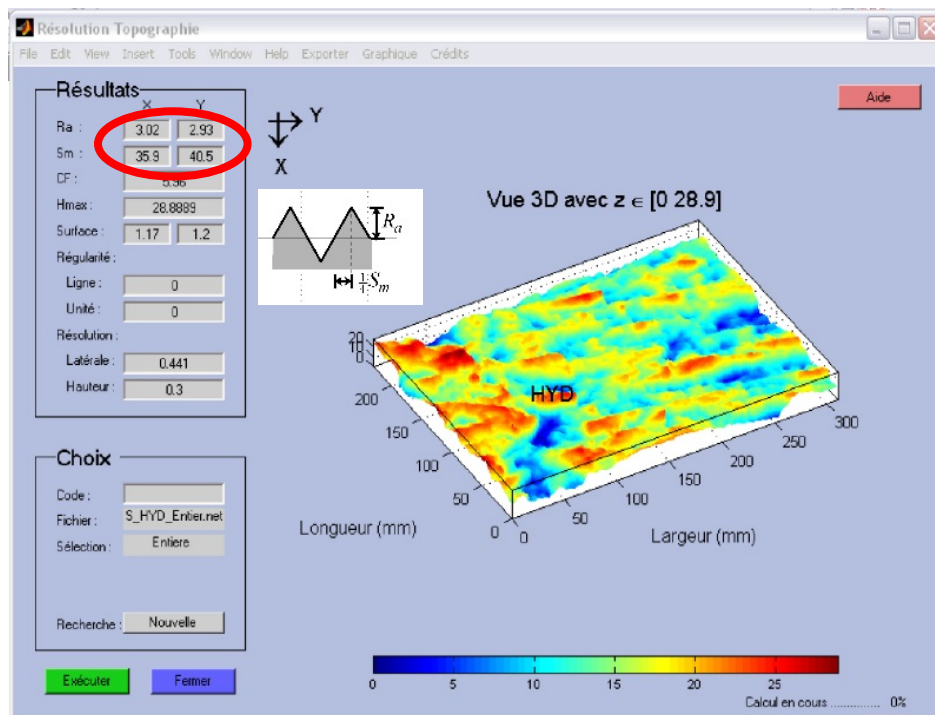


*Figure 3-8: Computer interface for optical profilometry data treatment (example shown: test slab prepared with water jetting)*
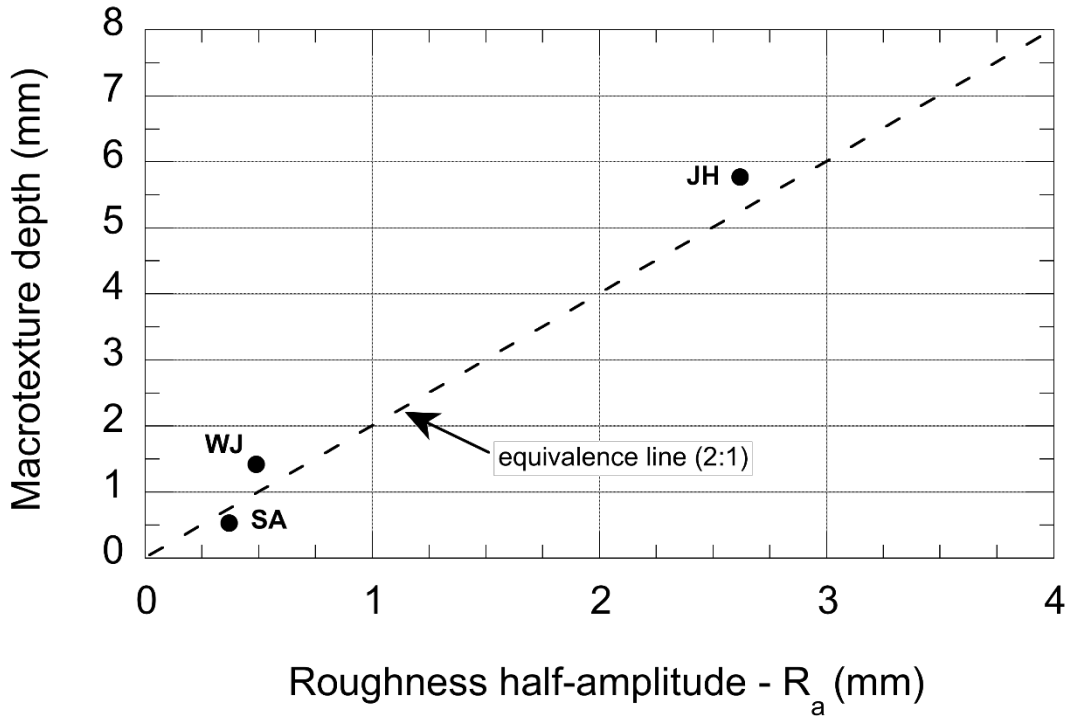
*Figure 3-9: Correlation between macrotexture depth (ASTM E965-06) and optical profilometry characterization (USBR program) (note: SA – sandblasting; WJ – water jetting; JH – jackhammering)*

Mechanical integrity of the substrate

Data recorded from the Schmidt hammer soundings performed on the substrate subjected to various types of surface preparation are summarized in Figure 3-10.  According to Courard et al. [4], data variability, rather than absolute values, would provide an indication of the presence and importance of defects in the substrate.  As a matter of fact, on surfaces exhibiting significant waviness, the rebound recorded with the Schmidt hammer is affected negatively.  The comparatively low estimated strength values for both water jetted and jackhammered are thus not surprising.  Nevertheless, it can be observed in the figure that the latter are characterized by a significantly larger variability, which may potentially reflect the presence of surface defects induced by the hammer tip.  Such damage induced into the substrate by jackhammers and the various types of impact breakers, generally referred to as bruising, was assessed in a previous study [7].
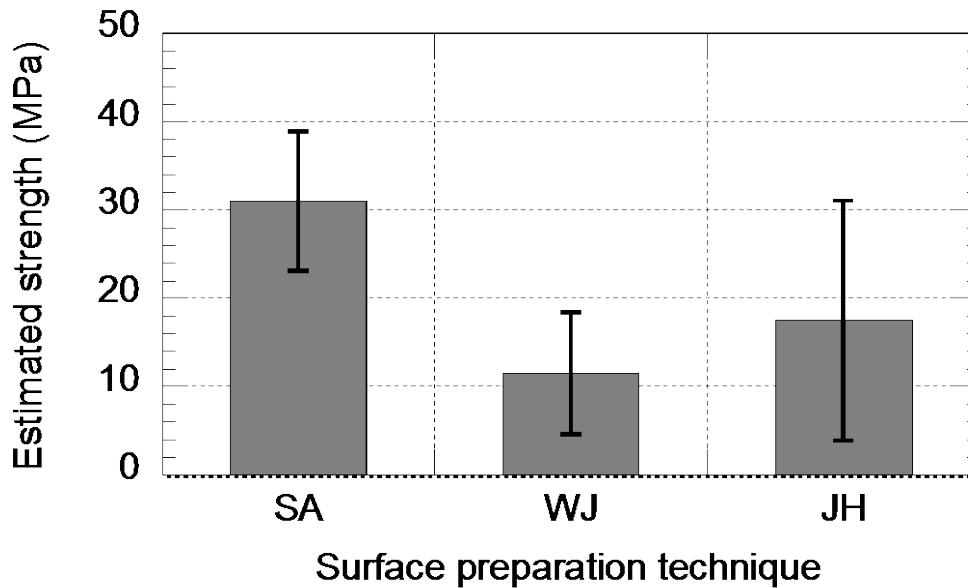
*Figure 3-10: Results of Schmidt hammer soundings (ASTM C805) performed after surface preparation to evaluate the mechanical integrity of the exposed concrete surface (USBR program)*
*(note: SA – sandblasting; WJ – water jetting; JH – jackhammering)*

Bond strength

The detailed results of the various bond strength tests performed on the experimental slabs are presented in Tables 3-6 and 3-7, and in Figures 3-11 to 3-14.

*Pull-off testing*

When comparing the pull-off test results obtained on sandblasted and water jetted surfaces in Figures 3-11 to 3-14, the small increase in surface roughness generated with the latter method apparently resulted in a slightly stronger bond. However, even though jackhammering yielded a much coarser surface profiles than the two other investigated surface preparation methods, the pull-off strength values recorded for the corresponding repaired slabs rank as the lowest.

As far as the relationship observed between pull-off strength and substrate roughness is concerned, the graphs of Figures 3-15 to 3-17 hereafter all exhibit similar trends, irrespective of the method used for characterizing roughness.

It appears that bond strength slightly increases with the level of roughness of the substrate, provided that no or limited damage is induced. In fact, the failure location distributions (Table 3-7 and Figure 3-12) tend to show that from sandblasting to water jetting, failure is somehow pushed away from the interfacial zone down into the substrate (75 %), while with jackhammering, the tendency is reversed and failure occurs preferentially in the interfacial area (70 %). This is consistent with the respective levels of variability found for the Schmidt hammer data on the different surface preparations (Figure 3-18).

The reversed tendency observed for the aggressive jackhammering technique has to be attributed to the presence of disseminated defects left on the surface after completion of the jackhammering operations, as confirmed by the microanalysis findings of a previous investigation [7]. Where the

extent of the defects induced in the substrate becomes significant, the positive influence of increased roughness is found to be completely offset mechanically by the adverse effects of bruising.

Table 3-6: Summary of pull-off test results for the 40-MPa concrete substrate series (USBR program)

| Surface preparation method | Test parameters | Location of failure | | | | | Overall |
|---|---|---|---|---|---|---|---|
| | | Core end | Substrate | Interface | Repair | Dolly | |
| Sandblasting (SA) | Number of test results (%) | 20 (57.1) | 2 (5.7) | 10 (28.6) | 1 (2.9) | 2 (5.7) | 35 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.91** (0.24) | **2.04** (0.16) | **1.29** (0.45) | **1.63** - | **1.20** (1.02) | **1.69** (0.46) |
| Water jetting (WJ) | Number of test results (%) | 27 (75.0) | 6 (16.7) | 1 (2.8) | 2 (5.6) | 0 (0) | 36 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.94** (0.32) | **2.01** (0.30) | **1.99** - | **1.83** (0.13) | **-** (-) | **1.96** (0.30) |
| Jackhammering (JH) | Number of test results (%) | 4 (11.1) | 7 (19.4) | 25 (69.4) | 0 (0) | 0 (0) | 36 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.57** (0.25) | **1.03** (0.27) | **1.35** (0.47) | **-** (-) | **-** (-) | **1.31** (0.44) |

Table 3-7: Summary of torque test results for the 40-MPa concrete substrate series (USBR program)

| Surface preparation method | Test parameters | Location of failure | | | | | Overall |
|---|---|---|---|---|---|---|---|
| | | Core end | Substrate | Interface | Repair | Dolly | |
| Sandblasting (SA) | Number of test results (%) | 2 (5.7) | 11 (31.4) | 12 (34.3) | 10 (28.6) | 0 (0) | 35 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.21** (0.00) | **1.51** (0.64) | **1.28** (0.65) | **2.14** (0.46) | **-** (-) | **1.60** (0.71) |
| Water jetting (WJ) | Number of test results (%) | 8 (22.2) | 10 (27.8) | 8 (22.2) | 10 (27.8) | 0 (0) | 36 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.80** (0.84) | **1.93** (0.35) | **2.34** (0.57) | **2.28** (0.53) | **-** (-) | **2.05** (0.56) |
| Jackhammering (JH) | Number of test results (%) | 8 (24.2) | 6 (18.2) | 10 (30.3) | 7 (21.2) | 1 (3.0) | 33** (100) |
| | Avg. strength [MPa] (std. dev.) | **1.57** (0.53) | **1.89** (0.93) | **1.46** (0.49) | **2.66** (1.11) | **6.30*** (-) | **1.84** (0.85) |

\*   value discarded in calculating the overall average strength
\*\* for one test core, the failure location was not reported
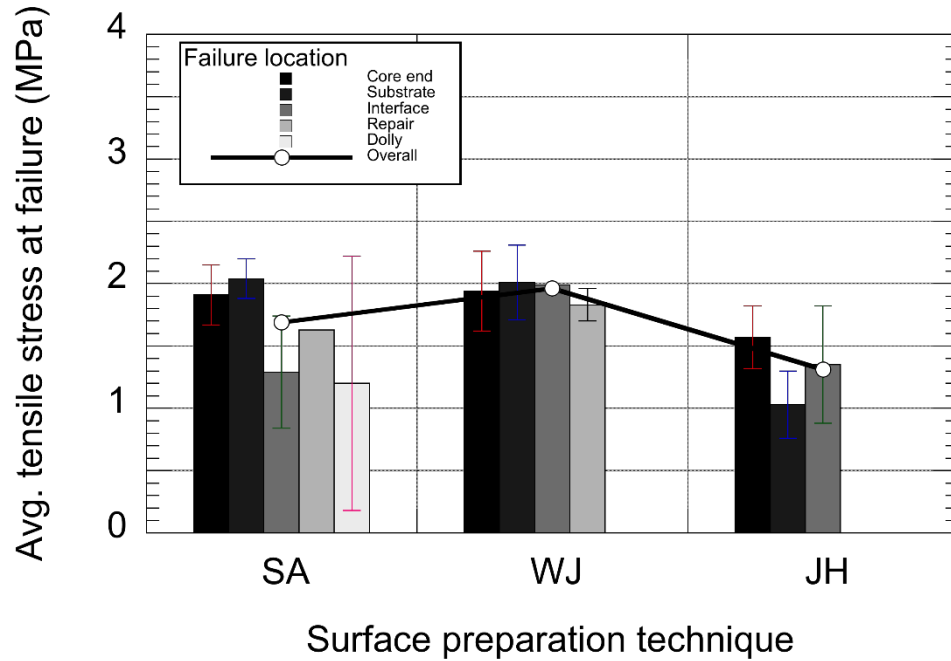
*Figure 3-11: Results of pull-off tests performed on 40-MPa substrates after repair (USBR program)*
*(note: SA – sandblasting; WJ – water jetting; JH – jackhammering)*
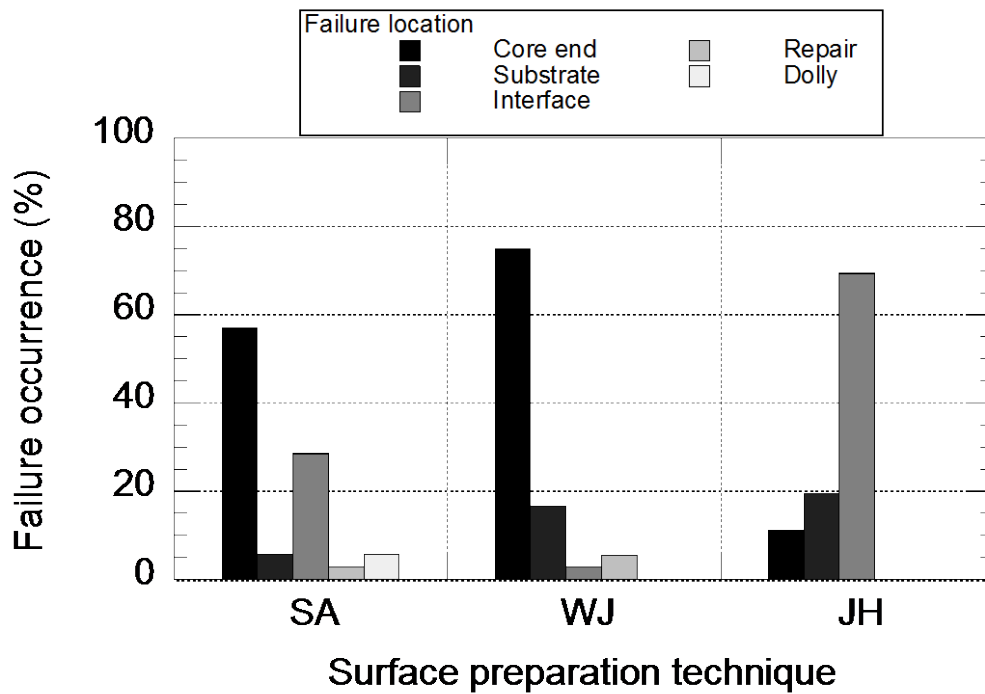


*Figure 3-12: Distribution of failure location in pull-off tests performed on 40-MPa substrates after repair (USBR program)*
*(note: SA – sandblasting; WJ – water jetting; JH – jackhammering)*
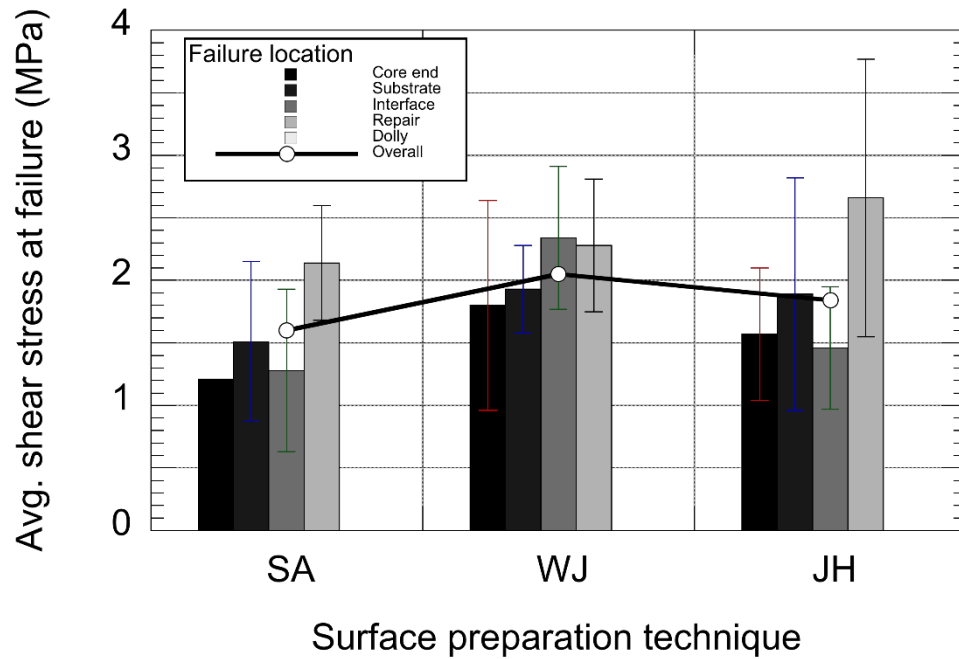
*Figure 3-13: Results of torque tests performed on 40-MPa substrates after repair (USBR program)*
*(note: SA – sandblasting; WJ – water jetting; JH – jackhammering)*
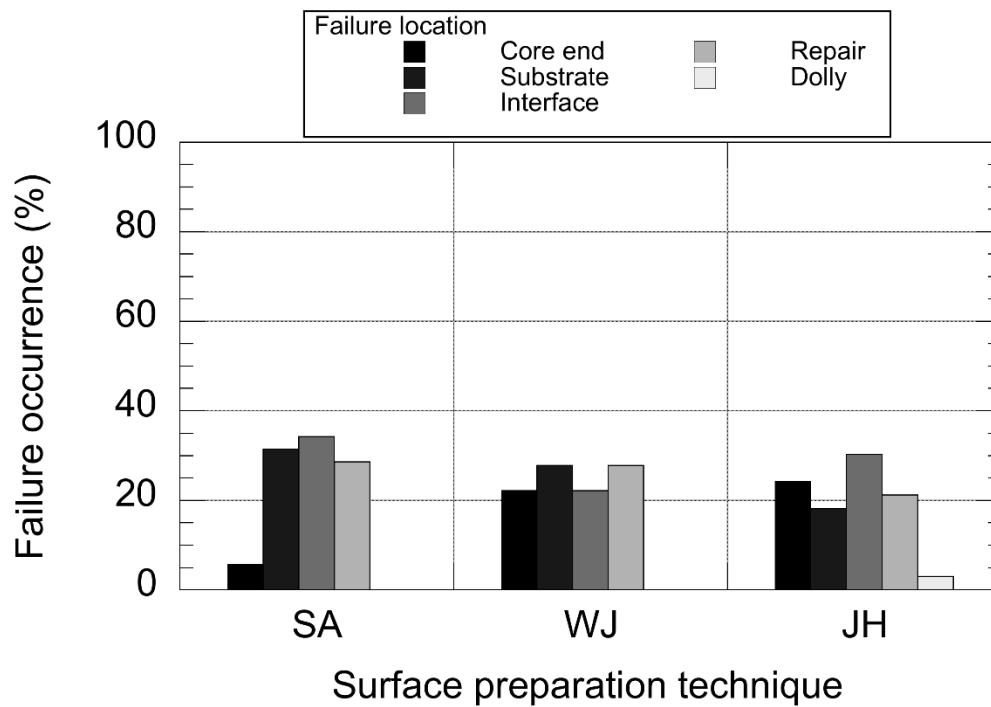


*Figure 3-14: Distribution of failure location in torque tests performed on 40-MPa substrates after repair (USBR program)*
*(note: SA – sandblasting; WJ – water jetting; JH – jackhammering)*
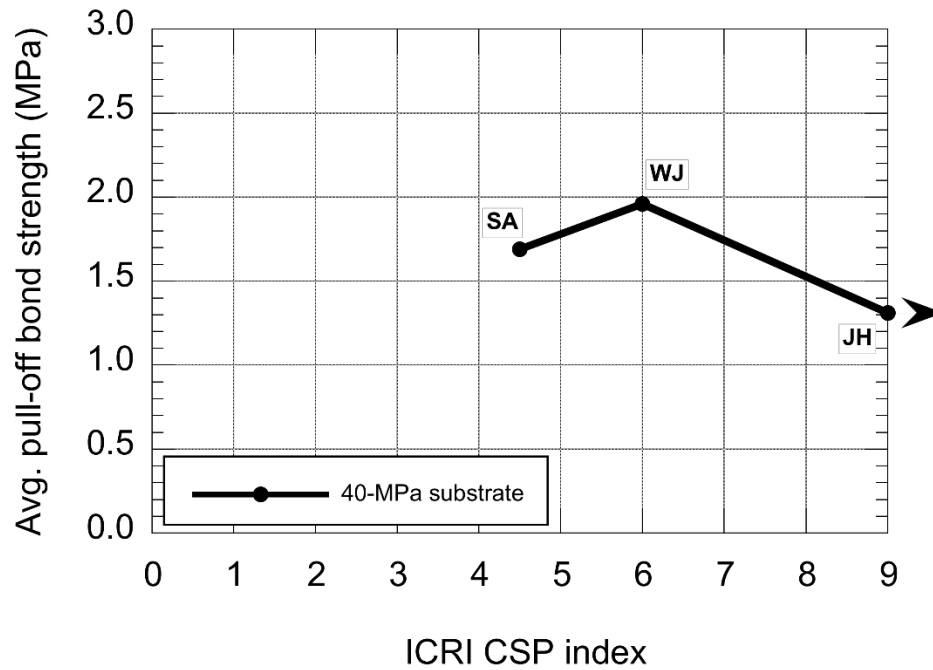
*Figure 3-15: Results of pull-off tests (ASTM C1583) performed after repair as a function of the substrate CSP index (ICRI No. 310.2R-2013) generated by various preparation techniques (USBR program)*
*(note: SA – sandblasting; WJ – water jetting; JH – jackhammering)*
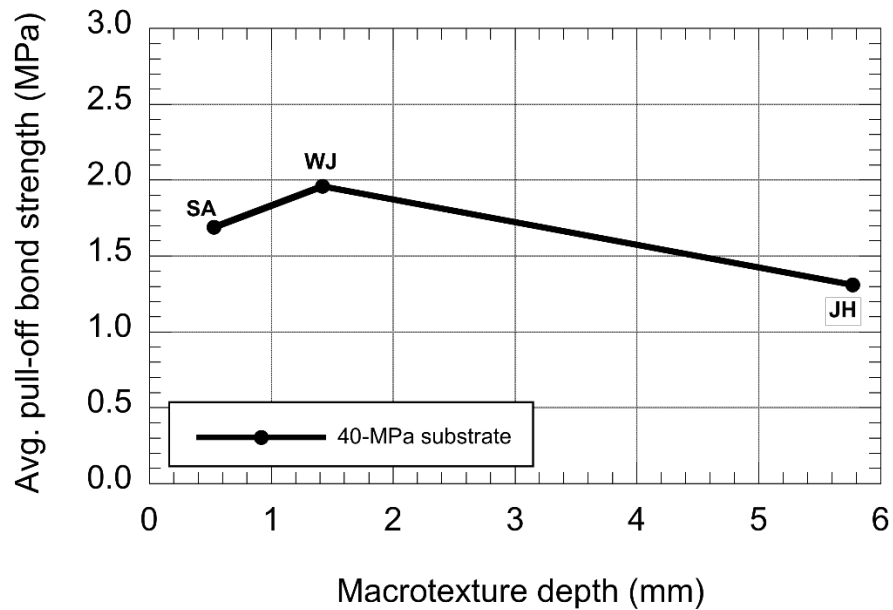


*Figure 3-16: Results of pull-off tests (ASTM C1583) performed after repair as a function of the substrate Macrotexture depth (ASTM E965) generated by various preparation techniques (USBR program)*
*(note: SA – sandblasting; WJ – water jetting; JH – jackhammering)*
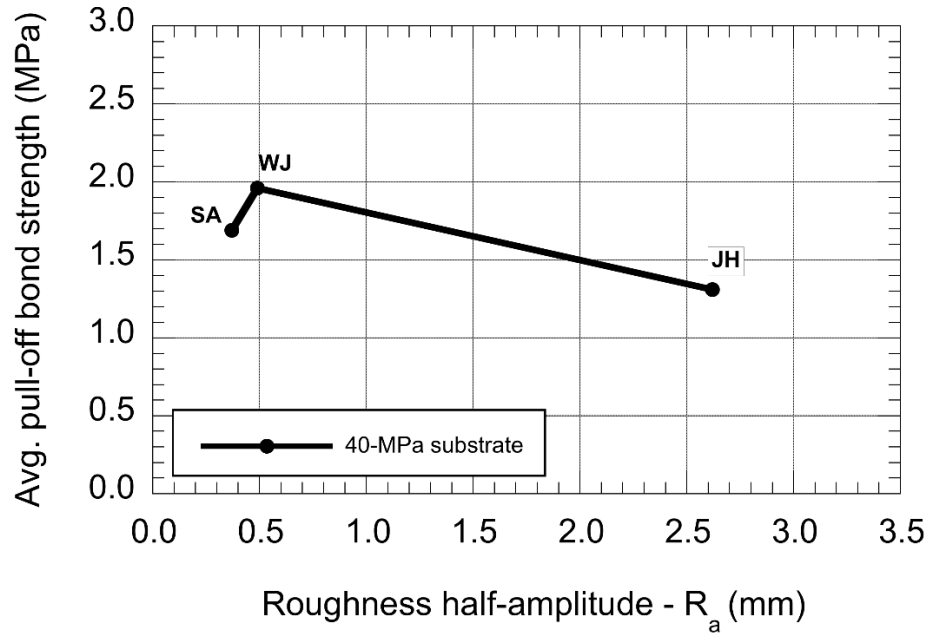
*Figure 3-17: Results of pull-off tests (ASTM C1583) performed after repair as a function of the substrate Rₐ value generated by various preparation techniques (USBR program)*
*(note: SA – sandblasting; WJ – water jetting; JH – jackhammering)*
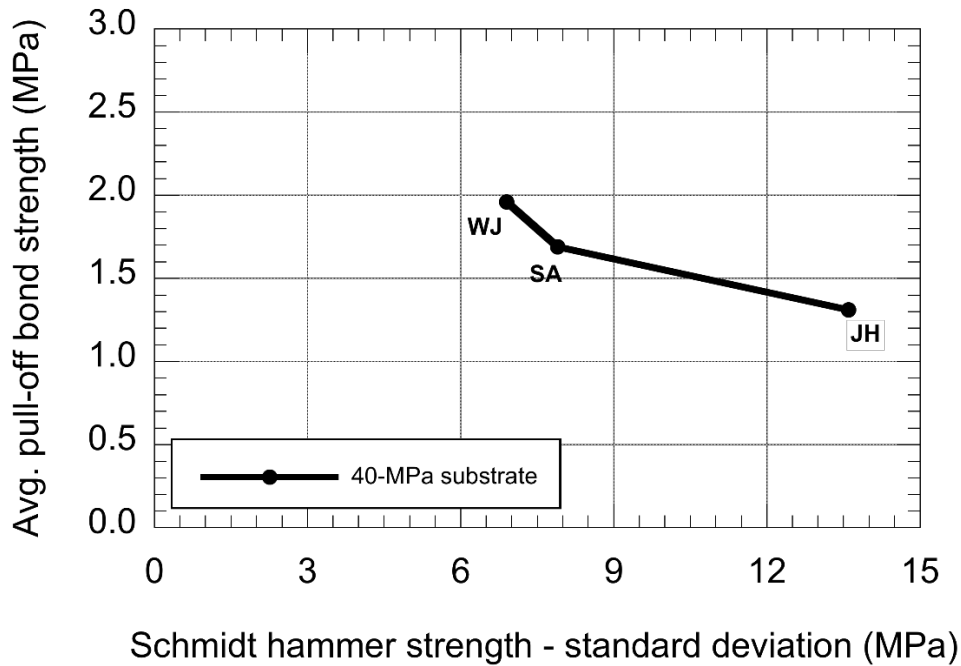


*Figure 3-18: Results of pull-off tests (ASTM C1583) performed after repair as a function of the variability of the Schmidt hammer data (ASTM C805) yielded for different surface preparations (USBR program)*
*(note: SA – sandblasting; WJ – water jetting; JH – jackhammering)*

Section 3 - Evaluation of Relationships between Tensile Bond, Shear Bond and Substrate      19
Roughness Parameters (Task 2)

*Torque testing*

The torsional shear bond test results presented in Table 3-8, Figures 3-13 and 3-14 exhibit trends that are similar to those observed for the pull-off results in terms of average bond strength, but not with regards to the failure location distribution.  In fact, contrary to the direction tension case, failure under a torsional load appears to be more or less uniformly distributed along the length of the test cores.

The relationship observed experimentally between shear bond strength and roughness is plotted on the graph of Figure 3-19.  Based on this graph, roughness appears to play a more important role in shear bond than in tension bond, which is somewhat consistent with what could be intuitively expected.  Nevertheless, when considering the shear bond to tensile bond strength ratio as a function of the $R_a$ value (Figure 3-20), the sharp increase observed for the coarse jackhammered substrates must thus be interpreted with caution.  While a positive influence of increased roughness upon shear bond strength cannot be dismissed, it appears that the damage induced in the vicinity of the surface is more detrimental to the bond in direct tension than in shear.

The shear bond to tensile bond strength ratios recorded experimentally are observed to be in the lower portion of the typical range (1.2 – 2.0) reported by Vaysburd et al. [9].  It should be stressed that in many instances, shear tests involve the use of a normal compressive force, which necessarily translates into a smaller corresponding tensile stress for a given imposed shear load, and ultimately, a larger shear-to-tension stress ratio.  In addition, when approaching the ultimate shear loading, the presence of a normal force tends to stabilize the specimen and prevent premature failure.  In the shear bond test procedure carried out as part of the present investigation, no normal force is applied against the testing surface.  This may explain why the bond values recorded in torsional shear are close to the tensile bond values determined with the pull-off experiment, in particular for flatter bond interfaces.  It may thus also explain, at least in part, the increased variability of the bond test data obtained in torsion as compared to those yielded with the pull-off test.

### 1.4. Test program conducted at Laval University

Surface roughness

The surface roughness characteristics corresponding to the five different surface preparation profiles, as obtained using the ICRI CSP index (ICRI No. 310.2R-2013) and optical profilometry respectively, are summarized in Table 3-8.

As can be seen in the table, the ICRI CSP plates merely cover the roughness values recorded for scarifying, all other techniques were out of range for the given concrete and experimental conditions.  As convenient a tool as these templates can be, with the existing scale, their use is limited to surface treatment applications where little of the existing concrete is actually removed (see Section 2).  In fact, they are intended for surface treatments, not really for repair.

