Development of correlation between compressive strength and ultrasonic pulse velocity in roller-compacted concrete for dams

BY

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Declaration

I, Thea Gouws, declare that this thesis represents my own unaided work, and that the thesis has not previously been submitted for academic examination towards any qualification. Furthermore, it presents my own opinions and not necessarily those of the Cape Peninsula University of Technology.

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Signed this ______14______ day of __________December__________ 2023

Abstract

Concrete is a fundamental building material that impacts construction productivity, time, cost, and structural durability. Dam construction projects, mainly using concrete, require careful consideration of time and cost factors when selecting an economically viable concrete type. Roller-compacted concrete (RCC) used to construct dams has become a modern choice, particularly for gravity dams, but the cost-effectiveness depends on various factors. Concrete testing dates back to the 19th century and mainly focused on assessing concrete quality and longevity through compressive strength tests. Compressive strength tests are crucial for evaluating concrete properties using destructive and non-destructive methods. The U.S. Army Corps of Engineers (USACE, 1998) emphasizes laboratory investigation to characterize RCC properties, particularly in the context of the Elk Creek Dam. However, a research gap exists regarding the correlation between ultrasonic pulse velocity (UPV) and compressive strength in RCC dams, underlining the need for comprehensive testing methodologies to assist engineers in non-destructive testing of RCC strength.

The study aimed to establish relationships between various compressive strengths of High-cementitious Roller-Compacted Concrete (HCRCC) used in dam construction by implementing both destructive (DT) and non-destructive (NDT) testing on laboratory HCRCC. After that, to establish a correlation between the DT and NDT testing results and validate the identified data through filed data.

This research utilized two different High-cementitious Roller-Compacted Concrete (HCRCC) mix designs, varying in the total quantity of cementitious materials used. The two designs, 15/38-365 and 20/38-90 met the design requirements for Roller-Compacted Concrete (RCC). Applying a variable by replacing fly ash with cement for the total cementitious content resulted in eight distinct HCRCC mixtures. A total of 72 specimens were cured for 7, 14, and 28 days. The research conducted non-destructive testing (NDT) methods, conducting Ultrasonic Pulse Velocity (UPV) and Rebound Hammer (RH) tests for each specimen. Subsequently, following NDT, each specimen underwent a compressive strength test using the DT method. Field data collection occurred at the De Hoop Dam and Springs Grove Dam, chosen for their construction using Inverted Roller Compacted Concrete (IVRCC), ensuring an accurate reflection of measurements specific to RCC dam structures.

A multiple regression model with a linear correlation was developed with the laboratory dataset without outliers, resulting in a strong positive model with an excellent R2 of 0,93. Expressing that 94% of the change in compressive strength was a function of non-destructive testing methods evaluated, with an error of 6% over the range evaluated. Further investigations are required for different RCC dam design mixes and the non-destructive testing thereof.

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Terms and Concepts (Technical Terms)

Term / Constants	Explanation / Definition
Abrasion	Process of wearing a way of a surface by friction. In the context of this thesis, abrasion means the wearing away of the concrete surface due to the movement of water over specimens.
ASTM	American Society for Testing and Materials.
BS EN	The European Standard (EN) has the status of a British Standard.
C & CI	Cement and Concrete Institute.
Crusher Run Aggregate	Is a blend between fine and course aggregate to the maximum size of 5 mm.
Coarse Aggregate	Coarse aggregate is all aggregate coarser than 4.75 mm.
Concrete Grade	Concrete grade defines the specific required strength of the concrete and the maximum size of aggregate to be used, i.e., Grade 20/38. The first number (20) indicated the strength of concrete of 20 MPa and the second number (38) indicated the nominal maximum size of the coarse
Curing	Process of maintaining moisture content and temperature of concrete to permit complete hydration of cementitious products after the concrete has been placed and compacted. In this context, curing means the curing of concrete specimens, after compaction.
CVC	Conventional Vibrated Concrete. CVC also referred to as skin concrete.
DT	Destructive Testing. Destructive testing is the type of method to be used for conducting tests of concrete, which is destructive of nature (i.e., pull-out test, compressive strength testing by crushing of sample).
DWS	Department of Water and Sanitation.
DWS:CS	Department of Water and Sanitation: Construction South.
FA	Fly-ash (FA), flue ash, coal ash, or pulverised fuel ash is the powder formed from the combustion coal in the coal-fired electricity power stations.
Facing concrete	Facing or skin concrete an impermeable layer of concrete placed against the formwork, or other surface forming the external face of RCC.
Fine aggregate	Fine aggregate is all aggregate passing 4,75 mm sieve.
GGBS	Ground Granulated Blast Furnace Slag.
IV-RCC	Immersion-Vibrated Roller Compacted Concrete or also called IVRCC. For this type of RCC no grout enrichment is required. IVRCC is placed with consistency can be compacted by immersion vibrators.
Mix design	This is the specific quantities of materials to be used for the mixing of concrete, calculated according to the project specified specifications. The type of materials available influence the mix design.
Mortar	The absolute volume of paste and fine aggregate.

MPa	Megapascal is the measure of the compressive strength of concrete at a certain age required, that is calculated as the force over the area applied.
NA	Not applicable (means when in the given context there is not value.)
NDT	Non-destructive testing. This is the type of test method used to test concrete characteristics without damage to the concrete structure, with limited or non-damage to the concrete surface as well (i.e., visual inspection, ultrasonic pulse velocity, impact echo, rebound hammer etc.).
OPC	Ordinary Portland Cement.
Paste	A mixture of cementitious material and water that binds the aggregates together to make concrete. It is also defined for RCC as the absolute volume of cementitious materials, liquid admixtures, and free water.
Paste/Mortar Ratio (P/M)	For RCC paste mortar ratio (P/M) is defined as the absolute volume of paste over the mortar.
Pozzolan	This is a fine-grained material which possesses cementitious in nature, and can be used as cement extenders, but it requires calcium hydroxide to react. GGBS and FA are some Pozzolan materials. Pozzolan is also known as cementitious materials.
RCC	Roller-Compacted Concrete: Concrete with a low slump, transported by high-capacity equipment, spread out and compacted with a smooth drum roller in layers. Also, refer to as rollcrete and mostly written as, roller compacted concrete or roller-compacted concrete. Where reference is made in this document to RCC, it is only RCC used for the construction of a dam wall.
RH	Rebound Hammer (as described in BS EN 12504-2 and BS EN 13791). A handheld device that is used to measure the hardness of concrete surfaces or penetration resistances. Also known as a Rebound Hammer or Schmidt Hammer or a Swiss hammer or concrete hammer test. The readings taken with RH is called rebound index number.
RN	Rebound Number. The rebound hammer produces a measure known as the rebound number, which is correlated to the surface hardness of concrete.
SABS	South African Bureau Standards.
SANS	South African National Standards.
Sand/Aggregate ratio (S/A)	The sand aggregate ratio is the volume of fine aggregate over the total volume of aggregate, also referred as fine aggregate – coarse aggregate ratio.

Specified Strength	The design RCC Strength, represents the compressive crushing strength on a 150 mm specimen after curing in a bath of water at a constant temperature of between 22° – 25° for a specific number of days, since mixing. Mix 15/38-365 in this document was design according to Spring Grove dam specifications to reach strength of 15 MPa at 365 days age. Mix 20/38-90 in this document was design according to De Hoop dam specifications to reach strength of 20 MPa at 90 days age.
UPV	Ultrasonic Pulse Velocity (as described in BS EN 12504-4) is the test to check the quality of concrete and rocks, by measuring the velocity of an ultrasonic pulse passing through the concrete structure or rock.
USACE	United States Army Corps of Engineers also known as U.S. Army Corps of Engineers.
Vebe	The test method was performed to measure the workability and consistency of concrete. It is also known as VeBe, Vebe consistometer, Vee-Bee, Vee Bee, Vee Bee consistometer or loaded Vebe consistency. For RCC loaded Vebe needs to be done in accordance with ASTM C1170M.
w/c ratio	The water-cement (w/c) ratio is also known as the water-binder ratio, referring to the binder as the total contingent of cementitious materials.

Chapter 1 Introduction

The comprehensive evaluation and examination of concrete properties is a fundamental requisite within the construction industry around all concrete alternatives. Among these, Roller-Compacted Concrete (RCC) is a specialised category predominantly employed in dam and road construction applications. In structural assessment, destructive testing (DT) methods are conventionally employed to appraise extant concrete structures.

This research initiative is designed to scrutinise the primary relationship between destructive tests, mainly focused on compressive strength, and non-destructive tests, encompassing ultrasonic pulse velocity (UPV) and rebound hammer (RH) assessments, made-to-order clearly for RCC formulations destined for dam construction. This introductory chapter provides a comprehensive background to the research problem, emphasising the motivation behind the necessity for further examination into non-destructive testing (NDT) methodologies. Furthermore, this chapter shall investigate the core research query, delineate the study's main goals, and lay out its specific objectives.

1.1 Background and motivation

Concrete is a typical building material highly favoured by the construction industry. The utilisation of concrete in construction projects deeply impacts productivity, time, cost, and the overall durability of structures. Thus, it is of principal importance to consider the numerous factors that can influence concrete, ensuring the deployment of the most efficient concrete mix designs to construct enduring structures.

For dam construction projects with concrete, it is imperative to consider both time and cost factors. Therefore, selecting an economically viable concrete type, considering the placement rate and the materials that contribute to a durable structure, is of immense benefit to any construction undertaking (Shaw & Perrie, 2021: 849). Shaw (2017) states that the first concrete dam was constructed between 2750 and 3000 BC. Over the resulting times, the technology for dam construction has undergone significant advancements. Technological progress has not only enhanced concrete testing but has also expanded the applications of dam construction.

Moreover, the growing global population's need for water access and storage has driven the rapid evolution of dam infrastructure. In this context, Roller-Compacted Concrete (RCC) dam construction stands out as the most modern and preferred method, particularly for gravity dams (Shaw, 2017). It is a prevalent misconception that RCC is always the most cost-effective choice for concrete dam construction (Shaw, 2017). In truth, the cost of dam construction centres on a multitude of factors, including the complexity of design, the site's geographical location, material availability, and the stipulated timeframe for construction.

The history of concrete testing can be traced back to the 19th century, specifically between 1835 and 1850, when the first tensile and compressive strength tests on concrete were conducted (Ali & Kakpure, 2019). The necessity for concrete testing emerged to assess the quality and longevity of concrete employed in construction industries.

The compressive strength test is a key method for assessing concrete strength and quality. This method, categorized as destructive testing (DT), involves measuring the compressive strength of concrete specimens. Additionally, non-destructive testing (NDT) techniques, such as ultrasonic pulse velocity (UPV) and the rebound hammer (RH), are employed to evaluate the mechanical properties of hardened concrete.

It's worth noting that in the past, RCC used in dam construction was often associated with low-quality and low-strength mass concrete. However, in recent years, RCC for dams has experienced a transformation, enabling the production of a range of RCC concrete qualities (Shaw & Perrie, 2021: 849).

The U.S. Army Corps of Engineers (USACE, 1998) emphasises the importance of laboratory investigation in characterising the strength and constitutive property behaviour of Roller-Compacted Concrete, as exemplified by the Elk Creek Dam in Trail, Oregon. This investigation involved conducting various mechanical property tests and an extensive non-destructive ultrasonic pulse velocity (NDT UPV) tests on large-diameter core samples. The objective was to understand and characterise the strength properties of the interfaces between the RCC lifts. However, the study does not show any correlations between DT and NDT.

Notably, there is a prominent gap in research regarding the correlation between UPV and compressive strength in the context of RCC dams. As such, conducting compressive strength tests on diverse RCC dam concrete mix designs are imperative, in conjunction with ultrasonic pulse velocity and rebound hammer tests. This multi-layered approach is essential to understand these test results and establish correlations comprehensively. Such correlations will prove helpful for engineers when investigating the RCC strength of dams using NDT testing methodologies.

1.2 Research problem

The absence of an established correlation between the compressive strength assessments, ultrasonic pulse velocity measurements, and rebound hammer tests in Roller-Compacted Concrete (RCC) remains a critical research gap.

1.3 Research question

What is the correlation between UPV and compressive strength to enable NDT tests on RCC dam walls, and how can a relationship be establish between RH, UPV and compressive strength?

1.4 Objectives

The study pursued the following objectives to establish the relationship between different compressive strengths of Roller-Compacted Concrete (RCC), ultrasonic pulse velocities, and rebound hammer test results for dam construction:

- Prepare High-cementitious Roller-Compacted Concrete (HCRCC) mix designs.
- Conducting destructive (DT) and non-destructive (NDT) testing on HCRCC.
- Establishing a correlation between the outcomes of DT and NDT testing.
- Validating the identified correlation with field data.

1.5 Significance

The significance of this research lies in its use of non-destructive testing (NDT) methods for assessing Roller-Compacted Concrete (RCC) in the context of dam construction. NDT techniques are not commonly employed for evaluating the quality of concrete in dam structures. This study offers valuable insights into the feasibility of adopting NDT methods as a viable alternative to traditional destructive testing (DT) approaches. It establishes the feasibility of NDT for determining the compressive strength of RCC in dam construction, thus positioning NDT as a more cost-effective and environmentally sustainable choice for future concrete quality assessment.

1.6 Delineation

The research primarily focused on the assessment of the compressive strength of Roller-Compacted Concrete (RCC) using both destructive (DT) and non-destructive (NDT) test methods in laboratory experiments. Laboratory specimens were precisely prepared to facilitate the conduction of compressive strength tests, rebound hammer tests, and ultrasonic pulse velocity tests.

Subsequently, NDT field tests were executed, specifically around rebound hammer and ultrasonic pulse velocity tests conducted on the non-overspill wall sections of *De Hoop Dam* and *Spring Grove Dam* as part of this research study.

It's essential to note that the laboratory experimental phase solely aimed to establish the correlation between ultrasonic pulse velocity (UPV) and compressive strength. Notably, this research encountered limitations concerning the timeline, preventing test data acquisition beyond the 28-day mark. The omission of tests conducted after the 28-day interval is acknowledged and is suggested as a recommendation for further exploration in Chapter 6. Furthermore, the mortar paste ratio was not determined in this study.

1.7 Organisation of thesis

This thesis is structured into six chapters, each serving a distinct purpose to address the study's objectives comprehensively. The following is an outline of the organization of this thesis:

Chapter 1 – Introduction
The first chapter introduces the roller compact concrete dam testing necessity. It provides insight into the research origins, outlines the main research objectives, and explains the scope and boundaries.

Chapter 2 – Literature Review
In this chapter, an extensive review of pertinent literature is conducted, focusing on Roller-Compacted Concrete (RCC). It delves into the materials used in RCC and expounds upon the essential properties of this concrete variant. Additionally, this chapter explores the existing body of knowledge regarding both destructive and non-destructive testing methodologies employed to assess the mechanical properties of RCC, with a particular emphasis on compressive strength. Furthermore, it identifies gaps in the existing literature, paving the way for the research's unique contributions.

Chapter 3 – Research Methodology
This chapter describes each testing procedure methodically, with detailed specifications and methodologies, that was applied for laboratory and field tests.

Chapter 4 – Experimental Results of RCC

This chapter presentation of research findings. It summarises the outcomes of laboratory experiments and field testing. The results are methodically structured and articulated to align with the project's research question, aims, and objectives.

Chapter 5 – Multiple Regression on DT and NDT results

In this chapter, the research results are subject to an in-depth analysis. The data and findings are carefully scrutinised, compared, and contextualised to comprehensively summarise the study's discoveries.

Chapter 6 – Conclusion and Recommendations

This chapter provides a summary of the research project. It addresses the research question and summarises the findings and insights from the literature review and research. Conclusions are derived from the experimental and field results, findings, and literature review. Recommendations for possible future research outlined through the study are identified.

Chapter 2 Literature review

This chapter pertains to the comprehensive literature review conducted within the scope of this particular research project. The review examines the existing literature on Roller Compacted Concrete (RCC), the requisite materials, the fresh and hardened properties, and mix designs. Furthermore, the chapter elaborates on the findings extracted from the literature regarding test methods applied to assess the hardened properties of concrete, with a specific focus on non-destructive testing techniques.

2.1 Introduction

Concrete dam structures may experience various issues and potential failures, restricting from design inadequacies and construction practices. Ensuring these dam structures' safety and reliability prevents potential disasters. Consequently, to mitigate the risks associated with dam structure failure, it is imperative to prioritise the careful construction, operation, and ongoing maintenance of concrete dams (Bukenya et al., 2014).

The overall condition of a concrete dam can be assessed by evaluating the quality and status of the concrete itself. While visual inspections and monitoring are commonly employed techniques for this purpose, their application can be notably challenging when dealing with larger concrete dam structures (Bukenya et al., 2014). Thus, visual inspection primarily serves as an initial step in identifying areas that permit further scrutiny through testing methodologies. These assessments may include destructive testing methods, such as core drilling, and non-destructive evaluations, including techniques like seismic tomography and impact echo. The objective is to gain a comprehensive understanding of concrete conditions and to monitor these conditions over time.

Non-destructive evaluation techniques applied to concrete dam structures offer valuable insights into the concrete materials' quality, the concrete's mechanical properties, and the concrete's ongoing deterioration (FPrimeC, 2017).

2.2 Roller-compacted concrete

A diverse selection of concrete types finds application in modern construction practices on all sides of conventional and non-conventional alternatives. Conventional concrete, characterised by a slump range falling between 30-175 mm and a typical density, is considered the norm. In contrast, non-conventional concrete materials and techniques offer the potential to expedite the construction process and enhance structural durability. The concrete type selection centres on factors such as the availability of materials and equipment, the chosen application method, and the fundamental design prerequisites. Roller compacted concrete (RCC), as categorized by Evans, 2021: 519, stands out as a representative of the non-conventional concrete category.

RCC, in the core, is concrete compacted, utilising rollers. It is a rigid, zero-slump concrete conveyed, positioned, and compacted using earth-fill construction machinery, as per the United States Army Corps of Engineers (USACE) in 2000. RCC primarily comes into play when extensive placement areas require coverage and there are minimal restrictions on the concrete placement rate. Its composition parallels conventional concrete, comprising fine and coarse aggregates, cement, pozzolan, admixtures, and water. Nevertheless, the ratios of these constituents differ from those used in conventional concrete. In contrast to traditional concrete, RCC lacks slump or is typically engineered for a specified low slump allowance tailored to a particular project's demands, contingent on the intended application and use case (Adaska, 2006; Avallone, Jahn & Marazzini, 2019).

RCC offers a straightforward, swift, and cost-effective method for concrete placement, featuring high-speed construction rates, reduced labour expenses, time efficiency, and cost savings on materials, as developed (Avallone, Jahn & Marazzini, 2019).

RCC is mainly close to two distinct construction contexts: dam construction and pavement construction. While similarities exist between RCC usage for dams and pavements, disparities also manifest. RCC for pavement construction boasts a higher cement content and employs smaller maximum-size aggregates than its corresponding item in dam construction (Rahmani, Sharbatdar & Beygi, 2020). Thin layers of RCC are laid during pavement construction, with the thickness corresponding to the designed road top layer, thus necessitating smaller maximum-size aggregates. Roads, as they stand the weight of heavy traffic loads, demand a larger quantity of cementitious materials. On the other hand, RCC deployed in dam construction fills substantial material volumes to establish mass in the dam wall. A concise summary of RCC formulations and treatments for these distinct purposes, pavement and dam construction, can be found in Table 2.1.

Table 2.1 Main differences between RCC for pavement and dam for construction purposes

	RCC Pavement	**RCC Dams**
	Formulation	
Cementitious materials	100-450kg/m³	70 -250kg/m³
Water	84-154	96-122
Maximum size aggregate	19mm	37.5 – 40 mm
w/c ratio	0.3-0.8	0.57-1.51
Concrete strength (MPa)	30-40	4-40
	Treatment	
Placed and compacted in:	150 - 250mm layers	300mm layers

2.2.1 History of roller-compacted concrete dams

RCC has been used to construct dams worldwide since the early 1980s. At the end of 2019, more than 800 large dams had been constructed using RCC worldwide (Shaw & Perrie, 2021:849).

De Mistkraal Dam and Zaaihoek Dam were built using RCC in South Africa in 1984 and 1985, respectively. After that, 27 other dams were built in South Africa using RCC (Geringer, 2008; Malcolm Dunstan & Associates, 2022). Figure 2.1 shows the timeline from the start of the first RCC dam construction in South Africa to 2011. To date, South Africa is currently the country which has constructed the most dams using RCC. (Malcolm Dunstan and Associates, 2022).

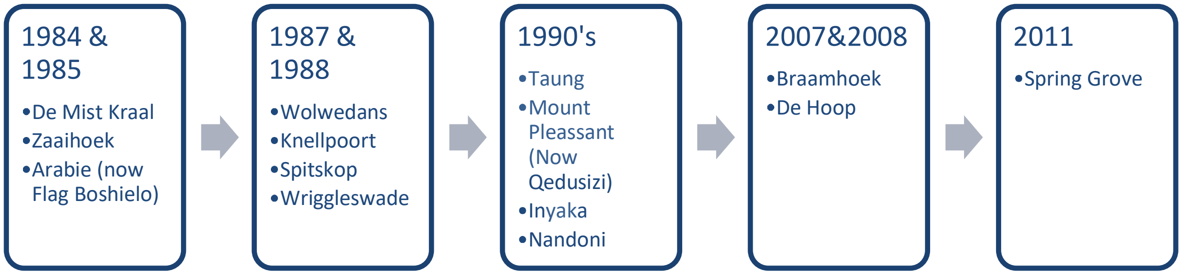

Figure 2.1 Timeline for the start of some RCC dams in South Africa (Malcolm Dunstan and Associates, 2022)

The initial application of Roller Compacted Concrete (RCC) in dam construction involved a lower-quality, lower-strength bulk material. However, advancements have enabled the production of relatively high-strength, high-quality RCC over the years. Experience and testing have demonstrated that the properties of hardened RCC are comparable to those of traditional mass concrete (USACE, 2000). The primary disparities lie in the fresh concrete state, where RCC features reduced water and cement content, resulting in lower binder and higher aggregate content. RCC is primarily substituted for conventional concrete in the construction of gravity dams due to its cost-effective capability to fill substantial volumes, meeting the required mass for the dam wall (Shaw & Perrie, 2021:849).

In the early implementation of so-called RCC dams in South Africa, such as during the construction of Wolwedans Dam, RCC was employed as a filling material within the dam's core. Initially, the permeability of conventional RCC was considered inadequate. Consequently, a layer of conventional skin concrete was applied to the exterior (upstream and downstream faces) of the RCC to ensure compliance with dam design permeability standards. This skin concrete layer was constructed using conventional concrete. Figure 2.2 illustrates the placement of the skin concrete on the external surfaces of the internal RCC dam wall at Wolwedans Dam in the Western Cape, South Africa.

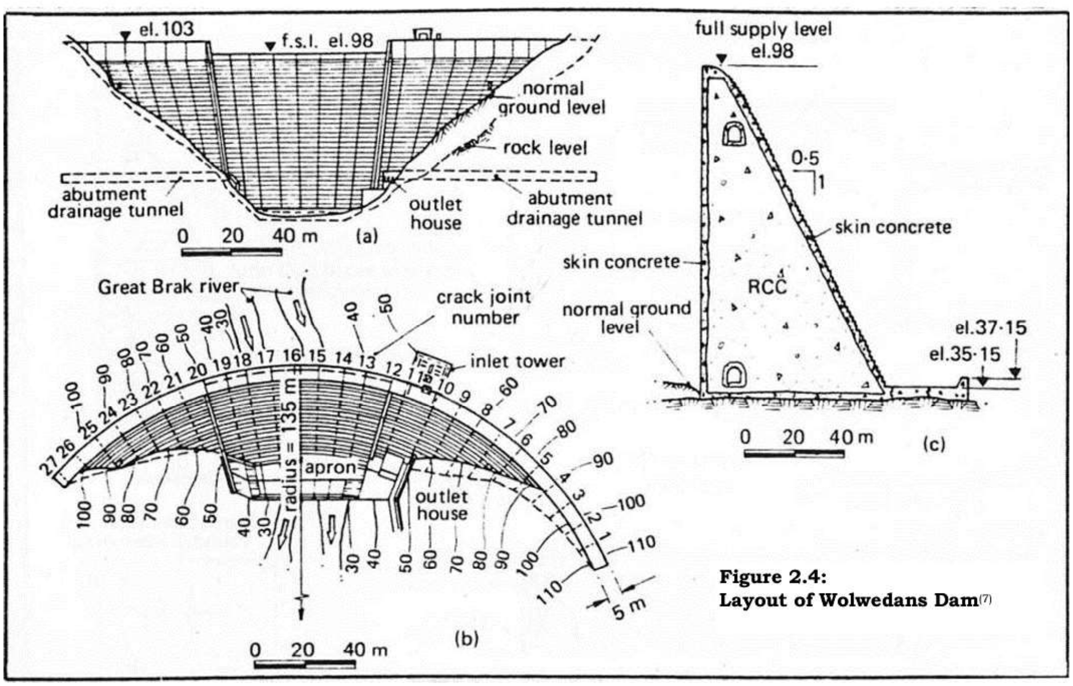

Figure 2.2 Layout of Wolwedans dam wall (Shaw, 2010.)

However, research indicates that the RCC mixture design for dams in South Africa underwent a notable transformation in the early 2010s, particularly in constructing the De Hoop, Spring Grove, and Neckartal dams in Namibia. In these instances, a "high-pasted RCC" with sufficient density and workability was employed, obviating the need for skin concrete application. This innovation in the wet paste form of RCC is commonly referred to as Immersion-Vibrated Roller Compacted Concrete (IVRCC), as introduced by Van Niekerk (Van Niekerk, 2012). Figure 2.3, presented below, captures an excerpt from an article discussing IVRCC and the research conducted in the context of the De Hoop Dam.

In order to keep South Africa at the forefront of Roller Compacted Concrete dam construction, the Contractor utilized international expertise in order to assist with concrete mix development. This resulted in the Department becoming a world leader in RCC (*Roller Compacted Concrete*) Construction Technology in effectively developing IV-RCC (*Immersion-Vibrated Roller Compacted Concrete*).

IVRCC is a high-paste concrete mix that can be placed by means of large equipment such as Bulldozers and Vibratory Rollers (*10T*) but can also be vibrated against the formwork by means of hand-held Poker Vibrators. This eliminates the need for "skin concrete".

The development of IVRCC is truly recognized in world of RCC as a ground breaking achievement and will change RCC dam construction as we know it.

Figure 2.3 IVRCC explained in De Hoop Dam article (Van Niekerk, 2012)

Extensive research was conducted during the De Hoop dam project to attain a uniform and sufficiently dense RCC that could exhibit impermeability, shown Figure 2.4. The ultimate goal was to dispense with the necessity for skin concrete on the upstream and downstream faces. This groundbreaking approach eliminated the need for applying skin concrete to these surfaces, leading to substantial time savings in the construction process. Notably, this wet paste RCC mixture design marked the maiden application of its kind in any dam construction within South Africa (Van Niekerk, 2012).

IV-RCC is deliberately deposited in layers measuring 300 to 400 mm in thickness. These layers are deposited, spread, and compacted using earth-moving equipment, roller vibratory machines, or conventional concrete immersion poker vibrators readily available at construction sites. This compaction method is preferred when the formwork contains the concrete during the pouring process. Mobile vibratory rollers are deployed to compact the remaining concrete. Notably, during construction, new concrete lifts can be placed immediately after the preceding lift has been compacted, yielding significant time-saving advantages in the dam construction process. RCC dam construction stands as the most contemporary approach to concrete dam construction (Van Niekerk, 2012).

Figure 2.4 De Hoop Dam is a RCC gravity dam on the Steelpoort River, Limpopo (DWS, 2020)

Modern RCC for dam construction is divided into three categories of RCC, namely: low cementitious RCC (LCRCC), medium cementitious RCC (MCRCC), and high cementitious RCC (HCRCC). The classification is done on the total cementitious material content in kilograms per cubic meter (kg/m^3). IV-RCC RCC can be placed in the high-cementitious RCC class. As per the below Table 2.2, the three categories are defined.

Table 2.2 Classification of RCC dams' mixture (Shaw & Perrie, 2021: 852)

Classification	**Total Cementitious material content (kg/m^3)**
Low-cementitious RCC (LCRCC)	< 100
Medium-cementitious RCC (MCRCC)	>100 and < 150
High-cementitious RCC (HCRCC)	< 150

Previously, high-cementitious RCC was termed high-paste RCC, medium-cementitious RCC was termed medium past RCC and low-cementitious RCC was termed lean paste RCC. As per the identification of the name, the low-cementitious RCC has low amounts of cementitious materials per mix, which causes the mixes to be drier in consistency, resulting in a less workable material. The advantage of the leaner paste RCC is its low internal temperature during hydration and low elastic modulus. High-cementitious concrete has high amounts of cementitious materials and performs just as well as conventional concrete in a dam (Shaw & Perrie, 2021).

There are various RCC guidelines which are used all over the world for RCC dam construction. Some of these documents provide guidelines on the proportions and ratios of materials to be used to design an RCC mixture. As per Shaw & Perrie (2021), typical mixture proportions guidelines are shown in Table 2.3.

Table 2.3 Mixture proportions for the three different RCC Classifications (Shaw & Perrie, 2021)

Materials	LCRCC	MCRCC	HCRCC
Portland cement	72	80	87
Supplementary cementitious materials	9	37	108
total cementitious materials	<100 kg/m^3	100-150 kg/m^3	> 150 kg/m^3
Water	122	116	111
w/c ratio	1,51	0,99	0,57
Sand : aggregate ration	can vary between 0.3 - 0.47		
Vebe time	>30 sec	>20 sec	10-20 sec
Total paste	210-240 l/m^3		
Paste : mortar range	0,35-0,43		0,37-0,45
Volume	range between 85 l/m^3 - 125 l/m^3		

The cementitious content plays a crucial role in shaping the concrete mix's paste. The concrete paste content encompasses the cementitious content and all fine materials passing through a 75-micron sieve. Furthermore, the water-cement and aggregate ratios vary for different categories of RCC, as outlined in Shaw and Perrie (2021:852).

As expressed by Jansen in 1988, RCC mix design must merge incompatible materials, ensuring an adequate paste volume to promote cohesion between lifts and minimising paste volume to mitigate the risk of thermal cracking (Jansen, 1988).

Like all construction materials and methods, RCC possesses its advantages and disadvantages. Among the benefits of RCC is the reduced cement content, leading to diminished heat of hydration. RCC influences a relatively broad spectrum of aggregates that may not meet the typical standards for conventional concrete. These factors collectively contribute to economic advantages and enable a high-speed implementation of RCC (Rahmani, Sharbatdar & Beygi, 2020). Nevertheless, there are drawbacks to using RCC in dam construction, including the potential for water seepage, challenges in achieving smooth finishes, and the requirement for skilled personnel well-versed in RCC techniques.

2.3 Mixture design and proportions

Roller Compacted Concrete (RCC), while sharing common constituents with conventional concrete, differentiates itself through distinct mixture proportions (Calis & Yildizel, 2019). Its final performance hinges on factors such as aggregate quality, shape, and the overall paste content (Shaw, 2002).

RCC's mixture design is characterized by its adaptability, reflecting diverse methodologies driven by regional conditions, specific requirements, and personal preferences among practitioners (Shaw, 2002). Four principal RCC design methods are outlined by Calis and Yildizel (2019):

- US Army Corps of Engineers' (USACE) Practice: This method presents a systematic, step-by-step procedure for RCC mixture design, employing parameters like water-cement ratio, strength, aggregate size, and water volume to align with desired strength levels.
- High Paste Method: Comprising three key steps, it initiates the selection of aggregate gradation based on specific compaction energy. The second step calculates the paste volume to fill the gaps

between aggregates for the required workability. Finally, it determines the water-cement ratio and pozzolanic content to achieve the desired strength.

- RCC Dam Method: Two fundamental principles guide this approach. Firstly, it emphasizes minimizing cement content while achieving the desired strength, including fly ash, to reduce hydration temperature. Secondly, it underscores a higher sand-to-aggregate ratio than traditional mass concrete to ensure adequate compaction and prevent segregation.
- Maximum Density Method: This method begins by selecting the aggregate form 90% of the mix volume (Calis & Yildizel, 2019). Subsequently, the remaining mix design is calculated.

In the context of South Africa, the Department of Water Affairs and Forestry relies on the principles outlined by the US Army Corps of Engineers for RCC mixture design. This approach considers strength and durability requirements specific to the structure, material availability, transportation logistics, and construction equipment, all of which are pivotal factors in the construction of RCC dams in the region (USACE, 2000). By leveraging these methodologies, RCC design can be made-to-order to meet various construction projects' unique demands and conditions.

The following basic considerations to keep in mind, according to USACE (2000), when deciding and designing a RCC mixture is:

- Durability: durability is influenced by the cementitious material content, aggregate quality, and percent compaction.
- Strength: the structure's design determines the strength of conventional concrete.
- Workability: This is determined by the ease of placing RCC and compacting successfully without harmful segregation.
- Heat generation: this is associated with low water and cement content. The maximum amount of pozzolan material to be used gives the strength, durability, and economic construction requirements.
- Aggregate: the largest practical size aggregate should be used. However, with that said, the larger the size of aggregate used, the more likely it is for segregation to occur during construction. The type of aggregate, the quality and the quantity of aggregate used influence the mixture design.

In order to achieve an RCC mixture that aligns with project specifications and concrete performance criteria, it is imperative to conduct a mixed design process. The US Army Corps of Engineers (USACE) offers a comprehensive, standardized procedure for developing RCC mixture designs, as outlined in the Roller-Compacted Concrete Engineer Manual (USACE, 2000). Although the USACE method is an industry-standard across dam construction, minor adjustments to mixture proportions can be accommodated during field applications. These adjustments should be grounded in visual assessments and Vibe test results, ensuring that any modifications maintain the desired performance characteristics while adhering to the project's specific requirements (USACE, 2000).

2.4 Constituent of roller-compacted concrete

Constraints of the type of concrete to be mixed, the use of the concrete and the environmental location of the structure to be built have all influenced the final (optimum) concrete mix design to be used during construction. Each material has its quality strength and determines the life and usage of concrete (Ofuyatan, et al., 2021).

The mix design for RCC generally consists of a high amount of coarse aggregate (normally large-size aggregates), a minimal amount of fine aggregate (sand), Portland cement, fly ash as pozzolan material, water, and admixture to hydrate the mix. For the purposes of this research project, the following materials were considered and used for this specific concrete mix design.

2.4.1 Cementitious materials

Portland cement serves as the primary cementitious component in concrete. Nevertheless, most concrete blends incorporate supplementary cementitious materials (SCM), called cement blends. SCM constitutes a portion of the cementitious constituents employed in concrete. These materials encompass fly ash (or pulverized fuel ash), ground granulated blast furnace slag, silica fume, calcined clays, and natural pozzolans (Juenger & Siddique, 2015).

In the context of Roller-Compacted Concrete (RCC), SCM plays a pivotal role, as various combinations of these materials are employed to enhance chemical resistance against sulphate attack, mitigate potential alkali reactivity, and bolster resistance to abrasion on aggregates. The selection of SCM employs significant influence over the development of RCC's strength (Adaska, 2006).

The total cementitious content profoundly impacts the workability of the concrete. Augmenting the amount of cementitious materials can concurrently bolster compressive strength. Furthermore, the durability of the concrete is contingent on the overall cementitious content (The Concrete Institute, 2018). Nonetheless, an excessive proportion of cementitious materials in a mixed design can generate heightened hydration heat, shrinkage, and the propensity for cracking in the concrete. Therefore, carefully using cementitious materials is imperative to avert adverse effects on the mix design.

Ordinary Portland Cement (OPC)

Cement functions as the binding agent, undergoing a chemical reaction with water to produce a cementitious gel that effectively binds the constituent materials within the concrete mixture. As the concrete cures, this cementitious gel solidifies. The specific characteristics of the cement employed in this research project can be found in Appendix B.

The market offers a range of cement types, each tailored to meet specific project demands. The categorization of cement types is visually presented in Figure 2.5 below:

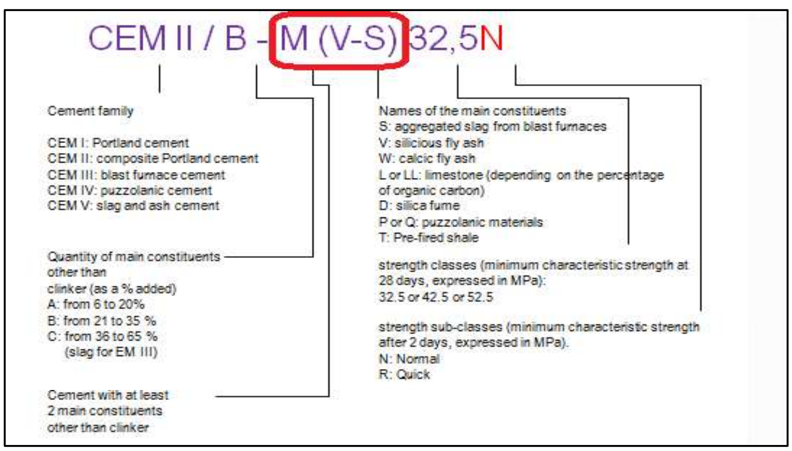

Figure 2.5 Cement classification meaning (Cement & Concrete SA, 2022)

Fly-ash

Fly ash (FA), or Pulverised Fuel Ash (PFA), is a by-product of coal-fired furnaces at power generation stations. According to Heyns (2013), there are only three places in South Africa where fly ash gets classified for use in concrete: Lethabo Power Ash, Kendal Power Ash and Matla Power Ash.

