

**KARADENİZ TECHNICAL UNIVERSITY
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES**

DEPARTMENT OF CIVIL ENGINEERING

**DEFINING BEST COMPACTION TECHNIQUES AND COLD LONGITUDINAL JOINT BONDING
SOLUTIONS FOR ROLLER-COMPACTED CONCRETE (RCC) PAVEMENTS BASED ON
ANALYTICAL AND EXPERIMENTAL ANALYSES**

DOCTORATE THESIS

Murat NAS

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TRABZON**



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Murat NAS

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This thesis endeavors to establish a paradigm for constructing longitudinal joints in RCC pavements. The extensive duration of this study is attributed to the integration of engineering and academic aspects. I loved every minute of it, however hard it had been. I have diligently kept abreast of emerging developments in the thesis topic throughout the writing process, ensuring its contemporaneity. I am confident that I have successfully achieved this goal, resulting in a current and relevant thesis. My intention is to pave the way for future researchers interested in the core subject of this study. Moreover, I am confident in the effectiveness of conveying my ideas to the potential reader. I trust that this work and its conclusive remarks will serve as a catalyst for future valuable research endeavors.

In conclusion, I aspire for this contribution to impact the realm of RCC pavement technology positively. If this study proves beneficial, it would bring me great satisfaction.

Murat NAS
Trabzon, 2024

THESIS STATEMENT

I hereby declare that all the information in this thesis entitled “Defining Best Compaction Techniques and Cold Longitudinal Joint Bonding Solutions for Roller-Compacted Concrete (RCC) Pavements Based on Analytical and Experimental Analyses” has been obtained and presented in accordance with the academic rules and ethical conduct under the supervision of Prof. Dr. Muhammet Vefa AKPINAR. I also attest that, as per required by these rules and conduct, I have fully cited and referenced all the material and results that are not original to this work. The content of this thesis solely mirrors the views and findings of the author, who is responsible for the factuality and the accuracy of the knowledge presented herein. This thesis has not been submitted to any other institution for any other degree. I concede any legal consequences in advance in case of the statements above appear otherwise. 30/09/2024

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PhD. Thesis

SUMMARY

DEFINING BEST COMPACTION TECHNIQUES AND COLD LONGITUDINAL JOINT BONDING SOLUTIONS FOR ROLLER-COMPACTED CONCRETE (RCC) PAVEMENTS BASED ON ANALYTICAL AND EXPERIMENTAL ANALYSES

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Cold longitudinal joints are the weakest elements in any pavement structure. Limited research has addressed techniques for durable longitudinal joints in hot-mix asphalt (HMA) pavements, but no specific study exists for roller-compacted concrete (RCC) pavements. This thesis investigates the ideal geometric arrangements of cold joints (90° , 60° , 45° , 30° , notched wedge-N1, doubly-notched wedge-N2) for RCC pavements in terms of compactability and mechanical properties. It also examines the bonding performance of cold longitudinal RCC joints and evaluates which reinforcement material (water spraying-W, cement slurry-S, epoxy-based repair mortar-E) yields better results. Several finite element analyses (FEA) and lab experiments were employed. Incorporation 1% polypropylene fiber to RCC raised the rupture modulus (MR) by about 15% without significantly boosting compressive strength. Of all alternatives, models cast as a whole, without cold joints, gave the best overall performances, indicating fresh longitudinal joints should be the first choice whenever possible. The hierarchical benefit of joint interface reinforcement methods, compared to samples without reinforcement (NO), was: $E > S > W > NO$. The best compaction performance order obtained from FEAs was: $90^\circ > 60^\circ > N2 > N1 > 45^\circ > 30^\circ$. The order of the highest MR values was: $90^\circ \geq 45^\circ > 60^\circ > N1 > 30^\circ > N2$. While the E generally yielded the best MR results in plain RCC samples, it did not provide the same benefit in fiber-reinforced samples. A cost-benefit analysis manifestly suggests that methods S and W are preferable to the E.

Key Words: Roller-compacted concrete (RCC), Cold longitudinal joints, Joint Construction

Doktora Tezi

ÖZET

ANALİTİK VE DENEYSEL ANALİZLERE DAYALI OLARAK SİLİNDİRLE SIKIŞTIRILMIŞ BETON (SSB) KAPLAMALAR İÇİN EN İYİ SIKIŞTIRMA TEKNİKLERİ VE BOYLAM SOĞUK DERZ BAĞLANMA ÇÖZÜMLERİNİN BELİRLENMESİ

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Soğuk boyuna derzler, bir yol kaplama yapısının en zayıf elamanları olarak kabul edilir. Bitümlü sıcak karışım asfalt (BSK) kaplamalarda dayanıklı boyuna derzler için en uygun yapım tekniklerini ele alan sınırlı sayıda araştırma bulunmakla birlikte, silindirle sıkıştırılmış beton (SSB) kaplamalar için belirgin bir çalışma mevcut değildir. Bu tez, SSB kaplamalarda soğuk derzlerin (90°, 60°, 45°, 30°, tek kademeli kama-N1, çift kademeli kama-N2) ideal geometrik düzenlemelerini sıkışabilirlik ve mekanik özellikler açısından araştırmaktadır. Ayrıca soğuk boyuna SSB derzlerinin kenetlenme performansını incelemekte ve hangi takviye malzemesinin (püskürtme su -W, çimento bulamacı-S, epoksi esaslı tamir harcı-E) daha iyi sonuçlar verdiğini değerlendirmektedir. Çeşitli sonlu eleman analizleri ve laboratuvar deneylerinden yararlanılmıştır. SSB'ye %1 polipropilen lif katılması, eğilme dayanımını (MR) %15 kadar artırmış ancak basınç dayanımını önemli ölçüde artırmamıştır. Tüm alternatifler arasında, soğuk derzsiz olarak dökülen modeller genel performans açısından en iyi sonuçları vermiş, taze (bir döküm) boyuna derzlerin mümkün olduğunca ilk tercih olması gerektiğini göstermiştir. Takviye yöntemlerinin derz kenetlenme performansı üzerindeki fayda sıralaması, takviyesiz örnekler (NO) ile karşılaştırıldığında şu şekildedir: $E > S > W > NO$. En iyi sıkışma performans sıralaması, $90^\circ > 60^\circ > N2 > N1 > 45^\circ > 30^\circ$ ve en yüksek MR değerleri sıralaması $90^\circ \geq 45^\circ > 60^\circ > N1 > 30^\circ > N2$ şeklindedir. E genellikle yalnız SSB örneklerinde en iyi MR sonuçlarını verirken, lif takviyeli örneklerde aynı faydayı sağlamamıştır. Maliyet-fayda analizi, S ve W'nin, E ye göre daha tercih edilebilir olduğunu açıkça göstermektedir.

Anahtar Kelimeler: Silindirle sıkıştırılmış beton yol, Soğuk boyuna derz, Derz inşası

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LIST OF SYMBOLS AND ABBREVIATIONS

Symbols

α	: Angle of the slanted plane
α_{cr}	: Critical bonding angle
c	: Cohesion
f_c	: Compressive (uniaxial) strength
f_{tb}	: Relative bond strength (MPa)
MC Model	: Mohr-Coulomb model
MR	: Modulus of Rupture or bending strength
μ	: Coefficient of friction
p	: Deviatoric stress; average normal stress from stress path quantity ($p = \sigma_1 + \sigma_3$)
q	: Deviatoric stress; largest shear stress from stress path quantity ($q = \sigma_1 - \sigma_3$)
σ_0	: Stress required to cause failure through the bonding plane
$\sigma_1, \sigma_2, \sigma_3$	: Principal stresses
σ_D	: Stress component along the space diagonal
σ_E	: Horizontal stress component
σ_F	: Vertical stress component
σ_i	: Mean pressure (confining pressure)
σ_{min}	: Minimum stress relating to α_{cr}
σ_n	: Normal stress
τ_{cr}	: Shear stress at bond failure
τ_n	: Shear stress
θ	: Lode angle ($\theta = \tan^{-1}(\sigma_F / \sigma_E)$)
φ	: Friction angle between two bodies

Abbreviations

AASHTO	: American Association of State Highway and Transportation Officials
ACI	: American Concrete Institute
ANN	: Artificial Neural Network

ASTM	: American Society for Testing and Materials
BS	: British Standards
CDP	: Concrete Damaged Plasticity
EN	: European Norm
FEA	: Finite Element Analysis
FEM	: Finite Element Method
GPR	: Ground Penetrating Radar
HMA	: Hot Mix Asphalt
IDOT	: Illinois Department of Transportation
LRFD	: Load and Resistance Factor Design
NAPA	: National Asphalt Pavement Association
NDT	: Non-Destructive Testing
PC	: Portland Cement
PVA	: Polyvinyl Alcohol
PWL	: Percent Within Limits
RC	: Reinforced Concrete
RCC	: Roller-Compacted Concrete
TCMB	: Turkish Cement Manufacturers' Association

1. INTRODUCTION

1.1. Background and Significance

In pavement engineering, the quest for improved design and innovative yet cost-effective construction practices has been a topic of interest among engineers for ages. In parallel to the prevailing trend that first appeared in North America and Europe, there has been a growing concern about Türkiye's concrete pavement construction practices. As Türkiye undergoes relatively steady economic development and social enrichment over the years, the evident necessity to expand road networks compels engineers to seek cutting-edge, reliable, and remarkably affordable construction techniques. Without further ado, throughout this study, the term “pavement” refers directly to the road pavement, not to airfield pavement, runway, or any other transport structure.

In general, producing cost-effective and purpose-driven concrete has always been a topic of interest for all engineering purposes. At this juncture, a particular type of concrete known as roller-compacted concrete (hereafter referred to as RCC) comes to the forefront with its versatile and tailorable opportunities for engineers. According to the American Concrete Institute's Concrete Terminology Manual (ACI, 2013), the term RCC is specifically defined as "a concrete compacted by roller compaction" and also as "a type of concrete that will support a roller while being compacted in its unhardened state." The way things are, RCC gets its name from a heavy vibratory steel drum and rubber-tired roller used to compact it into its final state (ACI, 1995).

RCC has thus far served as a primary or complementary multipurpose construction material in civil engineering for many years. Namely, RCC has, as yet, been used to construct concrete gravity dams and as a robust surface course material in pavements. A pavement constructed by laying down and compacting fresh RCC material is referred to as roller-compacted concrete pavement (hereafter RCCP). In addition, RCC is employed for various non-major engineering purposes, encompassing a wide range of construction operations, including embankment overtopping protection, spillway construction, slope protection, and even stabilizing foundation soils (Choi & Hansen, 2005).

From the very beginning, RCC has attracted a great deal of interest in the pavement construction industry, and it is accepted as an economical alternative to the other pavement

genres due to its groundbreaking opportunities, such as superfast construction with sustainable materials. In historical terms, one of the allegedly first examples of RCCPs was built in Sweden early in the 1930s. RCCPs then appeared in North America at the beginning of the 1940s (Piggott, 1999). The emergence of RCC as a pavement construction material stemmed from a straightforward motivation: the limited resilience of asphalt pavements against heavy-duty applications, particularly by the military and forestry (logging) industry. In that sense, this challenge accelerated the use of RCC in public pavements, which were first constructed in out-of-the-way places and later appeared in local road networks in the US (Harrington et al., 2010; Piggott, 1999).

As with any pavement genre, longitudinal joints are used in RCCPs to mainly prevent random longitudinal cracking. The cracking commonly originates from a compound effect due to loading and constrained warping of the slab (MERO, 2013). Longitudinal joints are regarded as the weakest element of the pavement structure and one of the most critical aspects of its resilience and strength. Nevertheless, there is much more unknown than known on this issue, and we encounter complexity in these known unknowns. For example, there were no technical specifications for RCCPs in Türkiye for many years. The General Directorate of Highways, regarded as the sole authority of the state highways in Türkiye, has not yet released any standard specifications for RCCPs. On the one hand, in 2018, the Turkish Cement Manufacturers' Association (TCMB) released a due code called Technical Specification for RCCPs (TCMB, 2018). In 2020, a new chapter, General Technical Specification for Roller-Compacted Concrete Pavement, was added to the General Technical Specifications for Construction Works, Machinery, and Electrical Installations, the primary authoritative specifications used throughout Türkiye. While they provide several valuable limitations and recommendations for manufacturers and practitioners, these specifications do not address any design and construction issues for longitudinal joints in RCCPs. On the other hand, several state-basis specifications (e.g., IDOT (2014)) have long been used for RCCP constructions in the US. Due to locally available material properties and zonal climatic conditions, the US-originated specifications vary from state to state to some extent. Even though there are many, they do not fully represent the answer to the question of “what the most appropriate longitudinal joint construction technique for RCCPs is”. Furthermore, our understanding of the robust longitudinal cold joint construction technique is, at best, based on subjective crumbs of knowledge. Much of the knowledge circulating here and there is derived from existing lessons acquired through past experiences and scientific studies

conducted initially for hot-mix asphalt (HMA) pavements (HMAPs), albeit without due comparative engineering judgment and reliable data.

While significant research has been conducted on various aspects of cold longitudinal joints in HMA pavements, too little research has been done on similar problematic issues in RCC pavements. As RCCP construction technology has enjoyed a boom in its frequency of use in Türkiye and worldwide for a few decades, numerous problematic issues related to construction techniques have come to the fore. Of these protracted problems, this study focuses mainly on understanding the behavior of initial compaction and the best way to angulate the tapered surface of cold longitudinal joints to be constructed in RCCPs. In that vein, this study is henceforth the first of its kind to explore this conundrum from a mechanistic perspective, employing both model-based lab tests and Finite Element Modeling (FEM). Specifically, this thesis endeavors to establish a novel paradigm for the construction techniques of cold longitudinal joints in RCCPs.

1.2. Pavement Joints

Joints are known to be one of the vital elements of any pavement, whether rigid or flexible. Their primary and straightforward function is to play an integral role in extending the lifespan of the road's main body. Pavement joints must fulfill several crucial tasks to keep the surface course from possible cracking for several reasons (e.g., relative movement of the pavement, differential settlement, freeze-thaw cycles, water ingress to subbase and subgrade, rapid warming up and cooling down of slab). Hence, most concrete pavements without joints would likely develop extensive cracks within a couple of years after the initial placement. Fundamentally, the pavement joints, the representation of which is set out in Figure 1, can be classified into two main categories regarding their orientations. Longitudinal joints, which constitute the main research topic of this study, are the interfaces leading to vertical discontinuities between two adjacent lanes along with the pavement slab, each of which is ordinarily placed fully parallel to the pavement's centerline. They are primarily determined as absolutely indispensable elements of pavement construction when two adjacent lanes are mainly constructed in an echelon form. Transverse joints, on the other hand, are positioned at a right angle to the pavement's centerline. Their primary role in concrete pavements is to transfer loads from traffic and other stresses through aggregate interlock, a crucial element also required in RCCPs. In conventional concrete pavements,

transverse joints serve to facilitate load transfer and aggregate interlocking, depending on the chosen mechanical system. The Federal Highway Administration (FHWA, 2019) classifies concrete pavement joints based on their primary purpose, including expansion joints, construction joints, isolation joints, and contraction or control joints. However, all joint types are nothing more than the specific use of the longitudinal or transverse joint. As is well known, the contraction of a concrete pavement slab is an almost inevitable physical process during which the concrete material shortens due to shrinkage. Of other joint types, contraction joints are created as human-made vertical discontinuities that are either sawed, formed, or tooled groove in a concrete mat. They constitute the most common type of joint and are mainly constructed in PCCs. In PCCs, no kind of wired reinforcing steel is used; instead, the dowel bars are used to transfer the loads properly. Besides, tie bars are concurrently used to keep the two-echelon lanes connected safely. However, they also contribute to the load transfer capability of the joint nevertheless (AASHTO, 1993; Thom, 2014).

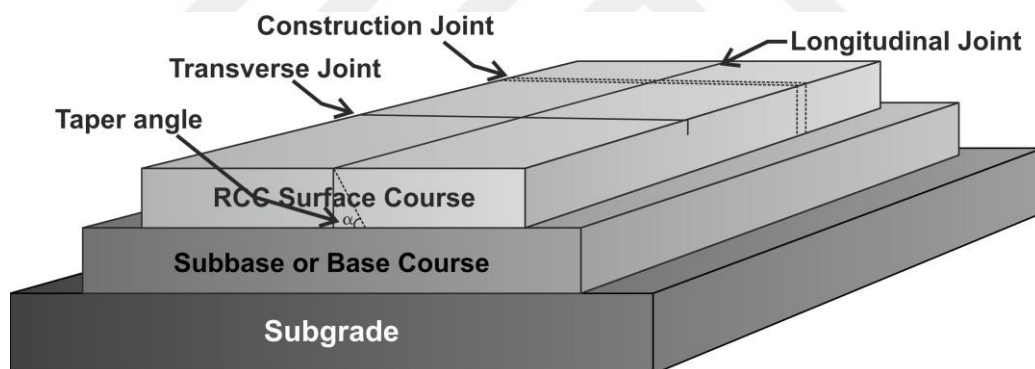


Figure 1. Basic joint types in a road pavement structure

One of the key differences between RCC and PCC installation is the absence of reinforcing elements, tie bars, or dowel bars in RCCPs. RCCPs rely on the load transfer capability of joints since they lack exterior details to facilitate load transfer. Transverse contraction joints are used to accommodate the natural shrinkage of the concrete slab. A slightly different version of transverse joints is the skewed contraction joint, which is cut at obtuse angles to the longitudinal direction of traffic flow in order to transfer the axle loads to a different side of the pavement slab. Like the transverse contraction joints, expansion joints are constructed in the pavement's transverse direction to allow any kind of physical movement in the same direction due to temperature change and moisture variation in time.

Although not a design-based joint type, pure construction joints are created either in the transverse or longitudinal direction when the ongoing pavement construction needs to be temporarily halted. However, it has long been known that each pavement joint creates almost apparent discontinuities in the pavement mat.

Early examples of RCC pavements were allowed to crack naturally without joints, prioritizing construction and asset maintenance economy. The contraction and saw-cut joints were then used to create some control joints, not to let the main body of the pavement crack. Not only sawing but also sealing contraction joints are common practices. In RCC pavements, the spacing between consecutive saw-cut joints is typically twice as large as that in PCC with an equivalent wearing course thickness (Shoenberger, 1994).

1.3. Principles of Longitudinal Joint Construction Problem

Much as extensive research has been conducted over several decades to determine the most suitable cold longitudinal joint construction technique for HMA pavements, no consensus has as yet been reached about which technique is the best. Like many similar institutions worldwide, several different state departments of transportation (DOTs) in the US use their own experiences (Zofka et al., 2008). Despite the passing of several consecutive years, limited research has been aimed at defining proper longitudinal joint construction techniques for RCCPs. Specifically, there is a lack of scientific perspective regarding how different initial rolling methods relate to the behavioral mechanisms of longitudinal joints during roller compaction. The primary focus of the current investigation is to determine whether a direct relationship exists between raveling and premature cracking in longitudinal joints. These issues are predominantly attributed to inadequate material density in and around the cold longitudinal joints. Due to the absence of well-established longitudinal joint construction techniques for RCCPs, my approach in this study is to leverage the known similarities between (HMAPs and RCCPs. I will then explore alternative novel techniques to address this challenge.

1.3.1. Compaction of the First Lane

One of the most critical challenges in both HMAPs and RCCPs is achieving proper compaction of the free edge in the first lane. Through an end-plate (edger plate), the paver

screed can create a sloped face (taper) either arbitrarily or in the desired shape as the paving is performed. Consequently, it set not only ideally bonded but also centrally aligned adjacent faces of longitudinal joints. The slopes of tapered surfaces in longitudinal joints are typically formed to a maximum of 60 degrees, especially in HMAP construction practices (Benson & Scherocman, 2006). To the best of my knowledge, the tapered surfaces could be formed in RCCPs at angles similar to those created in HMA pavements. Because of this, if we recall that the interior angle of the conventional slump cone is about 65-70 degrees, and now that the RCC ideally tends to make almost no collapse (zero slump), similar angulation techniques employed in HMAPs become available in RCCPs. Figure 2 shows a schematic of possible alternative initial roller positions that can be used to compact the first lane of an RCCP.

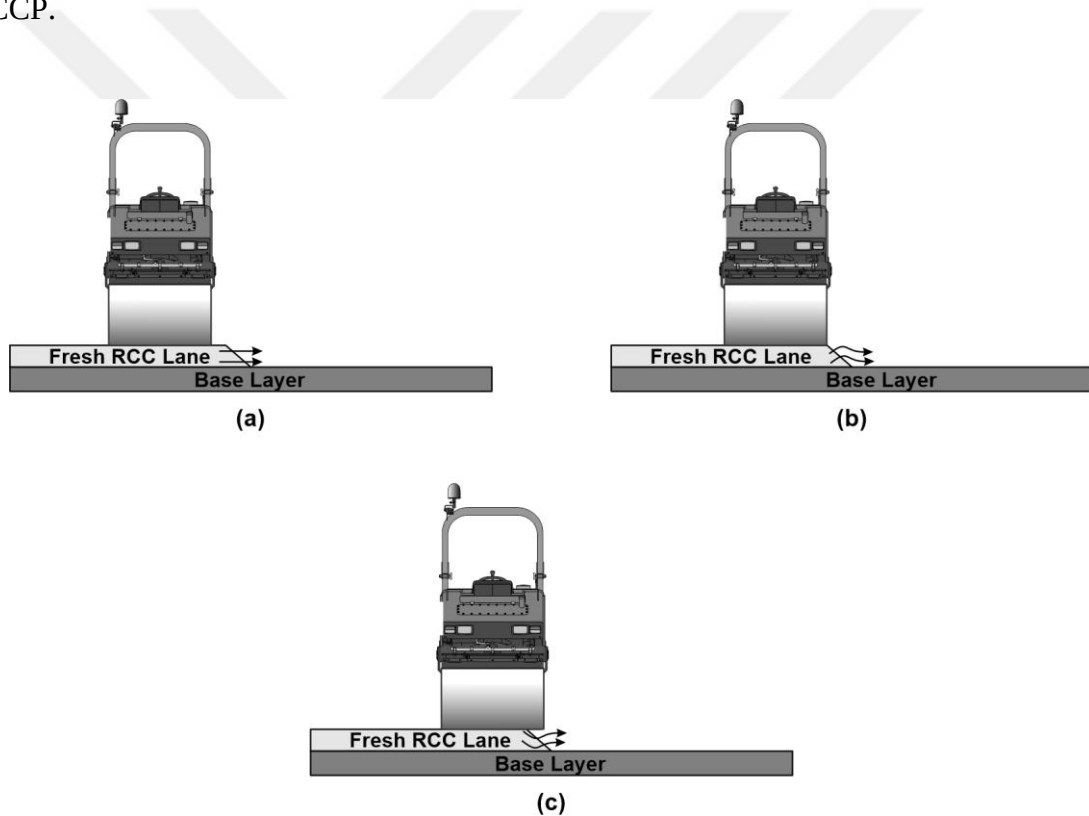


Figure 2. Alternative initial roller positions to compact fresh RCC lane: (a) The outermost edge of the tandem drum roller remains inside the unsupported edge of the first lane, being slightly off the edge, (b) The edge of the drum is on the unsupported edge, and (c) The drum slightly overhangs from the edge

The first technique set out in Figure 2a refers to the initial position of the roller whose outermost edge is off the unsupported edge of the fresh RCCP. In this technique, the whole drum rests on the fresh lane, and only a portion of the lane is not compacted in the first

operation. The second technique in Figure 2b represents a different position of the roller, whose outer edge is right over the unsupported edge of the first lane. This position is well regarded as one of the most inadequate techniques used in HMAPs. Because of this, as the roller compacts the HMA mix, the material beneath the drum often exhibits rapid and irregular sideways movement to the right. This phenomenon results in the formation of protrusions, leading to non-linear relaxation in the RCC matrix. Consequently, this process yields a significantly lower and unevenly distributed in-place longitudinal joint density than the mat. The third alternative for the initial roller position used in HMAPs illustrated in Figure 2c refers to an initial position of the roller whose outermost edge is slightly away from the unsupported edge of the first lane. The commonly implemented overhang distance in this technique is about 250-500 mm (Shin & Carboneau, 2010). However, the third technique (as shown in Figure 2c) creates another contentious practice, and past experiences showed that the HMA mix, which is directly under the overhang part, instantly tends to expand, moving in the transverse direction. As a result of the transversal expansion in the matrix, dilation occurs, increasing the likelihood of sudden and premature crack initiations due to poor compaction and low density, particularly in the vicinity of the longitudinal joint.

It is often pronounced that the initial position of the roller compactor and its working principle (e.g., static vs. vibratory) are both unreservedly influential in the performance of the initial compaction. In other words, the initial and primary compaction phase significantly influences the entire compaction process, impacting the final density gradient both in the joint zone and across the entire mat. Several studies have specifically aimed to determine the optimal proximity of the roller compactor to the unsupported edge of the lane for achieving better compaction in HMA pavement technology. In conventional HMA pavement construction, a typical and widely-used distance from the drum's end to the lane's free edge is 152 mm (6 inches) (Akpinar & Hossain, 2004). This measurement directly signifies how close or far the outer edge of the drum should be from the pavement's free end to achieve the best possible initial compaction. However, in everyday RCCP practices, the distance from the drum to the free end of the lane is typically considered to be two to three times greater than what is applied in HMA pavements (Shoenberger, 1994).

1.3.2. Compaction of the Second Lane

How the second lane adjacent to the first lane gets compacted in pavement construction determines a thick demarcation line between the concept of old and fresh longitudinal joints in concrete pavement technology. Creating cold longitudinal joints becomes necessary in pavement construction facilities, except in cases where the echelon paving technique is used. In principle, for the construction of fresh longitudinal joints, thoroughgoing compaction of RCC should essentially be carried out within 45 minutes of the initial mixing and 15 minutes of the spreading. In another saying, the whole process should take up to 60 min to create ideally fresh joints (Harrington et al., 2010). If the formation of a cold joint is unavoidable, the compaction procedure should be carefully planned and executed. With this in mind, cold longitudinal joints in RCCPs become indispensable if at least one hour has elapsed since the placement of the first lane.

Comprehensive details regarding the compaction procedures for fresh and cold longitudinal joints in RCCPs have been limited In the existing literature (Harrington et al., 2010; Pittman, 1986; Shin & Carboneau, 2010). The typical compaction procedure employed in RCCP construction for fresh longitudinal joints is illustrated in Figure 3. These methods have historically been developed based on anticipated mechanical behaviors of fresh RCC, primarily derived from on-site observations. Shoenberger (1994) emphasized the importance of laying RCC in a specific manner to create fresh longitudinal joints efficiently within a single uninterrupted shift. This approach involves leaving a 300-450 mm section of the first lane uncompacted (as shown in Figure 3a), which initially increases the apparent thickness of the uncompacted fresh region. The final thickness of the fresh zone largely depends on the height of the screed used in constructing the second lane. Consequently, the screed's height is adjusted to match the thickness of the uncompacted part of the first lane. Once the second lane is placed adjacent to the first, it is compacted by a roller whose centerline typically aligns with the surface projection of the fresh longitudinal joint. Ultimately, both edges of the adjacent lanes are compacted simultaneously. Achieving the desired preset density within the inner portion of the lane (mat density) and ensuring a smooth surface along the joint zone may require multiple roller passes (Harrington et al., 2010). Achieving the target density and surface smoothness may necessitate more passes than commonly practiced, exposing the centerline proximity to additional roller passes compared to the far edges.

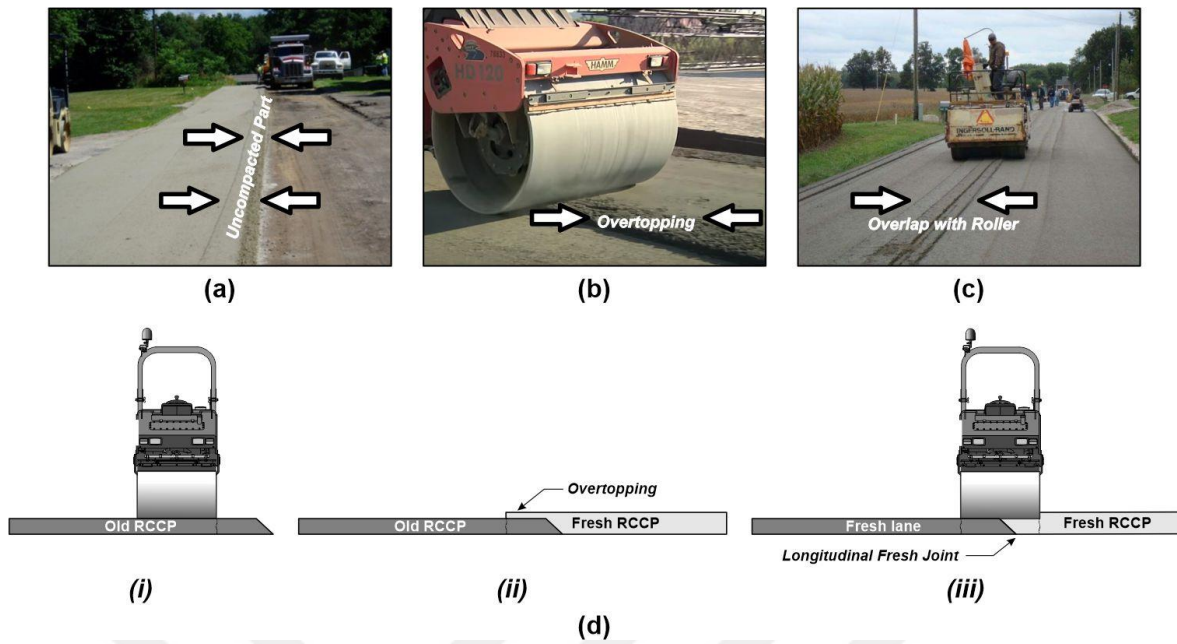


Figure 3. Construction stages of a fresh joint: (a) Leaving an uncompacted edge of the first lane, (b-c) Compaction of the fresh joint between the two adjacent fresh lanes (Shin & Carboneau, 2010), and (d) Schematic for the construction of fresh joints in RCCP

The second lane is laid and compacted to match the final thickness of the adjacent strip. In that regard, the second lane is intentionally cast slightly thicker than the first lane, resulting in some overtopped material. Typically, the overtopping material is, by and large, placed along the edge of the old lane (as shown in Figure 3b and Figure 3d-ii). It is widely believed that the quantity of overtopping material plays a crucial role in the quality of longitudinal joints, both in HMA and RCC pavements. However, it's important to note that this belief is based mainly on previous engineering judgments, primarily derived from semi-intuitive and empirical data. This widespread belief available among engineers has not yet been extensively tested for accuracy. In some cases, a tapered wheel attached to the roller drum is used during the compaction of the first lane. Subsequently, a paver is employed to lay the second lane, resulting in an overtopping zone along the paving direction, precisely aligning with the surface projection of the longitudinal joint, as illustrated in Figure 4.

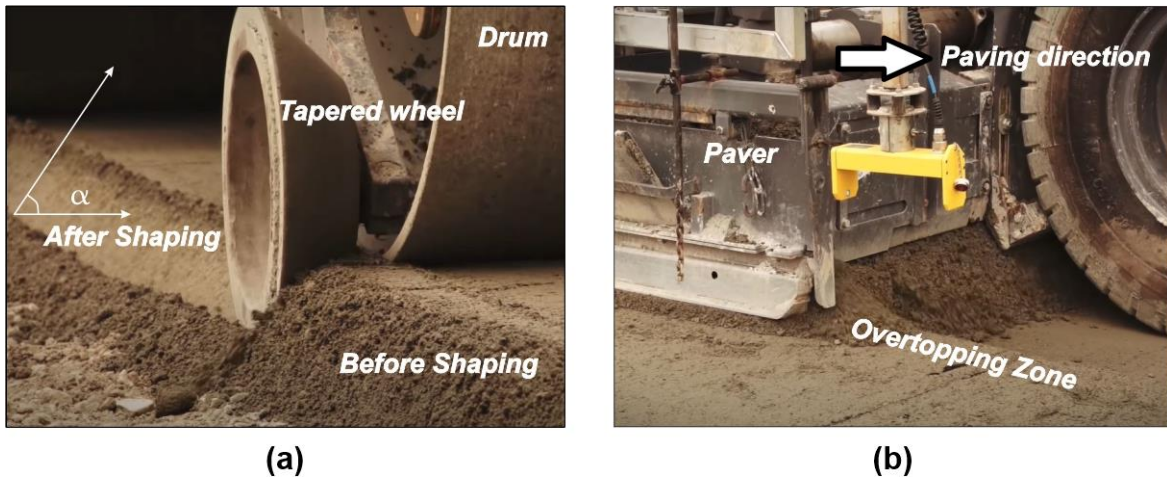


Figure 4. Construction stages of a fresh joint: (a) Shaping the joint face, (b) Laying down the fresh RCC over the second lane, some portion of which rests on the longitudinal joint

Several yet unresolved issues persist in pavement construction, particularly in managing the overtopping material within the overtopping zone. When the second lane is laid with excessive thickness onto the first lane, including some overlap, the roller may either plunge or dip into the overtopped material, depending on its position. Conversely, if the overtopping material is too thin, it becomes prone to cracking under the roller, leading to inadequate joint density. To address this, it is common practice in HMA pavements to maintain a preferred overtopping thickness ranging from 25.4 mm to 38.1 mm (1 to 1.5 inches). Chen et al. (2013) demonstrated that pushing additional overtopping material onto the surface projection of the longitudinal joint significantly improves joint density in HMA pavements. Excess material can be removed using a shovel or similar tools. While raking the material toward the edge of the second lane is possible, it is generally recognized in HMA pavement construction that avoiding any disturbance or raking of the joint is crucial for joint durability (Benson & Scherocman, 2006). In contrast, Shoenberger (1994) proposed the removal of the free edge of cold longitudinal joints through saw-cutting, either partially or full-depth, starting at a distance of 4 to 6 inches away from the joint (as shown in Figure 5e). This process, while involving a form of trimming, should be performed after full compaction of the first lane. Full-depth saw-cutting is often preferred but can potentially lead to micro-cracks and pre-deformations in the concrete. Nevertheless, it aids in creating a pseudo-right-angle longitudinal joint. Alternatively, paving shoes can practically be used to establish an angled or perpendicular interface surface (as depicted in Figure 5b-c). Paving

shoes are plates typically oriented at a 15 to 30-degree angle with respect to the vertical axis of the joint.



Figure 5. (a) Removal of a full-depth-trimmed unsupported edge of the first lane (creation of a right angle longitudinal cold joint), (b) Example of using an angled-paving-shoe as casting the second lift of two-layer RCCP (Harrington et al., 2010), (c) Example of using a different shoe type to create an acute-angled taper of the joint (Park et al., 2020), (α : the slope angle of the longitudinal joint)

Typical rolling patterns, commonly employed for compacting cold longitudinal joints in RCCPs, are illustrated in Figure 6. While it is widely acknowledged that various methods have been used in practice, including those distinct from the specific procedure depicted in Figure 6, to the best of my knowledge, no well-documented study is available that aims to

compare these different methods and provide direct recommendations for their practical implementation.

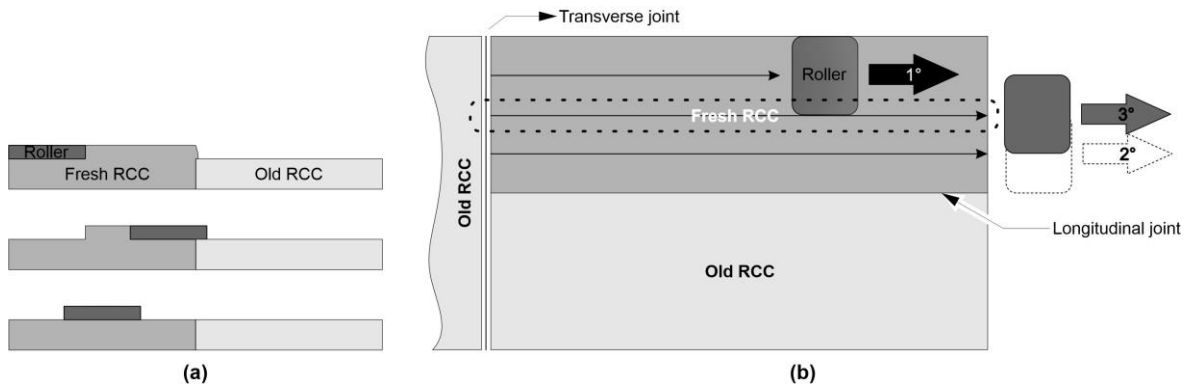


Figure 6. An example of a rolling and paving sequence for a 2-lane RCCP with a cold longitudinal joint (adapted from Shin & Carboneau (2010)): (a) Cross-section view, (b) Plan view

While longitudinally thickened edges are commonly favored in multilayer RCCPs, it's important to note that they can occasionally initiate visible reflective cracks on the pavement surface. These cracks are often attributed to partial or insufficient bonding between the bottom and top lifts of the pavement. However, it is essential to emphasize that longitudinally thickened edges offer significant advantages, particularly for lanes subjected to heavy-duty operations and extreme load repetitions. Intentionally creating these thickened edges, forming a raised (bumped) zone typically positioned within the 450- or 600-mm portion of the lane, can be beneficial in such cases.

1.3.3. Longitudinal Joint Construction Techniques Used in HMA pavements

A far-going compendium was gathered and presented hereunder upon comprehensive literature research about the techniques used to date in longitudinal joint construction of HMA pavements. Inspired by these techniques, several potential longitudinal joint construction techniques for use in RCCPs were defined. The terms bond coat and tack coat are commonly used interchangeably in practice to bind two adjacent layers in RCC technology. However, bond and tack coats are justifiably thought to serve different functions in asphalt pavements, depending on the specific elements and parts of the pavement structure where they are applied. While the terms bond coat and tack coat are often used

interchangeably in practice, it is worth noting that in the original methods compiled and reiterated below, "bond coat" is preferred over "tack coat". In short, the term bond coat is a synonym of tack coat in British English. The term "tack coat" is not typically used in concrete pavement technology. However, tack coats are more commonly associated with asphalt pavements, which are clearly crucial in promoting adhesion between asphalt layers. This preference is primarily based upon the common usage of the tack coat, which aligns more closely with the techniques developed for asphalt pavements in the USA. The known methods developed to date for HMA pavements are as follows:

- I. Rolling from the hot side: In this technique (see Figure 7a.), the compaction is made through the roller first from the hot side while the large portion of the roller wheel is on the hot side with an overlap of 152 mm (6 inches) in the cold lane.
- II. Rolling from the cold side: In this technique (see Figure 7b.), as the roller is in static mode, the rolling is carried out as the large portion of the roller is on the cold lane and with about 152 mm (6 inches) of the roller is on the hot side of the longitudinal joint.
- III. Rolling from the hot side 6 inches away from the joint: In this technique (see Figure 7c.), as the roller is in vibratory mode, the initial pass is performed as the roller wheel is entirely on the hot lane, albeit about 152 mm (6 inches) off from the joint. This technique is recommended chiefly by several practitioners who prophesy that it makes much stiffer lifts possible. However, the relatively thick HMA makes the entire lift prone to being pushed onto the joint, causing the longitudinal joint to be denser.

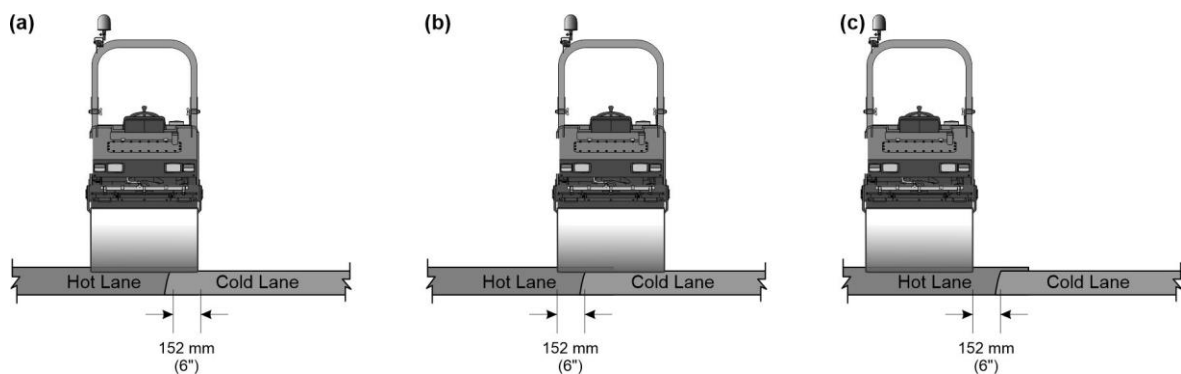


Figure 7. (a) Rolling from the hot side, (b) Rolling from the cold side, and (c) Rolling from the hot side 6 in (152 mm) away from the joint

- IV. Tapered (12:1) joint with 12.5 mm offset without a tack coat: This technique, also called the Notched Wedge joint or Michigan Wedge joint (see Figure 8), is implemented by overlapping two inverted wedges relative to each other. Simplistically, the Michigan Wedge joint is constructed by creating a taper with 1:12 (vertical/horizontal) that ends with a vertical notch at the end of the first-paved lane. No tack coat is used in between the cold and hot sides.