Overall, the comparative roughness data determined for the various surface preparation techniques are consistent with data from a previous study [8].  Comparatively, the recorded surface roughness half-amplitude values ($R_a$) obtained by optical profilometry shifted slightly

towards the rougher side in comparison with what was obtained in the USBR program (for the corresponding preparation methods). The observed shift is likely associated with differences in the aggregates and characterization devices used in the two test programs.
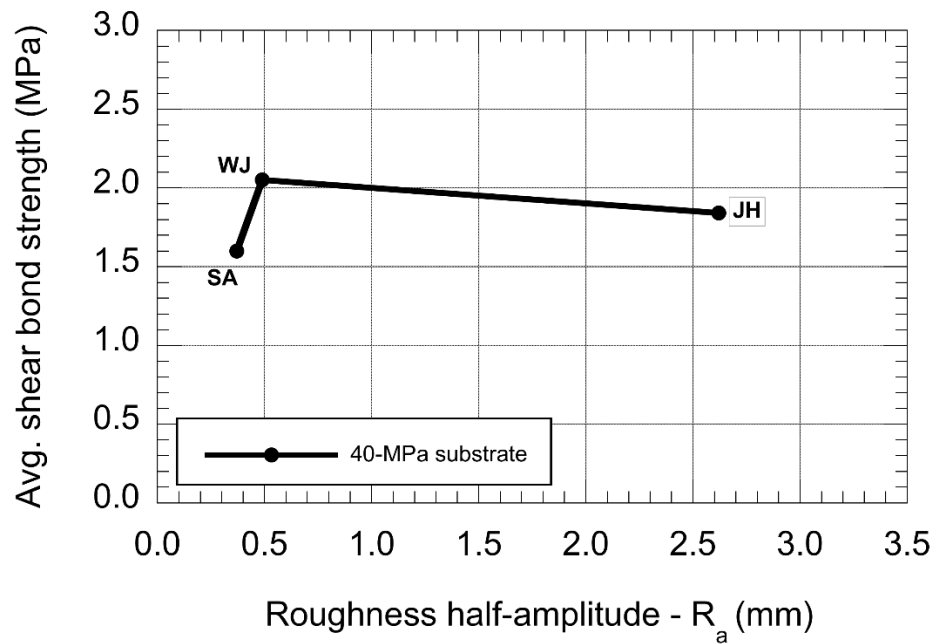


*Figure 3-19: Results of torque tests performed after repair as a function of the substrate roughness generated by various preparation techniques (USBR program)*
*(note: SA – sandblasting; WJ – water jetting; JH – jackhammering)*
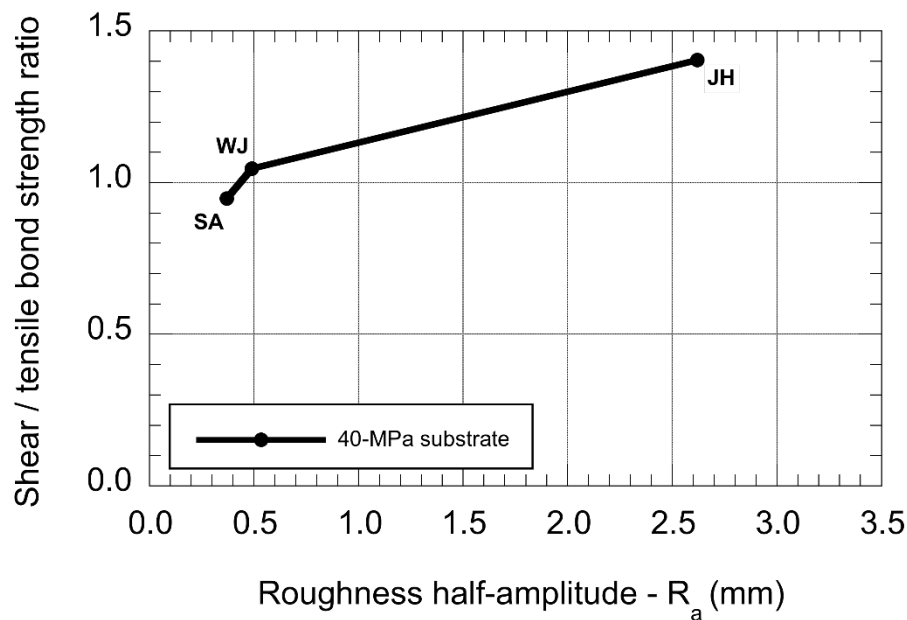


*Figure 3-20: Shear bond to tensile bond strength ratio after repair as a function of the substrate $R_a$ value generated by various preparation techniques (USBR program)*
*(note: SA – sandblasting; WJ – water jetting; JH – jackhammering)*

Section 3 - Evaluation of Relationships between Tensile Bond, Shear Bond and Substrate Roughness Parameters (Task 2)     21

*Table 3-8: Summary of surface roughness test results (UL program)*

| Test | Parameter | Substrate nominal strength | Surface preparation | | | | |
|---|---|---|---|---|---|---|---|
| | | | Scarifying | Shotblasting | Sandblasting | Water jetting | Jackhammering |
| ICRI CSP profiles (ICRI No. 310.2R-2013) | Avg. CSP index (1-9) | 20-MPa | 6.5 | > 9 | > 9 | > 9 | > 9 |
| | | 30-MPa | 6 | > 9 | > 9 | > 9 | > 9 |
| Optical profilometry | Half-amplitude $R_a$ (mm) | 20-MPa | 0.25 | 0.66 | 0.69 | 2.09 | 3.00 |
| | | 30-MPa | 0.22 | 0.62 | 0.66 | 1.54 | 2.22 |

The $R_a$ values determined experimentally are plotted on the graph of Figure 3-21. For sake of comparison, the $R_a$ values recorded for the ICRI CSP rubber templates are also displayed on this graph. The largest half-amplitude values (1.50 – 3.75 mm) were obtained with the jackhammer and water jetting, while the lowest values were recorded respectively for the scarified, the shotblasted and the sandblasted surfaces (< 1 mm). It can also be observed that for all slabs and templates, surface roughness is uniform, with most data points sitting on or close to the equality line (where the values determined in the X-direction and the Y-direction are equal).

It must then be emphasized that the *meso*-roughness level, which is directly related to the aggregate size distribution of the substrate concrete, is being considered here. The large waviness observed for instance on both water jetted and jackhammered surfaces is extracted from the calculation by filtering. Nevertheless, the recorded $R_a$ values suggest that water jetting and jackhammering both leave more and larger exposed aggregate particles than the other techniques.
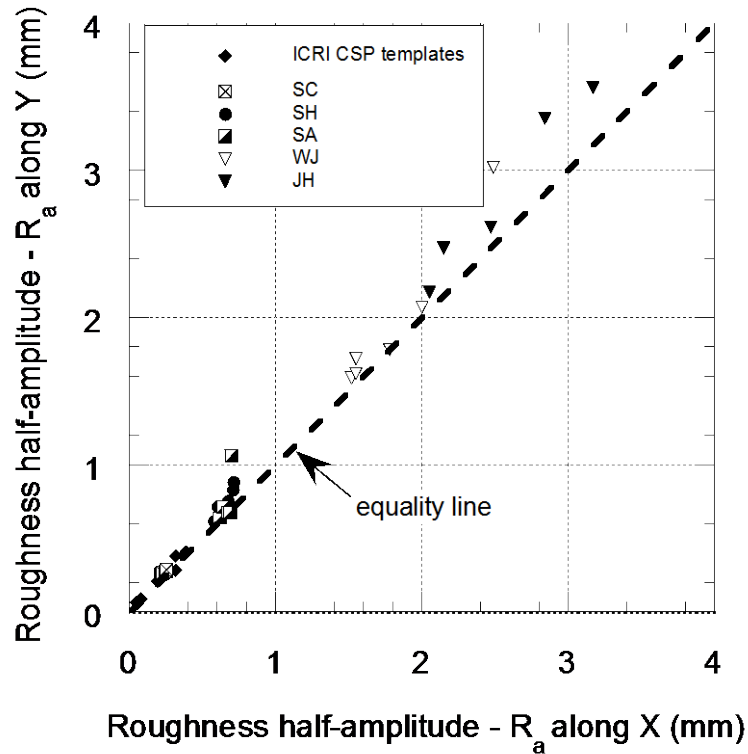
*Figure 3-21: Results of roughness evaluation performed after surface preparation by optical profilometry on both 20-MPa and 30-MPa substrates (UL program)*
*(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*

Mechanical integrity of the substrate

The average tensile strength values recorded in the substrate pull-off tests performed for the various types of preparation are summarized in Figure 3-22.  Overall, the results obtained with the 20-MPa and 30-MPa substrates respectively are consistent with the standard characterization test results summarized.  It can further be observed that for a given substrate quality, the average pull-off strength values obtained with sandblasting, shotblasting and scarifying are all close from the corresponding base concrete splitting tensile strength.  It can be asserted that the corresponding substrates were virtually undamaged by the surface preparation operations.  Actually, in most of the tests, failure occurred at the bottom of the core, far from the surface.
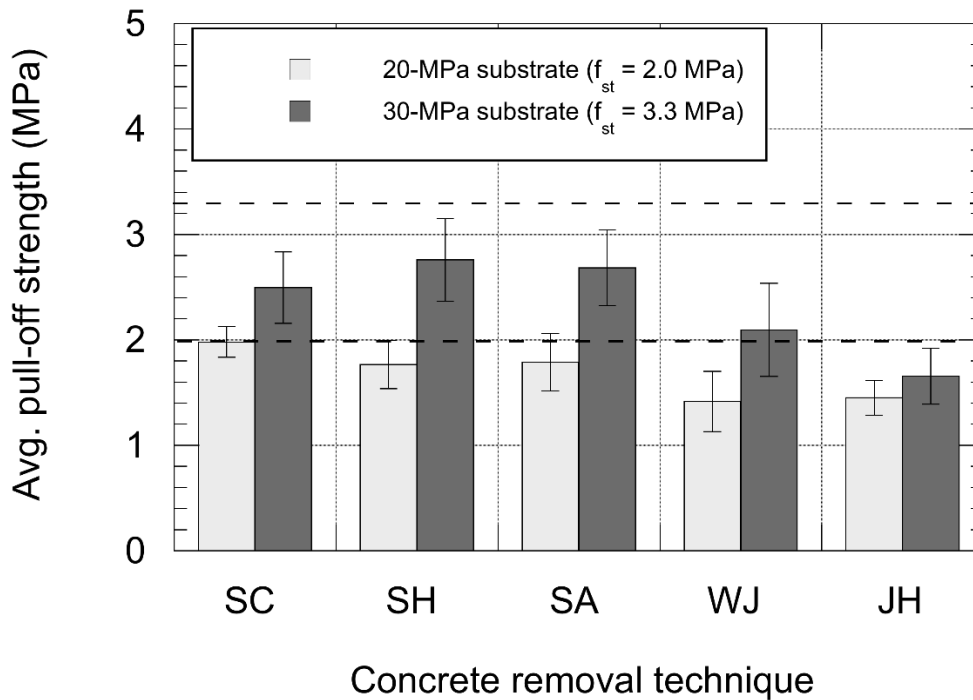
*Figure 3-22: Results of pull-off experiments (CSA A23.2-6B modified) performed after surface preparation to evaluate the mechanical integrity of the exposed concrete surface and comparison with average the splitting tensile strength ($f_{st}$) value determined for each base mixture (UL program)*
*(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*

In the case of water jetting, the lower recorded strength values are most likely not due to damage, as the bond strength test results subsequently show, but rather to a pull-off test bias for that given type of surface profile. Indeed, the waviness created by water jetting was particularly important, and although special care was taken to glue the dolly adequately and to ensure proper alignment of the testing device, it could apparently not fully compensate.

In comparison, the average cohesion values recorded for the jackhammered slabs are significantly lower, especially in the 30-MPa series. Again, this is assumed to be a consequence of surface defects induced by the hammer tip, as reflected by the prevalent number of pull-off specimen failure occurrences near the surface.

Bond strength

The detailed results of the various bond strength tests performed on the experimental slabs are presented in Tables 3-9 to 3-12 and in Figures 3-23 to 3-30.

*Pull-off testing*

Except for the slabs prepared by jackhammering, the pull-off test results (Tables 3-9 and 3-10) are close to the corresponding substrate pull-off strength values (see Figure 3-22), for both slab series. In the 20-MPa slabs, where it is particularly close, failure of the pull-off specimens occurred systematically in the substrate (again, except for the jackhammered slabs). In the 30-MPa slabs, with the difference in strength between the repair material and substrate material being smaller, failures were drawn a little more towards the interfacial zone.

On jackhammered slabs, irrespective of the substrate concrete strength, the differences observed in terms of pull-off strength magnitude and failure location are manifest. Even though lightweight hammers (15-lb) were used, the recorded pull-off strength values are significantly lower and most of the time (> 83 %), failure occurred in the interface area. As for the correspondingly weaker superficial pull-off strength values, this clearly has to be attributed to the presence of disseminated defects left within the substrate surface layer after completion of the jackhammering operations.

Concerning the relationship between pull-off strength and substrate roughness, the recorded data tend again to show that pull-off strength increases with increasing values of $R_a$, provided that no or limited damage is induced (Figure 3-31). Where the extent of damage becomes significant, as in the case here of jackhammered slabs, the positive influence of increased roughness is erased by the detrimental effects of bruising.

*Table 3-1: Summary of pull-off test results for the 20-MPa substrate series (UL program)*

| Surface preparation method | Test parameters | Location of failure | | | | Overall |
|---|---|---|---|---|---|---|
| | | Substrate | Interface | Repair | Dolly | |
| Scarifying (SC) | Number of test results | 45 | 1 | 0 | 2 | 48 |
| | (%) | (93.8) | (2.1) | (0) | (4.2) | (100) |
| | Avg. strength [MPa] | **1.69** | **1.81** | - | **1.88** | **1.70** |
| | (std. dev.) | (0.30) | (-) | (-) | (0.35) | (0.29) |
| Shotblasting (SH) | Number of test results | 48 | 0 | 0 | 0 | 48 |
| | (%) | (100.0) | (0) | (0) | (-) | (100) |
| | Avg. strength [MPa] | **1.75** | - | - | - | **1.75** |
| | (std. dev.) | (0.21) | (-) | (-) | (-) | (0.21) |
| Sandblasting (SA) | Number of test results | 47 | 0 | 0 | 1 | 48 |
| | (%) | (97.9) | (0) | (0) | (2.1) | (10) |
| | Avg. strength [MPa] | **1.75** | - | - | **1.13** | **1.74** |
| | (std. dev.) | (0.16) | (-) | (-) | (-) | (0.19) |
| Water jetting (WJ) | Number of test results | 46 | 2 | 0 | 0 | 48 |
| | (%) | (95.8) | (4.2) | (0) | (-) | (100) |
| | Avg. strength [MPa] | **1.82** | **1.78** | - | - | **1.81** |
| | (std. dev.) | (0.19) | (0.11) | (-) | (-) | (0.19) |
| Jackhammering (JH) | Number of test results | 4 | 44 | 0 | 0 | 48 |
| | (%) | (8.3) | (91.7) | (-) | (-) | (100) |
| | Avg. strength [MPa] | **1.39** | **1.18** | - | - | **1.20** |
| | (std. dev.) | (0.22) | (0.31) | (-) | (-) | (0.31) |

*Table 3-10: Summary of pull-off test results for the 30-MPa substrate series (UL program)*

| Surface preparation method | Test parameters | Location of failure | | | | Overall |
|---|---|---|---|---|---|---|
| | | **Substrate** | **Interface** | **Repair** | **Dolly** | |
| Scarifying (SC) | Number of test results (%) | 12 (25.0) | 36 (75.0) | 0 (0) | 0 (0) | 48 (100) |
| | Avg. strength [MPa] (std. dev.) | **2.48** (0.40) | **2.38** (0.29) | - (-) | - (-) | **2.40** (0.32) |
| Shotblasting (SH) | Number of test results (%) | 22 (45.8) | 20 (41.7) | 4 (8.3) | 2 (4.2) | 48 (100) |
| | Avg. strength [MPa] (std. dev.) | **2.53** (0.24) | **2.57** (0.29) | **2.62** (0.27) | **2.21** (0.11) | **2.54** (0.26) |
| Sandblasting (SA) | Number of test results (%) | 39 (83.0) | 3 (6.4) | 5 (10.6) | 0 (0) | 47 (100) |
| | Avg. strength [MPa] (std. dev.) | **2.64** (0.37) | **2.49** (0.57) | **2.95** (0.16) | - (-) | **2.67** (0.37) |
| Water jetting (WJ) | Number of test results (%) | 34 (70.8) | 13 (27.1) | 1 (2.1) | 0 (0) | 48 (100) |
| | Avg. strength [MPa] (std. dev.) | **2.64** (0.26) | **2.43** (0.38) | **2.94** (-) | - (-) | **2.59** (0.31) |
| Jackhammering (JH) | Number of test results (%) | 1 (2.1) | 47 (97.9) | 0 (0) | 0 (0) | 48 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.27** (-) | **1.47** (0.47) | - (-) | - (-) | **1.48** (0.47) |

*Table 3-11: Summary of torque test results for the 20-MPa substrate series (UL program)*

| Surface preparation method | Test parameters | Location of failure | | | | Overall |
|---|---|---|---|---|---|---|
| | | **Substrate** | **Interface** | **Repair** | **Dolly** | |
| Scarifying (SC) | Number of test results (%) | 6 (50.0) | 5 (41.7) | 1 (8.3) | 0 (0) | 12 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.06** (0.35) | **1.36** (0.46) | **1.32** (-) | - (-) | **1.24** (0.42) |
| Shotblasting (SH) | Number of test results (%) | 5 (41.7) | 6 (50.0) | 0 (0) | 1 (8.3) | 12 (100) |
| | Avg. strength [MPa] (std. dev.) | **0.87** (0.21) | **1.11** (0.38) | - (-) | **3.12** (-) | **1.18** (0.68) |
| Sandblasting (SA) | Number of test results (%) | 2 (16.7) | 9 (75.0) | 1 (8.3) | 0 (0) | 12 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.52** (0.10) | **1.22** (0.34) | **1.40** (-) | - (-) | **1.32** (0.35) |
| Water jetting (WJ) | Number of test results (%) | 4 (33.3) | 6 (50.0) | 1 (8.3) | 1 (8.3) | 12 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.98** (0.60) | **1.20** (0.54) | **2.25** (-) | **1.01** (-) | **1.53** (0.66) |
| Jackhammering (JH) | Number of test results (%) | 0 (0) | 12 (100.0) | 0 (0) | 0 (0) | 12 (100) |
| | Avg. strength [MPa] (std. dev.) | - (-) | **1.23** (0.41) | - (-) | - (-) | **1.23** (0.41) |

*Table 3-12: Summary of torque test results for the 30-MPa substrate series (UL program)*

| Surface preparation method | Test parameters | Location of failure | | | | Overall |
|---|---|---|---|---|---|---|
| | | **Substrate** | **Interface** | **Repair** | **Dolly** | |
| Scarifying (SC) | Number of test results (%) | 2 (16.7) | 8 (66.7) | 2 (16.7) | 0 (0) | 12 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.15** (0.25) | **1.90** (0.75) | **3.22** (0.71) | **-** (-) | **2.00** (0.91) |
| Shotblasting (SH) | Number of test results (%) | 1 (8.3) | 8 (66.7) | 0 (0) | 3 (25.0) | 12 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.56** (0.00) | **1.63** (0.33) | **-** (-) | **2.55** (0.71) | **1.86** (0.58) |
| Sandblasting (SA) | Number of test results (%) | 4 (33.3) | 7 (58.3) | 1 (8.3) | 0 (0) | 12 (100) |
| | Avg. strength [MPa] (std. dev.) | **2.10** (1.46) | **1.63** (0.25) | **1.39** (-) | **-** (-) | **1.77** (0.82) |
| Water jetting (WJ) | Number of test results (%) | 2 (16.7) | 5 (41.7) | 2 (16.7) | 3 (25.0) | 12 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.41** (0.37) | **1.70** (0.53) | **2.25** (0.61) | **2.48** (0.65) | **1.89** (0.61) |
| Jackhammering (JH) | Number of test results (%) | 2 (16.7) | 10 (83.3) | 0 (0) | 0 (0) | 12 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.15** (0.08) | **1.39** (0.61) | **-** (-) | **-** (-) | **1.35** (0.56) |
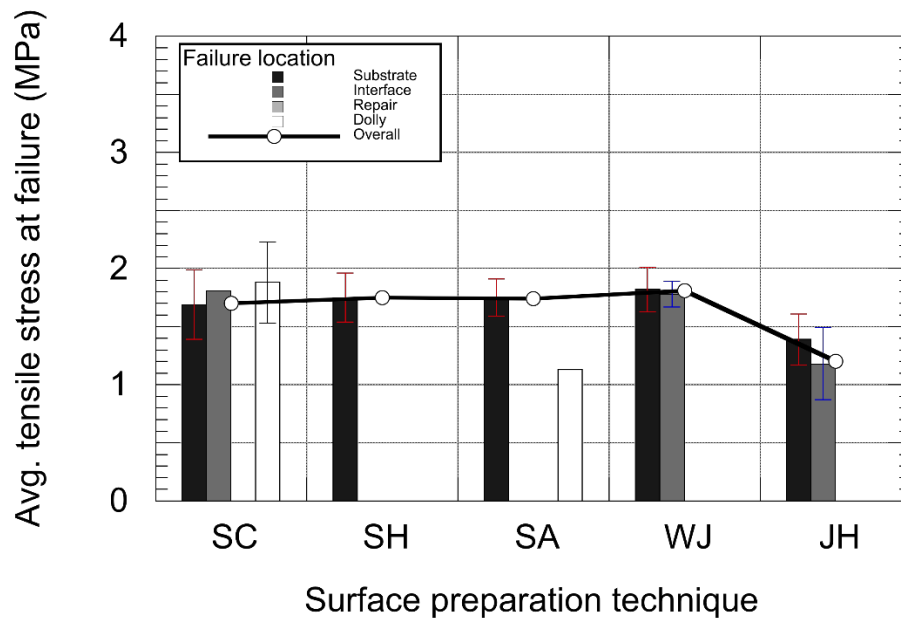


*Figure 3-23: Results of pull-off tests performed on 20-MPa substrates after repair (UL program)*
*(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*
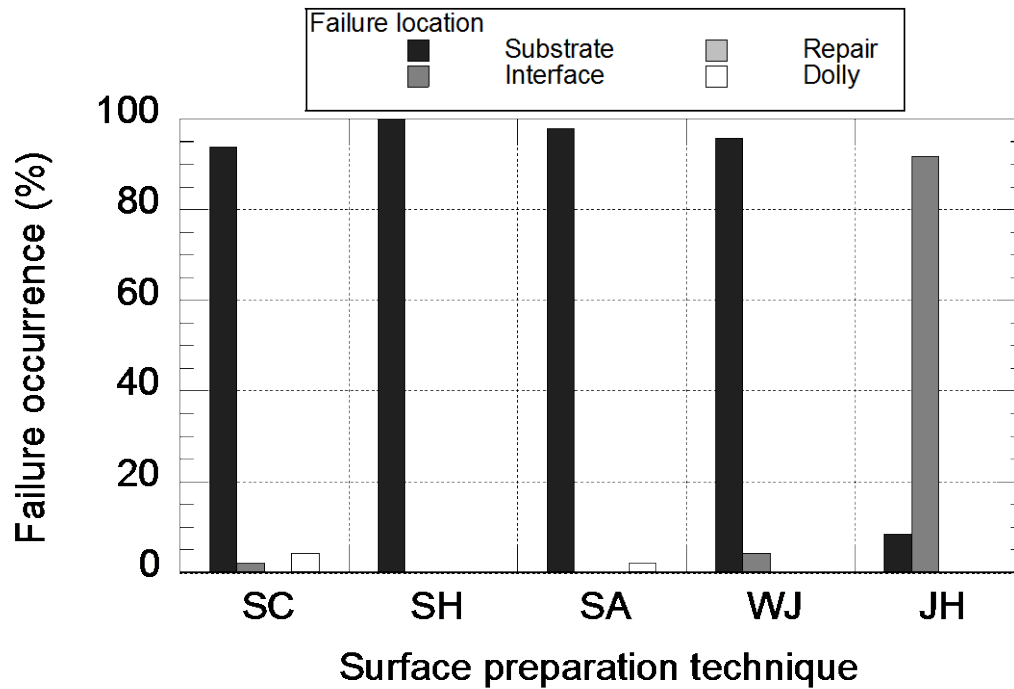
*Figure 3-24: Distribution of failure location in pull-off tests performed on 20-MPa substrates after repair (UL program)
(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*
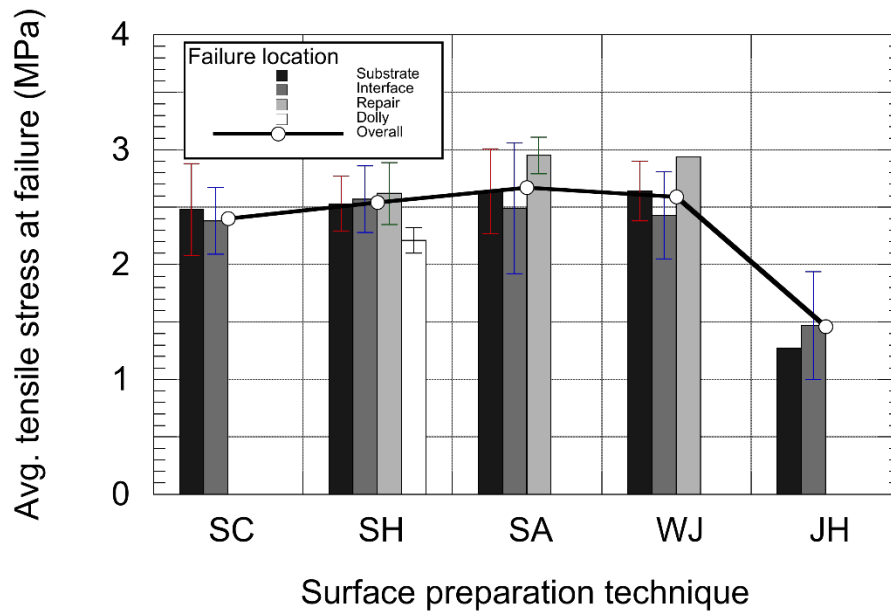


*Figure 3-25:Results of pull-off tests performed on 30-MPa substrates after repair (UL program)
(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*
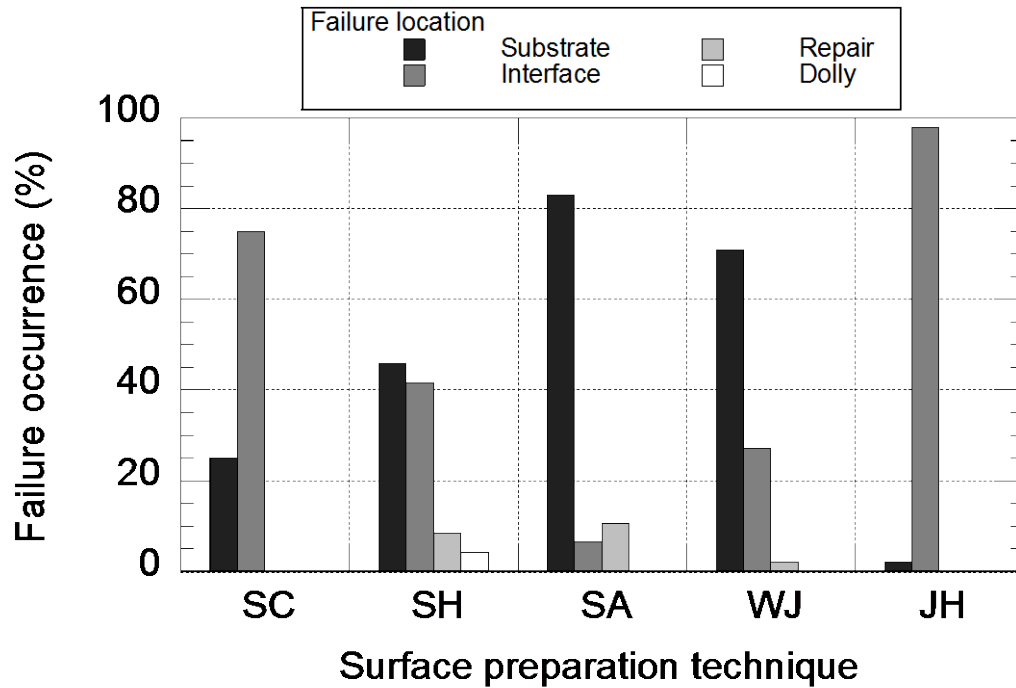
*Figure 3-26: Distribution of failure location in pull-off tests performed on 30-MPa substrates after repair (UL program)*
*(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*
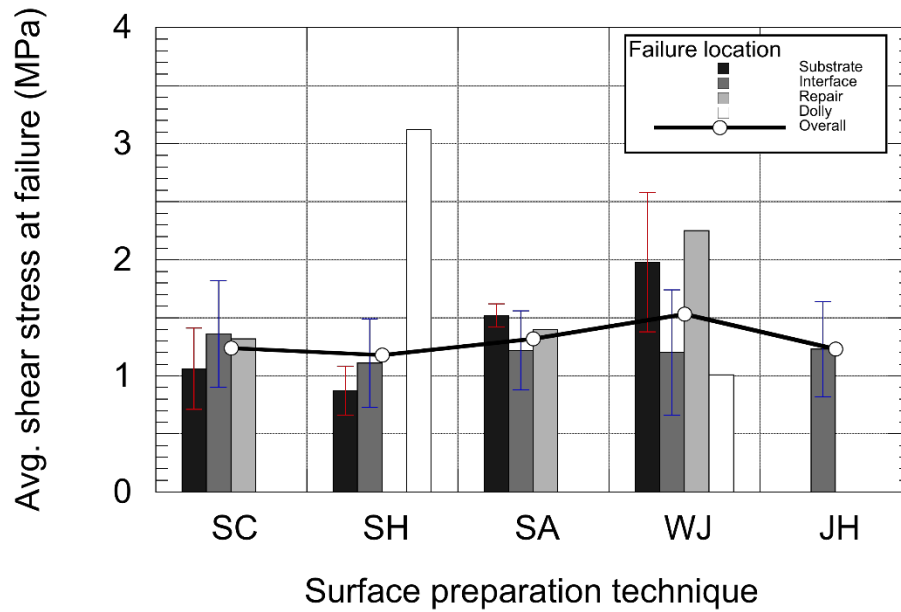


*Figure 3-27:Results of torque tests performed on 20-MPa substrates after repair (UL program)*
*(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*

Section 3 - Evaluation of Relationships between Tensile Bond, Shear Bond and Substrate     29
Roughness Parameters (Task 2)

*Figure 3-28: Distribution of failure location in torque tests performed on 20-MPa substrates after repair (UL program)*
*(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*
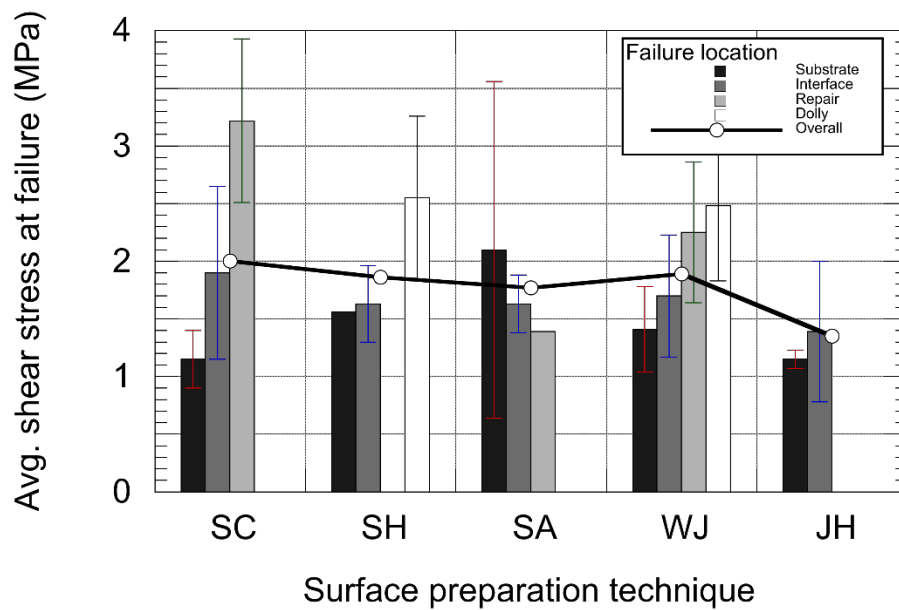


*Figure 3-29: Results of torque tests performed on 30-MPa substrates after repair (UL program)*
*(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*
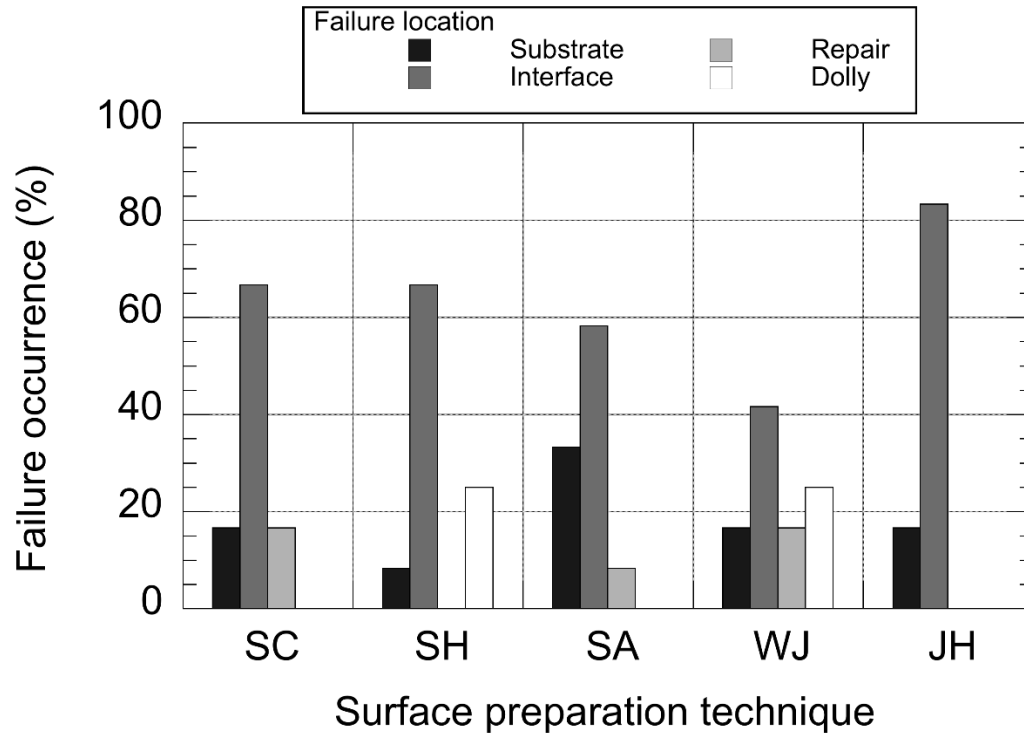
*Figure 3-30: Distribution of failure location in torque tests performed on 30-MPa substrates after repair (UL program)*
*(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*
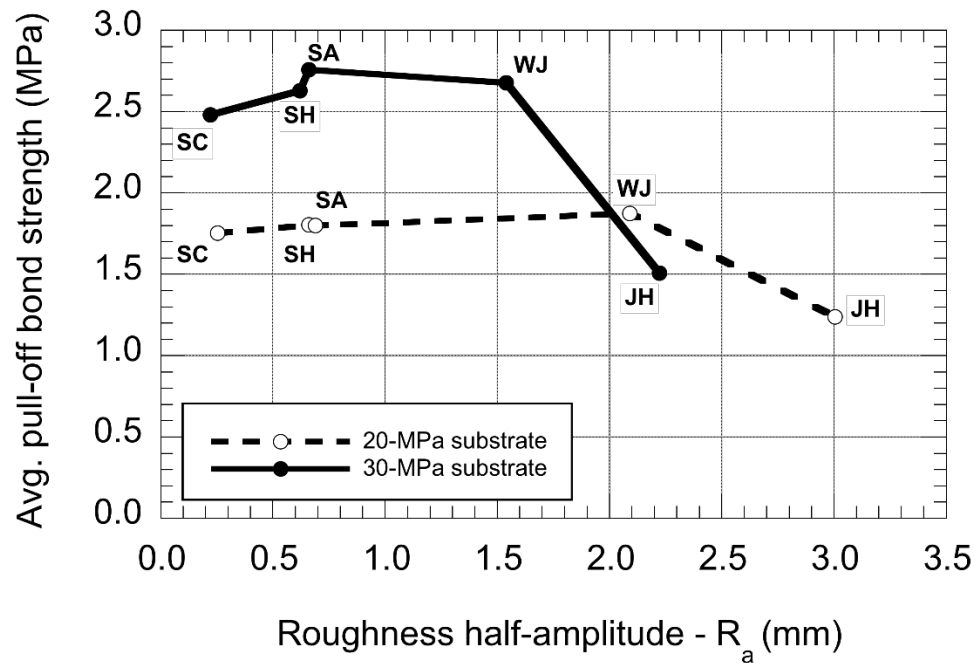


*Figure 3-31: Results of pull-off tests (ASTM C1583) performed after repair as a function of the substrate roughness generated by various preparation techniques (UL program)*
*(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*

*Direct tensile testing*

The average bond strength results in direct tension for both slab series prepared with the various investigated concrete removal techniques are displayed in Figure 3-32. The observed trends are similar to those exhibited by the pull-off tests, although the recorded values are different. In comparison with the pull-off results, the values recorded on the 20-MPa substrates shifted up, while those recorded on the 30-MPa substrates shifted down, such that the two direct tension curves are almost superimposed. Given the configuration of the tensile specimens, which promotes bond failure, the recorded values are likely more representative of the actual bond strength in tension. On that basis, the experimental results suggest that the substrate strength has little influence on the magnitude of the bonding forces developed at the interface.