In South Africa, fly ash is categorized into three primary grades: unclassified, air-classified, and twice classified, each with distinct applications:

- Unclassified fly ash is used directly in its raw form, sourced from power plants, primarily for producing bricks and blocks.
- Air-classified fly ash is predominantly employed as a cement extender in concrete and mortars. It undergoes sieving through a 45-micron sieve and is further classified as N and/or S based on the percentage of retained ash when wet sieved on a 45-micron mesh, adhering to the standards specified in SANS 50450-1. Category N designates fly ash with less than 40% mass retained on the sieve, while category S applies to fly ash with less than 12% mass retained.
- Twice classified or double classified fly ash undergoes a double-sieving process.

Fly ash's performance in concrete is linked to its physical, mineralogical, and chemical attributes. Consequently, the choice of fly ash type and grade significantly impacts the quality of the resulting concrete. Classified fly ash exhibits a spherical particle shape, which enhances the workability of concrete, aiding in compaction and density. These fly ash particles intermingle with water and cement particles, catalysing the hydration process. When fly ash reacts with calcium hydroxide, it forms a stable compound and releases heat energy during hydration, lowering hydration temperatures. This reduces the presence of non-durable calcium hydroxide (lime) in concrete and transforms it into calcium silicate hydrate (CSH), as illustrated in Figure 2.6.

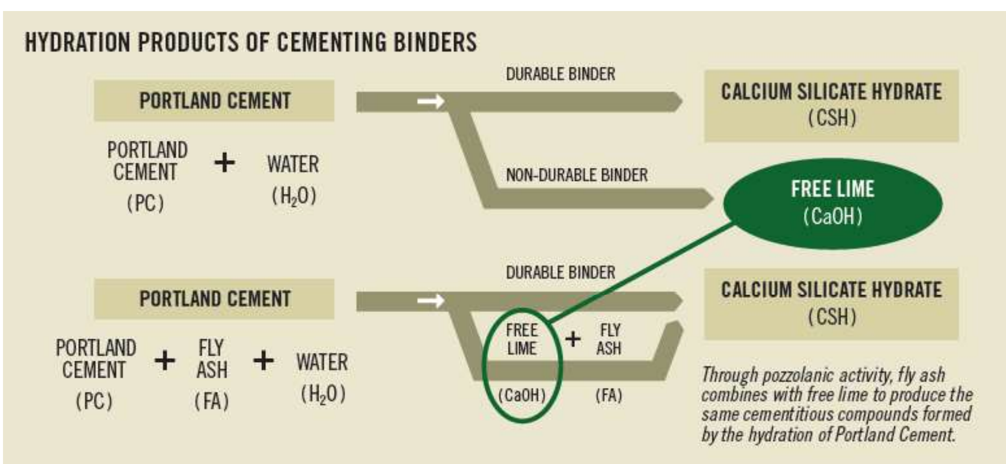

Figure 2.6 Hydration products of cementing binders (Headwaters Resources, 2020)

The applicability of fly ash in RCC was ascertained (Yerramala & Bubu, 2011). Cement typically achieves its maximum strength within a 28-day timeframe. As lime formation occurs, it triggers the availability of cement hydration, with concurrent reactivity of fly ash. Consequently, concrete incorporating fly ash may exhibit slightly lower initial compressive strength than pure cement-based concrete. However, over a year, the former may surpass the latter regarding compressive strength. This enduring reaction of fly ash in the presence of moisture contributes to prolonged and enhanced strength in concrete.

Figure 2.7 illustrates the comparative strength profiles of cement-only and cement-blended mixtures. Concrete compositions with pure cement (depicted by the black line) achieve high strength early on, while mixtures containing 30% fly ash and a 70% blend of cementitious content (represented by the blue line) attain their peak strength at later stages. Despite the longer time required to reach full strength, the cementitious blend mixture ultimately surpasses the strength of the cement-only counterpart.

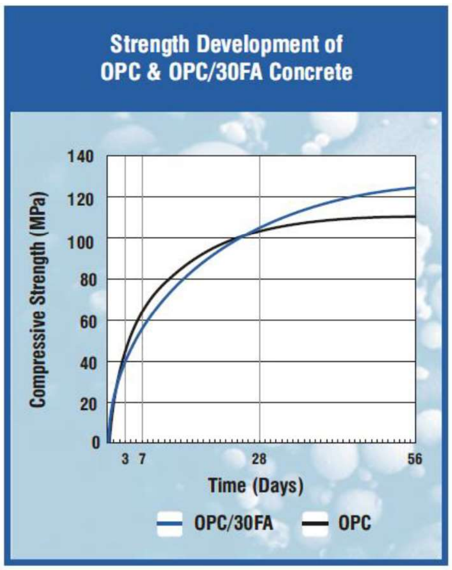

Figure 2.7 Strength development of cement concrete and cement with fly ash concrete (Ash Resources, 2014)

Therefore, concrete made with fly ash has higher compressive strength within a year than cement-only concrete (Gingos and Mohamed, 2011). Increasing the fly ash content in concrete mix can still provide acceptable compressive strength while ensuring durable concrete (Zulu & Allopi, 2014).

Overall, pozzolan material reduces the heat generation in the concrete, increases the workability in the fresh concrete state, increases the fresh concrete placement time, and reduces CO_2 emissions (Avallone, Jahn & Marazzini, 2019).

2.4.2 Water

Water - cement ratio

The water-cement (w/c) ratio is also known as the water-binder ratio, referring to the binder as the total contingent of cementitious materials. It is well known that the water-cement ratio and cement content influence the concrete properties considerably. The reaction between the water and cementitious materials depends mainly on the water-cement ratio of the concrete mixture. The cementitious materials react with the water to form a microstructure. The cement hydration process will fill gaps between the cement particles and form a cement gel. Therefore, concrete with a low water-cement ratio (i.e., 0,3) will fill up gaps rapidly and achieve a strong, dense microstructure. In contrast, concrete with a high water-cement ratio (i.e. 0,69) will take longer to fill gaps and achieve lesser strength and a less dense microstructure. The cement paste undergoes hydration and will fill up available space in the matrix.

As per Rahmani, Sharbatdar and Beygi (2020), the water-cement content ratio directly influences compressive strength, tensile strength, and the static modulus of elasticity. Furthermore, the cement-water ratio affects concrete's water absorption rate, as Gingos and Mohamed (2011) noted. Reducing the water-cement ratio has the consequence of prolonging the time required for Vebe and ultrasonic pulse velocity (UPV) testing. Additionally, it leads to enhancements in compressive strength, tensile strength, flexural strength, and the static modulus of elasticity within Roller-Compacted Concrete (RCC), as corroborated by Rahmani, Sharbatdar & Beygi (2020).

Conversely, Angelucci (2013) posits that increasing the water-cement ratio accentuates capillary pores' prominence as the cement paste undergoes hydration. Hence, the water-cement ratio and cement content emerge as two pivotal factors influencing concrete properties, as Rahmani, Sharbatdar & Beygi (2020) emphasised.

2.4.3 Aggregates

The properties of RCC are influenced by the quality and grading of the aggregates used in the mixture (Rahmani, Sharbatdar & Beygi, 2020). Grading influences the workability, void ratio, and ease of compaction (Van Wyk & Croucamp, 2014). The type and/or quality of aggregate use affects the properties of concrete as it constitutes the largest part of the volume of concrete (Al-Dulaijan et al., 2002).

Aggregates are classified into two primary categories: coarse and fine aggregates, and they are employed in specific ratios within concrete design. As the proportion of coarse-to-fine aggregates increases, the Vebe time also increases. The selection of aggregates wields a substantial influence on the overall quality of RCC, as Mohammad and Nikmohammadi (2017) highlighted.

Alternatively referred to as maximum size aggregates (MSA) in RCC, coarse aggregates can range in size, with maximum dimensions extending to 75mm. However, contemporary practices predominantly favour aggregates up to 38mm to mitigate segregation tendencies associated with larger aggregates, as emphasized by Shaw (2010).

Fine aggregates span a size spectrum ranging from 0.15mm to 4.75mm. The grading and quality of fine aggregates have a notable impact on concrete workability and compaction. An excessive presence of fines can diminish workability, necessitating increased water content, consequently undermining concrete strength, as Shaw and Perrie (2021) cautioned.

Shareef, Raju and Cheela (2019) elucidate that the choice of aggregate type can exert a considerable influence on concrete strength, and substituting one type of aggregate for another can also affect workability and overall concrete properties.

Depending on the intended application of concrete, mix design may call for a combination of fine and coarse aggregates (suited for load-bearing structures), exclusively coarse aggregates without fines (ideal for durable drainage applications), or fines alone (suited for slurry or grouting applications requiring workable concrete). RCC, primarily used in load-bearing structures, necessitates a blend of fine and coarse aggregates.

2.4.4 Concrete chemical admixtures

Chemical admixtures are used in concrete to improve the quality. Admixture is also used to change the concrete properties, which cannot be changed by the cement, aggregate, and water used. Using concrete admixtures and additives influences fresh and hardened concrete properties (Avallone, Jahn & Marazzini, 2019).

RCC's most generally used admixtures are water-reducing plasticisers and air-entraining admixtures. The desired properties of the RCC mixture and the results of laboratory tests will indicate which admixture to use.

2.5 Roller-compacted concrete properties

The nature, quality, and quantity of each constituent incorporated into a mixed design significantly impact concrete's mechanical, chemical, and physical attributes. These factors are pivotal in achieving the desired concrete characteristics specified by the end user, as articulated by Saleh, Rather and Jabber (2017). Consequently, the choice of ingredients employed in concrete formulation can potentially compromise some of the sought-after properties of concrete. RCC properties are categorised into two distinct domains: fresh concrete properties and hardened concrete properties, as depicted in
Figure 2.8 .

Regarding its constituents and anticipated properties, RCC closely resembles conventional concrete, as supported by Calis and Yildizel (2019). However, the consistency of RCC distinguishes itself from conventional concrete due to unique mix proportions, yielding distinctive properties that culminate in a robust material, as detailed by Habib et al. (2021).

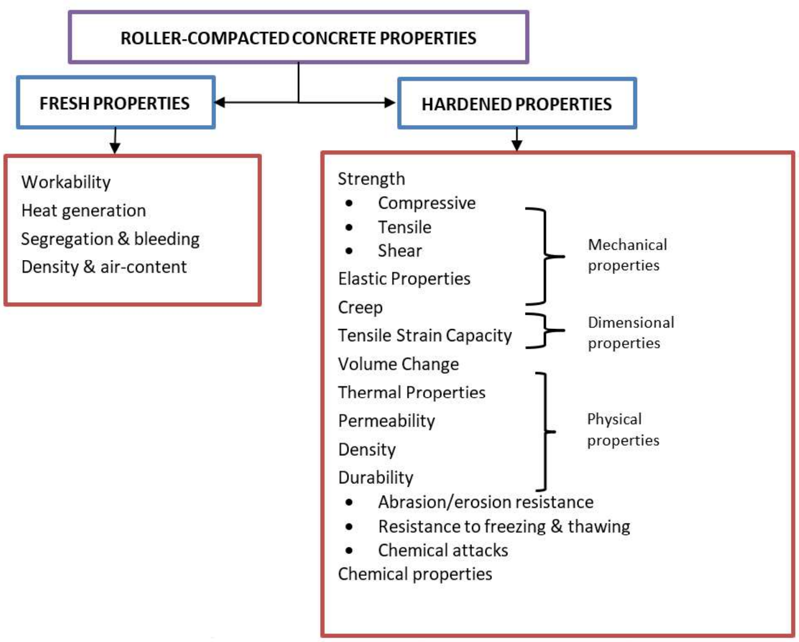

Figure 2.8 Roller-compacted concrete properties for dams

2.5.1 Fresh roller-compacted concrete properties

Concrete is in a fresh state from when it is mixed until it sets (Addis, 1997). The fresh state is also known as the plastic state of concrete. Fresh concrete properties are very important as they determine the quality of the hardened concrete (Addis, 1997). The fresh concrete properties are examined once the concrete constitutes have been approved for use in the final concrete mixture. Examining the fresh RCC properties such as Vebe, air content, and paste content, to name a few, will determine the ease with which RCC can be constructed, and therefore, the desirable design RCC hardened properties will be achieved.

2.5.1.1 Segregation and bleeding of fresh concrete

Segregation is when the heavier materials settle to the bottom, and the lighter slurry rises to the top, leading to adverse consequences in concrete properties. It decreases concrete strength and high porosity and increases permeability (Avallone, Jahn and Marazzini, 2019). Segregation tends to be evident in RCC mixtures with lower cementitious content (Baghdady and Khan, 2018).

Bleeding is a specific form of segregation where water within the concrete migrates to the surface (Addis, 1997). Typically, it is a surface layer of clear or slightly greenish water. However, the absence of this visible layer does not necessarily indicate the absence of bleeding. Using substantial volumes of pozzolanic materials and admixtures can extend the concrete's setting time, increasing the likelihood of bleeding (Crosswell and Brouard, 2021).

Crosswell and Brouard (2021) identify several factors contributing to excessive bleeding:

- Voids beneath aggregates, which diminish concrete strength and elevate porosity.
- Sand streaking, where water ascends along the sides, carries finer sand and cement particles, resulting in an uneven finish.
- Settlement cracking.
- Surface laitance, where water and fine particles migrate to the surface. When troweled into the concrete, it produces a porous, dusty surface.

To prevent segregation in RCC, it is imperative to create a cohesive mixture by precisely controlling the aggregates and moisture content within the blend (Baghdady and Khan, 2018).

2.5.1.2 Density and air content of fresh concrete

The density of fresh concrete is also the unit mass of concrete. This is influenced by the raw materials, water content, air content, and compaction of the concrete (Croswell & Brouard, 2021). The change in water content by 10 l/m^3 will change the density roughly by 15 kg/m^3, and the increase in air content will reduce the density by roughly 25 kg/m^3.

Concrete, after compaction, contains a small percentage of entrapped air, typically around 0,5 to 1 % (Crosswell & Brouard, 2021). For RCC, the allowed entrapped air content on the 37,5 mm portion of the mixture fraction is, on average 1,1% (USACE, 2000).

2.5.1.3 *Generation of heat*

Heat generation is when the temperature of the concrete is not controlled and goes higher than the allowed temperature range during hydration, and stress is produced, which forms cracks in the concrete. If the temperature falls again below the allowed range of temperature, the hydration process is slowed down, and the concrete will take longer to reach full strength (Ballim & Otieno, 2021). It is, therefore, important to control the temperature of concrete. The heat generation is influenced by the cementitious materials in the mixture (Baghdady & Khan, 2018).

A major concern for RCC dam construction is the generation of heat by the main dam structure, which should be considered when designing the structure (Baghdady & Khan, 2018). The hydration heat of the cement causes a rise in temperature (Avallone, Jahn & Marazzini, 2019), while the concrete gains its strength.

The goal is to design an RCC mixture with the lowest possible cement content, with the maximum amount of pozzolan materials consistent with the strength, durability, economic and construction requirements to prevent heat generation (Baghdady & Khan, 2018; USACE, 2000).

2.5.1.4 *Workability*

Workability, or consistency, is a way of determining the capacity of RCC to be placed and compacted without segregation (Baghdady & Khan, 2018). The workability of a mixture is affected by cement content, fly-ash, water, and aggregate content (Baghdady & Khan, 2018). The size of the stone and the quantity of stone used influence the workability. If the stone content is too high, there might not be enough paste to cover the stone and, therefore, compact difficult (Addis, 1997). If the fines modules of the concrete are too low, the concrete tends to lack cohesiveness, but if the fines modules are too high, the concrete tends to be sticky (Addis, 1997). Using fly-ash as part of cementitious content provides improved workability (Avallone, Jahn & Marazzini, 2019; Fleming, 2000). The water content of the concrete mixture influences the Vebe time of the concrete (Rahmani, Sharbatdar & Beygi, 2020), and the materials used in the concrete mixture influence the water requirements of the concrete mixture.

The mixture consistency and/or workability is measured for RCC using a modified Vebe consistency meter. Depending on the RCC mixture design and requirements, the Vebe consistency of 5 – 30 seconds is desired, which will contribute to uniform density, good bonding between lifts, and easy compaction (Baghdady & Khan, 2018). However, RCC mixtures with a Vebe consistency greater than 30 seconds have also been placed successfully (USACE, 2000). Mixture proportions can be adjusted to establish better Vebe times (USACE, 2000).

2.5.2 Hardened roller-compacted concrete properties

The properties of hardened RCC are generally similar to mass concrete (Adaska, 2006). The differences are due to lower water content in RCC; void content in aggregate and other materials might slightly differ (USACE, 2000). Figure 2.6 indicates the hardened properties of RCC as per the USACE (2000). The variation of RCC properties might differ from project to project due to the range of aggregate qualities, lower cementitious material content, amount of pozzolan used, in general, the materials quality and the compaction of RCC (USACE, 2000).

The properties of concrete at early ages are significantly different from those of mature concrete properties. All concrete in a hardened state should be strong, durable, and dimensionally stable (Abdo, 2008). The different hardened properties can be divided into the following properties, namely, mechanical, physical, durability, dimensional and chemical, as shown in
Figure 2.8 .

2.5.2.1 Chemical properties of concrete

The mixture of sand, stone, cementitious materials, admixture, and water in concrete has chemical constitutions and contributes to the chemical makeup of the particular mix design and final material product. Some of the chemical challenges in concrete are the reaction between the cement and water, the cement and fly ash, and the reaction of aggregates with the alkali hydroxides in concrete (Robinson, 2019). These ingredients are discussed in section 2.4 of this literature review.

2.5.2.2 Dimensional properties of concrete

Dimensional properties are creep and elastic properties, which are the elastic properties of RCC (Baghdady & Khan, 2018). Elastic properties can be divided into the modulus of elasticity and Poisson's ratio. Modulus of elasticity can then be sub-divide into modulus of elasticity of CMC, tensile and sustained modulus of elasticity (US Army Corps of Engineers, 2000).
Creep is a time-dependent deformation due to sustained loading over a long term (Baghdady & Khan, 2018).

2.5.2.3 Durability properties of concrete

Durability is the abrasive/erosion resistance, frost resistance and chemical attacks to which RCC dams are exposed, leading to deterioration of the concrete (Baghdady & Khan, 2018). The durability of the concrete is influenced by the quality of aggregates used and the compressive strength of the concrete.

2.5.2.4 Physical properties of concrete

Physical properties are volume change, which is the swelling and shrinkage and thermal expansion of concrete, thermal properties, water permeability, concrete density, and the tensile strain capacity.

a) Hardness of concrete

The hardness of concrete can be seen as the hardness of the aggregates used in the concrete and the paste of the concrete. Low-strength concrete cement paste is relatively weak in hardness and tends to abrasion. Hardened cement paste is influenced by the porosity of the cement paste, which is related to excess water from the workability of the fresh concrete, which is not used up during the hydration process.

b) Permeability

Permeability of concrete is the quantity of water, air and other substances that penetrate through the concrete pores. RCC's permeability is largely influenced by total mixture proportioning, placement method, and compaction degree. High cementitious materials content mixtures have a lower permeability than low cementitious material content mixtures (USACE, 2000). Hardened RCC permeability is comparable to conventional concrete (Adaska, 2006). Typical values for RCC range from 1,5 to 150 x 10^{-8} mm/sec (USACE, 2000).

c) Density of concrete
The density of RCC is the mass per unit volume and relies on the aggregate's degree of compaction and density. RCC has a low water content and low entrained air, so it has a higher density than conventional concrete. Conventional concrete density of approximately 2400 kg/m³ and RCC concrete have arranged between 2424 to 2500 kg/m³ (Baghdady & Khan, 2018).

d) Volume change and thermal properties
Volume change in RCC is experienced as the concrete's shrinkage, thermal expansion, and contraction. For RCC, the volume changes are less than conventional concrete, resulting in less cracking. However, with respect to thermal considerations, due to the chemical reactions of the cementitious materials, there is heat rise which causes expansion (Adaska, 2006).

e) Tensile strain capacity
Tensile strain capacity is the maximum tensile strain that the concrete can withstand without forming cracks. Tensile strains developed from external loads as well as by volume change of concrete (USACE, 2000).

2.5.2.5 Mechanical properties

Mechanical properties are the strength of concrete, which is compressive, tensile, shear and flexural. RCC's strength and elastic properties vary depending on the mixture proportions and components, with emphasis on aggregate quality and cementitious content affecting the strength and elastic properties, as well as the compaction of RCC (USACE, 2000). RCC's strength can be tested using compacted specimens or specimens cored from the dam wall (USACE, 2000). According to Calis and Yildizel (2019), compressive strength test needs to be done on RCC.

a) Tensile strength
The tensile strength can be subdivided into direct tensile strength, lift joint direct tensile strength, splitting tensile strength, flexural strength, and dynamic tensile strength (US Army Corps of engineers, 2000).
Tensile strength is used to determine the loading design and is compared to compressive strength. The tensile strength between the lift joints is important (Badhdady & Khan, 2018).

b) Shear strength
Shear strength can be subdivided into parent shear strength and lift joint shear strength (US Army Corps of engineers, 2000). For RCC dams, the shear strength between the lift joints properties is important (Badgdady & Khan, 2018).

c) Compressive strength
The strength properties depend on the degree of compaction, aggregate quality and grading, cementitious content, and type of cementitious materials used for a RCC mixture (Avallone, Jahn & Marazzini, 2019). Good and correct compaction is essential for all RCC. Depending on the percentage of voids, the percentage of strength loss due to insufficient compaction varies by as much as 80% loss in strength (USACE, 2000).

Compressive strength in RCC is used to measure the overall strength mixture properties and gauge the durability and quality of the concrete (Avallone, Jahn & Marazzini, 2019). During construction, the compressive strength test is used as a measurement to monitor the variability of the mixture and to confirm the achievement of the design properties of the mixture. The cementitious material content, type of cementitious material, water-cement ratio, grading and quality of the aggregate, and compaction mainly influence the compressive strength. Generally, the compressive strength specified for RCC dam mixtures ranges from 6,9 MPa to 27,6 MPa at 1-year age (USACE, 2000).

2.6 Testing roller-compacted concrete properties

Different tests and methods are used to test fresh as well as hardened concrete quality and durability.

2.6.1 Fresh roller-compacted concrete properties tests

As RCC is a zero-slump concrete, Vebe test is done to determine the fresh property regarding workability for RCC. Once RCC is mixed, it is critical to test the Vebe test. Other tests that might also be required are materials gradation, initial and final setting times, RCC moisture content, temperature and compacted density (CIGB ICOLD, 2022). These fresh property tests need to be done on different frequencies. For this research project only loaded Vebe test will be done.

2.6.1.1 Vebe consistency meter

The Vebe test equipment was developed by Swedish engineer V. Bahrner (Zongjin, 2011). Loaded Vebe test is used to determine the consistency and compatibility of fresh RCC. This test only applies to RCC with a maximum aggregate size of 50 mm or less. Alternatively, the test can be performed on the RCC mixture after passing the 50 mm sieve (ASTM C1170M). It is important to accurately record the vibration time, defined as loaded Vebe time (CIGB ICOLD, 2020). The time recorded shall be in seconds. There are two different procedures to follow for the loaded Vebe test, test A for very stiff to extremely dry RCC and test B for stiff to very stiff RCC, where the difference between the two tests is the total masses are loaded for the Vebe times. The total mass includes the transparent disc (plastic base plate) and metal shaft (ASTM C1170M). See Figure 2.9 for a section view of the Vebe consistency meter apparatus. Vebe test measures the time required for the ring of mortar to cover the transparent disc. For loaded Vebe time required less than 20 seconds, procedure B will be followed, using a surcharge mass of 12,5 kg. Where loaded Vebe time exceeds 30 seconds, procedure A with a surcharge mass of 22,7 kg will be used (ASTM C1170M).

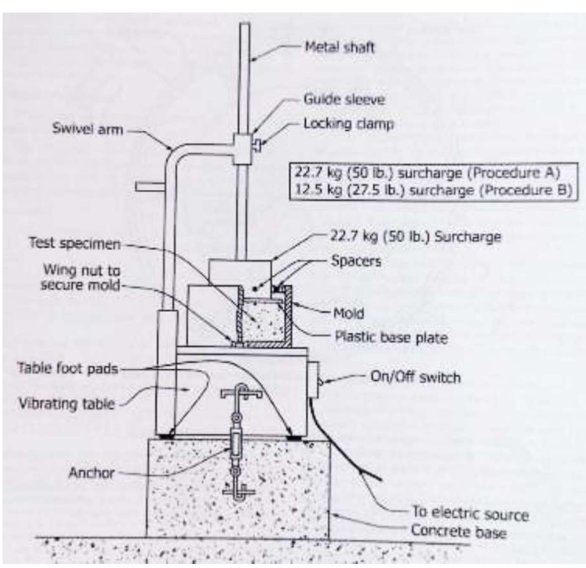

Figure 2.9 Section of Vibrating Table- Vebe Consistency Test (ASTM C1170M)

2.6.2 Test methods for hardened roller-compacted concrete properties

Hardened concrete signifies solidified concrete, initiating gaining strength and durability. It encompasses concrete ranging from a few hours old to structures that have endured for several years.

Traditionally, the emphasis was primarily on employing destructive testing methods to evaluate the quality of hardened concrete. However, recent research has unveiled the efficacy of non-destructive testing methods in assessing concrete quality. Among the critical destructive testing methods used to gauge concrete's mechanical characteristics and quality, the compressive strength test assumes a pivotal role. This test is instrumental in determining the concrete mix's compressive strength, a factor that significantly influences the structural lifespan, thereby impacting the efficiency and effectiveness of the structure. Figure 2.10 illustrates the dichotomy between the two main categories of test methods, destructive and non-destructive. Both are essential for evaluating the quality and resilience of hardened concrete.

As outlined by CIGB ICOLD (2020) standards, it is mandatory to assess several hardened properties of Roller-Compacted Concrete (RCC) against the prescribed design values. These assessments encompass various compressive strength tests, tensile strength tests, evaluations of elastic modulus, and measurements of permeability, all conducted at different frequencies.

In the context of this research project, the focus will be exclusively on the mechanical properties related to compressive strength. This will involve a comparative analysis of compressive strength tests alongside non-destructive assessments, specifically ultrasonic pulse velocity and the rebound hammer test, concerning their correlations with the compressive strength test results.

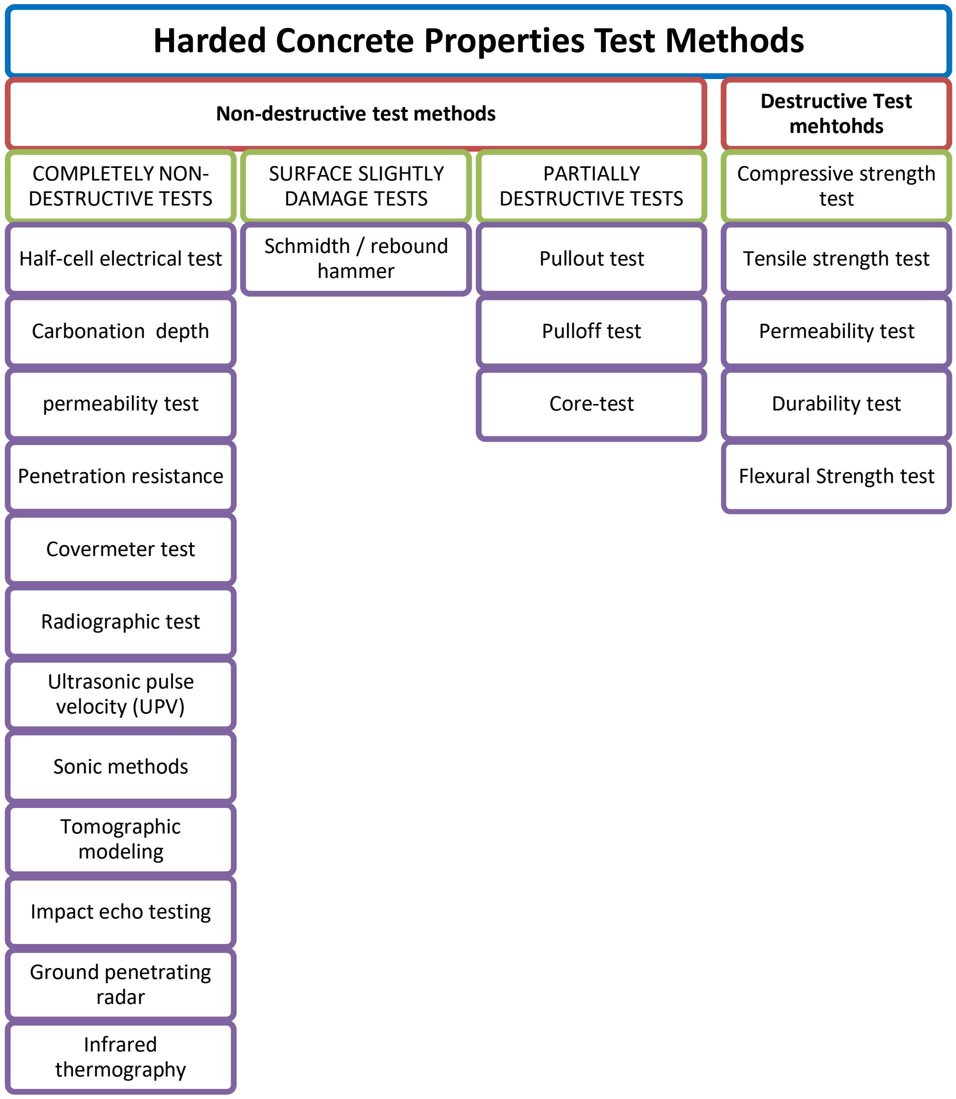

Figure 2.10 Summary of non-destructive and destructive test methods

2.6.2.1 Non-destructive testing

Non-destructive testing (NDT) is testing concrete strength without damaging the structure. NDT is used in the concrete industry to verify different parameters, such as density, elastic modulus, strength of hardened concrete, surface hardness, surface absorption, reinforcement details, and sometimes detecting voids, cracking, and delamination (International atomic energy agency, 2002). There are 13 different NDT test methods, which are more cost-effective than destructive testing. Non-destructive testing techniques are sensitive to the physical properties of the concrete and provide only an indirect way towards the mechanical performance of the concrete (Breysse, 2012).

a) Ultrasonic Pulse Velocity (UPV)

Ultrasonic pulse velocity measures the velocity of an ultrasonic pulse passing through concrete over a known distance. The ultrasonic pulse velocity test is measured with an electro-acoustical transducer on both sides of the concrete. One prob is called the transmitter, and the other prob the receiver. The transducer sends out a series of waves and detects the longitudinal waves. Figure 2.11 shows the three possible placement arrangements for the transducer, transducer prob on the concrete in an ultrasonic pulse velocity test (International atomic energy agency, 2002).

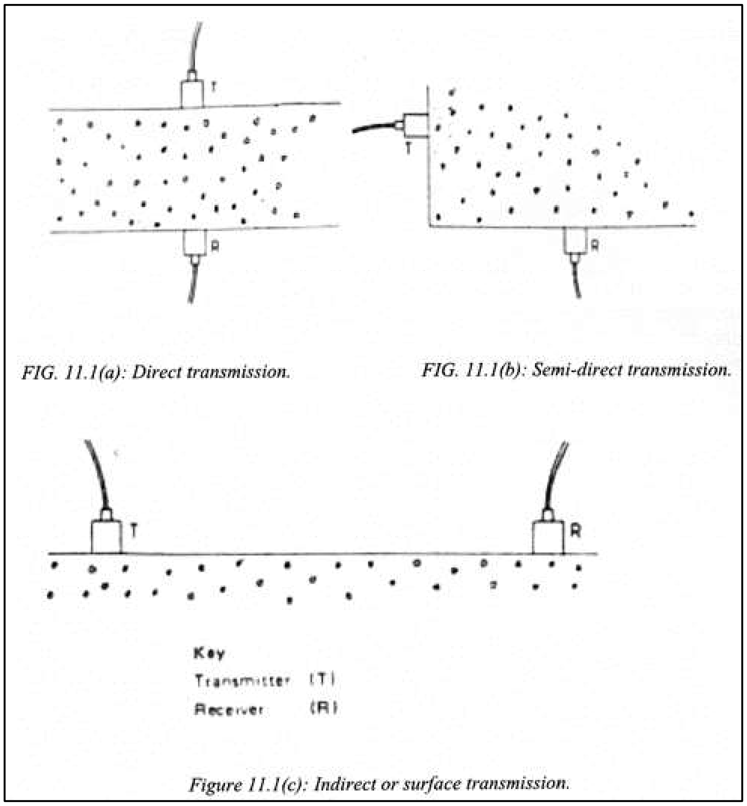

Figure 2.11 Three arrangements for transmission probes. (International Atomic energy agency, 2002:102)

There are three placement arrangements for measuring UPV, as indicated in Figure 2.11. Ideally, the direct transmission position should be prioritised, as it has the maximum energy transfer between the transducers, ensuring accuracy of velocity measurements. The semi-direct arrangement has intermediate sensitivity between the transducer arrangement and a potential reduction in the accuracy of the measurement over the path length. Indirect measurement should be reserved for cases where one face of concrete is accessible, or to measure a depth of a crack, or when the overall quality of the concrete surface is of interest. The indirect arrangement is the least sensitive for a given path length, with an amplitude typically about 2% or 3% of that direct transmission produces. Additionally, the indirect UPV arrangement measurements are often influenced by the concrete near the surface (International Atomic Energy Agency, 2002).

The UPV test method involves a piezoceramic source that is electronically pulsed to generate ultrasonic waves, which travel through the concrete element from the transducer to the matching receiver. The wave form is recorded at the receiver. There are three types of propagating mechanical waves, namely:
P-waves (compressional waves or longitudinal waves), S-waves (shear waves) and surface waves (Carette & Staquet, 2015).

The relationship between these velocities depends on the material properties of the concrete. In the context of the UPV test, these velocities are typically measured experimentally, and the UPV calculated using the formula shown in Equation (2.1):

$$V = \frac{L}{t} \tag{2.1}$$

Where:
V = pulse velocity (measured in meter per seconds (m/s));
L = distance between centres of the transducer faces (measure meter (m)); and
t = transit time (measure in seconds (s)).

The relationship between compressional waves and shear waves in concrete can be expressed using Poisson's ration as shown in Equation (2.2):

$$V_p = \sqrt{\frac{2V_s^2(\nu - 1)}{2\nu - 1}} \tag{2.2}$$

Where:
Vp $=$ P-wave velocity
Vs $=$ S-wave velocity
$\nu =$ Poisson's ratio

It is important to note that the specific formula for the relationship between P-waves and S-waves in concrete may vary depending on the assumptions made in the model used for the concrete's behaviour. The formulas provided here are general representations based on elastic wave propagation theory in isotropic materials.

It is useful to use UPV measurements to estimate the concrete strength. However, the relationship between compressive strength and UPV is affected by a number of factors. Some of these factors include the age of the concrete, the curing conditions, moisture conditions, mix proportions, the type of aggregate used for concrete, and the type of cement used in the concrete. It is, therefore, important to establish a correlation between compressive strength and UPV of the particular type of concrete under investigation. With this said a relationship between the compressive strength and UPV P-wave to determine the quality of the concrete using a density concrete of approximately 2400 kg/m3 is indicated in Table 2.4, which Whitehurst suggested according to the International Atomic Energy Agency (2002). According to Proceq, Screening Eagle class notes, the classification of concrete quality for UPV S-waves is also indicated in Table 2.4.

Table 2.4 Classification of concrete quality for UPV measurements

Quality of concrete	P-wave velocity (km/s)	S-wave velocity (km/s)
Excellent	>4.5	>2.8
Good	3.5 – 4.5	2.1 – 2.8
Doubtful	3.0 – 3.5	1.7 – 2.1
Poor	2.0 – 3.0	< 1.7
Very poor	< 2.0	

Some of the references suggest that concrete quality with a velocity below 3.0 km/s is considered of poor quality. Therefore, there may be variations in velocity criteria for concrete quality grading, with the consensus that velocities below 3.0 km/s generally indicate poorer quality of concrete.

The ultrasonic pulse velocity method has been used for over 60 years and is considered one of the oldest NDT techniques (Kencanawati et al., 2018).

The velocity of the ultrasonic pulse depends on the shape and size of aggregate, concrete voids, and the quality of the concrete through which they pass elastic properties and mechanical strength. The compressional wave transmitted through the concrete might undergo scattering due to aggregate. This forms a complex wave when it reaches the receiver transducer (Naik, Malhotra & Popovics, 2004). Therefore, the ultrasonic pulse velocity test method can be used to measure the concrete's properties and the concrete's uniformity, to mention a few. High velocity obtained on the concrete measure means a good quality in terms of the concrete's density, homogeneity, and uniformity (International atomic energy agency, 2002).

Factors affecting the UPV test can be divided into two categories: factors resulting from the concrete properties and factors from other influences. The factors of the concrete that can influence the concrete are the aggregate type, grading, size and content in the mixture, the cement type used, the water-cement ratio, admixtures used and the age of the concrete. The other factors can be the contact between the transducer and concrete, the temperature of the concrete, the moisture and curing condition of the concrete, the path length, size and shape of the specimen and even other vibrations in surroundings (like the use of a jackhammer) to name some.

b) Rebound hammer

A Schmidt hammer is a device to measure the surface hardness of concrete by rebound of the elastic mass of the concrete surface against which the mass impinges, as shown in Figure 2.12 (Brencich et al., 2013).