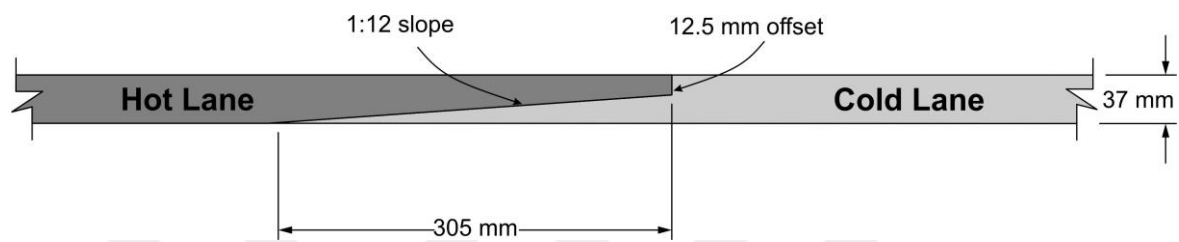


Figure 8. Typical scheme for Michigan Wedge joint (i.e., 12.5 mm offset and 12:1 taper), adapted from Kandhal & Mallick (1997c)

- V. Tapered joint (12:1) with 12.5 mm offset with tack coat: In this technique, the order of implementation is diametrically similar to the Michigan Wedge joint except for a tack coat. The tack coat is implicated on the taper's surface right before the placement of the adjacent wedge, which is also the confined edge of the hot lane.
- VI. Edge restraining device: In this technique (see Figure 9), a cleaver-like apparatus is used to restrain and compact the joint area when placing the first lane. The principal merit of this device is that it increases the material density in the edge zone. The compaction is then performed, as is recommended in the rolling technique from the hot side (I).

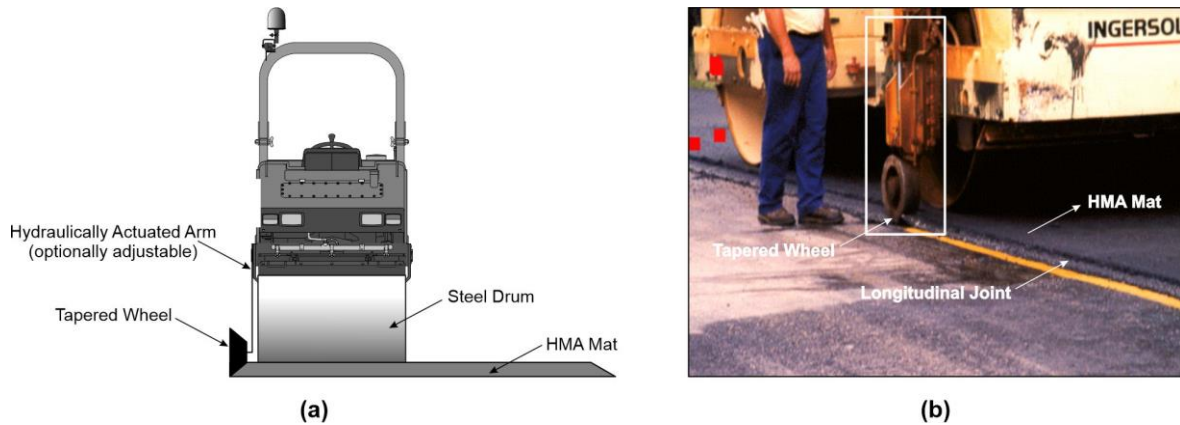


Figure 9. (a) Scheme for edge restraining device (tapered wheel) installed on the roller, (b) Example of use of edge restraint device in HMA coating, adapted from Fleckenstein et al. (2002)

- VII. Cutting wheel with a tack coat: In this technique, as the name suggests, after compaction, 38–51 mm (1½-2 inches) of the unconfined edge of the first lane is removed while the mixture is still in a formable state, exhibiting a degree of plasticity. Immediately following the removal of the lane's edge, a tack coat is applied to the interface surface.
- VIII. Cutting wheel without a tack coat: This technique is identical to the prior technique (the cutting wheel with a tack coat, VII), but no tack coat is used in any phase.
- IX. Joint maker: This method takes its name from the tool used in the implementation of the technique. Namely, a styling device mounted on a corner of a screed's side called a joint maker, as seen in Figure 10. The procedure that results in an extrusion just before the screed concurrently aids in pushing excess material onto the joint simultaneously. A kick-plate mounted to the screed is also used to move the imbricated HMA backward without a man-force. Thus, the compaction is performed as determined in the compaction technique from the hot side.

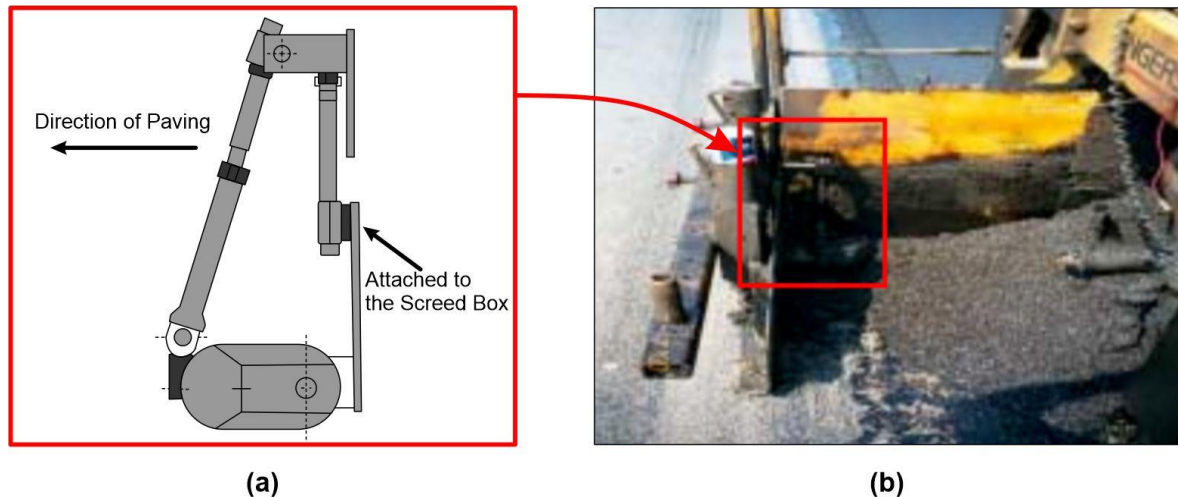


Figure 10. (a) Scheme for edge restraining device (tapered wheel) installed on the roller, adapted from Kandhal & Mallick (1997c), (b) Example of use of edge restraint device in HMA coating, adapted from Fleckenstein et al. (2002)

- X. Tapered (3:1) joint with vertical 25 mm offset: In this technique, the unconfined edge of the first lane is constructed in the shape of a vertical offset (a slip or a step) of 25 mm created at the top of the joint. The rest is formed with a taper (3:1) using an angled-iron under the drag device (assembly). The tack coat is then applied to the surfaces right before the placement of the second lane. The pavement is then finally compacted from the hot side.
- XI. Rubberized asphalt tack coat: This method takes its name from the nature of the adhesive layer used in the technique. Before placing the second lane, a rubberized asphalt tack coat is applied to the surface of the unconfined edge of the first lane. The typical thickness of the tack coat is about 3 mm, and the rolling is then performed from the hot side.
- XII. New Jersey wedge (3:1): This technique comprises a sloping-wedge and is mainly called “New Jersey Wedge” in practice. A sloping steel plate mounted to the inner corner of the paver screed, which is an extension to the main screed, is utilized to create the wedged form. During the second lane placement, an infrared heater is used to heat the edge of the first-laid lane to a surface temperature of about 90 °C, and the second lane is imbricated into 50.8–76.2 mm (2 to 3 inches) of the first lane. The imbricated material is then pushed backward 76.2–101.6 mm (3 to 4 inches) from the edge of the cold mat, and the pavement is then finally compacted from the hot side.

In relation to the abovementioned techniques, various studies emphasize different methods as superior, particularly their efficacy in the specific environments where they were tested. Baker et al. (1990) published the findings of a comprehensive five-year study that focused on five projects related to bituminous pavements. Their investigation centered on comparing the effectiveness of wedge joints with conventional joints, which were either considered to have arbitrary shapes or to be right-angled. Their conclusion revealed that wedge joints exhibited a superior density gradient across the joint compared to standard butt joints. Moreover, they underscored the substantial technical advantages of wedge joints over traditional butt joints regarding resistance to deformation caused by distress. Baker et al. (1990) also observed that wedge joints eliminate the potential for vertical drop-offs, thus providing a safer wear course for drivers. However, it is worth noting that while they highlighted the merits of wedge joints, they did not provide specific geometrical figures, and thus, a direct comparison was not feasible.

A few years later, the rolling from the hot side technique (150 mm (6 inches) away from the joint) emerged as the preferred variant, gaining unanimous acceptance among engineers (Kandhal & Rao, 1994). Besides, Kandhal & Rao (1994) observed that the Michigan wedge joint and cutting wheel technique yielded notably favorable results compared to other tested methods. Subsequently, Kandhal & Mallick (1996) conducted a comprehensive cross-state study in Colorado and Pennsylvania, assessing seven distinct longitudinal joint construction techniques. Their findings revealed that the tapered joint technique, featuring a 3:1 ratio and a vertical 25 mm offset in conjunction with a tack coat, ranked as one of the most superior among all tested methods. The second-best technique, as reported in the same publication, involved using a cutting wheel with a tack coat and a 3:1 taper. Furthermore, when evaluating three rolling techniques across four separate projects, Kandhal & Mallick (1997b) concurred that 'rolling from the hot side,' positioned 152 mm (6 inches) away from the joint, consistently yielded the most favorable results, aligning with the observations made in the Michigan project. McDaniel et al. (2012) comprehensively assessed the advantages and drawbacks of longitudinal joint construction techniques commonly employed in HMA pavement construction. According to their findings, echelon paving, while effective in preventing cold joint formation, increases overall costs. This increase is primarily attributed to the need to duplicate various components of the construction process, including work crews, paving equipment, and compaction rollers. Within the context of rolling patterns, it is essential to note that the optimal choice can vary depending on the specific construction

circumstances, and experience plays a vital role in determining the most suitable approach. Regarding joint styles, butt joints are the most familiar ones. When employing the butt joint technique, achieving a clear free edge demarcation involves lifting the adjacent strip, a practice that typically leads to water ingress over time. Tapered joints, on the other hand, effectively mitigate edge drop-off concerns but necessitate meticulous compaction both within and around the wedge. Using edge restraining devices enhances the longitudinal joint's density but demands high craftsmanship. While effective, the cutting wheel technique can result in some material wastage and requires additional sawing and debris removal during execution. Moreover, incorporating a tack coat for adhesive and sealing purposes can contribute to an overall cost increase. However, the incremental cost may be minimized if the tack coat is applied consistently throughout the pavement. Nonetheless, conducting an early cost-benefit analysis before adopting a tack coat is imperative, considering both potential benefits and total expenses.

1.3.4. Longitudinal Joints in RCC Pavements

Of all types of pavement joints, longitudinal joints, which either make the wearing course last longer or get the pavement quickly exposed to damage, are not only crucial but potentially the weakest elements of any pavement. Upon a detailed study, Mallick & Daniel (2006) noted that if longitudinal joints are not constructed in sufficiently good quality, the joint itself tends to become the weakest element of the surface course, making the pavement highly prone to the propagation of distresses and permanent damage. To the best of my knowledge, the most compelling argument on this topic was highly likely presented by Foster et al. (1964). Their research first posited that the way to properly judge the suitable longitudinal joint construction technique for HMA pavements differs from what was thought to be erstwhile. Having been fused by Foster et al. (1964), the longitudinal joint construction problem became a vital research topic in pavement engineering.

Remarkable progress has been achieved in developing construction techniques to enhance the performance of longitudinal joints in pavements, albeit with a greater focus on the related problems in asphalt pavements. Several kinds of research have as yet examined issues of the longitudinal joint construction in HMA pavements to elucidate the best longitudinal joint construction techniques (Akpınar & Hossain, 2004; Baker et al., 1990; Brock & Skinner, 1997; Brown, 2006; Denehy, 2005; Fleckenstein et al., 2002; Foster et al.,

1964; Hasan et al., 2015; Kandhal & Mallick, 1996, 1997c; Kandhal & Rao, 1994; Scherocman, 2002; Toepel, 2003; Williams, 2011). Given that HMA pavements comprise the majority of global pavements, it is imperative to promptly address the current deficiencies in the technical requirements for longitudinal joints in HMA pavements. Be that as it may, while longitudinal joints constitute significant segments of RCCPs and are prone to cracking or raveling for various reasons, the problem of longitudinal joint failure has not yet been extensively solved. Namely, there has been little, if any, improvement made on this issue to date. Most of our technical know-how on longitudinal joints for RCCPs is based on some experience and limited knowledge, most of which was gained from the research made explicitly for HMA pavements. Several possible longitudinal joint types, some of which are inspired by those used in HMA pavements, are set out in Figure 11.

The first and foremost question that comes to mind is whether or not any parallelism exists between RCCP and HMA pavements regarding longitudinal joint construction techniques. Upon initial examination, several design suggestions proposed for the longitudinal joints of HMA pavements appear to be inapplicable to RCCPs. For example, Kandhal & Mallick (1997b) noted tapered joints with a 1/12 slope as one of the best practices in longitudinal joint construction for HMA pavements. Considering the typical thickness of the surface course in an HMA pavement, which ranges from approximately 70 to 120 mm, the tapered joint with a 1/12 slope seems relatively applicable. However, adopting the same slope without modification for RCCPs, whose total thickness ranges from approximately 150 to 400 mm, appears to pose a challenge (the latter margin is for a two-layer construction). Moreover, current longitudinal joint construction techniques applicable to RCCPs share similarities with those used in HMA pavements, although they are not identical due to differences in material properties and mechanical behavior.

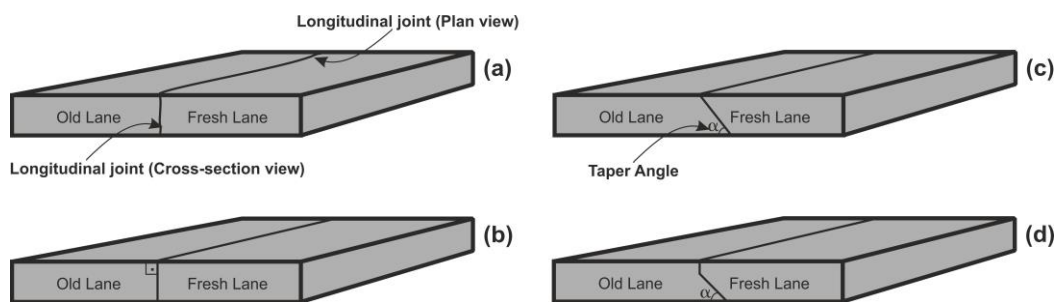


Figure 11. Common types of longitudinal joints that can be used in RCC Pavements: (a) Ordinary, (b) Cut-Back or perpendicular, (c) Wedge, (d) Notched wedge

Of very few early studies on longitudinal joints in RCCPs, Rollings (1988b) expounded that even a slight decrease in the mat density of RCCP directly causes a significant decline in the longitudinal joint's flexural and compressive strengths. Upon examining several problematic longitudinal joints exposed to raveling and several other distress, Ragan et al. (1990) ascertained that vulnerability to distress directly originated from the apparent decrease in the material density that created the RCC longitudinal joints. This finding is parallel to the remarks made by Foster et al. (1964) for HMA pavements long before. Upon those first and pioneering works, the U.S. National Center for Asphalt Technology (NCAT) initiated a long-term research project comparing several longitudinal joint construction techniques in 1992. After completing all phases, several documents (Kandhal & Mallick, 1996, 1997c; Kandhal et al., 2002; Kandhal & Rao, 1994) were released. However, no consensus on naming a particular technique as the best one was reached after these studies, nor has it been reached to date. Furthermore, the U.S. National Asphalt Pavement Association released a report (NAPA, 1998) on the same matter; however, it did not recommend a specific technique as superior or inferior.

When assessing the sequence of lane placement in HMA pavements, joints are categorized into two fundamental types: hot and cold. These concepts of hot and cold joints have been integral to asphalt pavement technology since its early days (Kandhal & Rao, 1994). The term "hot joint" typically refers to a scenario in which the paving of two adjacent lanes occurs consecutively. Conversely, a "cold joint" refers to a situation in which the first lane is initially paved and compacted before the second lane. It is commonly expected that hot joints are generally favored over cold ones in HMA pavements (DAV, 2001). However, in the realm of RCC pavements and other concrete pavement types, the material exhibits neither significant temperature fluctuations over time nor potential impacts on rigidity. Consequently, the terminology used in concrete pavements employs the concepts of "fresh (unhardened)" and "hardened (set)" to describe material properties, rather than the "hot" and "cold" terms. The U.S. Army Corps of Engineers (USACE, 1988), introduced a novel perspective against the terms "hot and cold" by suggesting the words "old and new" for the lanes in RCCPs. In summary, within RCCP practices, when a joint is constructed within 60 minutes after the first lane's placement, it is referred to as a cold joint, sometimes termed as a "hardened" joint (Harrington et al., 2010; IDOT, 2014). However, it is worth noting that throughout this study, the terms "Old" and "Fresh" joints were consistently used, in line with the terminology put forth by Zollinger (2016).

1.3.5. Factors Leading to Joint Deterioration in RCC Pavements

There are several potential sources of cracks in various types of pavements, which can broadly be categorized into a couple of primary groups, including fatigue of the pavement structure (partial or all-out), material shrinkage, subgrade soil movement, construction defects, aging, and exposure to distresses due to severe environmental conditions (Colombier, 2004). The surface course of a pavement is typically considered a mat resting on an ideally elastic or elastoplastic base layer. The two opposite far edges of the mat are often modeled as unconfined. Under normal circumstances, namely in the absence of wear or settlement in the base layer, significant deformations or displacements at the pavement's free ends should ideally not occur, even if they are not confined. In these instances, where no apparent deformation occurs in the base layer or subgrade directly beneath the longitudinal joints of the surface course, the question of why cracks predominantly develop in and around these joints instinctively comes to mind. All the more so, when these joints are positioned along the pavement's centerline, it becomes a practical reality that they are less exposed to repetitive axle loads. In that sense, the lack of flexural rigidity or uneven distribution around the longitudinal joint zone is more pronounced as the most significant weakening and incapacitating factor of the mat than other mechanical aspects.

Past experiences highlight a critical issue: the apparent lack of joint rigidity in concrete and HMA pavements suggests unevenly distributed material density within longitudinal joints. It directly underscores the importance of material properties for the resilience of the pavement mat. Fundamentally, higher material density, directly achieved by minimizing air voids and ensuring a smoother surface, is essential for optimal pavement resilience (ERMCO, 2013; Harrington et al., 2010; Huang et al., 2010; IDOT, 2014; Pittman & Ragan, 1986).

Because of the low moisture content of fresh RCC mixtures, there is a higher probability of low drying shrinkage, leading to natural cracks in RCC that are larger than those in conventional concrete (Pittman, 1994). While the early examples of some RCC pavements were initially allowed to crack naturally to reduce construction and maintenance costs, it is widely recognized that joint construction, whether longitudinal or transverse, can help mitigate crack initiation. A joint spacing of 6 meters is typically employed for 200-mm thick RCC pavements. In RCC pavements, particularly in the transverse direction, sawed joints have been found to be effective in reducing the abrupt initiation of random cracking

(Harrington et al., 2010). Shrinkage-related crack initiations in RCC pavements usually lead to small-scale cracks, typically less than 3 mm thick. However, these cracked areas continue to support load transfer through aggregate interlock (Halsted, 2009).

1.4. Problem Statement

When a pavement is of multiple lanes constructed in several phases, either due to geometrical constraints regarding the paving width or other reasons, the longitudinal joints serve as the interface between the previously constructed lane and the newly constructed one (DAV, 2001). As with any other design-based element of pavements, the longitudinal joints must be of "acceptable quality," particularly for the whole pavement body's resilience. However, answering questions about "what quality benchmark must be met" and "what the optimum longitudinal joint construction technique to be implemented in RCCPs is" remains challenging. Existing research on the performance of longitudinal joints in RCC pavements is notably limited. In contrast, investigations into similar issues within asphalt pavements have been ongoing for years (Kandhal & Mallick, 1996, 1997c, 2007; Kandhal et al., 2002; Kandhal & Rao, 1994; Roberts et al., 1991), yielding valuable but often inconclusive insights. A recurring lesson learned from numerous experiences underscores that longitudinal joints should be considered the most potential weak points in pavement structures.

The proneness to cracking and, therefore, vulnerability potential is primarily attributed to suboptimal compaction in the vicinity of the longitudinal joint, leading to reduced material density and increased permeability. Consequently, a significant loss of the pavement's overall resilience becomes obvious (Foster et al., 1964). Based on tests and observations on HMA pavements, the material density gradient across the joint is primarily attributed to the comparatively weaker unconfined edge during the initial lane paving. Subsequently, as the adjacent (second) lane is paved, a relatively higher density is achieved on the side of the confined edge (Kandhal & Mallick, 1996, 1997a; Kandhal & Rao, 1994; Roberts et al., 1991). Furthermore, surface irregularities in longitudinal joints represent an often-overlooked concern in RCCPs. The rapid lane-changing during compaction creates significant disturbances, leading to water accumulation in and around the joint zone. This issue is particularly problematic in HMA pavements, as it can result in poorly constructed joints prone to premature raveling and even cracking (Kandhal & Mallick, 1996). The typical

cracks that occur in longitudinal joints and their propagation patterns in various pavements are depicted in Figure 12.

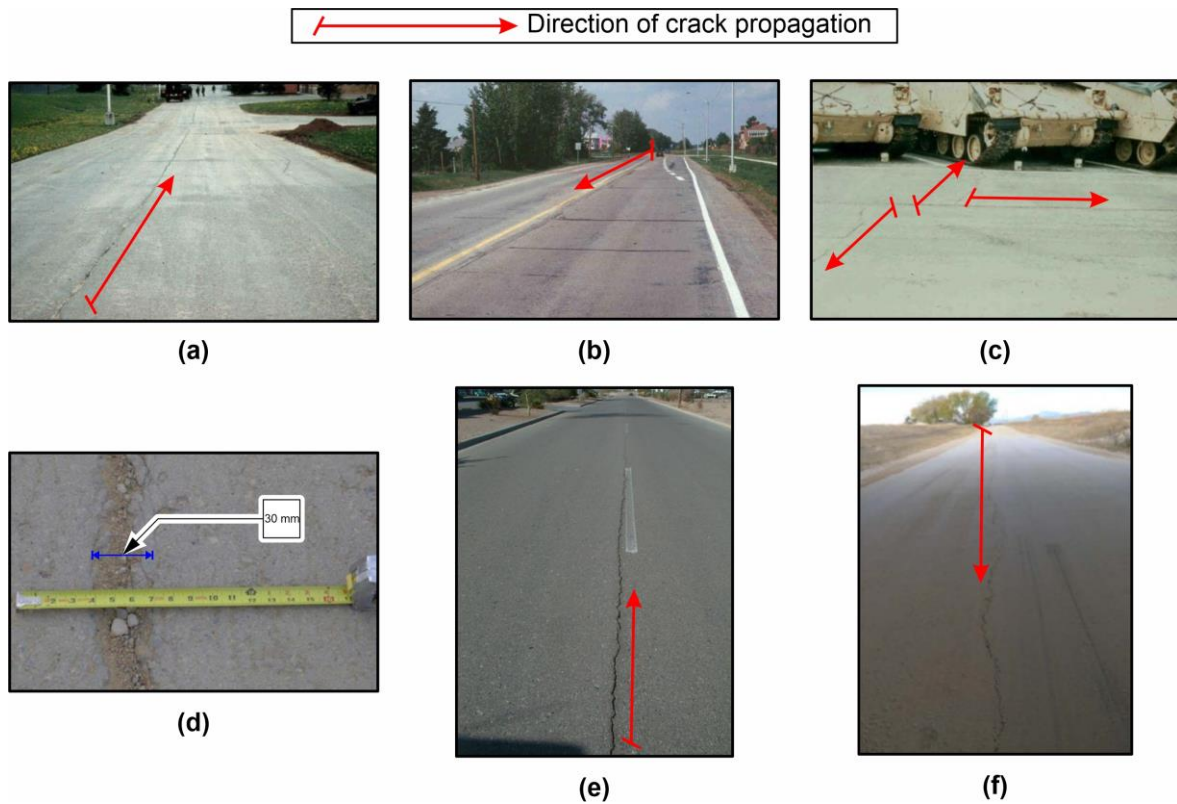


Figure 12. Typical cracks in longitudinal joints and their propagation along and off the joint

This study aims to provide comprehensive insights into longstanding inquiries by addressing several pivotal questions. Specifically, I seek the answer to the question: "Why is it likely not to reach a good degree of compaction in the longitudinal joints of RCC pavements?" and "Which cold longitudinal joint construction technique, when coupled with adjacent surface reinforcement treatment and employing different slope angles of the tapered adjacent surface, yields more robust longitudinal joints." Additionally, I explore the question of whether a suitable cold longitudinal joint construction technique exists capable of eliminating possible irregular material attribute distribution at and near the joints. Furthermore, I tried to investigate the extent to which the initial position of the roller, often considered ineffective in practice, influences the ultimate performance of RCC compaction.

1.5. Research Objectives

This study has two primary objectives: (1) To establish the ideal cold longitudinal joint construction technique for RCC pavements, achieving the highest possible mechanical resilience and uniform material density, thereby minimizing air-void scattering in and around the joint. This approach aims to reduce the likelihood of premature cracking initiation within the joint, enhancing the overall strength and resilience against potential distress. (2) To determine the most effective shape of the adjacent face (butt, tapered, or notched wedge) and the reinforcement treatment technique that yields a resilient cold longitudinal joint, with the goal of maximizing the cold joint's strength and resilience.

1.6. Scope

A comprehensive research strategy encompassing two main approaches was devised to achieve the objectives of this study:

(1) Laboratory-Scale Experimentation: In this phase, scaled-down laboratory samples are prepared to simulate various longitudinal joint scenarios. This phase employs different taper angles and notched-wedge geometry within different reinforced interfaces. The mechanical properties of RCC and RCC longitudinal joint models are rigorously assessed using the model samples.

(2) Numerical Analysis: This phase involves comprehensive numerical analysis, primarily driven by model-based Finite Element Analyses (FEAs). This phase aims to determine the most effective longitudinal joint construction techniques based on the results of deformation outputs from the FEAs. The ultimate goal is to establish the most effective, mechanistically optimized, and ideal construction technique for cold longitudinal joints in RCC pavements.

1.7. Limitation

As with any engineering problem, theoretical approaches based on ideal or hypothetical conditions often fall short of fully resolving complex issues. However, to maintain the brevity and feasibility of this study, it is necessary to limit the number of experiments conducted and the range of parameters investigated. In this context, Finite Element Modelling (FEM) offers engineers a valuable tool for optimizing the number of

parameters while reducing the need for an extensive array of physical experiments. Given the inherent variability among physical models, FEA emerges as a sophisticated and rational computational tool, offering insights that would be challenging to attain through physical experimentation alone. I utilized the open-source and free-of-charge FEA software called Calculix PrePoMax v2.0.0. This software supplies the user with a powerful pre- and post-processor that helps the user accelerate the FEM workflow, particularly for one of the powerful solver engines called the Calculix FEM solver, which is under GNU General Public License Version 2. I utilized Calculix PrePoMax to explore longitudinal joint finite element models encompassing various joint shapes and to examine the impact of the initial compaction position on the first-run compaction performance of the roller drum. However, it is essential to note that FEA, due to its hypothetical nature and idealized physical conditions, may yield results with some degree of uncertainty. Additionally, the concrete-to-concrete bonding phenomena existing in the longitudinal joint interface using various bonding agents (bond coats) are examined to determine the extent to which bonding is adequate and effective. At this juncture, it is essential to note that despite the wide variety of bonding agent alternatives available in terms of type and features, I limited the number for ease of comparison. Considering that testing all possible longitudinal models and their taper angles is not practically feasible, it thus falls beyond the scope of this study. Nevertheless, it is apparent that both the taper angle and its shapes and the interface reinforcement materials employed and tested for effectiveness are typically the first choices that come to mind for engineers and are the easiest for them to obtain in practice.

1.8. Thesis Organization

This thesis consists of eight themed chapters, one of which is the references and another is the appendix. Introduction (Chapter 1): This chapter provides the background and context for the study, presenting the main objectives in detail. Literature Review (Chapter 2): This chapter extensively covers the topics related to the thesis problem, including RCC material properties and testing methodologies. Research Methodology (Chapter 3): This chapter outlines the analytical and experimental procedures conducted in the study. Results and Discussion (Chapter 4): This chapter summarizes the study's results and presents in-depth discussions. Conclusions (Chapter 5): This chapter draws upon the entire thesis, providing final remarks and discussing their potential use in practice and future development.

prospects. Recommendations for Future Works (Chapter 6): This final conceptual chapter offers recommendations for future studies, highlighting potential gaps and yet unresolved issues. The reference list and appendices come as follows.

A flow chart in Figure 13 illustrates the research methodology employed in this study, including key steps such as data collection, experimental procedures, analysis techniques, and result interpretation.

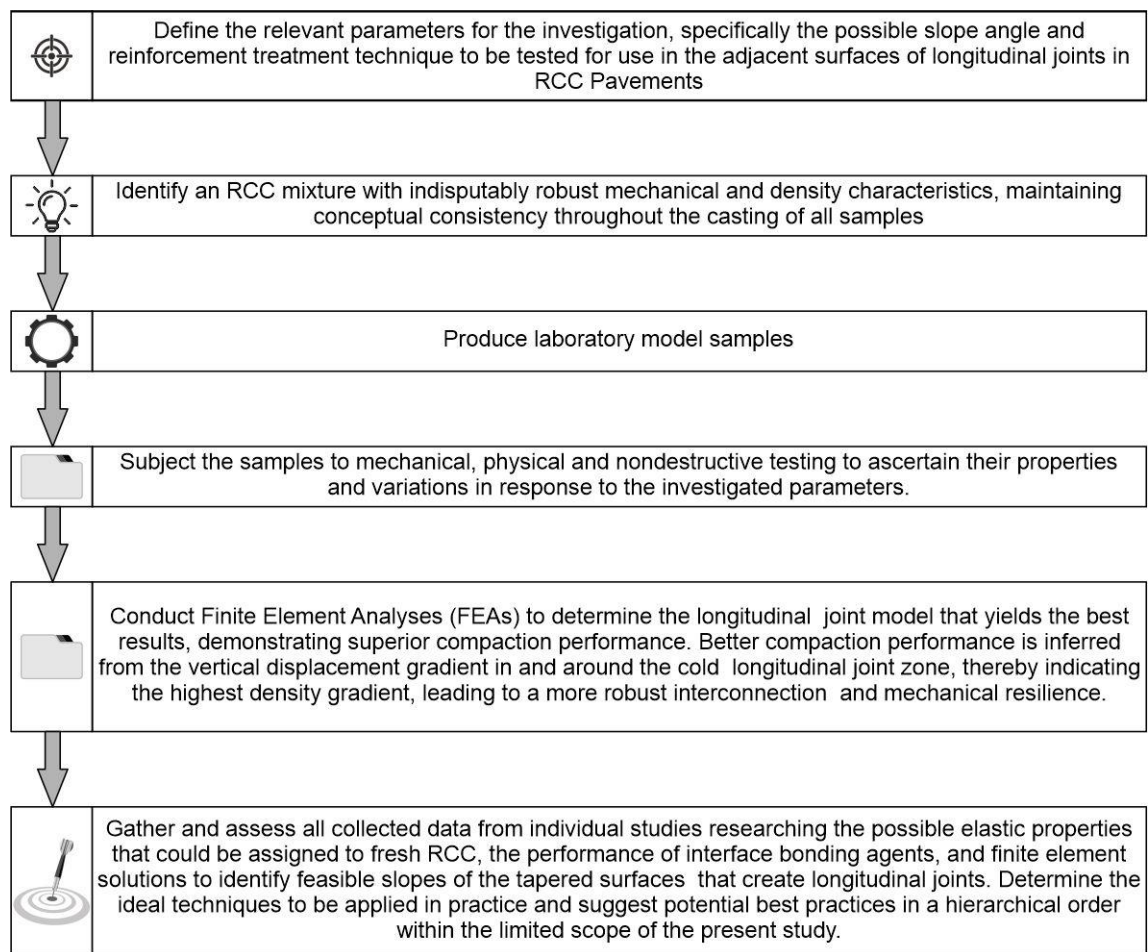


Figure 13. Flow chart for the research methodology employed in this study

2. LITERATURE REVIEW

This chapter presents an extensive literature review on hardened and fresh RCC attributes and the longitudinal joint construction techniques applicable to RCC pavements. The review aims to establish the foundation of scientific knowledge that underpins the present study. Below, I provide in-depth details on the relevant literature across several pertinent subchapters.

2.1. Basics for RCC and RCC Pavements

RCC, commonly referred to as zero-slump concrete, demonstrates material properties that encompass various characteristics derived from different materials, as depicted in Figure 14a. In Figure 14b, the general appearance of fresh RCC is displayed. With the exception of material proportioning, all ingredients employed in RCC are analogous to those utilized in conventional concrete. A noteworthy trait of RCC pavements is that formwork or highly advanced construction machinery is unnecessary. This characteristic closely resembles that of HMA pavements.

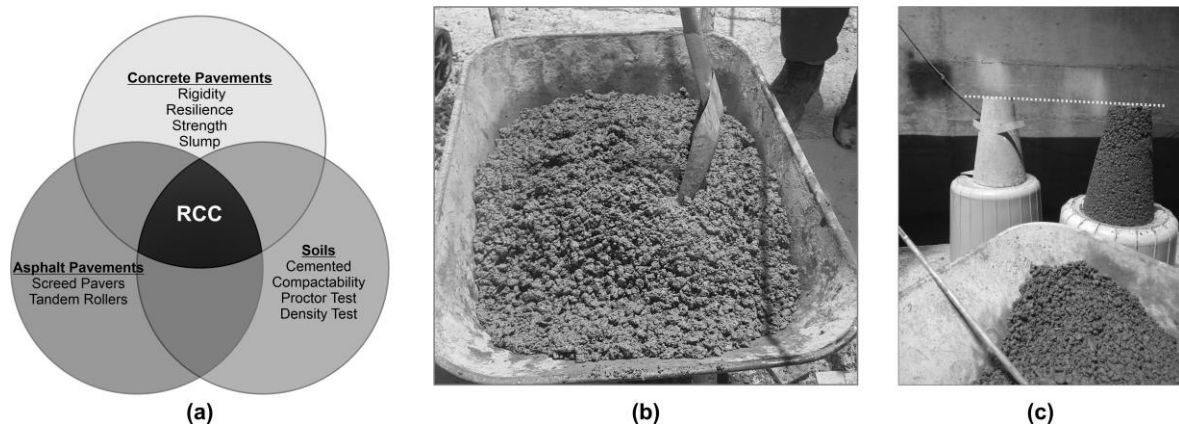


Figure 14. (a) Venn diagram for the nested and discrete features of RCC pavements, (b) Common physical appearance of fresh RCC, and (c) Representation of the zero-slump feature of RCC

The utilization of RCC in the construction of pavement surface courses represents a relatively recent technological advancement that has been evolving for decades (ACI, 1995). Despite claims of its historical origins dating as far back as the 1930s, the first

groundbreaking RCCP, as indicated in the related literature, was introduced in North America in 1942 (ACI, 1995; Anderson, 1986; Vahidi & Malekabadi, 2012). However, the consolidated terminology was just published in 2001 by the American Concrete Institute (ACI) with the Report ACI 325.10R, which provides an in-depth terminological description of RCC pavements (Berry et al., 2001). The early examples of RCC pavements were initially recommended for roads expected to endure heavy-load and low-speed traffic (i.e., tracked vehicles or high-pressure tire traffic) since no load transfer mechanism and joint spacing routines were unknown then (Shoenberger, 1994).

In addition to its use in low- or medium-speed roads (e.g., municipal streets), RCCPs have long been used for several different purposes to build structures, to wit: highway weigh stations, aircraft parking ramps, truck and automobile-parking slots, airport aprons, docks, container ports, military vehicle roads, multimodal transport hubs, rather substantial industrial facilities such as log and automobile-sorting yards (ACI, 1995; Delatte, 2004). In RCCP construction, RCC is laid over the base layer through asphalt-type high-density screeds, and no formworks and surface finishing are needed. The commonly used RCCP construction procedure is set out in Figure 15.



Figure 15. Consequent construction phases of an RCCP (adapted from Jofré (2019))

In RCCPs, an extra-wearing course is not typical, yet it seems somewhat contrary to its underlying philosophy. However, in some cases, a thin HMA overlay is constructed to

improve the RCC surface's smoothness or is proposed to achieve some maintenance-type benefits (ACI, 1995). The surface textures of RCCP, conventional concrete (plain cement) pavement (PCC), and HMA pavement are shown in Figure 16, respectively.

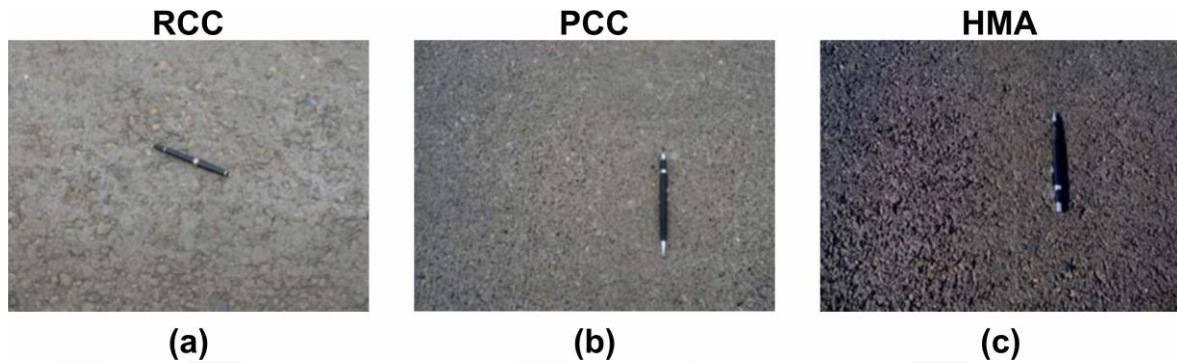


Figure 16. Typical surface textures of (a) RCC Pavements, (b) PCC Pavements, and (c) HMA pavements (Shin & Carboneau, 2010)

Fundamentally, on the one hand, there have been four primary requirements that any RCC must fulfill. First, RCC must have a high material density and a minimum amount of air voids. Second, a desirable degree of concrete-to-concrete bond must be achieved. Third, the heat of hydration must be maintained at a certain level during and after construction. Fourth, hardened RCC must be of a mechanical capacity so that it can safely and soundly compensate for the tensile strength with which to tamp down the thermal cracking initiation.

Using pavement research facilities based on accelerated pavement testing, Mahdi et al. (2020) showed that RCC pavement is the most outstanding candidate for low-to-medium roadway, mainly when the overweight traffic load is underway. From the strength, cost, and easy constructability standpoints, the desire to construct RCC pavements has dramatically increased for several decades. Of all types of pavement, it is the strongest candidate to become Türkiye's most preferred pavement type over time, particularly for local road networks and roads to be paved in the countryside.

2.1.1. Merits and Drawbacks of RCC and RCC Pavements

As with any construction materials and engineered structures, both RCC and RCC pavements possess distinct properties, encompassing inherent advantages as well as certain disadvantages. To summarize, the commonly cited merits of RCC pavements are as follows:

RCC pavements do not necessitate forming or finishing processes, eliminating the need for these labor-intensive tasks. They also do not require steel reinforcement, making the construction process more straightforward and cheaper than other rigid pavement alternatives. Furthermore, they exhibit a robust aggregate interlock, resulting in high mechanical resistance within the joints. The low water-to-cement (w/c) ratio further enhances this intrinsic property, which bestows low permeability and high material density upon placement, even without extensive compaction. The roller compaction method, aimed at bolstering durability and reducing permeability, imparts increased mechanical strength to the pavement, endowing it with a rigid character capable of withstanding highly concentrated loads generated by heavy vehicles and freight.

RCC is regarded as a valuable pavement construction material due to the wide range of mixture materials available for its production. Notably, RCC can be created using a variety of cement alternatives. Moreover, RCC pavements are often praised for their environmental friendliness compared to Portland cement concrete pavements (PCCPs) and HMA pavements. The well-known low cement requirement in RCC production reduces carbon emissions typically associated with cement manufacturing (Harrington et al., 2010). In essence, when ready-mixed concrete production is established in a given area, the potential for RCC production becomes a viable option. However, it is worth noting that if RCC were produced solely with specialized aggregates, it might not be cost-effective due to the expenses associated with aggregate production and transportation. In a simple sense, it can be claimed that if ready-mixed concrete is produced somewhere, the potential of RCC production becomes certain. From another angle, if RCC were made using only specialized aggregates, it would not likely be economical due to aggregate production and transport costs.

One significant advantage of RCCPs is the close resemblance of the basic paving system used in its construction to that of HMA pavements, though not identical. The key benefit of RCCPs over other pavement types lies in their capacity to reduce the total construction cost and time required significantly. This efficiency results from reasonably integrating both asphalt and concrete pavement technologies. RCC is transported to the construction site using dump trucks and loaded into the pavers. The material is then meticulously compacted using a vibrating roller, achieving the highest possible level of compaction. A fundamental distinction between RCCP and HMA pavement is that RCC is

typically placed in a single lift, with a thickness ranging from 200 to 300 mm, making it two to three times thicker than PCCP (Delatte, 2014).

From a moisture content perspective, RCC is a notably dry concrete with virtually zero-slump, a feature that aligns with its distinctive construction method of compaction through a roller (Delatte, 2014). Additionally, as RCC contains less cement paste compared to Portland cement concrete, it tends to produce less bleed water and exhibits reduced shrinkage (Vahidi & Malekabadi, 2012). Furthermore, RCC pavements offer exceptional skid resistance without additional surface finishing. From an economic perspective, RCCP construction results in substantial cost reductions, often amounting to 10 to 30 percent less than the total cost of PCC pavement construction (Pittman, 1996; Shoenberger, 1994). Moreover, the potential service life of RCCPs is regarded as more prolonged than that of HMAPs, extending up to 40-50 years (Zarrin & Shirazi, 2015).