As for the pull-off test results, the direct tensile test results in Figure 3-32 suggest that the average bond strength in tension slightly increases with the substrate roughness amplitude as long as little or no damage is induced by the surface treatment. This is actually confirmed by the results obtained with the artificially profiled slabs (20-MPa slab series), as illustrated in Figure 3-33. In absence of superficially induced damage, it clearly shows that increasing the surface of contact between the substrate and the repair material leads to a stronger bond, at least in tension.

*Torque testing*

The torsional shear bond test results presented in Tables 3-11 and 3-12, Figures 3-27 to 3-30 exhibit trends that are overall similar to those observed for the pull-off results in terms of average bond strength, but not with regards to the failure location distribution. Failures in torsion were not shown to be distributed along the length of the test cores as it was in the USBR program, but it was again more dispersed than the pull-off failures.

The torsional shear bond test results are presented as a function of the substrate roughness parameter $R_a$ in Figure 3-34. Both in terms of magnitude and trends, they show similarity with the pull-off data, with the substrate strength, roughness and mechanical integrity appearing as influential parameters. Based on this figure, the influence of roughness appears to be somehow subtler than in the case of pull-off testing. Hence, the relationship between the shear bond to tensile bond ratio and the $R_a$ value (Figure 3-35) does not exhibit any definite trend revealing for instance whether shear bond strength may be more or less favorably influenced by the substrate roughness than tensile bond strength. Again, in appraising such data, it must be kept in mind that the results obtained on the jackhammered surface are largely affected by the preparation-induced damage within the substrate.
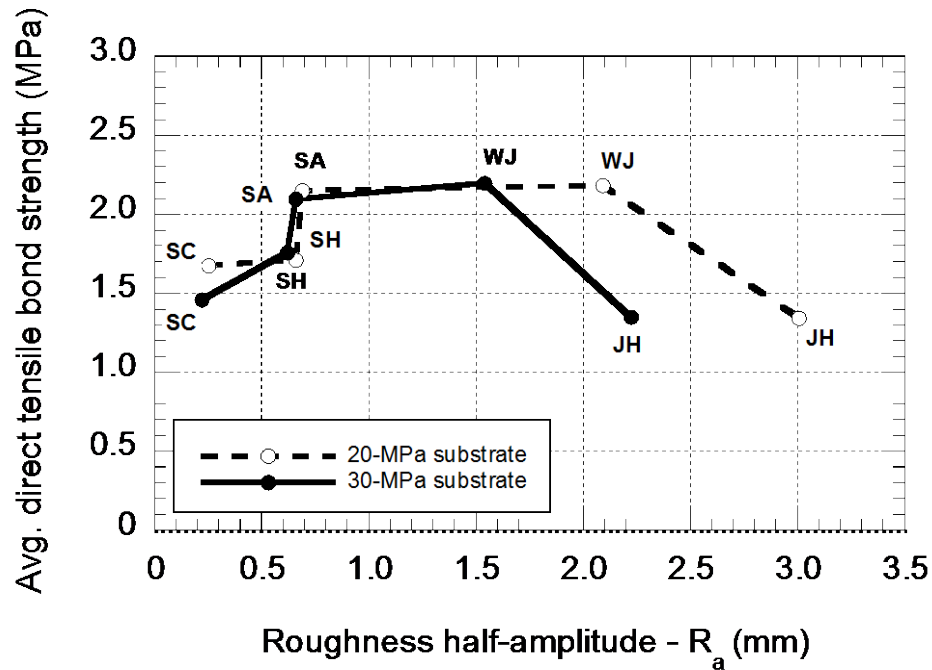
*Figure 3-32: Results of direct tensile tests (CRD-C 164) performed after repair on cores extracted from the slabs as a function of the substrate roughness generated by various preparation techniques (UL program)*
*(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*
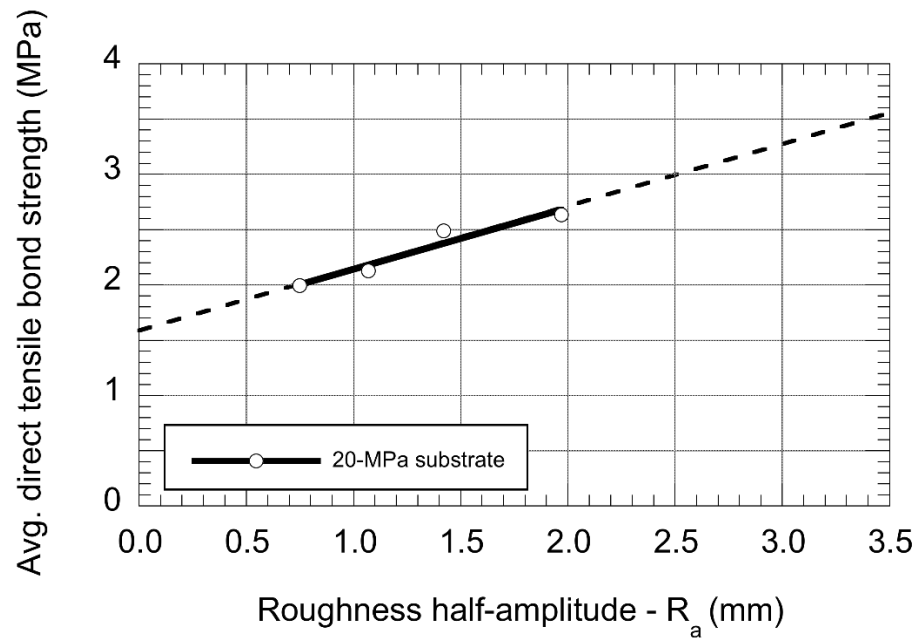


*Figure 3-33: Results of direct tensile tests performed after repair on cores extracted from the artificially profiled 20-MPa test slab (UL program)*

Section 3 - Evaluation of Relationships between Tensile Bond, Shear Bond and Substrate     33
Roughness Parameters (Task 2)
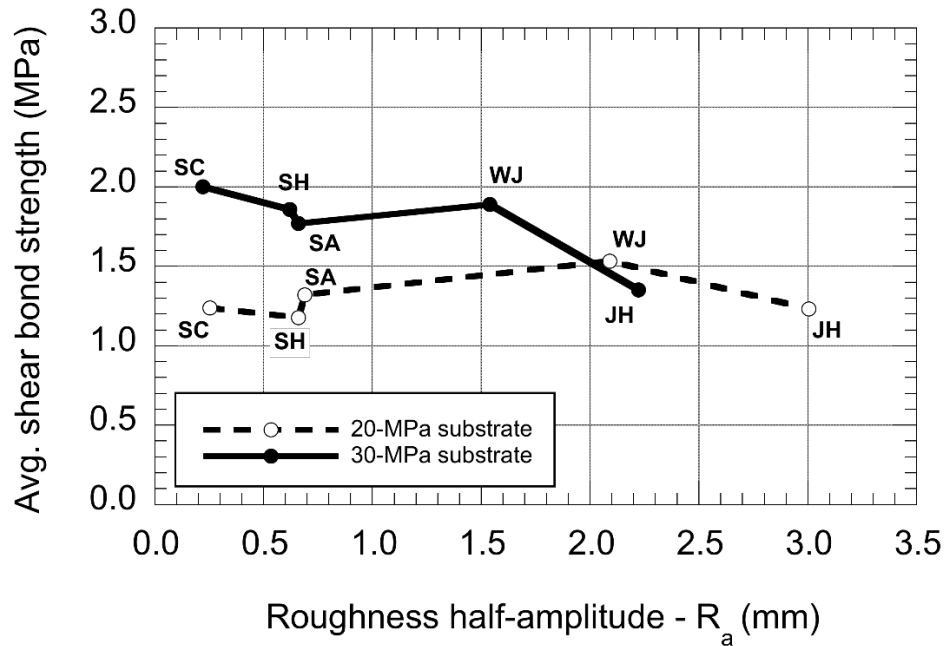
*Figure 3-34: Results of torsional bond experiments performed after repair as a function of the substrate roughness generated by various preparation techniques (UL program)*
*(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*
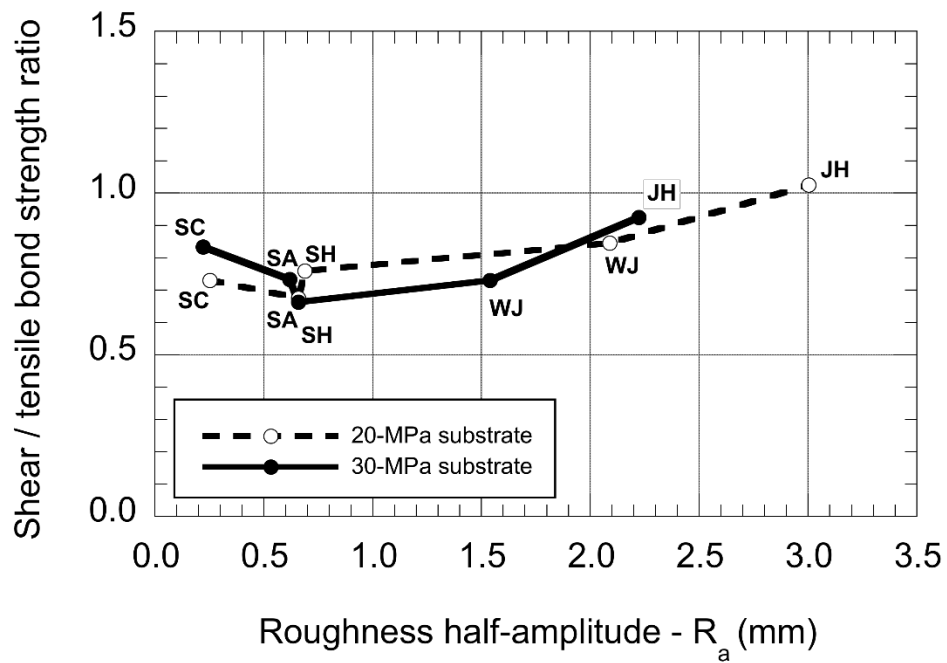


*Figure 3-35: Shear bond to tensile bond strength ratio after repair as a function of the substrate $R_a$ value generated by various preparation techniques (UL program)*
*(note: SC – scarifying; SH – shotblasting; SA – sandblasting; WJ – water jetting; JH – jackhammering)*

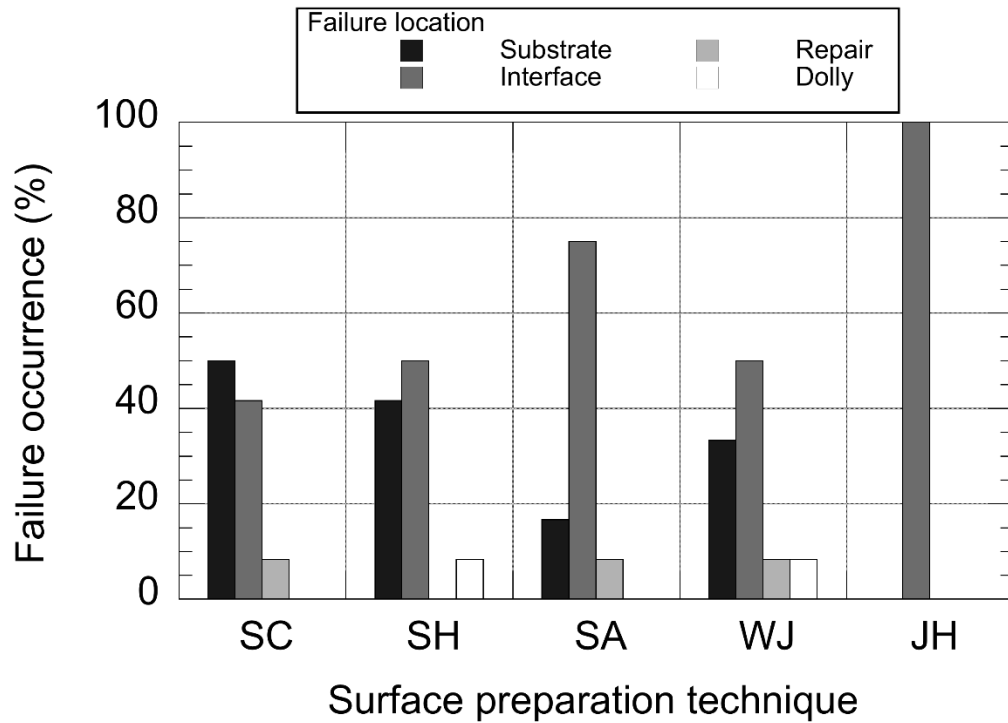It can be observed that the experimental shear bond to tensile bond strength ratios obtained in this program are even lower than those presented earlier in the test program conducted at USBR, with values ranging this time from 0.66 to barely 1.0. This finding further supports the statements made earlier related to the absence of a normal load in the torsional bond test procedure. Clearly, the experimental relationships generally reported between typical shear tests and direct tensile testing does not apply here, in the absence of a normal load.

## Conclusions

Bond strength of concrete repairs depends on a number of parameters. It has been clearly shown that when substrate-induced damage is prevented or kept below a certain level, tensile bond strength increases with the substrate coarseness. Still, one of the most important parameters apparently remains the mechanical integrity of the substrate. In that regard, it must be stressed that the use of impacting methods such as jackhammering leaves significant damage at the surface, which can easily outweigh the benefits of an increased roughness.

When considering the relationship between interfacial pull-off bond and shear bond strengths in composite repair overlay systems, the test results yielded during the experimental research and summarized herein do not exhibit the same trends as often reported or described in the scientific documentation (again, it must be stressed that reported hard data comparisons are extremely scarce). No general correlation between the two physical characteristics could be established due to the fact that the various surface preparation techniques result in different types of profiles and induced defects. The combination of these parameters influence pull-off bond and shear bond strength measurements in different ways.

Relating interface shear and tension test results in a highly heterogeneous medium such as a concrete composite system is in fact questionable, as both are subject to different combinations of bond mechanisms, which are affected to varying degrees by the interface and substrate characteristics (adhesion, friction, interface roughness and geometry, mechanical integrity of the substrate, etc.). In addition, there are some considerations inherent to the experimental test methods for bond assessment, which strongly influence the actual state of stress building up inside the material and ultimately leading to failure. For instance, many of the shear test procedures developed for concrete involve a normal force acting in at least one direction. In the investigated torsional bond test procedure, no such normal force is acting, making the comparison of the data difficult with some previously published works.

Overall, it can be concluded that bond strength testing is a convenient and useful in-situ method for evaluating the mechanical integrity of the concrete surface prior to repair, as well as the repair bond strength. A reliable evaluation of these properties can be obtained, provided that a minimum number of tests are performed, with adequate equipment and properly conducted test procedure.

Still, the tensile pull-off test has potential shortcomings, which must be considered in the analysis of any result. The first potential problem – addressed subsequently in this report – is misalignment of the pulling force, which leads to stress concentrations and can exert a significant influence on recorded bond strength values. Another problem that is commonly encountered with tensile pull-off tests is that failure often occurs outside the interfacial zone, either in the repair

layer or within the existing substrate.  When such a failure occurs, the recorded maximum stress merely represents a lower bound value for interface bond strength.  A third problem is that the coring operation may damage the interface between the repair and the substrate, which is likely to affect bond and thereby reduce the measured pull-off strength.

Among the two bond test methods compared in the present investigation, the pull-off test is the only one commonly used in practice because the equipment is widely available and it is relatively easy to carry out in the field.  Torsional tests may also be performed on site without too much difficulty, but they are very seldom used for a number of reasons, the most significant being the non-existence of specifications in that respect in repair guidelines and the lack of a standard test procedure.

Further work and analysis including bond ageing will allow more definite conclusions which will allow for broader recommendations for concrete surface preparation prior to repair.

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ASTM C496 / C496M-11, Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens, ASTM International, West Conshohocken (PA), USA, 2004, www.astm.org

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ASTM C1583 / C1583M-13, Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method), ASTM International, West Conshohocken (PA), USA, 2013, www.astm.org

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CRD-C 164-92, Standard Test Method for Direct Tensile Strength of Cylindrical Concrete or Mortar Specimens, U.S. Army Engineering Research and Development Center (ERDC), Vicksburg, Miss, USA, 1992, http://www.publications.usace.army.mil

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CSA A23.2-13C, Splitting Tensile Strength of Cylindrical Concrete Specimens, A23.1-14/A23.2-14 – Concrete Materials and Methods of Concrete Construction / Test Methods and Standard Practices for Concrete, CSA Group, Toronto, ON, Canada, 2014, http://www.csagroup.org

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# Section 4 - Evaluation of the Effect of Misalignment upon Pull-Off Testing Results (Task 3)

# Contents

# Figures

# Tables

# Section 4 - Evaluation of the Effect of Misalignment upon Pull-Off Testing Results (Task 3)

This section reports results of Task 3. The experimental work was carried out both at the University of Liège, ArGEnCo Dept, GeMMe Building Materials, Belgium and at the U.S. Bureau of Reclamation, Denver (CO), U.S.A.

## Introduction

Repairing and overlaying of deteriorated concrete structures are intended to extend their useful service life, to restore their load-carrying capacity and stiffness, and/or sometimes to increase their load-bearing capacity [1]. In order to satisfactorily achieve any of these objectives, full composite action of the repaired structure is a prerequisite, which implies the development of a sufficiently strong and lasting bond between the existing substrate and the newly cast material [2,3].

The concrete repair process usually involves the removal of deteriorated or contaminated material and surface preparation prior to application of a repair material [4]. The residual surface characteristics can significantly affect the bond strength and long-term performance of a repair system. Although it is not a common practice yet, mechanical integrity of the prepared concrete substrate should be assessed prior to repair as part of QC operations [5-11].

The pull-off test is a simple and effective test for evaluating both the mechanical integrity of the substrate prior to repair [12, 13] and the interface bond strength in the composite repaired structure. As any other direct tensile loading experiment for concrete, the results yielded with test procedure are sensitive to different parameters. In fact, it is even more sensitive because it is carried out in field conditions. In a previous research effort by some of the authors [12, 14], the influence of different test parameters upon the recorded strength was investigated, namely the dolly size (thickness, diameter), the core drilling depth, the loading rate, and the number of tests. Diameter of the dolly and core depth were found the most significant parameters affecting the measured tensile strength [15-19]. Geometry of the dolly and core drilling depth into the substrate were also found to be critical factors when testing for bond in repair systems [16].

Another potentially influential parameter of the pull-off test, namely the test alignment, has not received much attention yet. Still, the primary requirement in any direct tension test method is to ensure the pulling force is aligned with and parallel to the specimen axis at all times in order to avoid bending effects. Two main causes may usually induce misalignment in a pull-off experiment [20]: inclination of the core axis caused by inaccurate core drilling (Figure 4-1a)) and inclination of the pulling force caused by inaccurate positioning of the dolly (Figure 4-1b)). Real world, on-site conditions often limit the capability of the personnel performing the test to avoid misalignment situations. Pull-off test misalignment very often arises from difficult on-site conditions, such as a highly irregular support preventing a proper installation of the drilling system, leading to inaccurate coring. Special devices can help limiting the risk for loading

misalignment.  For instance, with the *Limpet* device, the load is applied through a guiding rod [13].

Austin et al. [14] investigated the effect of misalignment on recorded pull-off strength data.  The average eccentricity in their experiments was 1.5 mm [0.059 in.] at a depth of 50 mm [1.97 in.], translating into an angle of inclination of 1.7°.  The study concluded that such a misalignment caused an increase in maximum stress of the order of 20% at the core periphery [6].  Cleland and Long [15] performed numerous tests on cores drilled to a depth up to 40 mm into the repair substrate and inclination to the vertical of up to 20° in order to evaluate what effect it has on the measured pull-off bond strength.  The authors proposed a correction factor to be applied to the measured results based on the magnitude of the inclination angle:

$$F_{lr} = \frac{1}{[1-(\frac{8 \cdot \tan \alpha}{D}) \cdot y]} \tag{4-1}$$

where parameters $\alpha$, $D$, $y$ are the angle of inclination of the coring axis (with respect to an axis normal to the surface), the core diameter, and the coring depth respectively, as shown in Figure 4-2.



a) Core axis inclination                                       b) Load inclination
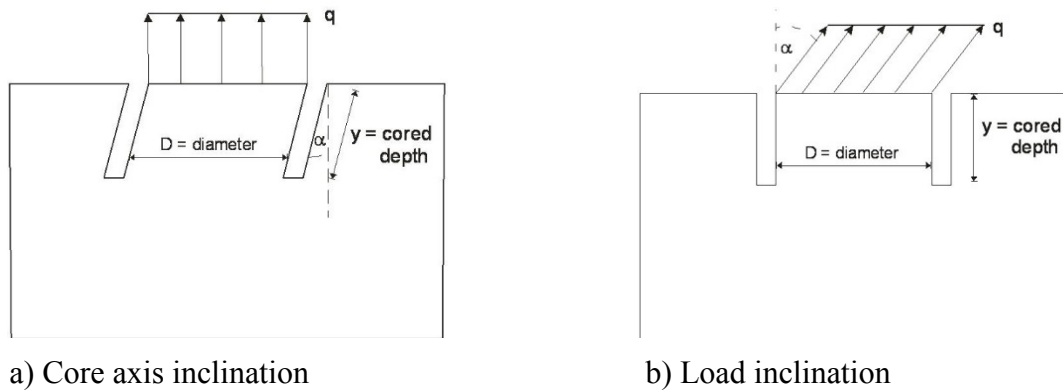
*Figure 4-1: Sources of misalignment in a pull-off test*
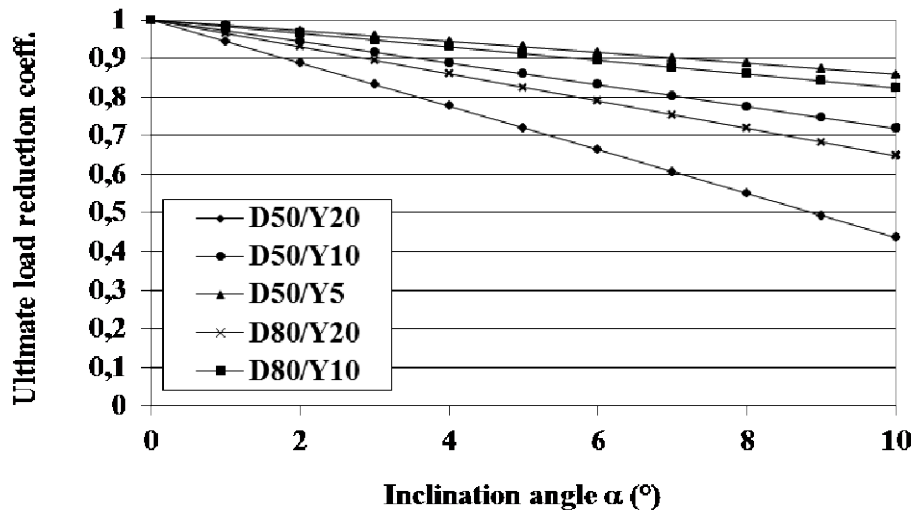


*Figure 4-2: Influence of the load inclination (from Cleland [15])*

Misalignment in pull-off tests may have a substantial influence upon test result for angles of inclination of more than 5° (Figure 4-2).  Reduction in core depth or increase in dolly diameter tends to minimize the negative effects of misalignment.  It should be stated, however, that the above conclusions are strictly theoretical in nature, as they do not take into account such factors as potential stress relaxation and the possibility that the core brittle zones are not necessarily corresponding to the stress concentration zones.

These are only geometrical and theoretical considerations.  The research work reported in this paper was intended to verify these conclusions by means of numerical simulations and experimental assessment.

## Objectives

Practical experience with in-situ pull-off testing shows that it is next to impossible to drill cores exactly at 90° to the surface and install dollies with the adhesive perfectly parallel to the tested concrete surface, even with the greatest care.  Moreover, a misalignment angle up to 5 degrees cannot be easily detected by human eye.  In order to evaluate the pull-off test result sensitivity to theses parameters, an experimental program aiming at answering the following questions was undertaken:

- What is the influence of minor load misalignments, i.e. within naked-eye detection capability, upon pull-off strength test results?
- Do coring and pulling load misalignments influence the results in different ways?

The results are anticipated to provide guidance towards improved reliability of pull-off strength test results and adapted means, if required, to ensure that the test results are valid.

## Methodology

The objective of this program was to evaluate the effect of coring and/or load misalignment upon the results yielded in pull-off tests, either for the assessment of (a) quality/integrity of a concrete substrate (monolithical), or (b) bond strength in a repair system (composite).  A theoretical analysis based on finite-element numerical calculations was first carried out to determine whether the core axis and load misalignment could influence the pull-off test results in a different fashion and assess the overall sensitivity of the results to the experimental bias.  Test programs were then conducted in the laboratory, involving experiments on both monolithic concrete substrates and composite repair systems.

The following parameters were addressed in the numerical analysis and/or laboratory experiments:

- Coring axis inclination angle;
- Pulling force inclination angle;
- Coring depth into the substrate.

The numerical and experimental test programs are summarized in Table 4-1. In each case, the test parameter values were selected to cover the range of possibilities encountered in practice: coring / pulling misalignment is investigated up to an angle that can be detected by the naked eye, whereas coring depth values are representative of most common standard procedures for pull-off testing.

*Table 4-1: Numerical and experimental test program variables*

| Test parameter | Numerical simulations | Laboratory tests | |
|---|---|---|---|
| | Monolithical slab | Monolithical slab | Repaired slab |
| Coring axis inclination angle | 0°, 2°, 4° | 0°, 2°, 4° | 0°, 2°, 4° |
| Pulling force inclination angle | 0°, 2°, 4° | 0° | 0° |
| Coring depth | 15 mm, 30 mm | 15 mm, 30 mm | 100 mm |

## 1.5. Numerical calculations

Finite element (FEM) calculations were performed using the *Lagaprogs* software [21] (tool developed at the University of Liège, Belgium) to predict the stress development within and around the cored area in a concrete substrate, assuming a perfectly elastic behavior, isotropic concrete properties, and isothermal conditions [22]. With these assumptions, it was not possible to evaluate the theoretical ultimate load and the maximum load considered in the analysis was limited to 50 percent of the ultimate load (corresponding to a testing stress of 0.50 MPa [72.5 psi]).