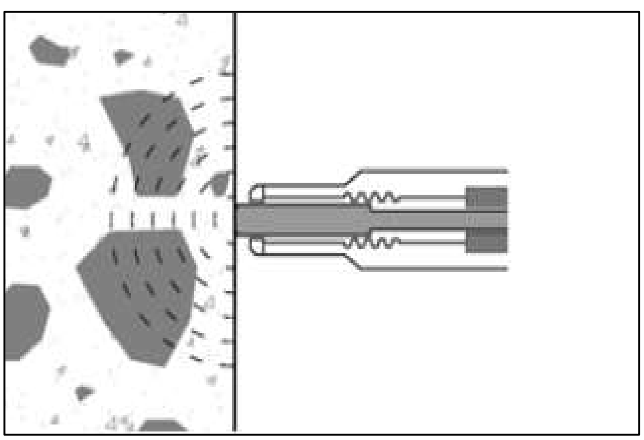

Figure 2.12 Effects of the rebound test (Brencich et al., 2013)

The non-destructive assessment of concrete strength has a historical extraction dating back to 1934. Ernst Schmidt's pioneering work in the 1950s finished developing the Schmidt hammer device, an innovative tool for conducting non-destructive compressive strength tests based on rebound index measurements. This innovation exited the traditional ball penetration test, as it offered direct scale readings without necessitating surface measurements. Subsequent refinements and enhancements have enabled the digital recording of concrete amend coefficients using Schmidt hammers (Szilágyi & Borosnyói, 2009).

Energy dissipation is a fundamental aspect of the rebound hammer's operation. It occurs due to wave reflection, mechanical wave attenuation within the hammer, and the concrete's energy loss during compression under the plunger tip. This inherent energy dissipation renders the rebound hammer a reliable tool for estimating concrete strength (Szilágyi & Borosnyói, 2009).

The usefulness of the rebound hammer extends to its usability in various orientations, including horizontal, vertical overhead, vertical downward positions, and intermediate angles. The basis of the rebound index value is the position of the hammer's impact with the force of gravity. Consequently, it is imperative to record the readings following the direction of the strike. For instance, the rebound index value obtained on a floor (in a vertically downward position) is anticipated to be smaller than the value recorded on a roof soffit (in a vertically overhead position) (International Atomic Energy Agency, 2002).

Despite its advantages in terms of speed and cost-effectiveness, the rebound hammer test is susceptible to the influence of certain concrete properties. These variables can impact the accuracy of rebound hammer measurements (International Atomic Energy Agency, 2002; Szilágyi & Borosnyói, 2009).:

- Quality of the concrete surface – the smoothness of the surface, if the surface is rough, it needs to be smoothened.
- Size, shape, and rigidity of the specimen – if the concrete does not form part of a large mass, any movement of the impact of the hammer will lead to reduction in the rebound measurement.
- Age of the specimen – if was found that the rebound number for 7-day old concrete was higher than for 28-day old concrete. Therefor it is important that a direct correlation between compressive strength and rebound number for concrete mixture needs to be determined. Rebound hammer testing should not be done on low strength concrete at early ages when the concrete strength is still lower than 7 MPa.
- Moisture content of the concrete – rebound number is lower for dried concrete than for the same concrete being soaked in water and tested on a saturated dried surface.
- Type and amount of coarse aggregate – even if the same type of aggregate is used in a concrete mix, the correlation curve can differ if the amount of aggregate differs.
- Type of cement and amount of cement – the correlation curve for different cement types and amount of cement use will differ, as the compressive strength will differ.
- Compaction of the structure
- Curing method
- Temperature and stress state

It is taken that the higher the rebound number, the higher the compressive strength of the concrete, it is only useful if a correlation was developed between the rebound number and the concrete compressive strength that is being tested (International Atomic Energy Agency, 2002). The aim of the rebound hammer test is to find a relationship between concrete surface hardness and the compressive strength (Szilágyi & Borosnyói, 2009). Rebound hammer test can be used in the field and in the laboratory (International Atomic Energy Agency, 2002).

In determining concrete quality grading, the correlation curves of the rebound hammer play a significant role, providing an estimate of the compressive strength of the concrete. Various research studies, including those by Malek (2020) and Yahya et al. (2018), refer to a similar table format for assessing the quality of concrete based on rebound hammer values, as illustrated in Table 2.5.

Table 2.5 Classification of concrete quality for Rebound number (Malek (2020) and Yahya, et al., (2018))

Quality of concrete	Average rebound number (RN)
Very good hard layer	>40
Good layer	30 – 40
Fair	20 – 30
Poor concrete	< 20
Delaminated	0

This classification uses the average rebound number to categorize concrete quality. For instance, a rebound number greater than 40 indicates a very good hard layer, while a rebound number less than 20 suggests poor concrete quality. The value of 0 is assigned to delaminated concrete. This approach provides a practical and straightforward method for assessing concrete quality based on rebound hammer measurements, as supported by Malek (2020) and Yahya, et al., (2018).

2.7 Conclusion

RCC has become a crucial material in the construction of dam walls, offering a durable and efficient mass-fill option. Since its beginning in the early 1980s, the development of RCC mixture designs has advanced significantly. The initial formulations were characterised by stiff, low-paste concrete, but they have since evolved into more workable and dense blends, often referred to as "wet RCC."

The choice of RCC mixture design depends on several key factors, primarily the strength and durability requirements of the structure, as well as the availability of materials, transport methods, and construction equipment. Each of these elements is site-specific, dictated by the requirements of the particular dam construction project.

RCC mixtures primarily comprise cementitious materials, including Ordinary Portland Cement (OPC) and Supplementary Cementitious Materials (SCM), water, high-quality aggregate materials, predominantly coarse aggregates, and various chemical admixtures. The construction of RCC dams in South Africa hinges on key considerations such as durability, strength, workability, heat generation, and aggregate quality.

RCC exhibits distinct properties in both its fresh and hardened states. Fresh properties encompass critical aspects such as segregation and bleeding of the concrete, density, air content, heat generation, and workability. Meanwhile, hardened properties include mechanical strength, chemical attributes, dimensional characteristics, physical attributes, and overall durability.

Numerous testing methods are available to assess the mechanical properties of fresh and hardened RCC. The Vebe consistency meter is employed to evaluate fresh properties, while the mechanical property of compressive strength can be determined using Non-Destructive Testing (NDT) and Destructive Testing (DT) methods. DT testing, however, results in permanent damage to the concrete, making NDT techniques a more economical and non-destructive alternative.

Ensuring the high-quality construction of a safe dam structure is of utmost importance, necessitating a thorough understanding of the concrete's properties. While compressive strength is widely recognised and used to assess concrete quality, it is a destructive testing method that can prove costly and time-consuming.

Non-Destructive Testing has seen significant developments since the early 1900’s, offering alternative means to evaluate concrete quality. In particular, the combined use of UPV (Ultrasonic Pulse Velocity) and RH (Rebound Hammer) testing has yielded more reliable results. Research has also revealed correlations between NDT test results and the compressive strength of certain concrete types. However, it is essential to establish a specific correlation for each concrete mixture design.

Existing research has not revealed a clear correlation between compressive strength and ultrasonic pulse velocity results for RCC used in dam construction. Hence, this research project aims to investigate the relationship between NDT ultrasonic pulse velocity results and DT compressive strength test results for RCC. The objective is to determine whether a correlation exists, ultimately saving time and costs on future dam investigations and maintenance.

Chapter 3 Research methodology

The assessment of concrete characteristics and quality is a well-established global practice employing various techniques and methodologies. Nevertheless, there is a notable lack of information regarding the application of non-destructive testing methods in characterising Roller Compacted Concrete (RCC) dams. Furthermore, no previous attempts have been made to establish a correlation between the conventional destructive compressive strength testing and the non-destructive methods of Rebound Hammer (RH) and Ultrasonic Pulse Velocity (UPV) for RCC dams. Consequently, this study aims to ascertain the correlation between compressive strength, rebound hammer values, and ultrasonic pulse velocity measurements for RCC dams.

This chapter delineates the comprehensive methodology employed in addressing the research investigation. It outlines the systematic approach taken to execute the experimental study, encompassing processes such as data acquisition through literature review, concrete mix design, and laboratory and field testing to assess the feasibility of developing the relationship between DT and NDT.

3.1 Research design

Experimental and field assessment procedures were carried out for this study. The experimental research used high cementitious RCC, with two different quantities of total cementitious material. Trial mixes were prepared to achieve optimised mixes, by adjusting the coarse and fine aggregate content in the RCC mixes. For field assessment, tests were conducted at De Hoop dam and Spring Grove dam.

The Spring Grove dam RCC mix design was used for the base mixture 15/38-365 mix design for this research. This RCC mix design is known and used and referred to as dry RCC mix. The base mixture 20/38-90, which had more cementitious material and water, is known as a wet-pasted RCC mix. This is not a known RCC mix design used to construct dams. The 20/38-90 base mixture was similar to the RCC mix recipe designed and used in South Africa to construct the De Hoop Dam.

These mix designs conformed to the design requirements for RCC, and standards of US Army Corps (U.S. Army Corps of Engineers. 2000 / USACE, 2000) and Department of Water and Sanitation (DWS) Standards (Department of Water and Forestry, 2005).

Field test was conducted at the De Hoop dam and Springs Grove dam, as these dams' original construction RCC mix designs was used as base mix designs for this research project and secondly as these two dams' are the only dams currently in South Africa, that was constructed using IVRCC and therefore will give a true reflection of RCC dam structures measurements results. If the test was conducted on other so called RCC dam structures in South Africa the measurements would be done on conventional skin concrete and not on RCC. However, in the case of both mixtures, the final mix designs were slightly different due to the use of different aggregates.

3.2 Research methodology

The research is structured into two distinct phases. In the initial phase, the study involves the meticulous preparation of Roller Compacted Concrete (RCC) within a controlled laboratory environment. Subsequently, the laboratory analysis includes the determination of critical parameters such as compressive strength, Ultrasonic Pulse Velocity (UPV), and Rebound Hammer Index values for all the samples. These data are subjected to a comprehensive analysis through the utilization of a multiple regression model to uncover potential relationships.

In the second phase, a field-based Non-Destructive Testing (NDT) process is executed on two dams located downstream of non-overspill wall sections. This field-based NDT endeavour serves the purpose of validating the relationships previously established during the laboratory phase.

The research capitalizes on equipment from the Department of Water and Sanitation's Clanwilliam Dam Raising Project, as well as the construction laboratory and Civil Engineering laboratory equipment. This specialised equipment is employed for tasks including concrete mixing, Vebe testing, and the creation and meticulous curing of specimens. The procedural workflow is depicted in Figure 3.1. Notably, specimens are transported between the Department of Water and Sanitation's Clanwilliam Dam Raising Project's construction laboratory and the laboratory facilities of Cape Peninsula University of Technology on testing days. The laboratory facilities at Cape Peninsula University of Technology are utilised for conducting UPV, Rebound Hammer Index, and compressive strength testing of the specimens. Finally, UPV and Rebound Hammer field tests are carried out at the non-overspill sections of the De Hoop Dam and Spring Grove Dam walls.

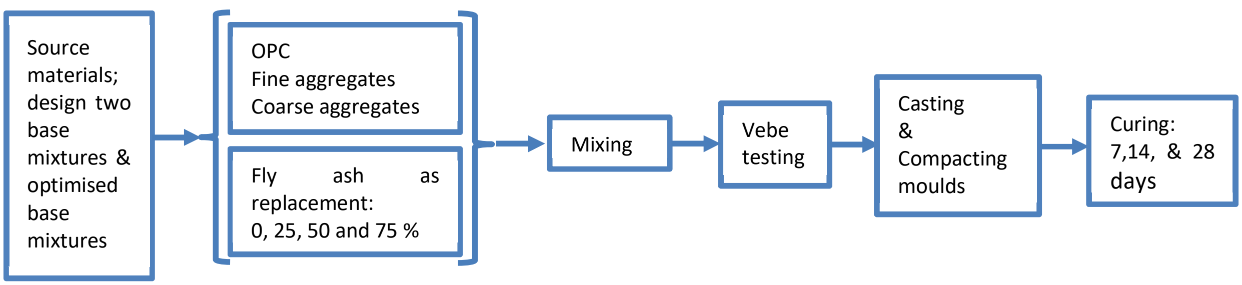

Figure 3.1 Experimental procedure followed for making RCC

3.2.1 Materials used for experimental laboratory testing

To prepare the samples, cement and fly ash were employed as the primary cementitious materials. Additionally, crushed and graded coarse aggregates with a maximum size of 19 mm, crusher run, river sand, an admixture, and potable water were incorporated into the trial mixtures.

3.2.1.1 Cementitious materials

Portland cement CEM I: Riebeeck West Suretech 52.5 N

The cement used in this study was procured from the Riebeeck PPC Factory, located just outside Riebeeck West in the Western Cape. It falls under the product range known as 'Suretech.' As previously elaborated in Chapter 2, Section 2.4.1, the notation '52,5' designates the strength gain at 28 days, while 'N' denotes the class of cement, with 'N' representing ordinary early-strength cement. This specific type of cement, CEM I, predominantly comprises clinker and incorporates an additional 5% of limestone as a constituent. Please refer to Appendix B for the accompanying datasheet.

The fly ash, classified as Class F and bearing the name 'Durapozz,'. It adheres to the stipulations outlined in SANS 50450-1. The utilisation of fly ash in this study was subject to variations, in line with literature findings that indicate fly ash's impact on water absorption in concrete, a factor directly influencing the durability of the concrete.

3.2.1.2 Aggregate

Fine Aggregate: The river sand was sourced from a local supplier in the Clanwilliam area in the Western Cape. The combination of mixture 20/38-90 and Mixture 15/38-365 consists of 70% river sand and 30% crusher run. A 5 mm crusher run of Dolomite aggregate type was added to the river sand to get grading in typical target fine aggregate grading, as shown in Figure 3.2. The targeted typical grading for RCC developed by the US Army Corps Engineering (2000) for fine aggregate was used. The full grading analysis is in Appendix C.

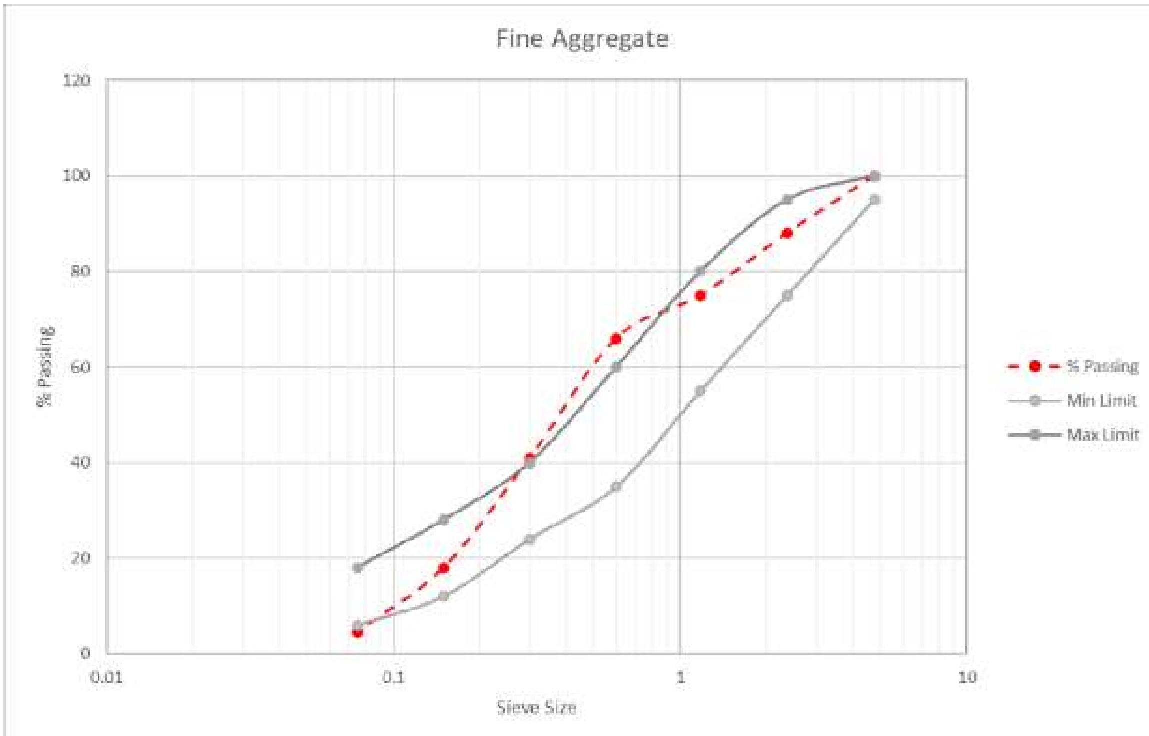

Figure 3.2 Fine aggregate particle distribution curve

Coarse Aggregate: The coarse aggregate employed in this study was procured from a local quarry, Cape Lime, located in the Vredendal region within the Western Cape. The coarse aggregate in this research comprises crushed Dolomite aggregate, available in 20 mm and 10 mm sizes.

The composition of the 20/38-90 mixture consists of 95% 20 mm aggregate and 5% 10 mm aggregate. In the case of the 15/38-365 mixture, it comprises 90% 20 mm aggregate and 10% 10 mm aggregate. These specific aggregate proportions were chosen per the established grading criteria delineated in the US Army Corps Engineering (2000) guidelines for coarse aggregates. Notably, for the 20/38-90 and 15/38-365 mixtures, the 10 mm and 20 mm aggregate components deviate from the prescribed minimum allowable limits. A visual representation of the aggregate grading for these mixtures is provided in Figure 3.3, while for a more comprehensive grading analysis, please refer to Appendix C.

Figure 3.3 Coarse aggregate particle distribution curve

3.2.1.3 Admixture

The plasticiser influences the concrete mixture by elevating the concrete slump while preserving the water content. This plasticiser facilitates improved concrete compaction, thereby reducing the required vibration time. In this study, we utilized a water-reducing plasticizer developed by Chryso specifically for Roller Compacted Concrete (RCC), known as "Plast RCC," in the various trial mixtures.

The choice of plasticiser ratio between the trial mixtures of 20/38-90 and 15/38-365 is contingent on each mix's distinct total cementitious content. A range of 0.3 to 0.5 litres per 100 kg of cementitious materials (including extenders) was applied to calculate the quantity of extenders to be added to each mix.

3.2.1.4 Water

Concrete standards/specifications specify drinking water quality water for concrete mix designs, as impurities in the water can significantly affect the concrete's chemical properties (SANS 51008:2006). Thus, portable water from the DWS Clanwilliam construction site laboratory at room temperature was used to prepare the RCC mixes.

3.2.2 Research equipment

This section outlines the apparatus used during the experimental phase. These apparatus and instruments were methodically calibrated and maintained in adherence to established standards and the manufacturers' recommendations.

A comprehensive list of standard laboratory equipment utilised throughout the experimental procedures includes sieves, scales, laboratory pans, bags, buckets, and containers. Additionally, a concrete mixing machine, slump measurement equipment, Vebe consistometer machine, vibrating tables with associated weights, cast iron cube moulds, curing bath along with a temperature control heater and water pump, temperature gauges, compressive strength testing machines, rebound hammer, and electro-acoustical transducer (UPV machine) were employed.

Subsequent sections in this research methodology will delve into the specific details of the apparatus used to obtain results and equipment that played a significant role in the study.

3.2.2.1 Vebe consistometer

The consistency of stiff concrete in its fresh state was measured using a Vebe consistometer machine.

The Vebe consistometer machine comprises several components, including a cylindrical metal container with a 240 mm diameter and a height of 200 mm. This container has handles and brackets for secure clamping onto a vibrating table. Additionally, there is a frustum cone-shaped mould, similar in shape to a slump mould, featuring handles located at two-thirds of its height. A transparent disc, 230 mm in diameter and 10 mm thick, is horizontally attached to a rod. This rod fits vertically into a sleeve mounted on a swivel arm, which can be firmly fixed in position using a screw. A funnel is positioned on this swivel arm and placed over the mould, centred within the metal container. A weight is then positioned on top of the transparent disc. For a visual representation of the Vebe consistometer machine, please refer to Figure 3.4.

The combined mass of the transparent disc, rod, and weight is adjusted to a specific value depending on the type of concrete under examination. In this project, the total mass was set at 12 kilograms. To facilitate the testing process, a tamping rod is utilised for creating the slump within the mould, and a stopwatch is an essential instrument for accurate test timing.

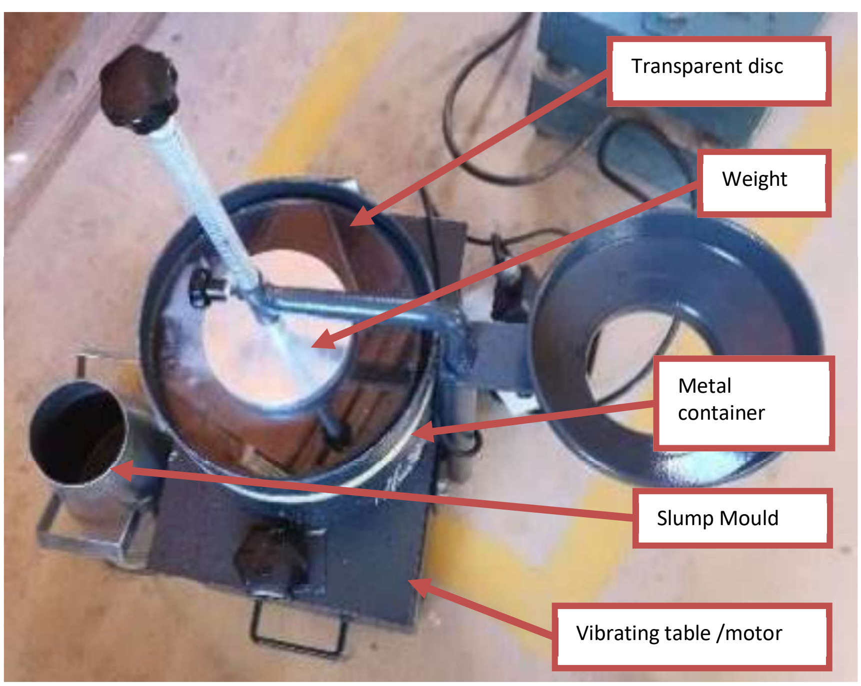

Figure3.4 Vebe Consistometer Machine

3.2.2.2 Ultrasonic pulse velocity

An ultrasonic pulse velocity instrument (Electro-acoustical transducer), as shown in Figure 3.5, was used to transmit waves through concrete, based on measuring the travel time of the compression stress waves through the concrete over a known distance. The variation in the concrete density, elastic properties, and other flaws in the concrete lead to changes in the pulse velocity (Shaw & Perrie, 2021). The apparatus consists of a pulse generator, a transmitter and receiver (pair of transducers), an amplifier, a time measuring circuit, a time display unit and connection cables.

The waves from the transducer transmitter to the receiver transducers are measured over a distance (L) and the transit time (t) it takes to travel through the concrete. The compressional wave through the concrete is therefore calculated using Equation (2.1):

$$V = \frac{L}{t} \tag{3.1}$$

Where:
V = pulse velocity (measured in m/s);
L = distance between centre's of the transducer faces (measure meter (m)); and
t = transit time (measure in seconds (s)).

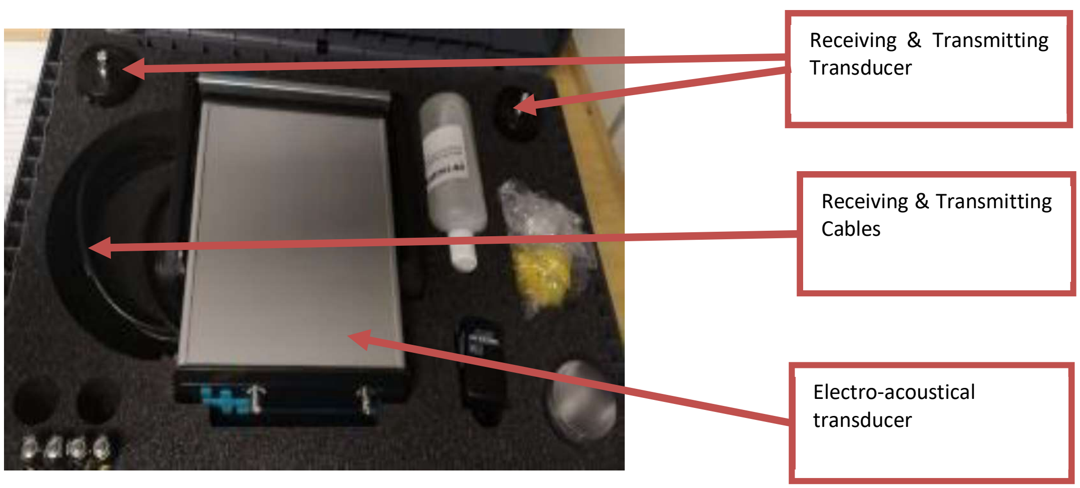

Figure 3.5 Electro-acoustical transducer (UPV instrument)

3.2.2.3 Rebound Hammer

The instrument referred to as the rebound hammer, also known by various names such as the Schmidt hammer, Swiss hammer, or concrete hammer test, quantifies concrete's surface hardness or strength. This instrument operates on the principle that the degree of rebound exhibited by a spring-loaded mass upon impact determines the surface hardness. The rebound hammer is versatile in its application and suitable for laboratory and field testing. Our study employed a digital rebound hammer device featuring key components such as the mechanical hammer housing (outer body), the plunger, the hammer mass, and the primary spring. Additionally, it incorporates the hammer lock button for securing the hammer mass to the plunger rod and an electronic interface equipped with a keyboard for displaying rebound measurements from the hammer mass. The recorded data is expressed in terms of rebound numbers. For a visual representation of the internal structure of a rebound hammer, please refer to Figure 3.6 (a), which provides a schematic section view from the inside of a rebound hammer (FPrimeC, 2019) as well as Figure 3.6 (b) photographs of the rebound hammer that was used for this research.

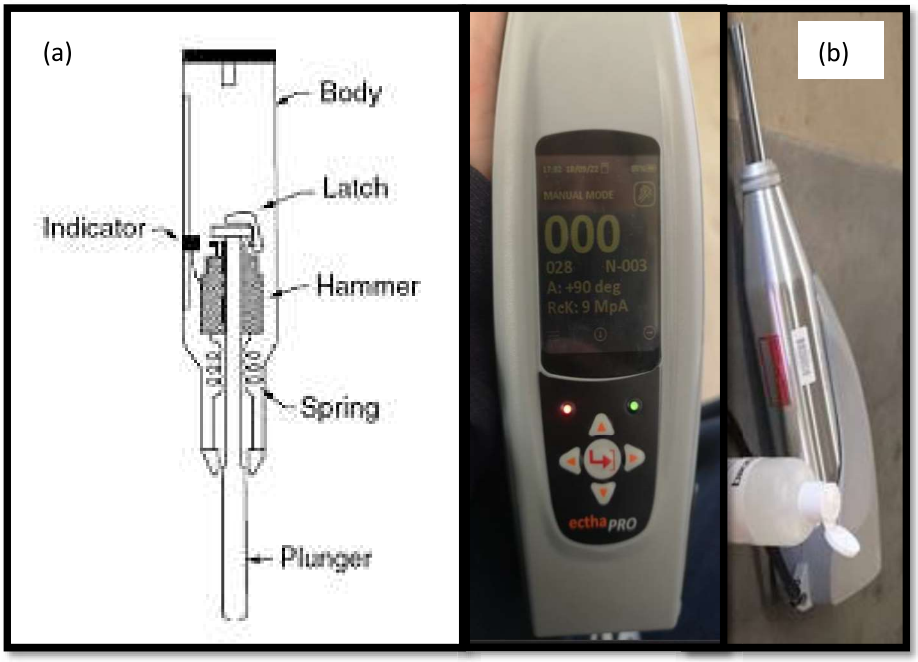

Figure 3.6 Rebound Hammer

3.2.2.4 Compressive strength machine

As shown in Figure 3.7, the compression testing machine was used to apply a pressure of 250 kN/min to the concrete specimens to measure the compressive strength of the concrete. Different machine sizes can crush different sizes of concrete specimens to a rate that the machine is calibrated. The machine used for this research can crush a maximum size of 150 mm specimens. The compression machine applies a load to the specimens without shock and increases the rate uniformly and continuously at a specific rate until the specimens fail. The maximum load applied before the failure of the specimen is recorded, as well as the appearance of the specimens and the type of failure.

Figure 3.7 Compressive strength test machine

3.2.3 Laboratory procedure

A 40-litre batch was used for each unique trial mixture. Concrete blends were prepared using a compact electric concrete mixer within the DWS laboratory. Precise measurements of all materials were taken prior to commencing each trial. The mixing process began by adding one-third of the water to the mixer drum, introducing coarse aggregate, fine aggregate, and all cementitious components. The plasticizer was blended with another one-third of the water and subsequently incorporated into the mix. The trial mixture underwent thorough mixing, and the remaining water was cautiously added. The study encompassed five distinct testing methods, as detailed below.

3.2.3.1 Moisture content

The moisture content of the material was calculated using the method specified by SANS 3001 GR20.
This is done to determine the moisture content of the fine aggregate to be used in the concrete mix. Furthermore, it is used before mixing the concrete to subtract the moisture content percentage in the fine aggregate from the total water content according to the mix design. This is done as the calculations for the mix design work on the dried mass for the fine aggregate and if the actual aggregate to be used is not dry. If this is not done, the total water content of the concrete mix will be more than the designed water content and thus will change the water-to-cement ratio of the concrete.

3.2.3.2 Vebe Test

This test is only done on the RCC trial mixes of fresh concrete.
The following process was followed following ASTM C1170M specifications to perform the Vebe tests:

Step 1: Follow the same slump method by placing the cone on the left side of the metal container, as shown in Figure 3.9.a.

Step 2: Remove the cone from the concrete sample.

Step 3: Determine the measurement in a slump by placing the transparent disc over the concrete sample and read the slump measurement on the rod graduated scale after recording the slump reading.

Step 4: If the record results are within specification, place the correct weight on the transparent disc and make sure everything is in place, as shown in Figure 3.9 b.

Step 5: Loosen the transparent disc screw so that the disc can easily slide down in the container but only rest on the concrete. Simultaneously, loosen the disc screw, switch on the vibrating table, and start the stopwatch simultaneously.

Step 6: Record the time it takes for the disc to move down on top of the concrete. Watch the concrete remoulding in the container until the disc is completely coated with cement grout, as shown in Figure 3.9 c and Figure 3.9.d.

Step 7: Stop the stopwatch and vibrating table and record the time to cover the disc with cement grout. According to the specification, this time should be between 10-25 seconds for RCC. An average time of 11 seconds was recorded for RCC 20/38-90, and an average time of 20 seconds was recorded for RCC 15/38-365.

Figure 3.8 Vebe tests done in the laboratory

3.2.3.3 *Making and Curing of Concrete Specimens*

The subsequent procedure adhered to the guidelines specified in SANS 5861-3 for the fabrication and curing of test specimens:

The concrete specimens employed in this research were 150 mm x 150 mm x 150 mm square concrete cube specimens selected based on their intended application. These specimens were instrumental in evaluating both ultrasonic pulse velocity and compressive strength.

Once the Vebe test on the fresh concrete yielded acceptable results, the remaining trial mixture was allocated to constructing concrete specimens. Nine specimens were produced, with three sets per trial mix, aligning with the stipulations of SANS standards. The square 150 mm x 150 mm x 150 mm moulds, crafted from cast iron, were utilised to ensure zero grout loss and to withstand the compaction of concrete specimens on the vibrating table. A visual representation of some of these prepared specimens can be observed in Figure 3.10.

It is imperative to adhere to the specifications outlined in SANS 5860 when embedding concrete specimens to prevent undesired dimension tolerances, deviations in specimen squareness, and the loss of water or grout during the specimen creation process.

The meticulous creation of concrete specimens holds significant importance, as it directly impacts the resultant compressive strength of the specimens. Any deviations, such as non-square moulds or incorrect concrete compaction, can lead to aberrant crushing values for a given specimen.

Figure 3.9 Cube specimen moulds prepared for concrete.

As shown in Figure 3.11 for the RCC trial mixes, all the specimens were compacted by using a vibration table and compaction weight. The conventional concrete was compacted using a small poker vibrator. This compaction method should be applied for the minimum duration whilst making the specimens. Moulds should be filled with three different layers. The required duration of vibration will depend on the workability of the concrete and the strength of the vibration machine. Vibration should only be applied until no large air bubbles are released and the surface of the concrete is relatively smooth with a shiny appearance. After the concrete specimens were float finished, the specimens were set aside to settle for three days in a curing area covered with a damp-proof material. Specimens were clearly marked and grouped separately after each trial mix, as shown in Figure 3.12.

Figure 3.10 Compaction of RCC done on vibration table.

Figure 3.11 Marking and grouping of specimens

3.2.3.4 Curing of specimens in a water bath

After three days, the concrete specimens were carefully removed from the steel moulds and placed in a water curing bath to cure for 7, 14 and 28 days in accordance with SANS 5861-3. Figure 3.13 shows the curing of concrete specimens in the water bath.

After removing the specimens from their moulds, it was observed that some specimens exhibited a change in granular characteristics compared to others. A more comprehensive analysis of this discrepancy will be provided in Chapter 4 and Chapter 5.

It is paramount to exercise caution during the extraction of concrete specimens from their moulds and their subsequent curing process. Any inadvertent damage sustained during these phases can potentially compromise the strength and durability of the specimens, rendering them unsuitable for further testing.

Figure 3.12   Specimens in curing bath

When testing concrete specimens, the specimens should still be saturated with only the surface moisture dried with a cloth. All specimens need to be weighed, measured, and inspected for any physical damage on the specimen surfaces before any test is conducted on the specimens.

3.2.3.5 Ultrasonic pulse velocity

Before performing UPV measurements, the UPV machine was calibrated according to manufacture specifications. The UPV measurements were performed with a digital pulser-receiver unit, where the data collected stated the transmission time of the pulse application, the received signal strength and the pulse velocity in P-wave measured. The voltages and transducer frequency settings were on auto.

The UPV test was conducted on all trial mix specimens after 7, 14 and 28 days of curing. The following process was followed following BS EN. 12504-4 standard specifications for conducting a UPV test:

Step 1: Specimens are measured and marked for position to place the transducers.

Step 2: Determine the transducer arrangement; direct transmission was used for the project.

Step 3: Measure the path length and put that distance into the Electro-acoustical transducer.

Step 4: The coupling agent, Petroleum Jelly, is applied on the transducer's faces and the concrete test surface. This is done to ensure good acoustical contact.

Step 5: Place the transducers firmly against the concrete surface directly opposite each other for direct transmission, in a straight line between the centres of the transducer face.

Step 6: Start the machine to take readings. Take readings until the minimum pulse velocity reading is obtained with the highest received signal level. Note down the transmission time. Figure 3.14 indicates the readings taken.

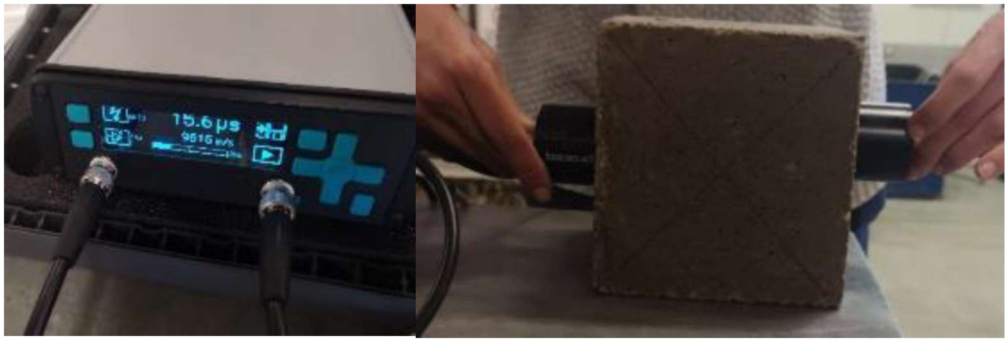

Figure 3.13 UPV test done on a specimen

3.2.3.6 Rebound Hammer

The Rebound hammer test was conducted on all trial mix specimens after 7, 14 and 28 days of curing. The following process was followed in accordance with BS EN. 12504-2 specifications to conducting a Rebound hammer test:

Step 1: Concrete specimen is placed in the compressive press machine and pressed to 1 N.

Step 2: The rebound hammer is set-up to take impact readings horizontally.

Step 3: The rebound hammer is held perpendicular to the flat and smooth surface, as shown in Figure 3.15.

Step 4: A minimum of 5 impact readings is taken on this surface, with none of the impact points closer than 25 mm from each other or the edge of the concrete specimen.

Figure 3.14 Top view from Rebound hammer test done on a specimen in compressive strength machine

3.2.3.7 Compressive Strength Test

A compressive strength test was done for all trial mixes following SANS 5863:2006.

The test specimens consist of 150 mm x 150 mm x 150 mm concrete specimens cured underwater in a water bath for 7, 14 and 28 days. Prior to the testing, the mass of each specimen was determined. Each specimen was tested while still saturated. During testing, a compression load was applied to each specimen without shock at 250 kN/min to failure.

The maximum load applied at which each specimen failed was recorded. The type of failure is also identified and noted. After that, the compressive strength of each specimen was calculated. The average compressive strength per set was then calculated to the nearest 0,5 MPa and checked if the highest and lowest values of the different specimens per set did not exceed 15% of the average as per SANS 5863 guidance. The average result was then obtained as the compressive strength for that number of curing days and trial mix. Figure 3.16 illustrates a specimen under pressure testing.

Figure 3.15 Compressive strength test

3.2.4 Field procedure

A site visit was conducted at two locations; the De Hoop dam in Limpopo and the Spring Grove dam in the Midlands of KwaZulu-Natal, situated northwest of the Mooi River. During this visit, non-destructive testing (NDT) was exclusively carried out on the sections of the dam walls that were accessible without overflows.