Compared to PCC pavements, one salient drawback of RCC pavements is their potential weakness in load transferability at joints. This issue arises due to the absence of dowel and tie bars, commonly used in conventional pavements that contribute to higher load transfer capabilities (Rollings, 1988a). Additionally, although RCCPs often achieve the desired level of skid resistance, they may occasionally fall short of meeting specific surface smoothness requirements (Shoenberger, 1994). In the early stages, the challenge of producing a sufficiently smooth surface for high-speed traffic hindered the widespread use of RCCPs. Consequently, many RCCP constructions were initially limited to low-speed traffic applications. A recurring obstacle to employing RCC for high-speed highways has been the surface's inherent roughness, which results in elevated noise levels due to tire-wearing course interactions.

The rideability of RCC pavements can be significantly enhanced by incorporating a thin asphalt layer. This addition improves the quality of pavement rideability and enhances its overall resilience against distress. Among the various distress factors, the freeze-thaw effect is widely recognized as one of the most critical elements affecting pavement surface texture and skid resistance (Piggott, 1999). Furthermore, Jofré (2019) emphasizes that RCC becomes an economically viable alternative for motorways and main highways, particularly when used in conjunction with diamond grinding. In addition, applying a thin asphalt overlay in RCCPs provides an ideal solution for creating a robust wearing course capable of meeting the requirements of high-speed traffic.

2.1.2. Mix Design Methods for RCC

Mix design for any concrete refers to a meticulous process determining the proportions of the ingredients used. It serves as the initial step in RCC production, an aspect that should never be underestimated. The mix design significantly impacts the ultimate performance of the concrete product, both in its fresh and hardened states. Sengun (2019) emphasized that compactability is influenced by mixture design and the compaction techniques employed, underscoring the paramount importance of mixture design. RCC mix design can be accomplished using various methodologies. Two primary methods, the "consistency" or "concrete" approach and the "soils approach" or "soil-compaction" approach, have been widely practiced. These methods have distinct characteristics and have been effective in RCC production for an extended period. On the one hand, as its name would suggest, the consistency approach places importance on achieving a mixture with the right consistency. This consistency ensures practical workability, essential for the entire RCC pavement construction process. Various parameters such as water-cement (w/c) ratio, aggregate gradation, sand/aggregate (hereinafter s/a) ratio, and the amount of substitutive pozzolan can be adjusted while preparing alternative mixes (Dolen, 1998). The Vebe time serves as a fundamental benchmark to determine whether the desired consistency is achieved. Optimal Vebe times for RCC typically fall within the range of 10 to 45 seconds, but, be that as it may, practical reports often indicate variations between 15 and 25 seconds (Choi & Hansen, 2005). On the other hand, the soil compaction approach leverages the similarities between the nature of RCC and granular cemented soils. Both share characteristics such as being mixtures with zero-slump properties; they are field cast using earthwork equipment and compacted through rollers. While there are distinct differences in hardened concrete properties, including compressive strength, tensile strength, shear strength, permeability, unit weight, abrasion resistance, and durability, the similarities have led to a simplified viewpoint known as the soils approach or soil-compaction approach. In most cases, this approach is referred to as the "simplified approach" (Reeves & Yates, 1985). Choi & Groom (2001) provided a comprehensive explanation of the method, offering an in-depth overview. In sum, the soil compaction approach follows a sequence that typically includes choosing suitable aggregates for RCC, specifying a range of cement and/or fly ash contents, determining the target moisture content, casting RCC into cylinders for strength and durability tests, and selecting the required cementitious content based on laboratory test

results. In this approach, moisture content is typically expressed as a percentage of the material's dry weight. While the soil compaction method is practical and straightforward for determining RCC mix proportions, it does not inherently assess concrete workability (Chhorn et al., 2019). Regardless of the method used to define mixture proportions, it is essential to consider some fundamental requirements, including constructability, mechanical strength, durability, and economic feasibility (Khayat & Libre, 2014).

2.2. Material Properties and Their Impacts on RCC Pavements

In the strictest sense, any pavement layer's material properties directly impact the service life of pavements. When selecting the right pavement materials, RCC must be produced using suitable and well-graded aggregates, the correct cement, and the appropriate water content. Furthermore, successful compaction is essential to provide the required strength to withstand traffic loads and environmental stresses. RCC pavements exhibit robust strength and durability characteristics, often equal to or exceeding those of PCCPs (Khayat & Libre, 2014). The main ingredients of the RCC mixture are the same as those of conventional slip-formed pavement concrete, albeit with different mixing ratios. The difference in mixing ratios creates one of RCC's well-known emblematic properties called the degree of compactability. The primary ingredients of an RCC mixture are similar to those of conventional slip-formed pavement concrete but with different mixing ratios. These varying mixing ratios give rise to one of RCC's distinctive features, known as the degree of compactability. Achieving the proper proportionality among the ingredient materials is crucial for producing high-quality RCC mixtures (Chhorn et al., 2017).

It is important to note that RCC mixtures used in pavements differ slightly from those used in constructing RCC dams. The key distinction lies in the characteristic properties of the concrete materials, specifically aggregate sizes, aggregate gradations, water-to-cementitious material ratio, and the construction method (Piggott, 1999). For more in-depth information on RCC for mass concrete, I recommend referring to ACI 207.5R. (ACI, 1999), while ACI 325.10R (ACI, 1995) primarily focuses on RCC's use in pavements. The material properties specific to RCC are discussed below in relation to their direct and indirect effects on longitudinal joint construction and performance.

2.2.1. Impact of Aggregate Selection on Fresh RCC

In RCC production, the choice of an appropriate aggregate hinges on a multitude of factors, encompassing local availability, project budget, RCC mixture volume, and the desired workability, strength, and durability. It is imperative to recognize that aggregates constitute a substantial portion, approximately 75-85%, of RCC. Their inherent properties profoundly influence construction procedures and the overall performance of RCC pavements (ACI, 1995). The visual representation of the disparities in typical appearances and mixture proportions between RCC and PCC matrices is set out in Figure 17.

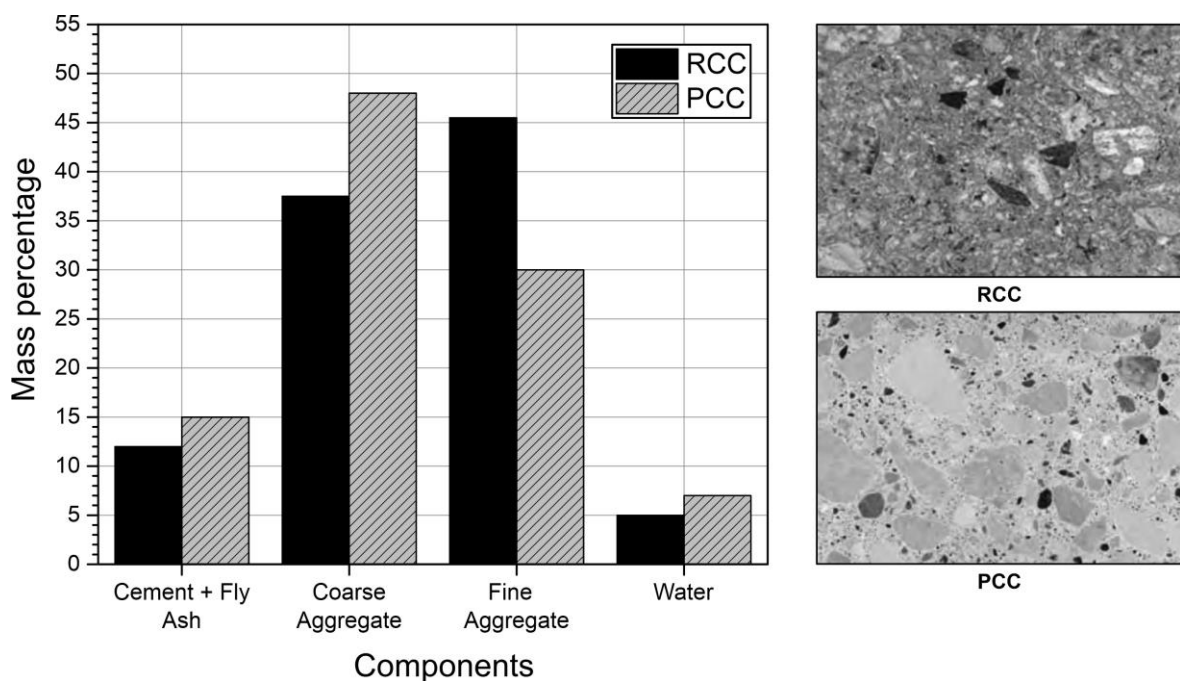


Figure 17. Similarities and disparities of matrices and components of RCC and PCC (adapted from Harrington et al. (2010))

As with any other type of concrete, the aggregates must be free of harmful substances and comply with the primary standard known as ASTM C33 (2018). The usability of aggregates is further assessed based on their performance against the Los Angeles abrasion loss, ASTM C131 (2006), and sulfate soundness loss, ASTM C88 (2018). In RCC, aggregates are classified into two categories: fine (finer than 4.75 mm or No.4 sieve) and coarse (larger than 4.75 mm). The crucial classification criterion is the proportion of sand to gravel (Choi & Groom, 2001), which is directly determined by the percentage passing through the No.4 sieve (4.76 mm), distinguishing between fine and coarse components (ACI,

1995). Because the #4 sieve creates a demarcation line distinguishing the amount of gravel and sand(+silt) for optimal RCC performance, aggregates should exhibit a well-graded characteristic, transitioning from coarse to fine particles, including a portion of fine aggregates. Notably, the significant distinction between RCC and PCC lies in the quantity of fine aggregate employed. Figure 17 visually illustrates this disparity in physical appearances, and the visual disparities in the appearance of the matrixes become even visible to the bare eyes.

In achieving better RCC mixtures, fines should be non-plastic ($PI < 5$), and their nominal diameters should be finer than 0.075 mm, as specified by the No. 200 standard sieve (ACI, 1995). The generally accepted range for fine aggregate in RCC falls between 5% and 10%. The plasticity index of fines is of paramount importance since plastic fines can increase water retention and the water-to-cement (w/c) ratio, potentially leading to a reduced bond between cement paste and coarse aggregates (Choi & Groom, 2001). An alternative suggestion, proposed by Halsted (2009), is to maintain the finer portion of the aggregate between 2% and 8% of the total aggregate by weight. Fine aggregates serve as ideal fillers in RCC, promoting the creation of a densely compacted matrix. This phenomenon leads to an increase in the cement paste content of the matrix, thereby simultaneously reducing the potential for segregation (Choi & Groom, 2001; Harrington et al., 2010; Marchand et al., 1997; Ragan, 1988). However, an overabundance of fine aggregates can pose challenges over time. In the initial service years, fine aggregates may protrude from the pavement surface, giving the RCC texture a resemblance to asphalt concrete surfaces (Harrington et al., 2010; Zollinger, 2016). The recommended minimum gravel content for coarse aggregates should be equal to or greater than 40%, with a suggested lower limit for minimum sand and silt content at 35% (Choi & Groom, 2001). In contrast to conventional concrete, the sand and coarse aggregate proportions in RCC are inverted, with sand comprising 50% to 55% of the total aggregate by weight and coarse materials constituting 45% to 50% (Zollinger, 2016).

Using crushed aggregates in RCC, with specifications similar to those used in standard ready-mix concrete, offers distinct advantages over other aggregate types. In RCCPs, the effectiveness of load transfer at longitudinal joints relies on a combination of aggregate interlock and shear forces generated at the cracked conjugate surfaces. Hence, aggregates with pronounced angularity are favored for achieving superior aggregate interlock, enabling RCC to efficiently distribute the loads within cracks and along the non-separated joint faces

(FHWA, 2019). It is well-established that as aggregate angularity increases, the necessary compaction energy decreases, along with the probability of achieving satisfactory compaction. Consequently, compacting RCC mixtures composed of crushed aggregates proves more challenging than those made with uncrushed aggregates (Harrington et al., 2010). Nevertheless, uncrushed aggregates, particularly those with finer particles, enhance compactability. In the realm of concrete and its aggregates, ASTM C125-21a (2021) provides standard terminology and guidelines for determining the maximum aggregate size, focusing on two closely related definitions. The first and more familiar definition is the "maximum size," denoting the smallest sieve opening through which the entire aggregate mass must pass. The second definition, "nominal maximum size," signifies the minimum sieve opening through which the aggregate is "allowed" to pass. In the context of the nominal maximum size, the term "allowed" or "permitted" introduces a specific perspective. Here, the aggregate does not need to pass entirely through the designated sieve opening, but a predefined maximum proportion of the aggregate can be retained on this sieve.

For maximum size-oriented gradations, an equation, which is mainly known as the "Talbot Equation" and was allegedly developed by Fuller & Thompson (1907), $P = 100(d/D)^{0.45}$, has long been in use for classification of aggregates. Where; P represents the percentage of aggregate passing through the target sieve (by weight), d signifies the size of the aggregate under consideration, and D is for the maximum (known) aggregate size. To that viewpoint, the power value of 0.45 is commonly adopted for achieving optimal particle density (Roberts et al., 1991). This choice aligns with the perspective of those who devised the formula, suggesting that the power factor should approximate 0.5 as closely as possible. Consequently, the resulting gradation curve produced by this method is commonly termed the "0.45 power curve". The essence of this approach revolves around the concept of "preferred" gradation, allowing the attainment of maximum density by facilitating increased particle-to-particle contact throughout the aggregate mass, thereby enhancing overall structural durability. The utilization of the power gradation line, or the maximum density line, enables the calculation of weight percentages corresponding to various particle sizes, culminating in improved aggregate properties. In the simplest sense, when the final aggregate gradation curve becomes steeper, the aggregate is thought to have been graded more uniformly than before (Bian et al., 2016). Namely, a 0.45-power gradation curve can facilitate the modification of the entire aggregate mix to achieve a well-graded nature.

Regarding the allowable maximum nominal size, the initial RCC mixtures designed for use in pavement construction in the United States featured a maximum aggregate size of 38.1mm (1.5 inches). Subsequently, the suitable maximum nominal size was reduced to 19mm ($\frac{3}{4}$ inch) (Pittman, 1988). It is crucial to note that not all RCC mixtures are the same. Despite some exceptions, a noticeable distinction exists in the maximum aggregate size utilized in RCC mixtures for pavement applications versus those intended for gravity dams. Aggregates for RCC mixtures used in dam construction should have even a diameter of less than 76mm to ensure proper spreading and compaction without hindrance.

Several early recommended aggregate gradation curves have long served as valuable references for identifying the most appropriate aggregate proportions for use in RCC production. These curves encompass both upper and lower limits for aggregate gradation. A highly effective approach for determining the optimal aggregate proportioning in the pursuit of enhanced RCC production involves the use of the s/a ratio. The recommended gradation limits give rise to distinct band-like regions, primarily characterized by this s/a ratio. Specifically, an s/a ratio of 30% predominantly represents the lower-limit gradation, while an s/a ratio of 70% signifies the upper-limit gradation. The term "within-limit gradation" is employed when the s/a ratio stands at 50%. Various aggregate gradation limits proposed by select authoritative bodies, alongside their recomputed ideal gradation patterns based on the 0.45-power approach, are presented in Figure 18. Concurrently, Figure 19 delineates the s/a percentages implicitly specified by each of the gradation bands outlined in Figure 18.

Indeed, numerous grading curves can be drawn, most of which fall within the suggested gradation bands. Song & Lee (2015) emphasized the importance of aggregates aligning with the recommended bands, although they acknowledged that these recommendations may not always suffice, particularly in terms of achieving high strength. Hence, they proposed the utilization of 0.45-power curves, which is in line with the recommendations of Zollinger (2016). On the one hand, Chhorn & Lee (2018) showed that aggregate gradations adhering to the recommendations of the Portland Cement Association, PCA (2004), are adequate for producing high-strength RCC, as depicted in Figures 18 and 19. On the one hand, Chhorn & Lee (2018) also observed that even when the gradation of an RCC mixture deviates from the recommended band, such as $s/a = 50\%$, it is still possible to achieve high-strength RCC. Furthermore, Song & Lee (2015) pointed out that certain fine particles, capable of passing through the #60 standard sieve with a 0.25 mm opening, do not

necessarily need to conform strictly to the recommended bands. This observation is notably substantiated by the research of Rao et al. (2014).

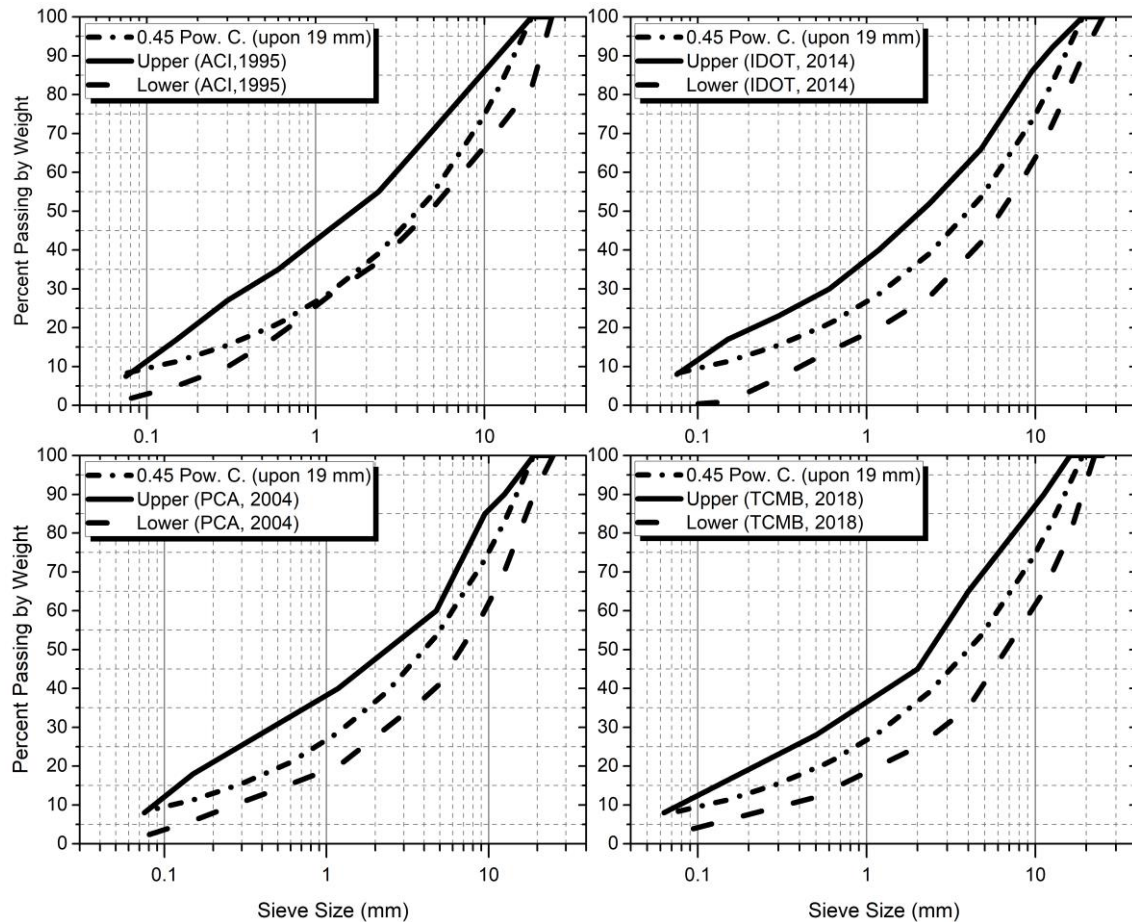


Figure 18. Previously recommended aggregate gradation limits for use in RCC pavements by reference institutions

In sum, the adequacy of the recommended gradation bands has long been a subject of scrutiny. From this point onwards, the recommended gradation bands should be thought of as nothing but some beneficial recommendations. As such, it should not be surprising if these bands undergo modifications and refinements over time. Hashemi et al. (2018) conducted an extensive study to ascertain the impact of the coarse-to-fine (C/F) aggregate ratio on the characteristic properties of RCC in the presence of varying cement content. This investigation thoroughly examined different RCC mixtures at both their fresh and hardened states. Their research findings revealed that an increase in the C/F ratio led to a substantial tripling of the Vebe time, which measures the workability of the concrete. Furthermore, they observed that as the cement content was heightened, the Vebe time decreased.

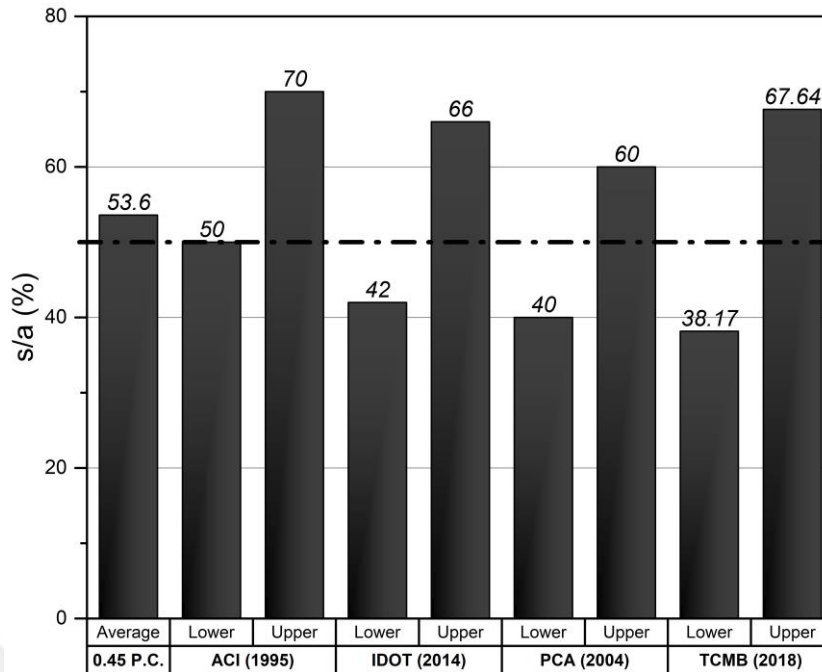


Figure 19. Variation in sand/aggregate (s/a) ratio among suitable aggregate gradations recommended by reference institutions

Additionally, an escalation in the C/F ratio was associated with a marked reduction in porosity. The study identified a practical range for the C/F ratio, falling between 1.2 to 1.4. In light of these results, the authors recommended a mixture comprising 12% cement and a C/F ratio of 1.2 as a favorable composition.

2.2.2. Influence of Cement Choice on Fresh RCC

In the context of cement selection for Portland cement concrete, it is a well-established norm that any cement to be employed must adhere to the standards outlined in ASTM C150 (2019). There are five distinct types of cement available for various applications. Type I cement, known as ordinary Portland cement, is commonly used in general engineering applications as long as there is no sulfate hazard. It does not pose the risk of excessive heat generation during hydration. Type II cement, often referred to as modified general-purpose Portland cement, is chosen when working with mass concrete in hot weather conditions. It exhibits a significantly lower heat production capacity compared to Type I cement. Type III cement is recognized as high-early-strength Portland cement. This type finds its utility in scenarios where cold weather might impede hydration. However, it tends to have a slightly higher shrinkage rate compared to Type I and II. Type IV cement, known as low-heat

cement, is particularly beneficial for massive concrete applications, such as concrete gravity dams. Its strength development pattern is slower than those of other cement types. Type V cement is sulfate-resistant and is the preferred choice when there is a possibility of sulfate attack, especially in alkaline soils. As with conventional concrete, The United States Army Corps of Engineers (USACE, 2000) provided a useful guideline, as depicted in Figure 20, for selecting the appropriate cement content in RCC mixtures based on the available curing time and target compressive strength.

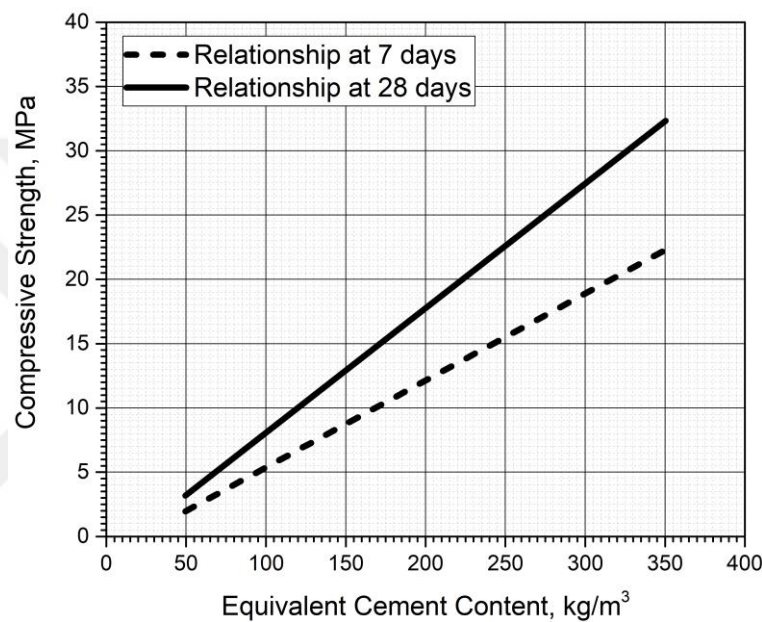


Figure 20. USACE (2000) procedure for cement content selection for an RCC mixture (reproduced from Amer et al. (2004))

Type I, II, and IV cement can be used to produce RCC mixtures (Ragan, 1988). However, the traditional approach leans toward using Type I or Type II cement due to its limited heat generation potential (Choi & Hansen, 2005). Besides, quite predictably, Type III cement is generally not practical in RCCP constructions since it directly extends the allowable time for the fresh mixture's workability and further decreases the heat of hydration at the initial stages of hardening (Aghaeipour & Madhkhan, 2019). The rate of strength development among various cement types varies significantly with age. Initially, lower-heat cement tends to exhibit slower strength development compared to ordinary cement (Andriolo, 2003). Due to its lower cement content, RCC offers distinct advantages, especially in mass concrete applications. Therefore, it minimizes the possible setbacks on account of the heat of hydration. To put it another way, if less cement is used in concrete

production, less heat of hydration is generated. Chhorn & Lee (2018) suggested that RCC mixtures should ideally contain around 12% cement content or slightly higher values. They observed that a cement content of approximately 12% helps regulate the influence of water content on RCC's strength. In essence, higher cement content leads to less variation in water content within the RCC mixtures.

2.2.3. Effects of Alternative Cementitious Materials on Fresh RCC

In RCC production, a substantial amount of paste content is inherently required. The fine materials in the mixture become quite crucial to occupy the voids between larger aggregates. Essentially, there are two approaches to achieving an adequate paste content: increasing the cement content or incorporating pozzolans into the mix as a substitute material. Much like conventional concrete, the primary cementitious materials in RCC are derived from Portland cement. Nevertheless, a range of alternative materials can replace cement, including fly ash, ground granulated blast furnace slag, limestone dust, silica fume, cement kiln dust, and various natural pozzolans such as volcanic ash and tuffs. Among these alternatives, fly ash, particularly Class F fly ash (Choi & Groom, 2001), is one of the most widely used cementitious materials in RCC production, and its use must meet the requirements outlined in ASTM C618-19 (2019).

Using alternative cementitious materials has garnered considerable attention among engineers, primarily to reduce the cement content required for various concrete mixtures. By incorporating cement-substitution materials, several advantages can be realized, particularly with respect to cost-effectiveness in pavement construction and enhanced durability, which directly correlates with reduced maintenance costs for pavements. Additionally, these cement-substitution materials offer environmentally friendly benefits by contributing to reduced greenhouse gas emissions.

Common sense dictates that the total amount of cementitious materials ordinarily employed in RCC is considered to be about 12-15% of the overall mixture. Several researchers have highlighted the use of fly ash in RCC production, noting that it is the most potent substitute material. Notably, the inclusion of fly ash has a positive impact on concrete by reducing water ingress and the heat generated during hydration, consequently improving durability and reducing the likelihood of crack formation (Zarrinkafsh & Shirazi, 2015). However, optimizing the quantity of fly ash used in RCC mixtures is of utmost importance

since an excessive amount may result in a slight reduction in compressive, splitting tensile, and flexural strength, as Pavan & Rao (2014) noted. Of a few pioneer works, Ragan (1988) suggested that the practical range of fly ash replacement should be set between 20 and 40% of the total weight of all cementitious material. Naik et al. (1995) recommended that the range for cement substitution with Class C Fly ash should fall between 20% and 50% of the total weight of all cementitious materials. In the case of Class F Fly ash, the substitution range can extend up to 40%. Choi & Groom (2001) revised these recommendations, modestly extending the limits to 25% to 60% of the total cementitious contents. While research on alternative cement substitution materials in RCC has been relatively scarce compared to the extensive focus on fly ash, noteworthy exceptions exist. Shamsaei et al. (2019) conducted an evaluation of the impact of incorporating ceramic and coal waste powders as cement substitutes in RCC. Their findings revealed that the introduction of ceramic and coal waste powders resulted in reductions in both the total unit weight and the Vebe time. Among the various substitution levels tested, a 5% substitution of ceramic waste powder by weight of the entire cementitious material emerged as the optimal choice. Furthermore, the use of ground granulated blast furnace slag (GGBFS) in RCC for pavement construction was investigated by Aghaeipour & Madhkhan (2020). They determined that the ideal substitution level, which maximizes mechanical properties, is up to 40% by weight of the entire cementitious material in the mixture.

2.2.4. Impacts of Chemical Admixtures on Fresh RCC

In the production of fresh RCC, maintaining a balanced interplay between compactability and extended workability is of utmost importance. Achieving prolonged workability is challenging, primarily due to potential transportation delays that can curtail the available compaction time. Therefore, the use of chemical admixtures becomes essential in certain situations. Compared to conventional concrete, chemical admixtures are not as commonly employed in RCC. ASTM C494 (2017) categorizes chemical admixtures for concrete production into several classes, including water-reducing, retarding, accelerating, high-range, and specific-performance admixtures. Retarding additives can be incorporated into RCC production to extend the workability time necessary for effective rolling operations. Moreover, Chhorn & Lee (2018) noted that when air-entrained agents and superplasticizers, such as Polynaphthalene sulfonate, are used in RCC mixtures in

appropriate proportions, these admixtures significantly prolong the workability period without causing a substantial loss in compressive strength.

2.2.5. Mixing Water Requirements for Fresh RCC

Similar to any concrete variant, the quality of water used in RCC adheres to the standards outlined in ASTM C1602 (2018). This standard specification for mixing water aligns with the requirements for water used in conventional concrete. It is a common practice to stipulate that the mixing water quality for RCC should be potable.

2.2.6. Effects on Workability Attributes of Fresh RCC

Workability is always one of the most sought-after quality benchmarks for concrete production. Attaining the ideal workability is an integral aspect of concrete production, and RCC is no exception to this need. Notably, RCC's workability requirements differ significantly from those of ordinary concrete, making it a critical consideration for successful construction. Chhorn & Lee (2018) emphasized the significance of moisture control in fresh RCC as a crucial step to prevent strength loss in hardened RCC. Sufficient mixing is necessary to uniformly scatter the water throughout the matrix to produce high-end RCC, albeit it is preferably used in a minimal amount (Pittman, 1986). Neville (2011) signified that once the concrete is compacted, in all respects, the effect of w/c on the strength of concrete becomes inversely proportional to strength, mostly known as Abrams's rule. Figure 21 illustrates the effects of optimum moisture content on concrete strength and w/c ratio, also highlighting the positive impact of vibration on compressive strength, albeit within a narrower range of w/c ratios compared to other compression methods.

Chhorn et al. (2019) signified that achieving workability in RCC is more challenging than attaining the desired dry unit density due to differences in the required water content. Zollinger (2016) suggested that the w/c ratio for workable RCC should typically fall within the range of 0.36 to 0.45, although lower values are generally more desirable. A simplistic and handy diagnostic test called “slump test or slump cone test” is routinely performed in accordance with ASTM C143 (2015) or AASHTO T 119-18 (2018) using a well-known cone-shaped test apparatus. It is commonly used to portray the ultimate slump characteristics of the mixture. Besides, it helps quantify fresh concrete's workability in terms of whether it

is acceptable to requirements. Namely, from a simplistic perspective, it is an auxiliary method for determining concrete's w/c ratio. However, RCC is notably a zero-slump concrete, meaning it must be stiffer and drier than conventional concrete.

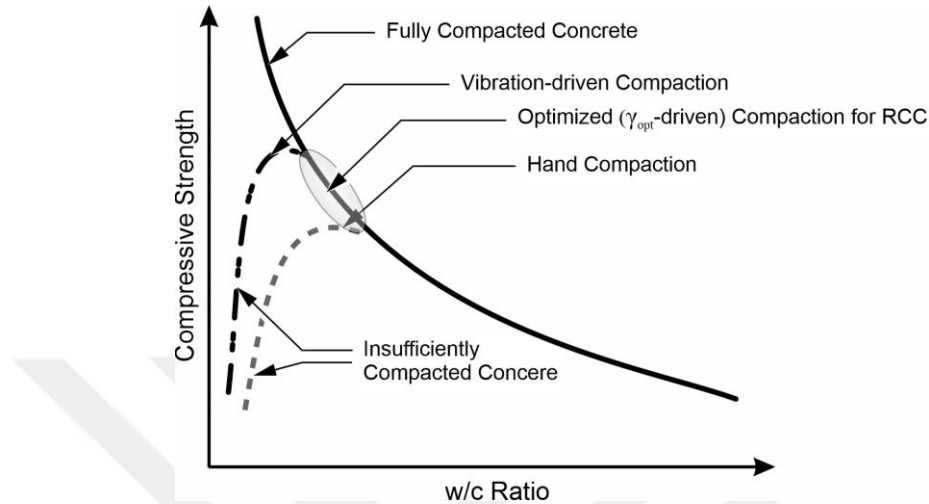


Figure 21. Relationship between w/c and compressive strength (adapted from Neville (2011))

Consistency is a paramount quality benchmark for RCC, especially when it is used in RCC pavements. Fresh RCC should be wet enough to allow mixing without any segregation and stiff enough to remain relatively stable when the vibratory roller is in operation. Workability directly influences the quality of compaction. Poorly compacted joints can markedly render the pavement susceptible to various distresses, such as premature cracking, raveling, and softening of the underlying granular base layers, as observed in HMA pavements. Since RCC innately represents nearly zero slump nature and is far stiffer than conventional concrete, the use of the slump test alone is not enough to express the full portrait of RCC's workability. Therefore, finding an alternative method to describe the workability of fresh RCC is crucial. With this in mind, the Vebe test, governed by ASTM C1170 (2014), emerges as a suitable alternative. The Vebe test, notably superior to the slump cone test due to the stiffness of the RCC mixture, is a highly versatile method for determining the behavior observed during the compaction process. Both tests share a similar underlying philosophy, albeit from different mechanical perspectives. Chhorn et al. (2019) note that the Vebe time increases with higher s/a ratios and a rise in water content. The standard and desired Vebe time for fresh RCC typically falls between 10 and 45 seconds, but in practice, it often falls within the range of 10 to 15 seconds (Choi & Hansen, 2005). Sengun (2019)

demonstrates that RCC with good and sufficient compactability for pavement construction can be achieved with 5-6% water and a Vebe time of 30 ± 10 seconds. Chhorn et al. (2019) noted that the Vebe time increases with higher s/a ratios and a rise in water content. The standard and desired Vebe time for fresh RCC typically falls between 10 and 45 seconds, but in practice, it often falls within the range of 10 to 15 seconds (Choi & Hansen, 2005). Sengun (2019) showed that RCC with good and sufficient compactability for pavement construction can be achieved with 5-6% water and a Vebe time of 30 ± 10 seconds.

2.2.7. Impact of Density and Compaction Degree on RCC

The bulk density or unit weight of concrete, commonly measured according to ASTM C138 (2017), is defined as the overall space occupied by the mass of concrete, consisting of solid particles and water (ρ_{bulk} =total mass of concrete/bulk volume occupied by concrete). The theoretical maximum constituent density (*TMCD*), calculated as percentages of each component used in the mix, including specific gravity, can be used to determine the density of RCC with the hypothesis that it is possible to remove all air voids in the mixture. The following expression (1) is commonly used to calculate the *TMCD*.

$$TMCD = \frac{P_c + P_f + P_{ca} + P_{fa} + P_w}{\frac{P_c}{D_{rc}} + \frac{P_f}{D_{rf}} + \frac{P_{ca}}{D_{rca}} + \frac{P_{fa}}{D_{rfa}} + \frac{P_w}{D_{rw}}} \quad (1)$$

In the given context, P denotes the percentage of the total weight of each material used in the mixture, and the subscripts are as follows. c is for cement, f is for fly ash, ca is for coarse aggregate, fa is for fine aggregate, and w is for water. Besides, the maximum achievable unit weight (γ) is then calculated using *TMCD* and the unit weight of water (γ_w) as per the following equation (2)

$$\gamma = TMCD(\gamma_w) \quad (2)$$

As is known, compaction is a contractive process that results in an increase in the density of the compacted material in the company with a decrease in air volume. The degree of compaction is determined using the dry unit weight of the material, and it is influenced by factors such as water content and the compaction effort applied (e.g., the weight of the

roller, frequency of vibrations, and the number of passes). From a soil mechanics perspective, when applying proper compaction effort, RCC's ideal maximum dry unit weight is achieved at an optimum water content. Besides, the concrete's time-dependent hardening is thought of as a possible reason for poor compaction achieved. In RCCPs, the initial position of the roller has some influence on the variation in material density around the joint zones.

Density is particularly critical for the durability of longitudinal joints, as it directly impacts the longevity and overall performance of pavement structures over time (Kandhal & Mallick, 1997c; Kandhal & Rao, 1994). In HMA pavements, inadequate compaction is attributed to the rapid cooling of the hot mix, leading to premature asphalt oxidation. To the best of my knowledge, Foster et al. (1964) were likely the first study to demonstrate that the density of the paved material decreases when approaching the longitudinal joint in HMA pavements. Consequently, as you move from the joint towards the free edge of the mat, the density progressively decreases until it stabilizes at a certain distance from the joint. Figure 22 illustrates the typical density gradient variation across the longitudinal joint of the pavement.

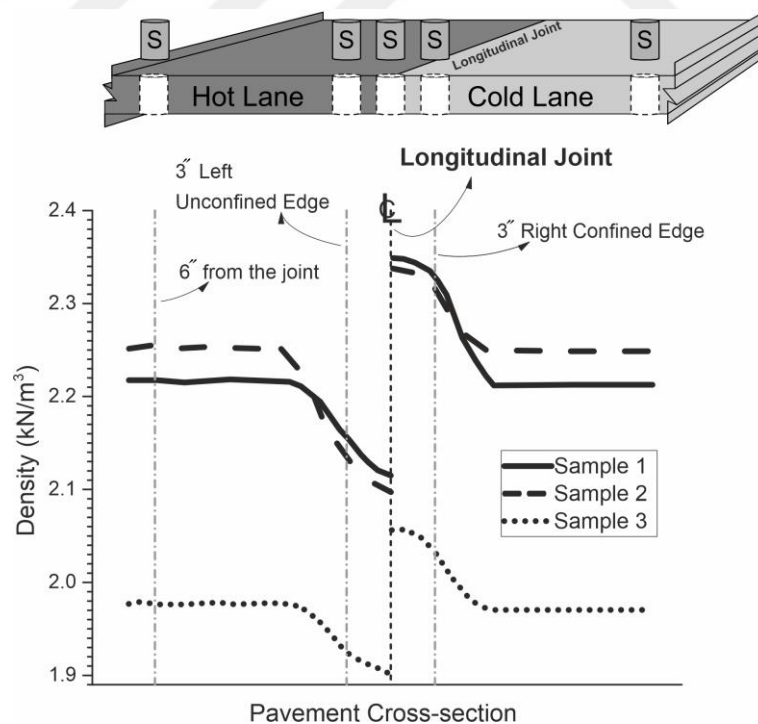


Figure 22. Typical density gradient across a longitudinal joint of a HMA pavement (not to scale, adapted and reproduced from Foster et al. (1964))

Several state departments of transportation in the United States established quality benchmarks for the density gradients of the pavement joints in HMA pavements. These standards are available from different sources (McDaniel et al., 2012; Williams, 2011). According to Figure 23, the minimum allowable relative joint density should be 87% of the main lane, which is off from the joint zone when these standards are compiled and plotted. In other words, a specific percentage of the theoretical maximum specific gravity should be achieved for the pavement joints to be accepted for quality. Missouri has one of the highest standards for the quality benchmark of longitudinal joint acceptance among all the states, as shown in Figure 23.