The pull-off testing experiment was addressed as a two-dimensional plane strain problem. The typical boundary conditions and loading scheme considered in the simulations are presented in Figure 4-3. The load was assumed to be distributed uniformly over the specimen top surface, implying that the results are not influenced by the dolly material characteristics and geometry. Figure 4-4 shows an example of the mesh used for the FEM-based simulations (example shown: angle of inclination of 4° and a core depth of 30 mm [2.36 in.]). The 2-D analysis was performed over the longitudinal cross section. As can be seen in the figure, three different mesh sizes were used depending on the area: 1) within the core and below; 2) in the slab outside the core; 3) immediately below the saw cut. The mesh implemented within and right below the cored area was denser than in the surrounding slab bulk concrete, in order to study more finely the local stress distribution in the critical areas, especially in the vicinity of the cut. An even finer squared mesh was used immediately below the saw cut (under points A and B in Figures 4-4 and 4-5), the size of the element corresponding to the thickness of the saw blade.
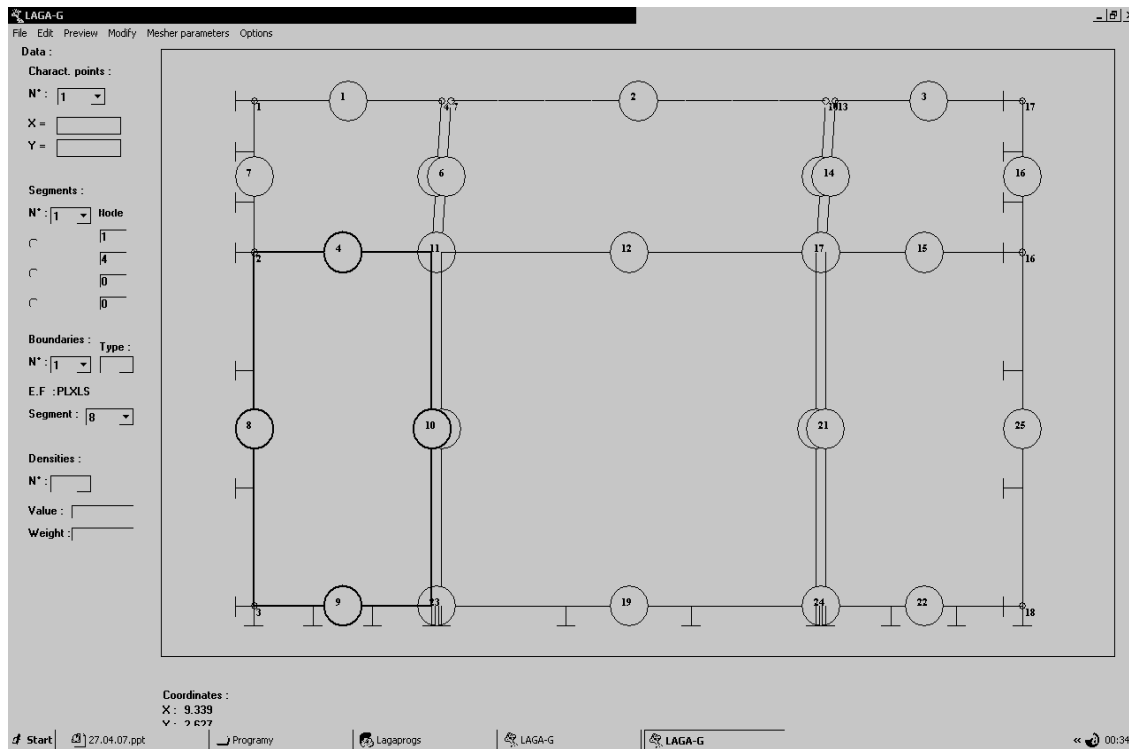
*Figure 4-3: Example of boundary conditions used in the analysis (case: pulling load with an angle of inclination 4°; core depth of 30 mm)*

The concrete physical characteristics assumed in the analysis were the following:

- Elasticity modulus: 30 GPa [4350 lb/in²];
- Poisson ratio: 0.20;
- Density: 2500 kg/m3 [4215 lb/yd³];
- Test load to yield an average stress of 1 MPa [145 psi]: 7.85 kN [1,770 lbs].
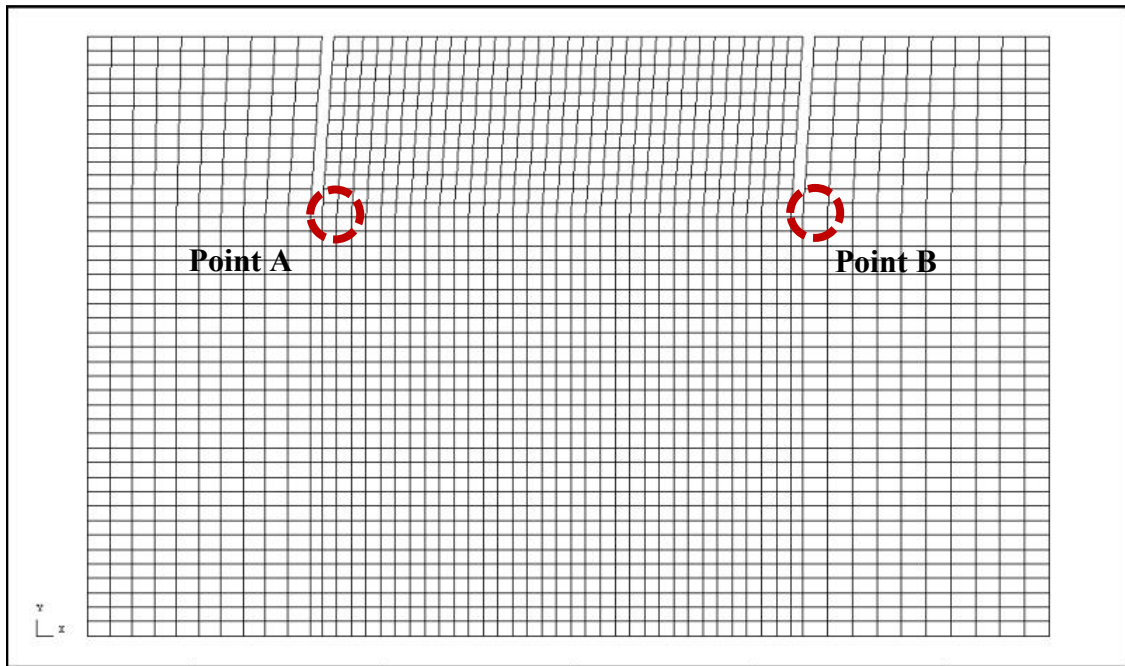
*Figure 4-4: Example of FEM mesh used in the analysis (case: pulling load with an angle of inclination 4°; core depth of 30 mm)*
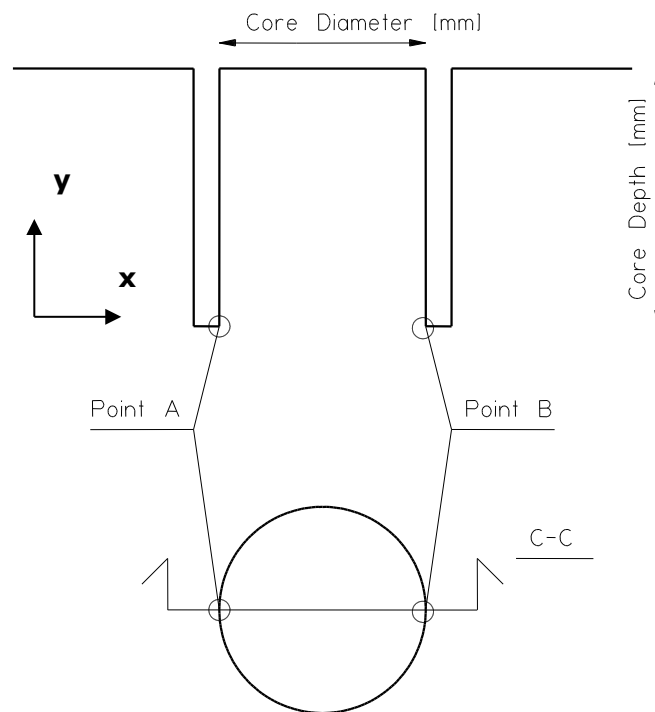


*Figure 4-5: Geometry and points (A and B) of analysis*

Analysis of the stress distribution in the critical areas of the cored substrate is expected to help evaluate the sensitivity of test results to misalignment and to determine whether load inclination and coring axis shift exert similar influence.

## 1.6. Laboratory experiments

The laboratory experiments were carried out in separate test programs conducted respectively at the University of Liège, Belgium (ULg) and at the U.S. Bureau of Reclamation (USBR), Denver (CO), USA. In the ULg test program, tests were performed on monolithic test slabs to assess the influence of misalignment on tensile pull-off strength data and to compare the results with modeling. In the USBR test program, tests series were conducted on repaired slabs.

Experiments on monolithic test slabs - ULg program

A series of six 600 × 400 × 100 mm [23.6 × 15.8 × 3.94 in.] concrete test slabs were prepared for Part I using three different ordinary Portland cement concrete mixtures, C30/37, C40/50, and C50/60, named after their respective design strength in MPa units (cylinder strength / cube strength). The concrete mixture composition details are summarized in Table 4-2. During the initial 48-hour period after casting, the slabs were covered with polyethylene (wet burlap inserted after 24 hours). At 48 hours, they were demolded and stored in lime-saturated water up to 28 days. Five pull-off tests have been carried out for each concrete composition.

The three mixtures were characterized for compressive strength at 28 days. The results are summarized in Table 4-3.

*Table 4-2: Concrete mixture compositions (ULg program)*

| Constituent / characteristic | | Mixture | | |
|---|---|---|---|---|
| | | C30/37 | C40/50 | C50/60 |
| CEM I 52,5N | [kg/m$^3$] | 275 | 325 | 375 |
| Water | [kg/m$^3$] | 192 | 186 | 182 |
| Crushed sand (0-2 mm) | [kg/m$^3$] | 765 | 729 | 676 |
| Crushed limestone (2-8 mm) | [kg/m$^3$] | 255 | 230 | 206 |
| Crushed limestone (8-14 mm) | [kg/m$^3$] | 569 | 576 | 601 |
| Crushed limestone (14-20 mm) | [kg/m$^3$] | 390 | 401 | 412 |
| W/C | | 0.70 | 0.57 | 0.49 |

Note: 1 kg/m$^3$ = 1.685 lb/yd$^3$; 1 mm = 0.03937 in.

*Table 4-3: Compressive strength determination at 28 days (ULg program)*

| Concrete mixture | $f_{c\ 28d}$[1] [MPa] (psi) | Standard deviation $s_n$ [MPa] |
|---|---|---|
| C30/37 | 50.1 (39.6) | 1.5 |
| C40/50 | 60.9 (48.1) | 1.0 |
| C50/60 | 65.4 (51.7) | 1.9 |

[1]  Tests performed on 150×150×150 mm cubes per EN 12390-3; each data corresponds to the average of 5 test results; equivalent 150×300-mm cylinder strength in parentheses
Note: 1 MPa = 145.0 psi.

After 28 days of moist curing, the concrete slab surfaces were prepared by sandblasting for pull-off testing.  The surface roughness was then evaluated with the sand-patch test method (EN 13036/EN 1766/ASTM E 965).  The texture depth values recorded for the three different concrete mixtures were comparable, the overall average being equal to 0.90 mm [0.035 in.].

Consistent with the numerical analysis conducted previously, EN 1542 tensile pull-off tests were conducted on test specimens prepared with different core depths and inclinations.  Core depths of 15 mm [1.18 in.] and 30 mm [2.36 in.] and coring axis inclination angles of 0°, 2° and 4° were again evaluated.  The different core inclinations were achieved using the special device shown in Figure 4-**6**a), which allows controlling the inclination of the core drill axis (Figure 4-6b)) with a precision of 0.1°.  Taking into account the maximum aggregate size of the concrete mixtures (20 mm), 80-mm diameter cores were drilled for pull-off testing (80 mm diameter and 30 mm [2.36 in.] thick steel dollies).  Steel dollies were carefully installed using epoxy resin (Figure 4-6c)) and the pull-off test device was then positioned on the concrete substrate (Figure 4-6d)) [12].  Prior to testing, the adhesive was allowed to cure for 24 hours.  Once the testing rig was installed and connected to the dolly, the pulling load was increased at a constant rate of 0.05 MPa/s [7.25 psi/s] until failure.
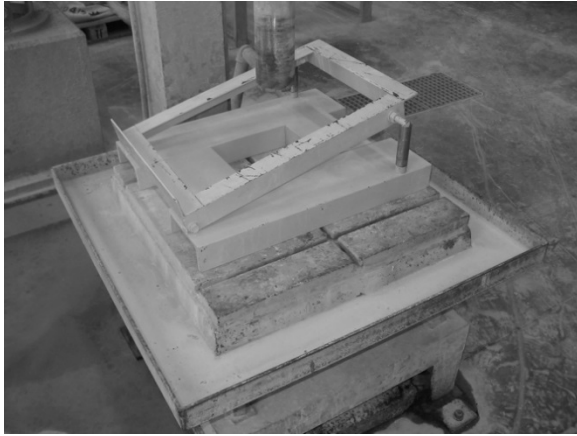
a) Special device for controlling the coring axis inclination

b) Slab positioning for coring at an angle of 4°

c) Dolly installation

d) Positioning of the pull-off test device

*Figure 4-6: Pull-off test preparation (ULg program)*

In order to better appraise the results in view of pull-off test variability, series of complementary direct tensile strength test were performed on cores extracted from the test slabs.

After each pull-off test, the fracture surfaces were carefully examined. Exposed aggregate area has been selected as criteria for analysis in trying to find a possible correlation between low experimental pull-off strength values and the lack of adhesion between the paste and aggregates.

Experiments on repaired test slabs - USBR program

The test program conducted at USBR is summarized in Table 4-4. More details pertaining to the test variables, the test specimens, the surface preparation techniques and the test methods are provided in the following paragraphs.

A series of 12 concrete slabs (46 in. by 22 in. by 6 in., 1170×560×150 mm) were manufactured for the test program (Figure 4-7). The slabs were cast using a 40-MPa ready-mix concrete. The basic properties of both mixtures are displayed in Table 4-5. The slabs were exposed to drying at least six months to achieve relative dimensional stability, after what surface preparation was performed.

Three of the most common surface preparation techniques were selected for investigation: sandblasting (SA), water jetting (WJ), and jackhammering (JH), with the characteristics provided in Table 4-4. Sets of four base slabs were prepared with each of these techniques, as shown in Figure 4-8.

*Table 4-4: Experimental details (USBR program)*

| Item | Details |
|---|---|
| **Test specimens**<br><br>• Base slabs | - 1170×560×150 mm base concrete slabs<br>- 1 slab series:  12 slabs prepared with 40-MPa OPC concrete<br>- base slabs moist cured for 3 days after casting and exposed to drying for more than 6 months prior to repair |
| • Repaired test slabs | - repairs performed on the slab series using 3 different surface preparation methods and pre-wetted to *SSD*: 75-mm thick overlays with a 40-MPa OPC concrete mixture |
| **Investigated surface preparation techniques**<br>(surface prep. prior to repair) | - sandblasting (SA)                               (4 slab per series)<br>- 15-ksi handheld water jetting (WJ)      (4 slab per series)<br>- 15-lb handheld jackhammering (JH)    (4 slab per series) |
| **Characterization test methods for**:<br><br>• Bond strength (28-d) | - Pull-off test<br>- *Torque test* (addressed in section 2 of this report) |



*Figure 4-7: Preparation of test slabs (USBR program)*

After surface preparation, evaluation of surface integrity and characterization of surface roughness were performed.  The slabs were then repaired (75-mm (3 in) overlay) with the same 40-MPa concrete mixture.  The repair concrete mixtures properties are also summarized in Table 3-2.  The repaired specimens were moist-cured for 3 days, after what they were air-dried for at least 28 days, until the bond strength tests were carried out.

*Table 4-5: Concrete mixture characteristics and mechanical properties (USBR program)*

| Material | Test slab concrete mixtures | Repair concrete mixture |
|---|---|---|
| Nominal strength | 40 MPa (6000 psi) | 40 MPa (6000 psi) |
| Mixture characteristics | ASTM Type I cement 14-mm coarse aggregates | ASTM Type I cement 14-mm coarse aggregates |
| Fresh concrete properties | | |
|     Slump (mm) | 75 | 75 |
|     Air content (%) | 3.2 | 4.5 |
| Compressive strength[1] (MPa) | | |
|     7 d | - | 34.1 |
|     28 d | 38.7 | 39.2 |
| Splitting tensile strength[2] (MPa) | | |
|     7 d | - | 3.8 |
|     28 d | 3.1 | 4.4 |
| Elastic modulus[3] (GPa) | | |
|     7 d | - | 24.2 |
|     28 d | - | 26.1 |

[1]ASTM C39; [2]ASTM C496; [3]ASTM C469.

All 12 repaired test slabs were characterized for bond strength with a combination of pull-off tests and torsional shear tests (note: the latter are addressed in section 2 of this report) performed at least 28 days after placement of the overlays.  The template for the different tests is presented in Figure 4-9.  The tests were performed at specific locations from one test slab to the other, selected in such a way to assure better reproducibility of the results.  The pull-off strength tests were performed in accordance with the CAN/CSA A23.2-6B procedure, for the following coring axis inclination angles: 0°, 2° and 4°.  The angle of inclination was obtained by using the experimental setup shown on Figure 4-10, which allowed to core with precisely controlled inclination using mobile drilling machinery.  Coring was performed through the overlay and 25 mm (1.0 in) into the substrate.

| a) Sandblasting | b) Water jetting (15 ksi) | c) Jackhammering (15 lb) |
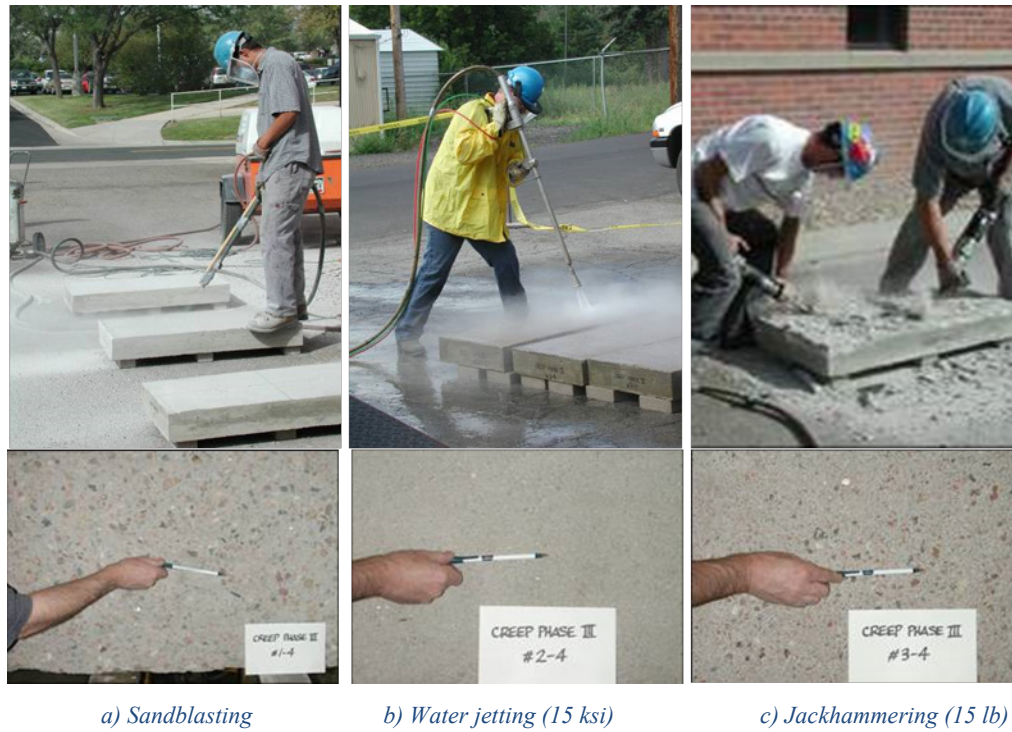
*Figure 4-8: Surface preparation of the test slabs (USBR program)*



P: pull-off test
A: pull-off test with 2° or 4° inclination
T: torque test
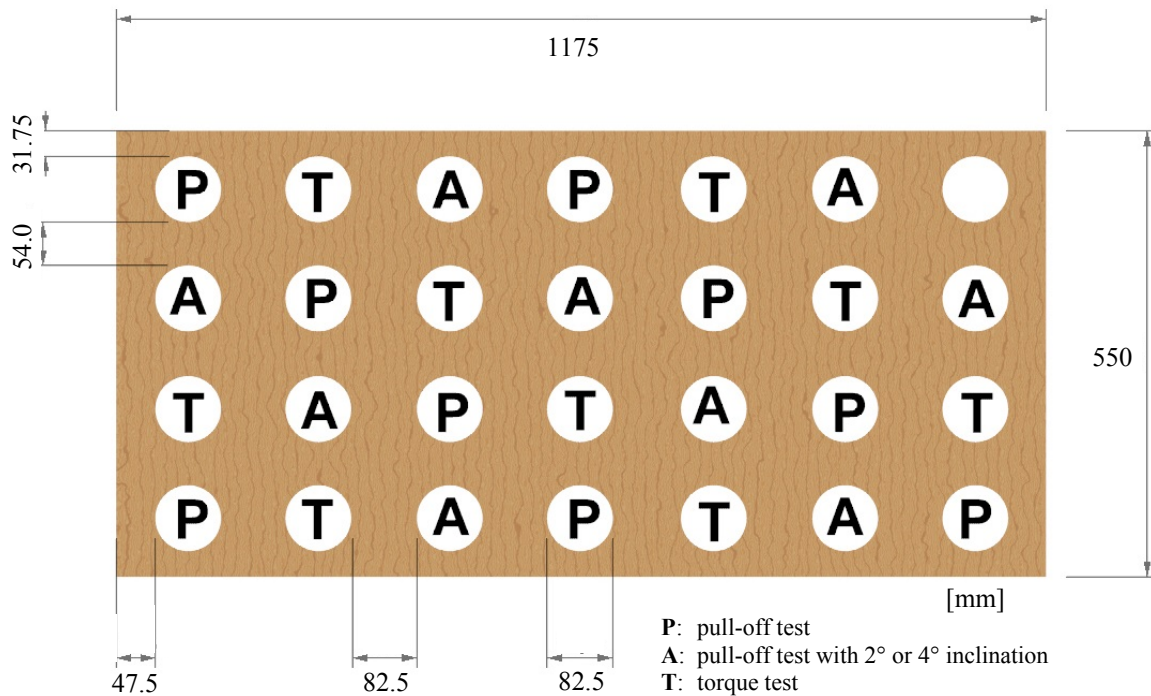
*Figure 4-9: Core-drilling template for mechanical bond testing (USBR program)*

*Figure 4-10: Experimental setup for coring under controlled inclination (USBR program)*

## Theoretical analysis

1.7. <u>Source of misalignment</u>

First, a sensitivity analysis was performed in order to establish whether the two possible sources of misalignment, i.e. coring misalignment and pulling misalignment, exert the same influence on pull-off test results.  Numerical simulations were carried out assuming only core inclination load inclination angles of 4° and a core depth of 30 mm [2.36 in.].  Results are summarized in Table 4-6.

For a given angle, both types of misalignment yield very similar results and it can be concluded that their influence upon pull-off test results is comparable.  A slight difference was found when comparing transverse stresses ($\sigma_x$), but it was sufficiently small to assume that it does not affect the pull-off strength data within its intrinsic range of variability.

*Table 4-6: Calculated pull-off test stress differentials induced by a 4° misalignment (7.85 kN [1,77 lb])*

| Testing conditions | Point A | | Point B | |
|---|---|---|---|---|
| | $\sigma_x$ [MPa] | $\sigma_y$ [MPa] | $\sigma_x$ [MPa] | $\sigma_y$ [MPa] |
| 4° – core misalignment 15 mm coring depth | 1.1 | 3.2 | 0.8 | 2.2 |
| 4° – load misalignment 30 mm coring depth | 1.4 | 3.2 | 0.6 | 2.2 |

Note: 1 MPa = 145.0 psi; 1 mm = 0.03937 in.

## 1.8. Influence of core depth and misalignment angle

Initially axi-symmetrical with respect to the vertical axis under a perfectly vertical load, the stress field induced by the pulling effort in the cored area becomes increasingly asymmetrical as the load inclination shifts from 0° to 2°, and then to 4° (Figure 4-11). Under a load perfectly aligned with the coring axis (0°), in addition to the absence of stress asymmetry, transverse stresses ($\sigma_x$) at the bottom of the core cut are very small. These stresses also increase when the angle of inclination increases, especially at the bottom of the core. The largest stress imbalance, either for axial ($\sigma_y$) or transverse ($\sigma_x$) load, occurs within the load plane between points located at the tip of each slit and identified as A and B (Figures 4-4 and 4-5), where the maximum and minimum stresses are found respectively.



a) 0° misalignment / 15 mm core

b) 0° misalignment / 30 mm core

c) 2° misalignment / 15 mm core

d) 2° misalignment / 30 mm core

e) 4° misalignment / 15 mm core

f) 4° misalignment / 30 mm core
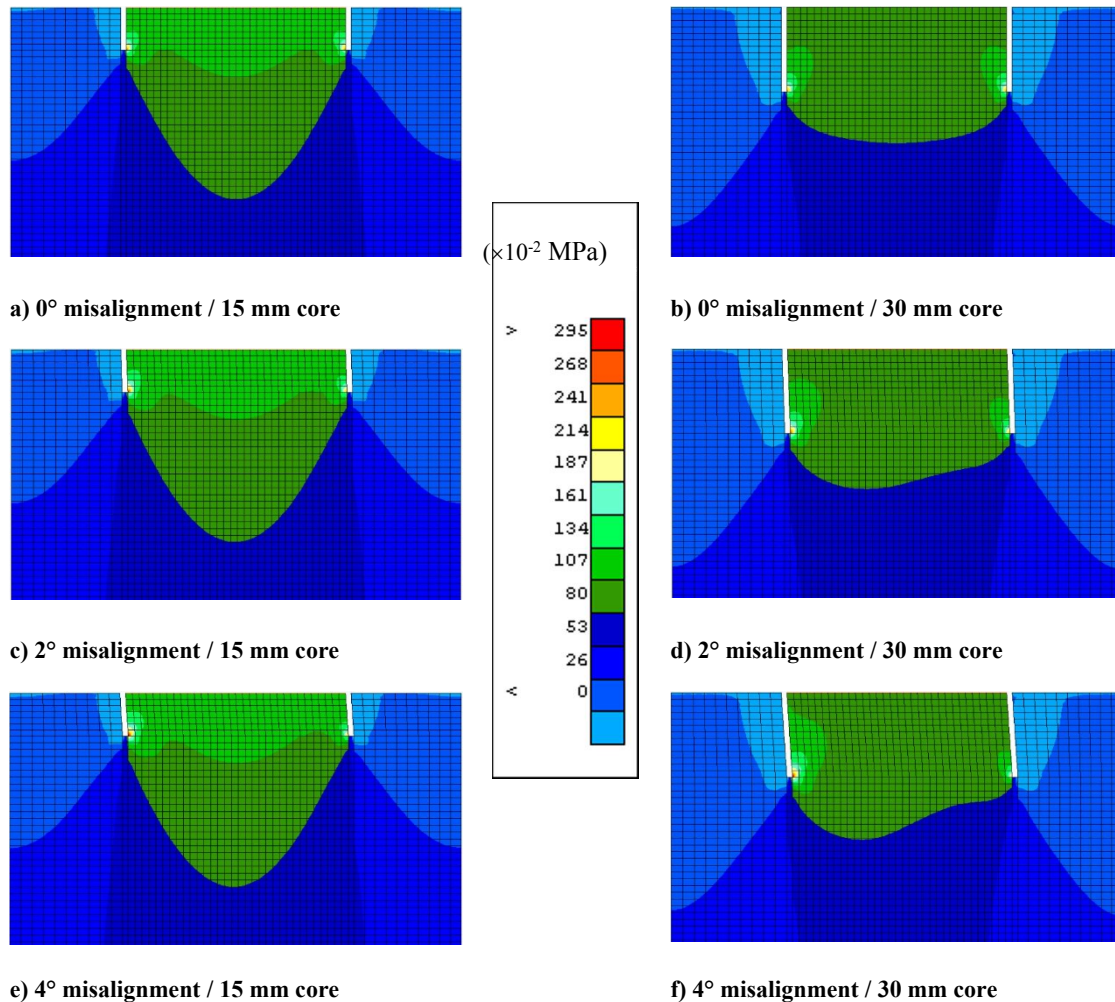
Note: 1 MPa = 145.0 psi; 1 mm = 0.03937 in.
*Figure 4-11: Axial stress ($\sigma_y$) distribution for misalignment angles of 0°, 2° and 4° and coring depths of 15 and 30 mm*

The severity of the stress imbalance obviously depends on the misalignment magnitude. Based upon the data summarized in Table 3, a 4° misalignment theoretically induces a significant axial stress ($\sigma_y$) differential at the bottom of the core. Stress distributions were calculated for different core depths and angles of inclination. As the value of the angle of inclination increases, the maximum axial stress increases at a progressively increasing rate (Table 4-7 and Figure 4-12).

Besides, it can be observed that the influence of the depth of coring is minor up to an inclination angle of approximately 10°, beyond which the axial stress imbalance appears to increase with the depth of coring.  This is in accordance with Cleland's findings [15].

At point A, a misalignment angle of 2° induces maximum axial ($\sigma_y$) stress increases of 6 and 9%, for core depths of 15 and 30 mm [1.18 and 2.36 in.] respectively, while a misalignment angle of 4° causes the axial stresses to increase by 14 and 19% for core depths of 15 and 30 mm [1.18 and 2.36 in.] respectively.  As a simple first-order assumption, it can be inferred that the corresponding pull-off strength values are reduced by 7 and 13% for a coring depth of 15 mm [1.18 in.] and by 8 and 16% for a coring depth of 30 mm [2.36 in.].

*Table 4-7: Axial stress ($\sigma_y$) amplification values calculated as a function of the misalignment angle of inclination and coring depth in a pull-off experiment*

| Misalignment angle (°) | Maximum axial stress ($\sigma_y$) amplification [%] | |
| --- | --- | --- |
| | Core depth [mm] | |
| | 15 | 30 |
| 2 | 7 | 9 |
| 4 | 15 | 19 |
| 10 | 41 | 39 |
| 15 | 57 | 84 |
| 20 | 89 | 117 |

Note: 1 mm = 0.03937 in.
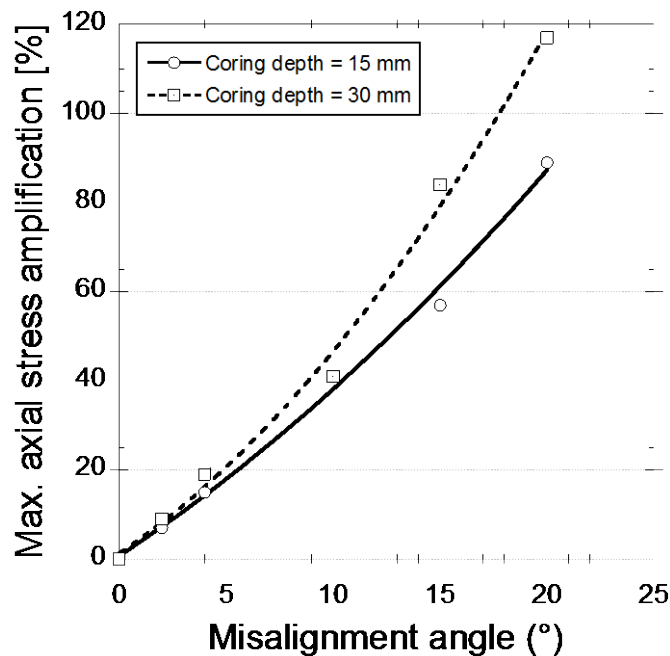


(note: 1 mm = 0.03937 in.)

*Figure 4-12: Theoretical axial stress ($\sigma_y$) amplification as a function of the misalignment angle of inclination and coring depth in a pull-off experiment*

It should be noted that the actual numerical results are dependent on the modelling assumptions and assumed material properties.  For instance, the use of different elastic modulus values would have yielded different results.

# Experimental results and discussion

### 1.9. Experiments on monolithic test slabs - ULg program

The effect of misalignment was evaluated experimentally through pull-off experiments.  The test results yielded under different conditions are summarized in Table 4-8, along with the results of direct tensile strength tests performed for comparison purposes on 50-mm (1.97-in.) cores extracted from the test slabs.  The direct tensile strength results recorded for the three mixtures are relatively close to each other and, contrary to the compressive strength data (Table 4-3), do not exhibit a systematic increase with the w/cm reduction.  It is not uncommon, given the non-linear relationship between tensile and compressive properties of concrete and the inherently more variable character of tensile strength determination.

In Table 4-8, it can be seen that for the given test conditions, the average recorded pull-off strength values for the three investigated concrete mixtures are also very close.  Besides, based on the comparison with direct tensile data for 0° misalignment and the shallowest core depth, the results yielded in the pull-off experiment provide a reliable appraisal of the actual substrate tensile strength.

In general, with regards to the influence of test misalignment, the pull-off test results exhibit trends that do not stand out as clearly as in the numerical analysis, likely due to the respective tensile testing and material variabilities, which are not taken into account in deterministic calculations such as those performed in this study.  In fact, the coefficients of variation of the recorded pull-off results, which are summarized in Table 4-9, are of the same order of magnitude as the calculated strength reduction due to testing misalignment (7 and 13% for 2° and 4° misalignments, and 15-mm cores; 8 and 16% for 2° and 4° misalignments, and 30-mm cores).  It thus appears normal to have less definite trends.  Besides, as found again in the simulations, a decrease in recorded pull-off strength values is systematically observed when increasing the core depth from 15 mm [1.18 in.] to 30 mm [2.36 in.].  For the 30-mm coring depth series, the effect seems to overshadow the influence of misalignment.