At the De Hoop dam, assessments involving Ultrasonic Pulse Velocity (UPV) and Rebound Hammer (RH) measurements were limited to the downstream non-overspill left embankment. In contrast, UPV and RH measurements were conducted at the Spring Grove dam on both the left and right downstream non-overspill embankments, as both were accessible for inspection and data collection.

3.2.4.1 Ultra sonic Pulse Velocity

Before conducting the Ultrasonic Pulse Velocity (UPV) measurements, the UPV machine was calibrated in strict accordance with the manufacturer's specifications. The UPV measurements were executed utilising a digital pulser-receiver unit, capturing data that included the transmission time of the pulse application, the received signal strength, and the measured pulse velocity. The settings for voltages and transducer frequency were set to automatic mode. When assessing the surface of the dam walls, solely indirect measurement methods were feasible. For the downstream steps of the dam walls, a semi-indirect approach was the preferred method, followed by an entirely indirect method.

The UPV test was administered on the non-overspill sections downstream of both dam walls. This procedure adhered to the guidelines outlined in the BS EN 12504-4 standard, using semi-direct and indirect methods, as indicated in Figure 3.17 and Figure 3.18 for conducting UPV tests. This procedure was the same as the procedure followed in the laboratory.

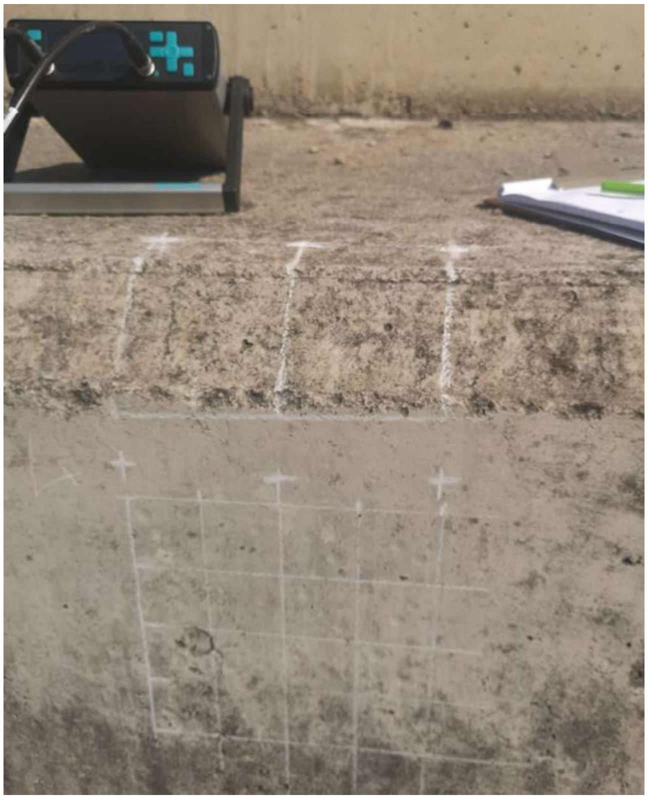

Figure 3.16 Identified position, labelled and sizes measured and mark-out for recordings

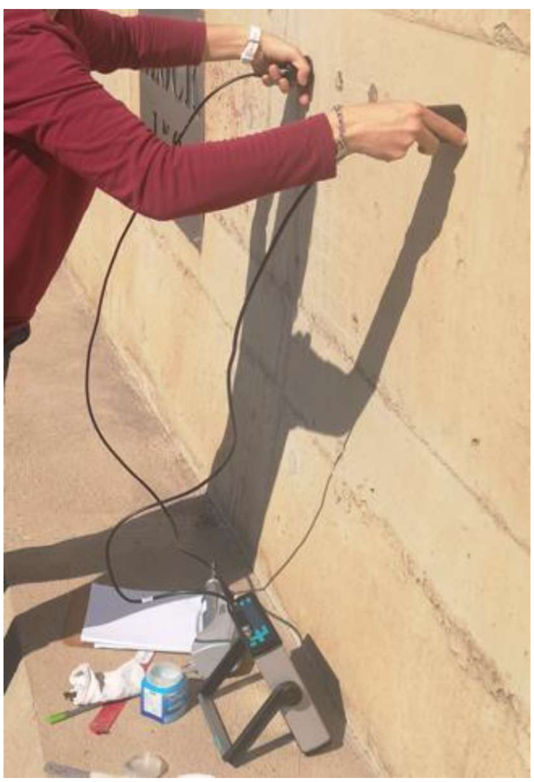

Figure 3.17 Indirect UPV measurement taken at De Hoop Dam wall

3.2.4.2 Rebound Hammer

The Rebound hammer test was carried out at identical locations where UPV measurements were previously taken on the dam structures. Specifically, a horizontal (0-degree) measurement method was employed for the top section of the dam walls, while a vertical downward (+90-degree) measurement method was utilised for the steps.

This testing process was executed strictly per the specifications delineated in BS EN 12504-2 and BS EN 13791:2007 for conducting a Rebound hammer test. This procedure was the same as the procedure followed in the laboratory as shown in Figure 3.18 and Figure 3.19.

Figure 3.18 RH test square marked out on dam wall

Figure 3.19 Rebound hammer test done on a dam wall in horizontal or 0-degree position

3.3 Optimisation of laboratory experimental base mixes

The mix design obtained from De Hoop and Spring Grove RCC was used as the basis to calculate two mixed designs in accordance with the RCC engineer manual (USACE, 2000) and Department of Water and Sanitation Standards for RCC dam specifications (Department of Water and Forestry, 2005). Four different mixes for each base mix design were prepared, with the major variant being the replacement of cement with an extender of fly-ash with 25% increments for each base mix. The control mix for each mixture made was 100% cement content. The actual mixing in the laboratory was batches of 0,04 m^3.

The original mix design for mixture one was based on the mix design used to construct the De Hoop dam. This mix was designed to reach 20 MPa at 90 days using a maximum coarse aggregate size of 38 mm, 70% FA and 30% Cement content, and a water-cement ratio of 058. It must be noted that a different type of aggregate was used for this research project than for the dam's construction. After the optimised mix was achieved, a 20 MPa RCC mix was designed, with a water cement ratio of 0,64 using a total cementitious content of 207 kg/m^3; fine and coarse aggregate content was kept constant at 877 kg/m^3 and 1366 kg/m^3 respectively. The mixture is named 20/38-90. Four different trial mixtures were done by systematically increasing the FA content and decreasing the cement content in the mix. The different FA replacement was done at 25 % intervals. The first trial mixture with 0% FA and 100% Cement was the control mixture and represented 20/38-90 FA0%. The second trial mixture with 25% FA and 75% cement was named 20/38-90 FA25%. The third trial mixture with 50% FA and 50% cement is named 20/38-90 FA50%. This mixture's fourth and last trial mixture is 75% FA and 25% cement, named 20/38-90 FA 75%.

For the second mix, the original mix design was based on the RCC mix used for the Spring Grove dam construction. This mix design reached 15 MPa at 365 days, with 68% of the total cementitious content made up by FA using a 38 mm maximum size coarse aggregate, with a 0.74 water cement ratio. A different aggregate type was used in this research project than for dam construction. After the optimised mix was achieved, 15 MPa RCC mix was designed, with a water-cement ratio kept between 0.7 to 0.84, using a total cementitious content of 160 kg/m^3; fine and coarse aggregate content was kept constant at 1200 kg/m^3 and 1100 kg/m^3 respectively. The mixture is called 15/38-365. Four different trial mixtures were made with variations between FA and cement ratio. The first trial mixture had 0% FA content and 100% cement, named 15/38-365 FA0%, used as the control mixture. The second trial mixture was 25% FA and 75% cement, 15/38-365 FA25%. The third trial mixture and 50% FA and cement, 15/38-365 FA 50%. The last trial mixture had 75% FA and 25% cement, 15/38-365 FA75%.

Due to the difference in total cementitious materials in the base mixture designs, the total volume of water (thus the water-cement ratio) and quantities of fine and coarse aggregate, the aggregate ratio for both these base mixtures differs. Changing what looks like one small thing in a concrete mix can result in a totally different mix with different results. The two design mixtures prepared in the laboratory are shown in Table 3.1 and Table 3.2. For each of the two mixtures, four (4) mixes were made, with the 100% cement content mixes being the control mix.

Table 3.1 RCC design 20/38-90 mix proportions for 0,04 m^3

Material (/0,04m^3)	Experiment Mix Number			
	20/38-90 FA0%	**20/38-90 FA25%**	**20/38-90 FA50%**	**20/38-90 FA75%**
Cement (kg)	8,28	6,21	4,14	2,07
FA (kg)	0	2,07	4,14	6,21
20 mm aggregate (kg)	51,91	51,91	51,91	51,91
10 mm aggregate (kg)	2,73	2,73	2,73	2,73
5 mm aggregate (kg)	10,52	10,52	10,52	10,52
River sand (kg)	24,56	24,56	24,56	24,56
Admixture (ml)	40	40	40	40
Water (l)	5,35	5,3	5,25	5,3

Table 3.2 RCC design 15/38-365 mix proportions for 0,04 m^3

Material (/0,04m^3)	Experiment Mix Number			
	15/38-365 FA0%	**15/38-365 FA25%**	**15/38-365 FA50%**	**15/38-365 FA75%**
Cement (kg)	6,4	4,8	3,2	1,6
FA (kg)	0	1,6	3,2	4,8
20 mm aggregate (kg)	39,6	39,6	39,6	39,6
10 mm aggregate (kg)	4,4	4,4	4,4	4,4
5 mm aggregate (kg)	14,4	14,4	14,4	14,4
River sand (kg)	33,6	33,6	33,6	33,6
Admixture (ml)	40	40	40	40
Water (l)	5,4	5,2	4,6	4,6

Aggregates grading

The grading analysis for the fine and coarse aggregates used for this project can be found in Figure 3.1 and Figure 3.2, respectively, in Chapter 3. All the different experimental trial mixes were conducted by using the same fine and coarse aggregate to ensure uniformity. The grading of the fine and coarse aggregate size distribution and the fines modules is important in any concrete design approach but is even more important for the design and use of RCC.

Replacement of cement with Fly Ash

The cement was replaced in increments of 25% volume by Fly-ash of the cementitious material. The weight of cementitious materials, plasticiser and aggregate was kept constant for the two base mix designs. The volume of fly-ash is expressed as a percentage of the cementitious material. The testing was done with 0, 25%, 50% and 75% fly-ash composite.

It was noted that with cement replacement with fly ash, less water might be needed and the consistency of each mix changes. The 50% fly-ash replacement trial mixes have the shortest Vebe time recorded and less water used for 20/38-90 and 15/38-365, respectively.

3.4 Conclusion

Laboratory experimental tests and field assessments were conducted for this research project. The mixture designs used in this research were aligned with the US Army Corps (US Army Corps of Engineers, 2000) and the Department of Water and Sanitation's RCC standard (Department of Water and Forestry, 2005).

Two high cementitious RCC mixtures with different quantities of total cementitious materials were designed and batched during the laboratory experiments. For each of the two main mix designs, four variants of mixtures were made with varying ratios of FA to cement. Vebe testing was conducted on each of these different mixtures to determine the workability of the RCC based on consistency. After that, a total of 36 concrete specimens were compacted and left to cure for 7, 14 and 28 days of age.

For each base mix type, cement was replaced with FA at the same contents of 25%, 50% and 75%, respectively. Due to the different total cementitious materials used for the two base mix designs, the aggregate quantity and water quantity needed differ as per the mixture design process. Therefore, the water-cement ratio quantities differed as well. The water-cement ratio for base mix design 20/38-90 was 0.64, and for base mix design 15/38-365 was between 0.7 and 0.84.

DT and NDT techniques were used for laboratory experimental testing to establish the relationship between compressive strength, pulse velocity, and rebound hammer index. De Hoop and Spring Grove dams were selected for field assessment due to their IVRCC construction, similar to the RCC concrete in laboratory experiments. NDT measurements were executed on downstream non-overspill embankment sections.

Data from these methods were methodically recorded and subsequently processed. These comparisons of the results obtained with compressive strength (MPa), UPV and Rebound Number (RN) will be presented in the following chapter.

Chapter 4 Experimental Results of RCC

This chapter offers a comprehensive overview of the results obtained through laboratory experiments and field tests. In the laboratory setting, two distinct bases Roller Compacted Concrete (RCC) mixture designs were used as experimental designs. This led to the formulation and testing of eight trial mixtures, resulting in 36 specimens subjected to rigorous examination.

Consistency was maintained across all 36 specimens, with uniform testing methods applied to each, encompassing Ultrasonic Pulse Velocity (UPV), Relative Hammer (RH), and compressive strength assessments. Furthermore, the field tests conducted at the De Hoop dam and Spring Grove dam walls focused on their downstream non-overspill sections, using the same UPV and RH evaluation methods.

4.1 Laboratory experiment conformity to RCC standards

In the context of the two distinct design mixtures, it is worth noting that the Fly Ash (FA) content exhibited variation, while all other pertinent parameters remained consistent. In the case of the 15/38-365 mixture, it is important to highlight the differentiation in water content.

Table 4.1 presents the precise mixture design proportions following the guidelines specified by the US Army Corps of Engineers manual for Roller Compacted Concrete (RCC) design, the standards the Department of Water Affairs and Forestry for RCC set forth, and the relevant FULTON guidelines. These tables also provide a comprehensive breakdown of the constituent components for both design mixtures, offering a detailed insight into the composition of these RCC formulations.

Table 4.1 RCC standards and guidelines and the trial mixes overview

Constituent	Standards and guidelines			Actual trial mixtures	
	US Army Corps of Engineers RCC Manual	**Department of Water and Forestry, DWS 0740**	**FULTON**	**20/38-90 RCC mix design**	**15/38-365 RCC mix design**
	Typical range by mass kg/m^3	Typical range by mass kg/m^3	Typical range by mass kg/m^3	Range by mass kg/m^3	Range by mass kg/m^3
Total cementitious material	120-200	NA	195	207	160
Water	107-140	NA	111	132	112-134
W/C Ratio	NA	0.5-0.7	0.57	0.64	0.7-0.84
S/A ratio	0.32-0.49	0.3-0.45	0.3-0.47	0.64	1
P/M ratio	0.31-0.56	0.36	0.37-0.45	0.38	0.4
Vebe Time	12-25 sec	10-25 sec	10-20 sec	7-15 sec	11-20 sec

The mixture design for RCC dams largely adhered to the standards and guidelines set forth in the RCC Engineering Manual (USACE, 2000) and Department of Water and Sanitation Standards for RCC dam specifications (Department of Water and Forestry, 2005). However, there was one exception, which is mixture 15/38-365, where the w/c ration and s/a ratio are higher that the maximum ratio guidance.

The schematic representation of the laboratory experimental procedure is clarified in Figure 4.1. This chapter's subsequent sections explain the outcomes obtained for each specific concrete mix, encompassing Ultrasonic Pulse Velocity (UPV), rebound hammer, and compressive strength examinations.

Furthermore, for in-depth analysis and reference, the comprehensive laboratory data concerning UPV assessments is precisely documented in Appendix D. Likewise, the corresponding rebound hammer results are documented in Appendix E, and the detailed records of compressive strength evaluations are presented in Appendix F. These supplementary appendices forms a comprehensive repository of the raw data, enabling further scrutiny and reference for interested readers.

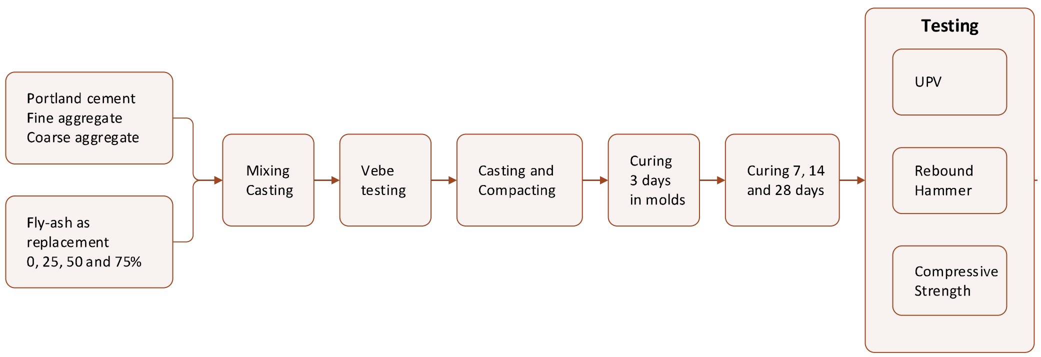

Figure 4.1 Laboratory experimental process followed flow chart.

Potential sources of laboratory experimental errors encompassed issues such as anomalies in compressive strength readings and rebound hammer assessments. Additionally, the susceptibility to inadvertent damages during the extraction of specimens from moulds, coupled with the challenges associated with their transportation to the CPUT laboratory, were acknowledged. The impact of weathering and fluctuating curing bath temperatures during the 28-day ageing period also introduced the possibility of minor variations in recorded measurements among specimens. In contrast, the potential for field test errors included variations in rebound hammer and Ultrasonic Pulse Velocity (UPV) readings, influence from environmental factors like heat, connectivity issues with the concrete, and limited access to dam walls for data collection. The potential for human error during experimental procedures also contributed to measurement discrepancies. Utmost care has been taken to minimise these errors in the present study.

Furthermore, an obvious contrast was observed in the granular characteristics of specimens from the 15/38-365 mixture compared to the 20/38-90 mixture during the demolding process, as visually depicted in Figure 4.2. Subsequently, these specimens exhibited signs of abrasion over the curing duration, resulting in a loss of smoothness, particularly evident in the 28-day cured specimens of the 15/38-365 FA50% and 15/38-365 FA75% compositions. This corrosion phenomenon, illustrated in Figure 4.3 between day 14 and day 28 of ageing, could be attributed to several factors, including an elevated volume of fine aggregate, a high concentration of fines in the mixtures, a substantial presence of fly ash, a relatively high water-to-cement (w/c) ratio, an excessive aggregate ratio, or over compaction of the specimens, among other potential contributors. Consequently, it is strongly recommended that a thorough re-evaluation of the 15/38-365 mixture design be undertaken.

Figure 4.2 Specimens from mixture 15/38-365 and 20/38-90 at day 3 of age

Figure 4.3 Specimens for mixture 15/38-365 FA75% at day 14 and day 28 of age

4.2 Ultrasonic pulse velocity laboratory test results

The following section shows UPV results for the different trial mixes on 7, 14 and 28 days. Direct velocity measurements were taken at 2 locations on each specimen and recorded. The direct path length for these measurements was through 150 mm specimen thickness. The measurements were taken in meters per second but converted to kilometres per second for analysis and comparison of results.

4.2.1 Mixture 20/38-90 UPV test results

The lowest recorded measurement from the UPV test results for various 20/38-90 mixtures' specimens consistently exceeded 4.0 km/s, as depicted in Figure 4.4. According to the criteria outlined by the International Atomic Energy Agency (IAEA) in 2002, as provided in Table 4.2, this corresponds to concrete of good quality. Analysis of Figure 4.4 reveals the following trends:

- In the case of the 20/38-90 FA0% mixture with 100% cement content (indicated in blue), the average UPV results using the direct method increased by 1.5% from 7 days to 14 days, and the average remained constant between days 14 and 28.

- For the 20/38-90 FA25% mixture, where 25% of cement was replaced with fly ash (FA), the results decreased by 0.4% from day 7 to 14 and decreased by 23.6% from 14 to 28 days.
- The 20/38-90 FA25% mixture with 75% cement content exhibited higher overall results than the 20/38-90 FA0% mixture with 100% cement content.
- In the case of the 20/38-90 FA50% mixture, where 50% of cement was replaced with FA, the results decreased by 2.1% from 7 days to 14 days and further decreased by 10.8% from 14 days to 28 days of age.
- The 20/38-90 FA50% mixture with 50% cement content yielded lower results than the 20/38-90 FA25% mixture with 75% cement content.
- The 20/38-90 FA75% mixture, with 75% cement content replaced with FA, displayed a 1.7% increase in results from 7 to 14 days and an additional 2.05% increase at 28 days of age.
- The 20/38-90 FA75% mixture with 25% cement content resulted in lower values than the 20/38-90 FA50% mixture with 50% cement content and recorded the lowest results among all trial mixes in the type A RCC mix design group.

The 20/38-90 FA0% mixture exhibited an overall increase, the 20/38-90 FA25% mixture experienced a decrease over time, the 20/38-90 FA50% mixture also exhibited a decrease over time, and the 20/38-90 FA75% mixture displayed an increase over time.

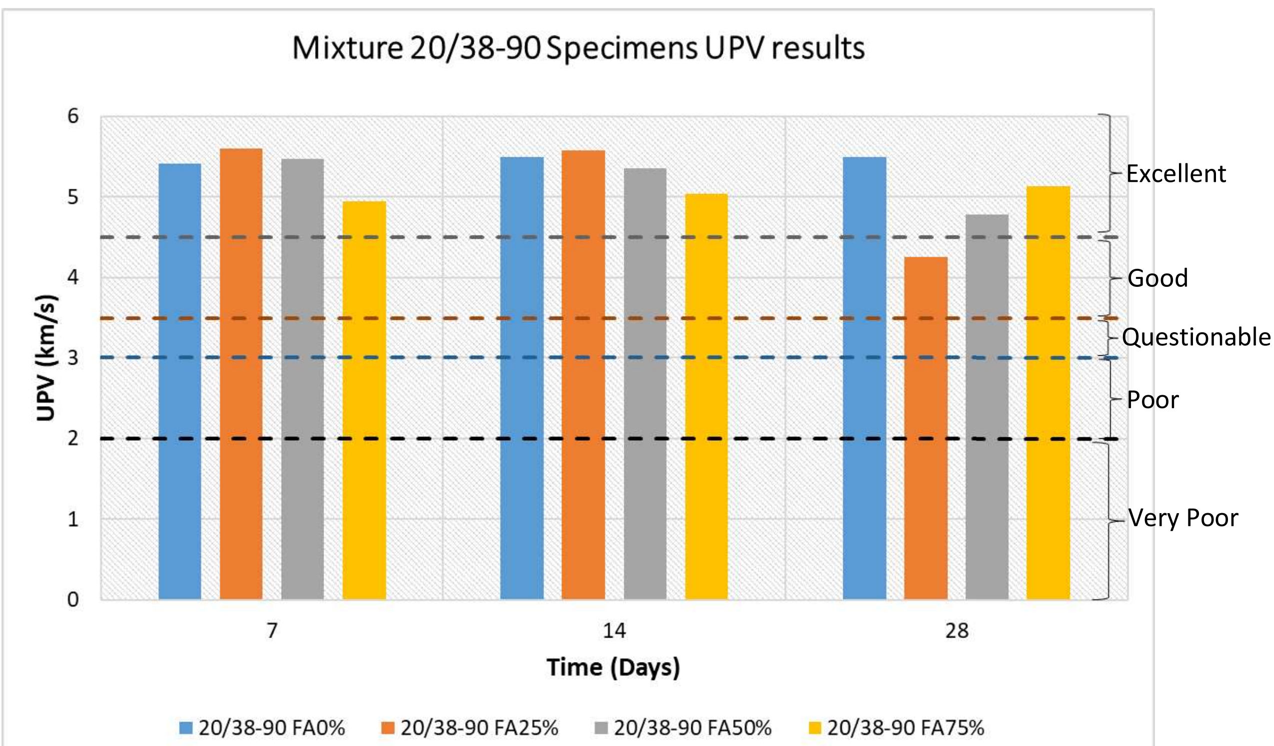

Figure 4.4 The direct transmission UPV results recorded for RCC mix 20/38-90 from 7 to 28 days.

4.2.2 Mixture 15/38-365 UPV test results

The UPV test results indicate that the lowest recorded values for the mixtures 15/38-365 FA0%, 15/38-365 FA25%, 15/38-365 FA50%, and 15/38-365 FA75% were consistently above 3.4 km/s, as illustrated in Figure 4.5. By referencing Table 4.2 and the findings presented in Figure 4.5, it can be affirmed that the concrete quality of the specimens is generally of high quality. Analysis of Figure 4.5 yields the following insights:

- For the 15/38-365 FA0% mixture, the UPV test results showed a 4.9% increase from 7 to 14 days, followed by a decrease of 4.97% from 14 to 28 days.
- In the case of the 15/38-365 FA25% mixture, there was a 4.08% increase in UPV values from 7 to 14 days, followed by a decrease of 4.9% between days 14 and 28.
- The 15/38-365 FA50% mixture exhibited a 2.9% increase in UPV values from 7 to 14 days, and a subsequent increase of 4.54% was observed at the 28-day mark.
- In the case of the 15/38-365 FA75% mixture, UPV results showed a substantial 17.66% increase from 7 days to 14 days. However, there was a minor 0.27% decrease in results between days 14 and 28.

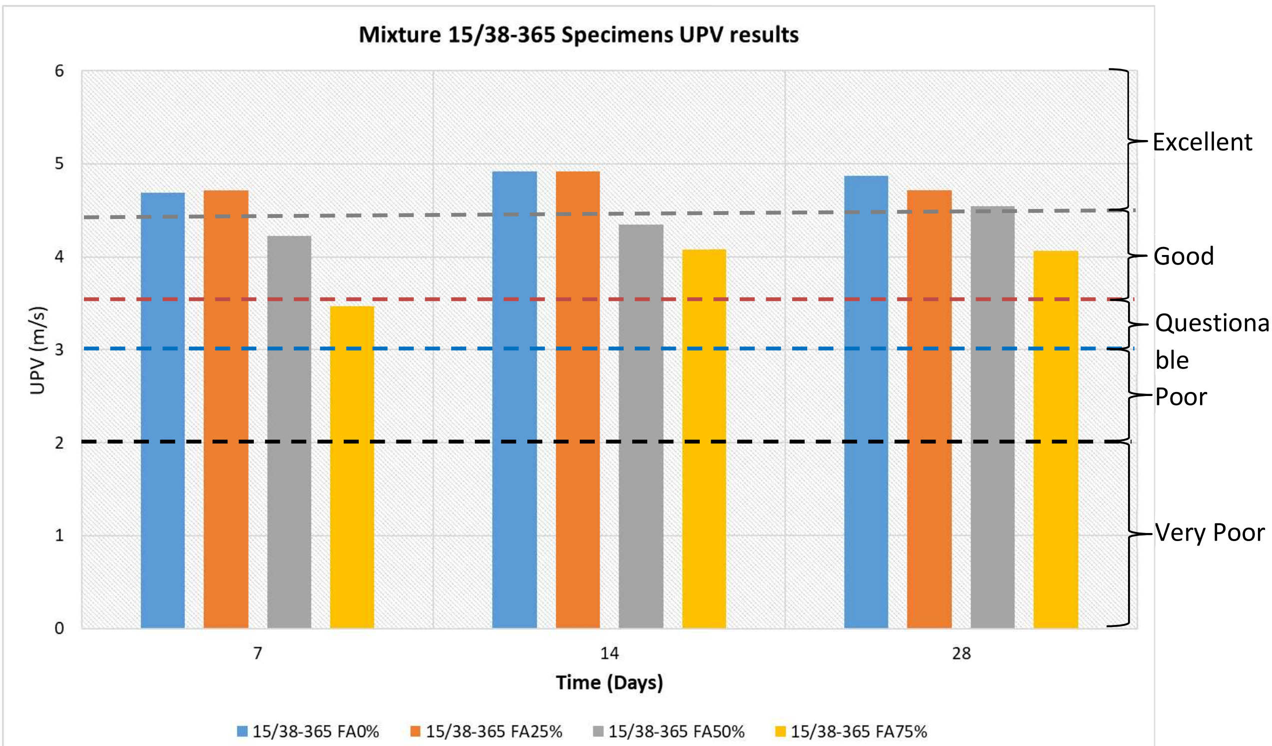

Figure 4.5 The direct transmission UPV results recorded for RCC mix 15/38-365 from 7 to 28 days

4.2.3 Both mixture 15/38-365 and mixture 20/38-90 UPV test results conformity

Direct UPV's obtained through the specimens are presented in Table 4.3 and in Figure 4.6 which represent the average measurement recorded per day age group specimens. UPV measurement is an in-line concrete quality classification as shown in Table 4.2 as per International Atomic Energy Agency (2002).

Table 4.2 Classification of the quality of concrete on the basis of ultrasonic pulse velocity results

GUIDELINES		ACTUAL TRIAL MIXTURE	
International Atomic Energy Agency (2002)		Mixture 20/38-90	Mixture 15/38-365
Quality of Concrete	UPV (km/s)	Overall UPV average (Km/s)	Overall UPV average (Km/s)
Excellent	> 4,5		
Good	3,5 to 4,5	Above 4,0	Above 3,4
Questionable (Slight porosity may exist)	3,0 to 3,5		
Poor (Loss of integrity is suspected)	2,0 to 3,0		
Very Poor (Loss of integrity exist)	<2,0		

Table 4.3 Summary of all different mixtures UPV results

Average Ultra-sonic Pulse Velocity Direct Method Readings (km/s)								
DAYS	**20/38-90 FA0%**	**20/38-90 FA25%**	**20/38-90 FA50%**	**20/38-90 FA75%**	**15/38-365 FA0%**	**15/38-365 FA25%**	**15/38-365 FA50%**	**15/38-365 FA75%**
7	5,588	5,726	5,587	5,148	4,917	4,996	4,374	3,611
14	5,644	5,706	5,462	5,168	5,224	5,380	4,543	4,282
28	5,590	5,327	5,345	5,261	5,409	4,955	4,743	4,253

For mixture 20/38-90 FA0%, 20/38-90 FA25%, 20/38-90 FA50% and 20/38-90 FA75% in Table 4.3 all the results were between 4,2 km/s and 5,6 km/s. Therefore, if referenced back to Table 4.2, it is considered good and excellent quality concrete. However, the results for mixture 20/38-90 FA0% don't increase with age but stay the same. This can be due to different factors, of which a few possible factors will be discussed. The problem could occur with the compaction on the specimens. The moisture content and curing condition could have influenced the UPV test reading. This mixture was also mixed with 100% cement content, and it is known that it reaches full strength in 28 days. Therefore, the cement gel formed by the cement particles busy reacting with water might have influenced the waves. For the 20/38-90 FA25% and 20/38-90 FA50% mixtures, the results decrease over the ageing of the specimens, mix 20/38-90 FA25% results for 28 days can be seen as outlier. The possible reasons for the results decreasing over time are as follows: the cement type, in the form of the combination of cement with fly-ash, where the fly ash starts to react with the cement after some time, and therefore forming a cement gel that could influence the waves path length. The moisture condition and the curing of the specimens could also influence the UPV measurements. For mixture 20/38-90 FA75% with 75% fly ash and 25% cement content, the UPV measurements increase over time. Mixtures with fly ash will reach higher compressive strength later than concrete mixes with only a cement content (Thomas, 2007). Therefore, with the low cement content, the concrete mixture will take longer than 28 days to reach its full compressive strength and a slight increase in UPV measurement over the ageing period.

For mixture 15/38-365 FA0%, 15/38-365 FA25%, 15/38-365 FA50% and 15/38-365 FA75% UPV measurement recorded in Table 4.3 is between 3,4 km/s and 4,9 km/s, thus can be taken as questionable, good and excellent concrete quality as per Table 4.2. Mixture 15/38-365 FA0% and 15/38-365 FA50% UPV measurements increase over the age period as was expected, as the cement reach to reach strength over time. Mixture 15/38-365 FA25% and 15/38-365 FA75% results increase from 7 days to 14 days, which is expected. However, it decreases to 28 days UPV results. This could be influenced by various factors like the compaction of the specimens, the curing of the specimens and moisture of the specimens, as well as the temperature of the specimens when measured. Mixture 15/38-365 FA75%, day 7 results are questionable and can be due to the high fly-ash volume and low cement content, and the specimen still in the curing stage for the specimen was still very weak, or there could be compaction and voids in specimens could have led to the low results recorded in these specimens to name a few possible reasons. Figure 4.6 line graphs also show that the lower the cement content, the lower the UPV results.

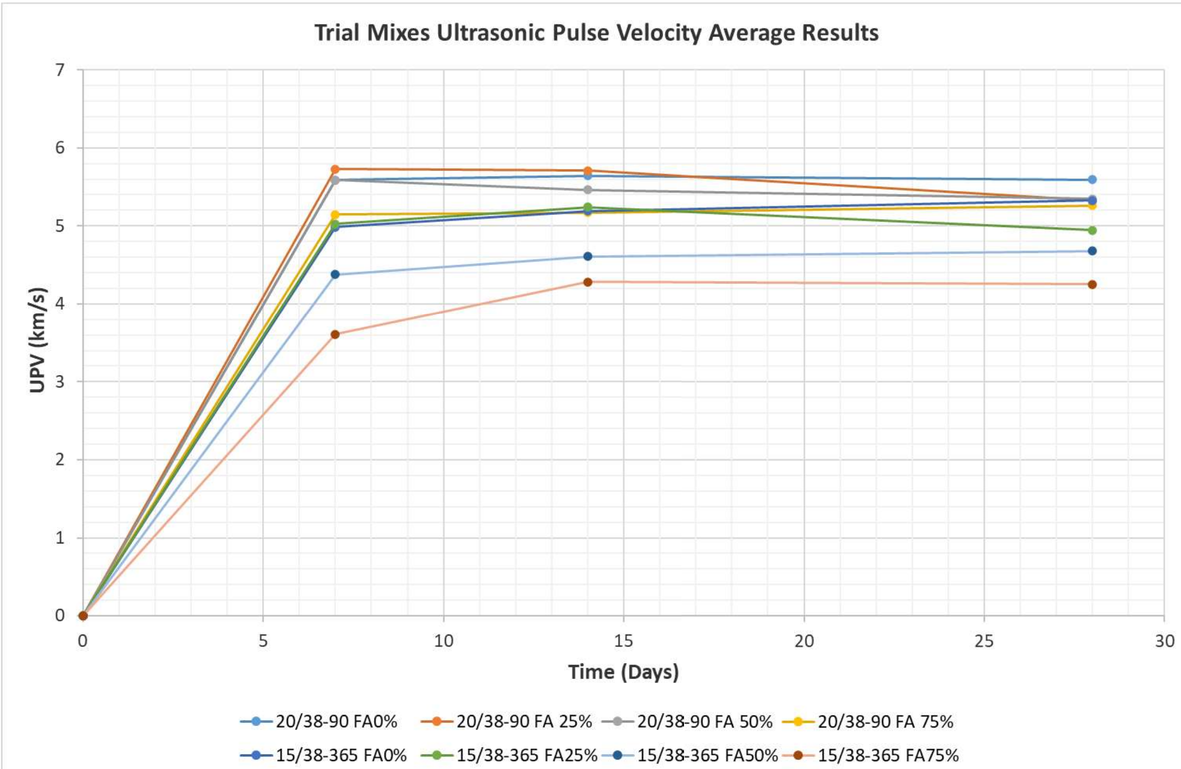

Figure 4.6 Relationship between ultrasonic pulse velocity and different cement content over period of 7 to 28 days

4.3 Rebound hammer laboratory test results

This section shows the rebound hammer results measured and recorded in rebound numbers for the different trial mixes on 7, 14 and 28 days. Each specimen cast from each trial mixes was tested by positioning the Rebound Hammer perpendicular to the surface of the specimen. This is termed the horizontal 0 degrees test method. At least 5 rebound hammer measurements were taken 25 mm apart on each specimen and recorded.

4.3.1 Mixture 20/38-90 rebound hammer test results

It is evident that all the results obtained from the rebound hammer tests conducted on various mixtures, namely 20/38-90 FA0%, 20/38-90 FA25%, 20/38-90 FA50%, and 20/38-90 FA75%, exhibited rebound index values exceeding 15, as illustrated in Figure 4.7. Notably, these results remained relatively consistent at the 7-day curing stage. However, significant differences emerged at the 28-day curing stage. Analysing the data from Figure 4.7, the following observations can be made:

- For the 20/38-90 FA0% mixture, the rebound index results increased by approximately 20.97% between the 7th and 14th days and remained constant between the 14th and 28th days.
- In the case of the 20/38-90 FA25% mixture, there was a notable increase of 29.4% in rebound index values from the 7th to the 14th day, followed by a subsequent decrease of 9.09% from the 14th to the 28th day.
- For the 20/38-90 FA50% mixture, the rebound index results demonstrated a 31% increase from the 7th to the 14th day, and then they remained stable between the 14th and 28th days.
- In the 20/38-90 FA75% mixture scenario, the rebound index results experienced a 19% increase from the 7th day to the 14th day and maintained a consistent level between the 14th and 28th days. Notably, all rebound number results for the 20/38-90 FA75% mixture remained below 20.