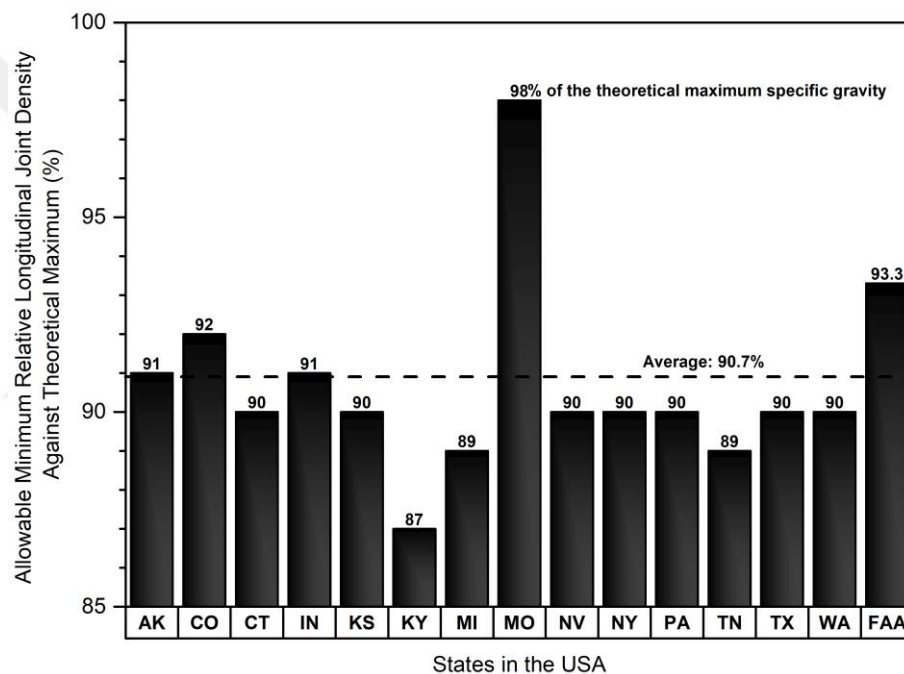


Figure 23. Density specification limits for longitudinal joints in HMA pavements across various state DOTs in the USA

From the compaction perspective, the presence of minimal air voids within the matrix indicates the achievement of high-density RCC. In this context, the significance of sand and cement paste, both of which have the potential to fill any gaps that may exist within the overall matrix, becomes more pronounced. If the aggregate gradation used in RCC production is poorly graded, often referred to as gap-graded, the overall material density tends to be lower compared to that of RCC produced with well-graded aggregates. Much as it can theoretically be made in a much more densely-packed form, the typical density range of RCC typically falls between 23.2 and 24.3 kN/m³ (Choi & Hansen, 2005; Harrington et

al., 2010). Schrader (1992) provided an impeccably explanatory and crucially important plot based on experimental data that illustrates how much of the strength at maximum density can be achieved in relation to the percentage of the obtained air-free density of RCC. I initially utilized this plot to conduct further data analysis. Through digitization, I developed two prediction models (a simple and a complex one) to further estimate the strength parameter at the maximum density based on the air-free density of an RCC sample. The modified figure, displaying the reprocessed data, is presented in Figure 24. This new plot can significantly facilitate the current study and future research efforts by offering an early means to assess and correlate the strength and air-free density of laboratory samples.

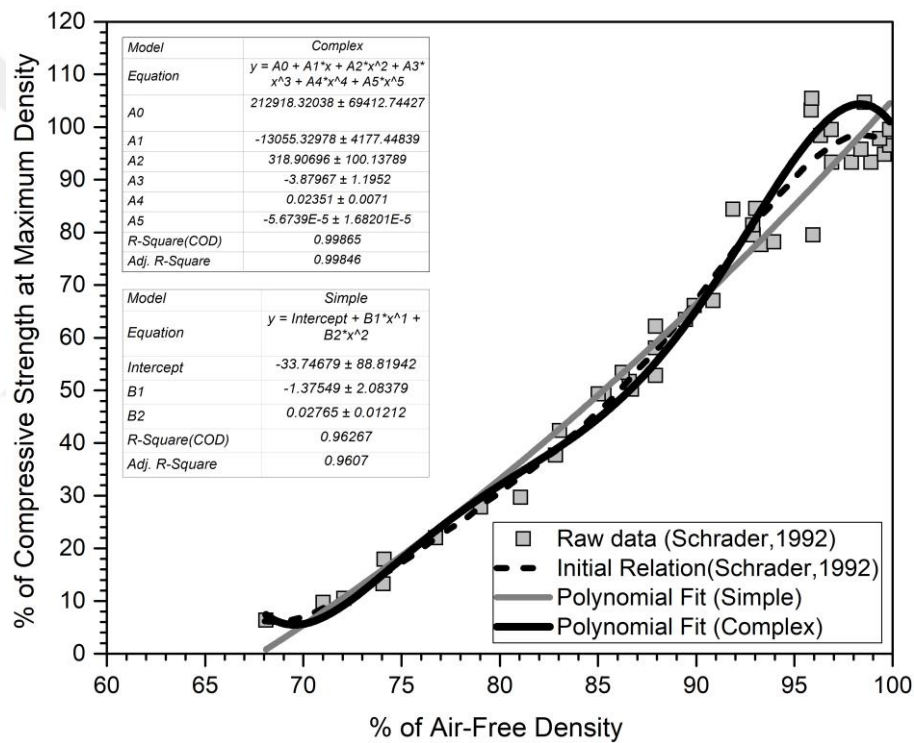


Figure 24. Relationship between percent air-free density and achievable strength at maximum density in RCC mixtures (Original data from Schrader (1992), reprocessed for this study to find new relations)

According to the data presented in Figure 24, the compressive strength of RCC shows only slight variation when compacted densities fall within the range of 97 to 100 percent of the theoretical air-free density (*TAFD*). Once the density falls below 96 percent, a significant loss in strength becomes noticeable. Schrader (1992) observed that this is why most specifications require a minimum density of at least 98 percent of the maximum density, often determined using a modified Proctor test. Incidentally, it is worth noting as a

cautionary point that the density determined through the modified Proctor test is not the same as the theoretical air-free density since there are typically some voids in the mixture compacted by the Proctor hammer, typically ranging from 0.5 to 2 percent.

As is known, the compaction degree of the HMA samples taken from the longitudinal joint has thus far been used as a quality indicator of the compaction achieved, which is also a benchmark for the quality of longitudinal joints (Fleckenstein et al., 2002; Kandhal & Mallick, 1996, 2007; Kandhal & Rao, 1994). Regarding RCC, Chhorn et al. (2019) noted that maximum dry density decreases as the s/a ratio increases. Furthermore, well-graded aggregates can yield maximum dry density while maintaining workability. LaHucik & Roesler (2017) demonstrated that achieving the initially targeted laboratory density of RCC in the field condition is a critical factor for attaining the structural design capacity of RCCPs and sustaining it in the long run. They also noted that even a slight difference in the measured densities of the samples cast in the laboratory and in the field could result in up to a 45% difference in compressive strength. In this instance, the potential incongruity between the laboratory and field RCC properties can be reduced by using high-density pavers. In support of this, Ragan (1986) pointed out long ago that high-quality compaction is essential to the extent that it makes RCC more durable against potential freeze-thaw effects without the need for any air entrainment.

2.2.8. Impacts of Porosity, Permeability, and Sorptivity Features on RCC

Porosity and permeability are commonly used terms to evaluate whether the desired physical criteria for the quality of longitudinal joints in pavements are met. Porosity is an intrinsic property of a material, measuring the volume of voids (air and water) as a percentage of the total volume of the concrete mass. Permeability or hydraulic conductivity (k , [m/s]), on the other hand, measures how well water can pass through a porous medium and is closely related to the porosity feature. It also assesses the interconnection of these porous spaces within the matrix.

In pavement engineering, permeability is a more advanced concept compared to porosity when assessing the quality of longitudinal joints. Further, given the guiding knowledge gained from HMA pavements, permeability gradients in and around a longitudinal joint are considered a metric for overall construction quality. A less permeable joint ideally prevents water and foreign material intrusion into the pavement, reducing the

risk of premature distress (Mallick & Daniel, 2006). From this point on, the permeability has been utilized as a quality benchmark for assessing the soundness of longitudinal joints in HMA pavements, as indicated in several studies (Mallick et al., 2001; Mallick & Daniel, 2006). The Sorptivity test, governed by ASTM C1585 (2020), also known as the capillary absorption test, is used to assess a material's resistance to harsh environmental conditions that lead to distress. Unlike the permeability test, which measures water flow through a saturated porous medium under pressure, sorptivity evaluates a material's innate ability to absorb and transmit water through capillary suction.

Several studies have examined RCC's permeability performance. Based on laboratory test results, Banthia et al. (1992) examined the permeability performance of RCC, noting that the permeability of RCC (with a range of $k = 2 \times 10^{-12}$ to 72×10^{-12}) lags behind that of conventional mass concrete to some extent. Shafigh et al. (2019) showed that a 1% increase in RCC porosity led to a significant decrease in compressive strength (~ 8 MPa). Hashemi et al. (2018) shared the results of a case study that primarily used low fines content sand, resulting in more porous RCC pavement. They strongly recommended using sand without ultrafine particles in RCC mixtures for RCCPs. While RCC's permeability characteristics can vary depending on factors such as cement content and aggregate type, it often exhibits notable anisotropic permeability, meaning that permeability is slightly higher when water flow aligns with the direction of compaction. Notably, RCC's permeability is significantly higher in specific areas very close to the longitudinal joint compared to locations farther from the joints. This difference is primarily attributed to segregation at the bottom of the pavement (Choi & Hansen, 2005). However, laboratory experiments to determine RCC permeability are often considered expensive and time-consuming. Heidarneshad et al. (2021) used an artificial neural network (ANN) to establish a relationship for estimating RCC's permeability coefficient based on some simple physical parameters. They noted that the ANN approach allows for more efficient decision-making, enabling the identification of whether the RCC mixture will have acceptable permeability without the need for lengthy testing.

2.2.9. Influence of Air Void and Air Spacing Factor on RCC

Air voids form a random pattern within the fresh RCC matrix, much like in any composite material. These voids represent the percentage of empty spaces within the

aggregates and cement paste not occupied by any binder. The air void percentage is calculated by measuring the volume of air voids in the sample as a percentage of the sample's total volume. These voids can facilitate the compaction of granular mixtures when they are present in the right amount. In the absence of proper preparation and design, the air void percentage may occur coincidentally. However, it is generally desirable to manage the presence of air voids to the extent possible. Purposefully entrained air voids can enhance concrete's resilience against various distresses, such as freeze-thaw cycles, and can also improve workability. Nevertheless, it's a well-known fact that excessive voids can reduce concrete strength and weaken the matrix (ACI, 2016). According to the American Concrete Institute (ACI, 2000), efficient compaction can reduce one percent of the total air in the concrete matrix. Failure to achieve proper compaction could likely result in an approximately 5% decrease in compressive strength. In a supportive sense, Rollings (1988b) noted that when proper compaction is not achieved, particularly at the bottom of an RCCP, it tends to exhibit an overall strength that is 25% weaker than the rest of the pavement. Given that the bottom part of the RCC mat constitutes the most critical part of the pavement structure regarding proneness to tensile stresses over the years, ensuring efficient compaction from top to bottom becomes a crucial requirement.

Comparing the air void content in the material within the longitudinal joint and that away from the joint serves as a quality benchmark for evaluating longitudinal joints in HMA pavements (Fleckenstein et al., 2002; Kandhal & Mallick, 1996, 2007; Kandhal & Rao, 1994). To the best of my knowledge, this way of thinking is also relevant for assessing longitudinal joints in RCCPs, given the crucial role of air void content in the resilience of RCCPs.

2.2.10. Effects of Creep and Drying Shrinkage on RCC

The “creep” and “shrinkage” of concrete are commonly pronounced terms in concrete science. Although they involve different processes and causes, they are often addressed together. Creep, for example, is a time-dependent deformation that occurs in concrete due to continuous (prolonged) loading, and it is strongly influenced by factors like material properties and exposure time to the loading. On the other hand, drying shrinkage is caused by the contraction of capillary water retained within hardened concrete, resulting in physical contraction, which often leads to the initiation of cracks on the concrete surface (Rüsch et

al., 1983). In RCCP design and construction, more attention is given to drying shrinkage than creep. This is because, with similar or even lower cement content, RCC exhibits either equal or reduced drying shrinkage potential and volume change characteristics compared to conventional concrete (Pittman & Ragan, 1998; Tayabji et al., 1995).

2.3. Significant Factors on RCCP Longitudinal Joint Construction

In this section, the topics related to the construction of RCCPs and their longitudinal joints are discussed one after the other, creating a new compilation of purpose-specific literature.

2.3.1. Impact of Compaction Techniques on RCCPs

The base course of RCCPs, with technical requirements nearly identical to those of conventional concrete pavements, should exhibit sufficient stability to accommodate the fresh RCC mixture and facilitate effective compaction with rollers (Harrington et al., 2010; Piggott, 1986). Chhorn & Lee (2018) proposed that achieving a minimum of 98% compaction based on the standard modified Proctor test is a practical prerequisite for realizing RCC's maximum potential strength. This highlights the importance of attaining 98% of the maximum dry density of the RCC mixture during the compaction process to achieve the desired material properties. Furthermore, Nanni et al. (1996) emphasized that factors such as paver speed, rolling techniques, and moving (rolling) patterns significantly impact the load transfer efficiency. Additionally, the type of joint plays a crucial role in the long-term performance of RCCPs. More so, the type of joint significantly influences pavement performance, whether it is a fresh joint or not, and these effects become increasingly evident over time.

Piggott (1999) pointed out that the most severe deterioration is often observed in and around the longitudinal joints of RCCPs, particularly in the case of cold-formed joints. Cold joints, which form between adjacent lanes, one of which is cast the night before the other, are particularly vulnerable to distress initiation. After experiencing distress initiation, cold joints become much more susceptible to cracking than fresh joints. It is a flashing indication that if the density requirement and concrete-to-concrete bonding are not genuinely met in the longitudinal joint zone, the early cracking initiations become likely thereabouts. That is

to say, it could easily be claimed that there has been an apparent tie between the density gradient in the RCC longitudinal joint and the behavior of the total strength.

2.3.2. Influence of Lift Thickness on RCCP Performance

The lift thickness of an RCC layer is another crucial factor that significantly impacts the compaction of the fresh mixture. It is a known fact that lift thickness has a crucial effect on permeability and the time available for compaction of HMA (Hainin et al., 2013). The recommended lift thicknesses in RCCPs can vary among different studies, but a commonly adopted thickness for compacted RCC is around 200–250 mm (9–10 inches) (ACI, 1995; Harrington et al., 2010). The ERMCO (2013) guide specifies that the RCC thickness tolerance should be within ± 15 mm of the pavement's design height to avoid water ponding. If the required RCC thickness exceeds the limit of 250–300 mm, the layer can then be divided into multiple layers; namely, it is laid in a two-layer form (Choi & Hansen, 2005; Zarrin & Shirazi, 2015).

2.3.3. Impacts of Pavers on Compaction Performance in RCCPs

As commonly used in HMA pavements, pavers (screeds or free-floating screeds or laydown machines) play a crucial role in RCCP construction by providing an initial high level of density, typically in the range of 90–96 percent of the total density. Subsequently, vibratory rollers weighing 10–12 tons are employed to compact the fresh RCC layer. For the final rolling, relatively mid-sized rollers weighing 3 to 6 tons are used, which can be either dual-drum steel rollers or rubber-tired rollers (Zollinger, 2016). As with HMA pavements, high-density pavers are used to provide nearly all of the initial compaction in RCCPs. Delatte (2014) controversially suggested that little or no further compaction is necessary once the screed lays down the material. According to Delatte (2014), the shaping of the longitudinal joint is almost complete immediately after the screed has passed over the RCC mix. This perspective has sparked debates and discussions in the field. The great majority, myself included, believes that the density of RCC continues to increase to a certain degree during rolling, leading to improved density and strength. Therefore, the practice of rolling is considered valuable according to conventional approaches.

2.3.4. Impact of Equipment on Compaction Performance in RCC Pavements

Pavement compaction is typically achieved using three main types of rollers: static, vibratory, and oscillatory, with the option to use them individually or in combination. The compaction of HMA pavements was historically accomplished using steel-wheel static rollers, a practice that persisted until the late 1950s when vibratory rollers gained prevalence. Subsequently, advancements in technology led to the emergence of significantly heavier static rollers, which posed challenges as they caused aggregates and even pavements to crack. The introduction of oscillation technology in Germany in 1983 marked a significant development, eventually leading to its integration into rollers. By early 2003, oscillatory rollers were employed in asphalt pavements in the United States. In addition to vibratory models, these rollers utilize a mechanical system involving dual, opposed eccentric weights spinning around the same direction as the drum's axis to generate a rocking motion as they compact the layer (Edward, 2006). Particularly, BOMAG introduced an intriguing roller known as the Vario Roller, featuring a cleverly designed internal mechanical system that can rotate the axis of eccentric masses. This innovation enables the generation of vibrations in any direction, whether along vertical or horizontal axes. This unique capability empowers Vario Rollers to produce vibrations at angles previously unattainable (Caicedo, 2019). For a more comprehensive understanding, the operational principles of various roller compactor types are outlined in Figure 25.

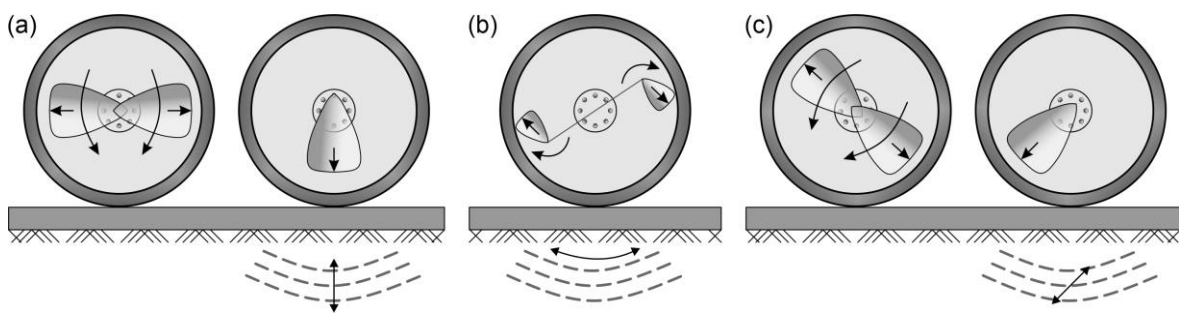


Figure 25. Different operating mechanisms of roller compactors: (a) Vibratory roller, (b) Oscillatory roller, (c) Vario- Roller (Caicedo, 2019)

The compaction performance of vibratory rollers is significantly impacted by their main features, including operation pace, frequency, and vibration amplitude. Figure 26 illustrates the characteristics of different amplitudes and frequencies produced by the roller compactors throughout compaction history. In accordance with this, using high-amplitude

and low-frequency rollers is recommended for less workable and somewhat thicker RCC lifts. Conversely, low-amplitude and high-frequency rollers are recommended for much more workable RCC mixtures, as suggested by ACI (2000).

Moreover, the number of drums on the vibrating rollers directly affects the required number of passes to achieve the desired compaction of RCC. Making four to eight passes with double-drum (dual-drum, tandem drum) rollers is generally considered sufficient for easily workable RCC mixtures. The counting of passes is as follows: if a trip is performed from point A to point B with a dual-drum roller, it is counted as a full trip. However, if a trip is made from point A to point B and then returning to point A with a single drum roller, it is assumed to be a full trip.

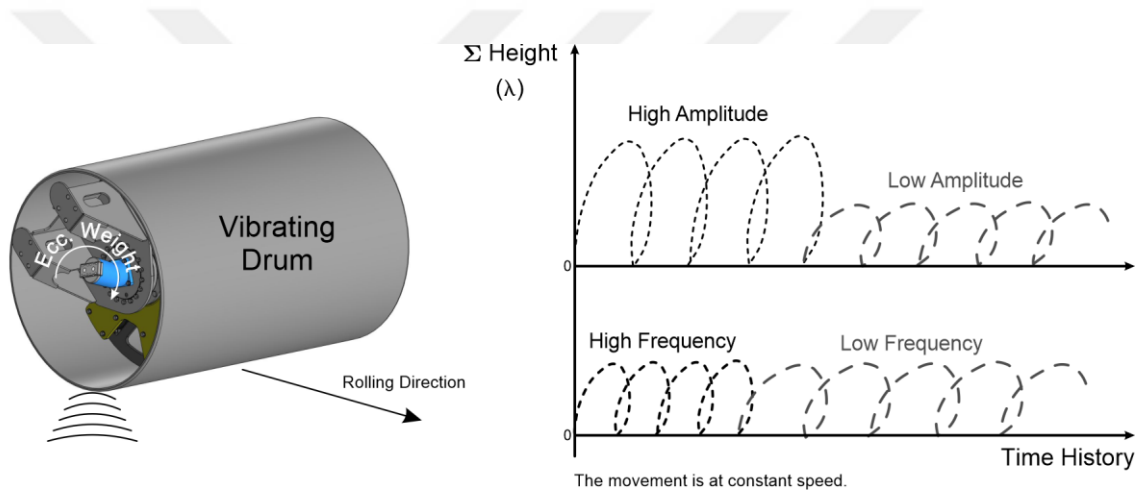


Figure 26. Typical amplitude and frequency patterns of the vibrating drums

Regarding the roller's speed, it is essential to note that the maximum allowable roller speed for effective compaction is approximately 3.2 km/h. If the roller is used at a significantly higher speed, increasing the number of passes is necessary to achieve the required uniform compaction (Withrow, 1988). When determining the static load of a vibrating roller, Withrow (1988) recommended dividing the compacted lifts' thickness into two sections. For lifts up to the maximum thickness of 150 mm, a static load of 20 kg/cm was suggested, while for thicker lifts exceeding 150 mm, a minimum static load of 27 kg/cm was recommended.

On the one hand, the ratio of frame-to-drum mass significantly affects compaction performance. Therefore, the upper limit for the frame-to-drum weight ratio should be carefully considered for improved design, equipment, and operation. The drum diameter,

which directly correlates with the amount of static load, should be specified before compaction. Whether a tandem or single-drum roller, any vibratory drum must be motor-driven to ensure sufficient drum traction (ACI, 2000). Subsequently, rubber-tired rollers, typically weighing 10 to 20 tonnes, are employed following the vibratory rollers. They are commonly used to smooth out minor imperfections, such as slight marks and minimal cracks, which tend to appear perpendicular to the direction of the vibratory rollers' movement (ACI, 2000). Soil stiffness, roller speed, excitation frequency, and the ratio between roller and drum mass are all significant factors that influence the interaction between the drum and the soil. These factors dictate the typical modes and operations of the system, as depicted in Figure 27. The figure illustrates the variation in amplitude and frequency of the drum during compaction, emphasizing the impact of these factors on the interaction between the drum and the soil, ultimately affecting the resulting density ratio.

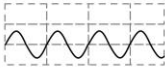
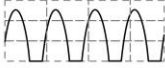
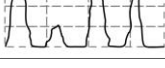
Drum Motion	Drum-Soil Interaction	Operating Condition	Soil Contact Force	Application of CCC	Soil Stiffness	Roller Speed	Drum Amplitude
Periodic	Continuous Contact	Continual Contact		Yes	Low	Fast	Small
	Periodic Loss of Contact	Partial Uplift		Yes			
		Double Jump		Yes			
		Rocking Motion		No			
Chaotic	Non-periodic Loss of Contact	Chaotic Motion		No	High	Slow	Large

Figure 27. Typical amplitude and frequency patterns of the vibrating drums (adapted from Adam (1996))

One of the rare examples in the literature is the work of Hirose & Yanagida (1984), who established a relationship between the typical amplitude and frequency modes of vibratory drums and their influence on density ratios. It is evident that the contact area, specifically related to the drum's length and width, and the drum's speed are inversely proportional to the vibration energy. Additionally, frequency, amplitude, and the total number of passes positively affect vibration energy. Figure 28 presents a novel modified relationship that facilitates fine-tuning the vibration energy and density relationship during

preliminary adjustments. It also illustrates a revised relationship I created based on the original data. This enables convenient vibration energy and density correlation adjustment during preliminary tuning. Alternatively, power tampers, often called "jumping-jack" tampers, or small-sized compactors, including vibrating-plate compactors, can also be employed for compacting RCC. On the one hand, power tampers are known for their superior compaction capabilities, typically delivering a minimum force per blow of 8.5 kN. They are generally considered suitable for lift thicknesses up to 230 mm. However, if the layer thickness exceeds 230 mm, power tampers may potentially cause some deterioration on the RCC surface.

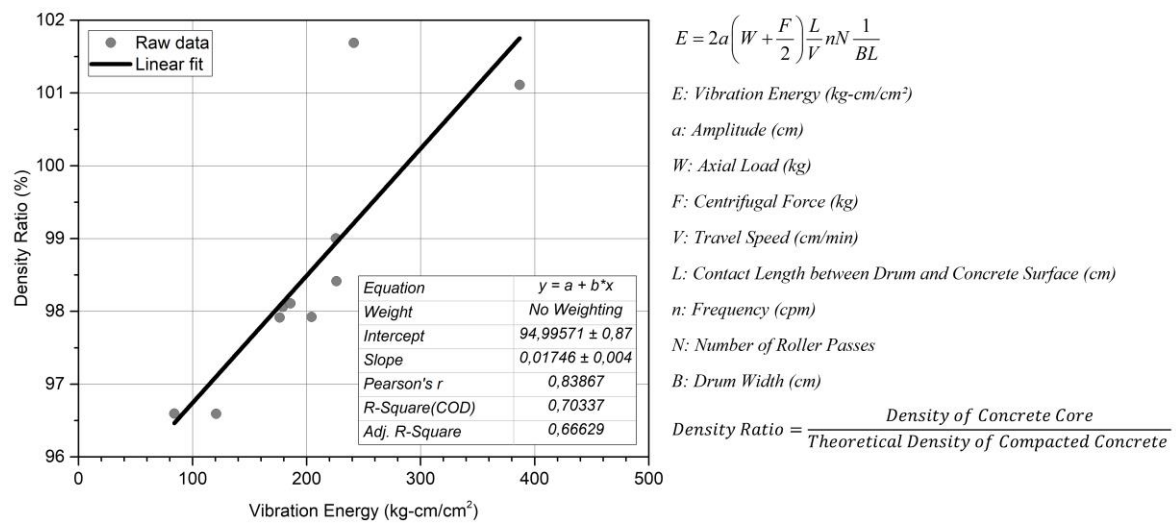


Figure 28. Typical amplitude and frequency patterns of the vibrating drums (adapted and restructured from Hirose & Yanagida (1984))

On the other hand, plate vibrators should ideally have a minimum mass of 75 kg and are suitable for RCC lift thicknesses ranging from 150 to 225 mm. In this thickness range, plate vibrators enable practitioners to achieve much smoother surfaces than power tampers (ACI, 2000). In some studies, engineers have attempted to develop project-specific compaction apparatuses replicating actual rollers to enhance in-field compaction performance (Marques Filho et al., 2008).

It is well-known that developing a simple compaction technique to replicate the performance of a 10-tonne vibratory roller on laboratory-produced samples has been a significant challenge. Various methods, such as ASTM C1435 (2014), have been proposed and standardized to simulate the compaction efforts of vibratory rollers in a laboratory

setting. Additionally, as regulated by ASTM C1176 (2013), vibrating tables can be used to cast RCC samples by simulating roller compaction through the vibration effect.

Kokubu et al. (1996) evaluated the compaction characteristics of RCC and established a new computation method based on a variable vibration table. This method examined the effects of vibration frequency, time, and acceleration on concrete properties. The study found that maximum compaction was achieved at low acceleration, and the vibration frequency did not affect the final filled volume. Particularly, aligning with the concept of the present study, Kokubu et al. (1996) highlighted that the sand/aggregate ratio and water content are crucial for optimal mix proportioning of roller-compacted concrete. The relationship between energy reduction and total water content was described by a decay function, identifying the optimal water amount as the point where energy reduction becomes minimal.

A critical concern that has long been under consideration is the extent to which alternative, smaller-sized compactors can replace 10- or 20-tonne vibratory rollers. Forssblad (1980) demonstrated a correlation and interchangeability between compaction methods, although the potential effects vary depending on the material being compacted (clays versus sands). Namely, Forssblad (1980) emphasizes that different soils require different stress levels during compaction. An example of this concept is illustrated in Figure 29a. While relatively low stresses are needed to trigger the movement for compaction (volumetric strains) among particles of non-cohesive soils, fine soils require much higher stress levels due to capillary effects that create bonding between particles. More so, this study suggests that, with the correct choice of compaction methods, successful compaction can be achieved without the need for heavy roller compactors. This indirectly supports the use of lighter compactors for modeling RCC material compaction in the laboratory. Tayabji & Okamoto (1987) observed that using a vibrating table to prepare RCC samples resulted in decreased overall strength and density due to the need to increase the moisture content in the fresh mix. It is worth noting that using pneumatic tampers to distribute grout throughout the RCC matrix can also be beneficial (ACI, 1999). Schrader (2003) conducted a comprehensive investigation into the limitations of various compaction techniques (i.e., pneumatic tamper, Hilti Kango, Vebe vibration table, internal vibrator, and rodding) compared to 10-tonne vibratory rollers regarding their potential effects on optimum water content, compressive strengths, and unit weights of RCC mixes. Based on the remarks of Schrader (2003), as presented in Figure 29b, the compaction technique significantly influences compaction performance. It not only affects the maximum achievable strength but also the moisture

content of the compacted mix. Furthermore, it suggests that as compaction energy decreases, the maximum achievable strength of RCC decreases, and the optimum moisture content tends to increase.

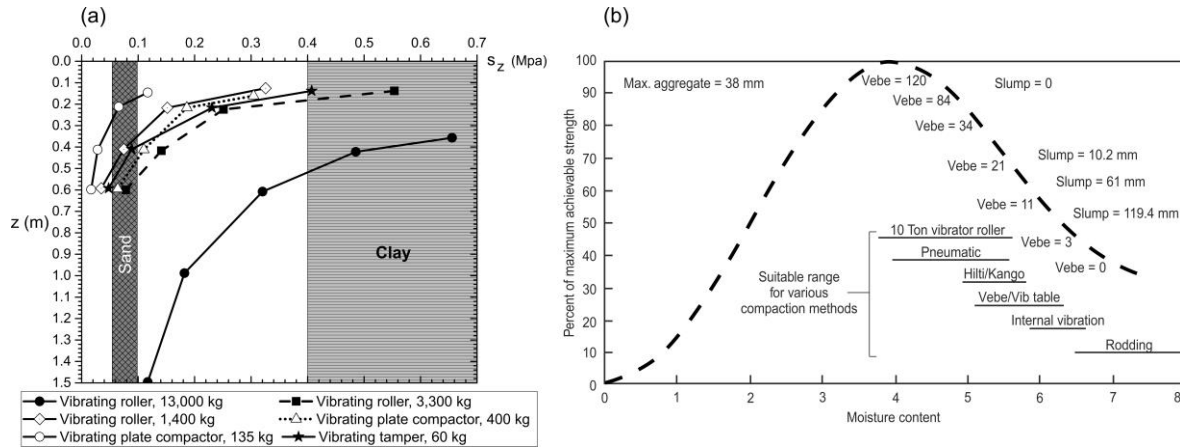


Figure 29. (a) Effects of different vibratory compactors on dynamic stresses in soil medium (adapted from Forssblad (1980)), (b) Curve for moisture content against the maximum achievable RCC strength depending upon various compaction techniques employed (adapted from Schrader (2003))

Several studies (e.g., Amer et al. (2003), Amer et al. (2004), and Shabani et al. (2021)) investigated the use of the gyratory compactor in producing RCC specimens with the desired density and optimizing the mix design process. In summary, Amer et al. (2004) observed that a 1 to 2% increase or decrease in the density of RCC with regard to the compaction method employed manifestly causes approximately 10% to 17% increase or decrease in strength. Shabani et al. (2021) proposed using the Superpave Gyratory Compactor (SGC) as a suitable surrogate for RCC production design. They found that while the gyration level for RCC mixtures has the least influence on mechanical capacity, the SGC has a substantial impact on the behavior of the fresh mixture. Therefore, they recommend using the SGC under field conditions with the same number of gyrations commonly adopted in typical SGC practices. A recent study by Selvam et al. (2023) compared the compaction performance of vibratory hammer, vibratory table, modified Proctor, and gyratory compactor in the laboratory to determine the extent to which they replicate actual infield compaction performed by rollers. Their research revealed that employing a vibrating hammer emerged as the most efficient approach for replicating field compaction, leading to improved dispersion of aggregates within the matrix and yielding higher strength values while

minimizing segregation tendencies. The modified Proctor method ranked second in this regard. Furthermore, they noted that although vibration tables and gyratory compactors demonstrated similar particle-to-particle distances compared to infield samples, they did not exhibit comparable mechanical and durability performance.

2.3.5. Effects of Different Rolling Procedures in RCCP Practice

Construction alternatives for longitudinal joints in RCCPs are discussed in a separate chapter in this study. However, a brief overview of rolling procedures is provided here to prepare the reader for further details. The compaction of fresh RCC is a fundamental part of the overall RCCP construction process. As a basic principle, the best rolling pattern is the one that is accomplished with the fewest number of passes, thereby yielding the desired material density (Ludwig et al., 1994).

Pittman (1986) emphasized the necessity of employing a rolling pattern and outlined a specific method for compacting RCC pavements. According to Pittman (1986), two consecutive passes should first be made, each comprising a backward and forward motion of the roller. In this phase, the roller touches on the paved lane's outer edge (located at the outer periphery of the full mat), while the roller drum protrudes from the lane with a 25- or 50-mm overhang. In doing so, it is thought that the roller ultimately creates an immediate consolidation at the unconfined edge, which later makes confinement for excessive lateral movement of the material during the subsequent rolling. Afterward, the roller is shifted 300 to 450 mm toward the inner edge to make two consecutive passes. The latter pattern directly creates a non-compacted edge belonging to the first lane, which is then used to set the screed paver's height for paving the adjacent lane. This approach offers the opportunity to simultaneously compact both sides of the fresh joint. The remaining non-compacted portion of the second lane, which is the center of the lane, should be compacted with two reciprocal passes. The entire pattern should be reiterated over the lane until four passes are accomplished. Whether the target density has been achieved determines the need for additional passes. If the interior edge is to be created as an old joint, it should be rolled in the same manner as the outer edge. Of course, utmost care should be taken to keep the surface flat in the joint zone, as no chamfering of the border is desired in this application.

Another compaction procedure covers one or two static passes followed by a 4- or 6-pass with a vibratory roller. Then, subsequent passes are employed with rubber tires, and a

steel drum is used to terminate the compaction and erase the existing marks created by the prior rollers. In this method, the roller's speed should not exceed 0.45 m/s (1.62 km/h) during the operation (Shoenberger, 1994). Lee et al. (2014) and Chhorn et al. (2017) explored alternative compaction methods for RCC. Lee et al. (2014) investigated some alternatives for better constructing a bicycle path using RCC and proposed a classification system for compaction methods, comparing them to one another. Their suggested method includes two passes with a tandem roller (backward and forward), three forward passes with a tandem roller without vibration, and then eight backward and forward passes with a tire roller with vibration. They particularly emphasized that this method works well for a 10 cm or thinner surface layer, and for thicker layers, different construction methods or equipment should be considered to achieve higher compaction energy.

Nevertheless, to the best of my knowledge, it is worth noting that in Türkiye, the majority of RCCP compaction processes traditionally involve using a single steel-drum roller with fewer than six passes. In Turkish practices, as with the technique employed for HMA pavements, any pause is not appreciated as the roller is in operation. Because it likely causes multiple ripples and permanent wave-shaped creases on the pavement's surface (Zarrin & Shirazi, 2015). In Türkiye, RCCP construction aims to complete a lane within a single shift, preferring to avoid additional construction joints whenever possible. To optimize and fine-tune the rollers' vibration properties for achieving the desired compaction level without the risk of over-compaction, fully autonomous (intelligent) compaction technology may be advisable (Anderegg & Kaufmann, 2004).

2.3.6. Impact of Curing Methods on RCCPs and Their Joints

For proper curing, several issues are addressed by the codes to achieve and optimize the desired throughputs. In the case of curing RCCPs, which occurs after the completion of rolling, the pavement surface should continuously be kept wet for no less than 24 hours. Pittman (1986) noted that, upon essential curing, RCCP should at least be cured for the first seven consecutive days. Keeping the surface wet enough can be achieved through a system, whether mechanical (e.g., sprinklers) or static (e.g., wetted sacks with water or chemical substances covering the lane). As with the use in conventional concrete, liquid membrane-like chemical compounds meeting ASTM C309-19 (2019) requirements can also be used for curing RCC. Given that the surface of RCC is relatively open, it quickly becomes prone to

drying much faster than conventional concrete surfaces (Delatte, 2014; Harrington et al., 2010).

2.3.7. Impacts of Short- and Long-term Weather Conditions on RCCPs

RCCPs generally demonstrate remarkable resilience against various forms of distress, especially when exposed to harsh winter climates. Similar protective measures should be applied during cold weather in RCCP construction, just as with conventional concrete casting and curing. However, it is essential to note that an increase in ambient air temperature can reduce the workability of fresh RCC, significantly limiting the available time for mixing and compaction (Halsted, 2009). Fleckenstein et al. (2002) observed that water or moisture within the air voids tends to freeze abruptly at low temperatures. This freezing action results in additional tensile stress within the pavement structure due to the gradual expansion behavior of water. This phenomenon often leads to cracking and other forms of distress, expediting the expected deterioration of the pavement. Due to the absence of bleed water, evaporation rapidly initiates water removal from the paste. Right after the final compaction, it is crucial to take indirect measures to prevent excessive evaporation (Harrington et al., 2010). Failing to do so can result in shrinkage cracking and the development of shallow micro-cracks, significantly affecting the quality of the pavement surface and ride comfort. Additionally, Zollinger (2016) emphasized that sole fatigue-driven failures in RCC pavements have been relatively rare to date.

Enhancing the freeze-thaw and deicer scaling resistance of RCC can be achieved through adherence to relevant specifications, which typically recommend the use of high-quality cement, suitable aggregates, meticulous placement with specified density, and proper curing (Piggott, 1999). The coefficient of thermal expansion, a critical physical property related to concrete-to-concrete contraction, is often used to characterize the thermal properties of coarse aggregates in the fresh mix of any concrete. Failures are attributed to in-body stress and can be mitigated through proper curing. While freeze-thaw durability is not the primary requirement for RCC, since strength-oriented tests dominate the design process, RCC demonstrates relatively good resistance due to the use of well-graded aggregates (Choi & Hansen, 2005). As aggregate sizes decrease, RCCPs tend to exhibit uniform surface texture characteristics, which directly contribute to better ride quality (Piggott, 1999). However, achieving a purely smooth surface texture is not always possible due to RCC's

limited workability and compactability, partly attributed to standardized aggregate gradation. On the one hand, regarding air-entrainment, a technique commonly used in conventional concrete production, it is worth noting that air-entraining agents do not significantly enhance RCC's freeze-thaw durability to the fullest extent (ACI, 1999; Hansen & Reinhardt, 1991). On the other hand, there is still limited research supporting the use of air-entrainment techniques to improve RCC's freeze-thaw resistance (Hazaree et al., 2009).

The standard casting procedure should be completed within 45 minutes when the ambient air temperature is below 30°C. If the ambient air temperature is 30°C or higher, the entire process should be completed within 30 minutes (Shin & Carboneau, 2010). It is crucially important to note that this guideline has been slightly revisited by the Illinois Department of Transportation (IDOT, 2014), which suggests that the ambient air temperature should be lower than 30°C but higher than 10°C, allowing the entire operation to be completed within an hour. RCC placement is considered acceptable if the air temperature is above 4°C. However, if the temperature drops and is expected to reach 7°C or lower, construction should be immediately halted to prevent negative temperature-related issues.

2.4. Design-Related Aspects of RCCP's Longitudinal Joints

The primary objective of this study is to identify potential construction techniques for longitudinal joints in RCCP and validate them through finite element analyses and model-driven laboratory tests. This section provides an overview of the critical issues related to the problem and discusses the methodologies that may impact the analyses.

2.4.1. Contact Mechanisms between Rollers and Compactable Medium

Contact mechanics is a complex system designed to analyze the combined physical effects when one object comes into physical contact with another. Heinrich Rudolf Hertz is widely acknowledged as the pioneer of this branch of mechanics. The origins of contact mechanics date back to the late 1800s, marked by the introduction of precise yet intricate formulations (Hertz, 1882a, 1882b) that were groundbreaking for their time. Subsequently, Joseph Boussinesq made a significant contribution to the field by approaching the study of solid elastic bodies from a different perspective (Boussinesq, 1885), ushering in a new era.

The evolution of theories in contact mechanics over the course of a century can be explored in the comprehensive paper by Johnson (1982). Additionally, I highly recommend that all readers delve into the excellent open-access book by Popov et al. (2019), which covers nearly all analytical solutions to various contact problems.

The most renowned theory to date is known as the Hertz theory, and the corresponding set of principles in contact mechanics is referred to as Hertzian contact. In a fundamental sense, there are several potential contact mechanisms for any physical medium in space, including point, line, and surface contacts (Fischer-Cripps, 2007). As elements in contact continuously undergo physical movement, their geometric shapes eventually transition into contacting surfaces, giving rise to a complete contact problem. Figure 30 provides a concise summary illustrating the geometric layout for contact mechanics discussed in this chapter.

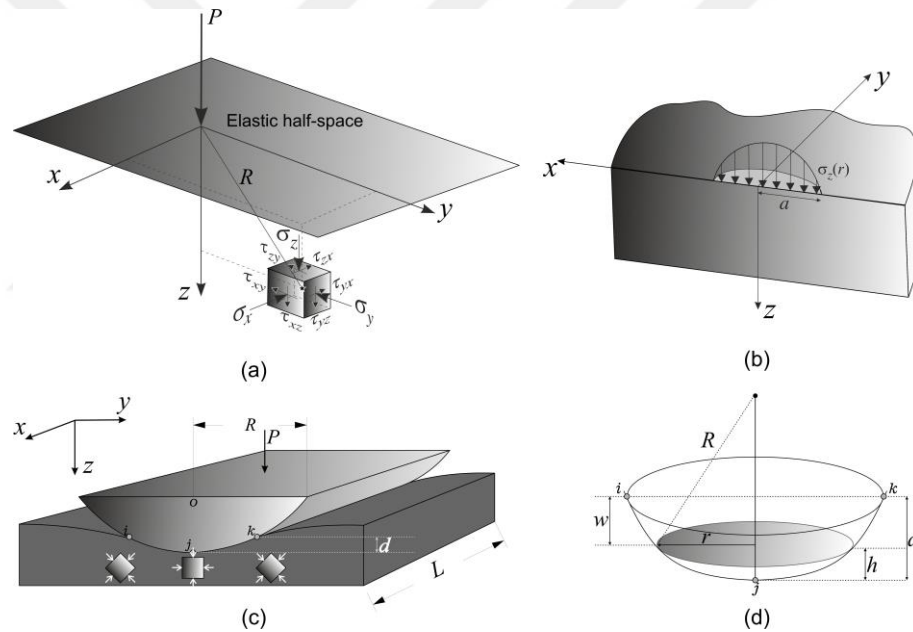


Figure 30. Geometric layout for contact mechanics based on a half-space and a cylindrical indenter: (a) Boussinesq solution in cartesian coordinates, (b) Half-space with and elliptical load, (c) Contact between half-space and roller, and (d) Compatibility of displacements (adapted from Caicedo (2019))

When addressing a typical mechanical contact problem, the solution is generally perceived to rely on stress distributions emanating from the stiffness and shapes of the bodies involved. The primary approach to problem-solving involves determining displacements between contacting bodies based on governing equations, with the radii of curvature of the contacting surfaces playing a crucial role in the calculations. However, this theory makes

several assumptions to simplify the problem. For instance, from the Hertzian perspective, the material is considered ideally homogeneous and isotropic, and the contacting surfaces are assumed to have no friction, being plain and continuous. It is further assumed that a small amount of strain initiates due to contact, primarily because the contact areas are much smaller than the size of the contacting bodies. Consequently, the problem is addressed within a hypothetically elastic half-space in and around the contact zone, modeled by elliptic equations.