*Table 4-8: Direct tensile test and pull-off test results (ULg program)*

| Concrete mixture | Avg. direct tensile strength[1] [MPa] | Test number | Pull-off strength [MPa] | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Coring depth | | | | | |
| | | | 15 mm | | | 30 mm | | |
| | | | Coring misalignment angle | | | | | |
| | | | 0° | 2° | 4° | 0° | 2° | 4° |
| C30/37 | 3.6 | 1 | 3.8 | 3.5 | 3.2 | 3.6 | 3.0 | 3.6 |
| | | 2 | 3.4 | 3.3 | 3.2 | 3.0 | 2.9 | 2.4 |
| | | 3 | 4.1 | 3.8 | 3.8 | 3.2 | 3.0 | 2.8 |
| | | 4 | 3.8 | 3.2 | 3.6 | 3.6 | 3.0 | 3.4 |
| | | 5 | 3.6 | 3.7 | 3.8 | 2.7 | 2.6 | 2.5 |
| | | 6 | - | 3.2 | 3.5 | 2.6 | 3.0 | - |
| | | Avg. | 3.8 | 3.4 | 3.5 | 3.1 | 3.0 | 2.8 |
| C40/50 | 3.9 | 1 | 3.7 | 3.3 | 3.4 | 2.9 | 2.9 | 3.4 |
| | | 2 | 4.1 | 3.9 | 3.8 | 3.3 | 3.1 | 3.6 |
| | | 3 | 4.0 | 3.2 | 3.7 | 3.7 | 2.9 | 2.9 |
| | | 4 | - | - | - | 3.0 | 2.5 | - |
| | | 5 | 3.6 | 3.4 | 2.8 | - | 3.6 | 3.3 |
| | | 6 | 3.9 | 3.8 | 3.3 | 2.6 | 3.2 | 3.2 |
| | | Avg. | 3.9 | 3.4 | 3.4 | 3.0 | 3.0 | 3.3 |
| C50/60 | 3.5 | 1 | 4.0 | 3.8 | 3.0 | 3.5 | 2.8 | 2.4 |
| | | 2 | 3.7 | 3.3 | 3.6 | 3.6 | 3.0 | 2.3 |
| | | 3 | 3.9 | 4.2 | 3.7 | 3.7 | 3.3 | 3.1 |
| | | 4 | 3.3 | 3.4 | 3.3 | 3.2 | - | 3.4 |
| | | 5 | 4.1 | 4.0 | 3.5 | 3.1 | 2.9 | 3.3 |
| | | 6 | 4.4 | 4.0 | 3.3 | 2.8 | 3.5 | 3.1 |
| | | Avg. | 3.9 | 3.9 | 3.4 | 3.3 | 3.0 | 3.1 |

[1] Test performed on 50-mm (2-in.) diameter cored cylinders; each data corresponds to the average of 5 test results.
  Note: 1 MPa = 145.0 psi; 1 mm = 0.03937 in.

*Table 4-9: Variability of the pull-off strength data (ULg program)*

| Concrete mixture | Pull-off strength COV (coeff. of variation) [%] | | | | | |
|---|---|---|---|---|---|---|
| | Core depth [mm] | | | | | |
| | 15 | | | 30 | | |
| | Misalignment angle | | | | | |
| | 0° | 2° | 4° | 0° | 2° | 4° |
| C30/37 | 8 | 9 | 9 | 13 | 7 | 17 |
| C40/50 | 5 | 9 | 12 | 13 | 13 | 9 |
| C50/60 | 10 | 10 | 9 | 9 | 10 | 17 |

Note: 1 mm = 0.03937 in.

In Figure 4-13, the experimental pull-off results of all three tested mixtures were averaged for each coring depth / misalignment combination and compared to the theoretical values, which were determined based upon the simulation results. It can be seen that the experimental results are quite close to the predicted values for the 15-mm deep coring series, while in the case of the 30-mm deep series, the recorded values seem to be little affected by misalignment and exceed slightly the calculations. Overall, it appears that the pull-off simulations provide a satisfactory level of accuracy for practical purposes, allowing a realistic prediction on the conservative side.

As for the type of failure encountered in the test program, more than 89% of the failures occurred at the bottom of the core, with only a few failures (6%) recorded in the body of the core. Detailed examinations of the fracture surfaces revealed interesting behavior: irrespective of the concrete mixture, the proportion of aggregate failures across the fracture surfaces in the test series performed with a coring depth of 15 mm [1.18 in.] was found to be systematically higher than in the 30-mm [2.36-in.] coring depth series. This observation is consistent with the higher pull-off tensile strength recorded in the former.

Conversely, the proportion of aggregate failures did not appear to be significantly affected by test misalignment.
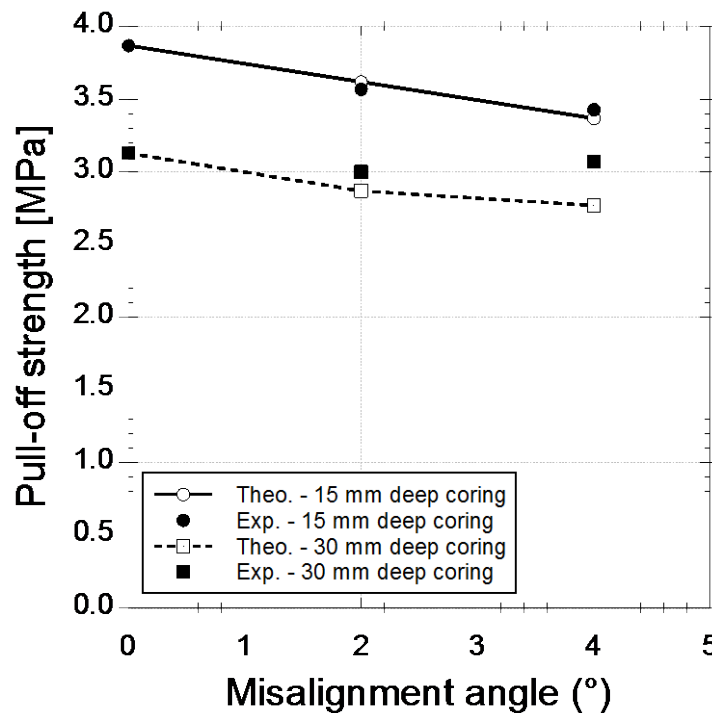


*Figure 4-13: Comparison of predicted and experimental pull-off test results*

### 1.10.   Experiments on repaired test slabs - USBR program

The detailed results of the bond strength tests performed with different alignment angles on the experimental slabs are presented in Tables 4-10 to 4-12, and in Figures 4-14 to 4-16.

The results obtained for repairs exhibit more or less the same tendency as those yielded before with monolithic specimens, the average recorded bond strength decreases as the coring misalignment angle gets larger (Table 4-13).  Again, these trends do not stand out as clearly as it did in the theoretical calculations performed for a monolithic substrate, owing to the intrinsic variability of the pull-off test data.  With average pull-off values two times smaller than those found previously in the case of monolithic specimens tested at ULg, the coefficients of variation in the present case are approximately twice as large (Figure 4-12).

*Table 4-10: Summary of pull-off test results obtained for the sandblasted slabs (USBR program)*

| Coring misalignment angle | Test parameters | Location of failure | | | | | Overall |
|---|---|---|---|---|---|---|---|
| | | Core end | Substrate | Interface | Repair | Dolly | |
| 0° | Number of test results (%) | 20 (57.1) | 2 (5.7) | 10 (28.6) | 1 (2.9) | 2 (5.7) | 35 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.91** (0.24) | **2.04** (0.16) | **1.29** (0.45) | **1.63** - | **1.20** (1.02) | **1.69** (0.46) |
| 2° | Number of test results (%) | 9 (50.0) | 3 (16.7) | 6 (33.3) | 0 (0) | 0 (0) | 18 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.46** (0.47) | **1.85** (0.20) | **1.00** (0.44) | - (-) | - (-) | **1.50** (0.46) |
| 4° | Number of test results (%) | 12 (70.6) | 1 (5.9) | 1 (5.9) | 1 (5.9) | 2 (11.8) | 17 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.60** (0.26) | **2.13** (-) | **0.91** (-) | **1.58** (-) | **1.15** (0.03) | **1.54** (0.34) |

*Table 4-11: Summary of pull-off test results for the water jetted slabs (USBR program)*

| Coring misalignment angle | Test parameters | Location of failure | | | | | Overall |
|---|---|---|---|---|---|---|---|
| | | Core end | Substrate | Interface | Repair | Dolly | |
| 0° | Number of test results (%) | 27 (75.0) | 6 (16.7) | 1 (2.8) | 2 (5.6) | 0 (0) | 36 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.94** (0.32) | **2.01** (0.30) | **1.99** - | **1.83** (0.13) | - (-) | **1.96** (0.30) |
| 2° | Number of test results (%) | 16 (88.9) | 1 (5.6) | 0 (0) | 1 (5.6) | 0 (0) | 18 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.84** (0.26) | **2.31** (0.00) | - (-) | **1.74** (-) | - (-) | **1.86** (0.27) |
| 4° | Number of test results (%) | 14 (77.8) | 4 (22.2) | 0 (0) | 0 (0) | 0 (0) | 18 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.49** (0.33) | **1.62** (0.31) | - (-) | - (-) | - (-) | **1.52** (0.32) |

From the failure location distributions (Tables 4-10 to 4-12, and in Figures 4-14 to 4-17) it appears that failure in the repaired elements is somehow pushed away from the interfacial zone down into the substrate as the coring misalignment angle increases, irrespective of the surface preparation method.

*Table 4-12: Summary of pull-off test results for the water jackhammered slabs (USBR program)*

| Coring misalignment angle | Test parameters | Location of failure | | | | | Overall |
|---|---|---|---|---|---|---|---|
| | | **Core end** | **Substrate** | **Interface** | **Repair** | **Dolly** | |
| 0° | Number of test results (%) | 4 (11.1) | 7 (19.4) | 25 (69.4) | 0 (0) | 0 (0) | 36 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.57** (0.25) | **1.03** (0.27) | **1.35** (0.47) | - (-) | - (-) | **1.31** (0.44) |
| 2° | Number of test results (%) | 2 (11.1) | 3 (16.7) | 13 (72.2) | 0 (0) | 0 (0) | 18 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.41** (0.37) | **1.15** (0.46) | **1.38** (0.33) | - (-) | - (-) | **1.40** (0.38) |
| 4° | Number of test results (%) | 7 (38.9) | 2 (11.1) | 9 (50.0) | 0 (0) | 0 (0) | 18 (100) |
| | Avg. strength [MPa] (std. dev.) | **1.44** (0.18) | **1.09** (0.26) | **1.21** (0.32) | - (-) | - (-) | **1.28** (0.29) |
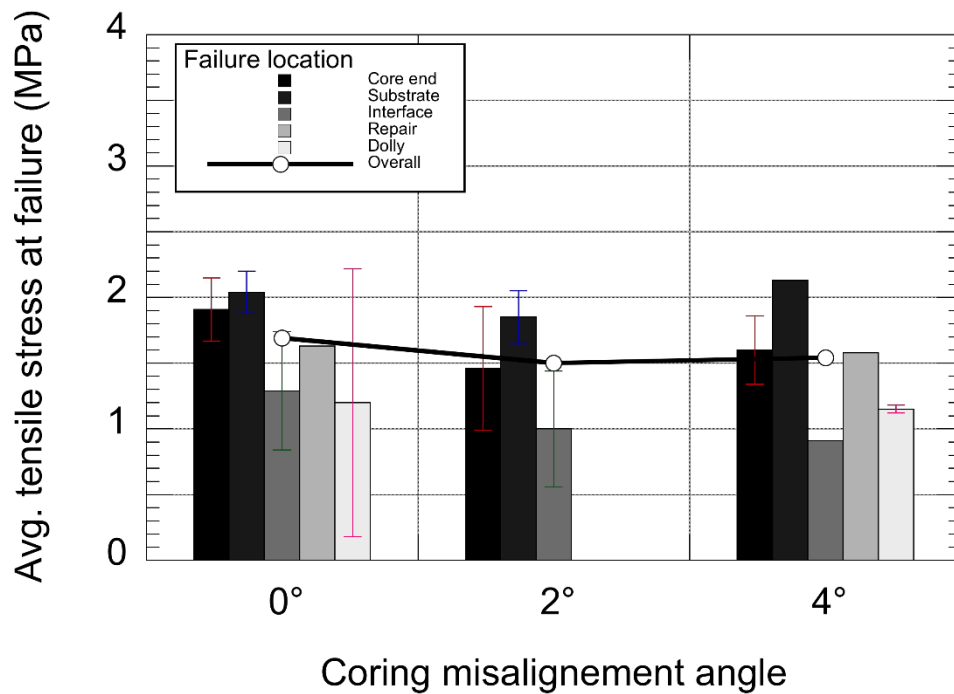


*Figure 4-14: Distribution of failure location in pull-off tests performed on sandblasted slabs as a function of the coring misalignment angle (USBR program)*

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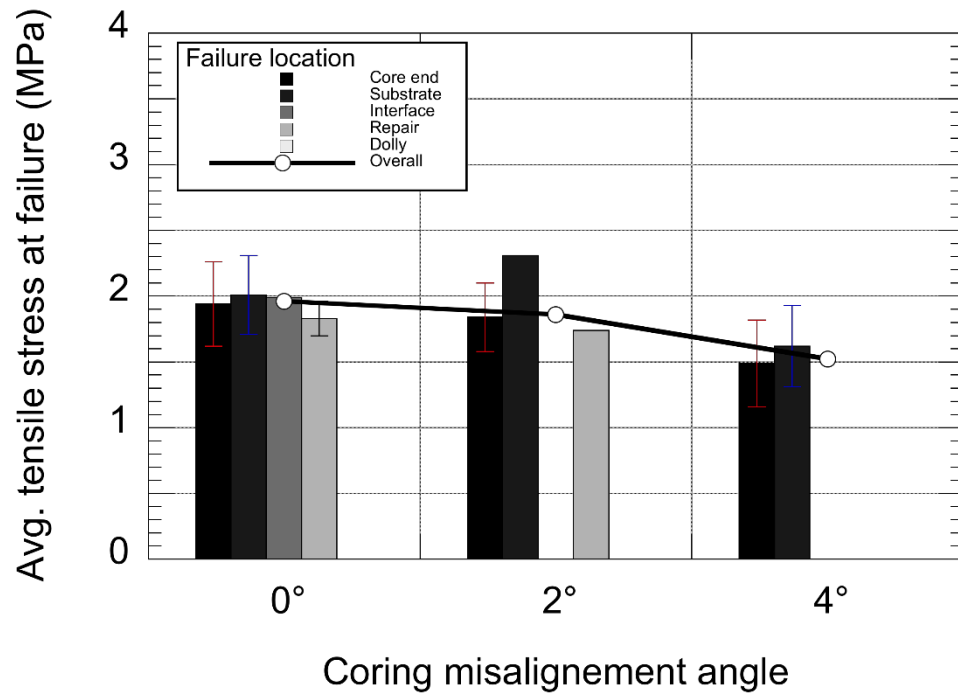
*Figure 4-15: Distribution of failure location in pull-off tests performed on water jetted slabs as a function of the coring misalignment angle (USBR program)*
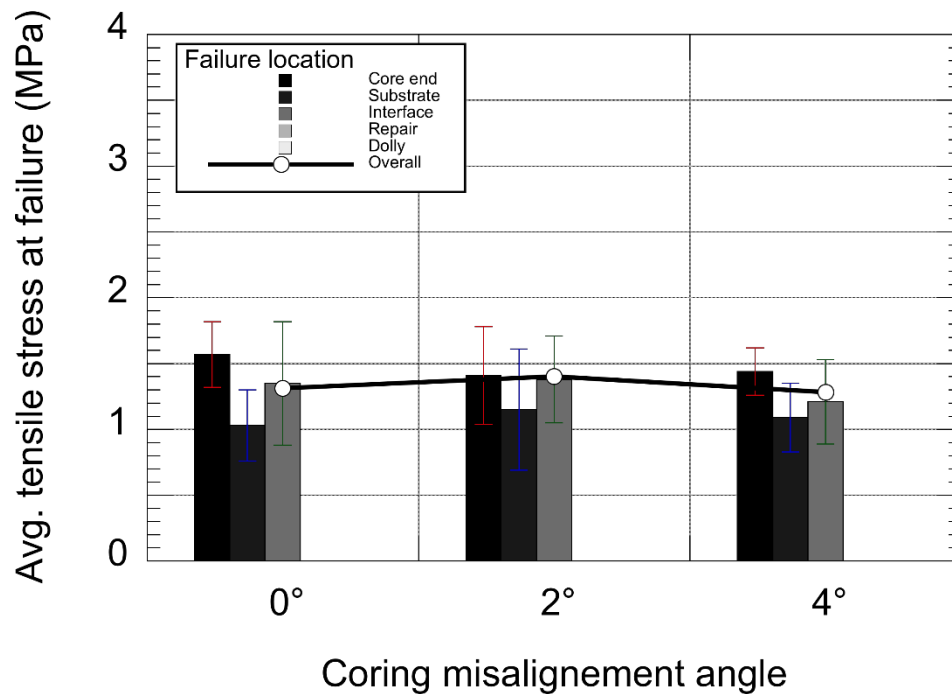


*Figure 4-16: Distribution of failure location in pull-off tests performed on jackhammered slabs as a function of the coring misalignment angle (USBR program)*

Section 4 - Evaluation of the Effect of Misalignment upon Pull-Off Testing Results (Task 3)    21
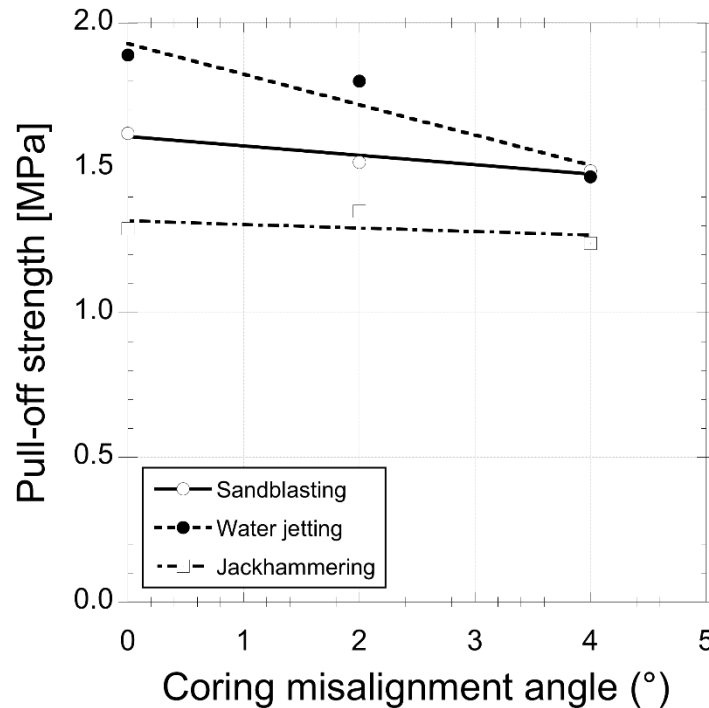
*Figure 4-17: Average pull-off test results obtained for the different surface preparation as a function of the coring misalignment angle*

*Table 4-13: Variability of the pull-off strength data (USBR program)*

| Surface preparation | Pull-off strength COV (coeff. of variation) [%] | | |
|---|---|---|---|
| | Coring misalignment angle | | |
| | 0° | 2° | 4° |
| Sandblasting | 27 | 31 | 22 |
| Water jetting | 15 | 15 | 21 |
| Jackhammering | 34 | 27 | 23 |

Note: 1 mm = 0.03937 in.

Overall, in the range of misalignment and other characteristics and factors studied, the experimental pull-off test data appears to be less sensitive than what was predicted for monolithical elements through numerical analysis. In the "*realcrete*" experiments, substrate surface imperfections, air voids, microcracks, cracks, non-uniformity of roughness, etc. may exert more influence on the recorded pull-off values than misalignment, at least within the investigated misalignment range (up to 4°).

Provided that core drilling is achieved accurately and the pull-off test is executed in overall good conditions, it can be concluded that small deviations from the vertical do not affect critically the pull-off strength evaluation in a repaired element. Within average naked-eye detection capability, the potential bias due to testing misalignment was generally found to be well below the theoretical reduced values calculated for monolithical elements, except for one single case (watter jetted substrate test at a 4° angle). Further, it must be stressed that it can only affect the pull-off strength evaluation on the conservative side. It may be useful to recall here that the 4° value has been assumed in this task to represent an easily detectable limit by the average human eye. This

can actually be appraised in Figure 4-18, which shows an extracted core specimen that was drilled with a 4° misalignment angle.



*Figure 4-18: Example of an extracted core specimen drilled with a 4° angle*

## Conclusions

The pull-off test is a convenient method for evaluating both the mechanical integrity of the concrete surface prior to repair and the repair bond strength.  A reliable evaluation of these properties can be obtained, provided that a minimum number of tests are performed, with adequate equipment.  The general trends observed in numerical analysis and experimental programs for both monolithic and composite repair systems reveal that the pull-off strength values decrease as the angle of misalignment increases.  The deeper the coring into the substrate, the greater is the effect of misalignment.

Based upon the various results generated in the two programs (ULg and USBR) conducted as part of this task, the following observations could be made:

- Up to a certain misalignment limit angle assumed to be detectable by the average human eye (4° in the present study), load and coring misalignments were not found to yield significantly different stress fields and, for practical calculation purposes, they can be addressed in a similar fashion;
- Results of simulations revealed that a distorted stress field is induced by pull-off testing misalignment, resulting in stress concentrations in an area at the bottom of the core slit: a 2° misalignment yield maximum stress increases of 6 and 9 % respectively for 15 mm and 30 mm [0.6 and 1.2 in.] coring depths, and the corresponding increases resulting from a 4° misalignment reach 14 and 19%;

- The experimental pull-off test program results are overall consistent with the theoretical calculations, although the observed trends are not as clear, owing to the experimental variability and to the added influence of the coring depth;
- The simulation results provide a conservative but realistic lower bound limit for evaluating the influence of misalignment upon pull-off test results, either in a monolithic or composite (repaired) concrete element: a 2° misalignment can be expected to yield a pull-off strength reduction of 7 to 9 % respectively for 15 mm [0.6 in.] and 30 mm [1.2 in.] coring depths, and the corresponding decrease resulting from a 4° misalignment reach between 13 and 16%;
- As for the failure mode, it can be concluded that within 4°, testing misalignment does not fundamentally change the failure mode characteristics in a monolithic element, whereas in a composite (repaired) element, it tends to push it away from the interface towards the core end.

The theoretical bias due to testing misalignment, below average naked-eye detection capability (±4°), was evaluated to reach up to approximately 15%. In so-called "*realcrete*" experiments, substrate surface imperfections, air voids, microcracks, cracks, non-uniformity of roughness, etc. may actually exert more influence on the recorded pull-off values than misalignment, at least within the investigated relatively low misalignment range of up to 4°. Thus, provided that core drilling is achieved quite accurately, and the rest of the pull-off test operations are performed properly, it can be concluded that generally, small deviations from the vertical do not significantly alter the pull-off strength evaluation.

From a practical standpoint (quality control), the results generated in this study indicate that pull-off test misalignment may cause the rejection of repairs with adequate bond strength. With a number of tests sufficiently large for statistical significance, one could specify a lower acceptance pull-off strength limit in the field – with the mean acceptance value being decreased for example by 5 to 10 % – to take into account the bias induced by testing misalignment. Nevertheless, in most cases, the amount of tests performed on site is quite limited and using the specified bond strength value as the field acceptance limit for quality control is conservative and is recommended.

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# Section 5 - Appraisal of Test Procedures for Evaluating the Optimum Moisture Conditioning of a Given Concrete Substrate (Task 4)

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# Section 5 - Appraisal of Test Procedures for Evaluating the Optimum Moisture Conditioning of a Given Concrete Substrate (Task 4)

The experimental work in this task was carried out at the GeMMe Building Materials Research Unit, in the Department of Architecture and Construction Engineering (ArGEnCo) of the University of Liège, in Belgium. Subsequent work has been undertaken at the Research Center on Concrete Infrastructure (CRIB), Laval University, Quebec City (QC), Canada and at the US Bureau of Reclamation in Denver (CO), USA, but it is not included in this report.

## General

The influence of surface moisture on the bond between existing concrete and repair is an issue of significant importance. Moisture condition of the concrete substrate surface at the time of application of repair material has a major influence on the moisture transport mechanism between the freshly applied repair material and existing concrete substrate.

*Saturated Surface Dry* (SSD) conditioning of the substrate prior to application of cementitious repair materials is usually recommended and used, which underlies the "layman's" instinctive procedures to avoid problems, rather than achieving the most effective bond. Various investigators came to the conclusion that different substrates and repair materials may require different interface moisture conditions at the time of casting to achieve optimum interfacial bond. The problem is that presently there is no test method to determine the optimum moisture condition for a given combination of substrate and repair material.

Water is one of the critical factors influencing bond development between concrete and repair materials: it may accumulate at the interface or migrate through it in either direction, as a result of mechanical (i.e. gravity), chemical (i.e. hydration) or physical (i.e. temperature gradients) driving forces.

Different moisture transport parameters affect the formation and behavior of the repair interfacial zone, such as diffusion and permeability coefficients, as the interface characteristics are indeed influenced by different forms of water interaction:

- First, moist conditioning of the substrate before the application of the repair system is a key consideration. Partial or total saturation of a concrete substrate is a common situation in repair works. Water along the interface may prevent adhesion to the repair system, with regard to polymer concrete (PC), polymer cement concrete (PCC) or cement concrete (CC) types [5];
- Second, water or aqueous solution movements may occur [3] due to migration and infiltration along the interface [4] or diffusion and capillary absorption from the zones to be repaired [5]. Resistance to these water movements will directly depend on the quality of the materials: W/C, porosity, etc.

In most situations, the saturation level at the interface appears to be a predominant factor in promoting the adhesion of the repair system.

## Background information

*Saturated Surface Dry* (*SSD*) conditioning of the substrate prior to application of cementitious repair materials is usually recommended and used, and underlies the "layman's" instinct to avoid problems rather than achieving the most effective bond. Various investigators have come to the conclusion that different substrates and repair materials correspond to different optimum interface moisture conditions at the time of casting. The problem is that presently, there is no methodology to determine the optimum moisture condition for a given combination of substrate and repair material.

When water is present at the interface between the repair and the concrete substrate, the thermodynamic equilibrium with respect to the surface free energy of each material is modified [7-9]. From a theoretical point of view, this requires a generalization of the Young and Dupré's equation [10], relative to a new liquid-liquid interface. Contact angle modification is a visible effect of the interaction between the two liquids and a solid surface. But what happens to the spreading conditions? If there is no spreading of one liquid at the expense of the other (equations 5-1 and 5-2), the equilibrium of forces requires that:

$$\gamma_{SA} = \gamma_{SB} + (\gamma_{AB} \cdot \cos\theta) \tag{5-1}$$

where $\gamma_{SA}$, $\gamma_{SB}$, $\gamma_{AB}$ and $\theta$ are the interfacial tension between solid S and liquid A, interfacial tension between solid S and liquid B, interfacial tension between liquids A and B, and contact angle of these liquids on the solid surface, respectively (Figure 5-1).
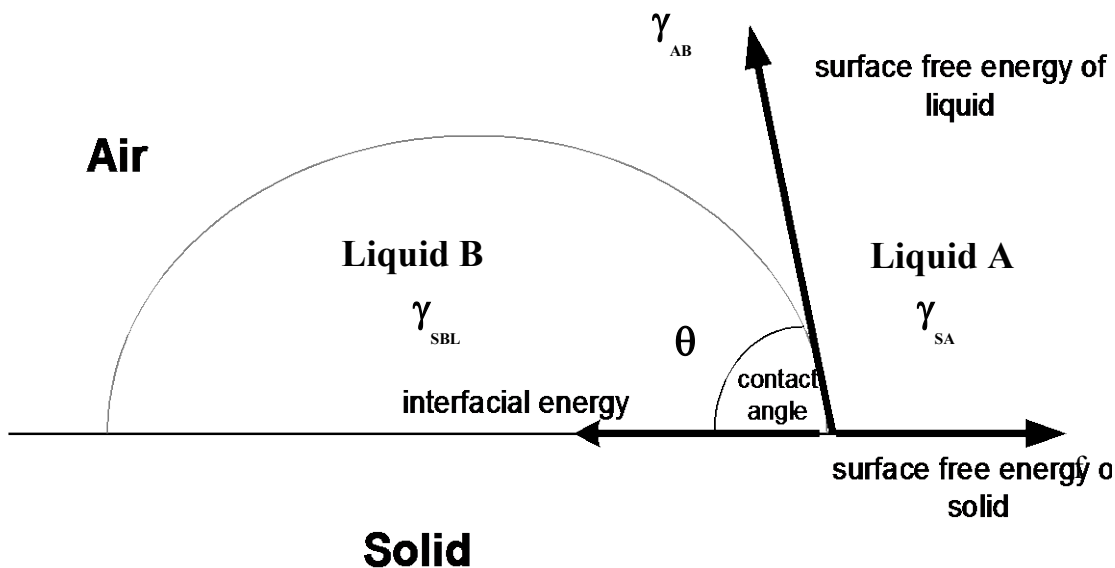


*Figure 5-1: Wetting of a solid surface by two non-miscible liquids (wetting effect favorable in this case to liquid B)*

It can be shown [11] that the liquid with the higher tension of adhesion ($\gamma_{x(L)} \cdot \cos \theta_{x(L)}$) will expel the other one from the surface. The calculation of the work of adhesion allows interesting interpretations, taking into account the variation of surface free energies in the presence of water [12]. The work of adhesion, which provides an estimation of the adhesion between a liquid and a solid in contact with each other, is described by the following equation:

$$W_{x(L)} = \gamma_{x(L)} \cdot (1 + \cos \theta_{x(L)}) \tag{5-2}$$

Table 5-1 summarizes data on the work of adhesion at the interface between concrete and acrylic- or epoxy-based resins, for both dry and moist conditions. A high specific work of adhesion value does not ensure *per se* adequate bond of the repair system to the concrete substrate [14]. However, it is an indication that helps determine whether minimum conditions for proper bonding are met: the lower the work of adhesion, the less chances there are of obtaining strong physico-chemical interactions between solid and liquid phases. This is a necessary, but insufficient condition for adequate bond [15].

*Table 5-1: Work of adhesion for interfaces without (W$_A$) and with (W$_{AL}$) water [13]*

| Interface | $W_A$ (mJ/m²) | $W_{AL}$ (mJ/m²) |
|---|---|---|
| Mortar / concrete | 87.8 | - |
| Acrylic / concrete | 74.1 | 22.7 |
| Acrylic / acrylic | 80.4 | 53.7 |
| Acrylic / hydrophobic treatment | 52.2 | 66.7 |
| Epoxy / concrete | 79.6 | 21.8 |
| Epoxy / epoxy | 92.4 | 53.0 |
| Epoxy / hydrophobic treatment | 56.0 | 42.2 |

## Objectives and methodology

The objective of this task was to develop a methodology to evaluate moisture level of the concrete substrate surface and to determine the optimum moisture conditioning for a given concrete surface such as to achieve maximum bond in a composite repair system. The governing criteria for the methodology development was its applicability to practical field use and accuracy. Eight (8) different concrete surface moisture levels and their effect on the bond strength were considered. Two methods were evaluated and compared.

## Experimental program

### 1.11.     General

Two concrete surface moisture test procedures were investigated, namely the *Initial Surface Absorption Test* (ISAT) inspired by an experimental device initially developed at Queen's University in Belfast [16], and the modified capillary suction test (MCST) developed at the University of Liège in Belgium [17]. The ISAT test procedure offers significant advantages over other existing techniques, making it an attractive alternative for non-destructive field testing: it is

compact, easy to handle, and the test duration is short (approximately 10 minutes). More common methods were also used as references.

Finally, the influence of moisture level on bond was evaluated by performing pull-off bond strength tests on concrete base slabs overlaid with two different cement-based repair mortars.

### 1.12. Initial Surface Absorption Test (ISAT)

Autoclam is a testing device designed for measuring the air and water permeability of concrete (Figure 5-2). It can be used in the laboratory as well as in situ: a metallic ring is fixed to the concrete substrate (Figure 5-2b)), and the amount of fluid (air or water) that penetrates into the concrete at a typical pressure of 0.5 kg/cm² is continuously recorded. Typically, the slope of the linear part of the curve between 5 and 15 minutes is used to determine a permeability index.