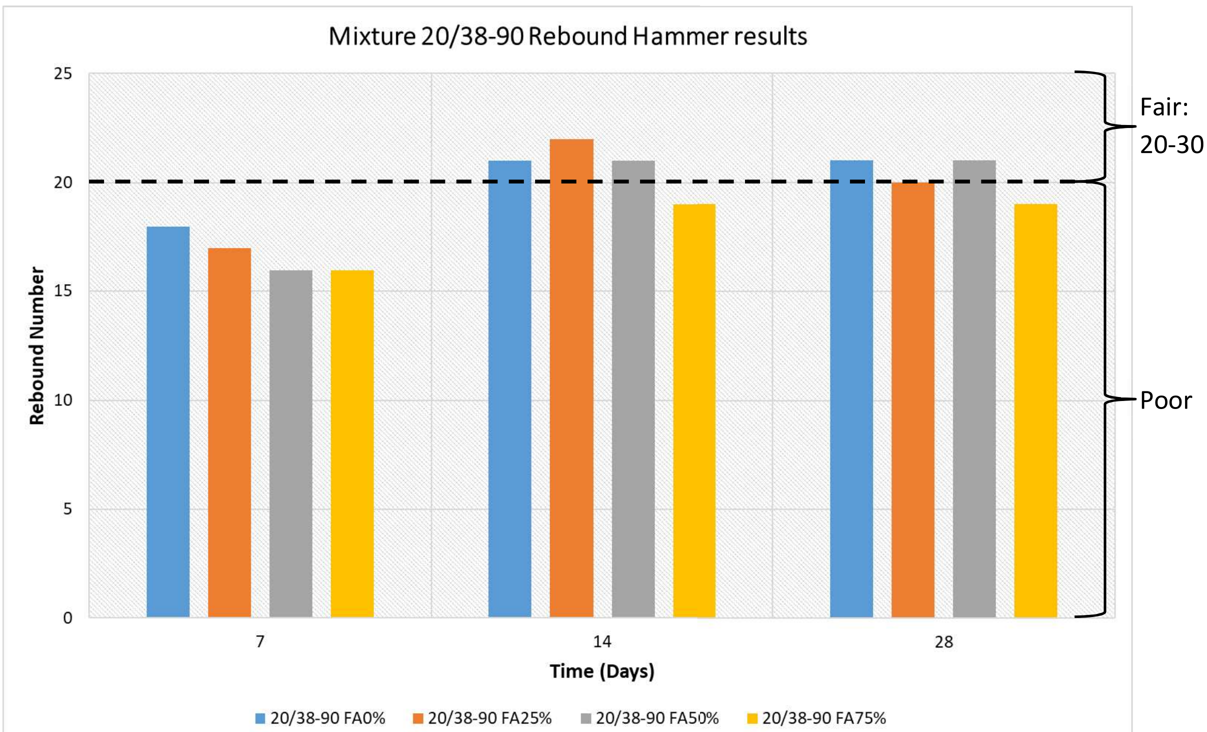

Figure 4.7 The rebound number for RCC mix 20/38-90 from 7 to 28 days

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4.3.2 Mixture 15/38-365 rebound hammer test results

All the rebound hammer test results obtained for the mixtures 15/38-365 FA0%, 15/38-365 FA25%, 15/38-365 FA50%, and 15/38-365 FA75% exhibited values exceeding 15, as depicted in

- Figure 4.8. Notably, these results remained fairly consistent during the 7-day curing period. However, distinctions in results emerged after 28 days of curing. Upon scrutinizing the data presented in

Figure 4.8, the following conclusions can be drawn:

- For the 15/38-365 FA0% mixture, the rebound index results displayed an approximate 10.5% increase between the 7th and 14th day, followed by a subsequent decrease of 4.76% from the 14th to the 28th day.
- In the case of the 15/38-365 FA25% mixture, a significant increase of 20.995% in rebound index values was observed between the 7th and 14th day, with a subsequent decrease of 4.76% from the 14th to the 28th day.
- The 15/38-365 FA50% mixture demonstrated a 20.016% increase in rebound index results from the 7th to the 14th day, followed by a 10% decrease from the 14th to the 28th.
- In the 15/38-365 FA75% mixture scenario, the rebound index results witnessed a 5.5% increase from the 7th day to the 14th day, and a further increase of 10.53% was observed from the 14th day to the 28th day.

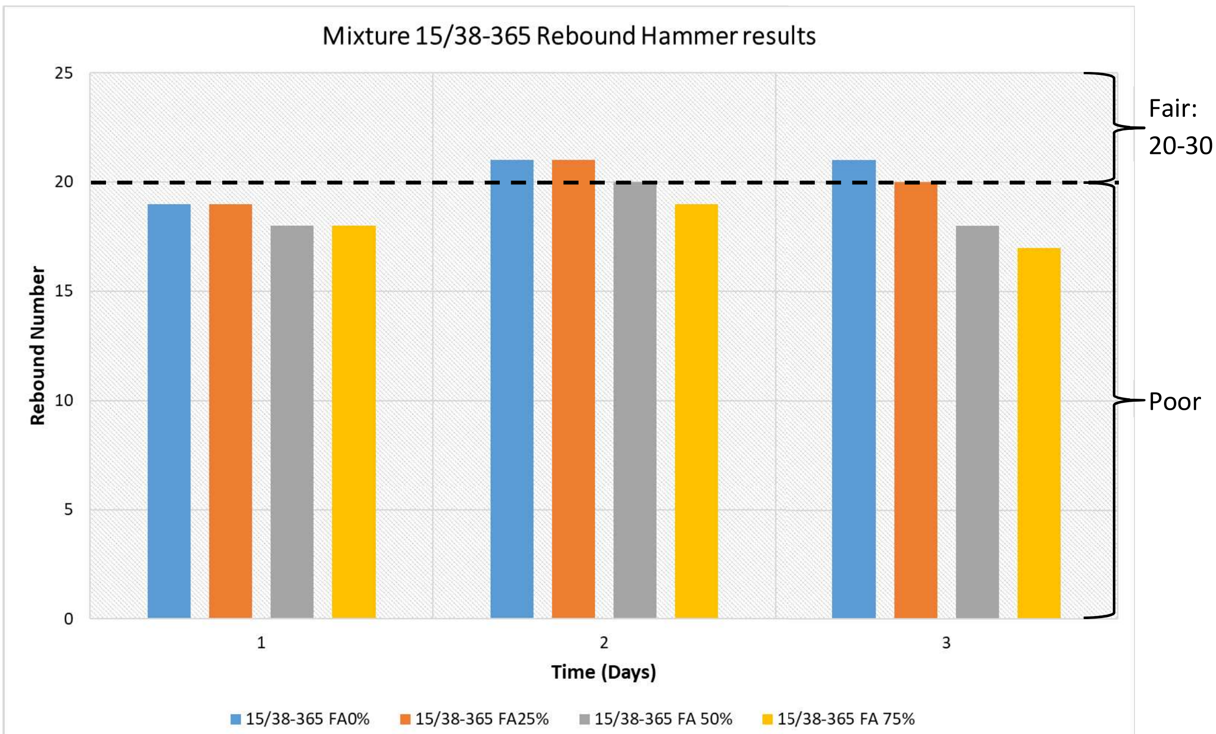

Figure 4.8 The rebound number for RCC mix 15/38-365 from 7 to 28 days.

4.3.3 Both mixture 15/38-365 and mixture 20/38-90 RH test conformity

Rebound hammer measurements were conducted using a digital rebound hammer unit, which recorded several data points, including the rebound number index, the average rebound index number for each specimen, and the number of impacts applied to the same specimen. The measurements were taken horizontally to ensure uniformity, and the rebound hammer was set to perform manual measurements at a 0-degree horizontal angle. Furthermore, the settings were configured to provide measurements regarding rebound number values.

A minimum of five rebound hammer measurements were taken on various locations of a single surface of each specimen, deliberately avoiding repeating measurements at the same spot. These measurements were spaced at least 25 mm away from the surface's edge and each other. The recorded rebound number

measurements could then be correlated with the manufacturer's calibration curve for the 0-degree angle, as illustrated in Figure 4.9 (Diagnostic Research Company, 2022). The average results for each trial mixture at the 7, 14, and 28-day testing intervals are presented in Figure 4.7 and
Figure 4.8. For the classification of concrete quality, the RH measurements align with the guidelines outlined by Malek (2020) and Yahya, et al., (2018), as depicted in Table 4.4.

Table 4.4 Concrete quality classification based on the rebound number results (Malek Jeddi, 2020)

GUIDELINES		ACTUAL RCC MIXTURE	
Malek Jedidi (2020)		Mixture 20/38-90	Mixture 15/38-365
Quality of Concrete	Rebound Number	Overall UPV average (Km/s)	Overall UPV average (Km/s)
Very good hard layer	> 40		
Good layer	30 to 40		
Fair layer	20 to 30		
Poor layer	<20	Between 16 and 22	Between 17 and 21
Delaminated	0		

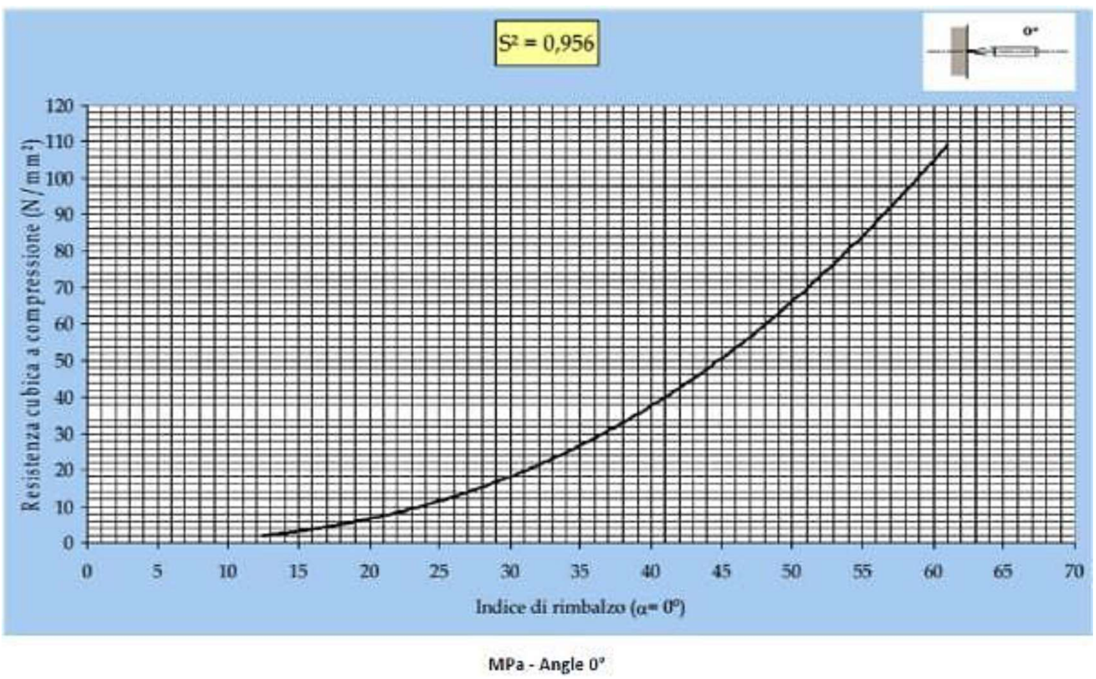

Figure 4.9 Correlation curve for estimated compressive strength of the concrete (Diagnostic Research Company, 2022)

Table 4.5 shows the average values recorded for all specimens' rebound numbers. It is visible from the results that results are similar throughout the different mixtures. This can be due to the concrete still curing and, therefore, needing to gain final strength; thus, the surface of the specimens is still soft. The rebound number results recorded for mixture 20/38-90 and 15/38-365 are similar.

Table 4.5 Summary of all rebound number values for the different mixtures

Average Rebound Hammer Results (Rebound Number)								
DAY	20/38-90 FA0%	20/38-90 FA25%	20/38-90 FA50%	20/38-90 FA75%	15/38-365 FA0%	15/38-365 FA25%	15/38-365 FA50%	15/38-365 FA75%
7	18	17	16	16	19	19	18	18
14	21	22	21	19	21	21	20	19
28	21	20	21	19	20	20	18	17

As shown in Table 4.5 and in Figure 4.10, the following is interpreted. For 20/38-90 FA0%, 20/38-90 FA50% and 20/38-90 FA75%, the values increase between 7 to 28 days however, for 20/38-90 FA25%, the results increase and then decrease again, this may be attributed to many factors that could have influence the measurement. Some of these factors can be the smoothness of the specimen's surface, the specimen's compaction, the moisture content of the specimen, the presence of aggregate close to the surface, maybe not being visible or air voids. For the different B mixtures, 14 days rebound measurement is higher than 7 days but lower than 28 days. Thus, it can be due to the surface smoothness that an aggregate was leading to a higher rebound number recorded or the compaction of the specimens, some factors that might have influenced the rebound number measurements.

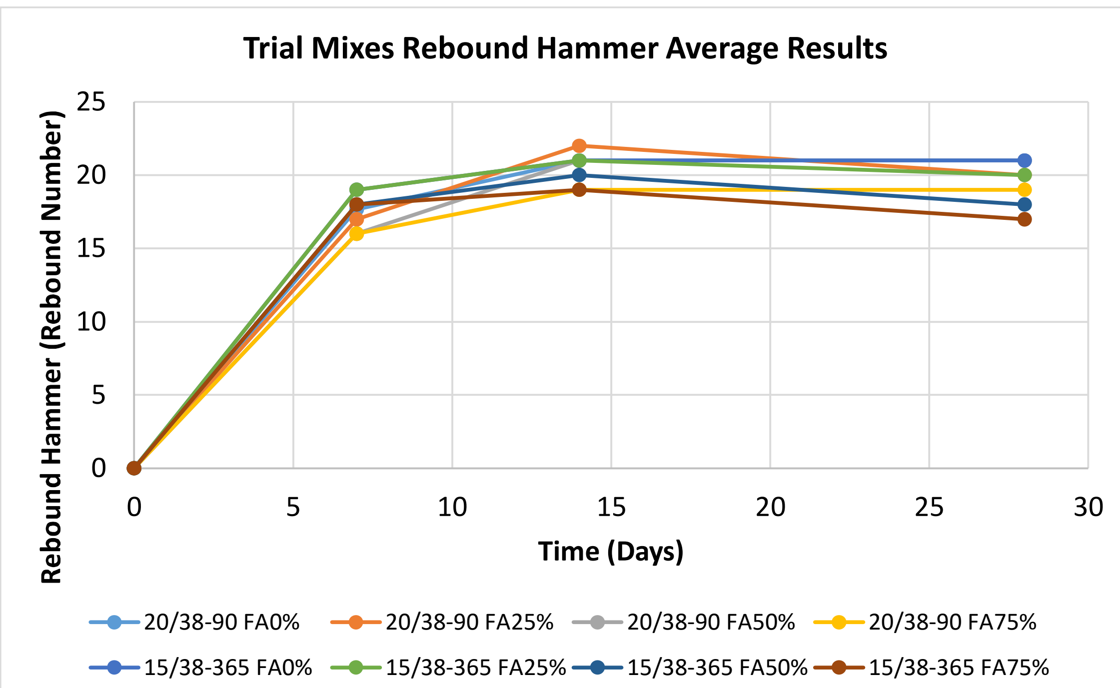

Figure 4.10 Rebound number values recorded for the different mixtures of time 7 days to 28 days.

4.4 Laboratory compressive strength test results

After the UPV and Rebound hammer test were done on the specimens, the compressive strength was tested for each specimen on 7, 14 and 28 days. The average compressive strength for mixture 20/38-90 and mixture 15/38-365 are presented in Figure 4.11 and Figure 4.12, respectively.

4.4.1 Mixture 20/38-90 compressive strength test results

It is evident from Figure 4.11 for mixture 20/38-90 (20 MPa with 70% FA at 90 days) that the mixes with 0%, 25% and 50% FA replacement achieve more than 20 MPa strength on 28 days and 75% FA replacement achieves 11,9 MPa and is predicted to reach the 20 MPa compressive strength by 90 days of curing age. The compressive strength increased with time. The compressive strength decreased with increasing fly ash proportions at 7, 14 and 28 days. For 20/38-90 FA0%, the compressive strength is lower at 14 days than the 7-day age strength.

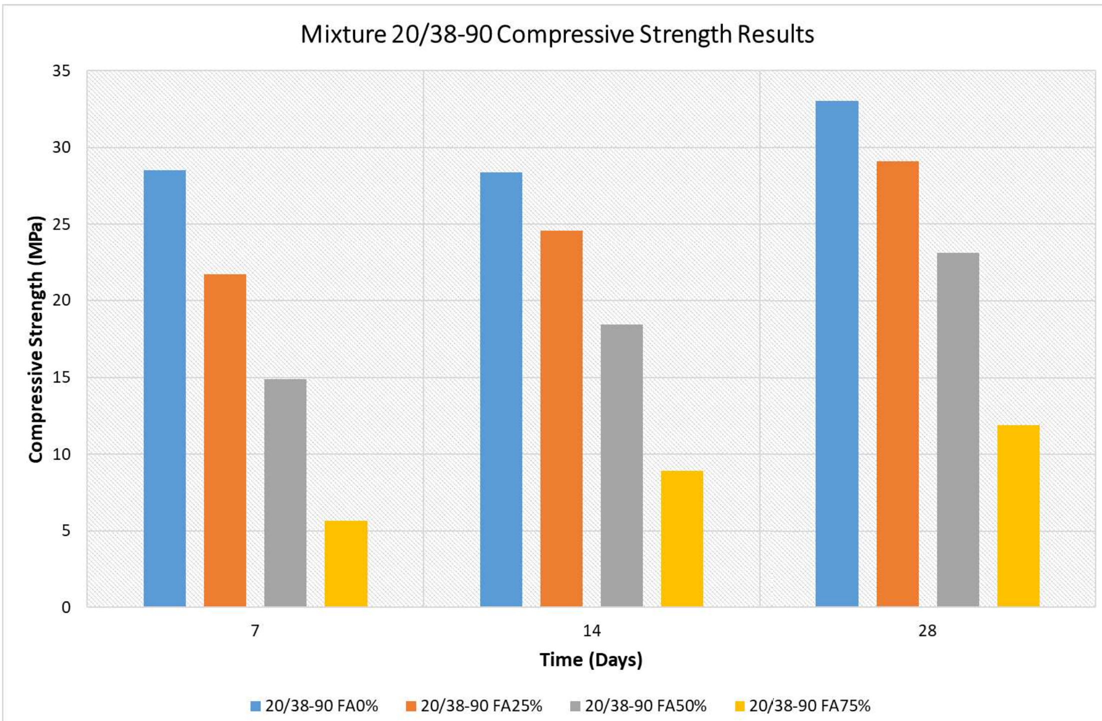

Figure 4.11 The compressive strength for RCC mix 20/38-90 from 7 to 28 days.

4.4.2 Mixture 15/38-365 compressive strength test results

From Figure 4.12 for RCC mix 15/38-365 (designed for 15 MPa at 90 days for 70% FA), it is evident that 0% and 25% replacement of FA achieves more than 15 MPa strength on 28 days. The 50% and 75% replacement with FA achieves 8MPa and 4,3MPa, respectively and is predicted not to reach 15MPa at 90 days. The compressive strength increased with time. The compressive strength decreased with increasing fly ash proportions at 7, 14 and 28 days. For 15/38-365 FA0%, the compressive strength is lower at 14 days of age against the 7-day age strength.

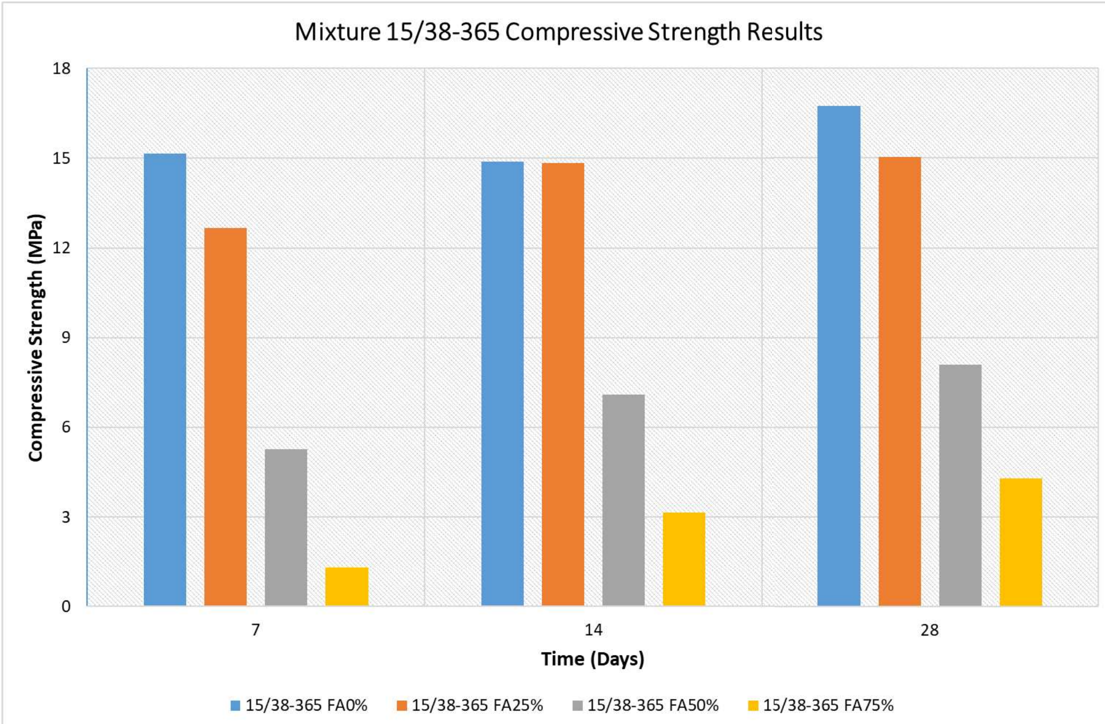

Figure 4.12 The compressive strength for RCC mix 15/38-365 from 7 to 28 days.

4.4.3 Mixture 15/38-365 and mixture 20/38-90 compressive strength test conformity

In South Africa, the total cementitious materials content used in RCC mixture design ranges from 110 to 207 kg/m3 (Shaw & Perrie, 2021: 858), and high-cementitious RCC is classified as total cementitious content exceeding 150 kg/m^3. Both mixtures 20/38-90 and 15/38-365 are classified as High-cementitious RCC according to standards. Figure 4.13 obtained from USACE (2000), RCC engineering manual, the historical data for RCC dams batched with pozzolan average compressive strength over the total cementitious content for different ages. Figure 4.13 provides a relationship between cement content and compressive strength of various equivalent cement contents with or without pozzolan. USACE also stated that the effect of pozzolan on RCC can only be determined in the laboratory and not be assumed. Therefore, the laboratory mixtures' equivalent cement content versus the compressive strength was plotted on the figure to see if it was in the design strength. The red line added to Figure 4.13 represents mixture 15/38-365 with 160 kg/m^3 total cementitious content, and the green line mixture 20/38-90 with 207kg/m^3.

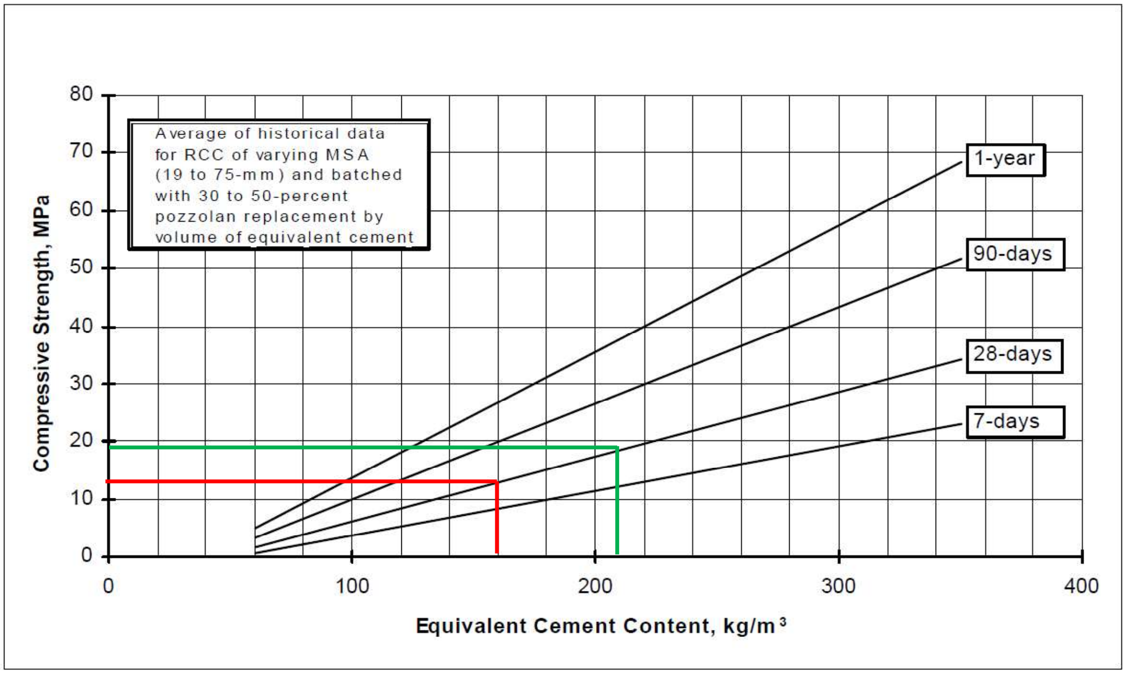

Figure 4.13 Equivalent cement content versus compressive strength for RCC batch with pozzolan (USACE, 2000: 3-2)

From Table 4.6 for mixture 20/38-90 with a total cementitious content and represented in green line in Figure 4.13, it can be taken that 20/38-90 FA50% with a 50% cement content, thus 50% pozzolan where fly ash was used, achieved 14,9 MPa on 7 days and 23,1 MPa on 28 days, which is slightly above the average as per Figure 4.13. It can thus be taken that the mixture 20/38-90 comply with RCC standards.

Table 4.6 Summary of compressive strength results for mixture 20/38-90

Compressive Strength in MPa (207 kg/m³ Cementitious material)		
Trial mixture	**7 days**	**28 days**
20/38-90 FA0% (100% CEM I)	28.5	33
20/38-90 FA25% (75% CEM I)	21.7	29.1
20/38-90 FA50% (50% CEM I)	14.9	23.1
20/38-90 FA75% (25% CEM I)	5.7	11.9

From Table 4.7 for mixture 15/38-365 with a total cementitious content of 160 kg/m^3 and represented in the red line in Figure 4.13, it can be taken that 15/38-365 FA50% with a 50% cement content. Thus, 50% pozzolan, in which fly ash was used, achieved 5,3 MPa on 7 days and 8,1 MPa on 28 days, below the average as per Figure 4.13. However, 15/38-365 FA25% with 75% cement content and 25% fly ash content achieved 12,7 MPa on 7 days and 15 MPa on 28 days, slightly above the average line. It can thus be taken that the mixture 15/38-365 comply with RCC standards.

Table 4.7 Summary of compressive strength results for mixture 15/38-365

Compressive Strength in MPa (160 kg/m³ Cementitious material)		
Trial mixture	**7 days**	**28 days**
15/38-365 FA0% (100% CEM I)	15.1	16.7
15/38-365 FA25% (75% CEM I)	12.7	15
15/38-365 FA50% (50% CEM I)	5.3	8.1
15/38-365 FA75% (25% CEM I)	1.3	4.3

Some different types of cement are available on the market. CEM I is a pure cement, whereas all the other types of cement are blends. Appendix B contains the cement characteristics for the cement type used in this research. There is also different fly ash available on the market in South Africa. Thus, it is of critical importance to use materials classified to SANS standards.

When mixed, the cement in a concrete mixture is the first to react chemically, which is why concrete with high cement concrete reaches high compressive strength early. But after about 28 days, the cement is said to react no more, and total strength is reached. However, fly ash only starts to react chemically when calcium hydroxide [Ca(OH)2] is created, as this chemical is needed to activate the fly ash in the mixture. Therefore, mixtures with fly ash will reach higher compressive strength later than concrete mixes with only a cement content (Thomas, 2007).

In Figure 4.14 it can be noted that the fly ash percentage content used in a mix design does influence the compressive strength of concrete. Table 4.8 summarises the average compressive strength per mixture per day.

Table 4.8 Summary of all rebound number values for the different mixtures

COMPRESSIVE STRENGTH (AVERAGES) (MPa)								
DAYS	**20/38-90 FA0%**	**20/38-90 FA25%**	**20/38-90 FA50%**	**20/38-90 FA75%**	**15/38-365 FA0%**	**15/38-365 FA25%**	**15/38-365 FA50%**	**15/38-365 FA75%**
7	28,5	21,7	14,9	5,7	15,1	12,7	5,3	1,7
14	28,4	24,6	18,5	8,9	14,9	14,8	7,1	3,1
28	33,0	29,1	23,1	11,9	16,7	15,0	8,1	4,3

In general, the compressive strength increased. The strength decreases as the cement content is lower and cementitious material content rises per mix.

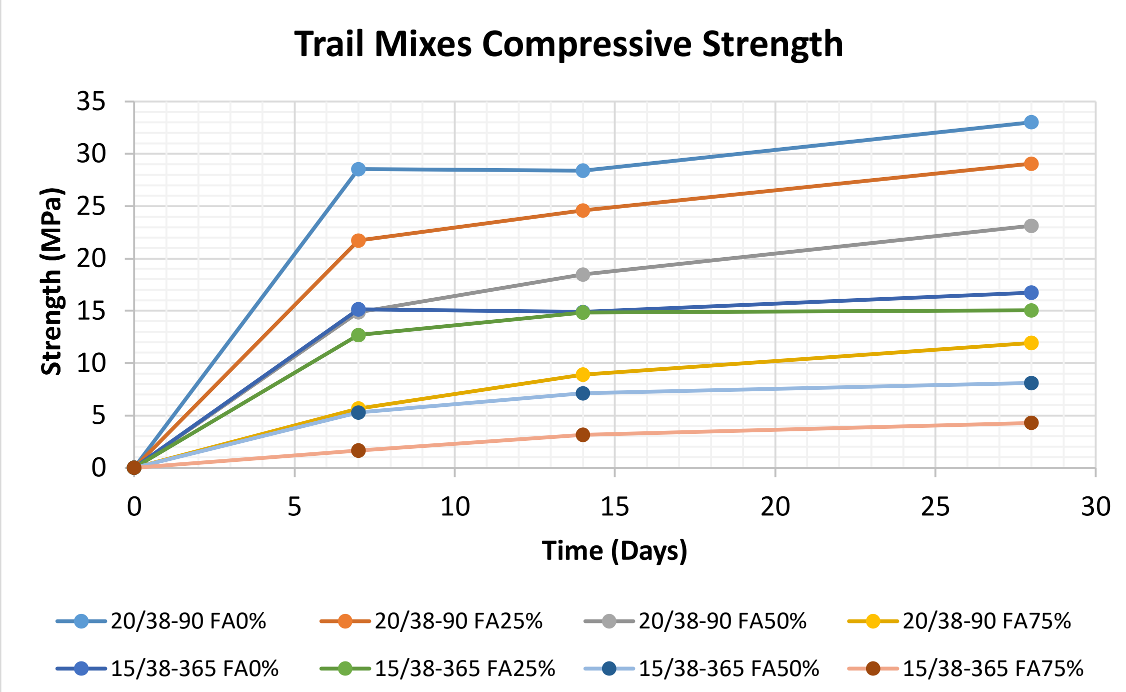

Figure 4.14 Compressive strength values recorded for the different mixtures of time 7 days to 28 days.

Figure 4.14 compares samples from various Roller-Compacted Concrete (RCC) mixes, illustrating the compressive strength achieved at 28 days. Notably, mix designs 20/38-90 FA0%, 20/38-90 FA25%, and 20/38-90 FA50% surpassed the 20 MPa compressive strength requirement at just 28 days, meeting the 90-day target early. However, it is predicted from Figure 4.11 for the 20/38-90 FA75% mix, s that it may not achieve the 20 MPa threshold at 90 days.

As seen in Figure 4.12 and Figure 4.14, mixtures 15/38-365 FA0% and 15/38-365 FA25% met the 15 MPa compressive strength requirement at 365 days, well ahead of the 28-day mark. In contrast, both mixtures 15/38-365 FA50% and 15/38-365 FA75% did not exhibit high compressive strength values. It is predicted from Figure 4.12, that mixture 15/35-365 FA50% and 15/38-365 FA75% may not reach the targeted 15 MPa at 365 days.

The summary of average compressive strength values for these mixtures at 7, 14, and 28 days is presented in Table 4.8 and visualized in Figure 4.14. Notably, for both the 20/38-90 and 15/38-365 mixtures, 20/38-90 FA25%, 20/38-90 FA50%, 20/38-90 FA75%, 15/38-365 FA25%, 15/38-365 FA50%, and 15/38-365 FA75%, compressive strength increased from 7 to 28 days. However, mixtures 20/38-90 FA0% and 15/38-365 FA0% decreased from 7 to 14 days before an increase to 28 days. Various factors, including compaction, curing issues, and specimen damage, may account for these trends.

Furthermore, the compressive strength decreases as the cement content is reduced and the proportion of cement replaced by fly ash increases. Consequently, the overall cementitious content in a mix design significantly influences concrete's compressive strength.

The higher the total cementitious material content in a trial mix, the smaller the observed increase in compressive strength between different age intervals. Conversely, mixes with lower total cementitious material content demonstrate a more significant increase in compressive strength as the age of the concrete advances.

Additionally, the total cementitious content in a mix design plays a pivotal role in concrete compressive strength; a higher total cementitious content correlates with greater compressive strength. Different cement-water ratios in the mixtures also impact compressive strength. Mixtures with higher total cementitious content possess lower water-cement ratios, while mixtures with less total cementitious material content exhibit higher water-cement ratios. A lower water-cement ratio is associated with higher compressive strength. The percentage of fly ash in the cementitious combination further influences compressive strength, with higher fly ash percentages leading to lower compressive strength values.

4.5 Field dam wall test results

It has been established that De Hoop Dam and Spring Grove Dam are currently the only two South African dams suitable for Roller-Compacted Concrete (RCC). Based on this information, Ultrasonic Pulse Velocity (UPV) and Rebound Hammer (RH) tests were conducted on the non-overspill sections of these dam walls. At De Hoop Dam, measurements were limited to the Left bank embankment, with 10 measurement positions identified and utilised. In contrast, test measurements were carried out at Spring Grove Dam on the right and left bank embankments, encompassing 14 test measurement positions. For both dam sites, the test positions were strategically chosen on the upper-right downstream dam wall section and downstream steps to ensure that the test results accurately reflect the quality of the dam wall concrete.

4.5.1 De Hoop dam test results

Access at De Hoop Dam was constrained to the Left bank embankment, where 10 test positions were identified and delineated. Positions 3 to 6 were specifically chosen on the downstream steps, while the remaining positions were marked on the upper dam wall section. Notably, the Rebound Hammer (RH) tests were performed at the same positions where the Ultrasonic Pulse Velocity (UPV) tests were executed.

UPV test results

Position 3-6 was on the steps where semi-direct measurements were taken. Three measurements preposition was taken from where an average of the three results was calculated and is used in Figure 4.15. Position 1,2, 7 and 10 measurements were taken on the dam top wall section, using the indirect measurement method to measure the concrete quality. 150 mm and 300 mm indirect measurements were measured for each portion, and an average was calculated, as shown in Figure 4.15.

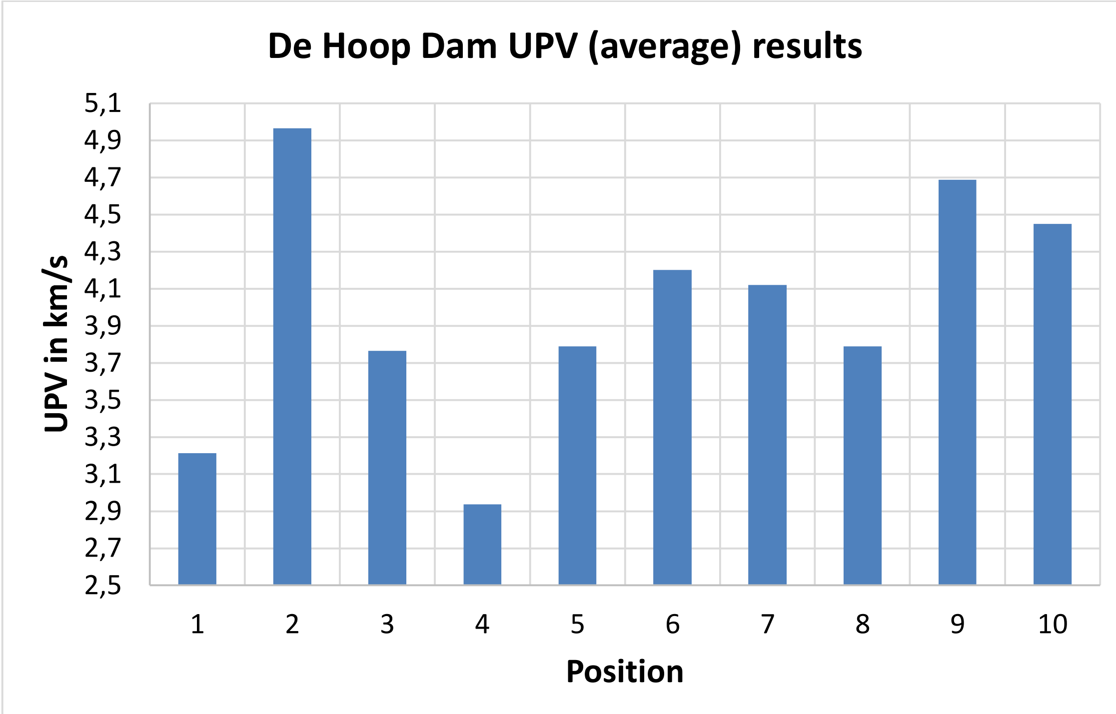

Figure 4.15 UPV results, taken at 10 different positions on the De Hoop Dam

Table 2.5 outlines the concrete quality based on the UPV results. However, it is essential to categorize the concrete quality considering S-wave measurements, given that the UPV assessment was conducted using semi-direct and indirect methods. Consequently, the results recorded for the steps (positions 3 to 6) fall within the range of 2,9 km/s to 4,1 km/s, where measurements exceeding 2,8 km/s are deemed excellent concrete quality. The remaining results for the steps consistently indicated excellent quality, surpassing the 2,8 km/s threshold. The test results in the top vertical dam wall section exceed 3,1 km/s, qualifying as excellent quality concrete. Consequently, the concrete quality at De Hoop Dam is considered excellent.