In pavement engineering, contact problems frequently arise as challenges in mechanical systems to be analytically addressed or in attempts to comprehend certain physical behaviors from a mechanistic-empirical standpoint. Contact mechanics proves to be a valuable tool in assessing various physical phenomena, spanning from modeling the California Bearing Ratio (CBR) test to understanding the compaction mechanics of roller compactors, even in the context of granular mediums. It is evident that contact can be broadly categorized into two distinct areas: Hertzian and non-Hertzian. For example, the cone penetration test (CPT), plate load, and CBR tests are analytically modeled from a non-Hertzian perspective. Conversely, the micromechanical behaviors of hypothetically round granular materials and the compaction problem of an elastic half-space by a roller compactor are more directly modeled as Hertzian.

Due to the brevity of this study, detailed derivations of lengthy and complex relations for all common problem types are not provided here, as it goes noticeably beyond the scope of the present study. However, only the portion directly relevant to the current research topic will be briefly mentioned. For in-depth exploration and comprehensive solutions to various contact problems, I highly recommend referring to the excellent books by Johnson (1985), Boresi & Schmidt (2002), and Fischer-Cripps (2007). These texts present numerous specific solutions and stand as some of the most compact and comprehensive resources ever written on contact mechanics.

The analysis is based on Hertz's contact theory, which assumes that each object in contact has a uniform material composition, adhering to Hooke's law. The materials are modeled as homogeneous, isotropic, and elastic. The solution is derived in an elastic half-plane under plane stress conditions, which are transformed into plane strain by substituting E with E^* , where: $E^* = E/(1 - \nu^2)$ and replacing ν with $\nu^* = \nu/(1 - \nu)$. Before loading, the surfaces are tangential, and the contact area is ellipsoidal. In the initial contact, points on the two surfaces are equidistant from the tangent plane. The semi-major and semi-minor axes

of the contact ellipse are denoted by a and b , respectively, with $k = b/a = \cos\theta$, $k' = 1 - k^2 = \sin\theta$. The contact problem can be addressed in one or two dimensions. Typically, a one-dimensional (1D) approach is used. For two contacting bodies, the contact modulus is $\frac{1}{E^*} = \frac{1-\nu_1^2}{E_1} + \frac{1-\nu_2^2}{E_2}$. The expression determines the equivalent radius of the two surfaces, $\frac{1}{R_e} = \frac{1}{R_1} + \frac{1}{R_2}$. For a flat surface (e.g., roller-to-road contact), R_2 approaches infinity, simplifying R_e to R_1 . For point contact, the radius of the contact area, a , is $a = \pi p_0 R_e / 2E^*$. Hertz's proposed pressure distribution is calculated using the expression $p = p_0(1 - (r/a)^2)^{1/2}$, where a represents the radius of the contact area, r is the radius measuring the pressure, and p_0 is the maximum pressure at the contact. The relative approach of points within the solids is denoted by δ and is given by $\delta = \pi a p_0 / 2E^*$. The total load F is related to the pressure by,

$$F = \int_0^a p(r) 2\pi r dr = \frac{2p_0\pi a^2}{3} \quad (3)$$

From this, the maximum pressure p_0 , contact radius a , and displacement δ are:

$$p_0 = 3F/2\pi a^2 = (6FE^{*2}/\pi^3 R_e^2)^{1/3} \quad (4)$$

$$a = (3FR_e/4E^*)^{1/3} \quad (5)$$

$$\delta = a^2/R_e = (9F^2/16R_e E^{*2})^{1/3} \quad (6)$$

In a more realistic scenario, such as a roller moving over a fresh roller-compacted concrete (RCC) lane, the contact is modeled as line contact rather than point contact. The contact area forms a rectangle rather than a circle, with the half-width of the rectangle given by $b = \left(\frac{4R_e}{\pi L E^*}\right)^{\frac{1}{3}}$. Where L is the contact length. The maximum pressure is $p_0 = \frac{2F}{\pi b L}$. In two-dimensional line contact, the relation between roller load and penetration into the half-space is $F = \frac{\pi a^2 E^*}{4R}$, where a is half of the contact width, and the pressure distribution along the x -axis is:

$$p(x) = \frac{2F}{\pi a^2} (a^2 - x^2)^{1/2} \quad (7)$$

The contact width and deformation can also be estimated using the following relations. For the use of the given equations, please refer to Johnson (1985), Slocum (1992), and Young et al. (2002).

$$d_{\text{calculated}} = \left(\frac{F}{\pi L E^*} \right) \left(1 + \ln \left(\frac{2\pi(L)^3 E^*}{F 2R} \right) \right) \quad (8)$$

$$a_{\text{zone}} = \left(\frac{2F 2R}{\pi L E^*} \right)^{1/2} \quad (9)$$

$$P_{\text{max}} = \left(\frac{2F E^*}{\pi L 2R} \right)^{1/2} \quad (10)$$

Where, E : Young's modulus of the material being compacted, ν : Poisson's ratio of the material being compacted, E_{roller} : Young's modulus of the roller material, ν_{roller} : Poisson's ratio of the roller material, F : Applied load exerted by the roller, L : Length of the drum or roller in contact with the material, R : Radius of the roller, E^* : Combined modulus of elasticity for the contact between the roller and the material, accounting for the properties of both materials, $d_{\text{calculated}}$: Deflection of the contact zone, a_{zone} : Half-width of contact zone, P_{max} : Maximum Hertzian pressure.

As an alternative, relations were given by Lundberg (1939) and then reexamined in Johnson (1985) that account for the material properties of both the roller and the compacted surface concerning the coordinate indices used to express deformations. The coordinate indices used to express where the settlements are calculated are as follows. $\delta(0,0)$ denotes the center of the contact area, $\delta(y)$ denotes the length-along distribution, $\delta(L)$ is for the particular case at the outer edge of the roller.

$$\delta(0,0) = \frac{F}{\pi L E^*} \left[1.8864 + \ln \left(\frac{L}{a} \right) \right] \quad (11)$$

$$\delta(y) = \delta(0,0) - \delta(0,y) = \frac{F}{2\pi L E^*} \ln[1 - (y/L)^2] \quad (12)$$

$$\delta(L) = \frac{F}{2\pi LE^*} [1.193 + \ln(L/a)] \quad (13)$$

2.4.2. Influence of Fiber Inclusion on the Performance of RCC Pavements

Various types of fibers, including natural feather, steel, and polypropylene-based, have been used in concrete production for a considerable time. In both conventional concrete and RCC, the primary purpose of incorporating fibers is straightforward: to inhibit crack formation and enhance the modulus of rupture, thereby reducing slab thickness and extending the economic lifespan. Two critical pieces of knowledge were established long ago: (1) savings in material and construction expenses offset the cost of fibers, and (2) an increase in fiber content in the concrete mix correlates positively with post-cracking performance, albeit within certain limits (Nanni, 1989). Kilic & Gok (2021) examined the variations in strength and durability properties of roller-compacted concrete when produced with varying types and amounts of polypropylene fibers. Their findings revealed that the inclusion of 0.20% fibers resulted in a 14.4% reduction in compressive strength, while a 0.50% inclusion led to a more significant reduction of 46.8%. Additionally, they observed that despite increasing water absorption characteristics and decreasing specific gravity, fiber-reinforced RCC exhibited good performance against freeze-thaw cycles. Rooholamini et al. (2018) applied response surface methods to investigate how the addition of macro-synthetic fibers to RCC pavements affected their mechanical properties. They found that adding these fibers to the mixture linearly decreased the Vebe time, but had no significant impact on compressive strength. Their research also highlighted two critical findings in pavement technology: (1) the type of fiber used had a more notable effect on fracture energy than flexural strength, and (2) the interaction between mortar and fibers influenced the post-cracking behavior of RCCPs. In contrast, Zhang et al. (2018) studied the effects of incorporating steel fibers and rubber particles into the mixture. Their findings suggested that adding steel fibers and rubber particles did not reduce the failure characteristics induced by freeze-thaw cycles in a potassium acetate solution, as all tested RCC samples already contained microcracks. Furthermore, they noted that the positive influence of steel fibers and rubber particles on the relative dynamic elastic modulus reduction was not self-evident when subsequent cement hydration was not considered.

2.4.3. Constitutive Theories for Modeling RCC Behavior

Constitutive models, typically characterized by stress-strain relations and mechanics laws, play a crucial role in understanding the behavior of materials such as soils, rocks, and concrete. All these materials exhibit frictional properties, and the confining pressure notably influences their characteristic plastic behaviors. Conventional constitutive models are termed pressure-dependent elastoplasticity in materials with innate frictional characteristics, indicating a mechanical phenomenon that is neither fully elastic nor friction-independent (Zhang et al., 2017b). Constitutive models for RCCs can be categorized based on their physical state, either fresh or hardened. The transformation of Coulomb's friction model to the Mohr-Coulomb model (hereafter referred to as MC) involves a slight improvement, which could be specified for clarity. The MC yield criterion, grounded in two fundamental phenomena, i.e., friction and cohesion, has a longstanding application in modeling granular materials within theoretical soil mechanics.

The MC model inherently assumes that the yield in the continuum is manifested as the principal stresses (τ and σ) on each planar surface, following the well-known relation, $\tau = c + \sigma (\tan\phi)$, where τ is the unit shear stress, c is the unit cohesion, σ is the unit normal stress, ϕ is the angle of internal friction. The same relation also expresses Mohr circles' envelope (failure envelope), as shown in Figure 31.

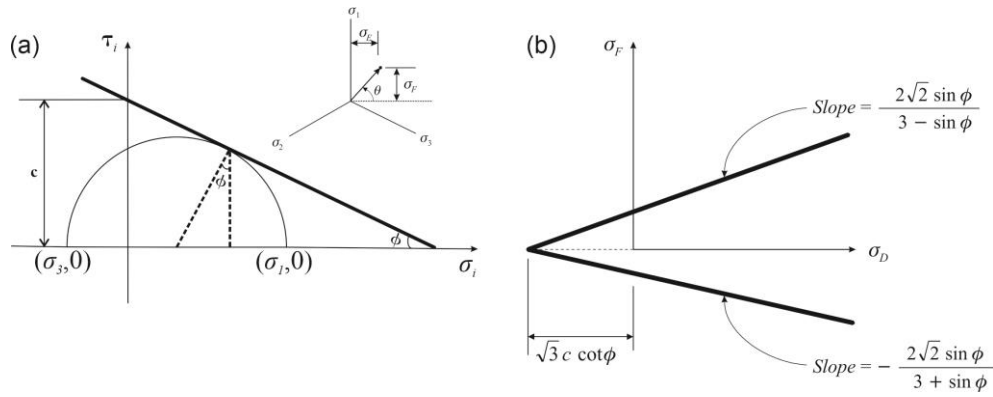


Figure 31. (a) Mohr-Coulomb yield surface on the Mohr's circle, (b) Long section through the Coulomb yield surface (adapted from Davis & Selvadurai (2002))

From the dashed triangle's geometry and using σ_1 , σ_3 , well-known constitutive relations can be derived, but they are not given here for brevity. More often than not, an envelope-oriented graph can be used instead of the traditional Mohr cycle. Assuming that

the failure has taken place due to principal stresses in order of $\sigma_1 > \sigma_2 > \sigma_3$. The Mohr circle at this critical state derived from σ_1 and σ_3 should be tangent to the line that touches the circle itself. With that said, the MC yield criterion can directly be transformed into the Tresca yield criterion by replacing ϕ with 0° and into the Rankine failure criterion by replacing ϕ with 90° (Caicedo, 2019).

Note that the subscripts D, F, E denote the three orthogonal vectors. While σ_1, σ_2 and σ_3 denote the principal stresses, σ_D, σ_E and σ_F denote the stress components. the component σ_D denotes the vector coinciding with the space diagonal. σ_E is for horizontal, σ_F is for vertical. It is also possible to use the polar coordinate system through the quantities $p = (\sigma_1 - \sigma_3), q = (\sigma_1 + \sigma_3), \theta$. In addition, θ denotes the Lode angle, and it is calculated by the equation of $\tan\theta = \frac{\sigma_F}{\sigma_E} = \frac{2\sigma_1 - \sigma_2 - \sigma_3}{\sqrt{3}(\sigma_3 - \sigma_2)}$ and it affects the deformation characteristics and failure patterns of solids, particularly metals (Xue, 2007).

The Lode angle has gained crucial importance and garnered interest over the years. Festugato et al. (2019) conducted a comparative analysis involving different cement amounts and Lode angles to characterize the mechanical behavior of artificially cemented sand subjected to multiaxial loading conditions. Their study highlighted that lightly-cemented sand mixtures exhibit non-isotropic mechanical behavior. This observation implies that there should be a minimum threshold for the amount of cement, suggesting the significance of cement quantity in fresh RCC production. Shang & Ji (2014) performed a comparative analysis to elucidate the mechanical behavior of various concrete types under multiaxial compression. Their findings indicated that the biaxial compressive strength surpasses the uniaxial compressive strength for all tested concrete types. Moreover, they demonstrated that the increase in biaxial compressive strength relative to uniaxial compressive strength varies with the lateral stress ratio, underscoring the role of the Lode angle in influencing the mechanical behavior of concrete.

The MC model is commonly applied in modeling fresh RCC, leveraging its compatibility with geomaterials. The hypothetical cohesive elastoplastic nature of fresh RCC aligns well with the characteristics of the MC model. While limited research exists on constitutive models for hardened RCC, Kalantary & Sadeghi (2003) suggested the utilization of a four-parameter concrete model, such as Hsieh et al. (1982), with some calibration (e.g., Jun Peng & Yonghong (1997)), for modeling RCC. This approach is deemed more accurate than conventional two-parameter models designed for ordinary concrete. Kalantary &

Sadeghi (2003) proposed a modification to enhance the MC model, introducing an equation (14) that tunes the internal friction angle.

$$\phi = 26.3 \times (0.61)^{(\sigma_i/f_c)} + 31.2 \text{ for } 0 \leq \frac{\sigma_i}{f_c} \leq 4 \quad (14)$$

Where ϕ denotes the angle of internal friction, σ_i is the mean pressure (equivalent to confining pressure), and f_c represents the compressive (uniaxial) strength. According to the provided relation, the estimated friction angle is approximately 58° for the unconfined state. Under confining pressure, when it is four times larger than the compressive strength, the angle drops to approximately 35° . Zhang et al. (2017a) aimed to characterize compressive behavior and develop a statistically-driven constitutive model for RCC under impact loading and observed that variations in aggregate gradation led to specific differences in the dynamic mechanical properties of RCC.

From a slightly different standpoint, Drucker & Prager (1952) (hereafter DP) provided an alternative yield function for a more accurate representation of perfectly plastic soil behavior. The DP model is formulated as a generalized version of the MC model. On the one hand, the DP model has become a classic in the mechanical modeling of granular media due to its utilization of a yield surface where mechanical behavior is purely elastic. Figure 32 illustrates the MC and DP yield surfaces in the deviatoric or orthogonal (π) plane. Unlike the isometric plot, stress variations' magnitude is readily observed on the π -plane, which is the primary projection of the yield surface for the specific case of hydrostatic state ($\sigma_1 = \sigma_2 = \sigma_3$). Notably, from Figure 32, it can be inferred that the DP yield surface is tangent to the Coulomb hexagon. Furthermore, several advanced mathematical models have been developed to analyze the behavior of hardened concrete under cyclic nonuniform loading. Among these models, the Concrete Damaged Plasticity (CDP) model stands out as one of the most intricate theories designed to simulate the behavior of hardened concrete. Hillerborg et al. (1976) laid the foundations of this comprehensive theory, proposing a method based on tensile testing, assuming that stresses operate through a fracture as long as it is slightly open.

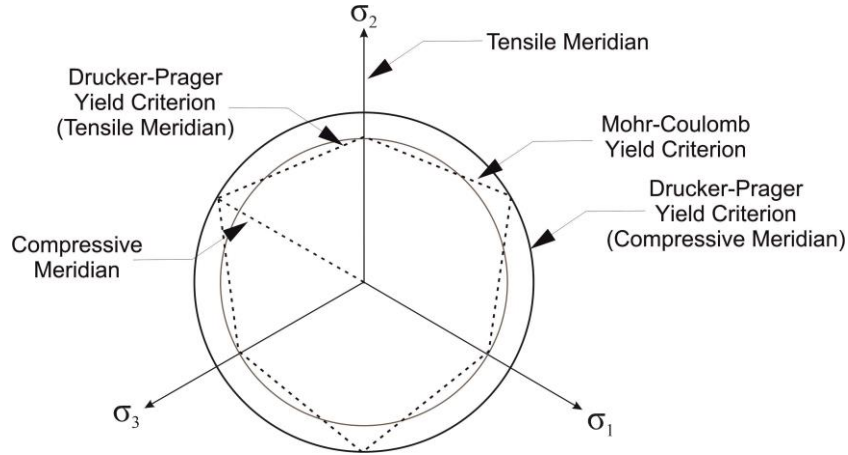


Figure 32. Mohr-Coulomb and Drucker-Prager yield surfaces in octahedral (π) plane

This assumption is considered in the context of energy absorption in the energy balance approach. One of the goals of the theories developed in the past was to find mathematical solutions for using larger mesh sizes to overcome the processing limitations of computers with limited capacity compared to today's more advanced capabilities. Lubliner et al. (1989) introduced the CDP model and initially named the theory the Plastic Damage Model for Concrete. Shortly afterward, Lee & Fenves (1998) defined the plastic damage model specifically for concrete structures subjected to cyclic loading. Although the theory of the CDP model is complex, Hafezolghorani et al. (2017) recently developed a simplified version to enhance its usability in modeling. For brevity, the complete derivation of the CDP theory is not provided here. Instead, the foundational principles of the CDP theory are explained from its inception.

Lubliner et al. (1989) asserted that any stress mechanism other than triaxial compression, where hydrostatic pressure prevails over stress deviator, inevitably leads to strain-softening in concrete and geomaterials. This stress mechanism ultimately results in a complete loss of strength, categorizing them as frictional materials with true cohesion (not apparent), sharing this characteristic with cohesive soils. Strain-softening is observed in both pure compression and tension, and despite the simplicity of the cohesive material approach, neither the MC nor DP theories can profoundly represent the actual stress-strain behaviors of concrete (Read & Hegemier, 1984). In that sense, this phenomenon explains the necessity of CDP theory. Finally, Lubliner et al. (1989) developed the CDP theory based on a novel yield criterion, effectively accounting for both elastic and plastic stiffness degradation observed in experimental data. Lee & Fenves (1998) explicitly stated that microcracking and

its formation govern the softening behavior of the material on a macroscopic scale. This softening results in strain redistribution within the medium, a phenomenon commonly modeled by pure plasticity theory (Chen & Han, 1988). Read & Hegemier (1984) determined that strain-softening, commonly deduced from laboratory studies, is not an inherent material attribute of rock, soil, and concrete. It can result in stress wave propagation issues for softening materials. Microcracking leads to stiffness degradation, which is difficult to capture using classical plasticity models. As a result, mechanical theories based on continuum damage fail to offer opportunities for tuning or controlling plain concrete structures subjected to cyclic or multiaxial loading. Stiffness degradation is a subject often disregarded but is crucial. Corrosion-induced cracks in reinforced concrete structures can reduce the stiffness of the cover concrete, leading to unexpected failures in the cover section of the elements. This is caused by softening in the stress-strain relationship of the yielded concrete, as noted by Zhong et al. (2010). Huang & Karihaloo (1993) observed that quasi-brittle materials, even in their undeformed and non-distressed form, are believed to have pores and microcracks. However, under low-magnitude loads creating diminutive tensile stress, these materials behave in a way that the impact of tensile stress on the material's mechanical reaction is not readily apparent.

2.4.4. Base and Subgrade Modeling in RCC Pavements

The subgrade and base course requirements in RCCPs closely resemble those for conventional concrete pavements. As a fundamental rule, these layers should be well-compacted to establish a robust and stable foundation upon which the RCC surface course will be placed. Moreover, in regions prone to seasonal extreme weather variations, such as heavy rainfall and freeze-thaw cycles, moisture content plays a pivotal role in preserving the strength of the base course and subgrade. In that sense, proper water drainage through stable and gentle infiltration is essential for base and subgrade courses (Pittman, 1986).

The temporally varied sharp loss in the bearing capacity of the base course due to various factors (e.g., repetitive loads resulting from heavy traffic, fluctuations in moisture content, soil matrix softening or dilation (expansion in volume due to shear), clayey bedding shrinkage, frost-induced heave, and landslides) can detrimentally impact not only the all-out performance but also the service life of the pavements. In light of this, it is essential to assess

whether the base course meets the necessary standards during the design phase before casting the surface course for any pavement (Colombier, 2004).

Designers model the base and subgrade within FEA software according to their preferences and engineering judgments. While several models are available for these courses, some commonly endorsed principles can guide the modeling process. In a broad sense, both elements are considered stress-dependent. The base and subgrade can be modeled for simplification, assuming both layers exhibit linear isotropic behavior. Additionally, the mechanical behavior of aggregate-made base courses is often modeled as stress-hardening material, while cohesive subgrade soils can be modeled as stress-softening materials. The cohesive soil approach can be employed when modeling the base and subgrade with available MC and DP failure models. Typically, other invariants of MC models are not frequently used in base modeling because the yield surfaces, such as those of Matsuoka & Nakai (1974) and Lade & Duncan (1975), are initially designed for cohesionless media.

The engineering judgment primarily relies on triaxial test data to define material strength and resilient modulus. A low modulus of subgrade reaction and a low resilient modulus typically indicate wet, soft, and cohesive subgrades. Proper compaction is key to obtaining a sufficiently rigid and stable base course. The base course must also retain its hypothetical solid characteristics during casting and be resilient to traffic loads and other external stresses (Rollings, 1988a).

In the case of heavy-duty pavements, Rith et al. (2018) conducted model-dependent finite element analyses to determine the long-term performance of a composite pavement body made up of the HMA surface course and RCC base course. Their findings demonstrated that the elastic modulus of the RCC-base proportionally influenced the overall performance of the pavements. Moreover, a high elastic modulus and increased RCC base thickness reduced the need for the topmost HMA layer (overlay) and mitigated rut depths in asphalt layers.

In summary, selecting constitutive models for the base course and subgrade is entirely a matter of the designer's engineering judgment, ideally based on known or hypothesized physical properties and potential stress-dependent variations.

2.4.5. Load Transfer Phenomenon in RCCP Longitudinal Joints

The term "load transfer" refers to the percentage of load initially applied to one slab carried by an adjacent unloaded slab. The load transfer capability of joints is typically determined by load transfer efficiency (LTE) or joint efficiency, expressed as either the shared deflection or stress. Joint efficiency is a fundamental feature of any pavement joint and is virtually essential in distributing loads and deformations between adjacent slabs. Figure 33 provides an illustrative representation of free edge conditions and load transfer from the adjoining slab.

A comprehensive guide from the American Association of State Highway and Transportation Officials for designing pavement structures (AASHTO, 1993) recommends a simple approach based on the proportioning of load or stress values from the neighboring and loaded slabs, respectively. The AASHTO method can be expressed either through shared deflection or stress. In this manner, joint load transfer efficiency based on deflection, d_{je} , is calculated as $d_{je} = 100(d_u/d_l)$, where d_u is the deflection at the joint of the unloaded slab, and d_l is the deflection of the loaded slab. With the same thought, load transfer efficiency based on the shared stress, s_{je} , is calculated as $s_{je} = 100(s_u/s_l)$, where s_u and s_l are the apparent stresses occurring in the unloaded and loaded slabs, respectively. An alternative method for calculating load transfer efficiency involves using the free edge condition and stress at the bottom of the loaded slab as $s_{je} = 100(1 - s_l/s_e)$.

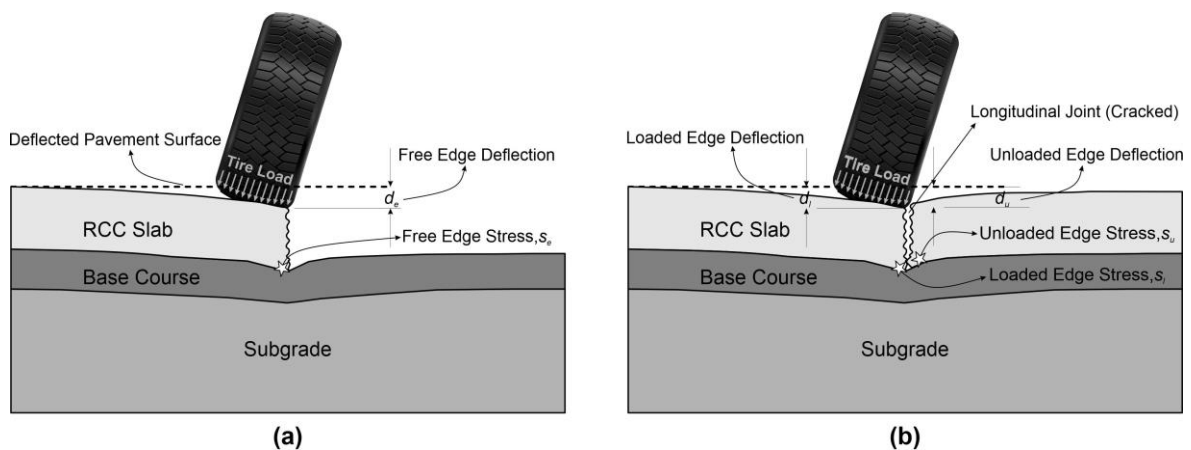


Figure 33. Schematic for load transfer efficiency: (a) Free edge condition, (b) Load transfer from the adjacent slab (adapted from ACI (1995) and USACE (1988))

For the evaluation of rigid pavement slabs, the stress modification factor ($J_e = 100/(100 + s_{je})$) can be used to describe how far the actual joint stress deviates from theoretically computed free-edge stress. This factor accounts for the actual joint stress in the slab due to a given load transfer efficiency. The actual slab stress at the joint (s_{act}) can be calculated by multiplying the stress modification factor (J_e) with theoretically computed free-edge stress (s_{fe}), to wit; $s_{act} = J_e(s_{fe})$. These calculations are mostly used to assess joint performance using results obtained from nondestructive tests (NDTs) once the road is open for use. In that regard, the US Corps of Engineers (USACE, 1988) suggested a relation [$LT = 2.27(LTE) + 43.8(LTE^2)$] for use in computing the percent load transfer (LT) using joint load transfer efficiency exists in between the two slabs.

In measuring the load transfer capability in longitudinal joints, it is crucial to incorporate a physical ajar, a depth-through discontinuity, between adjacent slabs. This ajar can take the form of a continuous, partial, or complete saw-cut to shape the joint, or it may result from a significant crack. This point of view helps address such weak areas between two adjacent lanes laid at different times. In this regard, in the case of cold joints, which are distinct from somewhat cracked pavement, the aggregate interlocking performance should be evaluated separately for each slab, highlighting the mechanical performance of each interconnected slab. Namely, the mechanical performance of each interconnected slab becomes significantly essential. When the joint is cold-formed, both sides of the lane remain connected at the joint area due to the bonding capability of fresh concrete with the existing concrete.

The load transfer efficiency typically ranges from 0 to 100 percent. Meanwhile, the actual load transfer falls within the range of 0 to 50 percent, indicating that the adjacent slab carries half of the load or stress. In concrete pavements, a 25% LTE is often considered a desirable benchmark for contraction joints. The effectiveness of the load transfer phenomenon at joints and cracks depends on the extent to which aggregate interlock is achieved at the joint. Stresses generated by applied loads are transmitted to the unloaded slab through shear stress at the vertical interface between two adjacent slabs. In most cases, LTE values have been observed to range between 12-18 percent for the construction joints of RCCP and 30-37 percent for PCC pavements (Rollings, 1988a). The US Army Corps of Engineers (USACE, 1988) recommended assuming a load transfer amount of zero at all joints in relatively thick slabs, which is a highly conservative approach. It is believed that the LTE is conceptually 25% for airfield pavements and parking areas for economical design.

Pittman (1996) conducted comprehensive research on 12 different RCC pavements using a heavyweight deflectometer (HWD) and found that joint efficiencies significantly decreased with increasing crack width, crack spacing, and dropping temperature. This finding has then partially appeared in the American Concrete Institute manuals (ACI, 1995), indicating the remarkable effects of ambient temperature on pavement joint performance regarding *LTE*. In a similar vein, Sok et al. (2018) also emphasized the critical factors influencing load transfer capability, including temperature variation, crack opening size, and maximum aggregate size. They further noted that an aggregate gradation with a 13-mm maximum size had a negative impact on load transfer efficiency, while a maximum size of 19 mm yielded far better results.

2.4.6. Previous Finite Element Analyses on RCC Pavements

Whether simple or complex, several computer codes have as yet been developed for FEAs for use in the context of pavement engineering. Examples include WESLAYER (Chou, 1981), ILLI-SLAB (Tabatabaie & Barenberg, 1980), J-SLAB (Tayabji & Colley, 1986), and ISLAB2000 (Khazanovich et al., 2000). Some studies (Darter et al., 2001; Uddin et al., 1997; Zaghloul et al., 1994; Zdiri et al., 2009a) primarily employed the Abaqus software to analyze rigid pavements and their interaction with the foundation bed.

In a broader context, the bottom of the RCC wearing course in pavements represents a potential weak zone within the pavement structure. This is primarily due to a gradual reduction in density resulting from the compaction process using vibratory rollers and the associated changes in thickness (Rollings, 1988a). Pietzsch & Poppy (1992) identified interrelated factors between the soil medium and vibratory rollers based on analytical perspectives. These factors pertain to soil compaction properties and the mathematical model that accounts for both the roller's nonlinear motion and the resulting ground deformation. From this perspective, their study serves as a cornerstone for future computer-based simulation studies aimed at a deeper understanding of the behavior of RCC and the underlying base when exposed to vibratory compaction. Additionally, Pietzsch & Poppy (1992) pointed out that if a pavement model is well-developed within a computer-aided design (CAD) environment, it can reduce the required number of tests to verify the targeted compaction level. Consequently, this process of elimination leads to cost-saving testing plans. Zdiri et al. (2009a) conducted comparative analyses to model the stress-strain

behaviors in RCCPs. They employed both a 3D model designed in Abaqus and a 2D model, comparing them with other analytical techniques, including closed-form formulas, PCA methods, and FEM. They concluded that the 3D modeling in Abaqus yielded slightly lower stress values than those obtained through other analytical methods. In conclusion, they attributed these discrepancies to variations in the indispensably fundamental assumptions inherent to each method. Furthermore, Famili & Vafaei (2016) conducted numerical analyses on RCCPs, and their results corroborated the findings of Zdiri et al. (2009a). Zdiri et al. (2009b) demonstrated that edge loading along the longitudinally unsupported edge of the pavement generated the highest stresses compared to other loading cases, such as loading at the center of the pavement and in the corner of the mat. Additionally, Suo & Wang (2013) conducted a series of Finite Element Analyses (FEAs) using ANSYS software on a flexible pavement model with an asphalt surface course resting on the RCC-treated base layer above a 6-meter-thick subgrade. Suo & Wang (2013) observed that an increase in the RCC-base thickness resulted in a reduced required thickness of the asphalt surface course. In another study, Vineela et al. (2015) conducted a series of model-based analytical computations using a tire load of 30 and 65 kN with a tire pressure of 0.5 N/mm² and 0.7 N/mm², applying each load configuration to the center, corner, and edge of the pavement. Of all the cases tested, they found that edge loading created higher stress initiation compared to interior and corner loadings, and corner loading resulted in higher deformations compared to other loading cases.

2.5. Strength Characteristics of RCC and RCCPs

Numerous test methods have been used to assess the quality and soundness of both RCC pavement structures and their longitudinal joints. These methods yield various independent or interdependent parameters that provide valuable insights into the characteristics of RCC pavements. The following sections provide a concise overview of the current state of knowledge in this field and explore potential avenues for our study.

2.5.1. Compressive Strength of RCC

A report on RCC called ACI 325.10R (ACI, 1995) recommends a minimum compressive strength of 27.6 MPa for RCC to be used in pavements. However, according to

the Illinois Department of Transportation (IDOT, 2014), there is a requirement for a minimum compressive strength of no less than 24 MPa after seven days of curing. Harrington et al. (2010) state that although RCC can be produced with much higher compressive strength, the typically acceptable range for hardened RCC is between 28 and 41 MPa. The American Concrete Institute Committee 201 (ACI, 2008) emphasized the desirability of higher compressive strength to enhance resistance to potential abrasion distresses. This is because abrasion resistance in concrete is closely linked to its compressive strength, making greater compressive strength crucial for achieving improved resistance to abrasion distresses.

2.5.2. Flexural Strength of RCC

The flexural strength of concrete is a crucial parameter that epitomizes the pavement's fatigue criteria, a common corollary to the repetitive loads due to heavy traffic. Although the center-point test of ASTM C293 (2016), the third-point test of ASTM C78 (2018), and the four-point test are used to measure the concrete's flexural strength, the third-point test is the most widespread test for measuring the flexural modulus of pavement elements. During the test, a load is incrementally applied to the sample through the controlled and gradual downward displacement of the bearing plate. Upon failure, the peak load is recorded, and the flexural strength is calculated using the formula, $f_r = PL/bh^2$, where L represents the flexural span between supports, h is the height of the concrete sample, and b is the width of the concrete sample. As with any other mechanical strength test, the loading rate is critical to ensure measurement accuracy throughout the experiment and to protect the specimen from excessive or shock loading. The loading rate is calculated using the equation $r = Sbd^2/L$, where r is the loading rate in N/min, S is the rate of increase in the maximum stress on the tension face in MPa/min, b represents the average width of the specimen as oriented for testing in mm, d is the average depth of the specimen as oriented for testing in mm, L is the average depth of the specimen as oriented for testing in mm. Typically, the loading rate of the bending test falls within the range of 0.9 to 1.2 MPa/min. Harrington et al. (2010) noted that the typical flexural strength of hardened RCC ranges from 3.5 to 7 MPa. In general, the flexural strength of RCC can be expressed as 12.5%–17% of its compressive strength. This means that the ratio of flexural strength to compressive strength in RCC is approximately

0.15, slightly higher than that of conventional concrete (0.10 to 0.12) (Khayat & Libre, 2014).

Meanwhile, the flexural strength (or modulus of rupture) denotes the maximum stress the material can withstand without breaking under bending. The flexural modulus is another descriptive parameter widely utilized in pavement design and testing. Flexural modulus refers to the material's stiffness in bending, representing the slope of the linear portion of the stress-strain curve. Since the flexural modulus of elasticity depends on the section's deflection, it can be deduced from the deflection test conducted on a loaded beam element supported at the left and right tips and loaded in the middle of the full span. The flexural modulus can approximately be calculated using the relation: $E = (PL^3)/48I\Delta$, where Δ is the mid-span deflection due to the load (P), L is the full span length, and I is the moment of inertia. It is commonly believed that an empirical relationship exists between compressive strength and flexural strength. ACI 325 (ACI, 1995) provides a basic formula similar to that of Tayabji & Okamoto (1987) for approximate conversion. In the original paper of Tayabji & Okamoto (1987), several project-specific correlations were developed, and the basic relation type was established, $f_r = C\sqrt{f_c}$. Where; f_r is the flexural strength of concrete in psi, C is an empirical constant, typically a multiplication factor between 8.0 and 12.0, f_c is the compressive strength of concrete in psi (1 MPa = 145.03773773 psi). Chhorn et al. (2018) developed a new conversion relationship between compressive and flexural strength of RCC, using the relation of $f_r = 0.678f_c^{0.605}$. This empirical flexural strength relation takes into account the number of curing days, which has a significant impact on both compressive and flexural strength. The same study also proposed an alternative relation, $f_r = 1.143f_c^{0.369}d^{0.11}$, to consider the effects of curing days in the estimations.

Shear strength parameters of RCC are fundamental in the joint zones since they influence the efficiency of the concrete-to-concrete bond (Xie et al., 2012). Studies by Hansen & Reinhardt (1991), Hansen et al. (1999), ACI (1999), and Choi & Hansen (2005) suggested typical strength parameters for RCC. According to studies above, these parameters include cohesion values between 830 and 4,140 kPa for RCC mass, 480 to 4,140 kPa for mortared RCC lift joints, 0 to 4,140 kPa for unmortared RCC lift joints, and friction angles between 35 and 70° for various RCC mixes.

2.5.3. Splitting Tensile Strength of RCC

The splitting tensile (or split-cylinder) test indirectly assesses concrete's tensile strength by splitting a cylindrical specimen into two halves under a diametrically applied load. As in the case of ordinary concrete, the splitting tensile strength for standardized cylindrical RCC specimens is defined by ASTM C496 (2017). The splitting tensile strength of the specimen is calculated using a simple formula ($T = 2P/\pi ld$), where T represents the splitting tensile strength in MPa, P is the maximum applied load indicated by the testing machine in Newtons, l is the length of the specimen in mm, and, d is the diameter of the sample in mm.

The splitting tensile test is commonly used in pavement engineering to simulate the mechanical interaction and physical coupling between the base and surface courses. It helps assess whether potential deformation and cracking remain within allowable limits. Nevertheless, the splitting tensile test is less frequently performed for pavements. Given that the nature of the three-point bending test comes from the philosophy of bending the beam samples, the flexural test is mainly used to delineate the tensile strength in pavements (Khayat & Libre, 2014). Hansen & Reinhardt (1991) noted that the splitting tensile strength of RCC typically falls within the range of 7% to 13% of the compressive strength, with an average value of 10%. Considering the demanding challenges of conducting a direct tensile test on concrete samples, the splitting tensile test is often preferred, albeit with the caveat that it yields slightly higher strength values than direct tensile tests. The loading rate is crucial for accurate measurements during the test, and ASTM C496 (2017) provides guidelines. Hence, the loading rate should remain within a specific, fixed range (0.7 to 1.4 MPa/min) until the point of failure to prevent the specimen from encountering excessive, insufficient, or shock loading. A useful conversion relation between compressive and splitting tensile strength of RCC was put forward by Chhorn et al. (2018) proposed a useful conversion formula between test-driven compressive and empirical splitting tensile strength of RCC, represented as $f_{sp} = 0.47f_c^{0.511}$, where f_{sp} is the splitting tensile strength of RCC in MPa, f_c is the compressive strength of RCC in MPa. Interestingly, Chhorn et al. (2018) found that curing time (age of the concrete) had no significant effect on the splitting tensile strength of RCC, which somewhat contrasts with initial engineering assumptions and their observations related to flexural strength. All the same, it is strongly assumed that the characteristic splitting tensile strength of RCC is slightly lower than the flexural strength.

2.5.4. Uniaxial Tensile Strength of RCC

To the best of my knowledge, a code-driven and code-compliant direct tension test designed explicitly for RCC samples has not been developed to date. Conducting a direct tensile test on concrete samples is known to be challenging, and research on tensile strength in RCC technology (Malkawi & Mutasher, 2003; Olivares et al., 2003; Ribeiro et al., 2001) is relatively limited. However, there are established uniaxial tension test methods for bituminous mixtures, as regulated by BS EN 12697-46 (BSI, 2020), and for intact rock core specimens, as outlined by ASTM D2936-20 (2020).

Presumably drawing inspiration from these standards, Malkawi & Mutasher (2003) put forward a similar procedure for RCC samples and introduced two correlation relations: $f_t = 0.15f_c^{0.744}$ and $f_t = 0.722f_{sp}^{0.9}$, where f_t represents the direct tensile strength, f_c is the compressive strength, and f_{sp} is the splitting tensile strength (all values in MPa). These correlations were designed to connect the direct tensile strength of RCC to its compression and splitting strength, with a specific focus on dam construction. Malkawi & Mutasher (2003) conducted a study to determine the direct tensile strength of longitudinal joints in RCC and establish its relationships with compressive and splitting tensile strength. This research emphasizes the potential of using direct uniaxial tension stress tests to model longitudinal joints. It also suggests the development of correlations between well-known mechanical properties for the same purpose.

2.5.5. RCC-to-RCC Bonding Strength

RCC pavements are occasionally constructed in multiple layers when a single-layer thickness exceeds the practical capabilities of the paver or when a single lift's applicable thickness cannot meet the strength requirements. Particularly in two-lift paving, the degree of bonding between the fresh and hardened layers at the concrete-to-concrete interfaces becomes a critical factor. This factor determines whether the pavement behaves as a monolithic (one-piece) structure or a segmented one composed of partially bonded layers. The bonding between two distinct RCC lifts, whether composed of the same concrete or not, is essential for the resilience of longitudinal joint zones. The interconnected adjacent surfaces (bonding area) may be vertical, horizontal, or elongated. Concrete layers can be cast either stacked (two-layer RCC) or side by side (longitudinal or transverse joints). While the

split tensile test is frequently used to evaluate strength between concrete lifts, it may not comprehensively capture the desired parameter. Hence, there is an evident need to employ new testing techniques for assessing the bonding characteristics of RCC-to-RCC.