*a)*          *b)*
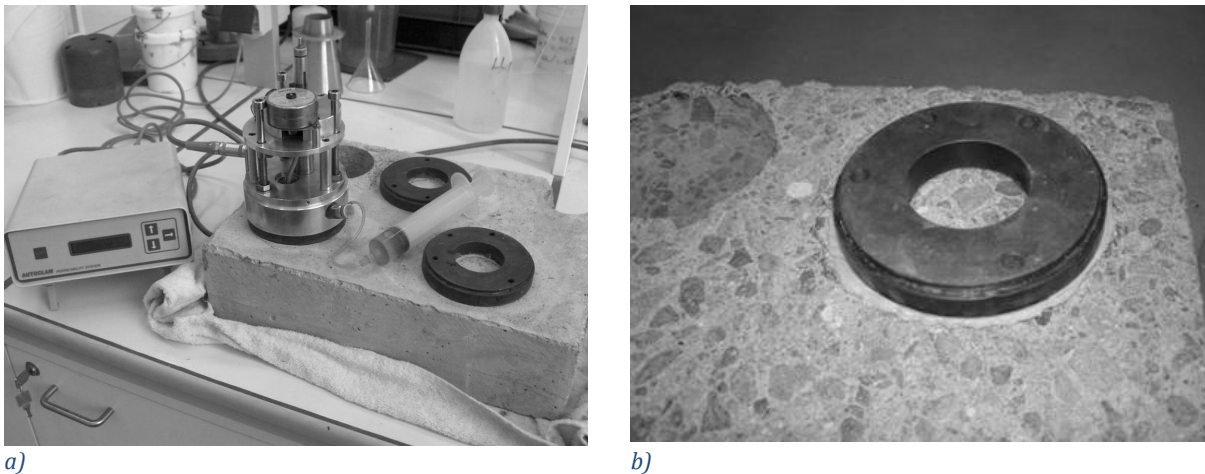
*Figure 5-2: Autoclam test device (CNS Electronics) used for ISAT testing –*
*a) experimental setup with electronic controller; b) gluing of the testing ring onto the concrete substrate*

### 1.13. Modified Capillary Suction Test (MCST)

The most commonly used test to analyze water transfer at the interface is the capillary suction test [18]. It is described by several standards, which differ from one another on the water level above the bottom surface of the concrete specimen and the time period of the measurement. Mass change is usually measured after 5, 15, 30 and 45 minutes, as well as after 2, 6 and 24 hours [19]. Mass is measured on samples wiped off with a damp tissue. However, capillary forces exist when contact is created between the concrete substrate and the repair material. The liquid phase from the repair material mixture is being absorbed into the capillaries of concrete. In the recently developed MCST test [17], the specimen mass change arising during the process of capillary suction is recorded in a continuous fashion (Figure 5-3).

Specimens are cast concrete cylinders (Ø = 80 mm; L= 200 mm) with the lateral face coated with epoxy resin (Figure 5-3b) to avoid moisture penetration and evaporation in the transverse direction: water comes up from the bottom until contact with the sample occurs. Mass change is continuously monitored (1 or 5 second(s)), and the measurements obtained from a weighing scale are saved through the use of appropriate computer software.

a)                                                      b)

*Figure 5-3: MCST testing – a) experimental setup; b) test specimen*

## Specimen preparation and conditioning

### 1.14.        Concrete substrate and sample preparation

Experiments were performed on three different concrete grades, respectively C30/37[2], C40/50 and C50/60.  The concrete mixtures were prepared with ordinary Portland cement and limestone crushed aggregates.  The main constituents, their proportions (by weight) and the resulting are summarized in Table 5-2.

---

[2] CX/Y where X and Y correspond to the nominal compressive strength values determined on cylinders and cubes respectively (EN206)

*Table 5-2: Concrete mixture composition*

| Constituent / Property | | Concrete mixture | | |
| --- | --- | --- | --- | --- |
| | | C30/37 | C40/50 | C50/60 |
| CEM I 52.5 N | (kg/m³) | 275 | 325 | 375 |
| Sand 0/2 | (kg/m³) | 765 | 729 | 676 |
| Crushed aggregate 2/8 | (kg/m³) | 255 | 230 | 206 |
| Crushed aggregate 8/14 | (kg/m³) | 569 | 576 | 601 |
| Crushed aggregate 14/20 | (kg/m³) | 390 | 401 | 412 |
| Water | (kg/m³) | 192 | 6 | 182 |
| W/C | (kg/m³) | 0.70 | 0.57 | 0.49 |
| Avg. Compressive Strength | (MPa) | 42.9 | 54.3 | 61.2 |

Eighteen 800×600×100-mm concrete slabs were cast (6 slabs for each concrete grade). After casting (24 hours), slabs were demolded and stored in water for up to 28 days. The average compressive strength of concrete was determined for each mixture using three 150×150×150-mm cube specimens (Table 5-2).

### 1.15. Surface preparation

For each tested mixture, two slabs were sandblasted (SB) and two others were water jetted at a 1000-bar pressure (HJ), all at 28 days, while two control slabs without any surface preparation (NT) were kept as reference. In the case of sandblasting (SB), the roughness of the surface was assessed using the Sand Patch Test (EN 13036-1:2002) and an average texture depth value of 0.60 mm was recorded. After thorough visual examination, only the test specimen prepared by water jetting exhibited some cracking. Still, the observed cracks [20] were limited and very small.

### 1.16. Specimen conditioning for moisture content

The concrete slabs were then subjected 8 different environmental conditioning, in such a way to obtain saturation levels ranging from 40 to 100% (Table 5-3). An example of the storage conditions is shown in Figure 5-4. The duration of conditioning was 3 months, in order to warrant a relatively homogenous degree of saturation at the surface.

Table 5-3: Storage conditions and saturation levels

| Storage conditions (Atm) | | Degree of saturation (%) |
|---|---|---|
| Atm 1 | Oven-dried (40°C) until constant weight and then stored at 23°C / 85% R.H. | 32 |
| Atm 2 | Stored outdoors (Belgium winter conditions, protected from rain) | 64 |
| Atm 3 | Stored in a climate-controlled room at 20°C / 100% R.H. | 100 |
| Atm 4 | Oven-dried (40°C) until constant weight and then immersed in water for 30 seconds and stored in a plastic bag | 41 |
| Atm 5 | Stored in water, taken out for 3 hours (23°C / 50% R.H.) and then stored in a plastic bag | 90 |
| Atm 6 | Stored in water, taken out for 1 hour (23°C / 50% R.H.) and then stored in a plastic bag | 92 |
| Atm 7 | Stored in water, taken out for 15 minutes (23°C / 50% R.H.) and then stored in a plastic bag | 96 |
| Atm 8 | Stored under standard laboratory conditions at 23°C / 50% R.H. | 42 |

The saturation level was evaluated by determining the water content (by weight) of small samples obtained by drilling and stored in the same conditions as those of the specimen slabs (Figure 5-4). The *Carbide bomb* test [21] was also used for comparison with the weight change method (Figure 5-5). The moisture content results yielded with both tests are presented together in Figure 5-6.



*Figure 5-4: Specimen storage layout for moisture pre-conditioning*

*Figure 5-5: Carbide bomb test*
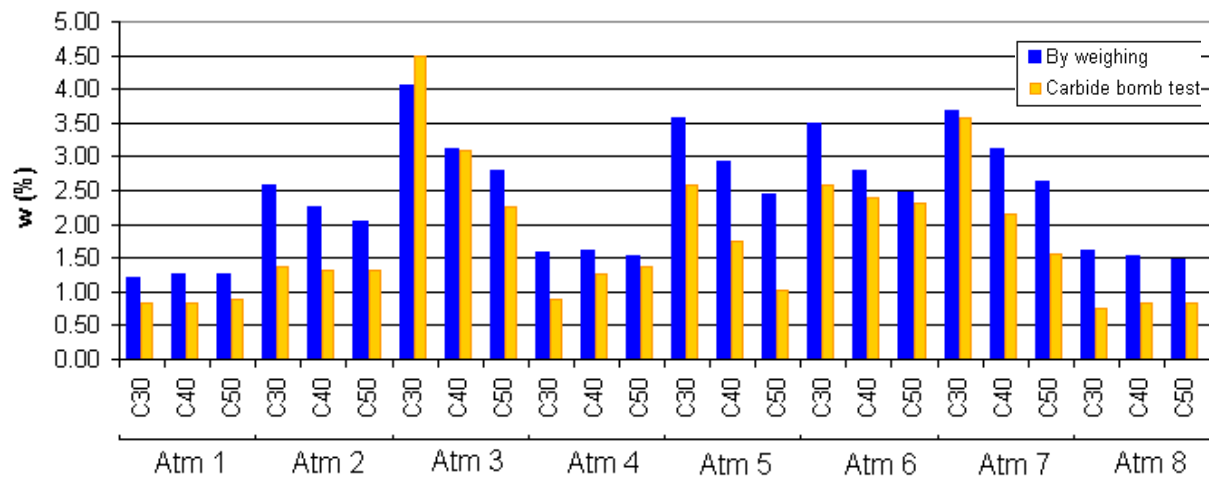
*Figure 5-6: Moisture content data obtained with the weight change method and the* Carbide bomb *test method after pre-conditioning in the various storage conditions*

# Test result analysis

### 1.17.    Water absorption test results

The water absorption permeability indices determined in the ISAT experiments for the various concrete mixtures pre-conditioned at various saturation levels are presented in the bar chart of Figure 5-7.  In Figure 5-8, it can be seen that the permeability index data exhibit overall good correlation with the degree of saturation, the index value decreasing more or less linearly as the moisture content increases (Figure 5-8).  It should be stressed however that the linear character of the relationship must be considered with caution, as the experimental results are characterized by relatively important dispersion, especially for saturation levels higher than 80% R.H.
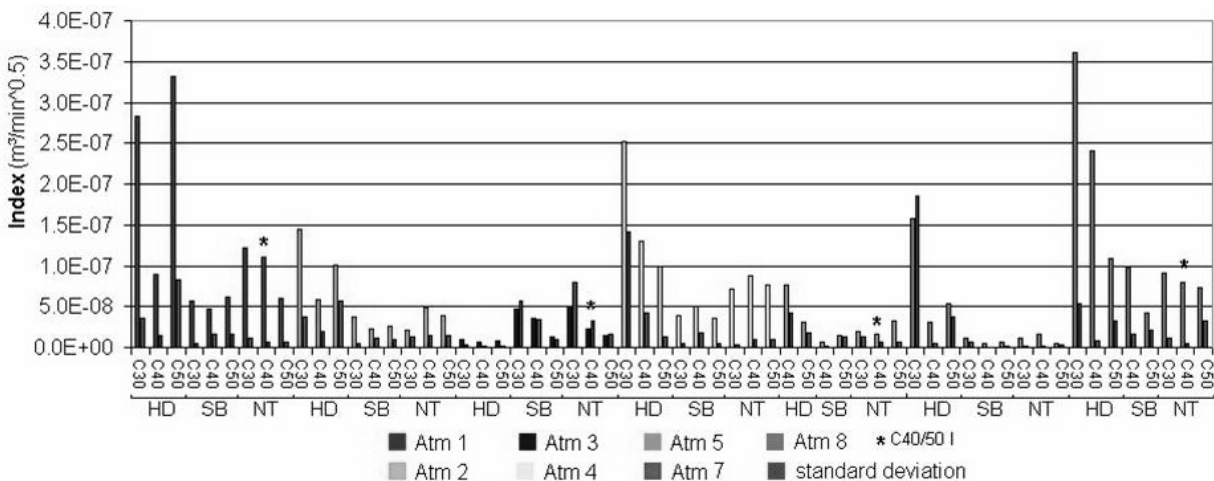


*Figure 5-7: Permeability indexes determined in the ISAT experiments for the different test program combinations (concrete mixture / surface preparation / storage conditions)*

A statistical analysis of the data did not reveal any significant influence of the concrete grade – analyzed using the average 28-day compressive strength value – upon this relationship between permeability and the degree of saturation determined experimentally. Conversely, for a given concrete mixture, the type of surface preparation is found to be a potentially influential factor. For instance, the recorded absorption rates were higher for surfaces prepared by water jetting (HD) in comparison with values obtained for sandblasted surfaces (SB), likely due to the presence of superficial cracking in the former.
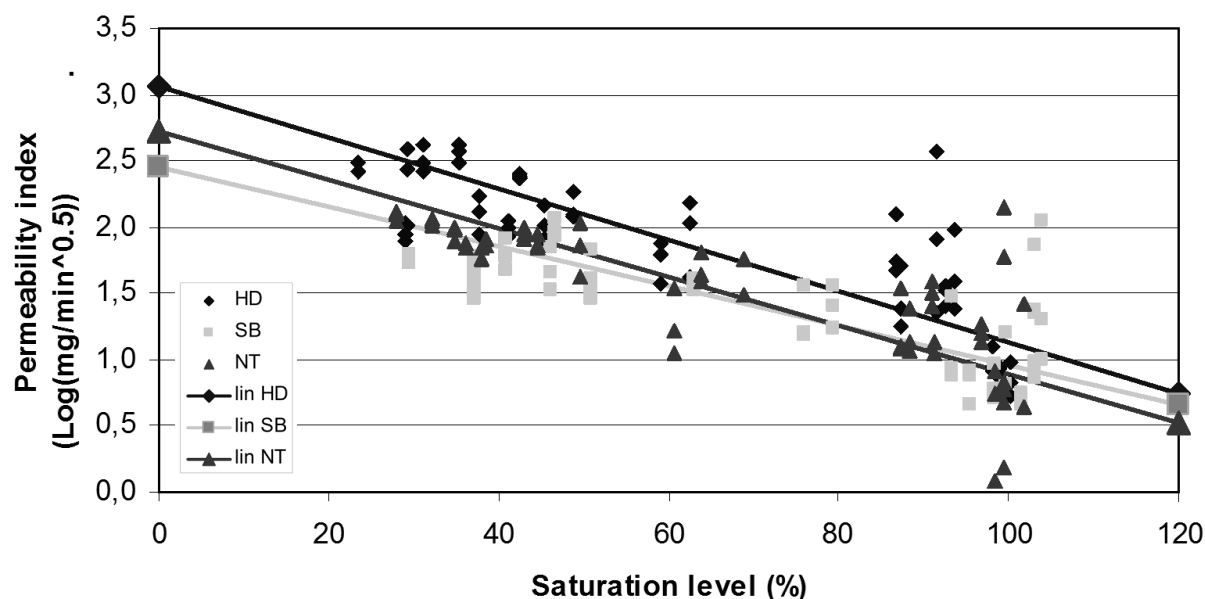


*Figure 5-8: Relationship between the ISAT permeability index and the degree of saturation of concrete*

Water capillary absorption data obtained in the MCST experiments are more consistent and exhibit clearer tendencies than those yielded in ISAT tests, as shown in Figure 5-9 for the water-jetted specimens pre-conditioned at 41% R.H. In this figure, it can be clearly seen that the denser the concrete (as inferred from the compressive strength value), the lower was the recorded absorption rate. Overall, the MCST procedure appear to be more easily controllable.

The MCST absorption coefficients are plotted as a function of the degree of saturation of concrete in the graph of Figure 5-10. The trends are quite similar to those observed previously for the ISAT index. The MCST absorption coefficient is almost inversely proportional to the degree of saturation of concrete and the relationship appears to be independent of the concrete grade. Furthermore, for a given concrete, the absorption coefficient can be influenced by the type of surface preparation. In comparison with sandblasting, water jetting induced a higher rate of water capillary absorption, which presumably can be again attributed to the presence of some bruising.
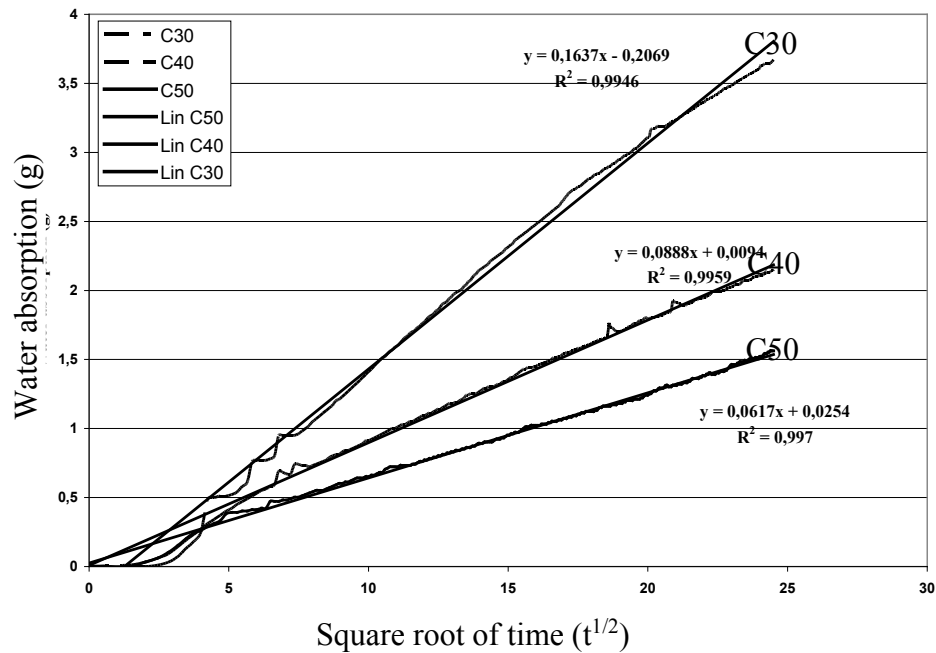
*Figure 5-9: Capillary absorption coefficients determined with MCST for water jetted (HD) specimens pre-conditioned at 41% R.H. (Atm 4)*
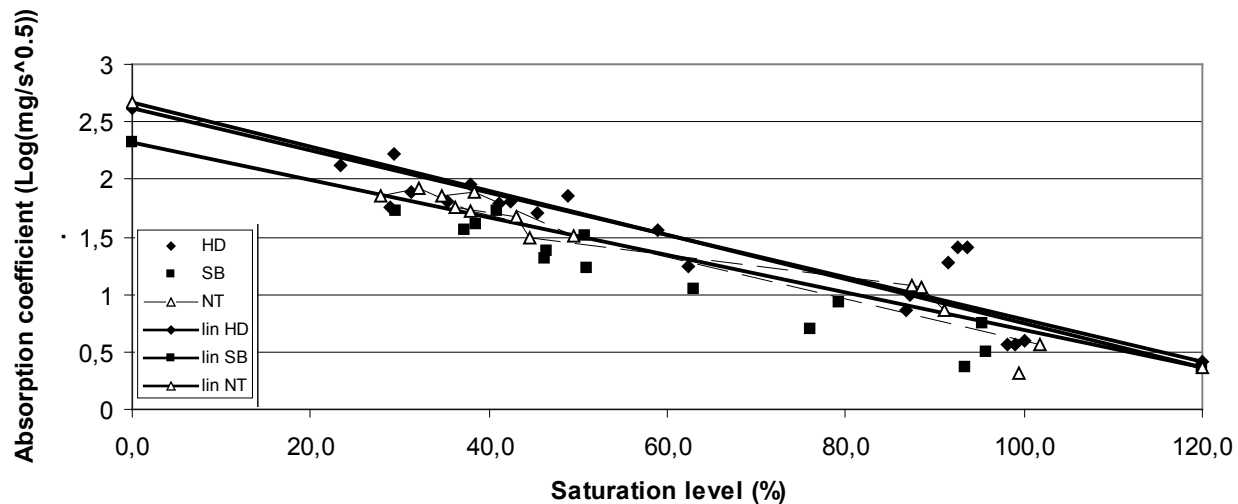


*Figure 5-10: Relationship between the MCST absorption coefficient and the degree of saturation of concrete*

### 1.18.    Comparison of ISAT and MCST test methods

The previous section stressed that similar trends were observed for the test data from the two methods as a function of the different variables considered in the experimental program.  This is further strengthened when analysing the experimental relationship between the ISAT permeability index and the MCST absorption coefficient (Figure 5-11).
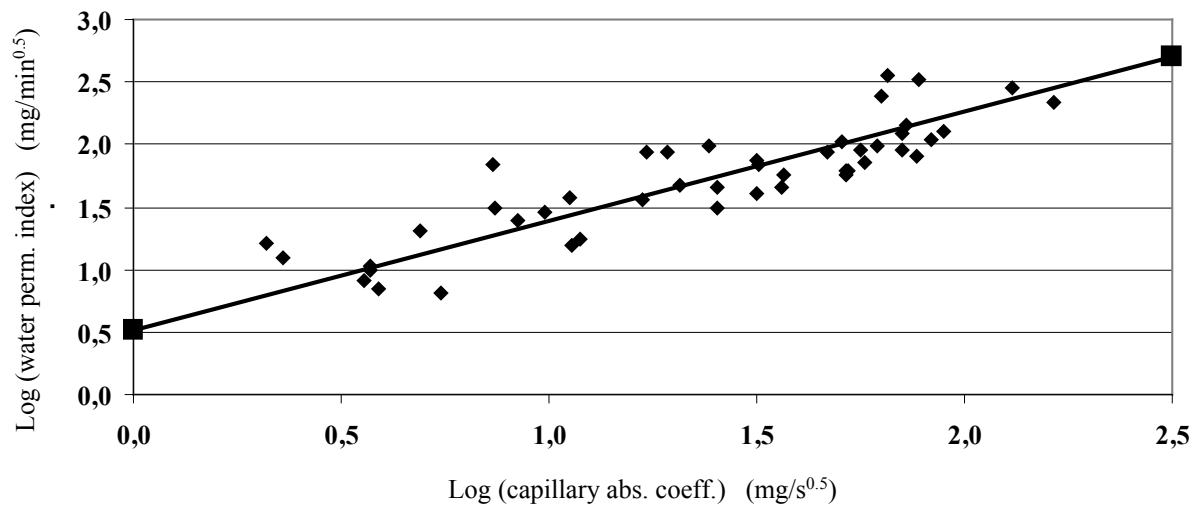
*Figure 5-11: Relationship between the MCST permeability index and the ISAT capillary absorption coefficient*

Still, it should be stressed that when expressing the results as a function of the degree of saturation of concrete, the coefficients of correlation are systematically higher with MCST data than with ISAT, as found in Table 5-4.

The higher variation and dispersion of results for ISAT may stem from the difficulty in performing the test, notably in achieving a watertight connection between the metallic ring and the substrate (Figure 5-2). Besides, dispersion of the results in this test can be aggravated by the following factors:

- Intrinsic concrete variability – the microstructure of the concrete may vary spatially, especially over the surface;
- Evaluation of the degree of saturation – even if the companion specimens are stored in the same conditions as those of the concrete slabs, their actual saturation level is most likely not exactly the same;
- Test conditions – tests are performed under laboratory conditions (50% R.H. and 23°C). This means that the specimens are taken out of their storage conditions during testing and water vapor exchanges may occur.

| Surface preparation | ISAT *vs.* saturation % (semi-log) | | MCST *vs.* saturation % (semi-log) | |
|---|---|---|---|---|
| | r | r² | r | r² |
| Water jetting (HD) | -0.80 | 0.64 | -0.90 | 0.80 |
| Sandblasting (SB) | -0.73 | 0.53 | -0.96 | 0.91 |
| No treatment (NT) | -0.78 | 0.62 | -0.96 | 0.93 |

When "out of the range" values are neglected, relatively good correlations are found, with correlation coefficients of 0.82, 0.73 and 0.90 for HD, SB and NT surface treatments, respectively (Figure 5-12).

## Bond test results

The influence of saturation level on bond between repair and substrate was evaluated by conducting series of pull-off tests for 2 types of PCC repair mortars referred to as Mortar A and Mortar B, with the following characteristics:

- Mortar A is a proprietary cement-based repair mortar having a 95/1000 water to powder content ratio and placed after the application of a bonding agent (cement-based slurry), with a characteristic bending strength of 10.9 MPa, a characteristic compressive strength of 48.36 MPa, and a density of 2.18;
- Mortar B is a proprietary polymer-modified mortar (acrylic) having a 105/1000 liquid to powder content weight ratio and placed after the application of a bonding agent (polymer-modified slurry), with a characteristic bending strength of 11 MPa, a characteristic compressive strength of 55 MPa, and a density of 1.87.
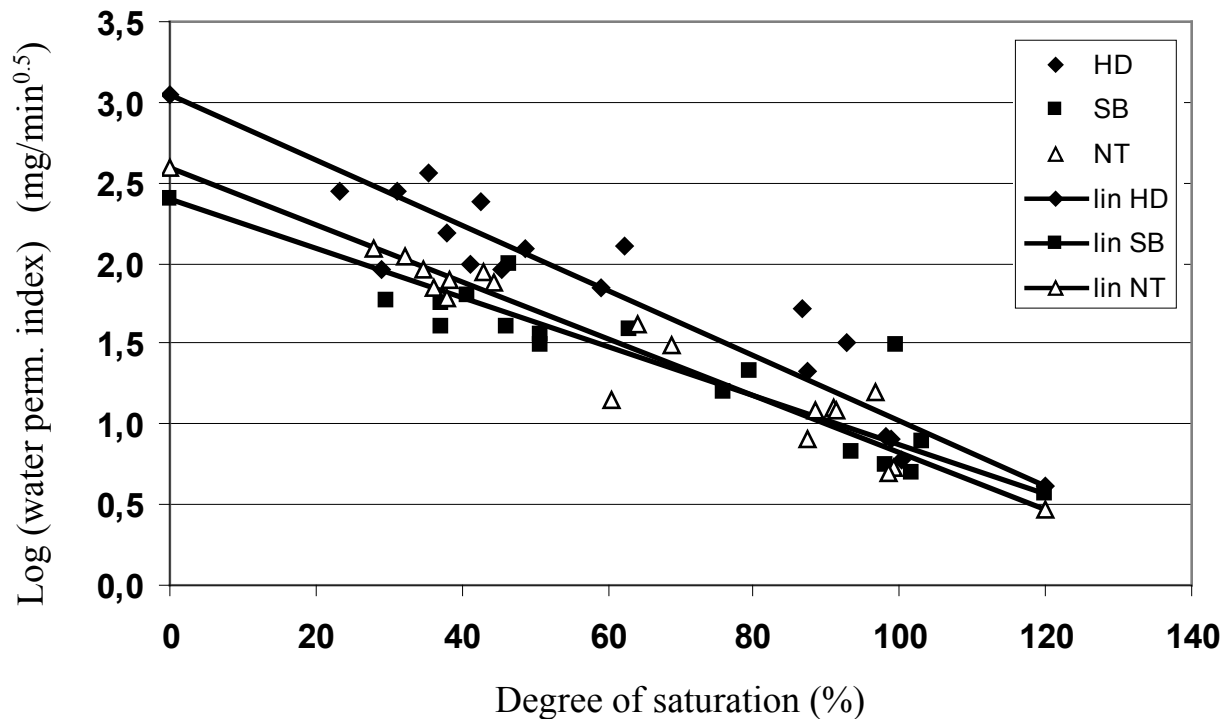
*Figure 5-12: Relationship between the ISAT permeability index and the degree of saturation of concrete, after elimination of out-of-range data*

The repair systems were placed on the sandblasted concrete slabs, after application of a bonding agent, either with a dry or wet consistency. The substrate concrete had an average tensile strength of 4.16 MPa. Concrete slabs were stored as aforementioned (Table 5-3) in order to reproduce different saturation levels covering a range from approximately 40 to 100% R.H.

The slabs were removed from the protecting bags they were kept in only once the mortar and/or slurry components were adequately mixed and ready to be poured. A 10-mm thick mortar layer was then placed on the test slabs positioned vertically, after application of slurries with wet and dry consistencies respectively on each half of the slab. The delay between application of the slurry and application of the mortar was about 30 seconds, in accordance with the manufacturer's recommendations. After placement, the test specimens were stored for 28 days at 23°C and 65% R.H. Three core samples (50 mm in diameter) were taken from each half slab, and bond strength was determined in accordance with EN 1542. For all test specimens, failure occurred within the concrete substrate, just below the surface (quasi-adhesive failures).

The test results are summarized in Tables 5-5 and 5-6 and plotted in the graphs of Figures 5-13 and 5-14. It can readily be seen that the recorded bond strength is influenced by the water saturation level of the substrate for both repair mortars A and B. The recorded bond strength is is observed to reach the highest values for intermediate saturation levels (grossly 55 to 90%). Below and above this intermediate range, a decrease in bond strength is experienced. A too low saturation level may hamper cement hydration in the repair material, while a moisture content close to saturation could adversely influence the attraction/repulsion force balance at the interface (complex interaction between the solid particles, liquid phase, porosity, kinetics of contact, and adhesion properties). In either case, the end result is a lower bond strength. Optimal moisture

conditioning of the substrate appears to be below 100% R.H., but extend over a rather wide range which could relatively easily be reached in field conditions.  Hence, provided that the moisture content within the substrate surface is kept away from extremes, specified bond strength could be achieved for a rather wide range of saturation levels and would not be very sensitive to slight variations of water content.

*Table 5-5: Bond strength results obtained for Mortar A overlays*

| Degree of saturation (%) | Avg. bond strength (MPa) | |
|---|---|---|
| | Dry slurry | Wet slurry |
| 50 | 1.41 | 1.34 |
| 60 | 2.90 | 2.72 |
| 70 | 2.95 | 1.41 |
| 100 | 1.09 | 1.43 |
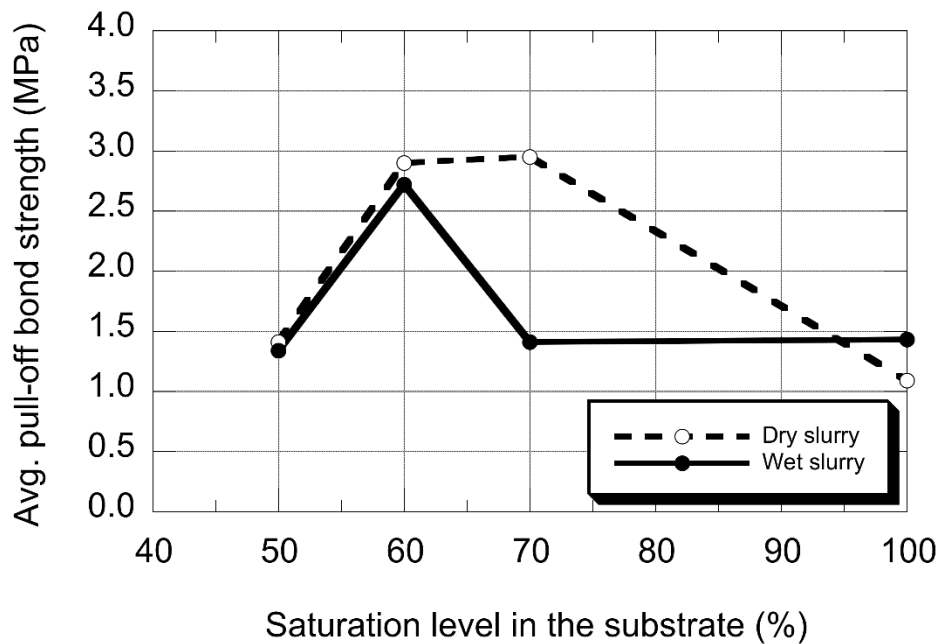


*Figure 5-13: Relationship between Mortar A bond strength and the concrete substrate saturation level*

In the case of Mortar A, the recorded bond strength values with dry slurry are somehow higher than the ones recorded with wet slurry (Table 5-5), a result which could be related to the fact that the water-based material (not a polymeric emulsion) potentially increased the effective W/C ratio (and, as a result, increased the porosity) in the interfacial zone.  For Mortar B, the influence of the slurry water content upon bond strength seems to be negligible for substrate saturation levels ranging between 70 and 90% (Table 5-6).  Out of this range, the wet slurry yielded better results.

*Table 5-6: Bond strength results obtained for Mortar B overlays*

| Degree of saturation (%) | Avg. bond strength (MPa) | |
|---|---|---|
| | Dry slurry | Wet slurry |
| 50 | 0.83 | 2.32 |
| 52 | 2.80 | 2.14 |
| 55 | 2.09 | 2.89 |
| 70 | 2.75 | 2.65 |
| 90 | 3.54 | 3.36 |
| 93 | 2.13 | 3.06 |
| 97 | 1.81 | 2.58 |
| 100 | 1.43 | 1.48 |
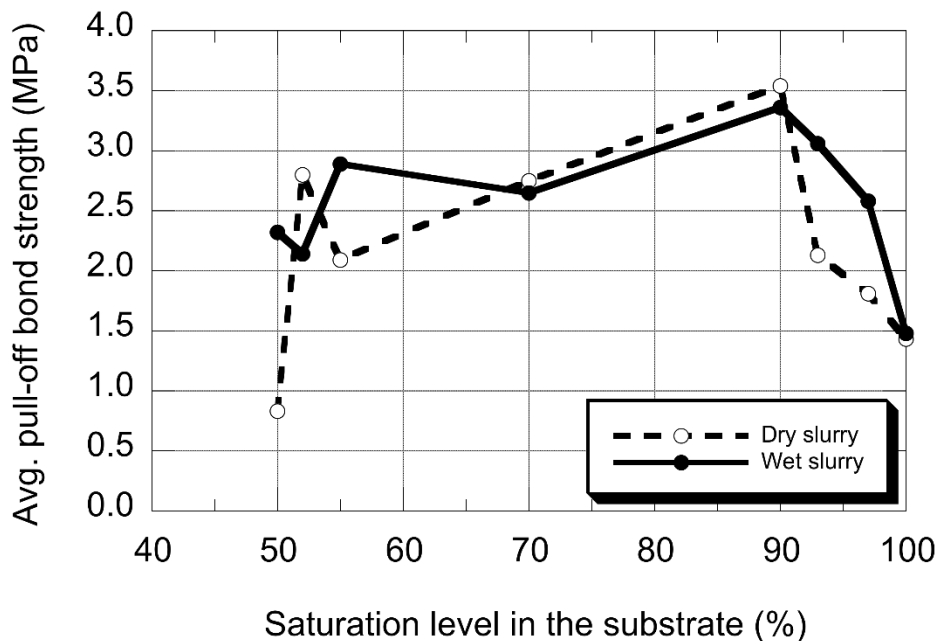


*Figure 5-14: Relationship between Mortar B bond strength and the concrete substrate saturation level*

Establishing the optimum moisture conditioning level for a specific concrete substrate is one of the critical factors affecting the bond strength in the overall quality and, potentially, the longevity of composite repair / overlay systems. Saturation levels of the concrete substrate surface at the time of repair material application in fact had a significant impact on the absorption of moisture and fine particles from the fresh (plastic) repair material, and as a result on the recorded bond strength results.