RH test results

All the results were measured using the 0-degree position of RH. Test results for position 3 to 6, which was taken on the steps, are lower than those measured on the top dam wall. Figure 4.16 shows the measured average rebound number results above 25 and below 35. As per Table 4.4, rebound number values between 20 and 30 are fair concrete layer quality and rebound number values between 30 and 40 are good concrete layer quality. The concrete at steps (positions 3 to 6) is generally between 20 and 30, and it can be said that the concrete layer quality at the steps is fair. The rebound number values for the top vertical dam wall section are between 30 and 35, and the concrete layer quality can be described as good concrete quality.

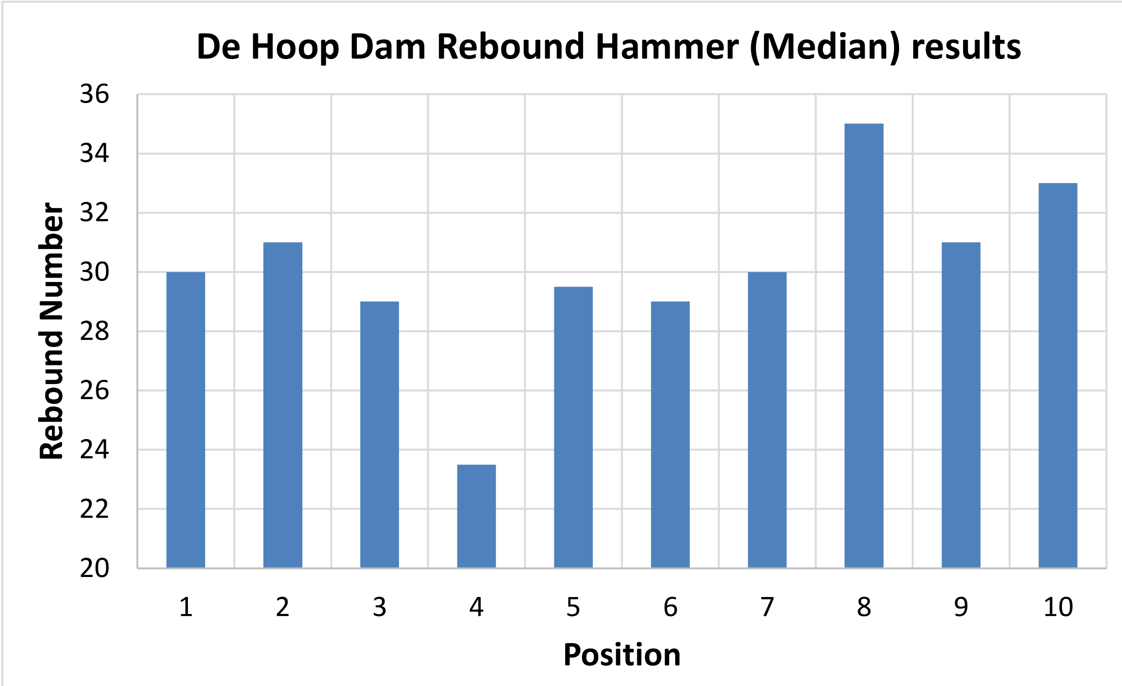

Figure 4.16 RH results, taken at 10 different positions on the De Hoop Dam

4.5.2 Spring Grove Dam test results

The dam wall was investigated where the test area on the left and right downstream embankment's top wall and steps were marked. After that, both UPV and RH tests were done at each test position. There was a total of 14 test positions marked out, with a total of 8 positions on the right bank embankment and 6 positions on the left bank. This was done to get an overall good representation of the dam wall concrete quality.

Spring Grove UPV results

Position 1-4 is the right bank steps, 5-8 corresponds to the right bank wall, 9-10 pertain to the left bank wall, and 11-14 relates to the left bank steps as illustrated in Figure 4.17, which shows the average results. Table 2.5 provides insights into the concrete quality based on the UPV results. It is crucial to note that concrete quality is determined from S-wave measurements rather than P-wave, given that the UPV measurement employed semi-direct and indirect methods. The recorded results, excluding position 9, fall within the 3,0 km/s and 4,6 km/s range. UPV measurements surpassing 2,8 km/s are deemed excellent concrete quality. Notably, the UPV recording for position 9 is 1,9 km/s, classifying it as medium concrete quality. Overall, except for position 9, the quality of the concrete, as indicated by the UPV measurements, is excellent.

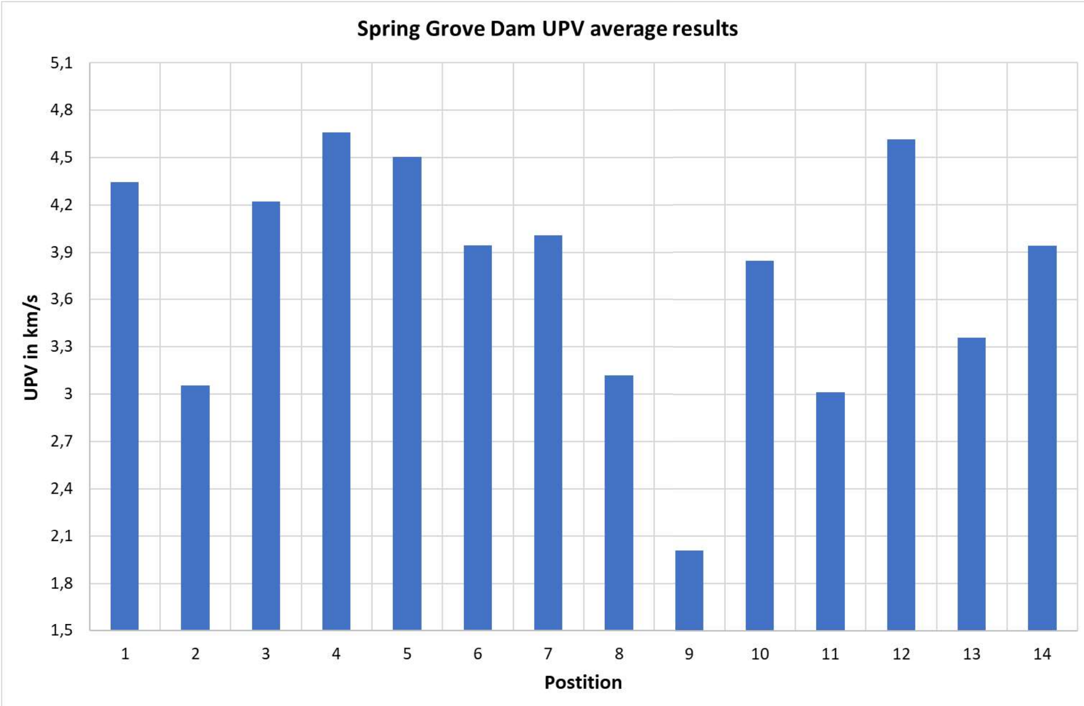

Figure 4.17 UPV results, taken at 15 different positions on the Spring Grove Dam

Spring Grove Dam RH test results

The measurements were taken at either 0 degrees or 90 degrees. The measurements for positions 5 to 10, taken at the top of the dam wall, were recorded at 0 degrees. The measurements for the rest of the step positions were recorded at 90 degrees. Figure 4.18 shows the results obtained from the measurements taken with the rebound hammer. The results vary between 20 and 30 rebound number values, except for test position 1. According to Table 4.4, these rebound number values between 20 and 30 indicate fair concrete layer quality.

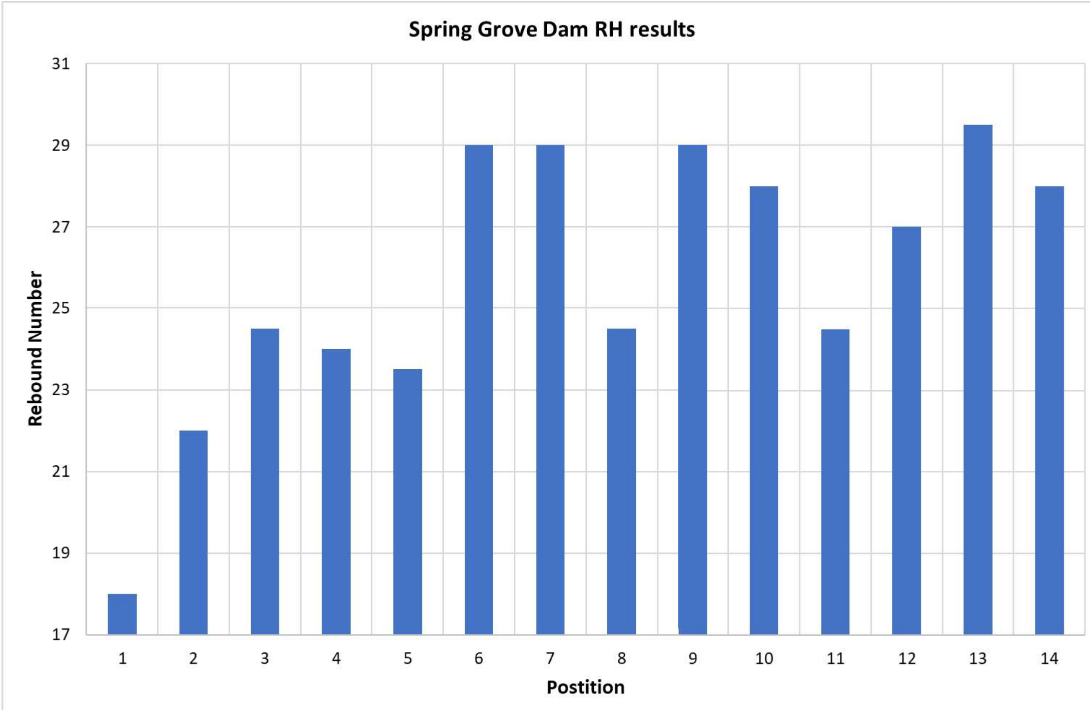

Figure 4.18 RH results, taken at 15 different positions on the Spring Grove Dam

4.6 Conclusion

This chapter presented the results obtained during the experimental laboratory testing and dam wall field testing. From the test results collected for the UPV tests, indicating the average results for each age day of the specimens where the different batches with a variant of FA content are presented in different colours. The average results for mixture 20/38-90 UPV is above 4,0 km/s and 15/38-365 is above 3,4 km/s. According to the International Atomic Energy Agency (2002), 3,5 km/s to 4,3 km/s is good quality concrete; therefore, both 20/38-90 and 15/38-364 concrete mixture design is considered good quality.

UPV for 20/38-90 is maximum at 25%FA for both 7 and 14 days. The UPV decrease with increasing FA concentration. However, at 28 days the behaviour changed where the UPV is at a minimum at 25%FA and increase with increasing fly ash concentration. For 15/38-365 UPV, the UPV is the highest for FA25% on 7 and 14 days. The UPV decrease as the FA increases.

The laboratory RH results for mixture 20/38-90 is above 16 rebound number, and for mixture 15/38-365 is above 17 rebound number. According to Malek (2020) and Yahya, et al., (2018), for both mixtures 20/38-90 and 15/38-365, the concrete layer quality is poor and can be described to the specimens' test at early ages and have not reached final curing.

The compressive strength results for 20/38-90 and 15/38-365 showed that for mixture 20/38-90, the average design mixture strength of 20 MPa at 90 days was reached except for mixture 20/38-90 FA75%, which is predicted to reach the 20 MPa compressive strength by 90 days of curing age. Mixture 15/38-365 reach for some of the different trial mixtures with a designed strength of 15 MPa at 365 days. Due to the coarseness of the specimens, it is not predicted to reach the design strength, especially for mixture 15/38-365 FA 75%.

Most recorded measurements for the De Hoop dam are above 3,5 km/s; therefore, the concrete quality is described as good. Most rebound number values for the De Hoop dam recorded are between 20 and 30, and the concrete layer quality is described, according to Malek (2020) and Yahya, et al., (2018), as fair. Spring Grove dam values recorded for UPV are mostly above 3,0 km/s, and the concrete quality is described as questionable where there is a possibility that slight porosity may exist. The rebound number values recorded for Spring Grove dam are primarily between 20 and 30, therefore considered a fair layer of concrete quality.

Overall, the compressive strength decreases as FA content increases in mix design. The UPV values recorded were the best for FA25% and decreased as FA content increased, where the RN stays more or less the same, which can be expected, as the concrete surface has not yet reached surface hardness.

The data obtained will be used to determine a correlation between the compressive strength in MPa for ultrasonic pulse velocity and rebound number, using Multiple regression methodology and is presented in the next chapter.

Chapter 5 Development of correlation using laboratory and field results

In this chapter, we explore the analysis and interpretation of results obtained from various tests and measurements, focusing on establishing correlations among key parameters in the context of Roller Compact Concrete (RCC) for Dams. The primary objectives of this chapter include exploring the relationship between destructive compressive strength and non-destructive test results, specifically Pulse Wave (PW) and Rebound Hammer (RH) values. Additionally, we aim to investigate the effectiveness of a multiple regression model in predicting compressive strength based on these non-destructive test results and laboratory compressive strength measurements.

5.1 Statistical analysis of data

The statistical analysis of data presented in Table 5.1 represent all the results collected in the laboratory, including comprehensive strength, UPV and rebound number values recorded, and encompasses all recorded outliers. In Table 5.2 the statistical analysis represents the laboratory results collected without the outliers.

5.1.1 Statistical analysis of all laboratory test data results

For the compressive strength, UPV and RH, descriptive statistics were computed based on the results conducted in Table 5.1 on 72 specimens. For UPV, two test readings was recorded per specimen, and the average of those two values was use. At least five test readings were recorded per specimen for RH, and the median value of these values of these readings was used.

The descriptive statistics in Table 5.1 provide a comprehensive overview of the distribution, central tendency, dispersion, and shape of the three variables in the dataset.

Table 5.1 Descriptive statistics of all laboratory results

Descriptive Statistics from laboratory results			
	Compressive strength (MPa)	**UPV (km/s)**	**Rebound number (RN)**
Mean	15,29	5,09	19,09
Standard Error	1,06	0,07	0,21
Median	14,92	5,17	19,00
Standard Deviation	8,97	0,56	1,75
Kurtosis	-0,87	0,28	-0,40
Skewness	0,34	-0,91	-0,36
Range	33,61	2,38	7,00
Minimum	0,68	3,52	15,00
Maximum	34,29	5,90	22,00
Count	72,00	72,00	72,00

Statistical analyses often rely on central tendency measures to understand a dataset's typical value. In statistics, the mean, median, and mode stand out as key measures representing central tendency. Each measure utilises a distinct method to determine the central point, providing valuable insights into where the bulk of values in a distribution are concentrated. Beyond merely indicating the majority of values, these statistics act as markers for the central location within the distribution. The discussion delves into the central tendency of the laboratory dataset's descriptive statistics, as detailed in Table 5.1.

For our dataset, the mean compressive strength is 15.29 MPa, UPV is 5.09 km/s, and the RN mean is 19.09. This metric serves as a measure of central tendency, representing the typical recorded value. However, it is important to note that the mean can be influenced by extreme values, and in our case, where outliers are present in the dataset, it may impact these computed values.

In our specific dataset, the median for compressive strength is 14.92 MPa, for UPV, it is 5.17 km/s, and for RN, it is 19. Unlike the mean, the median remains uninfluenced by extreme values. This characteristic enhances its robustness as a measure of central tendency, particularly when dealing with a dataset containing outliers.

The mean and median values are close together for all three datasets, which is a good sign that the datasets follow a symmetric distribution. However, for both compressive strength and RN, the mean is greater than the median, which indicates that the distribution is right-skewed.

A standard error of 1,06 MPa suggests the variability expected in the sample mean of compressive strength measurements. The standard error of 0,07 km/s of UPV indicates a low variability, and the standard error of 0,21 for RN is also low and, therefore, favourable.

In this case, the standard deviation for compressive strength is 8,97. This value is approximately 58% of the respective mean values, which is relatively high due to the dataset covering a range of compressive strength values of the specimens. The standard deviation for UPV is 0,56 km/s, roughly 11% of its mean value, and for RN, it is 1,75, approximately 9%. These percentages signify a low variability relative to the mean for UPV and RN.

For this dataset's compressive strength, a negative kurtosis value of -0,87 MPa implies a less peaked distribution with lighter tails than a normal distribution. They indicate that the compressive strength data may exhibit fewer extreme values (outliers) than a normal distribution. A negative kurtosis value of – 0,40 for RN indicates a less peaked distribution with a lighter tail than the normal distribution, suggesting that the dataset may have fewer extreme values than a normal distribution. A positive kurtosis value of 0,28 km/s for UPV is calculated.

A skewness value of 0,34 MPa for compressive strength suggests a mild positive skewness, indicating that the compressive strength data might have a slightly longer right tail. This implies that there may be a few higher values (outliers) on the higher end of the compressive strength distribution. A negative skewness of -0,91 suggests a substantial leftward skewness in the distribution of UPV. This implies that a concentration of lower values may contribute to a longer left tail. A negative skewness of 0,36 indicates a slight leftward skewness in the distribution of RN. This suggests that lower values tend to be more concentrated and contribute to a longer left tail.

5.1.2 Statistical analysis of all laboratory test data without outliers

For the compressive strength, UPV and RH, descriptive statistics were computed based on the results conducted in Table 5.2 on 65 specimens. For UPV, two test readings was recorded per specimen, and the average of those two values was use. At least five test readings were recorded per specimen for RH, and the median value of these values of these readings was used. The descriptive statistics in Table 5.2 provide a comprehensive overview of the distribution, central tendency, dispersion, and shape of the three variables in the dataset where outliers where removed.

Table 5.2 Descriptive statistics of laboratory results with outliers removed

Descriptive Statistics from laboratory with outlier removed			
	Compressive strength (MPa)	**UPV (km/s)**	**Rebound number (RN)**
Mean	15,52	5,11	19,07
Standard Error	1,10	0,07	0,22
Median	15,09	5,18	19,00
Standard Deviation	8,89	0,55	1,81
Kurtosis	-0,84	0,22	-0,52
Skewness	0,36	-0,93	-0,34
Range	32,71	2,31	7,00
Minimum	1,58	3,52	15,00
Maximum	34,29	5,83	22,00
Count	65,00	65,00	65,00

The mean compressive strength is 15,52 MPa, which is 1,5% higher than the mean for the dataset with outliers of 15,29 MPa. The mean UPV is 5,11 km/s, 0,4% higher than the 5,09 km/s calculated for the dataset with outliers. A 0,1% lower RN value of 19,07 was calculated against the 19,09 rebound number for the dataset, including the outliers.

The median value for compressive strength is 15,09 MPa, which is 1,1% higher than the 14,92 MPa measured for the dataset, including the outliers. The median for UPV is 5,18 km/s without the outliers, which is 0,2% higher than the dataset, including the outliers of 5,17 km/s. The RN median of 19,00 stayed the same as was calculated for a dataset with the outliers.

The standard error for compressive strength is 3,6% higher with 1,10 MPa than for the dataset with outliers. The standard error for the UPV and RN is the same as for the dataset, including the outliers. The standard deviation for compressive strength is 8,89 MPa, 57% of the respective mean values. The standard deviation for UPV is 0,55 km/s, 10,7% of its mean value, and for RN, is 1,81, 9,5%. The negative kurtosis value is -0,84 MPa for compressive strength, with a negative distribution for RN of -0,52. However, a positive distribution of 0,22 km/s for UPV is shown. The skewness for compressive strength is 0,36. The skewness for UPV and RN is negative with -0,93 km/s and -0,34.

The provided statistical measures for the two datasets are similar. These statistical measures collectively offer insights into each dataset's central tendency, variability, shape, and distribution, aiding in a comprehensive understanding of the data.

Furthermore, the multiple regression results are documented in Appendix H. This is to enable further scrutiny and reference for interested readers.

5.2 True representation of laboratory results

Pearsons ' method analysed correlations between measured variables, i.e., Compressive strength, UPV and RN, as recorded in the laboratory. Table 5.3 summarises the correlation coefficients for all the results acquired during the laboratory test. It is clear that there is a strong positive correlation between compressive strength and UPV, with a value of 0,773. The correlation between compressive strength and rebound number is weakly positive, as is the correlation between UPV and RN, with values of 0,366 and 0,219, respectively.

The compressive strength measurement is an assessment of the entire specimen obtained by crushing the specimen. Compressive strength is, therefore, representative of the entire specimen's strength.

An instrument takes RN measurement with a surface area of (50 mm^2), and only the surface directly underneath the hammer is assessed. This method is, therefore, limited to only measuring superficial strength and is extremely sensitive to local variations in strength compared with compressive strength. If relatively fresh concrete (i.e. not fully cured) is assessed, the incomplete bonding process will additionally result in an unrepresentative assessment of the specimens' (ultimate) strength. The comparatively weak correlation is, therefore, not surprising.

When measuring the UPV of a specimen, the ultrasonic waves characterise a substantial volume of the sample as they pass through it between the emitter and receiver. While not as comprehensive as the compressive strength, this is a substantially more representative assessment of the specimen's properties than the RH. Therefore, the strong positive correlation is likely due to the more realistic assessment of the strength inherent in the method.

Table 5.3 Pearsons' correlation of true representation of laboratory results

	Compressive Strength	**UPV**	**Rebound Number**
Compressive strength	1	NA	NA
UPV	0,773	1	NA
Rebound Number	0,366	0,219	1

UPV and compressive strength results are plotted in Figure 5.1. A linear trendline with a moderate correlation coefficient illustrates the positive correlation between the two variables.

RH and compressive strength results are plotted in Figure 5.2. A linear trendline with a weak correlation coefficient illustrates the positive correlation between the two variables.

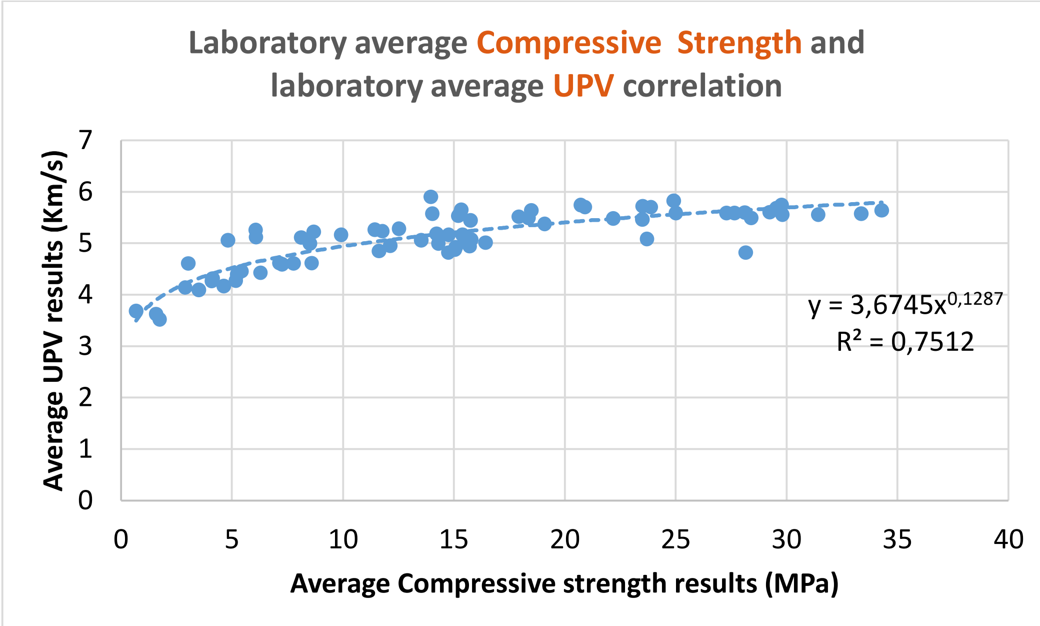

Figure 5.1 Correlation between compressive strength and average UPV results from laboratory results

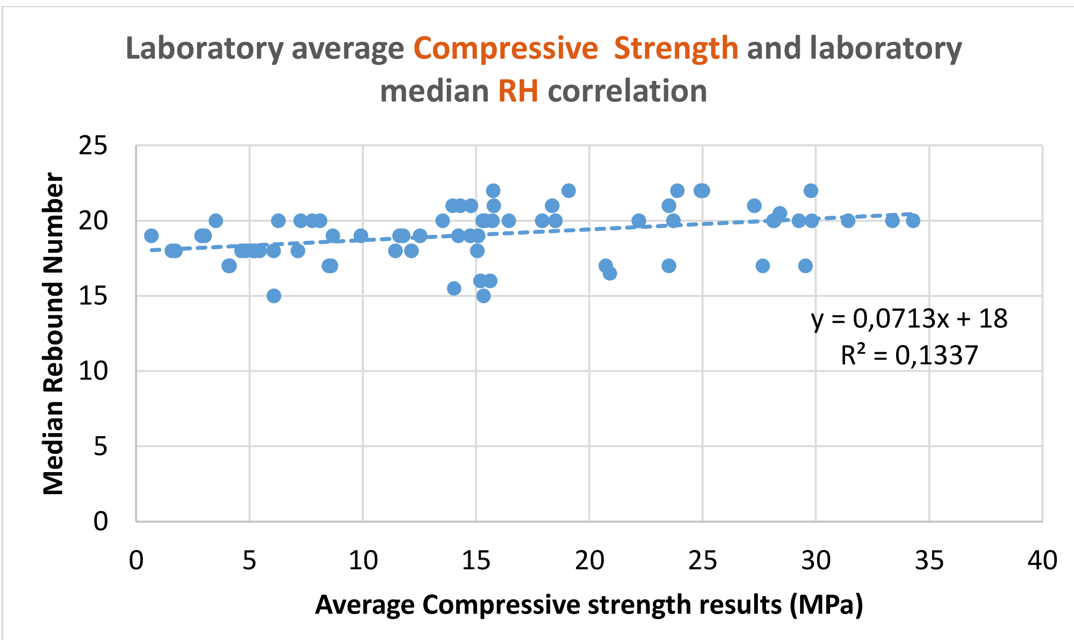

Figure 5.2 Correlation between compressive strength and median RH results from laboratory results

5.3 Representation of laboratory results with outliers removed

Pearsons ' method analysed correlations between measured variables as recorded in the laboratory, with outliers removed. Table 5.4 summarises the correlation coefficients, from where it is clear that there is a strong positive correlation between compressive strength and UPV, with a value of 0,831, which is increased from 0.773. The correlation between compressive strength and rebound number is weakly positive, with slight decreases from 0.366 to 0.357, whereas the correlation between UPV and RH remains the same. The Pearson's correlation coefficients between the UPV and RH stayed the same as when the outliers were not removed. The RH and compressive strength correlation is weaker with the outliers removed than without the outliers removed.

Table 5.4 Correlation between data where the outliers was removed from results

	Compressive strength	UPV	Rebound Number
Compressive strength	1	NA	NA
UPV	0,831	1	NA
Rebound Number	0,357	0,219	1

UPV and compressive strength results are plotted in Figure 5.3. A power trendline with a strong association correlation coefficient illustrates the positive correlation between the two variables.

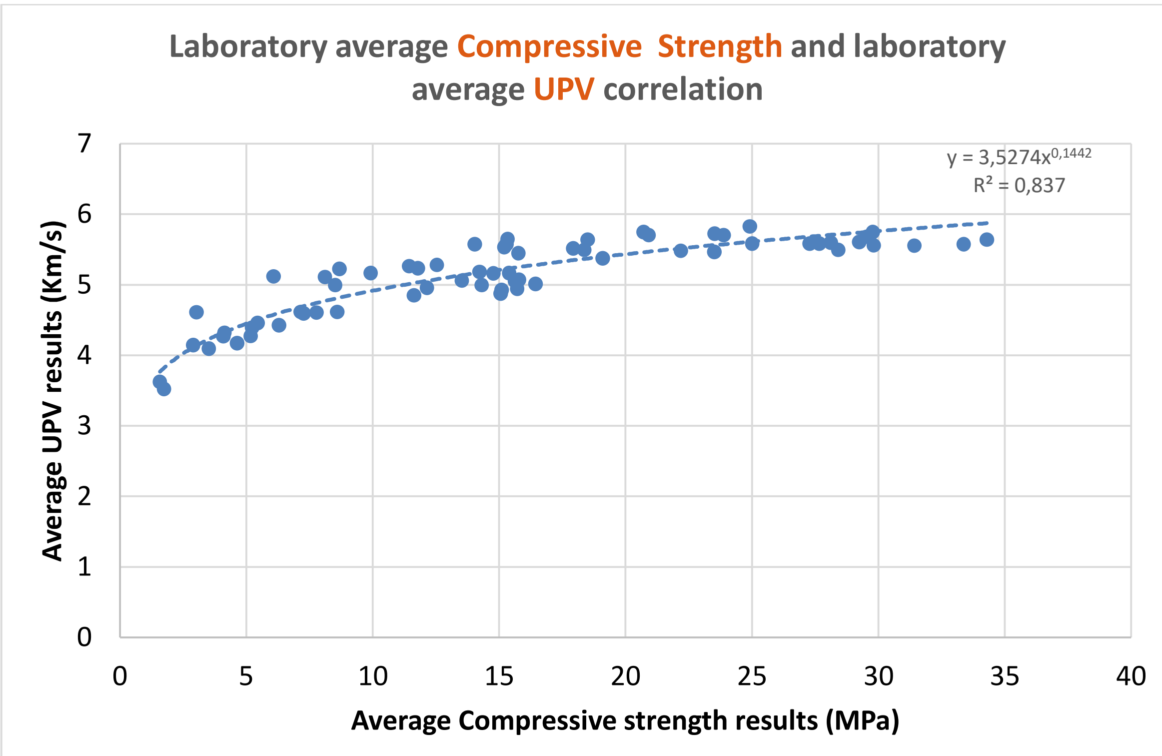

Figure 5.3 Correlation between compressive strength and average UPV with outlier removed from results recorded from laboratory tests

RH and compressive strength results are plotted in Figure 5.4. A linear trendline with a very weak correlation coefficient illustrates the positive correlation between the two variables. The correlation coefficient is lower in Figure 5.4 than in Figure 5.2, where the outliers were not removed.

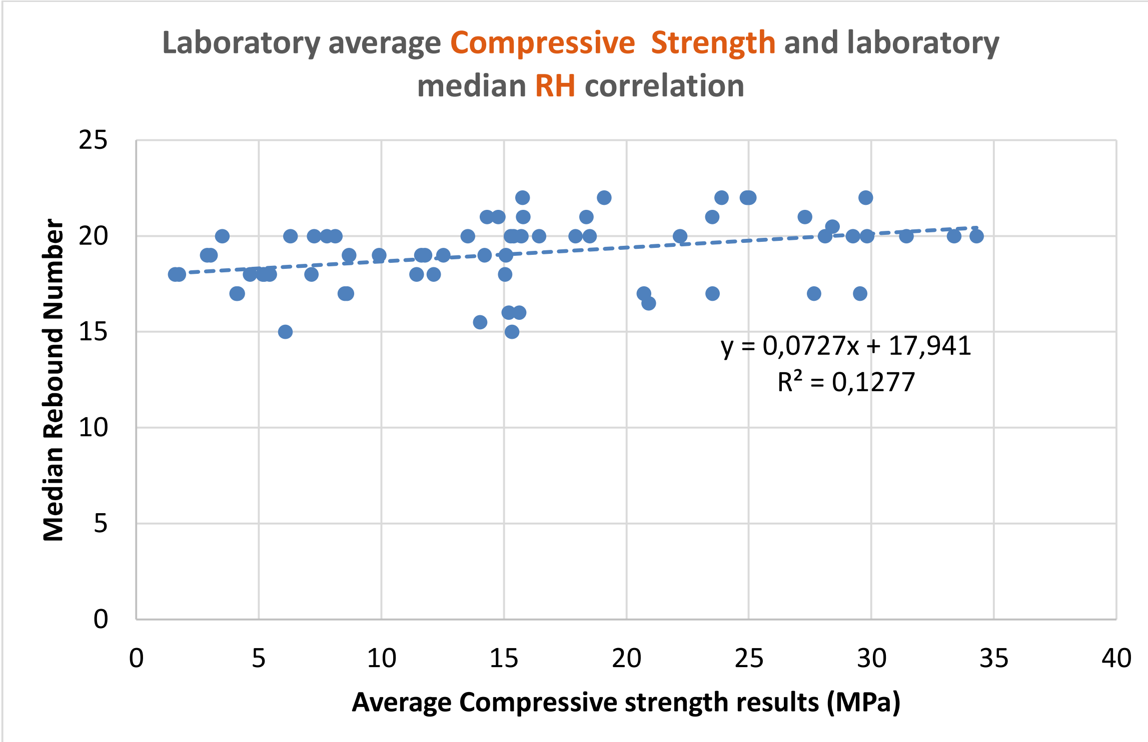

Figure 5.4 Correlation between compressive strength and median RH results with outliers removed from results recorded from laboratory tests

5.4 Multiple regression

This section discusses a model that represents a statistical framework. This model is designed to describe how intervening variables convey the influence from causal variables to outcome variables. This model aims to assess whether it is possible to derive a correlation between compressive strength measurements, UPV measurements and RN measurements. The goal is to understand how these different measurements relate to each other and whether they are correlated.

This section discusses developing and using a statistical model to explore the relationships between different measurements (compressive strength, UPV and RH) and understand how intervening variables may affect these relationships. The goal is to assess whether there is a correlation between these various assessments, which can be valuable for assessing the quality and properties of RCC.

The focus is on regression analysis of the test results, as discussed in preceding chapters, with outliers removed. For this purpose, data points over 1 standard deviation above or below the median value were considered outliers.

In the laboratory, a direct arrangement was employed to measure the UPV of specimens. An indirect or semi-direct arrangement was employed for the dam walls to measure the UPV of the RCC. The choice of an indirect or semi-direct arrangement method was used due to practical considerations in the testing environment.

The analysis involved conducting a simple multiple linear regression to establish a correlation between ultrasonic pulse velocity (UPV) and rebound hammer (RH) measurements with compressive strength, following removing outliers from the dataset. The regression equation (5.1) was derived, where the dependent variable, compressive strength, is predicted based on the model's coefficients. The R-squared's summary output for the multiple regression model was 72%. Thus, the UPV average and RH median variables explain about ¾ of the dependent compressive strength variable variation. Furthermore, the model is significant with a P-value of 5,08E-18, which is well below 0,5 or 0,1 with a 95% and 99% confidence level. Therefore, the model is statistically significant. The actual variable's P-values were as follows: 4,25 E -17 for UPV average and 9,10 E-3 for RH median, and therefore statistically significant at a 99% confidence level. The model coefficients for the intercept are -66,79, with an UPV average of 12,73 and a RH median of 0,91. Thus, Equation (5.1) provides a predictive tool for estimating compressive strength using UPV and RH, facilitating a comprehensive understanding of the interplay between these variables in the given context.

$$Compressive\ Strength = -66{,}8 + 12{,}73\ UPV\ (P - wave) + 0.91\ RH \tag{5.1}$$

Data collected in the laboratory on the specimens for UPV, was conducted with direct method. These velocity waves recorded was P or longitudinal waves. The field data collected at the dams for UPV was conduct using indirect or semi-direct method, the velocity waves recorded was shear (S) waves. Therefore, the multiple regression derived in equation (5.1) can not be used to model the field data.