In this study and practical terms, bond strength typically refers to the strength between two in-contact surfaces of the RCCs cast in different lifts. Despite some disparities in using RCC material in pavements and mass concrete works, ACI (2000) emphasizes the significance of RCC being free from segregation, cast through tightly bonded joints, and compacted to the desired density. The latter should always be as close as possible to the theoretical maximum density. The American Concrete Institute (ACI, 1995) recommends directly using cement slurry or grout, including sand and cement, as needed. Liu et al. (2018) investigated the interlayer shear strength characteristics of RCC by testing various interlayer treatment methods. They emphasized that the increasing time intervals between compactions significantly influence overall strength performance, especially the delay after the final set. In alignment with the findings of Kreuer (2006), they highlighted that achieving sufficient bond strength is the paramount and foremost benchmark for multi-lift RCC pavements, best accomplished by placing subsequent lifts within an hour.

The bond strength between concrete layers can be significantly improved using bonding-enhancing materials, such as cement slurry, mortar, and chemical resins. When contemplating RCC for dam construction, Xie et al. (2012) strongly recommend using mortared joints to enhance flexural strength in longitudinal joint zones. Alternatively, in studies focused on HMA by Wang & Oeser (2016), it was noted that the treatment of the vertical interface of cold longitudinal joints might be necessary to enhance joint quality. Liu et al. (2018) employed the Coulomb failure criterion ($\tau = f\sigma + c$, where the c is the cohesive stress of the shear surface, f is the friction coefficient, σ is the normal stress to the shear surface and τ denotes the shear stress) to assess the behavior of interlayer shear strength variations over time. Their observations revealed that, while the friction coefficient remained relatively stable, especially at peak stress ranges, the cohesive stress tended to decrease with increasing time intervals. Furthermore, they noted that the use of expansion agents and nanoscale silica (SiO_2) demonstrated more favorable performance compared to ordinary mortar alone. Concerning the bond strength in two-layer RCC pavements, Mohammed et al. (2019) found that wet-on-wet placements exhibited superior durability performance than wet-on-dry placements. Their study highlighted that the use of cement slurry significantly improved joint strength. Furthermore, they observed a noticeable reduction in bond strength

when delays were introduced between the casting of subsequent RCC layers. As of my knowledge, while there are no universally accepted benchmark values for the precise measurement of bond shear strength in RCC, Rollings (1988a) cautiously concluded, based on prior studies, that a bond (shear) strength of 1.38 MPa could be considered sufficient for fully-bonded RCC overlays.

Several tests have been proposed to date to measure the relative bond strength between concrete lifts (ACI, 2006; Tran et al., 2015) and have been utilized in different studies (Harris et al., 2011; Momayez et al., 2005; Yazdi et al., 2020; Zanotti et al., 2014; Zhang et al., 2020) in diverse ways. Approximately five commonly implemented tests include the punching shear test, slant shear test, bi-surfaces shear test, pull-off test, and direct shear test. Moreover, given the challenges of replication in typical laboratory environments, tensile bond testing and push-off testing (Austin et al., 1995; Courard et al., 2014; Tran et al., 2015) have been infrequently used to define bonding characteristics. The most common test setups for determining concrete-to-concrete bonding strength are illustrated in Figure 34.

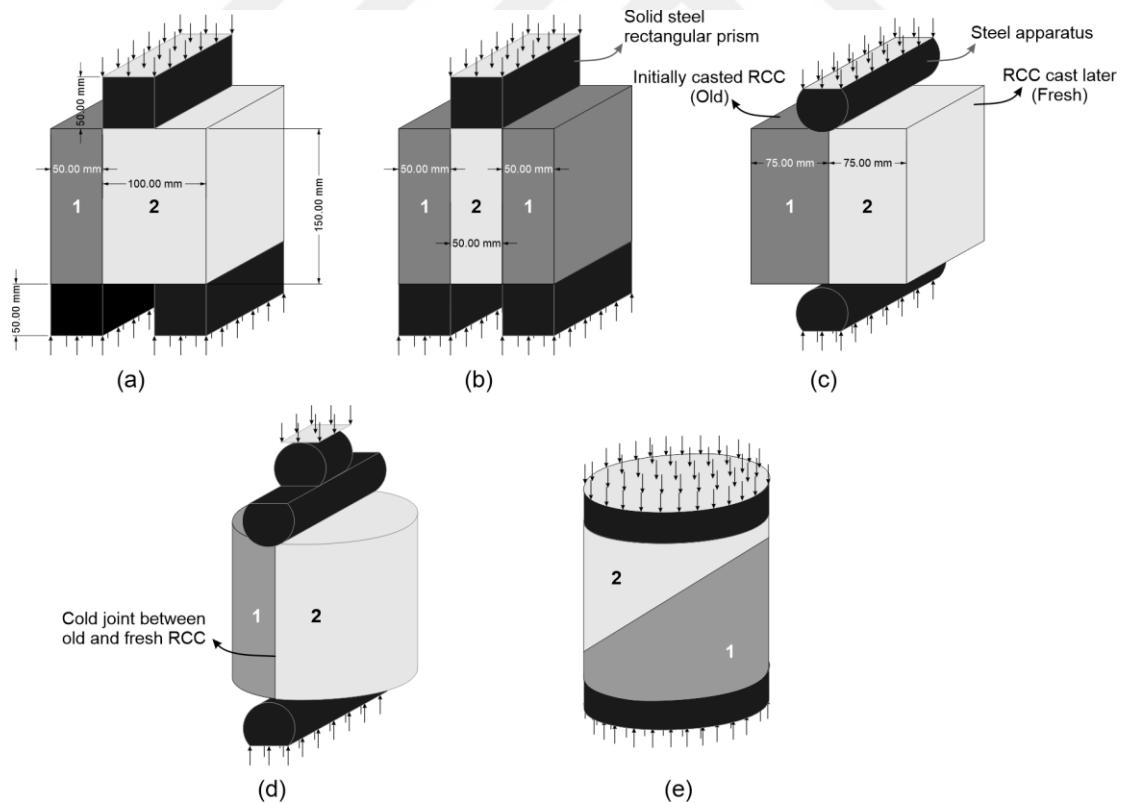


Figure 34. Test modes for determining the relative bond strength between hardened RCC lifts that create longitudinal joints: (a) Bi-surface shear test, (b) Double shear test, (c) Split prism test, (d) Splitting cylinder test, and (e) Slant shear test

For cubic samples, the bi-surface test (Figure 34a) can be used to determine the interface bond strength, similar to the one applied by Valikhani et al. (2020). As indicated by its name, resistant forces stemming from relative bonding are measured against increasing pressures applied to three quadrangular solid prisms during the bi-surface test. The apparatuses facilitating load application onto the sample in the split prism test depicted in Figure 34b-c can be used. However, Figure 34c draws inspiration from ASTM C1245 (2012). Notably, the test procedure illustrated in Figure 34c is inactive, having been withdrawn by ASTM in 2019 without an alternative test procedure being proposed. To the best of my knowledge, although the upper and lower load transfer elements are rigid enough not to bend, Figure 34b, which closely resembles the conventional Brazilian test, may be preferred over Figure 34c. A thorough examination of the standard guided the development of a loading apparatus aligned with the standard's objectives. This method involves the transfer of a linear load over a very short and infinitesimally narrow area to the full length of the element. While providing detailed explanations for all the tests mentioned above, including their controversial initial development dates, might not be practical for readers, I believe it might not be handy for readers if all the tests are explained in exhaustive detail. Tran et al. (2015) showed that the bonding strength against shear between new and old concrete lifts undergoes significant changes with variations in testing methods. They specifically highlighted that the bi-surface shear test emerged as the most suitable for determining shear bond strength. Geissert et al. (1999) demonstrated that the results of the classical split tensile test (ASTM C496, 2017) are comparable to the split prism test. The slant shear test (Figure 34d) has notably garnered considerable interest over the years. To the best of my knowledge, its initial form was conceived by Kreigh (1976) for use in cylindrical samples and later modified by Tabor (1978) for testing prisms. In the bonding tests, geometric factors come into play when aiming to convert compression force into shear and normal forces on the bonding plane. Austin et al. (1999) pointed out that despite the stereotyped inclination of 30 degrees to the vertical plane in most tests, the orientation of the bond surface has received little attention despite its significant impact on failure style and load. The bond angle, expressing the slope at which the minimum stress impairs the bond, is crucial. The failure load acting on the sample directly depends on the angle between the bonded adjacent planes, known as the bond angle (α). In other words, α is the slope angle between adjacent interfaces forming the longitudinal joint. Critical alpha values, represented by α_{cr} , similar to α but varying with surface roughness, have garnered significant attention.

Several studies highlight the significance of α_{cr} . Robins & Austin (1995) demonstrated the negative effects of a smooth bonding surface on strength. Tran et al. (2015) indicated that while the level of roughness significantly affects bond strength, increasing roughness beyond a certain point becomes futile. Figure 35 visually represents the bond plane angle against failure stress to adhesion alongside classified surface appearances. By utilizing Figure 35, a crucial conclusion can be drawn. For instance, a 10-degree increment in the inclination of the bond plane results in a significant increase in normalized stress with adhesion strength (σ_0/c). This indicates that as the bond surface becomes rougher, the critical bond angle (α_{cr}) tends to decrease substantially.

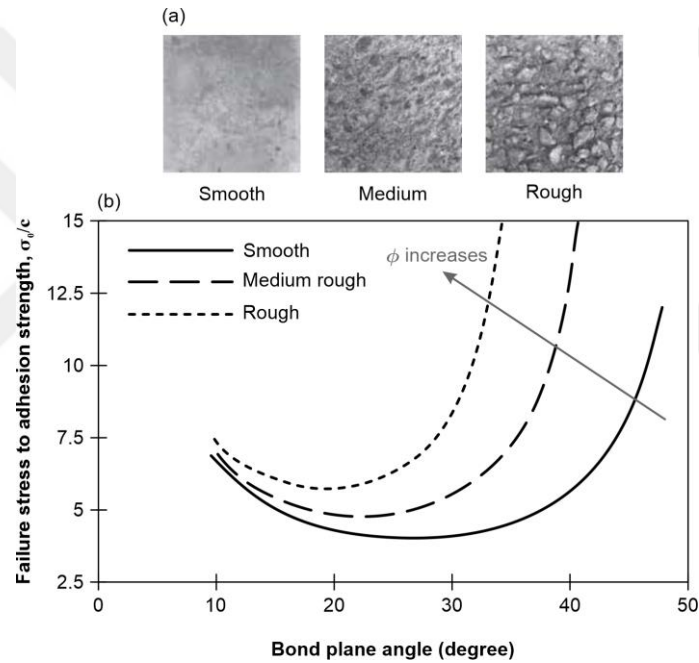


Figure 35. (a) Different bond surface appearances that impact the bonding strength (adapted from Tran et al. (2015)), (b) Relation between bond plane angle against failure stress to adhesion strength (adapted from Austin et al. (1999))

By elucidating its most general form, the slant shear test could provide insight into understanding the mechanisms of other tests. The most comprehensive representation of normal and shear stress on the bonded adjacent faces of the slant sample is illustrated in Figure 36. Well-known constitutive relations, easily derived using Mohr's Circle, are expressed in equation (15) as follows.

$$\tau_n = \frac{1}{2}\sigma_0 \sin(2\alpha), \sigma_n = \sigma_0 \sin^2 \alpha, \tau_n = c + \mu\sigma_n = c + \tan \phi \sigma_n \quad (15)$$

where σ_0 is the stress required to failure through the bonding plane, σ_n is the normal stress and τ_n is the shear stress, $\mu = \tan \phi$ is the coefficient of friction, adhesion is represented by c , and friction angle between two bodies is represented by ϕ . According to Mohr-Coulomb theory, σ_0 can be calculated using the angle of slanted plane as $\sigma_0 = c[\cot \alpha + \tan(\phi + \alpha)]$.

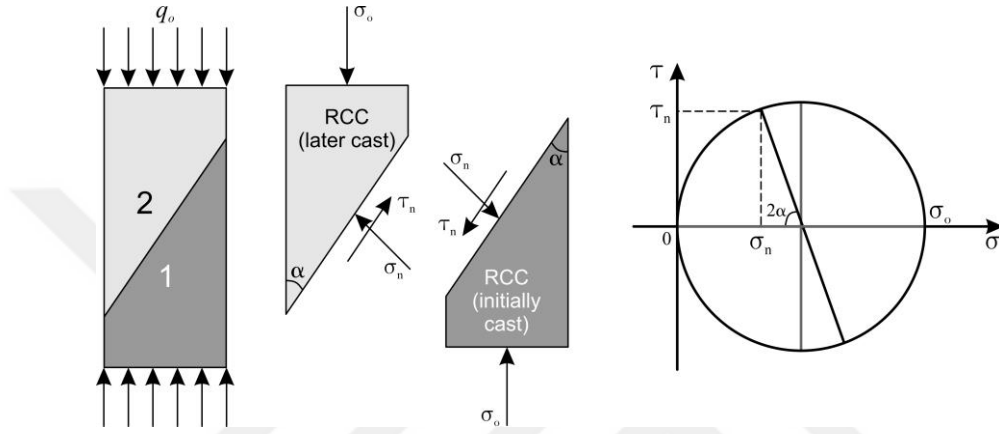


Figure 36. Free-body diagram and Mohr's circle from the slant shear test

Austin et al. (1999) introduced a method to define α_{cr} , determining the bonding angle based on the minimal load that results in bond failure. In brief, using the same terminology, α_{cr} can be expressed in equation (16), directly influenced by surface roughness, which in turn influences the friction angle.

$$\alpha_{cr} = 45 - \phi/2, \sigma_{min} = 2c \tan(45 + \phi/2), \tau_{cr} = c(1 + \sin \phi) \quad (16)$$

where σ_{min} and τ_{min} are the stresses relating to α_{cr} .

In principle, the standardized point load test method for the determination of RCC bond strength (ASTM C1245, 2012) recommends calculating the relative bond strength using this simple relation: $f_{tb} = P/D^2$, where f_{tb} is the relative bond strength (MPa), P is the maximum applied load that yields the bond surface (N), D is the specimen diameter at the bond surface (averaged), (mm).

While no specific bond strength scale has as yet been developed for RCC, the American Concrete Institute (ACI, 2006) has provided typically allowable strength limits for examining interface bond strength in concrete repair, as outlined in Table 1.

Table 1. Allowable minimum bond strength limits in concrete-to-concrete bonding (ACI, 2006)

Testing mode	Bonding strength (MPa)		
	After 1-day	After 7-day	After 14-day
Direct tensile test	0.5—1	1—1.7	1.7—2.1
Slant shear test	2.8—6.9	6.9—12	14—21

From another perspective, Austin et al. (1999) classified concrete interfaces as smooth, medium-rough, and rough based on their friction coefficients, with boundaries set at 0.75, 1.0, and 1.25, respectively. Compiled from various studies on concrete-concrete interfaces in the literature, Table 2 demonstrates the apparent effect of surface roughness on friction coefficients.

Table 2. Coefficients of friction for concrete-to-concrete surfaces as reported in various studies

References	Surface Roughness	$\mu^{\dagger}$
Franke (1986)	Smooth	0.7
Alexander et al. (1968)	Smooth	0.75—0.87
Robins & Austin (1995)	Smooth	0.8
	Smooth	1.42
Hussein et al. (2016)	Smooth	0.47*
	Mid-to-rough	1.09*
	Rough	1.44*
Tayeh et al. (2013)	Smooth	1.20*
ACI Comm (1991)	Rough	1.0
Alexander et al. (1968) and Robins & Austin (1995)	Rough	1.1 —1.4
$\dagger$ Values for interfaces subjected to either wire-brushed or sandblasted treatments are not included in the tabular values. * Values pertaining to high-strength concrete bonding		

However, the observation that similar friction coefficients are noted across different surface properties underscores the importance of the bonding angle and the specific test applied. Failure modes for yielded samples do not commonly adhere to a fixed pattern, and according to the results of Valikhani et al. (2020), similar samples can exhibit different

failure modes, classified as cohesive, adhesive, and mixed-type failures. At the heart of the matter, a naturally arising question is whether and to what extent fiber inclusion into fresh RCC can enhance interface bonding strength. There is an apparent scarcity of research on this topic. Drawing on the findings of Zanotti et al. (2014) regarding bond strength in cementitious fiber-reinforced repairs, augmented concrete intended for use in concrete repair with polyvinyl alcohol (PVA) fibers was beneficial in enhancing interfacial adhesion. However, no significant improvement in durability was observed against wet-dry effects.

2.5.6. Modulus of Elasticity and Poisson's Ratio of RCC

Typically, the Poisson's ratio for RCC ranges between 0.17 and 0.22, which is quite similar to that of ordinary (vibrated) concrete, whose Poisson's ratio typically ranges from 0.20 to 0.25 (Andriolo, 1998). The modulus of elasticity plays a crucial role in defining the rigidity of longitudinal joints in any pavement, constituting a benchmark for the resilience of the pavement body. Interestingly, in past studies, the modulus of elasticity was not consistently measured in samples from actual RCCP projects (Aghaeipour & Madhkhan, 2019). However, various tests conducted on RCC samples suggest that RCC's modulus of elasticity is either similar or somewhat higher than that of conventional concrete with similar cement content (Tayabji & Okamoto, 1987; Tayabji et al., 1995). Therefore, factors influencing the elasticity modulus of conventional concrete are also applicable to RCC. Li et al. (2002) signified that the modulus of elasticity, particularly determined through uniaxial tension tests, is rather related to the square root of the compressive strength and a ratio derived from dividing the maximum size of the aggregate with the dimension of the specimen, which is valid for any failure modes of the concrete either RCC matrix or interface zone.

In cases where actual elasticity modulus data is unavailable but judgment is required, empirical conversions can be used. For instance, the AASHTO LRFD Bridge Design Specifications code (AASHTO, 2012) suggests a conversion equation, $E_c = 33,000w_c^{3/2}\sqrt{f_c}$, for use in conventional concrete. In this equation, E_c is the modulus of elasticity in ksi, w_c is the unit weight of concrete in kcf (assuming w_c to be 0.145 kcf for normal-weight concrete), and f_c is compressive strength of concrete in ksi (1 ksi = 6.895 MPa, kcf = 16.02 Metric Ton). Furthermore, understanding the time-dependent behavior of RCC's modulus of elasticity, especially during the early stages, holds significant importance

for pavement engineers as it directly impacts the time required to put the road into service. Conrad et al. (2003) extensively studied the temporal variation in the modulus of elasticity of RCC and developed empirical prediction relations to estimate time-dependent strength development. These prediction relations for the modulus of time-dependent elasticity of RCC are expressed in relations (17-19).

$$E(t) = E_{c,28} \left(\frac{t}{17.6 + 0.37t} \right)^{1/7.35} \quad (17)$$

$$E(t) = E_{c,365} \left(\frac{t}{10.12 + 1.096t} \right)^{1/0.622} \quad (18)$$

$$E(t) = E_{c,365} (e^{-5-t^{-0.63}}) \quad (19)$$

Where t represents the elapsed time after casting, age, day, $E(t)$ signifies the age-dependent modulus of elasticity, $E_{c,28}$ denotes the modulus of elasticity at 28 days, $E_{c,365}$ denotes the modulus of elasticity at 365 days. Figure 37 illustrates a representative example of time-dependent strength development in RCC.

In conclusion, the smaller panels in Figure 37, zoomed in on the initial part of the larger panels, show that the initial two-day curing period significantly influences the strength development of RCC, with the standard deviation margins beginning to rise immediately after this period. More so, Figure 37b also highlights that the two relations established for 365-day strength development predict values that deviate from the actual trend. Additionally, the proposed relations for 28-day strength development, while eventually reaching the expected strength value, exhibit a slower approach starting from values higher than the actual ones.

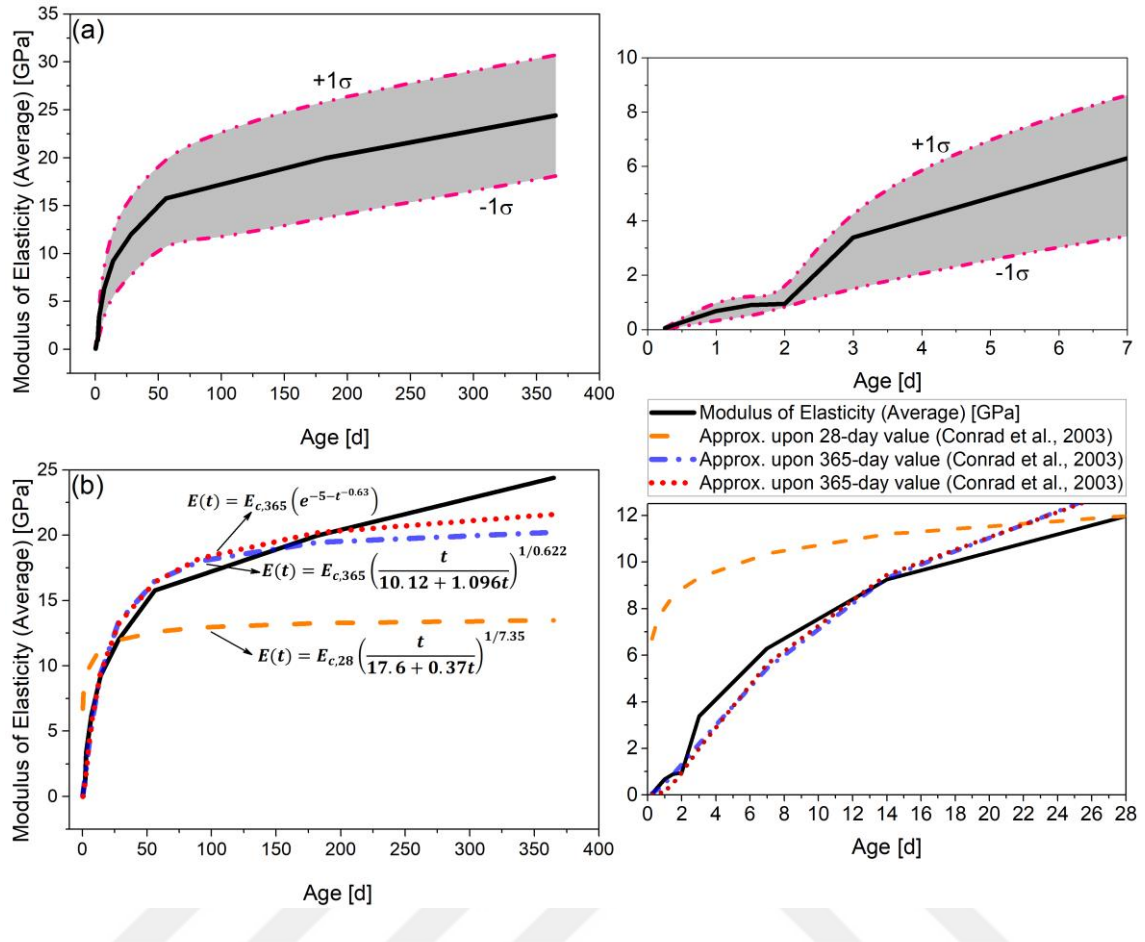


Figure 37. (a) Time-dependent strength development of RCC, (b) Trends of empirical estimation (raw data sourced from Conrad et al. (2003) and reanalyzed)

2.5.7. Non-Destructive Testing Methods for RCC Pavements

As its name suggests, non-destructive testing (NDT) means the sample of the material being tested does not get exposed to damage-generated exposure and, thereby, is not damaged during the testing process. The NDTs have gained significant prominence in the evaluation and quality assurance of RCC pavements. These advanced tools, stemming from the latest developments in concrete technology, are pivotal in ensuring the structural integrity and resilience of RCC pavements. The adoption of NDT techniques in RCCP has garnered growing interest due to their non-intrusive and non-disruptive nature. The NDT techniques are transforming how we assess the quality and condition of RCC pavements and will become even more crucial than today as technology advances.

Ground-penetrating radar (GPR), ultrasound pulse-echo, tomography, infrared thermography (IRT), and electrical resistivity, all of which belong to the newer generation of non-destructive testing and evaluation techniques, are frequently employed in concrete technology (Lin et al., 2018a). Given their non-intrusive and non-disruptive nature, interest in these methods is steadily growing within pavement technology. Additionally, a longstanding concern of pavement engineers has been the effectiveness of GPR in determining pavement layer thickness (Haas & Hudson, 2015). Maser & Puccinelli (2009) addressed this issue and demonstrated that GPR is reliable for measuring pavement thickness, typically within a deviation of around $\pm 8\%$. They also noted that calibrating the GPR tool before use can improve its accuracy. Joshaghani & Zollinger (2021) introduced a novel curing protocol for concrete pavement construction. They developed a new technique known as "percent within limits" (PWL) to assess curing soundness using ground-penetrating radar. In a pivotal study, Lin et al. (2018a) evaluated nine methods for the non-destructive assessment of concrete bridge decks with overlays. Among the nine NDT methods tested on seven overlay specimens, ground-penetrating radar emerged as significantly effective in detecting target defects in concrete specimens, whether bonded or debonded. While sounding and impulse response proved practical for detecting overlay debonding, they were less effective in identifying defects beneath bonded overlays. Similarly, electrical resistivity was found to be ineffective for overlays, likely due to the electrical insulation properties of the overlays. Ultrasonic imaging applications have long faced constraints imposed by fixed-velocity concrete structures. Nevertheless, Lin et al. (2018b) addressed these challenges and implemented ultrasonic imaging techniques to assess multi-layer concrete structures. This breakthrough is crucial as it expands the potential for onsite examination of single or multilayered concrete pavements. In another study conducted by Janků et al. (2019), delaminations in concrete bridges were investigated using infrared thermography, ground-penetrating radar, and ultrasonic pulse-echo methods. Among the methods tested, IRT was found to be the most suitable, considering the parameters and physical structure of the bridge. Its primary advantage lies in its ability to swiftly scan the entire bridge without requiring direct access to the structure. Consequently, it can inspect all areas of the bridge, even those above busy roads, without disrupting traffic. The only limitation and drawback of IRT is that measurements need to be conducted in suitable weather conditions to ensure a strong enough apparent temperature contrast for defect identification.

3. RESEARCH METHODOLOGY

3.1. Examined Alternative Longitudinal Joint Models for RCCPs

As previously discussed, twelve techniques have been utilized thus far for constructing longitudinal joints in HMA pavements, though many lack authenticity. While some techniques bear specific names, they can essentially be viewed as modified iterations of fundamental methods. The techniques employed in RCC pavements have predominantly been adapted from those already utilized in HMA pavements. This is due to the similarity in paving techniques and compaction methods between RCC and HMA. Thus, the construction techniques for longitudinal joints in RCC and HMA pavement extend far beyond mere coincidental resemblances. Occasionally, akin to HMA pavements, the free end of the RCC mat is left as laid by the paver, with or without pre- or post-shaping. The compaction process is determined based on the shaping technique employed. Shaping operations can be divided into two categories: those conducted when the joint is in a fresh state and those carried out when it is in a cold state. The broad discussion of fresh joints falls outside the scope of this thesis.

Arbitrarily shaped longitudinal joints formed by the laydown machine are rarely utilized in practice due to the challenge of predicting their mechanical behavior through scientific means. Consequently, models for arbitrarily shaped cold joints were not explored in this study. The post-shaping technique, simply referred to as "saw-cut joints", is considered the most rudimentary, time-intensive, and costly method, albeit occasionally preferred for creating right-angle butt-shaped longitudinal joints. Eventually, saw-cut joints were not employed or tested in this study. Regarding pre-shaping, the shaping process occurs either during the laying-down phase of the RCC or during compaction by the roller. The free edge of the mat is typically mechanically shaped using joint pressing tools known as joint makers. These joint-maker apparatuses can be mounted onto the paver screen or one of the roller compactor's drums. The primary aim is to define a cold longitudinal construction technique applicable for future full-scale applications and to illuminate unresolved issues. Figure 38 outlines the cold longitudinal construction techniques examined in this study, which are thought to be applicable to RCC pavement constructions in general.

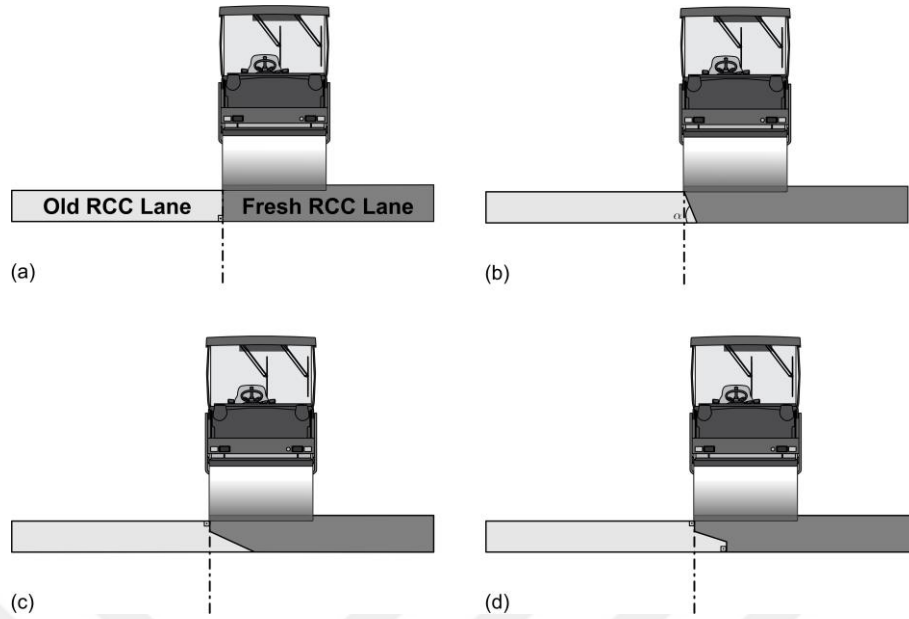


Figure 38. Cold Longitudinal Construction Techniques Examined in this Study: (a) Butt joint model (at a right angle), (b) Wedge joint models (at varied angles of 60°, 45°, 30°), and (c) Notched-wedge model, and (d) Double-notched wedge model

In this study, the preferences for slope angles were deliberate and not arbitrary. Examination of catalogs from edge pressing and restraining device manufacturers, such as HAMM (2017), revealed that ready-to-use tools are designed to create tapers at specific angles, as mentioned earlier. Additionally, it is impractical in RCC pavements to create longitudinal joints with relatively small inclination angles in very thin layers, a practice common in HMA pavements. Observations from various local or national RCC projects in Türkiye indicate that when small angles are used in RCC pavements, the horizontal component of the joint extends significantly more than the vertical, leading to the segregation of fresh concrete and potential breakage under roller drums. Thus, this study aims to test nearly all possible slope angles achievable using joint-maker devices.

The employed techniques are categorized based on the slope angle (90°, 60°, 45°, 30°) of the tapered adjacent surfaces, which create the visible longitudinal joint along the pavement axis. Additionally, notched wedge and double-notched wedge joints were modeled and employed for the first time in this study. The decision to test notched wedge joints is grounded in their similarity to counterparts long used in HMA pavements. Specifically, as RCC exhibits zero-slump characteristics, notched wedges if initially compacted with a robust screed, can remain stable and undeformed until rolling is performed. Moreover,

secondary shaping can be achieved using a joint-maker device mounted onto the roller drum, either maintaining or initially setting the shape of the notched-wedge joints.

Namely, six different shaping techniques and four adjacent joint interface treatment methods were employed, and thus, 24 different alternative longitudinal joint construction techniques were tested in this study. The beam elements bearing examined longitudinal joint models and interface reinforcement treatment methods employed in this study are shown in Figure 39.

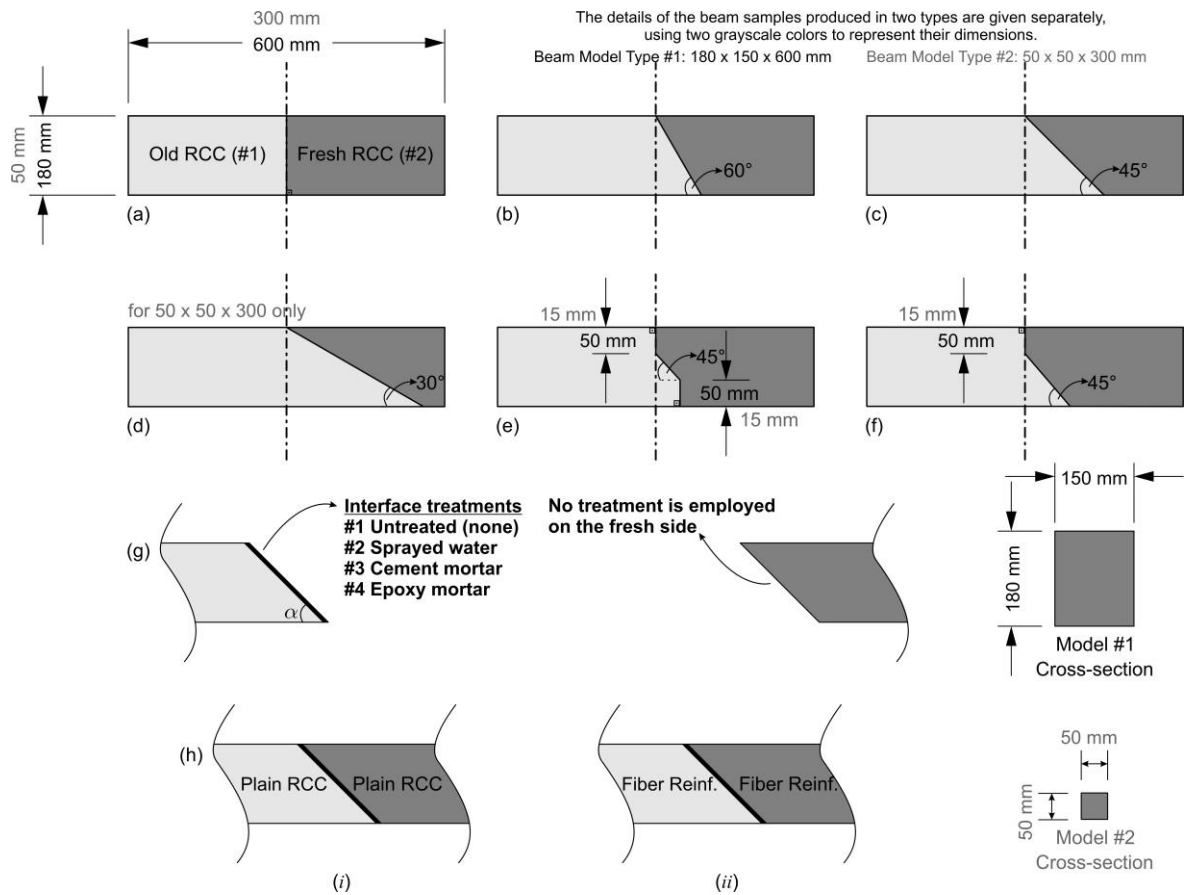


Figure 39. Beam samples with modeled longitudinal joints: (a-f) Beam sample models with different longitudinal joint types, (g) Adjacent interface treatment alternatives, (h) Types of concrete joined together and cross-section of the beams

3.2. Laboratory Test Program

The test program adopted in this thesis primarily comprises several laboratory tests aimed at examining key properties of RCC samples with different longitudinal joints. The principal goal of the experimental investigation is to gain a deep understanding of behavioral

patterns and subsequently address the impact of possible variations in RCC longitudinal joint construction techniques surveyed. Each part of the test program and the order of tests were mainly designed to focus on the problematic issues contemplated beforehand. Regarding mechanical and material properties, although the number of parameters affecting the experiments was relatively high, the tests were performed solely to observe variations in critical parameters. In this sense, the remaining parameters were circumscribed to obtain refined outcomes with as little uncertainty as possible. Several digital and mechanical recording and screening techniques were employed in some tests, and then the raw data were cross-correlated and compared using various statistical sensitivity techniques. Furthermore, an interface-capturing approach via video recording was utilized to track variations in the position of crack traces and surfaces. Table 3 provides brief information about the sampling and testing plan.

Table 3. The compendium of sampling and testing programs employed in this study

Test	Pertinent standards to be complied with	Age	Sample size	# samples
Compressive Strength	ASTM C39	7, and 28 days	150×300 mm cylinders 150×150 mm cubes	3 per model
Flexural Strength	ASTM C78	7, and 28 days	50×50×300 mm 150×180×600 mm	3 per model
Relative Bond Strength ¹	Split prism test (Inspired by ASTM C1245 [†])	28 days	150×150 mm cubes	3 per model
Relative Bond Strength ²	Bi-surface shear test	28 days	150×150 mm cubes	3 per model
Consistency of Concrete (Slump Cone Test)	ASTM C143	Zero/Fresh	220×230×310 mm cones	3 per mix
Ultrasonic Pulse Velocity Testing	ASTM C597	28 days	All	Each
Density, Absorption, and Voids	ASTM C642	7, and 28 days	50×50×80 mm Prisms	3 per model
[†] ASTM C1245 is currently a withdrawn code. The split prism test was employed using a similar aim with what C1245 aimed at.				

Because the cement type and amount of all cementitious materials were kept fixed in this study, an investigation into adiabatic temperature rise due to heat hydration was not conducted. Rapid temperature fluctuations were mitigated by the existing heating and cooling system in the labs where the tests were conducted, ensuring a constant ambient

temperature throughout the tests. This approach eliminated the effects of ambient temperature changes on both RCC batches and tests to the greatest extent possible.

3.3. Materials and Sampling

This section outlines the materials and their properties used in the trial mixes to determine the optimal RCC material in the laboratory. The primary criterion for material selection was the use of readily available materials found in almost every ready-mixed concrete plant or concrete laboratory, ensuring reliable repeatability for potential future trials. In summary, the material selection process involved several sequential steps to ensure the creation of the best possible RCC material in the laboratory. The approach followed was consistency, a long-known foolproof method aimed at achieving proper consistency to make the fresh mixture better compactable. The targeted consistency ensures practical workability, which is essential for the entire RCC pavement construction process. The detailed procedures and materials used for each stage are outlined below.

3.3.1. Aggregates Used

The aggregates used in this study were sourced from the stockpile of a large concrete plant (Albayrak Concrete Corp. in Trabzon). Initially, the aggregates were subjected to several fundamental tests, including Los Angeles abrasion (ASTM C131, 2006), Sodium sulfate soundness (ASTM C88, 2018), Clay lumps and friable particles (ASTM C142, 2017), Lightweight particles in aggregates (ASTM C123, 2014), Potential Alkali Reactivity (ASTM C1260, 2021), Plasticity (No. 40 sieve) (ASTM D4318, 2017). The aggregates demonstrated excellent performance, meeting all relevant criteria and passing all tests, thus confirming their suitability for use.

It is widely acknowledged that the sand portion of concrete aggregates may contain very fine materials such as silt, clay, and/or stone powder in trace proportions. Washing is crucial in the production of crushed stone at plant sites. The Methylene Blue Index (MBI) test (AASHTO T 330, 2007; ASTM C837, 2019) was employed to determine the presence of allowable extra fine materials in the sand, indicated by an acceptable MBI level. Although this test can quantify the very fine material content, it does not identify the specific type of material. The amount of positively-charged ions, or cations, that clay can contain depends

on its cation exchange capacity. However, as the clay content in the very fine material increases, the amount of methylene blue absorbed by the clay, with its greater surface area and cation exchange capacity compared to sand, also increases, resulting in a higher measured methylene blue value. Generally, if the very fine material content in fine sand exceeds 3%, normatively recommended tests include the Methylene Blue test and other fine-aggregate-targeted tests (EN 12620, 2002).

Of the tested aggregates, the one with an MBI of 0.75 was selected for further work. This choice indicates that the fines are suitable for most concrete applications, as evidenced by the very low MBI value, suggesting high-quality very fine aggregates with minimal clay-like content. Images from the experiment to determine the Methylene Blue Index are provided in Figure 40.



Figure 40. Defining the MBI level of the fines in sand

In accordance with the recent regulation of Turkish Cement Manufacturers' Association (TCMB, 2018) and the Turkish General Technical Specification of RCC Pavements (CSB, 2020), the aggregate mixture for fresh RCC production in this study was created by blending at least three different aggregate groups, all of which were basaltic. Specifically, for all RCC mixtures used in both initial trials and the final mix, natural basaltic coarse aggregate was employed, conforming to ASTM C33 (2018) with a nominal maximum size of 13 mm, a specific gravity of 2.68, and 24-hour water absorption of 0.64%, as defined by ASTM C127-15 (2015). The fine aggregate used was basaltic angular sand with a specific gravity of 2.64 and 24-hour water absorption of 0.81%, determined according to ASTM C128-15 (2015).

Considering the various suggested aggregate gradation bands (ACI, 1995; IDOT, 2014; PCA, 2004; TCMB, 2018) and applying the 0.45-power gradation curves for aggregate

sizes of 13-mm and 19-mm, the entire ready-mixed aggregate batch underwent assessment by sieving in sequential sieves. Sieve analysis of both fine and coarse aggregates followed ASTM C136 (2014) standards, resulting in a suitable gradation curve. The aggregate classes employed in this study for RCC production are illustrated in Figure 41. Additionally, Figure 42 collectively presents the gradation of mixed aggregates ultimately used in RCC mixtures and several feasible grading curves suggested by different institutions.



Figure 41. Aggregate samples used in the production of fresh RCC

The primary mixing ratios for coarse aggregate and sand with fines were 41.68% and 58.32%, respectively (comprising 56.83% sand and 1.37% fines that pass #200 sieve). The aggregate gradation characteristics were assessed using a 0.45-power maximum density gradation plot. The final aggregate mix is considered fine-graded if the grading curve remains on or above the 0.45 power curve. Conversely, if the curve falls below the 0.45 power curve, the aggregate is defined as coarse-graded. Figure 42b shows that the final

aggregate mixture, with a sand-to-total aggregate ratio of 58.32, exhibits well- and coarse-graded characteristics.