It should be stressed here that pure cement-based materials and polymer-based materials exhibit different behaviors when affected by water at the interface [22].

# Conclusions

The following conclusions were made from the experimental work performed in this task:

- ISAT is an attractive test method for performing a quantitative test to evaluate the saturation level of a concrete substrate: it is compact, cost-effective and rapid. ISAT results (permeability index) are not sensitive to concrete compressive strength, at least in the range from 30 to 50 MPa. They are influenced by the substrate surface quality, but it is difficult to conclude whether this is due to surface roughness, microcracking, or a combination of both. The relatively high variation and dispersion characterizing the ISAT test results may stem from the difficulty of performing the test on rough concrete surfaces (for instance after water jetting).

- The MCST test yielded clearer trends and less dispersed information than the ISAT test, as well as a better correlation with water content measurement.

- A good correlation was found between the water absorption index and the capillary absorption coefficient determined using both the ISAT and MCST tests.

- There is a large range of saturation levels (50 to 90%) where bond strength remains high and constant, which seems to limit the influence of environmental conditions on adhesion of cement-based repair systems. The bond strength is relatively low for low saturation levels ($\leq 50\%$), but it reaches higher values for saturation levels comprised grossly between 55 and 90% (this range likely varies depending on the nature of the overlaying material).

- When an acrylic emulsion is used as a bonding agent, the highest saturation levels induce a water film at the interface, which is incompatible with polymeric material and reduces the effectiveness of adhesion.

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# Section 6 - Evaluation of the Effect of Substrate Concrete Carbonation upon Repair Bond Strength (Task 5)

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# Tables

# Section 6 - Evaluation of the Effect of Substrate Concrete Carbonation upon Repair Bond Strength (Task 5)

This part of the report evaluated the influence of substrate concrete carbonation on repair bond strength development. The experimental work has been carried out at the U.S. Bureau of Reclamation, Denver (CO).

## Introduction

ACI defines "carbonation" as the "*reaction between carbon dioxide and a hydroxide or oxide to form carbonate, especially in cement paste, mortar or concrete; the reaction with calcium compounds to produce calcium carbonate* [1]." The alkalinity of the concrete initially rises to a pH of about 12.8 and higher due to the calcium hydroxide released during the cement hydration. However, alkalis in concrete exposed to the atmosphere react with acidic components of the atmosphere, particularly with carbon dioxide ($CO_2$). As a result, the calcium hydroxide is converted to calcium carbonate. The reaction of carbon dioxide with the calcium hydroxide in concrete is called "carbonation". Carbonation as a process mainly affects the capability of concrete to protect embedded steel reinforcement from corrosion. Therefore, the attention with regard to carbonation in reinforced concrete is essentially paid to its electrochemical effects.

One of the issues sometimes ignored or overlooked is the fact that carbonation also alters a number of physical properties of concrete [2]. Carbonation has an effect of strengthening and densification of cement-based materials, which is associated with forming calcium carbonate, and depends on the type of cement used in concrete. In Portland cement-based concrete, carbonation can lead to an increase in compressive strength exceeding 50 percent. When using cements incorporating natural pozzolans or supplementary-cementing materials such as silica fume, fly ash and slag, the changes in strength by carbonation are only marginal [3].

Another very important consequence of carbonation is a change in the void space and the permeability of carbonated concrete. The calcium carbonate fills the fine pores around the larger voids within the pore structure [2]. Blocked pores on a concrete surface may affect the wetting and suction of repair materials.

The term "adhesion" describes the condition in the boundary layer between two connecting materials with a common interface. Adhesion mechanisms can be divided basically into mechanical interaction, thermodynamic mechanisms, and chemical bonding [4]. Mechanical adhesion in repaired concrete members relies on the hardening of the repair material mixture inside the open cavities, pores and asperities of the substrate surface, and the resulting physical anchorage. The cement paste absorption into open pores of the substrate concrete plays an important role in the anchoring effect, as it draws paste into the substrate. Thus, the phenomenon of carbonation, by producing a denser surface layer with so-called "clogged" pore system, which reduces the absorptivity of the substrate concrete, might affect negatively the bond strength in repair systems.

Through proper surface preparation the carbonated concrete surface can usually be removed, thereby exposing a "fresh" non-carbonated surface. However, in some cases it involves extensive removal of otherwise sound concrete. Also, long periods of time between concrete surface preparation and repair placement may result in carbonation of the exposed substrate surface.

The influence of carbonation of the concrete substrate surface upon bond strength of concrete repairs has not been investigated extensively. In addition, from the limited published research data, there is no consensus regarding the effects of carbonation in the scientific community. According to Schrader [5], carbonation of the substrate can result in a soft surface and dusting, which may cause in turn poor bond strength if an overlay is applied. As a matter of fact, test results from Gulyas et al. [6] show that carbonation may decrease bond strength significantly. On the contrary, Block and Porth [7] found in their studies that a carbonated substrate does not affect the repair pull-off bond strength. The conflicting data and information reported in the technical documentation prompted the experiments performed in Task 5.

The objective of the task reported in this chapter was thus to evaluate the effect of carbonation of the concrete substrate upon tensile bond strength of repairs.

## Methodology and experimental program

In order to evaluate the effect of carbonation of the concrete substrate upon tensile bond strength of repairs, the following basic variables were selected for investigation:

- Concrete surface preparation technique;
    - Chipping hammer;
    - Sandblasting;

- Carbonation;
    - No carbonation (control specimens);
    - Carbonation in a controlled environment ($CO_2$ cabinet).

The experimental test program carried out at the U.S. Bureau of Reclamation (USBR), Denver, CO is summarized in Table 6-1. Further details pertaining to the test variables, the specimen sizes, the preparation and conditioning of these specimens, and the bond tests are discussed below.

*Table 6-1: Test program*

| Item | Details |
|---|---|
| **Test specimens**<br><br>    • Base slabs | - 406.4×406.4×101.6-mm base concrete slabs<br>- test slab series: 18 slabs prepared with 27.5-MPa ordinary portland cement (OPC) concrete<br>- base slabs moist cured for 3 days after casting and exposed to drying for more than 6 months prior to repair<br>- slabs submitted to 2 different surface preparation methods<br>- slab conditioning for 10 weeks after surface preparation:<br>    • 8 slabs protected from carbonation (control)<br>    • 10 slabs stored in a carbonation chamber |
|     • Repaired test slabs | - repair performed on pre-wetted slabs to achieve saturated surface dry (*SSD)*<br><br>- 75-mm thick overlays with a 27.5-MPa OPC concrete mixture |
| **Investigated surface preparation techniques**<br>(surface prep. prior to repair) | - sandblasting (SA)<br>- jackhammering (JH) |
| **Characterization test methods for**:<br><br>    • Bond strength (28-d) | - pull off test |

A series of eighteen plain concrete slabs (406.4×406.4×101.6-mm) were manufactured for the test program. The slabs were cast using a 27.5-MPa ready-mix concrete. The basic properties of the concrete mixture are displayed in Table 6-2. The slabs were first exposed to drying for six months in order to achieve relative dimensional stability, after what surface preparation and conditioning were performed.

*Table 6-2: Concrete mixture characteristics and mechanical properties*

| Material | Test slab concrete mixture | Repair concrete mixture |
|---|---|---|
| Nominal strength | 27.5 MPa (4000 psi) | 27.5 MPa (4000 psi) |
| Mixture characteristics | ASTM Type I cement 14-mm (½ in.) coarse agg. | ASTM Type I cement + black pigments (6% of Cement weight.) 10-mm (3/8 in.) coarse agg. |
| Fresh concrete properties[1] Slump (mm) Air content (%) | 125-175 4-7 | 125-175 5-8 |
| Compressive strength[2] (MPa) 7 d 28 d | 20.9 28.3 | 21.9 27.5 |
| Splitting tensile strength[3] (MPa) 7 d 28 d | n/a n/a | 2.34 2.92[5] |
| Elastic modulus[4] (GPa) 7 d 28 d | n/a n/a | 21.3 22.9 |

[1]Specified values; [2]ASTM C39, [3]ASTM C496, [4]ASTM C469; [5]at 10 days.

Two representative surface preparation techniques were selected for investigation: sandblasting (SA) and jackhammering (JH) with a 7-kg handheld chipping hammer. Sets of nine base slabs were prepared with each of these two techniques. The resulting surface profiles are shown in Figure 6-1.

In each set of prepared test slabs, four (4) control slabs were protected from carbonation (control), and five (5) slabs underwent controlled carbonation in a laboratory carbonation chamber. Throughout the conditioning period, the control slabs were covered with polyethylene sheets sealed with duct tape to prevent carbonation. The carbonation chamber used to condition the test specimens has a storage capacity of ten (10) slabs, thus accommodating five specimens of each set. The slabs were conditioned for 10 weeks, to reach a carbonation depth of at least 3 mm, which was assessed through phenolphthalein measurements performed on cores (Figure 6-2). As a matter of fact, as seen on the picture, the carbonation front had reached a depth of the order of 10 mm.

*Figure 6-1: Base slabs after surface preparation*
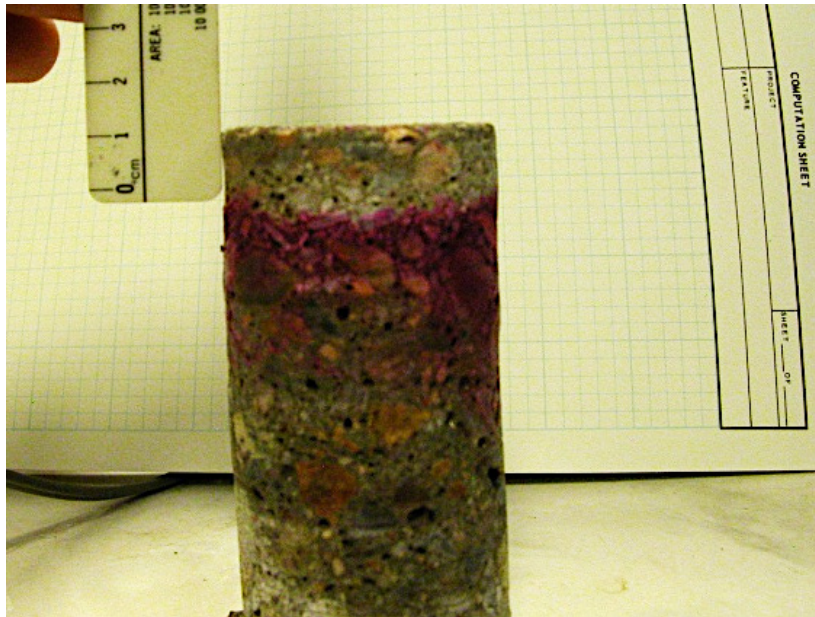


*Figure 6-2: Carbonation depth measurement on a freshly exposed concrete section using phenolphthalein*

All slabs were then overlaid (75-mm thick overlay) with a 27.5-MPa ready-mix concrete mixture similar to the one used originally for the base slabs, except for the maximum aggregate size was smaller (10 mm instead of 14 mm).

The repair concrete mixtures characteristics and properties are also shown in Table 6-2. The repaired specimens were moist-cured for 3 days, after which they were air-dried for at least 25 days, until the bond strength tests were carried out (at least 28 d). For bond strength evaluation, nine (9) pull-off bond tests were performed on each repaired slab. The coring layout used on each test slab is shown in Figure 6-3.
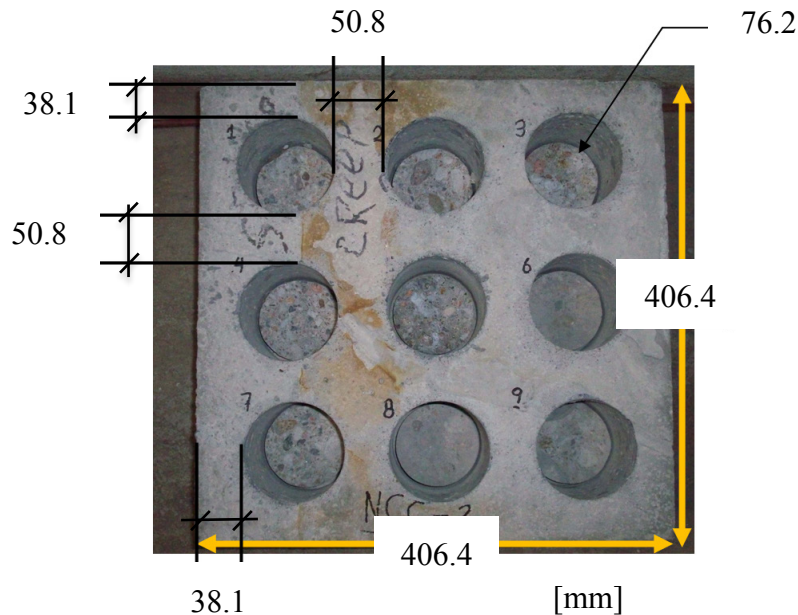


*Figure 6-3: Slab coring template*

The pull-off bond test layout and apparatus used in the program are shown in Figure 6-4. The repair material and the substrate had reached respectively 3 months and one year at the time the pull-off tests were performed.



*a) Dollies installed at the repair surface*          *b) Instrumented pulling device*

*Figure 6-4: Pull-off bond test – experimental layout*

# Results and analysis

The bond strength results yielded for the four test series are summarized in Table 6-3 and in Figures 6-5 and 6-6.

*Table 6-3: Summary of pull-off test results for the various investigated combinations*

| Surface preparation method | Test parameters | Location of failure | | | | | Overall |
|---|---|---|---|---|---|---|---|
| | | Core end | Substrate | Interface | Repair | Dolly | |
| Sandblasting non-carbonated | Number of test results (%) | 18 (54.6) | 6 (18.2) | 5 (15.2) | 1 (3.0) | 3 (9.1) | 33 (100.0) |
| | Avg. strength [MPa] (std. dev.) | **2.48** (0.29) | **2.56** (0.24) | **2.43** (0.40) | **2.65** - | **2.78** (0.19) | **2.52** (0.29) |
| Sandblasting carbonated | Number of test results (%) | 17 (38.6) | 1 (2.3) | 20 (45.5) | 3 (6.8) | 3 (6.8) | 44 (100.0) |
| | Avg. strength [MPa] (std. dev.) | **2.45** (0.22) | **2.33** - | **2.53** (0.32) | **2.68** (0.26) | **2.49** (0.20) | **2.50** (0.27) |
| Jackhammering non-carbonated | Number of test results (%) | 3 (9.1) | 1 (3.0) | 29 (87.9) | 0 (0.0) | 0 (0.0) | 33 (100.0) |
| | Avg. strength [MPa] (std. dev.) | **2.27** (0.34) | **2.47** - | **2.18** (0.32) | **-** (-) | **-** (-) | **2.20** (0.32) |
| Jackhammering carbonated | Number of test results (%) | 2 (11.1) | 0 (19.4) | 40 (69.4) | 0 (0.0) | 0 (0.0) | 42 (100.0) |
| | Avg. strength [MPa] (std. dev.) | **2.55** (0.30) | **-** - | **1.82** (0.38) | **-** (-) | **-** (-) | **1.86** (0.40) |
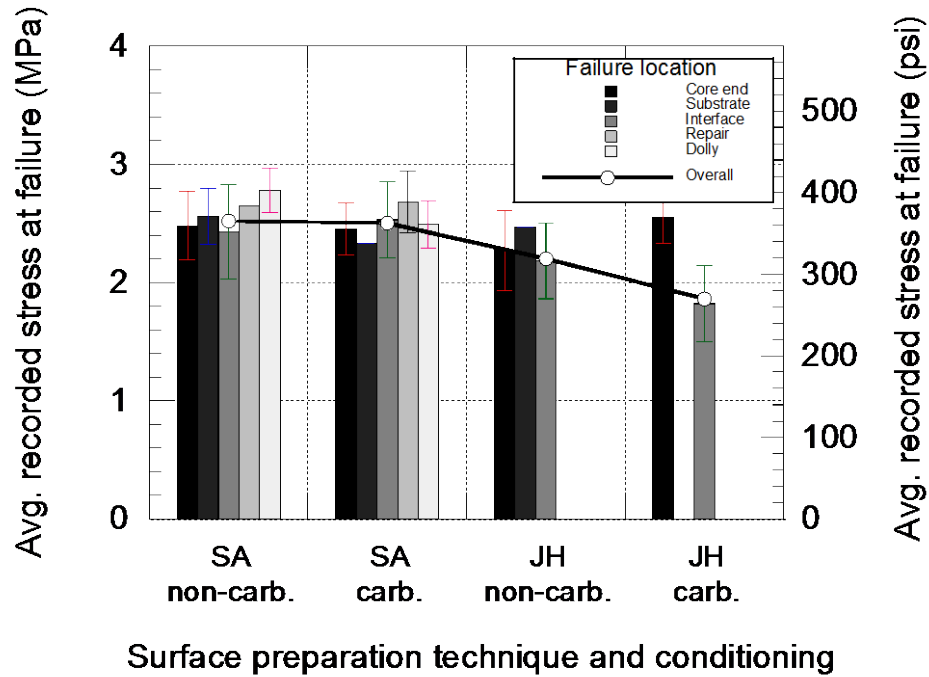
*Figure 6-5: Results of pull-off tests performed after repair for the various investigated combinations (note: SA – sandblasting; JH – jackhammering)*
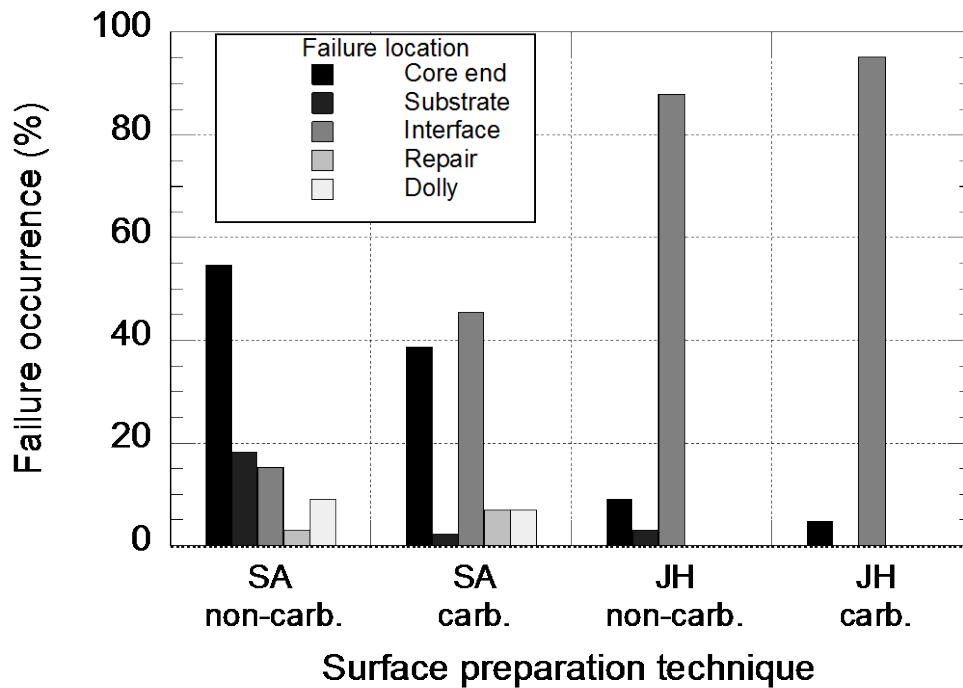


*Figure 6-6: Distribution of failure location in pull-off tests performed after repair for the various investigated combinations (note: SA – sandblasting; JH – jackhammering)*

Although the average values are a little larger, the bond strength test results obtained for the non-carbonated slabs are overall consistent with the data, trends and main conclusions reported in section 2 of this report, in particular the detrimental effect of bruising induced by concrete jackhammers.

For substrate surfaces prepared by sandblasting, no difference in bond strength is observed between carbonated and non-carbonated concrete substrates. There appears to be a slight shift with failures occurring more frequently at the interface than within the substrate in the case of carbonated substrates, but it did not affect the average strength. Actually, for the test slabs prepared by sandblasting, with or without carbonation, the recorded bond strength values are very similar irrespective of the failure location.

Conversely, for substrates prepared with a concrete jackhammer (chipping hammer), a significant reduction (16 %) in bond strength is observed for carbonated surfaces as compared to non-carbonated surfaces. Such a different behavior may be associated with the presence of micro-defects (bruising) induced into the surface prepared by chipping hammer. These defects may actually be harmful in two ways with respect to the subsequent occurrence of carbonation. First, microcracks present at the surface are pathways for easier and deeper $CO_2$ penetration. In addition, the chemical reactions involved in the carbonation process result in a negative volume change, which is generally referred to as carbonation shrinkage [8]. In a layer already weakened by bruising, the easier penetration of the carbonation front and the accompanying shrinkage likely induce further damage and, as a result, adversely affect bond strength development at the time of repair.

The results yielded in this task call for further attention with regards to the use of concrete jackhammers and chipping hammers for surface preparation prior to repair. Hammers have already been shown to induce some extent of microcracking in the prepared substrate, which is detrimental to repair bond. If carbonation is allowed to occur prior to repair, there clearly is a risk for the situation to worsen.

## Conclusions

The effects of concrete substrate carbonation have on the tensile bond strength of the repair material to the substrate were investigated. Based on the experimental data that were generated in this task, the following conclusions can be drawn:

- For substrate surfaces prepared by sandblasting there was no difference in bond strength found between carbonated and non-carbonated concrete surfaces;
- For substrate surfaces prepared by chipping a significant reduction (16%) of bond strength was documented for carbonated surfaces compared to non-carbonated;

The different effects of carbonation were attributed to the micro defects (bruising) of the surface prepared by chipping hammer, further aggravated by the carbonation process and inherent volume changes (carbonation shrinkage).

The most immediate practical outcome of this task is the added caution required when selecting a surface preparation method prior to concrete repair.  Concrete breakers inevitably leave some bruising within the superficial layer of the concrete surface being treated and its detrimental effect upon repair bond can only be amplified if carbonation is allowed to occur prior to repair.

However, it appears that carbonation may have little or no impact on bond strength for an otherwise sound, properly prepared concrete substrate surface.  It should be stressed here that the limited number of tests performed using a single repair material does not allow for broad conclusions about the overall effect of carbonation on tensile bond strength.  Other repair materials may not necessarily behave the same way in bond development to the carbonated surfaces.

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# Section 7 - Specification Guidelines for Surface Preparation of Concrete Prior to Repair (Task 6)

# Contents

# Section 7 - Specification Guidelines for Surface Preparation of Concrete Prior to Repair (Task 6)

This section provides guidance on the surface preparation of concrete prior to repair and overlay. It can be used as a starting point by individuals involved in developing project specifications who are competent to analyze the significance and limitations of these guide specifications' content and who will accept responsibility for the application of the material and provisions it contains.

The proposed guidelines were developed for surface preparation of existing concrete for repair/overlay with Portland cement concrete and pre-packaged cement-based materials. The document is based on the results generated in the various tasks of the project and the review of best practices and knowledge in the area of concrete repair.

## 2. General Introduction

These specification guidelines contain recommendations for surface preparation of concrete for repair and overlay. The document summarizes current knowledge, best practices and results of the research concerning the surface preparation of concrete prior to application of repair/overlay materials. The guide specifications are applicable to repairing damaged or deteriorated concrete structures, correcting design or construction deficiencies, or upgrading a structure for new uses, or to meet more restrictive code requirements.

The specification details removal of concrete, preparation of the concrete substrate surfaces for repair and quality control/quality assurance of the work performed. These guide specifications are recommended for design engineers and personnel who face the task of introducing the best practices for concrete surface preparation on repair and rehabilitation projects.

To achieve the goal of a durable repaired concrete structure the specifier of a given repair project needs to use equipment, techniques and procedures that are appropriate for the project objectives, deterioration mechanism(s), environmental conditions, structural circumstances and other local conditions and limitations which exist for the specific structure or part of the structure under consideration.

Repair geometries, locations, access, amount and spacing of reinforcement, climatic conditions, available equipment, local engineering and labor skills, local regulations, etc. have to be analyzed and addressed in properly tailored specifications.

The success of concrete repairs is dependent on determining the cause and extent of concrete distress or deterioration, establishing realistic repair objectives, and developing a repair strategy to address the problem. Typical steps of a systematic repair are as follows.

1. A condition survey with a scope consistent with the perceived condition of the structure and the owner's repair objectives, performed by qualified individuals, to document and evaluate visible and non-visible defects and damage as well as potential damage.

2. An assessment of the application and service conditions to which the concrete repair is, or will be, exposed.

3. Determination of the cause of the damage or deterioration necessitating the repair; for example, mechanical damage such as impact or abrasion; design, detailing or construction deficiencies; chemical damage, such as alkali-aggregate reaction; physical damage related to cycle of freezing and thawing or thermal movements; and corrosion of the steel reinforcement caused by improper placement, carbonation of the concrete, or chloride ingress into the concrete.

4. Determination of the repair objectives, including desired service; and durability planning including service life modeling.

5. Design of a repair project including appropriate specification for a particular project.

6. In the specific repair project, the specifier should consider outside constraints such as limited access to the structure; the operating schedule of the structure; any limitation imposed by the owner of the structure, including the cost; the required useful life of the repaired structure.

7. Consideration shall be given to the physical, chemical and electrochemical condition of the existing concrete substrate, the ability of the structure to carry loads, movement and vibration during repair, ambient conditions, and the characteristics of substrate materials and those of the repair materials and systems.

8. Safety and structural stability before, during, and after repair must be maintained in accordance with the specific project specifications and design.

The following requirements shall be met:

1. The achievement of the required condition of the substrate regarding cleanliness, roughness, cracking, tensile and compressive strength, chlorides and other aggressive agents, depth of carbonation, moisture content and temperature.

2. The achievement of the compatibility of the existing concrete and reinforcement with the repair and protection materials and systems, and compatibility between different repair and protection products, including avoiding the risk of creating conditions which may cause acceleration of corrosion.

3. The achievement of the specified requirements, characteristics and properties of repair materials and systems and the composite repair system regarding the fulfilment of their purpose to prolong the useful service life of the structure.

4. The achievement of the required repair application conditions regarding ambient temperature, humidity, wind force and precipitation, and any temporary protection when needed.

# 3. Concrete removal

### 3.1. Description

This section specifies procedures, equipment and requirements for the removal of concrete in areas designated for repair. The process of concrete preparation for repair is the process by which sound, clean, and suitably roughened surfaces are produced on concrete substrates. This process includes the removal of unsound and, if necessary, sound concrete and bond inhibiting foreign materials from the concrete and reinforcement surfaces, opening the concrete pore structure, and reinforcement damage verification and repair, if necessary.

Unsound or deteriorated concrete shall be defined as: concrete affected by spalling, delamination, disintegration and concrete in areas with severe cracking where active corrosion of reinforcing steel has been detected. "Unsound" concrete suggests that the material is in a reduced physical condition and hence relatively easy to remove. Alternatively "sound" concrete in all probability may be in physically good condition and involves considerable effort for its removal. Concrete contaminated with chlorides and/or carbonated concrete is usually physically sound concrete.

Concrete removal usually involves unsound material. However, some sound concrete is also removed to permit for adequate repair geometry, to remove contaminated concrete, to prepare embedded reinforcement, and to permit structural modifications. The effectiveness of various concrete removal techniques may differ for unsound and sound concrete and a combination of techniques may be necessary.

Proper attention to surface preparation is essential for a durable repair. Regardless of the cost, complexity and quality of the repair material and application method selected, the care with which concrete is removed and concrete reinforcement surfaces are prepared will often determine whether a repair project will be successful.

The methods used to remove the deteriorated or contaminated concrete and prepare the concrete and reinforcement to receive the repair material must not weaken the surrounding sound concrete and reinforcement.

### 3.2. Structural safety

Before starting removal of existing concrete, the effect of the removal on the structural integrity should be reviewed. In cases where removal of deteriorated concrete and/or severely corroded reinforcing steel can affect the load carrying capacity of the structure or its elements, a temporary shoring system should be provided to relieve the loads from the structure or its member being repaired. Caution needs to be exercised in order that the safety of the structure is not jeopardized by repair activities.

Details of shoring to be used shall be provided by the Contractor and shall be designed and sealed by a Professional Engineer; this does not, however, in any way relieve the Contractor of his responsibility for the safety and adequacy of the shoring system.

The limitations for concrete removal such as the depth, reduction of cross section, the amount of concrete removed from the top surface, etc. shall be subject to the restrictions described in the contract.

### 3.3. Precautions prior to concrete removal

The areas where concrete is to be removed shall be examined to determine if there are electrical conduits, utility lines, or other embedments which may be damaged during removal.

If required, the Contractor shall enclose work area with a suitable barrier to confine dust and debris inside the work areas. The enclosures shall be securely constructed-and inspected by the Contractor each working day to ensure that there are no holes or tears.

The Contractor shall ensure that the level of equipment exhaust fumes (such as from air compressors or portable generators) in enclosed or confined areas is within acceptable limits. If the fume level cannot be kept at an acceptable level using the existing exhaust fans, then the Contractor shall use other equipment or relocate the equipment so that the fumes can be properly exhausted away from occupied areas.

All necessary precautions shall be taken to ensure that dust or falling debris does not constitute a hazard to personnel, equipment, the structure, its occupants and the general public. Effective means of clearing dust and debris away from the working area shall be continuously implemented.

The extent and depth of concrete removal required shall be measured and recorded on drawings by the Contractor and agreed with the Engineer as the work proceeds.

### 3.4. Concrete removal geometry

The location, number, and extent of defects shown in the Contract are indicative only. The true location, number, and extent of defects requiring repair can only be assessed properly by close inspection and other testing during the course of concrete removal. The limits of each repair shall be marked with chalk or paint by the Contractor as a series of straight lines on the surface. The limits of each repair shall be subject to approval by the Engineer.

Areas requiring repair shall be modified to provide for simple layouts. The layouts shall be designed to reduce boundary edge length and eliminate acute angles. Excessive or complex edge conditions are usually produced by trying to closely follow the shape of the deteriorated concrete. Such edge conditions often result in shrinkage stress concentrations and cracking.

The perimeters of repairs that involve concrete removal shall provide right angle cuts to the concrete surface by saw cutting, chipping or hydrodemolition (water jetting).

### 3.5. Saw cutting

A saw cut along the perimeter of the area where concrete is to be removed shall be provided to reduce edge spalling and to provide a sound edge surface against which the repair material will be placed and compacted. The saw cuts shall be made to a depth of approximately ½ inch (13

mm) to 1 inch (25 mm). Where the cover to the reinforcing steel is low, cutting may have to be omitted in order to avoid damage to reinforcement.

The saw cut surfaces shall be roughened prior to application of a repair material. It is best achieved by sand or grit blasting at the same time as cleaning of exposed reinforcement. Care needs to be exercised when roughening the disc-cut surfaces to avoid damage to the repair cavity edges.

The advantages of the saw cutting procedure include the following:

- The saw leaves vertical edge faces;
- The forces experienced by the concrete during chipping are isolated within the sawed boundaries;
- Very little spalling of the remaining concrete occurs;
- Removing the deteriorated concrete within the sawed boundaries is usually easier and faster when the boundaries are sawed than when they are not sawed;
- Most crews are familiar with the method.

The disadvantages of the saw cutting procedure include the following:

- More workers are required than in the other procedures;
- Since water is usually used when sawing, the repair area is saturated for some time, possibly delaying the repair;
- Saw overcuts often occur and weaken the repair area and must be cleaned and sealed. Without overcuts, extra chipping work is needed at the corners
- The polished, vertical repair boundary faces need to be cleaned and roughened, or poor bonding may result;
- If more unsound concrete is later found beyond the sawed boundaries, the operation should be repeated to saw new boundaries causing extra work and further delays.