The laboratory average UPV data was converted to S-waves using Poisson's ratio formula to change between P-waves and S-waves as shown in equation (5.2).

$$\frac{V_p}{V_s} = \sqrt{(1 + 2\eta)} \tag{5.2}$$

Where:

V_p = Velocity of P-waves in concrete

V_s = Velocity of S-waves in concrete

η = Poisson's ration of concrete

The relationship between compressional waves and shear waves in concrete can be expressed using Poisson's ration as shown in Equation(2.2):

$$V_s = \frac{V_p}{\sqrt{\left(1 + \left(2(0{,}15)\right)\right)}} \tag{5.3}$$

Where:

V_p = Velocity of P-waves in concrete

V_s = Velocity of S-waves in concrete

After the UPV velocity waves for the laboratory data was converted from P-waves to S-waves, a simple multiple linear regression was performed to correlate UPV and RH measurements to compressive strength, based on the dataset with outliers removed. The derived equation is shown in Equation (5.4) predicted the model based UPV Shear waves and median rebound number.

$$Compressive\ Strength = -74{,}4 + 16{,}17\ UPV\ (S - wave) + 0{,}903\ RH \tag{5.4}$$

Another simple multiple linear regression was performed to correlate only UPV measurements to compressive strength as measured in the laboratory, based on the dataset with outliers removed. The derived equation is shown in Equation (5.5) predicted the model based on only UPV shear waves.

$$Compressive\ Strength = -60{,}54 + 16{,}19\ UPV \tag{5.5}$$

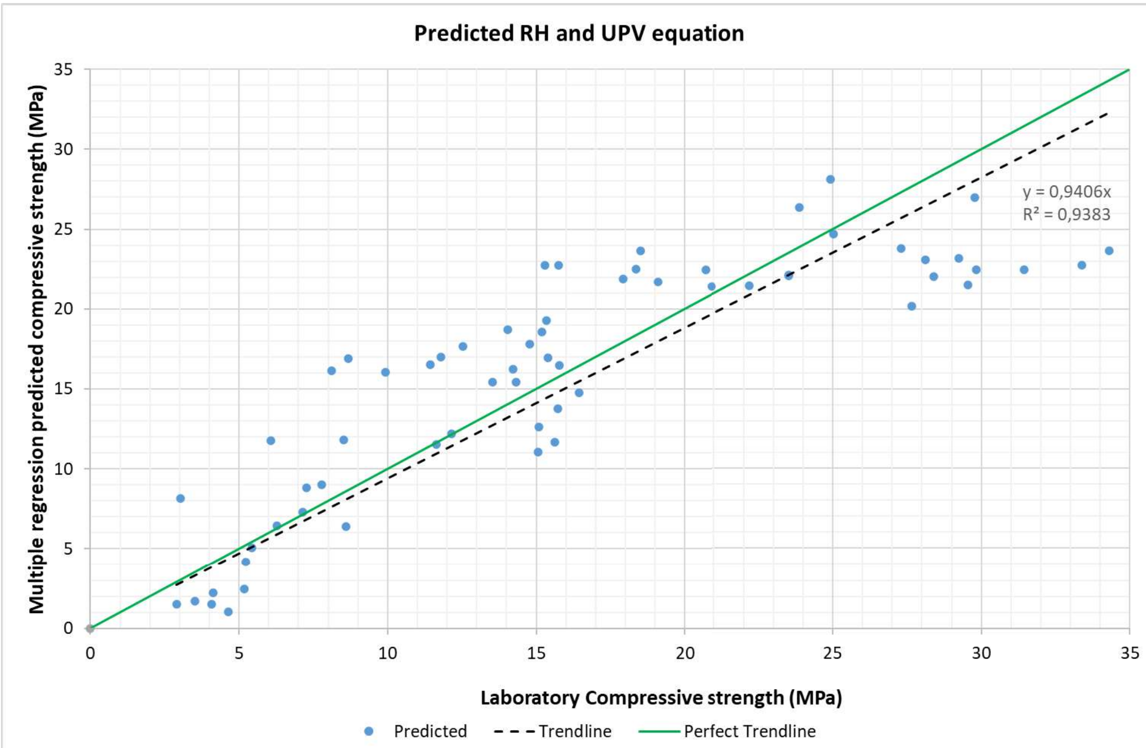

Figure 5.5 Multiple regression with only laboratory data

If the model was a perfect reflection of reality, the linear correlation would have had a fit with a slope of 1, intercept of 0 and R^2 of 1. The linear trend indicates that the model is slightly biased towards underprediction (slope <1), with an excellent R^2 of 0,93. Overall, the model explains 94% of the change in compressive strength as a function of the non-destructive methods evaluated, with an error in the region of 6% over the range evaluated.

If the accuracy of the non-destructive measurements is considered, it can be concluded that the model is an accurate prediction of real strength, with an estimated error of less than 10% and 94% accuracy.

The regression was then applied to field data collected from Spring Grove and De Hoop Dam, as discussed in Chapter 4.5.**Error! Reference source not found.** Dam field data in 1 standard deviation was taken (median value plus and or minus the std dev (x1) was used). The data collected on the steps of the dams were also discarded, reason for that is that the quality of the concrete on the steps is not good.

The steps of dams during construction are still during the fresh state of the concrete for shutter erecting as well as for access to the top of the dam wall. The steps might also not be finished off to the same quality as the concrete with a smooth shutter-finished face as the vertical face of the dam wall. Hence, the data collected on the steps of the dam were considered outliers.

The simple multiple linear regression performed is simulated strengths are graphically compared with measured in Figure 5.6, and compared with data from the field measurements. The best regression fit was performed using the data with the outliers removed to develop a linear correlation.

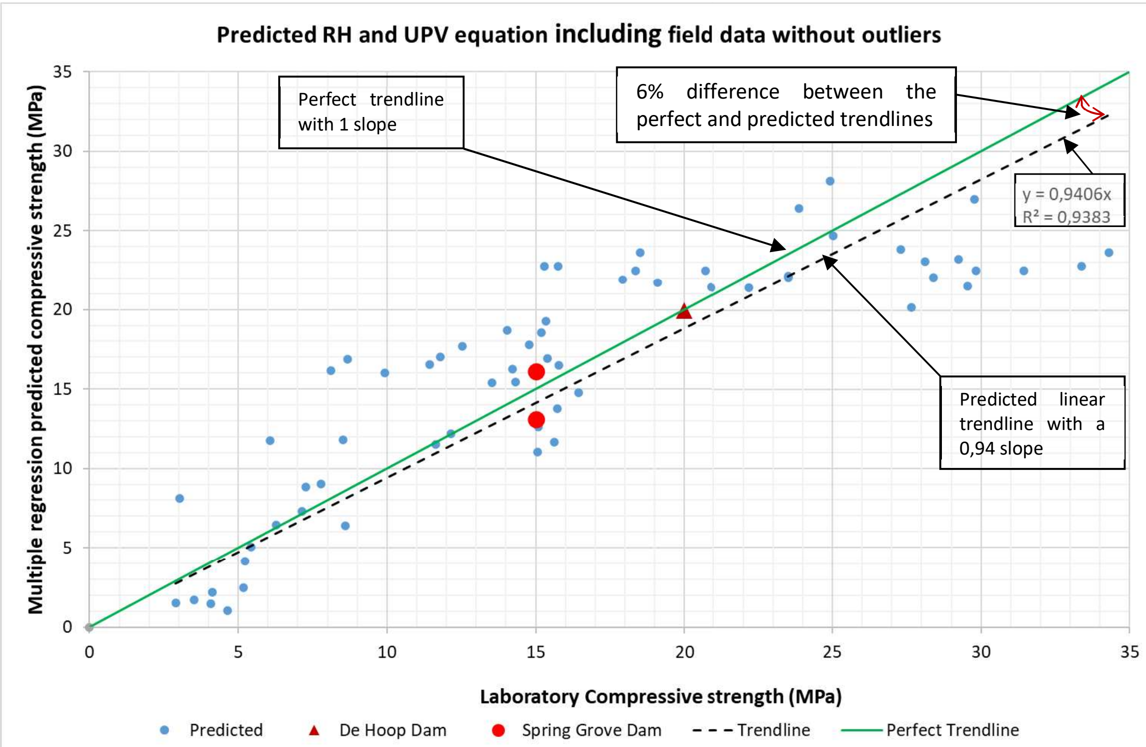

Figure 5.6 Indirect representation of all laboratory and field results through multiple regression

In Figure 5.6 the green line indicates the perfect trendline with a 1 slope, where if this was a perfect dataset the recorded measurements and the design compressive strength would have been the same values, therefore the predicted compressive strength would have been exactly the same as the design compressive strength. Unfortunately, this was not the case.

The predicted dam values were calculated in the multiple regression formula and plotted on the multiple regression graph indicated with the dash black line. This predicted trendline is slightly off the perfect green trendline. It can be that the actual RCC casted compressive strength was slightly higher or lower than that specified in the project specification. The difference between the perfect trendline and the predicted trendline is 6%. The predicted trendline is on a slope of 0,94, with a R^2 value of 0,94 as well.

The model explains that 99% of the De Hoop Dam compressive strength, as per the project specification, is a function of the non-destructive methods evaluated, with an error in the region of 1% over the range evaluated.

Further, the model indicates that for the Spring Grove dam, the compressive strength, as per the project specifications, is a function of the non-destructive methods evaluated, with error between -7% and 15% over the range evaluated.

5.5 Conclusion

The study involved a comprehensive analysis and interpretation of the results, aiming to establish a correlation between compressive strength (DT) and NDT such as UPV and RH. To achieve this, statistical techniques were employed to scrutinize and construct a model that delineated the relationship among various variables.

The investigation unveiled a positive correlation between the anticipated compressive strength derived from ultrasonic pulse velocity (UPV), relative humidity (RH), and laboratory-based compressive strength measurements. Overall, the model accounted for a remarkable 94% of the variance in compressive strength attributable to the non-destructive methods evaluated, demonstrating a margin of error within the vicinity of 6% across the assessed range.

Furthermore, the inclusion of field data in the model exhibited slight deviations from the established trendline, ranging from -7% to 15%. Nevertheless, this discrepancy still substantiates a commendable correlation and provides a means to estimate field compressive strength without resorting to destructive testing.

This correlation offers valuable applications in quality control and testing RCC dam concrete structures, ensuring that predicted strength aligns closely with the actual material performance. The robustness of the correlation attests to the reliability of these statistical measures in assessing the mechanical properties of concrete. The comprehensive dataset, including mean values, standard deviations, kurtosis, and skewness, contributes the understanding of the material behaviour, fostering informed decision making in the construction and engineering industries.

Chapter 6 Conclusions and recommendations

This research investigated the potential use of Non-Destructive Testing (NDT), specifically Ultrasonic Pulse Velocity (UPV) and Rebound Hammer (RH) testing, on Roller Compacted Concrete (RCC) for dam construction. The total cementitious content was varied to examine the mechanical properties of the concrete and determine its compressive strength. Additionally, the research seeks to establish correlations between NDT results and the compressive strength of the concrete. The testing of concrete characterisation is an industry need, to be conducted on all concrete and concrete types to produce concrete on a construction project, which effectively balances the requirements of productivity, time and cost and ensures the durability of structures. This need for a correlation between mix designs and NDT testing was the main reason for the research.

This research on NDT methods of testing RCC for dam applications could be utilised to form a basis for guidelines or specifications for using NDT testing methods on RCC for dam construction applications. Applying this research would lead to a reduction in cost, for cost of testing concrete quality during the construction phase of a project and also reduce the waste created by destructive testing of concrete.

The following conclusions and recommendations can be drawn from the results and findings from the experimental work.

6.1 Conclusions

Based on the results achieved from this experimental work the following conclusions are made:

1. The concrete mixtures have satisfied the fresh properties of RCC requirements regarding the Vebe time and sufficient paste.

2. Compressive strength results obtained for mixture 20/38-90 achieved the design compressive strength. Therefore, it is accepted that the results obtained for the UPV and RH can also be accepted with a degree of confidence.

3. For mixture 15/38-365, the compressive strength for 15/38-365 FA0% and 15/38-365 FA25% achieved the design compressive strength and therefor the UPV and RH results for these test mixtures can be accepted with a degree of confidence. 15/38-365 FA50% and 15/38-365 FA75% did not achieve the design compressive strength and it is also not predicted to reach 15MPa at 90 days.

4. The replacement of the total cement content with a ratio of fly ash, has effect on the workability of the concrete mixture, where up to 50% ration replacement of the total cement the workability of the concrete mixture is improved, but thereafter the workability reduces.

5. Variation in the total cementitious material content, as well as the different ratio of fly ash has a definite influence on the compressive strength of the concrete. The higher the total cementitious content in a concrete design, the higher the compressive strength, and the lower the total cementitious content the lower the compressive strength value. And when the total cement content is replaced on a ratio with fly ash, the compressive strength value is lower than for the same specimens with higher cement content.

6. The variation in the total cementitious materials content as well as the ratio of fly ash content in a concrete mixture has an influence on the UPV results. The higher the total cementitious materials in the concrete mixture the higher the tendency for high UPV readings. Where both mixture 20/38-90 and 15/38-365 the UPV readings was higher than for the mixtures with 25% replacement of fly ash than for both the mixtures on 100% cement content. However, the higher the ratio of fly ash replacement, lower UPV recordings measured.

7. A strong positive model were developed with an excellent R^2 of 0,93.

6.2 Recommendations

After conducting the testing procedure, collecting data, processing of results, the discussion and obtaining of conclusions on the work presented in this thesis, the following recommendations are made for future research on this topic:

1. With limited resource best possible results were obtained using two different labs. It is recommended that in an ideal testing situation, one laboratory be used to conduct all testing and another laboratory be used to verify and conduct checks on results.

2. For the purpose of this thesis and due to time constraints, compressive strength was tested for RCC for 7, 28 and 90. Further research may develop further understanding of the compressive strength development and correlation between compressive strength and non-destructive when testing at ages of RCC up to 365 days.

3. The prediction that mixtures 15/38-365 FA50% and 15/38-365 FA75% will reach design strength at 90 days, is not possible as it is uncertain on the long-term strength development influence of the aggregate and other factors that might influence the mixture (USACE, 2000).

4. There is a need to investigate the design mix of the mixture 15/38-365 FA50% and 15/38-365 FA75%, to adjust the aggregate, aggregate ratio, and water cement ratio of the mixture.

5. Further investigations are required for different RCC dam design mixes and the non-destructive testing thereof.

6. The developed model is best correlation between compressive strength and UPV (Longitudinal and shear Waves) and Rebound Hammer values. Various factors could have an influence on the compressive strength of concrete; therefore, the mixtures design, aggregate type, quality, and quantity needs to be revised and the model can be improve with addition of more data through testing.

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Appendices

Appendix A Approval to conduct research in the Department of Water and Sanitation in fulfilment of Master of Engineering in Civil Engineering

This letter gives approval to conduct research studies in the Department of Water and Sanitation.

water & sanitation

Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

B. Tshabalala	TshabalalaBa@dws.gov.za	021-8720591
021-8720593	Pers: T.Gouws 98759522	**DATE:** 23 June 2022

Acting Contract Manager: CD:PI
Department of water and sanitation
Private Bag X313
PRETORIA
0001

Attention: Mr. B. Tshabalala: Acting Contract Manager: Construction South

APPLICATION FOR APPROVAL TO CONDUCT RESEARCH IN THE DEPARTMENT OF WATER AND SANITATION IN FULFILLMENT OF MASTER OF ENGINEERING IN CIVIL ENGINEERING STUDIES
EMPLOYEE: ME T GOUWS: NUMBER 98759522

Attached please find a letter received from Me T Gouws in which she applies for approval to conduct research in the Department of Water and Sanitation in fulfilment of Master of Engineering in Civil Engineering studies.

I propose approval to conduct research in the Department of Water and Sanitation in fulfilment of Master of Engineering in Civil Engineering studies.

CVL 690 R _ RESEARCH PROJECT AND DISSERTATION

NAME: AC FRYLINCK
ASSISTANT CONTRACT MANAGER: CONSTRUCTION SOUTH
DATE: 23/6/2022

APPLICATION FOR APPROVAL TO CONDUCT RESEARCH IN THE DEPARTMENT OF WATER AND SANITATION IN FULFILLMENT OF MASTER OF ENGINEERING IN CIVIL ENGINEERING STUDIES
EMPLOYEE: ME T GOUWS: NUMBER 98759522 (Cont.)

(Dated 23/06/2022)

The granting approval to conduct research:

APPROVED / ~~NOT APPROVED~~

The employer hereby approves conduct research in the Department of Water and Sanitation in fulfilment of Master of Engineering in Civil Engineering Studies:

CVL 690 R _ RESEARCH PROJECT AND DISSERTATION

NAME: B. TSHABALALA
ACTING CONTRACT MANAGER: CONSTRUCTION SOUTH
DATE: 27 June 2022

Appendix B Cement Datasheet

PPC Cement SA (Pty) Ltd
PPC Building 148 Katherine Street
(Cnr Grayston Drive) Sandton Johannesburg
PO Box 787416 Sandton 2146 South Africa

Tel +27 11 386 9000
Fax +27 11 386 9001
www.ppc.co.za

Reg No 2009/005305/07

Riebeeck West SURETECH 52,5 N

Characteristic	Typical value	Specification: SANS 50197-1
Initial setting times (min)	110	≥ 45
Soundness (expansion) (mm)	1	≤10
Compressive strength:		
2 Days (MPa)	27.5	≥ 20
28 Days (MPa)	57.5	≥ 52.5
Chemical Properties of Clinker		
SiO_2 (%)	22.3	none
Al_2O_3 (%)	4.4	none
Fe_2O_3 (%)	3.3	none
CaO (%)	67.2	none
C_3S	58	none
C_2S	21	none
C_3A	7	none
C_4AF	10	none
Chemical Properties of Cement		
MgO (%)	0.8	none
SO_3 (%)	2.1	≤3.5
Na_2Oeq (%)	0.55	none
Chloride (%)	0.02	≤0.10

Comments:
Main constituents: Clinker and Limestone (5%)

……………………………………

Denzil Esau

Quality Assurance Chemist

Figure B.1 Data sheet for PPC Portland cement CEM I: Riebeeck West Suretech 52.5N.

Appendix C Aggregate Grading Results

This chapter provides all the laboratory records for the aggregates grading results. The data will be provided per aggregate size. For the 20mm aggregate and river sand, intensive tests were done. Due to limited time and small quantity of the 5 mm and 10mm aggregate were use only sieve analysis were conducted for 5mm and 10mm aggregate.

LAB JOB NO:		LAB SERIAL NO:		SAMPLE ID:	
SAMPLE DESCRIPTION:	SAND				

NORMAL MAX AGGREGATES SIZE (mm)	4.75
MINIMUM SAMPLE SIZE (kg)	1.0

PROCEDURE	6.1	Sieving and washing for the determination of the sieve analysis, dust content and fines modulus

Original Lab. Sample mass with container (g)	1976.0
Oven dry sample mass with container (g)	1974.0
Oven dry sample mass with container - after washing through the 0.075 mm and drying (g)	1972.0
Mass of container alone (g)	672.0 PAN3
Moisture content %	0.1%

Mass passing 0.075 mm after sieving (g)	45.5	SIZE VALIDITY	
		TEST VALIDITY	

Sieve Size mm	Mass Retained (g)	% Retained	% Passing	Bottom Spec % <	Top Spec % <
75.0					
53.0					
38.0					
26.0					
19.0					
13.2					
9.5					
6.7					
4.8					
2.36					
1.18	1.5	0.1	100.0		
0.600	140.0	10.8	89.2		
0.300	436.5	33.7	55.5		
0.150	415.5	32.1	23.4		
0.075	255.0	19.7	3.7		
< 0.075	47.5	3.7			
TOTAL	1296.0				

FM	
TYPE	
SHAPE	
TEXTURE	
REMARKS	

DUST CONTENT =	%

Figure C.1 Sieve analysis, fines content and dust content of river sand sample 1.

LAB JOB NO:		LAB SERIAL NO:		SAMPLE ID:	
SAMPLE DESCRIPTION:					

NORMAL MAX AGGREGATES SIZE (mm)	4.75
MINIMUM SAMPLE SIZE (kg)	1.0

PROCEDURE | 6.1 | Sieving and washing for the determination of the sieve analysis, dust content and fines modulus

Original Lab. Sample mass with container (g)	2030.0
Oven dry sample mass with container (g)	2028.0
Oven dry sample mass with container: - after washing through the 0.075 mm and drying (g)	2026.0
Mass of container alone (g)	726.0 PAN 5
Moisture content %	0.1%

Mass passing 0.075 mm after sieving (g)	56.5

SIZE VALIDITY	
TEST VALIDITY	

Sieve Size mm	Mass Retained (g)	% Retained	% Passing	Bottom Spec % <	Top Spec % <
75.0					
53.0					
38.0					
26.0					
19.0					
13.2					
9.5					
6.7					
4.8					
2.36					
1.18	1.0	0.1	99.9		
0.600	129.0	10.0	89.9		
0.300	399.0	30.8	59.1		
0.150	391.0	30.2	28.9		
0.075	317.5	24.5	4.4		
< 0.075	56.5	4.4			
TOTAL	1294.0				

FM	
TYPE	
SHAPE	
TEXTURE	
REMARKS	

DUST CONTENT = | %

Figure C.2   Sieve analysis, fines content and dust content of river sand sample 2.

LAB JOB No:		LAB SERIAL No:		SAMPLE ID:	
Sample Description :					

METHOD | P

RESULT NEEDED | RD

WETTING AGENT | W

		TEST 1	TEST 2
Mass of the saturated surface-dry aggregate, in grams	Ma		
Mass of oven dried aggregate, in grams	Mb	265.0	278.0
Mass of pycnometer plus aggrgate and water in grams	Mc	1779.5	1734.5
Mass of the pycnometer filled with water, in grams	Md	1607.5	1559.5
Test temperature, in ° Centigrade	T°C	22°C	22°C
density of water in g/cc	g / cc	0.99780	0.99780
Relative density of aggregate		2.84	2.69
	diff	0.15	
	Average	2.76	

Figure C.3   Relative density of river sand for sample 1 and sample 2.

LAB JOB No :		LAB SERIAL No :		SAMPLE ID	
Sample Description					

PROCEDURE 4,1

		SAMPLE A	SAMPLE B
HYGROSCOPIC MASS OF SAMPLE AS RECEIVED (g)	a	1670.0	1668.0
MASS AFTER WATER ABSORPTION FOR 24 HOURS AND SURFACE DRIED (g)	b	1686.5	1684.5
MASS OVEN DRIED TO CONSTANT MASS (g)	c	1663.0	1660.5
MASS HYGROSCOPIC WATER (g)	(a-c) = d	7.0	7.5
MASS ABSORBED WATER (g)	(b-c) = e	23.5	24.0
HYGROSCOPIC MOISTURE CONTENT %	(d/c)*100 = f	0.4	0.4
% ABSORPTION	(e/c)*100 = g	1.4	1.4
AVERAGE HYGROSCOPIC MOISTURE CONTENT %	average of f	0.4	
AVERAGE ABSORPTION MOISTURE CONTENT %	average of g	1.4	

Figure C.4 Water absorption for sample 1 and sample 2 of river sand

LAB JOB NO:		LAB SERIAL NO:		SAMPLE ID:	W 1046 (WTE) Item 1
SAMPLE DESCRIPTION:	19.0 MM STONE				

NORMAL MAX AGGREGATES SIZE (mm)	19.00
MINIMUM SAMPLE SIZE (kg)	3.8

PROCEDURE	7.3	Sieving and washing for the determination of the sieve analysis only

Original Lab. Sample mass with container (g)	4498.0	
Oven dry sample mass with container (g)	4492.0	
Oven dry sample mass with container: - after washing through the 4.75 mm and oven dried (g)	4478.0	
Mass of container alone (g)	690.0	PAN 2
Moisture content %	0.2%	

Mass passing 4.75 mm after sieving (g)	8.0

SIZE VALIDITY	
TEST VALIDITY	

Sieve Size mm	Mass Retained (g)	% Retained	% Passing	Bottom Spec % <	Top Spec %<
75.0					
53.0					
38.0					
26.0					
19.0	174.0	4.6	95.4		
13.2	1882.0	49.5	45.9		
9.5	1326.0	34.9	11.0		
6.7	356.0	9.4	1.6		
4.8	38.0	1.0	0.6		
< 4.75	22.0	0.6			
TOTAL	3798.0				

FM	
TYPE	
SHAPE	
TEXTURE	
REMARKS	

Figure C.5 Sieve analysis, fines content and dust content of 20mm aggregate sample 1.

LAB JOB NO:		LAB SERIAL NO:		SAMPLE ID:	W 1046 (WTE) Item 1
SAMPLE DESCRIPTION:	19.0 MM STONE				

NORMAL MAX AGGREGATES SIZE (mm)	19.00
MINIMUM SAMPLE SIZE (kg)	3.8

PROCEDURE	7.3	Sieving and washing for the determination of the sieve analysis only

Original Lab. Sample mass with container (g)	4514.0
Oven dry sample mass with container (g)	4510.0
Oven dry sample mass with container: - after washing through the 4.75 mm and oven dried (g)	4494.0
Mass of container alone (g)	714.0 PAN1
Moisture content %	0.1%

Mass passing 4.75 mm after sieving (g)	9.0	SIZE VALIDITY	
		TEST VALIDITY	

Sieve Size mm	Mass Retained (g)	% Retained	% Passing	Bottom Spec % <	Top Spec %<
75.0					
53.0					
38.0					
26.0					
19.0	186.0	4.9	95.1		
13.2	1294.0	34.3	60.8		
9.5	1606.0	42.5	18.3		
6.7	588.0	15.6	2.7		
4.8	76.0	2.0	0.7		
< 4.75	25.0	0.7			
TOTAL	3775				

FM	
TYPE	
SHAPE	
TEXTURE	
REMARKS	

Figure C.6 Sieve analysis, fines content and dust content of 20mm aggregate sample 2.

LAB JOB No:		LAB SERIAL No:		SAMPLE ID:	
Sample Description :					

METHOD	P
RESULT NEEDED	RD
WETTING AGENT	W

		TEST 1	TEST 2
Mass of the saturated surface-dry aggregate, in grams	Ma		
Mass of oven dried aggregate, in grams	Mb	469.0	455.0
Mass of pycnometer plus aggrgate and water in grams	Mc	1878.0	1868.0
Mass of the pycnometer filled with water, in grams	Md	1592.0	1590.0
Test temperature, in ° Centigrade	T°C	22°C	22°C
density of water in g/cc	g / cc	0.99780	0.99780
Relative density of aggregate		2.6	2.6
	crit	0.0	
	Average		

Figure C.7 Relative density of 20mm aggregate for sample 1 and sample 2 20mm.

LAB JOB No :		LAB SERIAL No :	SAMPLE ID
Sample Description			

PROCEDURE 4.2

		SAMPLE A	SAMPLE B
HYGROSCOPIC MASS OF SAMPLE AS RECEIVED (g)	a	2002.0	2000.0
MASS AFTER WATER ABSORPTION FOR 24 HOURS AND SURFACE DRIED (g)	b	1970.0	1975.5
MASS OVEN DRIED TO CONSTANT MASS (g)	c	1958.0	1963.0
MASS HYGROSCOPIC WATER (g)	(a-c) = d	44.0	37.0
MASS ABSORBED WATER (g)	(b-c) = e	12.0	12.5
HYGROSCOPIC MOISTURE CONTENT %	(d/c)*100 = f	2.2	1.9
% ABSORPTION	(e/c)*100 = g	0.6	0.6
AVERAGE HYGROSCOPIC MOISTURE CONTENT %	average of f	2.05	
AVERAGE ABSORPTION MOISTURE CONTENT %	average of g	0.6	

Figure C.8 Water absorption for 20mm aggregate sample 1 and sample 2.

LAB JOB NO:		LAB SERIAL NO:		SAMPLE ID:	
SAMPLE DESCRIPTION:		SAMPLE NUMBER:			

PASSING	RETAINED	SLOT	MASS	MASS RETAINED	MASS PASSING
53	37.5	26.5	7500		
37.5	26.5	18.75	2500		
26.5	19.0	13.25	230 1000	0,210	0,020
19	13.2	9.5	1690 500	1,602	0,288
13.2	9.5	6.6	120	0,110	0,010
9.5	6.7	4.75	50	0,044	0,006
6.7	4.75	3.35	20	0,016	0,004

SIEVE SIZE	% RETAINED	% PASSING	INDEX
53			
37.5			
26.5			
19	2.1	8.1	0.2
13.2	69.8	10.2	0.7
9.5	37.6	8.3	3.1
6.7	7.4	12.0	0.9
	FLAKINESS INDEX		4.9

Figure C.9   Flakiness of 20mm aggregate.

LAB JOB NO:		LAB SERIAL NO:		SAMPLE ID:	
SAMPLE DESCRIPTION:	10mm				

NORMAL MAX AGGREGATES SIZE (mm)	4.75
MINIMUM SAMPLE SIZE (kg)	1.0

PROCEDURE	6.1	Sieving and washing for the determination of the sieve analysis, dust content and fines modulus

Original Lab. Sample mass with container (g)	1273.5
Oven dry sample mass with container (g)	1273.0
Oven dry sample mass with container. - after washing through the 0.075 mm and drying (g)	
Mass of container alone (g)	272.0
Moisture content %	

Mass passing 0.075 mm after sieving (g)	5.4

SIZE VALIDITY	
TEST VALIDITY	

Sieve Size mm	Mass Retained (g)	% Retained	% Passing	Bottom Spec % <	Top Spec % <
75.0					
53.0					
38.0					
26.0					
19.0					
13.2					
9.5					
6.7	218.4				
4.8	259.9				
2.36	454.1				
1.18	54.5				
0.600	2.9				
0.300	1.1				
0.150	0.8				
0.075	2.6				
< 0.075	5.4				
TOTAL					

FM	
TYPE	
SHAPE	
TEXTURE	
REMARKS	

DUST CONTENT =	%

Figure C.10   Sieve analysis, fines content and dust content of 10mm aggregate sample.

LAB JOB NO:		LAB SERIAL NO:		SAMPLE ID:	
SAMPLE DESCRIPTION:	5 mm CRUSHER DUST				

NORMAL MAX AGGREGATES SIZE (mm)	4.75
MINIMUM SAMPLE SIZE (kg)	1.0

PROCEDURE	6.1	Sieving and washing for the determination of the sieve analysis, dust content and fines modulus

Original Lab. Sample mass with container (g)	772.6
Oven dry sample mass with container (g)	772.2
Oven dry sample mass with container: - after washing through the 0.075 mm and drying (g)	
Mass of container alone (g)	272.5
Moisture content %	

Mass passing 0.075 mm after sieving (g)	30.8

SIZE VALIDITY	
TEST VALIDITY	

Sieve Size mm	Mass Retained (g)	% Retained	% Passing	Bottom Spec % <	Top Spec % <
75.0					
53.0					
38.0					
26.5					
19.0					
13.2					
9.5					
6.7					
4.8					
2.36	194.0				
1.18	209.7				
0.600	36.9				
0.300	12.7				
0.150	6.5				
0.075	10.6				
< 0.075	30.8				
TOTAL					

FM	

TYPE	
SHAPE	
TEXTURE	
REMARKS	

DUST CONTENT =	%

Figure C.11 Sieve analysis, fines content and dust content of 5mm aggregate sample.

Appendix D Mixtures

This chapter provides all the laboratory records for the first mixtures before mixtures where optimised. aggregates grading results. The data will be provided per 20/38-90 and 15/38-365 mixture design.

Table D.1 RCC design 20/38-90 mix proportions for 0,01 m^3

Material	**Experiment Mix Number**			
	20/38-90 (/1m^3)	**MIX 1:20/38-90 (/0,01m^3)**	**MIX 2: 20/38-90 (/0,01m^3)**	**MIX 3: 20/38-90 (/0,01m^3)**
Cement (kg)	62	0,62	0,62	0,62
FA (kg)	145	1,45	1,45	1,45
Tota Cementitious content	207	2,07	2,07	2,07
Aggregate (19-38 mm) (kg)	883	13	13	13
Aggregate (5-19 mm) (kg)	483	0,68	0,68	0,68
Total Coarse Aggregate	1366	13,66	13,66	13,66
5 mm Aggerate (kg)		4,385	3,51	6,14
River sand (kg)		4,385	5,26	2,63
Total Fine Aggregate	877	8,77	8,77	8,77
Water (l)	122	1,35	1,3	1,3
W/C	0,589	0,6	0,63	0,63
Admixture (kg)	0,8	15 ml	15 ml	30 ml
Vebe (sec)		20	10	10

Table D.2 RCC design 15/38-365 mix proportions for 0,01 m^3

Material	Experiment Mix Number			
	15/38-365 (/1m^3)	MIX 1:15/38-365 (/0,01m^3)	MIX 2: 15/38-365 (/0,01m^3)	MIX 3: 15/38-365 (/0,01m^3)
Cement (kg)	50	0,5	0,5	0,5
FA (kg)	110	1,1	1,1	1,1
Tota Cementitious content	160	1,6	1,6	1,6
Aggregate (19-38 mm) (kg)	537	9,97	9,97	9,97
Aggregate (5-19 mm) (kg)	512	0,53	0,53	0,53
Total Coarse Aggregate	1049	10,49	10,49	10,49
5 mm Aggerate (kg)	537	6,42	7,7	6,42
River sand (kg)	747	6,42	5,1	6,42
Total Fine Aggregate	1284	12,84	12,84	12,84
Water (l)	118	1,3	1,5	1,18
W/C	0,7375	0,81	1,07	0,74
Admixture (kg)	0,8	15 ml	15 ml	15 ml
Vebe (sec)	-	20-22	17	20

Appendix E Ultrasonic Pulse Velocity Results

This chapter provides all the Ultrasonic pulse velocity (UPV) records and results. The data will be provided in tables per trial mixture.

Concrete mixtures were made with crushed Dolomite aggregate. Aggregate range between 20 mm to 5 mm size were used and river sand, together with, PPC Portland 52,5 N Riebeeck West Suretech cement and Plast RCC plasticizer. Four different mixtures were mix with different ratio of Fly ash per mixture.

Mixture 20/38-90 was made with the following aggregate ratio: 1366 kg/m^3 coarse aggregate and 877 kg/m^3 fine aggregate.

In Table D.1 RCC mixture 20/38-90 FA0% with 0% Fly ash cementitious content, 100 % PPC Portland composite cement and water cement ratio of 0,65 ultrasonic pulse velocity data is presented. The 7th day set of specimens, which is 20/38-90 FA0%.1, 20/38-90 FA0%,2 and 20/38-90 FA0%.3 signal pass were not 100 %, with no signal pass recorded for specimen 20/38-90 FA0%.1 first reading. Values range between 5415 m/s and 5703 m/s. For 14th day, specimen 20/38-90 FA0%.4, 20/38-90 FA0%.5 and 20/38-90 FA0%.6 had 100% signal pass, with lowest value recorded 5495 m/s and highest 5814 m/s. The 28th ultrasonic pulse velocity values on specimens 20/38-90 FA0%.7, 20/38-90 FA0%.8 and 20/38-90 FA0%.9 had 100% pass signal with values recorded between 5495 m/s and 5703 m/s.

Table E.1 Mixture 20/38-90 FA0% – Direct transmission UPV results

Specimen Identification	Direct transmission UPV					
Number	**Signal pass %**	**(m/s)**	**µs**	**Signal pass %**	**(m/s)**	**µs**
20/38-90 FA0%.1	-	5415	27.7	37%	5576	26.9
20/38-90 FA0%.2	100%	5556	27.0	77%	5618	26.7
20/38-90 FA0%.3	100%	5660	26.5	100%	5703	26.3
20/38-90 FA0%.4	100%	5495	27.3	100%	5703	26.3
20/38-90 FA0%.5	100%	5682	26.4	100%	5814	25.8
20/38-90 FA0%.6	100%	5535	27.1	100%	5639	26.6
20/38-90 FA0%.7	100%	5495	27.3	100%	5618	26.7
20/38-90 FA0%.8	100%	5576	26.9	100%	5576	26.9
20/38-90 FA0%.9	100%	5576	26.9	100%	5703	26.3

In Table D.2 RCC mixture 20/38-90 FA25% with 25% Fly ash cementitious content, 75 % PPC Portland composite cement and water cement ratio of 0,64 ultrasonic pulse velocity data is presented. The 7th day set of specimens, which is 20/38-90 FA25%.1, 20/38-90 FA25%.2 and 20/38-90 FA25%.3 signal pass were 100 % except for one recording on 20/38-90 FA25%.1, where values range between 5597 m/s and 5814 m/s. For 14th day, specimen 20/38-90 FA25%.4, 20/38-90 FA25%.5 and 20/38-90 FA25%.6 had 100% signal pass, with lowest value recorded 5576 m/s and highest 5952 m/s. The 28th ultrasonic pulse velocity values on specimens 20/38-90 FA25%.7, 20/38-90 FA25%.8 and 20/38-90 FA25%.9 had 100% pass signal with values recorded between 4261 m/s and 5660 m/s. For specimen 20/38-90 FA25%.7 UPV result recorded of 4261 m/s in 35,2 µs can be seen as outlier. The reason for this outlier results, might be due to air void inside the specimen, or the compaction of the specimen and or could have been the contact between the transducer and concrete.

Table E.2 Mixture 20/38-90 FA 25% - Direct transmission UPV results

Specimen Identification	Direct transmission UPV					
Number	**Signal pass %**	**(m/s)**	**µs**	**Signal pass %**	**(m/s)**	**µs**
20/38-90 FA25%.1	100%	5682	26.4	98%	5769	26.0
20/38-90 FA25%.2	100%	5682	26.4	100%	5814	25.8
20/38-90 FA25%.3	100%	5597	26.8	100%	5814	25.8
20/38-90 FA25%.4	100%	5618	26.7	100%	5792	25.9
20/38-90 FA25%.5	100%	5576	26.9	100%	5597	26.8
20/38-90 FA25%.6	100%	5703	26.3	100%	5952	25.2
20/38-90 FA25%.7	100%	4261	35.2	100%	5376	27.9
20/38-90 FA25%.8	100%	5576	26.9	100%	5639	26.6
20/38-90 FA25%.9	100%	5455	27.5	100%	5660	26.5

In Table D.3 RCC mixture 20/38-90 FA50% with 50% Fly ash cementitious content, 50 % PPC Portland composite cement and water cement ratio of 0,63 ultrasonic pulse velocity data is presented. The 7th day set of specimens, which is 20/38-90 FA50%.1, 20/38-90 FA50%.2 and 20/38-90 FA50%.3 signal pass were 100 % except for one recording on 20/38-90 FA50%.2, where values range between 5474 m/s and 5703 m/s. For 14th day, specimen 20/38-90 FA50%.4, 20/38-90 FA50%.5 and 20/38-90 FA50%.6 had 100% signal pass, with lowest value recorded 5357 m/s and highest 5576 m/s. The 28th ultrasonic pulse velocity values on specimens 20/38-90 FA50%.7, 20/38-90 FA50%.8 and 20/38-90 FA50%.9 had 100% pass signal with values recorded between 4777 m/s and 5495 m/s. For specimen 20/38-90 FA50%.7 UPV result recorded of 4777 m/s in 31,4 µs can be seen as outlier. The reason for this outlier results, might be due to air void inside the specimen, or the compaction of the specimen and or could have been the contact between the transducer and concrete.