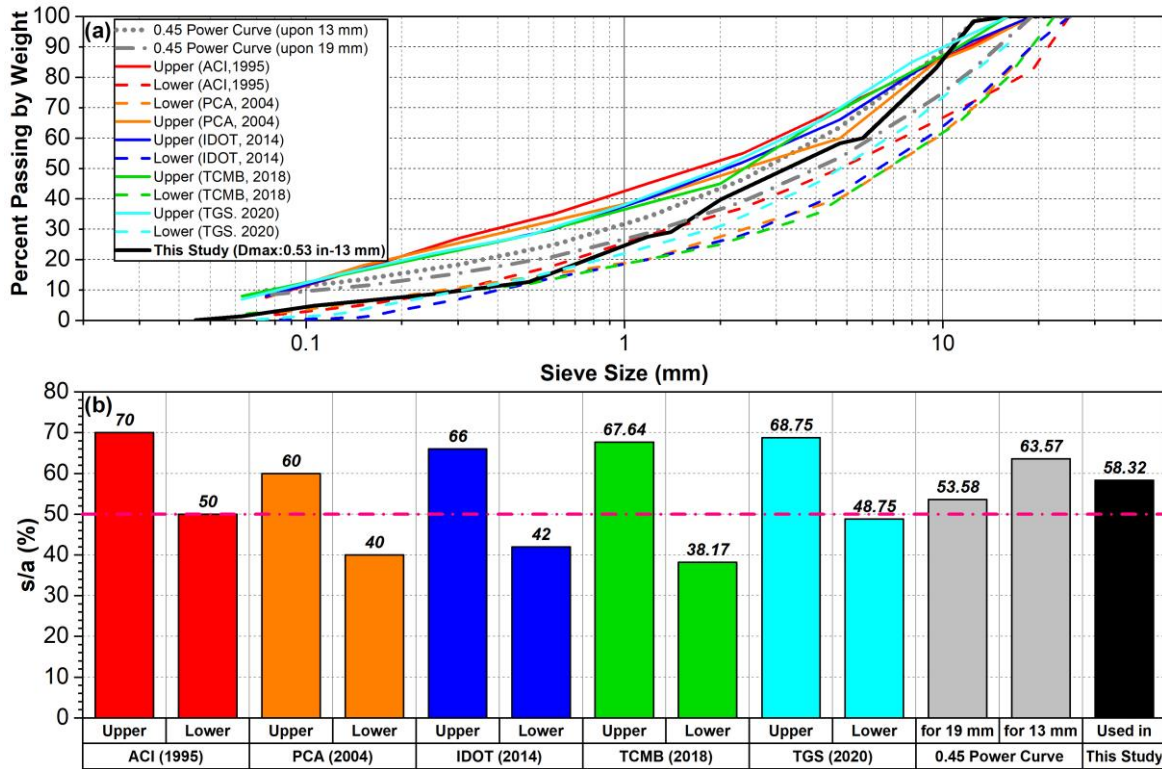


Figure 42. Comparison of grading curve used in RCC production with recommended limits for aggregate mixtures

The coefficient of uniformity ($C_u = D_{60}/D_{10} = 16.70$) and coefficient of curvature ($C_c = D_{30}^2/D_{10} D_{60} = 1.11$) were determined, and according to the unified soil classification system, the overall aggregate mixture falls within the limits of SW, defined as well-graded sands, gravelly sands, with little or no fines.

With a C_u value of 16.70, the particle size distribution of the aggregate mixture is relatively wide, indicating a broad range of particle sizes. A C_c value close to 1 indicates a well-graded aggregate with a smooth gradation curve, while values significantly higher or lower than 1.0 indicate poorly graded or gap-graded aggregates. The relatively smooth and well-graded particle size distribution curve is indicated by the C_c value of 1.11, close to 1. The fineness modulus (FM) of the fine aggregate was 3.07. While an FM of 3.07 falls within the well-graded range, it aligns well with the classification of SW (well-graded sand) regarding the particle size distribution.

3.3.2. Cement and Cementitious Materials Used

A single type of cement was consistently used throughout concrete production to mitigate the potential effects of cement-type variations on the RCC mixes. Commercially available Portland Cement CEM I 42.5 R (Portland Cement - Type I), meeting ASTM C150 (2019) standards and manufactured at Aşkale Cement Factory in Gümüşhane, was employed in all RCC mixtures. Additionally, no fly ash or other cement substitution materials were used. The chemical composition and physical properties of the cement used in this study are detailed in Table 4.

Table 4. Chemical composition and physical properties of the cement used in RCC

CEM I 42.5 R, %							
Chemical Composition				Mineral Composition		Physical Properties	
SiO ₂	20.3	TiO ₂	0.2	C3A	7	Specific gravity	3.15
Al ₂ O ₃	4.42	SrO	0.29	C3S	58	Blaine, m ² /kg	393
Fe ₂ O ₃	3	P ₂ O ₅	0.22	C2S	15	% passing 45µm	89
CaO	62.6	Mn ₂ O ₃	0.05	C4AF	9		
MgO	2.74	ZnO	0.02				
SO ₃	3.08	Cr ₂ O ₃	0.02				
K ₂ O	0.82	Loss on Ignition (LOI)	1.91				
Na ₂ O	0.29						

3.3.3. Water

Fresh tap water, meeting the requirements of ASTM C1602 (2018) and certified as potable and free from harmful substances, was utilized to produce fresh concrete mixes.

3.3.4. Admixtures Used

Modified polycarboxyl ether is crucial in producing high-performance water-reducing admixtures, commonly known as polycarboxylate superplasticizers (PCE-based

superplasticizers). These admixtures contain modified polycarboxyl ether molecules with a unique structure that efficiently adsorb onto cement particles, effectively dispersing them and reducing the concrete mix's viscosity. This feature enables easier placement and compaction, resulting in improved dispersion and a more uniform distribution of cement particles throughout the mix. As a result, these admixtures contribute to higher strength, increased durability, and improved workability of the RCC.

In this study, a superplasticizer (Zirve TrueChemical 42-07) and an air-entraining agent (Zirve Air FLOW 42) were sourced from ZIRVE Inc. in Konya. Both additives were incorporated into the mixture at a total rate of 0.7 kg/m^3 per unit weight of all cementitious materials, with each additive used in equal amounts of 0.35 kg/m^3 .

3.3.5. Fiber Inclusion

The fiber reinforcement product utilized in fiber-reinforced RCC samples was sourced from KORDSA. The fiber, commercially known as "KraTos FIBERS," is CE-certified, and its characteristic properties fully comply with the specifications outlined in ASTM C1116 (2015). In the fresh concrete incorporating fiber, 3 kg of fiber per m^3 of concrete was employed. The visual representation of the fibers and their technical specifications are presented in Figure 43.

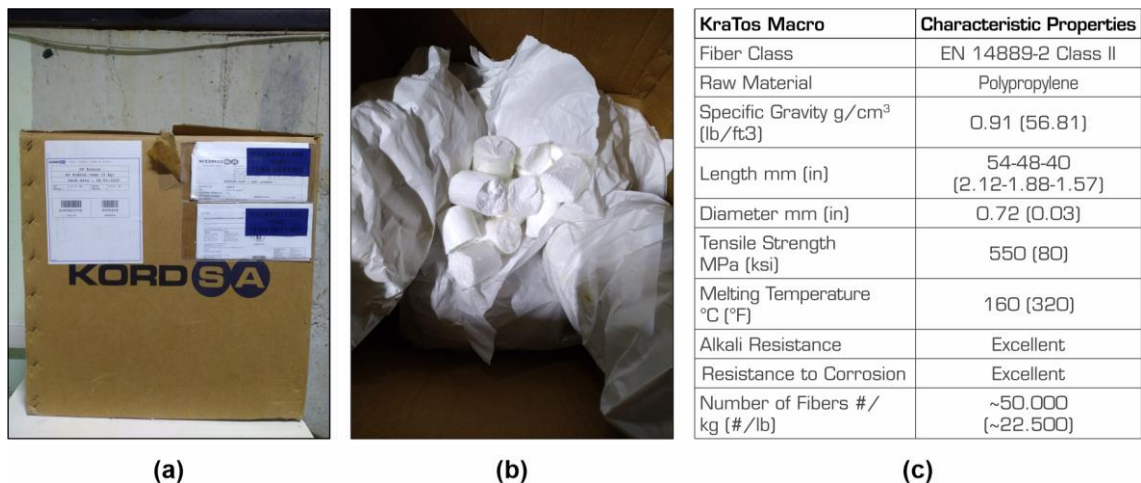


Figure 43. Characteristic properties of fiber used in this study

3.3.6. Adjacent Joint Surface Reinforcements Agents

In this study, the interface of two adjacent parts, commonly known as the old joint face, was prepared in several alternatives, including (#1) longitudinal joints with no bond coat, bare surface, (#2) wetted with sprayed water, (#3) coated with cement slurry with a cement/water ratio of 2.0 by weight, and (#4) coated with epoxy adhesive for structural bonding supplied by a commercial supplier (MasterBrace ADH 1406, a product of Master Builders Solutions from the Sika group). Some examples of adjacent surface reinforcement used in the cold longitudinal joints are shown in Figure 44.

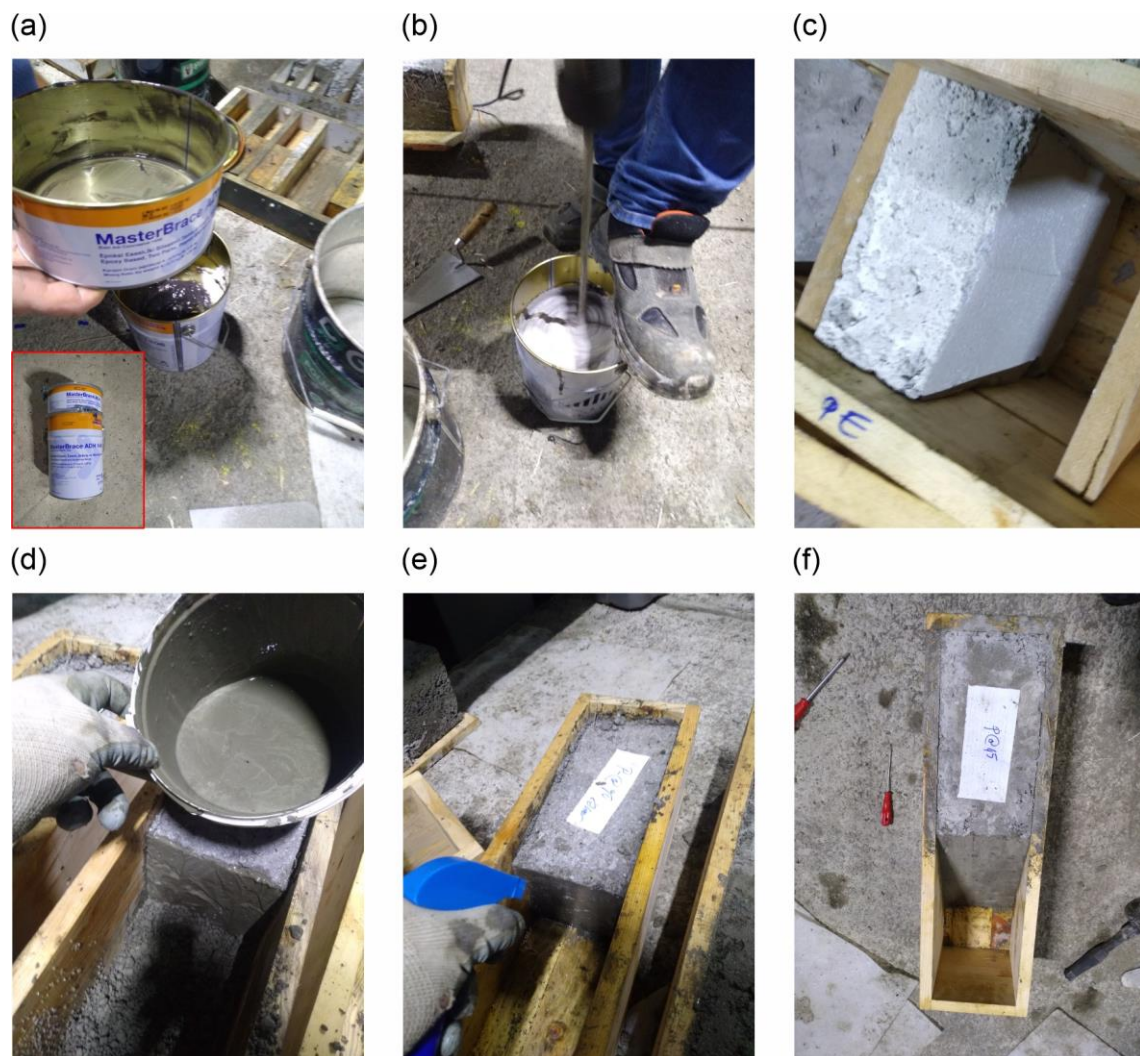


Figure 44. (a-c) Mixing the components and applying epoxy adhesive, (d) Using cement slurry, (e) water spraying, and (f) leaving the joint bare

3.4. Fresh RCC Production

3.4.1. Mix Design

The mix design process for RCC is a critical step in ensuring the performance and durability of RCC pavements. This chapter details the methodology, material selection, proportioning, and analysis used in developing an optimal RCC mix for further laboratory applications. The part below details the mix design process, material selection, and proportioning used in our study, adhering to current industry standards and best practices while aiming to balance strength, workability, and durability. Following the regulations of the Turkish Cement Manufacturers' Association (TCMB, 2018) and the Turkish General Technical Specification of RCC Pavements (CSB, 2020), our aggregate mixture comprised at least three different basaltic aggregate groups. The cement used was Portland Cement CEM I 42.5 R. Coarse aggregate is from Basaltic gravel, whose nominal maximum size is 13 mm, with a specific gravity of 2.68 and 24-hour water absorption of 0.64% (ASTM C127-15, 2015). Fine aggregate is from basaltic angular sand, whose specific gravity is 2.64, and 24-hour water absorption is 0.81% (ASTM C128-15, 2015). Water is potable water free from impurities. Admixture was of two components comprised of superplasticizer+ air-entraining admixture. The initial mix design was developed for a 10-cubic-decimeter (10 liters) sample. The absolute volume method was employed to determine the material proportioning. The baseline mixture was primarily based on the composition used by Park et al. (2020). Table 5 presents the final mix proportions. Figure 45 presents the sample preparation area and setup.

Table 5. Final proportions of fresh RCC mix used in this study

Component	Cement	Coarse Aggregate (Gravel)	Fine Aggregate (Sand)	Water	Admixture (superplasticizer+ air-entraining)	Total
Weight (g)	3,000	8,891	12,439	357	18	24,705
Percentage (%)	12.12	35.91	50.24	1.44	0.07	100

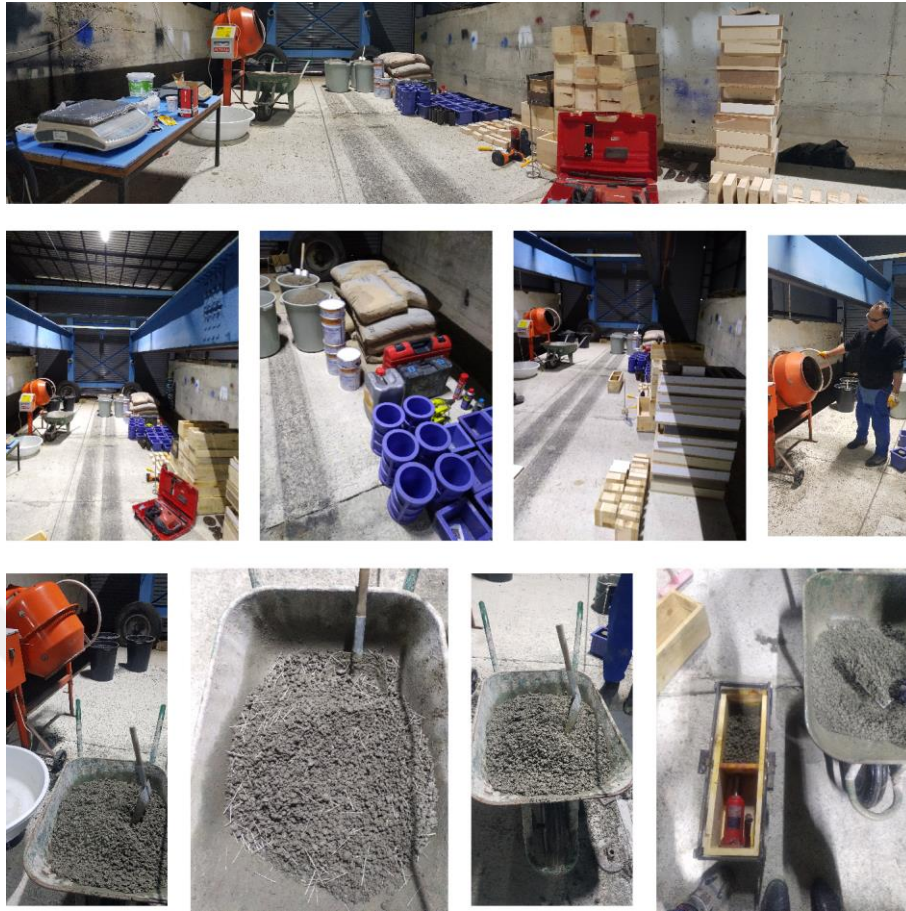


Figure 45. A general view of the sample preparation area and setup

It is well established that varying cement content affects both the fresh and hardened properties of RCC. Fakhri & Amoosoltani (2017) demonstrated that increasing the cement content from 250 to 400 kg/m³ in fresh RCC mixtures results in a noticeable increase in strength. In RCC technology, a cement content of 280 kg/m³ is considered within the acceptable range, while 240 kg/m³ is deemed below the acceptable limit, and 320 kg/m³ is classified as above the acceptable limit. However, since the primary goal of this study is not to investigate the impact of different cement contents on the mechanical performance of RCC mixtures, a single cement content was chosen for the experiments. The cement content used in this study was 300 kg/m³, which is within the acceptable range and rather close to the upper limit. The water-to-cement ratio (w/c), $w/c = \frac{\text{Weight of Water}}{\text{Weight of Cement}}$, was calculated as follows: $w/c = 357 \text{ g} / 3,000 \text{ g} = 0.119$ or 11.9%. This low w/c ratio is characteristic of RCC mixes, contributing to high strength and durability while maintaining the desired consistency for roller compaction. The aggregate-to-cement ratio was determined as (Coarse Aggregate

+ Fine Aggregate) / Cement = (8,891 g + 12,439 g) / 3,000 g = 7.11. This high aggregate-to-cement ratio is typical for RCC mixes, contributing to its unique properties and suitability for pavement construction. The sand-to-aggregate ratio (s/a) was calculated as Fine Aggregate / (Coarse Aggregate + Fine Aggregate) = 12,439 / (8,891 + 12,439) = 0.5832 or 58.32%. This ratio indicates a relatively high proportion of fine aggregate, which can help achieve better compaction and reduce voids in the mix. The Theoretical Maximum Constituent Density (TMCD) was calculated using the following equation: $TMCD = (P_c + P_{ca} + P_{fa} + P_w) / \left(\frac{P_c}{D_{rc}} + \frac{P_{ca}}{D_{rca}} + \frac{P_{fa}}{D_{rfa}} + \frac{P_w}{D_{rw}} \right)$, Where: P_c , P_{ca} , P_{fa} , P_w are the percentages of cement, coarse aggregate, fine aggregate, and water, respectively. D_{rc} , D_{rca} , D_{rfa} , D_{rw} are the specific gravities of cement, coarse aggregate, fine aggregate, and water, respectively. $TMCD = (12.12 + 35.91 + 50.24 + 1.44) / (12.12/3.15 + 35.91/2.68 + 50.24/2.64 + 1.44/1.0) = 99.71 / 38.22 = 2.61 \text{ t/m}^3 = 2.61 \times 9.80665 = 25.56 \text{ kN/m}^3$. The maximum achievable unit weight (γ) was calculated as $\gamma = TMCD \gamma_w = 2.61 * 1 = 2.61 \text{ t/m}^3$ or $\gamma = 2.61 \times 9.80665 = 25.56 \text{ kN/m}^3$, where γ_w is the unit weight of water. The developed RCC mix design exhibits several notable characteristics. The low water-to-cement ratio of 11.9% is significantly lower than typical conventional concrete mixes. This low water content is crucial for RCC to achieve the desired consistency for roller compaction while maintaining high strength and durability. The high aggregate content, represented by the aggregate-to-cement ratio of 7.11, is consistent with RCC best practices, promoting optimal density and strength characteristics. The sand-to-aggregate ratio of 58.32% is relatively high but between the typical early recommended range of 50-70% for RCC mixes. Incorporating admixtures (0.07% by mass) aligns with modern RCC mix designs that utilize chemical admixtures that enhance workability without compromising strength or durability. The calculated maximum achievable unit weight of 25.56 kN/m³ is in line with typical RCC densities reported in the literature, which range from 2400 to 2600 kg/m³. The RCC mix's final natural density is 23.42 kN/m³. Because of its lower water content and how compaction works, the RCC usually has some air space left in it even after compaction. Unlike the theoretical maximum density, which assumes zero voids, these voids lower the total density. Moisture loss from evaporation may occur during the installation and compaction of RCC, thereby influencing the density. While the actual density accounts for nearly unavoidable loss, the TMCD assumes perfect moisture conditions. In addition, laboratory compactions and on-site compaction frequently yield less TMCD; nonetheless, the actual compaction may

differ from the calculations made theoretically under laboratory conditions. This mix design provides a solid foundation for further laboratory testing and trials.

3.4.2. Molding, Compaction and Sampling

There are two primary methods for molding RCC samples: using a vibratory hammer, as regulated by ASTM C1435 (2014), or a vibrating table, as specified by ASTM C1176 (2013). Both methods simulate roller compaction through vibration effects. However, Tayabji & Okamoto (1987) demonstrated that increased moisture content in the fresh mix, when using the vibrating table, leads to a decrease in the overall strength and density of the final sample. Therefore, fresh RCC samples were compacted in this study using an electric vibratory hammer. According to ASTM C1435, adopted as the standard method for vertical molding, a fresh and ready-to-cast RCC mixture is compacted in molds using an electric vibratory hammer equipped with a shaft and plate. This standard is specifically designed for molding RCC in cylindrical molds. However, various mold types and sizes may also be used similarly to conventional concrete. Figure 46 illustrates the equipment utilized for sampling and compacting the fresh concrete.



Figure 46. Apparatuses used in the sampling and compaction process of the fresh concrete

Considering that it is not always possible to use full-size roller compactors in the laboratory environment, alternative methods have been proposed to prepare laboratory-scale molded samples of RCC. Of these methods, the most used one is the compaction of the RCC with a vibratory plate since it can be used in all kinds of field conditions. This method is described in detail only for cylindrical samples in ASTM 1435. An assortment of tamping plates were designed and produced, so much so that all possible molding forms could be available. The details of several tamping plates are shown in Figure 47.

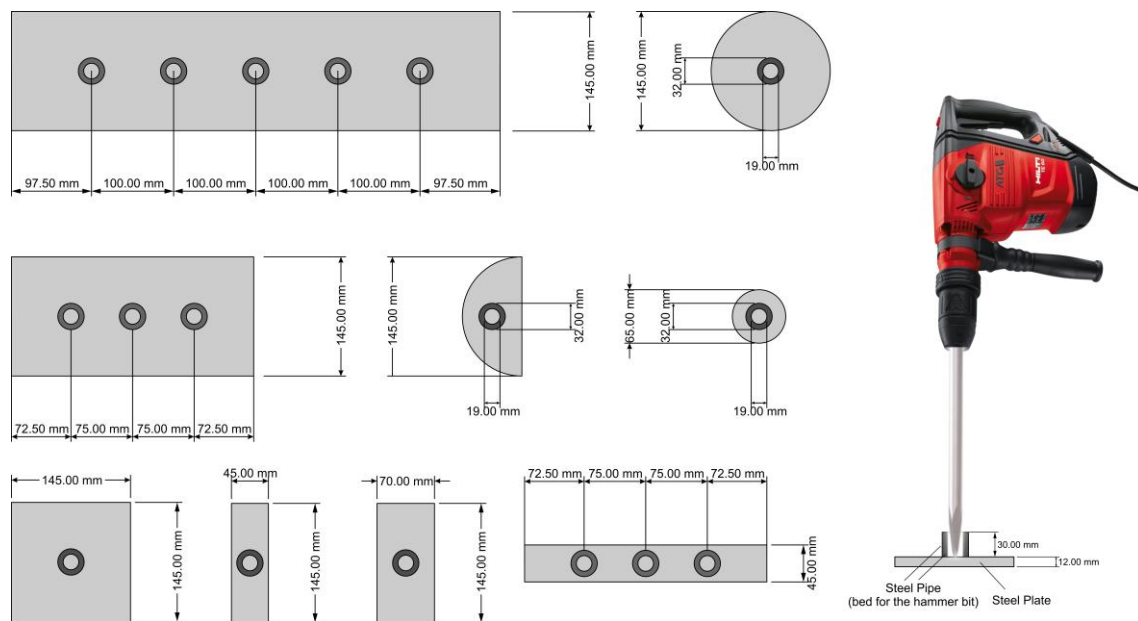


Figure 47. Tamping plates used in compaction process of the fresh concrete

Before beginning the evaluation, it would be helpful to summarize the essential points given in the core reference. While C1435 specifies that notes and footnotes (excluding those in tables and figures) should not be considered mandatory requirements of this practice, they have already become an established standard practice. C1435 paves a pathway employing a technique based on using an electric vibrating hammer equipped with a shaft and circular plate. This method requires using the electric vibrating hammer to compact a stiff to very dry consistency concrete mixture in four consecutive lifts and molding test specimens solely vertically in cylindrical molds. The practice cautions against the use of rodding or internal vibration as they are ineffective methods for consolidating the RCC to the required density. In this manner, the density of fresh concrete can also be determined using test specimens. The vibrating hammer used for compaction purposes should have a

mass (excluding the tamping plate and shaft) of 8.5 to 13.5 Kg, a minimum power input of 900 W, and a maximum output of 2000 to 2000 ± 200 impacts per minute. Fresh concrete shall have a maximum aggregate size of 50 mm or less. This means that this method can also be used as a surrogate method for RCC mixtures for pavement construction. For each batch of concrete mixture, all the molding processes of the concrete shall be completed in less than 45 minutes, as recommended in ASTM 1435. The vibrating hammer we used was a brand-new device, younger than one-year-old, and no repair was done. It had been used for less than 300 hours. In that sense, no calibration was needed according to the specifications expressed in the standard practice. Mold sleeves were allowed if reusable molds were used. From then on, a wooden sleeve (sheath) was prepared and placed inside a solid steel beam box, serving as steel armor. The inner sleeve was tightened using the left- and rightmost clamps so that the mold was held firmly in the sleeve. First, enough concrete in the mold is placed to fill one-fourth of its whole volume after consolidation, with a total mass of approximately 3.4 kg. Upon beginning the compaction, as the concrete is tamped down, the vibrator dawn drags, and the mortar starts topping on and above the tamping plate. There comes a time when the vibrating hammer should be stopped when the liquidlike mortar ring has fully formed around the tamping plate. If most of the mortar ring has not formed after 20 seconds, the vibrating hammer is stopped, and the next layer of concrete is applied. If the tamping plate keeps running downward up to a level below the topmost level of the mold, the hammering is terminated. The same steps are implemented for the second and third layers of concrete, filling the mold to approximately half and three-quarters of its volume. For the fourth shift, the mold is overfilled by mounding the concrete on the top. Additional concrete is placed so that the top surface can be handled to be smooth surface and within a level with the top of the mold. Finally, the uppermost part of the concrete is struck off with the vibrating hammer and tamping plate. While the hammer is working and the plate is literally in contact with the edge of the rim of the cylinder mold, the plate is moved back and forth at the top of the to get rid of the excess material beyond the rim by a driving force up until the topmost surface is smooth and flat with the top of the roller mold.

A sampling process was developed for the tests conducted on fresh RCC samples, as no standard currently exists for casting fresh RCC samples in a laboratory environment. Several views from the molding process are shown in Figure 48.



Figure 48. Some views from the molding process

A procedure was devised to address this by combining elements from several well-established standards (ASTM C1435, 2014; ASTM D1632, 2017; ASTM D1633, 2017). ASTM C1435 outlines how to mold cylindrical samples in 150/300 mm molds, while ASTM D1632 focuses on molding and curing soil-cement samples for compression and flexure tests. ASTM D1633 specifies the procedure for conducting compressive tests on molded soil-cement cylinders. This study used the cylindrical mold specified in ASTM D1632 to create lab-scale samples that could easily be handled during testing. The samples were removed from the molds using a hand-jack extruder. While the compaction procedure was aligned with the guidelines of ASTM C1435 and D1632, it differed slightly from the method in ASTM C1435. A 6.8 kg dropping weight machine (similar to a Proctor hammer with a 15

lb mass) is typically used in compacting mixtures. However, a custom tamping plate was employed in this case, with a diameter 5 mm smaller than the mold's, which remains within ASTM C1435 limits. After extracting the mold samples, relevant measurements (mass, final dimensions) were taken. Any samples that exhibited spalling or honeycombing were discarded, and new samples were prepared. The inner surfaces of the molds were lubricated with oil to facilitate sample removal. Cylindrical specimens were capped to get tested in an unconfined compression testing machine.

ACI (2000) noted that no standard procedure exists for molding and producing RCC beams in a laboratory environment. Additionally, sawed beams extracted from actual RCC pavement test sections generally exhibit lower strength parameters compared to laboratory-made beams. In a unique study, Ferrebee et al. (2014) proposed and implemented a casting procedure for producing beam specimens based on the method used for RCC cylinder specimens. They designed a solid beam mold with dimensions of 150 mm in depth, 80 mm in width, and 700 mm in length and a tamping plate featuring three slots. One slot was centered in the plate, while the other was positioned 100 mm from each end. The large beam specimens were cast in three equal lifts, while the small ones were cast in two. For each lift, compaction was performed using a vibrating hammer, with a maximum of 20-second compaction time per lift, following the details outlined in ASTM C1435 (ASTM C1435, 2014). Compaction was applied through the slots in the tamping plate. A sequential vibration process was followed for each lift, with compaction applied at 5-second intervals in four stages. The first 5 seconds were applied to the middle slot, followed by 5 seconds at one end, 5 seconds at the other end, and then another 5 seconds at the middle slot. In line with the method used by Ferrebee et al. (2014), we employed a similar alternating compaction procedure (both centric and asymmetric) for all beam samples. This involved using a tamping plate with evenly spaced slots into which the rod was inserted for compaction. Before placing the overtopped material as a final operation, the last lift of the RCC mixtures on the beams was cast with a thickness 20% greater than the design thickness, as Rollings (1988a) recommended. Specimens without cold joints were kept in the steel molds for 24 hours after compaction. Following this, they were submerged in a water bath maintained at 23.2°C until testing. No elevated temperatures were applied to accelerate curing, ensuring the specimens underwent standard curing conditions typical of RCC. Specimens with cold joints, cast in a separate mold, followed the same curing regime as the non-jointed samples. The first lift was kept continuously moist for beam specimens with cold joints using a water-

saturated burlap on the side opposite the joint. After 24 hours, the second lift was cast directly onto the first, and both sections were kept moist with the burlap (sack cloth) Twenty-four hours after the second casting, the specimens were removed from the molds and placed in the curing bath for further hydration and strength development.

3.5. Analytical Procedure for Calculating Electric Hammer Compaction Time to Simulate Roller Compaction

This section establishes an analogy between rollers and electric vibratory hammers using an approach based essentially on plainly Newtonian physics. Additionally, calculating the time required to use an electric hammer instead of an actual roller compactor (conjectured) will be explained. This approach aims first to determine the energy transmitted by an actual roller compactor at a specific speed to a given contact area. Then, it seeks to calculate how long a vibrating hammer should be applied to fresh RCC through a load transfer plate atop the mold to produce an equivalent amount of energy for compaction.

The fundamental equations used in the computational process are presented below, followed by an example calculation to illustrate their application. Parametric calculations are expanded, and comparative analyses are presented in the relevant results and discussion section. Our basic equations are nothing but the mechanical motion equations known to everyone. Assume that L is the length of the contact surface in between the roller and compactable half-plane, simplified to the length of the roller, and b is the width of the contact area whose centerline could theoretically align with the drum's center. The contact width can be computed using theories like Hertzian contact theory, as I did, or through finite element modeling. The contact area can be computed as $A_R = Lb$. Given that the velocity of the roller is v , the required unit time to transfer the impact energy through per contact area is calculated as $t = b/v$. While the frequency is denoted by f , the total number of strokes (N_s) produced by the roller mechanism and exerted by the single (front) drum on the surface is calculated as $N_s = f \times t$ in times. We know the amplitude of the roller, $\hat{u}_R$, and the dynamic force produced, F_R , according to the active mode of the roller. The total energy (E_R) being transferred to the contact surface by the drum is $E_R = F_R \times \hat{u}_R \times N_s$ in Newton $\times$ meter. Now we can calculate the compaction energy acting on per unit area of the contact surface as $E_{R,unit} = E_R/A_R$ in N/m. At this stage, the unit compaction energy exerted by the drum onto the contact area to be simulated by the electric vibrator is defined.

The main entry of the surrogate computation is the mass of the electric vibratory hammer. It should represent the total mass of the device, which comprises the hammer itself, the breaker bid, and the tamping plate attached to the bid. Assume that it is m kg. Because the vibrative forces are produced upon the total mass, and the resulting energy is to be exerted by the tamping plate onto the fresh RCC beneath the plate. The latitude ($\hat{u}_{h,2}$) of the acceleration components produced in three directions by the hammer is used. However, taking the impact acceleration as the resultant component of the triaxial vibration more realistically represents the machine's actual mode. While the triaxial vibration value is A_h , the impact force, F_h , is calculated from Newton's second law of motion, $F_h = A_h m$. As expected, impact energy, E_h , can be calculated by $E_h = m \times V_h^2 / 2$, where V_h is the velocity at impact. Now that we know the total mass and impact energy given in the device specifications, we can put the value in the relevant place, and impact velocity can then be found. We know that the area of the transfer plate, A_{tp} , attached to the bid of the hammer. Besides the compaction energy produced by the hammer, we can also calculate the unit energy exerted onto as per unit area of the transfer plate as follows: $E_{h,unit} = E_h / A_{tp}$ in N/m. More so, we know from the previous computation steps that the unit compaction energy scattered over the area of the contact area by the roller's drum is $E_{R,unit}$. If we equalize the unit compaction energy exerted by the roller to the unit compaction energy produced by the electric hammer, we can find the required total energy needed to be produced by the hammer, $E_{R,unit} = E_h / A_{tp}$ the E_h in Nm. We can back-calculate the number of strokes using the same relation employed in the roller compaction phase. While the relation is $E_h = F_h \times \hat{u}_h \times N_h$, the sole unknown term becomes the number of strokes (N_h) that the hammer produces, which can now be calculated straightforwardly. Number of strokes is the spin-off term for the cyclic motion. We know that frequency is the reciprocal of the period, representing the number of oscillations per unit of time. Therefore, if we count some time while the machine is working within a specified frequency, the number of stokes is calculated by the relation of $N_h = f \times t$. So, we now know the number of strokes and the hammer's frequency. If the known parameters are plugged into the relation, t , which is the required working time for the vibratory hammer to produce the same unit energy by the actual roller, is defined

The specifications of the actual roller compactors and electric vibratory hammers employed in the parametric analyses are given in Table 6. The selection of equipment brands

and models is entirely independent of commercial considerations. The machines referenced are either already in our possession or local council-owned units that are readily accessible. Besides, these brands were chosen for subsequent project phases due to their availability and the possibility of low-cost rental if needed. It is important to note that this study does not endorse or criticize any particular machine or device. The data and calculations presented herein are purely factual and based on real-world performance without intent to promote or disparage any equipment manufacturer.

Table 6. Compactor specifications employed in computations

Device	Size of the Transfer Medium	Velocity (average)	Centrifugal Force (F_s)	Frequency (f)	Amplitude (up and down motion) ($\hat{u}$)
AMMANN AV 110 X (In mode#2 ^{**})	1.7 m Length $\times$ 1.3 m Diameter Drum (Static load: 30.9 kg/mm)	1, 2, and 3 km/h	110 kN	45 Hz (2700 cpm [*])	0.7 mm
AMMANN ARP 35 T4i (In mode#1 ^{**})	1.1 m Length $\times$ 0.8 m Diameter Drum (Static load: 14.5 kg/mm)	1, 2, and 3 km/h	22 kN	47 Hz (2820 cpm [*])	0.47 mm
HILTI TE 60-AVR Rotary Hammer (In hammering mode) ^{***}	By variable transfer plates, customized	—	0.81 kN	55 Hz (3300 cpm [*])	1 mm (approx.)
Makita Hr5201C Rotary Hammer (In hammering mode) ^{***}	By variable transfer plates, customized	—	0.27 kN	35.83 Hz (2150 cpm [*])	1 mm (approx.)
[*] 1 Hz = 1/60 cpm (cycles per minute), ipm (impact per minute) or vpm (vibrations per minute) ^{**} The number of operation modes is preferred according to the recommended specs for use in the compaction of the soil and soil-like granular materials rather than asphalt. ^{***} While the hammers have two modes, namely rotary drilling and chiseling without rotary effect, the machine is used in hammering mode without rotary motion throughout all the applications.					

3.6. Laboratory Tests Conducted in This Study

3.6.1. Density, Absorption, and Void Measurements of RCC Samples

This chapter presents the experimental methods and results for density, absorption, and void measurements of hardened RCC samples (plain and fiber-incorporated) following ASTM C642-21 (2021) with noted modifications. Figure 49 illustrates the experimental

setup and computational pathways. RCC samples were saw-cut into rectangular prisms, deviating from standard disk-shaped samples due to the nature of available specimens.

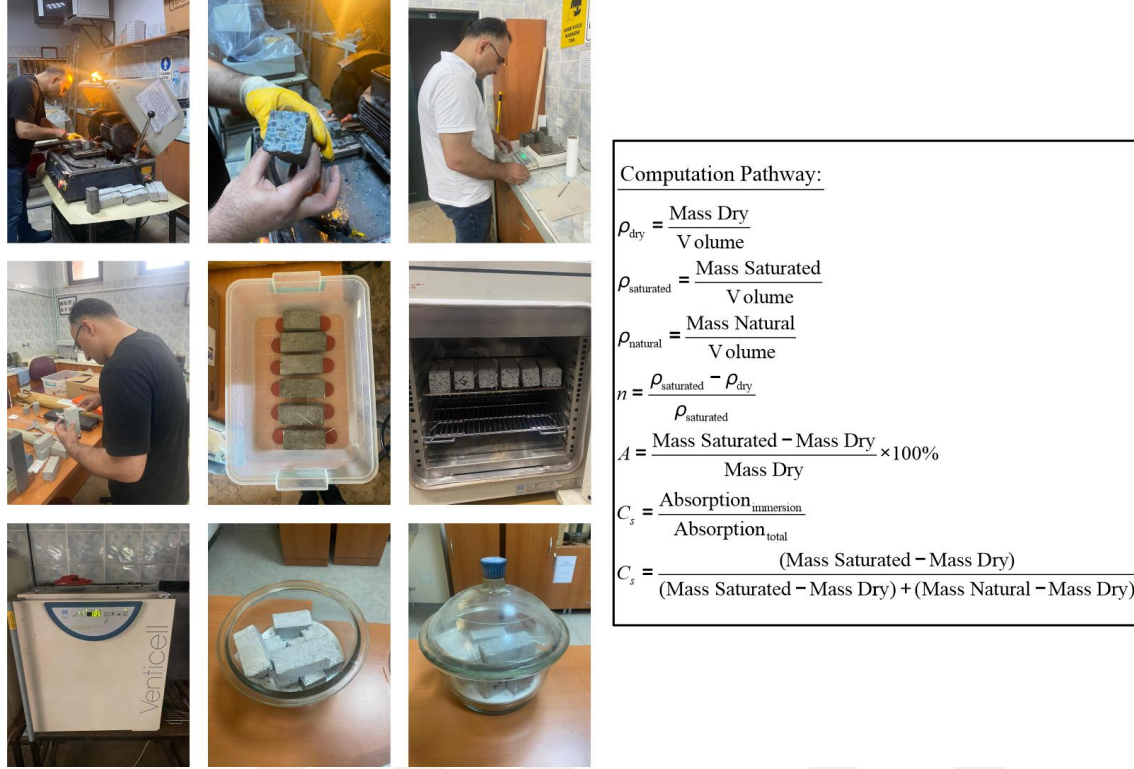


Figure 49. Test setup used in measuring density, absorption, and voids of RCC samples

Samples underwent a slightly modified saturation process employing partial submersion for 12 hours followed by full submersion for seven days, replacing the standard 5-hour boiling method to prevent potential damage to the RCC structure. Namely, the samples were placed in a water tub up to half the height of the samples, ensuring that each sample was partially submerged. This process allows the concrete to absorb water and helps prevent air from being trapped within the pores when the water level increases. After the initial wait period, more water was gradually added to the container until the samples were fully submerged, ensuring that the final water level was above the top surface of the samples by at least a few centimeters. The fully submerged samples were kept in the water for seven days to ensure complete saturation. After the complete saturation period, the samples were removed from the water and placed into a desiccator glass jar to prevent further ambient moisture exchange. The samples were then weighed again to obtain the saturated mass. All the samples were kept in the drying oven for 24 hours at $105 \pm 5^\circ\text{C}$. The thoroughly dried

samples were again placed in the desiccator and protected from the natural humidity effect until the measurements were finalized. The final measurements were made after defining the oven-dry mass. Precise mass measurements and temperature control are crucial to tamping down the uncertainty. Some potential error sources could be incomplete saturation due to the modified method, drying time/temperature variations, and minor material loss during handling.