3.6. Chip cutting

The boundaries in the chip cutting procedure are the same as in the saw cut procedure, except the repair boundaries are not sawed. The concrete in the center of the repair area is removed using a light jackhammer with a maximum weight of 15 lb (6.8 kg). The concrete near the repair borders is then removed using a light jackhammer with a maximum weight of 15 lb (6.8 kg) and hand tools. The work should progress from the inside of the repair toward the edges, and the chisel point should be directed toward the inside of the repair.

The advantages of the chip cutting procedure include the following:

- The rough vertical edge produced promotes bonding;
- There are no saw overcuts;
- It has fewer steps than the saw cut method;
- Spalling is controlled by using light hammers at the edges.

The chip and patch procedure may be faster because it has fewer steps; the patch boundaries are not sawed, and there are no saw overcuts to be cleaned and sealed. Once joint sawing is complete, the saw is not needed again, even if more unsound concrete is later found beyond the boundaries.

The disadvantages of the chip cutting procedure include the following:

- Sound concrete may be damaged by chipping hammers;
- Hammers can cause feathered (thin) repair material edges, which often leads to poor repair performance;
- Vertical sides are difficult to achieve.

Water-wash equipment shall be used to remove sawing slurry from the repair area before it dries.

3.7. Concrete removal techniques

3.8. General

Concrete removal methods are categorized by the way in which the process acts on concrete. The general categories are impacting, blasting, cutting, milling, pre-splitting, and abrading. ACI 546R Concrete Repair Guide provides a description of these categories, lists the specific removal techniques, and provides a summary of information on each technique.

Among the various concrete removal methods and categories, only breakers (jackhammering) and hydrodemolition (water jetting) are addressed in this section. Where a significant depth of concrete is to be removed, these are the main options available. Most of the other methods are intended to remove the skin concrete and / or to texturize the surface.

3.9. Jackhammering

Impacting methods with breakers (jackhammers) are the most commonly used concrete removal systems. They generally employ repeated striking of a concrete surface with a high energy tool to fracture and spall the concrete. Impacting methods include a wide range of devices, from hand-held chipping hammers to large machinery-mounted hydraulic breakers.

The hand-held breaker or chipping hammer is probably the best known of all concrete removal devices. Hand-held breakers are available in various sizes with different levels of energy and efficiency. The smaller hand-held breakers (15 pounds) are commonly specified for use in partial removal of unsound concrete or concrete around reinforcing steel, because they do little damage to surrounding concrete. The larger hand-held breakers (30-90 pounds) are used for complete removal of large volumes of concrete. Care shall be exercised when selecting the size of breakers to minimize the damage to existing concrete and its bond to embedded reinforcing steel.

Chipping hammers are typically classified by weight, even though breakers of similar weight do not necessarily generate the same impact force.

The percussive force used by pneumatic breakers to fracture concrete is primarily determined by the impact energy and the frequency at which the impacts occur. The impact energy is based on the mass of the piston, the size of the cylinder, and the inlet port diameter. Impact energy ranges from approximately 15 lbs. (7 kg) per blow for small tools to more than 180 lbs. (82 kg) per blow for large tools. The frequency of impact, or blows per minute, ranges from 900 blows per minute to more than 2,000 blows per minute, depending on the valve design.

Various cutting tools are available for use with hand-held pneumatic breakers. The shank end, which is inserted into the tool-retaining mechanism, is common to all. The cutting or working end can vary from a broad spade like blade to a sharp well-honed point. The vast majority of concrete removal work is done with a pointed tool, although a relatively narrow (3 in. to 4 in. [7.5 cm to 10 cm]) blade-type tool is sometimes used to remove cracked and deteriorated concrete.

The effects of the breaker on the concrete during the removal operation must be monitored to ensure minimal impact on the surrounding environment. The primary issues of concern are noise, dust, and flying debris.

The first step in the removal procedure is saw cutting the repair boundaries. The deteriorated concrete in the center of the repair is then removed using a light jackhammer with a maximum weight of 15 lbs. (6.8 kg). The work should progress from the inside of the repair toward the edges. When all unsound concrete in the repair area is removed and repair geometry is established the final procedure is to remove the concrete near the repair borders using a light jackhammer and/or hand tools.

Removal near the repair boundaries must be completed with hammers fitted with spade bits as other bits can damage sound concrete. Jackhammers and mechanical chipping tools should be operated at an angle less than 45 degrees from the vertical.

### 3.10.    Water jetting

The water jetting procedure uses a high pressure water jet to remove deteriorated concrete. A high-pressure water jet uses a small jet of water driven at high velocities commonly producing pressure of 10,000 to 45,000 psi (69 to 310 MPa) and above.

High-pressure water jetting (hydrodemolition) may be used as a primary means for removal of concrete when it is desired to preserve and clean the steel reinforcement for reuse and to minimize damage to the concrete remaining in place. Hydrodemolition literally disintegrates concrete, returning it to sand and gravel-sized pieces. This process works preferentially on unsound or deteriorated concrete and leaves a rough profile. Care must be taken not to punch through thin slabs or decks if unsound concrete exists in an area to be repaired.

High-pressure water jets in the 10,000 psi (70 MPa) range require 35 to 40 gal/min (130 to 150 L/min). As the pressure increases to 15,000 to 20,000 psi (100 to 140 MPa) the water demand will vary from 20 to 40 gal/min (75 to 150 L/min). The equipment manufacturer should be consulted to confirm the water demand. Ultra-high-pressure equipment operating at 25,000 to 35,000 psi (170 to 240 MPa) has the capability of milling concrete to depths of ⅛ inch to several inches (3 mm to approximately 50 mm).

Water jetting (hydrodemolition) should not be allowed for concrete removal if there is a possibility that unbonded post-tensioned systems are within the concrete removal zone. The only viable method of concrete removal in this situation is concrete removal using lightweight chipping hammers.

Two trial areas, one of sound concrete and one of deteriorated concrete, are then used to determine the appropriate water jetting operating parameters. These parameters include speed, pressure, and the number of overlapping passes. Using trial and error in the test areas, the water jet must be set up to prevent removing sound concrete unnecessarily. In the sound area, consistent concrete removal depth of ¾ inch behind the reinforcing bar shall be obtained. After successful cutting of the above test area, with specified depth control, the operation shall be moved to the deteriorated concrete and remove all deteriorated concrete. If a result is obtained which meets the specified requirements, these parameters shall be used as a basis for the production removal. If not, the Contractor shall repeat the trial process and recalibrate the equipment or replace the equipment until a result which meets the specified requirements is obtained. Once properly calibrated, the operating parameters should not be changed while water jetting the rest of the damaged concrete, unless the concrete changes (for example, a harder aggregate has been used in one section of the structure). If the concrete does change, the water jetting machine must be recalibrated using two new trial areas in the section with the different concrete.

All concrete within a marked for repair area should be removed to a minimum depth of 2 in (51 mm) with neat vertical faces. Then the repair area must be tested again for soundness. Any additional unsound concrete must be removed by continued water jetting.

The debris and slurry that result from the water jettting operation must be removed using a low-pressure water stream before the slurry dries and hardens on the surface of the cavity. If this is not done, the repair area may have to be refaced. Once dried, sandblasting may or may not be able to remove the dried slurry residue. Some moisture-sensitive materials may require the repair area be completely dry before placing the material.

The advantages of water jetting include the following:

- It requires fewer workers than the other procedures and large amounts of concrete can be removed relatively quickly;
- Once an experienced operator adjusts the operating parameters, only weak concrete is removed;
- The cavity surfaces produced are vertical, rough, and irregular, and enhance bonding;

The disadvantages of water jetting include the following:

- The finished surfaces are saturated. Placement must be delayed until the area dries unless the repair material is not moisture-sensitive;
- The fine slurry laitance remaining after the procedure requires careful attention during cleaning;
- A protective shield must be built around the repair area if the work is next to occupied areas;

- It can be difficult to control the depth of removal;
- Equipment is expensive;
- It can be difficult to obtain a good production rate; performance of water jetting equipment has been variable;
- The waste water and debris must be handled in an environmentally acceptable manner as prescribed by regulations.

***Important limitation of hydrodemolition in post-tensioned structures***

Although hydrodemolition will not physically damage steel tendons, it is not considered to be a viable concrete removal technique if there is a possibility of the high-pressure water coming into contact with tendons, anchorages, or both. Reasons why hydrodemolition is not considered to be a viable technique include:

1) Hydrodemolition of post-tensioned concrete elements may cause a safety problem. It is potentially dangerous because it may accidentally undercut embedded anchors and result in explosive release of prestressing force.

2) If any part of the tendon is exposed to high water pressure, water may penetrate into the tendon. The water jets will likely destroy the sheathing on the tendons, whether it is wrapped in paper, plastic, tubing, or extruded plastic. If the sheathing is damaged, the water has a direct path to the prestressing strand or wire, and corrosion may result.

3) Concrete repair projects commonly include replacement of post-tensioning strand. The water pressure used in water jetting equipment can force slurry into the sheathing. When slurry and other debris exist within the sheathing, installation of a new strand becomes very difficult. When the new strand is pushed into the existing sheathing, debris within the sheathing builds up ahead of the advancing strand. This buildup of debris can cause the sheathing to rip and "ball up" in front of the leading edge of the strand. This scenario makes strand replacement very difficult and compromises the corrosion protection or sheathing over the prestressing steel.

   More information can be found in ACI 423.4R.

### 3.11.    Treatment of reinforcing steel

The most frequent cause of concrete deterioration is the corrosion of embedded reinforcing steel. The evaluation of the condition of reinforcing steel exposed in the repair area and proper reinforcement treatment steps will ensure that the repair will not fail prematurely.

The first step in preparing reinforcing for repair or cleaning is the removal of deteriorated concrete or sound chloride contaminated concrete surrounding the reinforcement. Extreme care should be exercised to insure that further damage to the reinforcing or prestressing steel is not caused by the process of removing the concrete. Impact breakers can damage reinforcing steel if the breaker is used without regard to the location of the reinforcement. Once the larger areas of unsound concrete have been removed, a smaller chipping hammer (of the order of 15 lbs.) should be used to remove the concrete in the vicinity of the reinforcement. Care should be taken not to

vibrate the reinforcement or otherwise cause damage to its bond to concrete adjacent to the repair area.

All unsound concrete shall be chipped away.  If during the removal operation, reinforcing steel is exposed, then concrete removal around the bar shall continue to provide a minimum ¾ inch clear space between the rebar and surrounding concrete or ¼ inch larger than the maximum size aggregate in the repair material, whichever is greater.

Additional concrete removal shall be carried out along corroded exposed bars until a continuous length of 50 mm (2 in) of bar free from corrosion is exposed.  The limit of activite corrosion shall be assessed on a visual basis.  The edges of any additional areas removed shall be cut square as specified above.  The extent of concrete removal shall be agreed by the Engineer before any removal commences.

An additional length of uncorroded bar will have to be exposed if couplers or lap splices are to be used for replacement reinforcement.

### 3.12.     Concrete surface roughness

Interface roughness depends to a large extent on the method of substrate surface preparation. Mechanical methods of concrete removal normally leave the substrate surface much rougher than blast methods.  The magnitude of surface roughness for concrete repairs is commonly reported in terms of surface profile amplitude.

Unacceptably rough substrate profiles after concrete removal may be reduced through additional work using properly selected surface preparation technique.

The decisions about surface preparation and its roughness in particular, cannot be made without knowing the properties and application requirements of the selected repair/overlay material.

When applying proprietary materials (protective / waterproofing systems, coatings, mortars), the material's data sheet shall be consulted.  In many cases, ICRI Guideline No. 310.2R-2013, "Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Polymer Overlays" is explicitly referred to by the manufacturer.  The ten suggested concrete surface profile (CSP) chips provide a benchmark to aid in selecting, specifying and evaluating the required concrete surface profile, and therefore in achieving more systematically the desired results.  Each profile carries a CSP number ranging from a base line of CSP 1 (flattest) through CSP 10 (roughest).

In the general case of repairs performed with ordinary concrete, it has been shown that in the absence of substrate-induced damage, tensile bond strength increases with the substrate coarseness.  As a general rule, a concrete removal method generating the roughest profile and a sound surface shall be promoted.

# 4. Surface conditioning

4.1. General

This section addresses the final concrete surface and reinforcing steel preparation steps after concrete removal and prior to application of the repair material.

The preparation of the substrate for repair has to be suitable for the required condition of the substrate and the structural and safety status of the structure to be repaired, so that the realistic requirements of the completed repair, as specified, are satisfied.

In all repair types it is important that the new repair adheres well to the substrate concrete. In this respect, it is important that preparation of the concrete surfaces to receive the repair materials be given careful attention as the adhesion developed is as dependent on good surface preparation as on repair material characteristics. Clearly efforts to obtain good adhesion to a weak surface are futile since failure of the concrete surface is likely to occur. Conversely poor adhesion to a sound surface is possible if the surface is inappropriately prepared.

For a successful repair, the following conditions must be satisfied:

- The concrete must be strong and sound;
- The surface should receive the optimum moisture conditioning;
- The surface should be free of dust, laitance or any other foreign materials;
- The surface should have an open pore system;
- The surface temperature should be within suitable limits to permit proper wetting by the repair materials.

Unless cleaning is carried out immediately prior application of repair materials, the cleaned and otherwise prepared concrete and reinforcement surfaces shall be protected from contamination.

4.2. Surface Cleaning

Concrete removal methods may leave the surface to receive the repair material too smooth, too rough, too irregular, and without open pores. In these cases, procedures specifically intended for surface cleaning are necessary.

Microcracking (sometimes referred to as bruising) of the concrete surface is common when impact tools are used to remove concrete. A surface with bruising may weaken the bond between the existing concrete and the repair. In this case, a less aggressive method of surface preparation such as abrasive blasting (sand) or water jetting is necessary.

Concrete can be removed by a variety of methods such as chipping hammers, abrasive blasting, and hydrodemolition. Removal subjects the concrete substrate to a wide range of dynamic loads, and the resulting bruising will depend on the method used and the quality of the concrete. The depth of the bruised layer varies, but is typically on the order of 1/8 in. (3.0 mm). There are no criteria for the degree of bruising that reduces service life.

Pull-off testing of the repair system (surface repair and substrate) can be conducted to determine the bond strength. Excessive bruising may result in low pull-off strength with the failure surface running entirely through the substrate. Bruising is identified conclusively by microscopic examination of the concrete. This examination is typically performed on small samples by a concrete petrographer to identify severity of microcracking. To see bruising, a polished surface needs to be magnified 20 to 100 times, depending on the width of the cracks.

Bruising can be minimized by exercising care in the removal process and by avoiding techniques that experience has shown to cause bruising. Techniques to avoid include the use of scabblers, scarifiers, bush hammers, or large pneumatic hammers, especially those equipped with wide chisel tools.

Bruising can be minimized by using methods such as abrasive sand blasting, shotblasting or water jetting. Where the more damaging methods must be used to increase production or reduce costs, the damage can be mitigated somewhat by abrasive sand blasting, shotblasting or water jetting as a final preparation step for the top 0.10 in. Replacing the commonly used sand in abrasive blasting with alternative materials such as sintered slag, flint silicon carbide, or aluminum oxide can reduce damage.

The use of lightweight pneumatic-chipping hammers equipped with sharp, pointed tools can also reduce the magnitude of bruising.

### 4.3. Cleaning stages

First stage cleaning operations shall be commenced in a repair area after all necessary concrete removal has been completed. The remaining concrete surface must have laitance, partially loosened chips of concrete and the bruised concrete layer, removed by blasting.

If in the Engineer's opinion bruising and/or contaminants, or weathered and carbonated concrete surface, which might interfere with bond, are present on the prepared surface, second stage blasting and cleaning must be performed as directed by the Engineer prior to placement of the repair material.

The old weathered and carbonated concrete surface is usually removed during concrete removal operations and following first stage cleaning. However, long periods of time between these operations and repair material placement may result in new carbonation of the exposed surface.

The issue of the effect of carbonated surface on bond strength is controversial, as conflicting evidence has been reported in the technical documentation over the years. Theoretical analysis, however, lead to the opinion that carbonation does affect the bond strength since it not only densifies the affected concrete, but also changes the pore structure.

The experiments performed in the present study showed that carbonation may have little or no impact on bond strength for an otherwise sound, properly prepared concrete substrate surface. Besides, it confirmed that when some bruising is present within the superficial layer of the concrete surface being treated, carbonation amplifies its detrimental effect upon repair bond. Therefore, a recommendation that the carbonated (existing and/or incipient) surface should be removed is justified.

### 4.4. Cleaning techniques

These techniques consist of removing thin layers of surface concrete using abrasive equipment such as sandblasting, shotblasting, or high-pressure water jetting devices. Abrading techniques remove concrete by propelling an abrasive medium at high velocity against the concrete surface to abrade it as a final step in surface preparation. The process uses common abrasive medium as a primary abrading tool. The process may be executed in one of the following methods.

- Sandblasting – Sand blasting is the most commonly used method of cleaning concrete and reinforcing steel. The process uses common sand, silica sand, metallic sand or slag as the primary abrading agent.
- Shotblasting – Shotblasting equipment cleans concrete by projecting metal shot at the concrete surface at a high velocity. This equipment has the capability to remove finite amounts of sound or unsound concrete. The shot erodes the concrete from the surface. The shot rebounds with the pulverized concrete and is vacuumed into the shotblasting machine. The concrete particulates are separated out and deposited into a holding container to be discarded later while the shot is reused. The shotblasting process is a self-contained operation that is highly efficient and environmentally sound.
- Waterblasting – Water is sprayed at pressures between 5,000 and 15,000 psi (35-105 MPa). This technique is suitable for vertical and horizontal surface cleaning. It is the largely the same as hydrodemolition, except that smaller and hand held equipment is typically used.
- Waterblasting (with abrasive) – Water blasting with abrasives is a cleaning system using a stream of water at high pressure with an abrasive such as, aluminum oxide, or garnet introduced into the stream. This equipment has the capability of removing dirt or other foreign particles as well as concrete laitance thereby exposing the fine aggregate.

### 4.5. Maintenance of the Prepared Surface

After the substrate has been prepared, it should be maintained in a clean condition and protected from damage until the repair/overlay material is placed. Prepared areas should be protected from repair activities in adjacent areas. Mud, debris, cement, dust, etc., when deposited on a prepared surface, may act as a bond breaker if not cleaned up before the repair material is placed.

In hot climates shade should be provided, if practically possible, to keep the substrate cool, thereby reducing rapid hydration or hardening of repair material. In wintertime, necessary steps should be taken to provide sufficient insulation and/or heat to prevent the repair area from being covered with snow, ice, or snowmelt water.

### 4.6. Moisture content

The moisture condition of the substrate will determine the rate of movement of water to or from the repair mortar to substrate concrete due to the moisture imbalance between the two layers. Both the surface moisture condition and the moisture distribution inside the substrate are important.

For bonded overlays it is commonly reported and specified that the substrate surface has to be in *SSD* (saturated surface dry, i.e. pre-wetted, but surface-dry) condition prior to overlay application.  In such a condition, the substrate looks damp but contains no free water on the surface.  The surface absorbed all the moisture possible but does not contribute water to the repair material mixture at the time of placement.  However, no conclusive evidence is provided in the literature suggesting that this actually improves the quality of the bond.  If pre-wetting is done, then it needs to be ensured that the substrate surface has dried out completely before the overlay is applied as any water in the substrate surface pores will prevent mechanical interlock between substrate and overlay.

The results generated in the present study show that optimum moisture saturation levels for repair bond strength of polymer-modified repair mortars would lie somewhere between 55 and 90%.  When acrylic emulsion is used as a bonding layer, the highest saturation levels induce a water film at the interface, which is incompatible with polymeric material and artificially reduces the effectiveness of the adhesion.  This means that the moisture condition of the substrate for such repair materials would actually be dryer than *SSD*.

There are indications in the scientific documentation that substrate moisture conditions dryer than *SSD* would also be preferable for cement-based repair and overlay materials.  Unfortunately, the reliable user-friendly methodology for relatively easy evaluation of the optimum moisture condition of a given concrete substrate, presently is not available.  There clearly is a need for quantitative data in this area to provide more precise guidance.

In the meantime, the saturated surface dry (*SSD*) condition may not always be the best choice, but when experimental evaluation of the optimum moisture condition is not conducted, but it is a "safe" compromise.  Still, it must be realized that the *SSD* condition itself is a very subjective surface moisture quantity.  To what depth must the substrate be actually saturated? This clearly points to the effect of water in the concrete superficial zone and the difficulty of accurately evaluating the saturation level.

The optimum moisture condition will almost inevitably vary from substrate to substrate in otherwise equal conditions because development and performance of the bond depend to some degree on the way the substrate will affect the direction and rate of water movement between phases of the composite repair system.

4.7. <u>Bonding agents</u>

Bonding agents are not recommended for repairs and overlays employing cement-based materials.  Their use cannot compensate for inadequate surface preparation and may act as bond breakers when used inappropriately.

In some cases, the quality of the concrete surface preparation for repair is being neglected based upon the false assumption that poor workmanship can be mitigated by using a bonding agent.  In other cases, bonding agents are being specified and used as a "belt and suspenders" measure.

Bonding agents provide an additional operation and material layer that can cause failure, e.g. a bonding agent that is allowed to cure prior to material placement, becoming a bond breaker. In addition, a bonding grout may have a high water-cement ratio leading to a low strength and risk of adhesive failure within the bonding agent layer itself.

The use of bonding agents should be avoided since it leads to two interfaces and thus to the creation of two potential planes of weakness instead of only one.

# 5. Quality control

The integrity and ultimate performance of repairs and overlays is in large part determined by the quality of the existing concrete surface preparation. It is imperative that care be taken, specifications followed, and surface preparation quality control and related decisions be made by qualified personnel.

Qualified personnel are required for all testing and inspection operations, and shall be performed by the Engineer's representative, and not by the Contractor performing the surface preparation.

### 5.1. Evaluation of roughness

Recently, various techniques were developed for concrete surface profile characterization. The combination of different methods can yield a very accurate description of the "roughness" at various scale levels, depending on the roughness range to be analyzed. For instance, mechanical and laser lab-profilometers allow a more accurate micro-roughness characterization, while the investigated interferometric (optical) method provides a better description of the shape of the profile. Nevertheless, investigations of a range of concrete surface treatments with very precise laser and mechanical profilometers clearly indicated that the surface treatment technique does not have much influence on micro-roughness (high-frequency waves). This indicates that only the waviness parameters actually need to be determined for assessing surface roughness prior to repair.

Among the techniques available today, the best suited method for field assessment appears to be the Concrete Surface Profile developed by ICRI: it is rapid, easy to use and yields reliable information, irrespective of the surface orientation. However, its use is limited right now to surface up to 6 mm in terms of profile height, for which it was actually designed for: it is clearly not suitable for water jetting or jack hammering evaluation.

The advantages of Sand Patch Test method are the speed and its applicability in situ on a surface that has to be protected from wind and rain. The biggest limitations are range of validity (from 0.25 to 5 mm), which usually exclude surface with high holes and peaks and possibility of application only on horizontal surfaces. It gives however good correlations with statistical parameters like $W_a$.

Considering the fact that surface preparation essentially influences waviness, the optical method based on Moiré's pattern, which offers significant advantages in terms of production rate and surface area treatment capability, could in fact be used alone to perform the whole surface roughness characterization. The method directly yields a handful of reliable quantitative data, but the equipment available today is not adapted to daily field applications. Nevertheless, with the

rapid technological development in that field, the availability of suitable optical devices, likely automated, can be foreseen in the near future. This would allow even more rapid and objective assessment.

### 5.2. Evaluation of mechanical strength of the substrate concrete

There are numerous examples of repair and overlay projects where the specified bond strength is greater than that of the concrete substrate. Clearly it is pointless to expect the bond value to be greater (or even equal) than the tensile strength of the substrate concrete.

Many specified testing criteria for bond strength of completed overlays and surface repairs are in fact based on documented recommendations from organizations such as ACI, ICRI, RILEM, etc. and seldom on considerations related to the actual strength of the given concrete to be repaired. In cases when such criteria is not being met based on the tensile pull-off test results of the completed repair or overlay, it is very difficult to establish what went wrong: surface preparation, repair material quality, workmanship, environmental conditions, or a combination of some of these.

The benchmarks for the bond criteria are also often taken from the repair materials data sheets and relate to laboratory tests.

The expectations to meet these benchmarks at the jobsite, often under difficult real life working conditions, can be unrealistic. Therefore, sound engineering judgment is necessary. The specifications for a specific repair project shall not be blindly copied from other specifications or a material manufacturer's data sheet, because it may result in situations where it is not physically possible to achieve compliance with the specified criteria.

Thus more consideration needs to be given in specific project specifications to the requirements of the project. The test criteria shall consider the results of the existing condition evaluation carried out. The direct tension test of existing concrete should be performed as a part of condition evaluation program to allow the specifier to establish the realistic bond strength requirements.

To provide assurance that the surface preparation procedures were performed as specified, the tensile pull-off tests shall be performed on the prepared surface prior to repair application. The pull-off test should be done in accordance with the applicable provisions of the ICRI Guideline No. 210.3-2004 [1].

In cases where the tensile strength of the prepared substrate tested significantly deviates from the tensile strength of the existing concrete documented in the condition evaluation report, the data shall be analyzed by the Engineers, and additional surface treatments may be necessary.

### 5.3. Evaluation of the concrete surface cleanliness

Prior to repair, it is essential to make sure that the concrete surface is free of contaminants, dust, laitance, fragments of concrete, bruised concrete layer, etc. While it may sound simple, there was no unified or systematic approach until recently. In the wake of the newly developed ICRI's Concrete Surface Repair Technician (CSRT) Certification Program, a reference document [2] is

about to be issued by ACI (final editing stage) to provide guidance on how to carry out this evaluation adequately as part of a rigorous QC program for repair works.

### 5.4. Evaluation of the substrate moisture content

Investigations concerning the measurement of water saturation levels and their effect on the adhesion of cement and polymer cement concrete repair systems have shown that the Modified Capillary Suction Test (MCST) gives clearer, more accurate and lower dispersive information than the Initial Surface Absorption Test (ISAT), with a higher correlation for water content measurement (wet and dry weighing). Moreover, there is a very good correlation between the water absorption index and the capillary absorption coefficient determined using the ISAT and MCST tests, respectively.

MCST requires coring a sample of concrete and testing it in the laboratory. ISAT is an attractive option for performing a quantitative test to evaluate the saturation level of a concrete substrate: it is compact, cost-effective and quick. The higher variation and dispersion of results for ISAT may stem from the difficulty of performing the test with a rough concrete surface (after water jetting). Procedures are influenced by the surface quality, but it is difficult to conclude whether this is due to cracking or roughness.

In situ evaluation of the moisture content of concrete remains a challenge and no definite recommendations can be proposed. A simple method [3] is about to be issued by ACI (final editing stage) to support the newly available ICRI's Concrete Surface Repair Technician (CSRT) Certification Program. The method is solely intended to determine whether the surface is dry. Also, on-going research activities at USBR and Laval University notably are devoted to the use of simple electrical R.H. measuring devices to assess the moisture condition of concrete and identify quantitative criteria to be correlated with bond for different repair and overlay materials [4, 5].

### 5.5. Evaluation of adhesion

The pull-off test is a convenient method for evaluating both the mechanical integrity of the concrete surface prior to repair and the repair bond strength [1, 6-9]. A reliable evaluation of these properties can be obtained, provided that a minimum number of tests are performed, with adequate equipment. The potential bias due to testing misalignment, below average naked-eye detection capability, was assessed to reach up to approximately 15 percent. Yet, it can only affect the pull-off strength evaluation on the conservative side.

ICRI Guideline No. 210.3-2013 [1] arguably provides the most comprehensive technical guidance with regards to the specification and evaluation of bond for concrete surface materials. In the document, no recommendation is made on a universally acceptable value. Depending on the project, required bond strength values will typically range from 0.7 MPa (100 psi) to 1.7 (250 psi) and shall not exceed the existing concrete tensile strength. Whenever possible, the implementation of field trials is desirable. According to the ICRI Guideline, for all modes of failure, acceptance of pull-off test results should be based upon the following criteria:

- Where field trials (mock ups) are carried out;
  - Average pull-off strength of the specimens is above the required pull-off strength (90% of average field trial test value);
  - No specimen test below 75% of the average trial specimen test value;
- Where field trials (mock ups) are not carried out;
  - Average pull-off strength of the specimens is above the required pull-off strength;
  - No specimen tests below 75% of the required strength.

With a provision allowing single test values reaching 75% of the specified strength, the QC procedure implicitly takes care of the variability of the test associated with misalignment, which was found to reduce the recorded bond strength up to 15% within a reasonable visual detection limit (± 4°).

It is important to keep in mind that bond strength between a repair or overlay material and a concrete substrate is in fact a subtle property or, more accurately, characteristic to specify. Ultimately, the key requirement of a successful repair is adequate bond between the repair and existing substrate, which will remain intact throughout its service life.  At the present time, practical answers to the problems of bond may depend only on short term bond testing rather than on long term performance.  An initially achieved specified bond strength is only an indication of performance with the specified parameters.  There is no well-defined relationship between initial bond strength and the lasting interfacial bond in a repair system.  Longevity of the bond is influenced by many factors including substrate surface preparation and texture, relative volume changes of repair material, mass transport, service conditions, and quality of the underlying concrete.

# 6. Perspectives

Several fundamental aspects concerning concrete surface preparation prior to repair/overlay and bond strength development were addressed in this study.  Although the results and analysis resulted in a better overall understanding of the problem, a number of questions remain unanswered.  Studying the issues below would be the most effective way to provide more information to gain a better understanding to achieve optimum bond performance (strength and durability) in composite repair and overlay systems.

### 6.1. Substrate roughness

As discussed in sections 2 and 3 of this report, the CSP replicate system proposed by ICRI [10] is a useful tool for evaluating the roughness of a concrete surface prior to the application of a protective system, a coating or a layer of mortar.  However, the actual CSP plates are rather narrow with respect to the spectrum of CSPs obtained with actual surface preparation techniques. In fact, with the existing 10-level scale, its use is limited at present and is thus confined to surface treatment applications where very little material is actually removed and is irrelevant in many concrete repair applications.

It is possible to extend the range of the CSP replicate system in order to cover rougher profiles, typical of those obtained with water jetting and jackhammering. Moreover, it can be improved through a real quantitative approach, by using optical profilometry in combination. The identification of reference curves, similar to those presented by Perez et al. [11] but on a wider scale of surface roughness, will help broaden the range of application of this method to much coarser profiles and allow the identification of quantitative roughness criteria.

### 6.2. Moisture conditioning of the concrete prior to repair

Despite the work accomplished in this project, some fundamental issues remain unresolved with regard to moisture conditioning of the concrete substrate prior to repair. In daily repair practice, inevitably loose specifications and the absence of measuring tools actually result in a wide range of moisture conditions.

In order to develop proper specifications, it is necessary to gain a better understanding of the transport mechanisms between repair materials and concrete substrates and the influence of the moisture state of the substrate upon bond development.

Both the issuing and implementation of such specifications will, in turn, require the development of a test method to evaluate quantitatively the actual moisture condition of concrete in the laboratory, as well as in the field. The envisioned method would allow the determination of optimum conditions for a given concrete substrate, as well as quality control testing. The method needs to be simple and applicable to both laboratory and in situ conditions. In that regard, further investigation should be directed towards measurement techniques already available, such as electrical impedance devices (flooring industry) or superficially encased relative humidity probes.

### 6.3. Long-term bond

It must be emphasized that this study, as well as other reported work on the subject, is primarily dealing with "short-term" bond strength issues, not with the mechanisms and issues related to long-term bond behavior and durability. The short-term bond strength typically specified and evaluated can be used as an indication of the quality of workmanship (i.e., concrete surface preparation for repair, material selection, application, and curing). Long-term bond strength, however, is usually influenced by various other factors, among them environmental, loading, and fatigue conditions.

Therefore, it is desirable to pursue research efforts on those factors affecting long-term bond strength in concrete repair/overlay systems, notably the surface preparation parameters and characteristics.

### 6.4. Compatibility issues in repair/overlay systems

When compatibility issues are properly addressed in repair systems, durability of the bond is achieved, as it ensures a lasting coexistence of the repair material and substrate concrete.

Incompatibility issues cause premature debonding and repair failures. Unfortunately, at the present time, much confusion, misconceptions, and misleading guidance exist concerning compatibility of repair materials and the substrate concrete. These issues negatively affect the design, specification, implementation, and, as a result, service life of concrete repairs and overlays.

Development of reliable guidelines addressing compatibility issues – with special emphasis on the factors related to dimensional compatibility issues – is needed for the repair industry to evolve as an engineering discipline.

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