Table E.3 Mixture 20/38-90 FA 50% - Direct transmission UPV results

Specimen Identification	Direct transmission UPV					
Number	**Signal pass %**	**(m/s)**	**µs**	**Signal pass %**	**(m/s)**	**µs**
20/38-90 FA50%.1	100%	5474	27.4	100%	5597	26.8
20/38-90 FA50%.2	100%	5495	27.3	77%	5660	26.5
20/38-90 FA50%.3	100%	5703	26.3	100%	5597	26.8
20/38-90 FA50%.4	100%	5515	27.2	100%	5474	27.4
20/38-90 FA50%.5	100%	5455	27.5	100%	5576	26.9
20/38-90 FA50%.6	100%	5396	27.8	100%	5357	28.0
20/38-90 FA50%.7	100%	5396	27.6	100%	4777	31.4
20/38-90 FA50%.8	100%	5495	27.3	100%	5474	27.4
20/38-90 FA50%.9	100%	5474	27.4	100%	5455	27.5

In Table D.4 RCC mixture 20/38-90 FA75% with 25% Fly ash cementitious content, 75 % PPC Portland composite cement and water cement ratio of 0,64 ultrasonic pulse velocity data is presented. The 7th day set of specimens, which is 20/38-90 FA75%.1, 20/38-90 FA75%.2 and 20/38-90 FA75%.3 signal pass range between 58% - 100%, where values range between 4950 m/s and 5300 m/s. For 14th day, specimen 20/38-

90 FA75%.4, 20/38-90 FA75%.5 and 20/38-90 FA75%.6 had 100% signal pass, with lowest value recorded 5034 m/s and highest 5300 m/s. The 28th ultrasonic pulse velocity values on specimens 20/38-90 FA75%.7, 20/38-90 FA75%.8 and 20/38-90 FA75%.9 had 100% pass signal with values recorded between 5137 m/s and 5396 m/s.

Table E.4 Mixture 20/38-90 FA 75% - Direct transmission UPV results

Specimen Identification	**Direct transmission UPV**					
Number	**Signal pass %**	**(m/s)**	**µs**	**Signal pass %**	**(m/s)**	**µs**
20/38-90 FA75%.1	81%	5068	29.6	100%	5172	29.0
20/38-90 FA75%.2	76%	5226	28.7	100%	5300	28.3
20/38-90 FA75%.3	75%	4950	30.3	58%	5172	29.0
20/38-90 FA75%.4	100%	5155	29.1	100%	5300	28.3
20/38-90 FA75%.5	100%	5068	29.6	100%	5155	29.1
20/38-90 FA75%.6	100%	5034	29.8	100%	5300	28.3
20/38-90 FA75%.7	100%	5226	28.7	100%	5245	28.6
20/38-90 FA75%.8	100%	5137	29.2	100%	5396	27.8
20/38-90 FA75%.9	100%	5226	28.7	100%	5338	28.1

Mixture 15/38-365 was made with the following aggregate ratio: 1100 kg/m^3 coarse aggregate and 1200 kg/m^3 fine aggregate.

In Table D.5 RCC mixture 15/38-365 FA0% with 0% Fly ash cementitious content, 100 % PPC Portland composite cement and water cement ratio of 0,84 ultrasonic pulse velocity data is presented. The 7th day set of specimens, which is 15/38-365 FA0%.1, 15/38-365 FA0%,2 and 15/38-365 FA0%.3 signal pass was not all 100 %. Values range between 4688/s and 5068 m/s. For 14th day, specimen 15/38-365 FA0%.4, 15/38-365 FA0%.5 and 15/38-365 FA0%.6 had 100% signal pass, with lowest value recorded 4918 m/s and highest 5725 m/s. The 28th ultrasonic pulse velocity values on specimens 15/38-365 FA0%.7, 15/38-365 FA0%.8 and 15/38-365 FA0%.9 had 100% pass signal with values recorded between 4673 m/s and 6696 m/s.

Table E.5 Mixture 15/38-365 FA0% - Direct transmission UPV results

Specimen Identification	**Direct transmission UPV**					
Number	**Signal pass %**	**(m/s)**	**µs**	**Signal pass %**	**(m/s)**	**µs**
15/38-365 FA0%.1	100%	5068	29.6	100%	5034	29.8
15/38-365 FA0%.2	100%	5068	29.6	52%	4688	32
15/38-365 FA0%.3	100%	4870	30.8	80%	4777	31.4
15/38-365 FA0%.4	100%	4918	30.5	100%	5415	27.7
15/38-365 FA0%.5	100%	5725	26.2	100%	5172	29
15/38-365 FA0%.6	100%	5068	29.6	100%	5051	29.7
15/38-365 FA0%.7	100%	5155	29.1	100%	4870	30.8
15/38-365 FA0%.8	100%	4673	32.1	100%	4983	30.1
15/38-365 FA0%.9	100%	5263	28.5	100%	6696	22.4

In Table D.6 RCC mixture 15/38-365 FA25% with 25% Fly ash cementitious content, 75 % PPC Portland composite cement and water cement ratio of 0,81 ultrasonic pulse velocity data is presented. The 7th day set of specimens, which is 15/38-365 FA25%.1, 15/38-365 FA25%,2 and 15/38-365 FA25%.3 signal pass was not 100 %. Values range between 4717 m/s and 5226 m/s. For 14th day, specimen 15/38-365 FA25%.4, 15/38-365 FA25%.5 and 15/38-365 FA25%.6 had 100% signal pass, with lowest value recorded 4918 m/s and highest 6757 m/s. The 28th ultrasonic pulse velocity values on specimens 15/38-365 FA25%.7, 15/38-365 FA25%.8 and 15/38-365 FA25%.9 had 100% pass signal with values recorded between 4717 m/s and 5155 m/s.

Table E.6 Mixture 15/38-365 FA25% - Direct transmission UPV results

Specimen Identification	**Direct transmission UPV**					
Number	**Signal pass %**	**(m/s)**	**µs**	**Signal pass %**	**(m/s)**	**µs**
15/38-365 FA25%.1	100%	5226	28.7	100%	5137	29.2
15/38-365 FA25%.2	100%	4983	30.1	74%	4717	31.8
15/38-365 FA25%.3	100%	4934	30.4	80%	4983	30.1
15/38-365 FA25%.4	100%	5226	28.7	100%	4918	30.5
15/38-365 FA25%.5	100%	5051	29.7	100%	6757	22.2
15/38-365 FA25%.6	100%	5226	28.7	100%	5102	29.4
15/38-365 FA25%.7	100%	5000	30	100%	4886	30.7
15/38-365 FA25%.8	100%	4839	31	100%	5155	29.1
15/38-365 FA25%.9	100%	4717	31.8	100%	5137	29.2

In Table D.7 RCC mixture 15/38-365 FA50% with 50% Fly ash cementitious content, 50 % PPC Portland composite cement and water cement ratio of 0,72 ultrasonic pulse velocity data is presented. The 7th day set of specimens, which is 15/38-365 FA50%.1, 15/38-365 FA50%,2 and 15/38-365 FA50%.3 signal pass was between 42& and 57%. It was difficulted to obtain these results and might have been due to the concrete in it earlier curing ages, with high fly-ash content, rough surface, might even be due to poor connection between the concrete and UPV probes. These values range between 4225/s and 4491 m/s. For 14th day, specimen 15/38-365 FA50%.4, 15/38-365 FA50%.5 and 15/38-365 FA50%.6 had 100% signal pass, with lowest value recorded 4348 m/s and highest 4673 m/s. The 28th ultrasonic pulse velocity values on specimens 15/38-365 FA50%.7, 15/38-365 FA50%.8 and 15/38-365 FA50%.9 had 100% pass signal, except for 15/38-365 FA50%.7 with 97% pass signal, with values recorded between 4545 m/s and 5435 m/s.

Table E.7 Mixture 15/38-365 FA50% - Direct transmission UPV results

Specimen Identification	Direct transmission UPV					
Number	**Signal pass %**	**(m/s)**	**µs**	**Signal pass %**	**(m/s)**	**µs**
15/38-365 FA50%.1	43%	4425	33.9	42%	4491	33.4
15/38-365 FA50%.2	52%	4323	34.7	42%	4225	35.5
15/38-365 FA50%.3	50%	4360	34.4	57%	4425	33.9
15/38-365 FA50%.4	100%	4518	33.2	100%	4673	32.1
15/38-365 FA50%.5	100%	4630	32.4	100%	4587	32.7
15/38-365 FA50%.6	100%	4348	34.5	100%	4505	33.3
15/38-365 FA50%.7	100%	5435	27.6	97%	4559	32.9
15/38-365 FA50%.8	100%	4545	33	100%	4688	32
15/38-365 FA50%.9	100%	4644	32.3	100%	4587	32.7

In Table D.8 RCC mixture 15/38-365 FA75% with 75% Fly ash cementitious content, 25 % PPC Portland composite cement and water cement ratio of 0,72 ultrasonic pulse velocity data is presented. The 7th day set of specimens, which is 15/38-365 FA75%.1, 15/38-365 FA75%,2 and 15/38-365 FA75%.3 signal pass was between 17 % to 45%. Values range between 3464/s and 3695 m/s. It was difficulted to obtain these results and might have been due to the concrete in it earlier curing ages, with high fly-ash content, rough surface, might even be due to poor connection between the concrete and UPV probes. For 14th day, specimen 15/38-365 FA75%.4, 15/38-365 FA75%.5 and 15/38-365 FA75%.6 had 100% signal pass, with lowest value recorded 4076 m/s and highest 4983 m/s. The 28th ultrasonic pulse velocity values on specimens 15/38-365 FA75%.7, 15/38-365 FA75%.8 and 15/38-365 FA75%.9 had 100% pass signal, except for 15/38-365 FA75%.8 and 15/38-365 FA75%.9 record 97% signal pass, with values recorded between 4065 m/s and 4573 m/s.

Table E.8 Mixture 15/38-365 FA75% - Direct transmission UPV results

Specimen Identification	Direct transmission UPV					
Number	**Signal pass %**	**(m/s)**	**µs**	**Signal pass %**	**(m/s)**	**µs**
15/38-365 FA75%.1	35%	3464	43.2	45%	3580	41.9
15/38-365 FA75%.2	22%	3731	40.2	17%	3641	41.2
15/38-365 FA75%.3	43%	3695	40.6	43%	3555	42.2
15/38-365 FA75%.4	100%	4076	36.8	100%	4110	36.5
15/38-365 FA75%.5	100%	4237	35.4	100%	4983	30.1
15/38-365 FA75%.6	100%	4098	36.6	100%	4190	35.8
15/38-365 FA75%.7	100%	4335	34.6	100%	4202	35.7
15/38-365 FA75%.8	97%	4167	36	100%	4178	35.9
15/38-365 FA75%.9	100%	4065	36.9	97%	4573	32.8

Appendix F Rebound Hammer Results

This chapter provides all the Rebound hammer (RH) number records and results. The summary of the recorded results, that was calculated in accordance with ASTM C 805-02 will be provided in tables per trial mixture.

Concrete mixtures were made with crushed Dolomite aggregate. Aggregate range between 20 mm to 5 mm size were used and river sand, together with, PPC Portland 52,5 N Riebeeck West Suretech cement and Plast RCC plasticizer. Four different mixtures were mix with different ratio of Fly ash per mixture.

Mixture 20/38-90 was made with the following aggregate ratio: 1366 kg/m^3 coarse aggregate and 877 kg/m^3 fine aggregate.

In Table E 1 the summary of the rebound number where 20/38-90 FA0%.1, 20/38-90 FA0%.2 and 20/38-90 FA0%.3 represent 7 days ageing results, 20/38-90 FA0%.4, 20/38-90 FA0%.5 and 20/38-90 FA0%.6 were tested on 14 days and 20/38-90 FA0%.7, 20/38-90 FA0%.8 and 20/38-90 FA0%.9 tested on 28 days.

Table F.1 Mixture 20/38-90 FA0% - Summary of Rebound number per specimen and aging day

Specimen Nr.	**Rebound Number**	
	Average per specimen	**Average per curing age day**
20/38-90 FA0%.1	19	18
20/38-90 FA0%.2	17	
20/38-90 FA0%.3	17	
20/38-90 FA0%.4	20	21
20/38-90 FA0%.5	22	
20/38-90 FA0%.6	21	
20/38-90 FA0%.7	20	21
20/38-90 FA0%.8	21	
20/38-90 FA0%.9	21	

In Table E 2 the summary of the rebound number where 20/38-90 FA25% .1, 20/38-90 FA25%.2 and 20/38-90 FA25%.3 represent 7 days ageing results, 20/38-90 FA25%.4, 20/38-90 FA25%.5 and 20/38-90 FA25%.6 were tested on 14 days and 20/38-90 FA25%.7, 20/38-90 FA25%.8 and 20/38-90 FA25%.9 tested on 28 days.

Table F.2 Mixture 20/38-90 FA 25% - Summary of Rebound number per specimen and aging day

Specimen Nr.	Rebound Number	
	Average per specimen	Average per curing age day
20/38-90 FA25%.1	17	17
20/38-90 FA25%.2	17	
20/38-90 FA25%.3	17	
20/38-90 FA25%.4	22	22
20/38-90 FA25%.5	22	
20/38-90 FA25%.6	22	
20/38-90 FA25%.7	19	20
20/38-90 FA25%.8	20	
20/38-90 FA25%.9	20	

In Table E 3 the summary of the rebound number where 20/38-90 FA50%.1, 20/38-90 FA50%.2 and 20/38-90 FA50%.3 represent 7 days ageing results, 20/38-90 FA50%.4, 20/38-90 FA50%.5 and 20/38-90 FA50%.6 were tested on 14 days and 20/38-90 FA50%.7, 20/38-90 FA50%.8 and 20/38-90 FA50%.9 tested on 28 days.

Table F.3 Mixture 20/38-90 FA 50% - Summary of Rebound number per specimen and aging day

Specimen Nr.	Rebound Number	
	Average per specimen	Average per curing age day
20/38-90 FA50%.1	16	16
20/38-90 FA50%.2	16	
20/38-90 FA50%.3	16	
20/38-90 FA50%.4	20	21
20/38-90 FA50%.5	20	
20/38-90 FA50%.6	22	
20/38-90 FA50%.7	21	21
20/38-90 FA50%.8	20	
20/38-90 FA50%.9	21	

In Table E 4 the summary of the rebound number where 20/38-90 FA75%.1, 20/38-90 FA75%.2 and 20/38-90 FA75%.3 represent 7 days ageing results, 20/38-90 FA75%.4, 20/38-90 FA75%.5 and 20/38-90 FA75%.6 were tested on 14 days and 20/38-90 FA75%.7, 20/38-90 FA75%.8 and 20/38-90 FA75%.9 tested on 28 days. It was not possible to get results on 7day from specimens 20/38-90 FA75%.2 and 20/38-90 FA75%.3, this could be due to wrong operation of the rebound hammer, or as concrete were still curing with high fly-ash content, it could have need to soft to obtain readings.

Table F.4 Mixture 20/38-90 FA75% - Summary of Rebound number per specimen and aging day

Specimen Nr.	Rebound Number	
	Average per specimen	Average per curing age day
20/38-90 FA75%.1	16	16
20/38-90 FA75%.2	0	
20/38-90 FA75%.3	0	
20/38-90 FA75%.4	19	19
20/38-90 FA75%.5	20	
20/38-90 FA75%.6	19	
20/38-90 FA75%.7	19	19
20/38-90 FA75%.8	18	
20/38-90 FA75%.9	19	

Mixture 15/38-365 was made with the following aggregate ratio: 1100 kg/m^3 coarse aggregate and 1200 kg/m^3 fine aggregate.

In Table E 5 the summary of the rebound number where 15/38-365 FA0%.1, 15/38-365 FA0%.2 and 15/38-365 FA0%.3 represent 7 days ageing results, 15/38-365 FA0%.4, 15/38-365 FA0%.5 and 15/38-365 FA0%.6 were tested on 14 days and 15/38-365 FA0%.7, 15/38-365 FA0%.8 and 15/38-365 FA0%.9 tested on 28 days.

Table F.5 Mixture 15/38-365 FA0% - Summary of Rebound number per specimen and aging day

Specimen Nr.	Rebound Number	
	Average per specimen	Average per curing age day
15/38-365 FA0%.1	20	19
15/38-365 FA0%.2	19	
15/38-365 FA0%.3	18	
15/38-365 FA0%.4	21	21
15/38-365 FA0%.5	21	
15/38-365 FA0%.6	21	
15/38-365 FA0%.7	20	21
15/38-365 FA0%.8	21	
15/38-365 FA0%.9	21	

In Table E 6 the summary of the rebound number where 15/38-365 FA25%.1, 15/38-365 FA25%.2 and 15/38-365 FA25%.3 represent 7 days ageing results, 15/38-365 FA25%.4, 15/38-365 FA25%.5 and 15/38-365 FA25%.6 were tested on 14 days and 15/38-365 FA25%.7, 15/38-365 FA25%.8 and 15/38-365 FA25%.9 tested on 28 days.

Table F.6 Mixture 15/38-365 FA25% - Summary of Rebound number per specimen and aging day

Specimen Nr.	Rebound Number	
	Average per specimen	Average per curing age day
15/38-365 FA25%.1	19	19
15/38-365 FA25%.2	19	
15/38-365 FA25%.3	18	
15/38-365 FA25%.4	21	21
15/38-365 FA25%.5	21	
15/38-365 FA25%.6	21	
15/38-365 FA25%.7	20	20
15/38-365 FA25%.8	21	
15/38-365 FA25%.9	20	

In Table E 7 the summary of the rebound number where 15/38-365 FA50%.1, 15/38-365 FA50%.2 and 15/38-365 FA50%.3 represent 7 days ageing results, 15/38-365 FA50%.4, 15/38-365 FA50%.5 and 15/38-365 FA50%.6 were tested on 14 days and 15/38-365 FA50%.7, 15/38-365 FA50%.8 and 15/38-365 FA50%.9 tested on 28 days.

Table F.7 Mixture 15/38-365 FA50% - Summary of Rebound number per specimen and aging day

Specimen Nr.	Rebound Number	
	Average per specimen	Average per curing age day
15/38-365 FA50%.1	18	18
15/38-365 FA50%.2	18	
15/38-365 FA50%.3	18	
15/38-365 FA50%.4	20	20
15/38-365 FA50%.5	19	
15/38-365 FA50%.6	20	
15/38-365 FA50%.7	18	18
15/38-365 FA50%.8	18	
15/38-365 FA50%.9	17	

In Table E 8 the summary of the rebound number where 15/38-365 FA75%.1, 15/38-365 FA75%.2 and 15/38-365 FA75%.3 represent 7 days ageing results, 15/38-365 FA75%.4, 15/38-365 FA75%.5 and 15/38-365 FA75%.6 were tested on 14 days and 15/38-365 FA75%.7, 15/38-365 FA75%.8 and 15/38-365 FA75%.9 tested on 28 days.

Table F.8	Mixture 15/38-365 FA75% - Summary of Rebound number per specimen and aging day

Specimen Nr.	Rebound Number	
	Average per specimen	Average per curing age day
15/38-365 FA75%.1	17	18
15/38-365 FA75%.2	19	
15/38-365 FA75%.3	18	
15/38-365 FA75%.4	20	19
15/38-365 FA75%.5	19	
15/38-365 FA75%.6	18	
15/38-365 FA75%.7	17	17
15/38-365 FA75%.8	18	
15/38-365 FA75%.9	17	

Appendix G Compressive Strength Results

This chapter provides all the compressive strength results. Concrete mixtures were made with crushed Dolomite aggregate. Aggregate range between 20 mm to 5 mm size were used and river sand, together with, PPC Portland 52,5 N Riebeeck West Suretech cement and Plast RCC plasticizer. Four different mixtures were mix with different ratio of Fly ash per mixture.

Mixture 20/38-90 was made with the following aggregate ratio: 1366 kg/m^3 coarse aggregate and 877 kg/m^3 fine aggregate.

In Table F 1 the compressive strength results presented, where 20/38-90 FA0% .1, 20/38-90 FA0%.2 and 20/38-90 FA0%.3 represent 7 days ageing results, 20/38-90 FA0%.4, 20/38-90 FA0%.5 and 20/38-90 FA0%.6 were tested on 14 days and 20/38-90 FA0%.7, 20/38-90 FA0%.8 and 20/38-90 FA0%.9 tested on 28 days.

Table G.1 Mixture 20/38-90 FA0% - Compressive strength Results

Specimen Identification	Saturated Specimen Mass	Saturated Specimen Volume	Specimen Density	Specimen Failure Load	Specimen Compressive Strength
Number	**(kg)**	**(m^3)**	**(kg/m^3)**	**(kN @ 250 kN/min)**	**(MPa)**
20/38-90 FA0%.1	8.657	0.003454	2506.2	639.2	28
20/38-90 FA0%.2	8.627	0.003473	2484.3	622.4	28
20/38-90 FA0%.3	8.536	0.003447	2476.0	664.8	30
20/38-90 FA0%.4	8.665	0.003484	2487.1	632.6	28
20/38-90 FA0%.5	8.708	0.003457	2519.3	670.0	30
20/38-90 FA0%.6	8.726	0.003511	2485.0	614.0	27
20/38-90 FA0%.7	8.503	0.003409	2494.4	707.2	31
20/38-90 FA0%.8	8.567	0.003463	2473.6	751.0	33
20/38-90 FA0%.9	8.598	0.003477	2472.8	771.6	34

In Table F 2 the summary of the rebound number where 20/38-90 FA0%.1, 20/38-90 FA25%.2 and 20/38-90 FA25%.3 represent 7 days ageing results, 20/38-90 FA25%.4, 20/38-90 FA25%.5 and 20/38-90 FA25%.6 were tested on 14 days and 20/38-90 FA25%.7, 20/38-90 FA25%.8 and 20/38-90 FA25%.9 tested on 28 days.

Table G.2 Mixture 20/38-90 FA25% - Compressive strength Results

Specimen Identification	Saturated Specimen Mass	Saturated Specimen Volume	Specimen Density	Specimen Failure Load	Specimen Compressive Strength
Number	**(kg)**	**(m^3)**	**(kg/m^3)**	**(kN @ 250 kN/min)**	**(MPa)**
20/38-90 FA25%.1	8.862	0.0034	2570.6	529.2	24
20/38-90 FA25%.2	8.693	0.0035	2500.0	466.2	21
20/38-90 FA25%.3	8.629	0.0035	2499.7	470.6	21
20/38-90 FA25%.4	8.894	0.0036	2493.2	537.4	24
20/38-90 FA25%.5	8.521	0.0034	2481.5	562.8	25
20/38-90 FA25%.6	8.575	0.0034	2493.9	560.6	25
20/38-90 FA25%.7	8.546	0.0035	2467.5	633.6	28
20/38-90 FA25%.8	8.503	0.0034	2484.6	658.0	29
20/38-90 FA25%.9	8.643	0.0035	2498.8	671.0	30

In Table F 3 the summary of the rebound number where 20/38-90 FA50%.1, 20/38-90 FA50%.2 and 20/38-90 FA50%.3 represent 7 days ageing results, 20/38-90 FA50%.4, 20/38-90 FA50%.5 and 20/38-90 FA50%.6 were tested on 14 days and 20/38-90 FA50%.7, 20/38-90 FA50%.8 and 20/38-90 FA50%.9 tested on 28 days.

Table G.3 Mixture 20/38-90 FA50%- Compressive strength Results

Specimen Identification	Saturated Specimen Mass	Saturated Specimen Volume	Specimen Density	Specimen Failure Load	Specimen Compressive Strength
Number	**(kg)**	**(m^3)**	**(kg/m^3)**	**(kN @ 250 kN/min)**	**(MPa)**
20/38-90 FA50%.1	8.574	0.0034	2497.0	342.0	15
20/38-90 FA50%.2	8.686	0.0035	2498.0	315.8	14
20/38-90 FA50%.3	8.604	0.0034	2495.8	345.0	15
20/38-90 FA50%.4	8.559	0.0034	2497.6	413.2	18
20/38-90 FA50%.5	8.563	0.0035	2474.1	403.4	18
20/38-90 FA50%.6	8.521	0.0034	2488.1	429.6	19
20/38-90 FA50%.7	8.521	0.0034	2471.7	533.6	24
20/38-90 FA50%.8	8.522	0.0036	2384.3	499.2	22
20/38-90 FA50%.9	8.526	0.0034	2492.8	529.0	24

In Table F 4 the summary of the rebound number where 20/38-90 FA75%.1, 20/38-90 FA75%.2 and 20/38-90 FA75%.3 represent 7 days ageing results, 20/38-90 FA75%.4, 20/38-90 FA75%.5 and 20/38-90 FA75%.6

were tested on 14 days and 20/38-90 FA75%.7, 20/38-90 FA75%.8 and 20/38-90 FA75%.9 tested on 28 days.

Table G.4 Mixture 20/38-90 FA75% - Compressive strength Results

Specimen Identification	Saturated Specimen Mass	Saturated Specimen Volume	Specimen Density	Specimen Failure Load	Specimen Compressive Strength
Number	(kg)	(m^3)	(kg/m^3)	(kN @ 250 kN/min)	(MPa)
20/38-90 FA75%.1	8.437	0.0035	2431.2	136.8	6
20/38-90 FA75%.2	8.474	0.0034	2474.4	136.6	6
20/38-90 FA75%.3	8.504	0.0034	2479.9	108.4	5
20/38-90 FA75%.4	8.595	0.0034	2491.5	195.4	9
20/38-90 FA75%.5	8.526	0.0034	2502.8	182.6	8
20/38-90 FA75%.6	8.592	0.0035	2477.5	223.2	10
20/38-90 FA75%.7	8.495	0.0034	2469.1	265.2	12
20/38-90 FA75%.8	8.549	0.0034	2483.1	257.4	11
20/38-90 FA75%.9	8.508	0.0034	2497.5	282.0	13

Mixture 15/38-365 was made with the following aggregate ratio: 1100 kg/m^3 coarse aggregate and 1200 kg/m^3 fine aggregate.

In Table F 5 the summary of the rebound number where 15/38-365 FA0%.1, 15/38-365 FA0%.2 and 15/38-365 FA0%.3 represent 7 days ageing results, 15/38-365 FA0%.4, 15/38-365 FA0%.5 and 15/38-365 FA0%.6 were tested on 14 days and 15/38-365 FA0%.7, 15/38-365 FA0%.8 and 15/38-365 FA0%.9 tested on 28 days.

Table G.5 Mixture 15/38-365 FA0% - Compressive strength Results

Specimen Identification	Saturated Specimen Mass	Saturated Specimen Volume	Specimen Density	Specimen Failure Load	Specimen Compressive Strength
Number	(kg)	(m^3)	(kg/m^3)	(kN @ 250 kN/min)	(MPa)
15/38-365 FA0%.1	8.314	0.003443	2414.84	351.6	16
15/38-365 FA0%.2	8.365	0.003475	2407.27	338.8	15
15/38-365 FA0%.3	8.188	0.003486	2348.63	332.0	15
15/38-365 FA0%.4	8.257	0.003457	2388.75	346.4	15
15/38-365 FA0%.5	8.192	0.003459	2368.40	354.6	16
15/38-365 FA0%.6	8.040	0.003429	2344.52	304.4	14
15/38-365 FA0%.7	8.356	0.003523	2371.75	370.0	16
15/38-365 FA0%.8	8.303	0.003498	2373.88	344.0	15
15/38-365 FA0%.9	8.375	0.003461	2419.73	416.4	19

In Table F 6 the summary of the rebound number where 15/38-365 FA25%.1, 15/38-365 FA25%.2 and 15/38-365 FA25%.3 represent 7 days ageing results, 15/38-365 FA25%.4, 15/38-365 FA25%.5 and 15/38-365 FA25%.6 were tested on 14 days and 15/38-365 FA25%.7, 15/38-365 FA25%.8 and 15/38-365 FA25%.9 tested on 28 days.

Table G.6 Mixture 15/38-365 FA25% - Compressive strength Results

Specimen Identification	**Saturated Specimen Mass**	**Saturated Specimen Volume**	**Specimen Density**	**Specimen Failure Load**	**Specimen Compressive Strength**
Number	**(kg)**	**(m^3)**	**(kg/m^3)**	**(kN @ 250 kN/min)**	**(MPa)**
15/38-365 FA25%.1	8.370	0.0034	2442.36	320.0	14
15/38-365 FA25%.2	8.259	0.0035	2373.62	261.8	12
15/38-365 FA25%.3	8.353	0.0035	2403.95	273.2	12
15/38-365 FA25%.4	8.320	0.0034	2413.39	355.2	16
15/38-365 FA25%.5	8.335	0.0035	2400.21	314.4	14
15/38-365 FA25%.6	8.261	0.0035	2372.73	332.4	15
15/38-365 FA25%.7	8.155	0.0034	2367.07	353.6	16
15/38-365 FA25%.8	8.444	0.0035	2395.35	322.0	14
15/38-365 FA25%.9	8.207	0.0035	2350.90	339.6	15

In Table F 7 the summary of the rebound number where 15/38-365 FA50%.1, 15/38-365 FA50%.2 and 15/38-365 FA50%.3 represent 7 days ageing results, 15/38-365 FA50%.4, 15/38-365 FA50%.5 and 15/38-365 FA50%.6 were tested on 14 days and 15/38-365 FA50%.7, 15/38-365 FA50%.8 and 15/38-365 FA50%.9 tested on 28 days.

Table G.7 Mixture 15/38-365 FA50% - Compressive strength Results

Specimen Identification	**Saturated Specimen Mass**	**Saturated Specimen Volume**	**Specimen Density**	**Specimen Failure Load**	**Specimen Compressive Strength**
Number	**(kg)**	**(m^3)**	**(kg/m^3)**	**(kN @ 250 kN/min)**	**(MPa)**
15/38-365 FA50%.1	8.129	0.0034	2384.69	122.4	5
15/38-365 FA50%.2	8.052	0.0034	2360.54	116.4	5
15/38-365 FA50%.3	8.239	0.0034	2436.37	117.6	5
15/38-365 FA50%.4	8.148	0.0034	2409.46	163.4	7
15/38-365 FA50%.5	8.132	0.0034	2395.12	175.0	8
15/38-365 FA50%.6	8.082	0.0034	2355.23	141.4	6
15/38-365 FA50%.7	8.117	0.0034	2376.49	191.6	9
15/38-365 FA50%.8	8.084	0.0034	2343.34	160.8	7
15/38-365 FA50%.9	8.104	0.0034	2358.50	193.4	9

In Table F 8 the summary of the rebound number where 15/38-365 FA75%.1, 15/38-365 FA75%.2 and 15/38-365 FA75%.3 represent 7 days ageing results, 15/38-365 FA75%.4, 15/38-365 FA75%.5 and 15/38-365 FA75%.6 were tested on 14 days and 15/38-365 FA75%.7, 15/38-365 FA75%.8 and 15/38-365 FA75%.9 tested on 28 days. With specimen 15/38-365 FA75%.2 there were an error with the compressive machine, and the compression of the specimen and therefore the results for this specimen was omitted.

Table G.8 Mixture 15/38-365 FA75% - Compressive strength Results

Specimen Identification	**Saturated Specimen Mass**	**Saturated Specimen Volume**	**Specimen Density**	**Specimen Failure Load**	**Specimen Compressive Strength**
Number	**(kg)**	**(m^3)**	**(kg/m^3)**	**(kN @ 250 kN/min)**	**(MPa)**
15/38-365 FA75%.1	8.045	0.0034	2342.97	39.2	1.7
15/38-365 FA75%.2	8.016	0.0035	2316.07	15.3	0.7
15/38-365 FA75%.3	8.121	0.0034	2358.84	35.6	1.6
15/38-365 FA75%.4	8.089	0.0034	2349.54	79.0	3.5
15/38-365 FA75%.5	8.124	0.0034	2359.71	68.2	3.0
15/38-365 FA75%.6	8.036	0.0034	2334.15	65.2	2.9
15/38-365 FA75%.7	8.106	0.0035	2343.55	92.0	4.1
15/38-365 FA75%.8	8.118	0.0034	2378.30	104.4	4.6
15/38-365 FA75%.9	8.000	0.0035	2315.97	93.0	4.1

Appendix H Multiple regression

This chapter provides all the multiple regression results. The data will be provided in tables.

For the true representation, all 72 specimens' results recorded for UPV, RH and Compressive strength is used. Peasons correlation in shown in Table H1. The Multiple regression summary output is shown in Table H.2. The multiple regression ANOVA outputs is shown in Table H.3 and the coefficients outputs in Table H.4.

Table H.1 True representation Pearsons's correlation

	UCS	**UPV Avg**	**RH Med**
UCS	1		
UPV Avg	0,773	1	
RH Med	0,366	0,219	1

The below tables show the multiple regression outputs.

Table H.2 True representation Multiple regression summary output

Regression Statistics	
Multiple R	0,798
R Square	0,636
Adjusted R Square	0,625
Standard Error	5,442
Observations	71

Table H.3 True representation Multiple regression ANOVA output

ANOVA					
	df	***SS***	***MS***	***F***	***Significance F***
Regression	2	3519,484	1759,742	59,430	1,1851E-15
Residual	68	2013,507	29,610		
Total	70	5532,991			

Table H.4 True representation Multiple regression coefficients output

	Coefficients	***Standard Error***	***t Stat***	***P-value***	***Lower 95%***	***Upper 95%***	***Lower 95.0%***	***Upper 95.0%***
Intercept	-62,995	8,397	-7,502	1,755E-10	-79,750	-46,239	-79,750	-46,239
5,50	11,533	1,182	9,756	1,482E-14	9,174	13,891	9,174	13,891
20,5	1,018	0,380	2,682	0,0092	0,261	1,776	0,261	1,776

For the direct representation, outliers were removed where only 65 specimen's results recorded for UPV, RH and Compressive strength is used. Pearsons's correlation in shown in Table H.5. The Multiple regression summary output is shown in Table H.6. The multiple regression ANOVA outputs is shown in Table H.7 and the coefficients outputs in Table H.8.

Table H.5 True representation Pearsons's correlation

	UCS	UPV Avg	RH Med
UCS	1		
UPV Avg	0,831	1	
RH Med	0,357	0,219	1

The below tables show the multiple regression outputs.

Table H.6 True representation Multiple regression summary output

Regression Statistics	
Multiple R	0,850
R Square	0,723
Adjusted R Square	0,714
Standard Error	4,752
Observations	65

Table H.7 True representation Multiple regression ANOVA output

ANOVA					
	df	***SS***	***MS***	***F***	***Significance F***
Regression	2	3657,703	1828,851	81,000797	5,0858E-18
Residual	62	1399,848	22,578		
Total	64	5057,551			

Table H.8 True representation Multiple regression coefficients output

	Coefficients	***Standard Error***	***t Stat***	***P-value***	***Lower 95%***	***Upper 95%***	***Lower 95.0%***	***Upper 95.0%***
Intercept	-66,797	7,578	-8,815	1,545E-12	-81,945	-51,650	- 81,945	- 51,650
UPV Avg	12,729	1,102	11,550	4,255E-17	10,526	14,932	10,526	14,932
RH Med	0,906	0,336	2,693	0,00910	0,233	1,579	0,233	1,579

For the indirect representation, outliers were removed where only 63 specimen's results recorded for experimental RH, and compressive strength, and experimental UPV results converted to shear UPV wave using Poisson's ratio. Pearsons's correlation in shown in Table H.9. The Multiple regression summary output is shown in Table H.10. The multiple regression ANOVA outputs is shown in Table H.11 and the coefficients outputs in Table H.12

Table H.9 True representation Pearsons's correlation

	UCS	**UPV Shear**	**RH Med**
UCS	1		
UPV Shear	0,831	1	
RH Med	0,343	0,192	1

The below tables show the multiple regression outputs.

Table H.10 True representation Multiple regression summary output

Regression Statistics	
Multiple R	0,852
R Square	0,726
Adjusted R Square	0,717
Standard Error	4,612
Observations	63

Table H.11 True representation Multiple regression ANOVA output

ANOVA					
	df	***SS***	***MS***	***F***	***Significance F***
Regression	2	3385,095	1692,547	79,559	1,331E-17
Residual	60	1276,448	21,274		
Total	62	4661,543			

Table H.12 True representation Multiple regression coefficients output

	Coefficients	***Standard Error***	***t Stat***	***P-value***	***Lower 95%***	***Upper 95%***	***Lower 95.0%***	***Upper 95.0%***
Intercept	-74,423	8,0115	-9,290	3,225E-13	-90,449	-58,398	-90,449	-58,398
UPV Shear Waves	16,165	1,4000	11,546	7,0036E-17	13,365	18,966	13,365	18,966
RH Med	0,903	0,3266	2,765	0,00756	0,250	1,556	0,250	1,556

For the indirect representation, outliers were removed where only 63 specimen's results recorded for experimental compressive strength, and experimental UPV results converted to shear UPV wave using Poisson's ratio. Pearsons's correlation in shown in Table H.13 The Multiple regression summary output is shown in Table H.14. The multiple regression ANOVA outputs is shown in Table H.15 and the coefficients outputs in Table H.16

Table H.13 True representation Pearsons's correlation

	UCS	**UPV Shear**	**RH Med**
UCS	1		
UPV Shear	0,831	1	
RH Med	0,343	0,192	1

The below tables show the multiple regression outputs.

Table H.14 True representation Multiple regression summary output

Regression Statistics	
Multiple R	0,831
R Square	0,691
Adjusted R Square	0,686
Standard Error	4,857
Observations	63

Table H.15 True representation Multiple regression ANOVA output

ANOVA					
	df	***SS***	***MS***	***F***	***Significance F***
Regression	1	3222,507	3222,507	136,600	3,27842E-17
Residual	61	1439,036	23,591		
Total	62	4661,543			

Table H.16 True representation Multiple regression coefficients output

	Coefficients	***Standard Error***	***t Stat***	***P-value***	***Lower 95%***	***Upper 95%***	***Lower 95.0%***	***Upper 95.0%***
Intercept	-60,542	6,574	-9,210	3,772E-13	-73,686	-47,397	-73,686	-47,397
UPV Shear Waves	16,909	1,447	11,688	3,278E-17	14,016	19,802	14,016	19,802

Appendix I Turnitin