3.6.2. Compressive Strength Measurements

Compressive strength measurements on RCC samples are conventionally carried out following an internationally recognized testing method outlined in ASTM C39 (2018). This method, commonly used for testing conventional concrete, involves using cylindrical specimen molds. While it is a common practice in field sampling, various mold sizes can also be employed. In instances where the specimen slenderness (expressed as the ratio of length-to-diameter, L/D) is equal to or less than 1.75, test results are corrected by multiplying with a correction factor provided in a relevant table within ASTM C39. This multiplication factor, typically less than 1.0, becomes crucial, especially if a soil approach is preferred to define the proportion of RCC. Staying within the limits defined by ASTM C39, cubic specimen molds of $150 \times 150 \times 150$ mm are consistently utilized throughout this study.

Additionally, cylindrical samples measuring 150×300 mm were cast to compare the test results with those obtained from the cubic samples. A tamper-mounted vibrating hammer was employed for all samples based on ASTM C1435 (2014) recommendations for compacting RCC in a mold. The testing device used for measuring compressive strength in this study is depicted in Figure 50.

The loading rate, representing the applied load onto the sample over time, is a crucial parameter regulated by ASTM C39 (2018). The loading rate must fall within a predefined range to prevent shock-like loading. In this study, a compressive load was consistently applied at a constant stress rate of 0.25 MPa/s onto the samples, ensuring a gradual application without abrupt shocks and allowing for the capture of residual plastic deformations.



Figure 50. The test setup of a universal testing machine for measuring the compressive strength of RCC samples

3.6.3. Flexural Strength Measurements

A third-point loading mechanism was employed to determine the flexural strength of RCC samples, whether theoretically from the mass or longitudinal joint zone. The flexural strength test was conducted in accordance with ASTM C78 (2018). For all flexure tests in this study, beam specimens measuring 50×50×300 mm and 150×180×600 mm were utilized. Upon failure, the peak load causing the failure was recorded and used to calculate the flexural strength, also known as the modulus of rupture. At the recommended stress rate (0.25 ± 0.05 MPa/s), the loading rate under both constant moment and fixed load conditions is calculated

using the equation $V = 2abh^2/3l$, where a is the stress rate (MPa/s), b is the width of the beam (mm), h is the height of the beam sample (mm), l is the length between the support spans (mm). The load rate, representing the speed of displacement applied to the sample, was maintained at a constant rate of 0.21 kN/s for 50×50×300 mm samples and 1.62 kN/s for 150×180×600 mm beam specimens. The load was applied evenly and incrementally to the sample. The flexural strength testing device (universal testing machine) used in this study is depicted in Figure 51. In the three-point bending test, given the sample's rectangular cross-section, the modulus of rupture is calculated as $f_r = 3Pl/2bh^2$, where P is the load at fracture, l is the length between the support spans, b is the width, and h is the height of the beam. The benchmark criterion for the failure mode, used to define the modulus of rupture, was assumed to be the peak load causing fracture within the middle one-third of the sample.

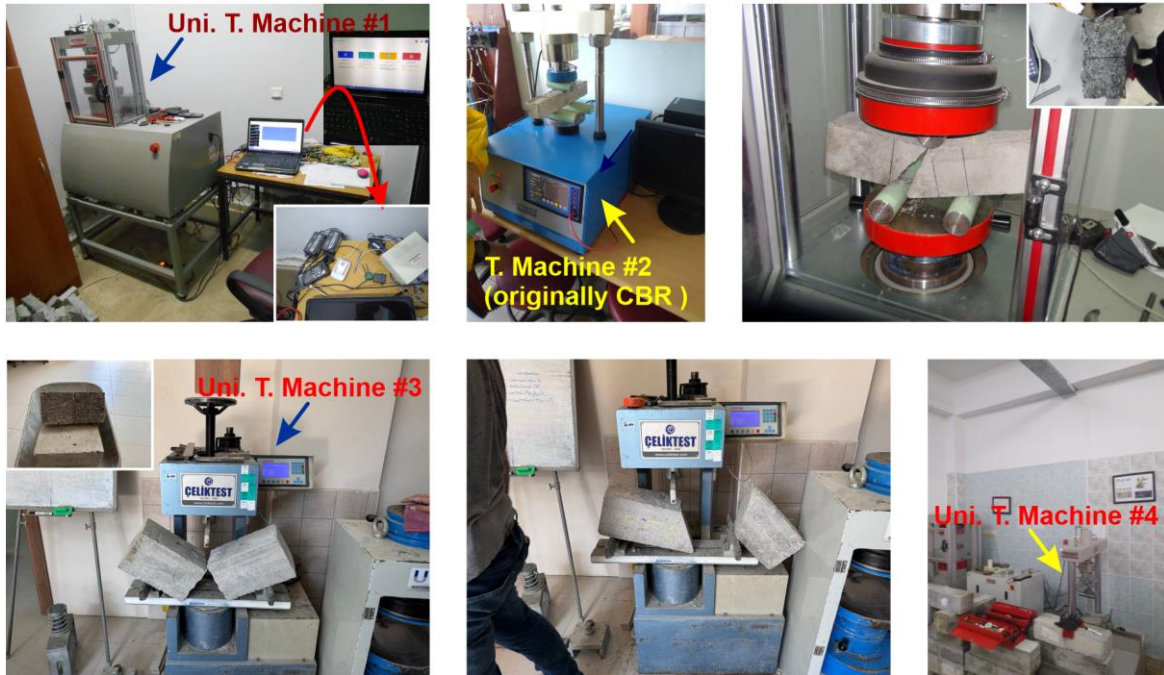


Figure 51. Flexural strength test devices used in this study for testing the samples

3.6.4. Measurement of Interface Bonding Between Old and Fresh RCC Joints

In this study, a concrete compaction machine was utilized in its original state, with additional apparatuses enabling the conversion of compressive force into shear force between the adjacent surfaces of old and new RCC. This approach facilitated the assessment of the impact of interface reinforcement treatment techniques on the bond strength between old and new RCC surfaces through various experiments. The objective was to explore the

relationships among the experiments and their practical applicability and assess their efficacy in explaining the measured values. To this end, Bi-surface shear test, Double-shear test, and Split prism test techniques were employed. All fresh RCC was produced using the same recipe throughout the study, with cubic sample molds employed in the entire process. Wooden wedges, lubricated with mold oil, were positioned before the casting of the first part and subsequently removed after the concrete hardened. Following the removal of these wedges, fresh concrete-RCC was deposited into the rectangular prism already allocated for new concrete pouring, and compaction was executed using the same technique at precalculated intervals of tamping time, thereby maintaining consistent tamping counts. The concrete compression testing machine used in the test maintained a loading rate of 0.25 ± 0.05 MPa/s. The test modes used to determine the relative bond strength between hardened RCC lifts in this study are illustrated in Figure 52. The apparatuses facilitating the application of load onto the sample in the split prism test were designed, directly drawing inspiration from ASTM C1245 (2012).

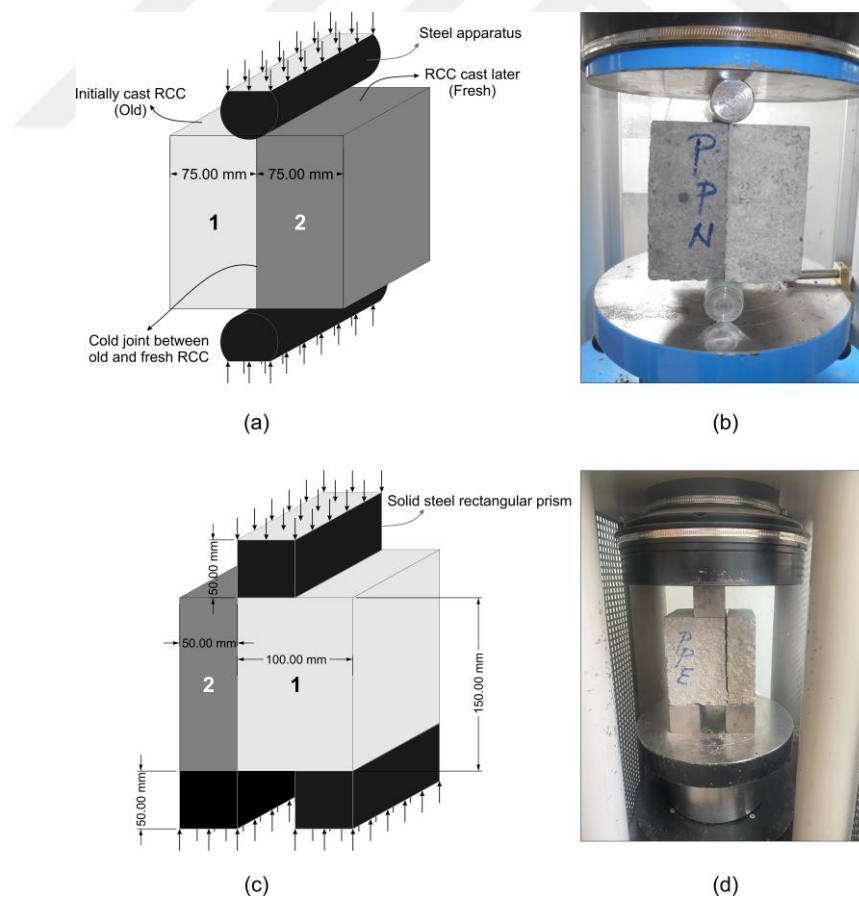


Figure 52. Testing modes used to determine the relative bond strength in between RCC cold longitudinal joints; (a-b) Split prism test, (c-d) Bi-surface shear test

3.6.5. Non-Destructive Testing and Measurements Techniques Employed

Two different non-destructive measurement methods were employed throughout this study. One method involves determining the P and S wave transmission times of the beam samples, both with and without joints, using ultrasound velocity measurement. Upon measuring the transmission distance (distance between probes) and determining the travel time, the P and S wave velocities were then calculated. The main objective here is to test the hypothesis that different cold joint geometries result in varying compaction performance, particularly when compacting the fresh lane. It is thought that the strength and overall physical properties of the initially compacted lane (i.e., the first-cast lane) would influence the entire element due to differences in compaction, and these properties were tested using various techniques. A comprehensive comparison was made with reference samples cast as a whole, without joints. In harmony with ASTM C597 (2016), the pulse velocity passing through the concrete medium is calculated as follows: $V = L/T$, where V is the pulse velocity in m/s, L is the distance between centers of transducer faces in m, and T is the transit time in seconds (originally in μs).

Two different measuring instruments, the Proceq Puntid Lab(+) and Proceq Pundit 200 (Proceq, 2021), were used for velocity measurements across four methods. Both devices were initially tested for calibration using their own calibration samples. They were also tested for P wave measurements and found to yield the same results. Consequently, P waves were measured with the Puntid Lab(+), while S waves were measured with the Proceq Pundit 200 using different probes. Both direct and indirect methods were employed in the measurements. By altering the transmission path, measurements were taken around the joint on various transmission paths, allowing for evaluations of the material properties behind and beyond the joint. It is known that the instruments measure the transit time using the shortest transmission distance at which communication between probes is first established. In that sense, each measurement was taken three consecutive times, and the average value was used. The measurement techniques employed and tools used are illustrated in Figure 53.

Another non-destructive method I employed is the use of the Schmidt hammer, also known as the Swiss hammer or rebound hammer. Both original N-type and L-type hammers were available in our laboratory. While N-type hammers are called original hammers, L-type hammers are designated as lightweight. We used the Proceq Silver Schmidt Type L Concrete Test Hammer, which measures the Q value with less energy and is suitable for

concretes with strengths up to 100 MPa. The Silver Schmidt Type L Concrete Test Hammer combines optical speed sensors and a lightweight alloy impact plunger to gather Q-value rebound data. The main reason for using this tool is its ability to quickly record and report sequential measurements in its memory. Additionally, the coil-spring mechanism of the original hammer is not durable enough for many sequential measurements (195 exercises per beam with some pauses). Measurements were taken on the side surface of the $150 \times 180 \times 600$ beam samples, perpendicular to the joint area rather than in the casting direction. A pointwise template was used to create a grid by marking equally spaced points on the beam.

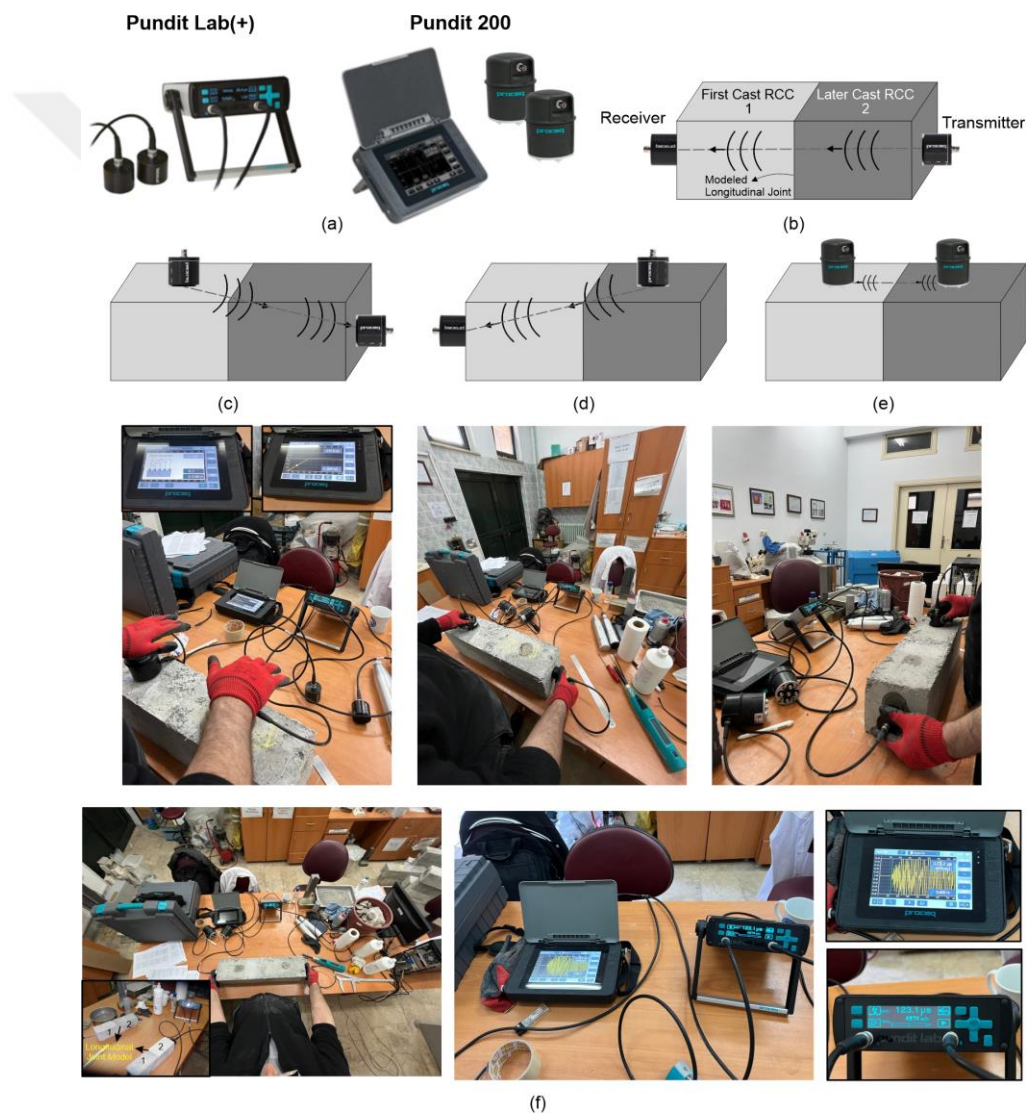


Figure 53. (a) Ultrasound pulse velocity measurement devices used, (b) Direct method, (c) Semidirect method-1, (d) Semidirect method-2, (e) Indirect method, (f) Test-through photos

Three rebound measurements were taken from each grid point, and the average was recorded. To put it simply, lateral surface areas of the beams were mapped as grids within an orthogonal coordinate system. A general Q value index for the beam models was determined by averaging the measurements for each beam, allowing for comparison between beams. An important detail is the elastic property distribution resulting from compaction, influenced by the joint shape itself rather than the joint interface reinforcement process. The effect of the interface reinforcement treatment (a thin layer applied to the beam dimensions) on the bending strength of the beam was assessed through bending tests. However, since the locally applied reinforcement material at the interface does not affect the overall compaction of the beam, this aspect is excluded from the evaluation, and only the geometrically varied joints were assessed. The tools, measurement techniques, and phases are detailed in Figure 54.

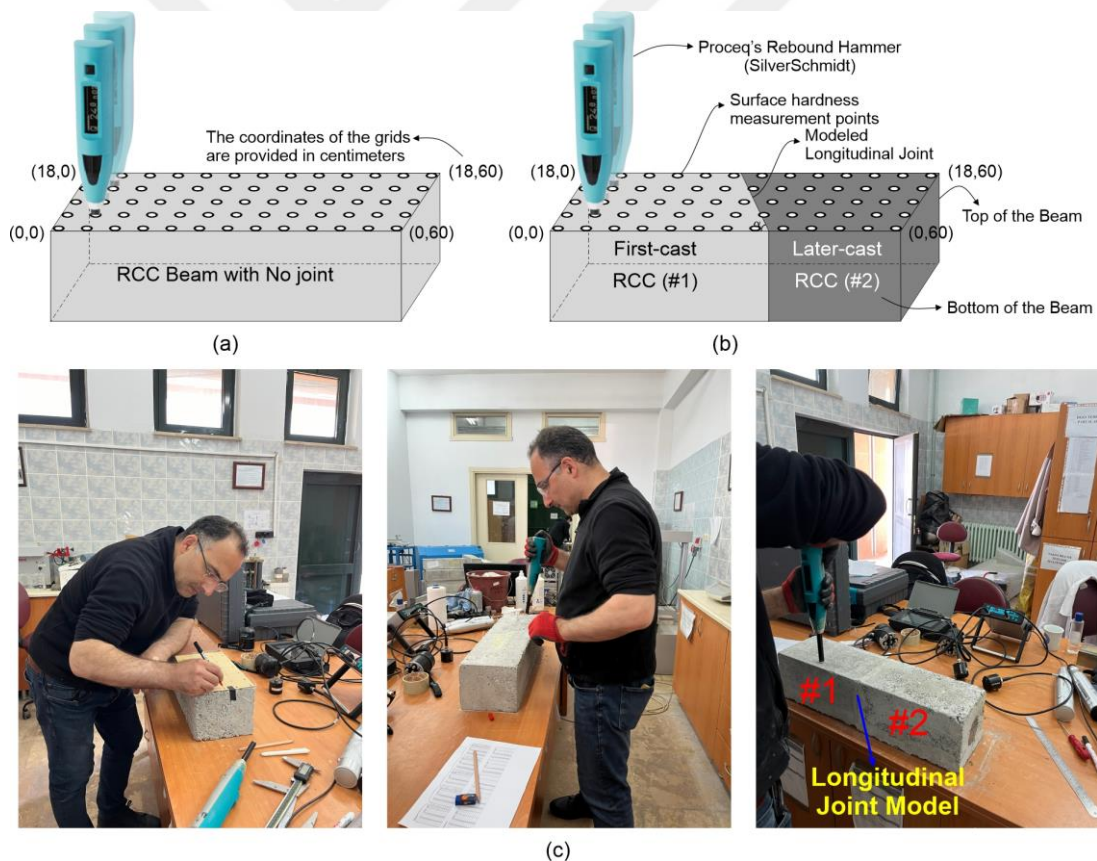


Figure 54. Schmidt hammers used in rebound tests and sample measurement techniques: (a) No-joint sample, (b) Sample with a modeled longitudinal joint, (c) Test preparation and testing photos

3.7. Optimization Algorithms Employed

This chapter delves into the fundamental principles and applications of mathematical optimization in engineering problems, mainly focusing on optimizing material properties under constrained deformation conditions. I begin by defining optimization and discussing various types of optimization problems, highlighting the basis of constrained optimization. Through a systematic presentation, I also explore the standard notation and mathematical formulations used in optimization, emphasizing the role of the objective function and constraints. The core mathematical model supporting the optimization process is explicitly grounded in Hertzian contact mechanics. The fundamental equations for combined modulus of elasticity, Poisson's ratio, computed and calculated vertical displacement, contact zone width, and maximum Hertzian pressure were presented and employed.

In its simplest form, optimization refers to a mathematically expressed problem designed to identify the optimal solution for effective problem resolution. In that sense, there must be at least one function with minimum or maximum values, which serves as the objective for optimization (Beale, 1988). Optimization problems manifest in various forms, including deterministic or nondeterministic, single or multi-objective, discrete or continuous, constrained or unconstrained, linear or nonlinear. More often than not, constrained optimization problems are preferred as they enhance our understanding of the behavioral nature of engineering models. The primary objective of mathematical optimization is to ascertain the best available solution or, frequently, solutions within a given domain that defines the search space. Namely, the search space encompasses potential candidate values that are systematically tested to determine their optimality (Martins & Ning, 2021).

The most common and straightforward notation for mathematical optimization algorithms is presented as follows. Let Ω be a constraint set referred to as a feasible set with x representing the decision variable. Initially, an objective function denoted as $f : \Omega \rightarrow \mathbb{R}$, is determined for optimization, where Ω is a set with elements mapping to real numbers. The set Ω consists of sub-members with $x_0 \in \Omega$ representing a specific element. At this juncture, the problem posed by f can be approached as either a minimization or maximization problem. Specifically, if $f(x_0) \leq f(x)$ holds for all $x \in \Omega$, the problem is classified as a minimization problem; conversely, if $f(x_0) \geq f(x)$ for all $x \in \Omega$, it transforms into a maximization problem. The notation 'for all $x \in \Omega$ ' denotes a state of subordination, and

'subject to' is abbreviated as 's.t.'. In summary, the above concepts can be succinctly summarized as $f: \mathbb{R}^n \rightarrow \mathbb{R}, \Omega \subseteq \mathbb{R}^n$. In general, the set Ω is given in a succinct functional form as $\Omega = \{x \in \mathbb{R}^n \mid g_i(x) \geq 0, i = 1, \dots, m, h_j(x) \geq 0, j = 1, \dots, k\}$, $g_i(x), h_j(x): \mathbb{R}^n \rightarrow \mathbb{R}$. Conventionally, $g_i(x)$ and $h_j(x)$ are called boundary condition functions. Quite simply, while $\forall x \in \Omega$, x_0 or x^* refers to a solution or global solution to the problem. Hence, the optimal value, f^* , denotes the infimum of the f over the domain of Ω . Namely, if the optimal solution x^* is found within the search space, the optimal value f^* is equivalent to $f(x^*)$. I highly recommend that readers consult the comprehensive book of Messac (2015), which I also used and referenced while writing the codes, for a detailed explanation, description, and syntactic procedures.

In the context of the code that I provided in the appendix, the primary goal is to optimize the modulus of elasticity (E) and Poisson's ratio (ν) of a frictionless half-space compactable medium under a constant drum. This problem involves the indentation of a drum into a space and is addressed using Hertz's contact theory. To this end, the task is accomplished through a sizeable iterative process by minimizing the difference between the calculated indentation displacement and the target displacement based on Hertzian contact theory. In our case, the objective function is formulated to minimize the absolute difference between the calculated displacement ($d_{\text{calculated}}$) and the target displacement (d_{target}). Mathematically, the objective function can be expressed as $f(x) = |d_{\text{calculated}}(E, \nu) - d_{\text{target}}|$. Where $x = [E, \nu]$ represents the candidate variables in the search space. The optimization problem handling includes a few constraints on the decision variables to ensure the feasible values for E and ν . These constraints are defined as follows: $g(x)_1: E_{\text{Lower bound}} \leq E \leq E_{\text{Upper bound}}$ and $g(x)_2: \nu_{\text{Lower bound}} \leq \nu \leq \nu_{\text{Upper bound}}$. These constraints must first be defined to ensure that the final solution remains within a physically meaningful range for the system, applied load, and materials. The optimization algorithm employed in the process is based on a ready-to-use famous MATLAB function called `fmincon`, a standard solver for constrained nonlinear optimization problems. The problem is solved iteratively using the GlobalSearch algorithm, which enhances the likelihood of finding a global minimum by exploring multiple starting points. Both for the sake of brevity and not to stray too far from the essence of the problem, the well-known theory for constrained optimality is not recapped here. However, some key components of the theory for reproducibility are given below.

In all cases, the Karush-Kuhn-Tucker (Karush, 1939; Kuhn & Tucker, 1951) optimality conditions are adopted as a necessary condition for an optimal solution in nonlinear programming, provided that some regularity conditions are satisfied. For the given problem, the KKT conditions are parametrized as (a) the gradient of the Lagrangian must be zero at the optimal point, (b) the constraints of the problem must be satisfied, (c) the Lagrange multipliers associated with the inequality constraints must be non-negative, (d) the product of each Lagrange multiplier and the corresponding constraint function must be zero. The auxiliary Lagrangian function is employed by the KKT conditions: $L(x, \lambda) = f(x) + \sum \lambda_{g,i} g_i(x) + \sum \lambda_{h,i} h_i(x)$. In the present problem, the Lagrangian can be written as $L(E, v, \lambda_E, \lambda_v) = |d_{calculated}(E, v) - d_{target}| + \lambda_E(E_{lower} - E) + \lambda_v(v_{lower} - v)$. The vector λ represents the Lagrange multipliers, combining the values of λ_g and λ_h with its length corresponding to the total number of constraints. The total number of restrictions is its length. The following are the KKT requirements are to be fulfilled: (i) $\nabla_x L(x, \lambda) = 0$, (ii) $\lambda_{g,i} g_i(x) = 0 \forall i$, (iii) $\{g(x) \leq 0, h(x) = 0, \lambda_{g,i} \geq 0\}$. The reader will find it more straightforward to grasp if some crucial parts are explained clearly. An optimality measure is a quantity that indicates how close a solution is to the optimal. Lagrange multipliers are used in constrained optimization to incorporate the constraints into the objective function. $\lambda_{g,i}$ represents the Lagrange multiplier associated with the i -th constraint, $g_i(x)$ represents i -th constraint function. The $\|\cdot\|$ is the norm of the expression, which denotes a measure of the length or magnitude of a vector in space.

Accordingly, the optimality measure for condition (i) is given by; $\|\nabla_x L(x, \lambda)\| = \|\nabla f(x) + \sum \lambda_{g,i} \nabla g_i(x) + \sum \lambda_{h,i} \nabla h_i(x)\|$. The term $\|\nabla_x L(x, \lambda)\|$ describes the norm of the gradient of the Lagrangian, a common measure of optimality in constrained optimization. Additionally, the optimality measure for the product $\lambda_g g(x)$ is represented by $|\lambda_g \cdot g(x)|$ which denotes the magnitude (absolute value) of this product. This measure quantifies the degree to which the constraints are active, indicating that when the measure is zero, either the constraint $g_i(x) = 0$ is precisely satisfied, or the corresponding Lagrange multiplier $\lambda_{g,i} = 0$, thereby fulfilling the complementary slackness condition. If a dual variable, namely λ , is positive (slack), then the corresponding primal constraint must be binding, meaning it is satisfied as an equality (tightness). Conversely, if a primal constraint is not binding (i.e., slack), then the associated dual variable must be tight (i.e., zero). This relationship captures the principle of complementary slackness, where a non-zero dual

variable indicates a tight constraint and a zero dual variable signifies a slack constraint. The mathematical model underpinning the optimization process is based on Hertzian contact theory, and the fundamental relations employed in the optimization code are as follows. For the use of the given equations, please refer to Johnson (1985), Slocum (1992), and Young et al. (2002).

$$E^* = \left(\frac{1 - \nu^2}{E} + \frac{1 - \nu_{\text{roller}}^2}{E_{\text{roller}}} \right)^{-1} \quad (20)$$

$$d_{\text{calculated}} = \left(\frac{F}{\pi L E^*} \right) \left(1 + \ln \left(\frac{2\pi(L)^3 E^*}{F 2R} \right) \right) \quad (21)$$

$$a_{\text{zone}} = \left(\frac{2F 2R}{\pi L E^*} \right)^{1/2} \quad (22)$$

$$P_{\text{max}} = \left(\frac{2F E^*}{\pi L 2R} \right)^{1/2} \quad (23)$$

Where, E : Young's modulus of the material being compacted, ν : Poisson's ratio of the material being compacted, E_{roller} : Young's modulus of the roller material, ν_{roller} : Poisson's ratio of the roller material, F : Applied load exerted by the roller, L : Length of the drum or roller in contact with the material, R : Radius of the roller, E^* : Combined modulus of elasticity for the contact between the roller and the material, accounting for the properties of both materials, $d_{\text{calculated}}$: Deflection of the contact zone, a_{zone} : Half-width of contact zone, P_{max} : Maximum Hertzian pressure.

3.8. Finite Element Modeling and Analyses

In this chapter, the main goal is to evaluate the effect of the initial roller position on the compaction performance of fresh concrete lanes in an RCC pavement model with cold longitudinal joints of varying taper angles and shapes in notched wedges. Detailed finite element models were developed using the open-source software PrePoMax CalculiX V.2.0, which utilizes the finite element solver engine CalculiX Version 2.21.

3.8.1. Model Geometry and Initial Parameters

A Cartesian coordinate system was used to create each finite element model in the modeling phase. The pavement models' fundamental physical and mechanical characteristics were kept constant throughout this task to make the results comparable. A schematic illustrating the primary pavement model and layer orientations is shown in Figure 55. In this research, the parameters to be investigated and optimized are (1) the taper angle; α , (2) the distance of the roller (front drum) relative to the longitudinal joint just in the first pass; β .

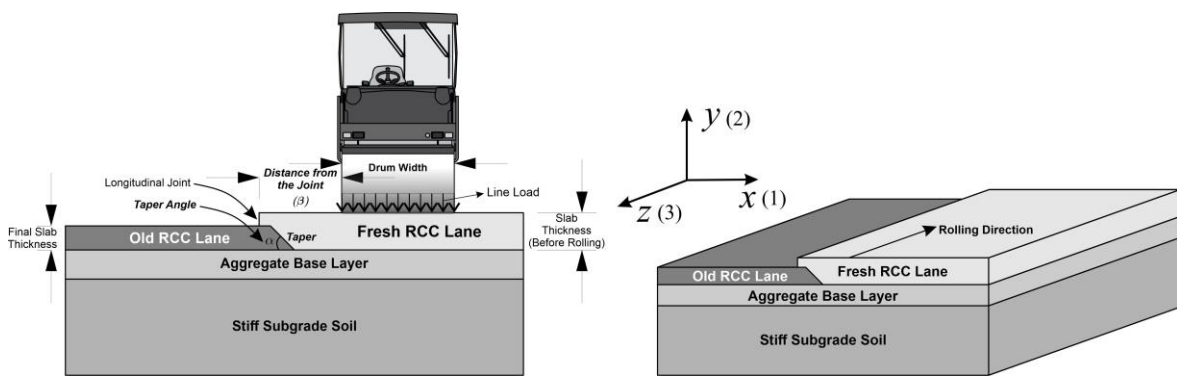


Figure 55. Schematic of the cold joint compaction problem investigated in this study

In modeling the variants, five different roller positions, or loading variants, were employed. The drum model that applies static load to the fresh RCC represents the front drum of the roller. Since the primary aim was to examine the effect of the initial roller position on compaction, the rear drum was not considered in the modeling. Literature, primarily based on HMA pavement compaction results, and observations from field compaction exercises indicate that the final compaction is largely determined by the initial compaction. After the initial compaction, the fresh material retains plastic deformations and consolidates, with subsequent compaction efforts having only a minor impact on final compaction performance, albeit just slight improvements are possible. In each model, the drum's position relative to the surface projection of the longitudinal joint was kept fixed. The longitudinal joint variants tested and the initial drum positions relative to the longitudinal joint are shown in Figure 56.

The selection of horizontal angles (90° , 60° , 45° , and 30°) for the longitudinal joints in the designed models is predicated on two salient factors. Firstly, examining catalogs from edge pressing and cutting device manufacturers, exemplified by HAMM (2017), reveals that

the available tools are engineered to create tapers at these specific angles. Consequently, these slope angles predominate in the practical implementation of longitudinal joints. Secondly, while HMA pavements permit the construction of longitudinal joints with relatively acute inclination angles in substantially thinner layers, the application of such angles in RCC pavements presents unique challenges. The significantly greater overall thickness of RCC pavements, when coupled with small joint angles, results in a horizontal joint component that far exceeds its vertical counterpart. This excessive horizontal extension increases the likelihood of segregation in the fresh concrete, making it more prone to cracking under the pressure applied by roller drums. Thus, the selection of joint angles represents a critical design parameter that must be optimized to ensure structural integrity and constructability in RCC pavements.

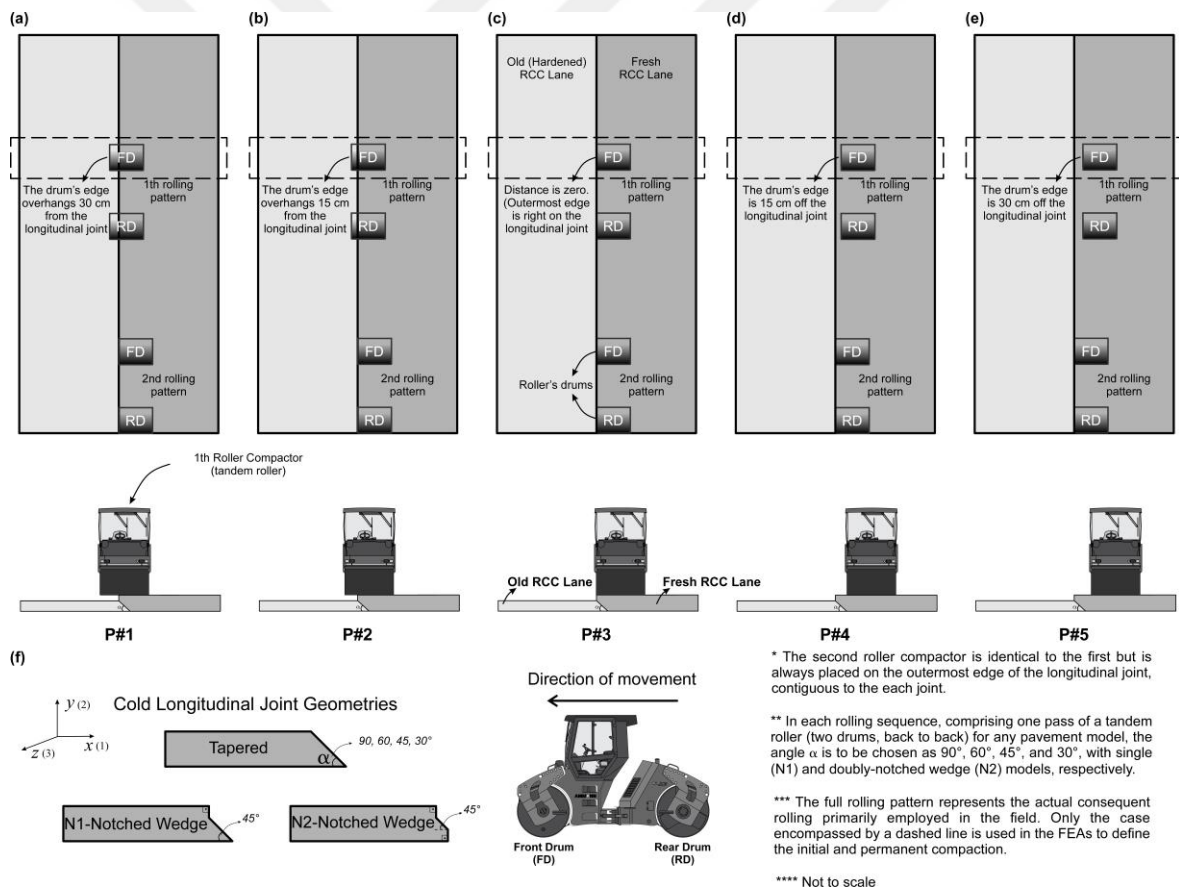


Figure 56. Tested positions (from P#1 to P#5) of the roller against the longitudinal joint

The final names of the individual models were assigned based on a straightforward pathway. In this approach, the drum position for each model was denoted by "P," while the position of the drum load was numbered from 1 to 5. Additionally, for each pavement model,

the slope of the longitudinal joint (angle of the tapered surface with the horizontal axis) was modeled at four different angles: 90, 60, 45, and 30 degrees. Two variants of the notched wedge were represented as N1 and N2. Model N1 is the classical notched-wedge model used in HMA pavement construction, albeit with different dimensions. Model N2 is the doubly-notched wedge model, which has sparked debate among engineers for years about whether it is superior to the single-notched wedge. Some engineers support N2, while many (me included) intuitively oppose it.

The deformations are denoted by "u" in three directions of the orthogonal axis system, labeled as u1, u2, and u3. Since the pavement model is in 2D, u3 is not applicable. The models are categorized based on their geometric features. For example, the name U290P1 corresponds to a model's vertical deformations, where the cold joint's slope angle is 90°, and the drum is positioned 300 mm off from the joint on the old side. In total, 30 numerical models (six cold joint models and five roller positions) were developed and solved in the PrePoMax CalculiX V.2.0 environment.

On the one hand, the first position (P#1) represents a particular case where the drum protrudes 300 mm (~12 inches) from the edge of the longitudinal joint. This 300 mm distance is not arbitrary; it is generally considered the initially preferred distance in RCC pavement constructions. On the other hand, the second position (P#2) represents a 150 mm overhang of the drum from the edge of the longitudinal joint, which is an overhang distance (~6 inches) typically used in HMA pavement constructions. In the third position (P#3), the outer edge of the drum is positioned directly above the surface projection of the longitudinal joint. This position is a frequently employed technique widely known as a standard approach in Turkish RCC pavement practice. The fourth position (P#4) has the drum 150 mm off the longitudinal joint, diametrically opposite to the overhang position adopted in P#2. The 150 mm distance used in P#4 is typical in HMA pavement constructions. The fifth position (P#5) differs slightly from the fourth. In P#5, the outer edge of the drum is 300 mm off the longitudinal joint aligned towards the inner part of the RCC mat. The 300 mm distance from the longitudinal joint to the drum's edge used in P#5 is consistent with RCC pavement constructions.

3.8.2. Implemented Drum Load and Contact Pressure

Mechanical models and properties of all the lanes included in the problem were held constant. In all models and elements, the force of gravity was activated to more accurately simulate real-life conditions, particularly to observe the vertical deformation of the fresh RCC layer during compaction efforts. The drum size, mechanical properties, and loading magnitude were kept constant for each resolved task. A single-type double drum roller (AMMANN AV 110 X TIER 3) was employed in the modeling phase. The physical and mechanical features are detailed in Figure 57 and Table 7.

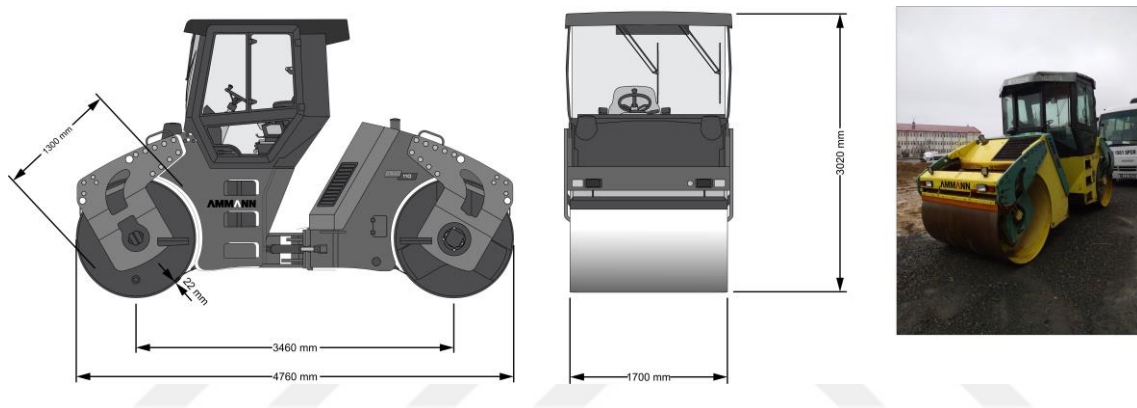


Figure 57. Roller dimensions used in finite element modeling

Table 7. Specifications for the roller compactor employed in modeling (Ammann, 2022)

MAXIMUM RECOMMENDED COMPACTED LIFT THICKNESS AT OPTIMAL WORKING CONDITIONS					
Model	Sand / Gravel	Mixed Soils	Silt	Clay	Stabilization
AV 110 X	0.42 m (17 in)	0.32 m (13 in)	—	—	0.22 m (9 in)
WEIGHT & DRIVING CHARACTERISTICS			COMPACTION FORCES		
Operating weight	10 400 kg (22 930 lb)		Frequency I	45 Hz (2700 VPM)	
Maximum weight	11 190 kg (24 670 lb)		Frequency II	55 Hz (3300 VPM)	
Static linear load of front drum	30.9 kg/cm (173 lb/in)		Amplitude I	0.7 mm (0.028 in)	
Static linear load of rear drum	30.3 kg/cm (169.7 lb/in)		Amplitude II	0.35 mm (0.014 in)	
Max. transport speed	11 km/h (6.8 MPH)		Centrifugal force I	110 kN (24 729 lbf)	
Climbing ability	30 %		Centrifugal force II	77/83 kN (17 310/18 659 lbf)	
Turning radius inner (EDGE)	4650 mm (183.1 in)		—	—	
Crab mode	160 mm (6.3 in)		—	—	

The maximum recommended compacted lift thickness under optimal working conditions for AV 110 X is detailed in Table 7. The free-body diagram detailing the drum load and the mechanical model interaction between RCC and the roller is represented in Figure 58.

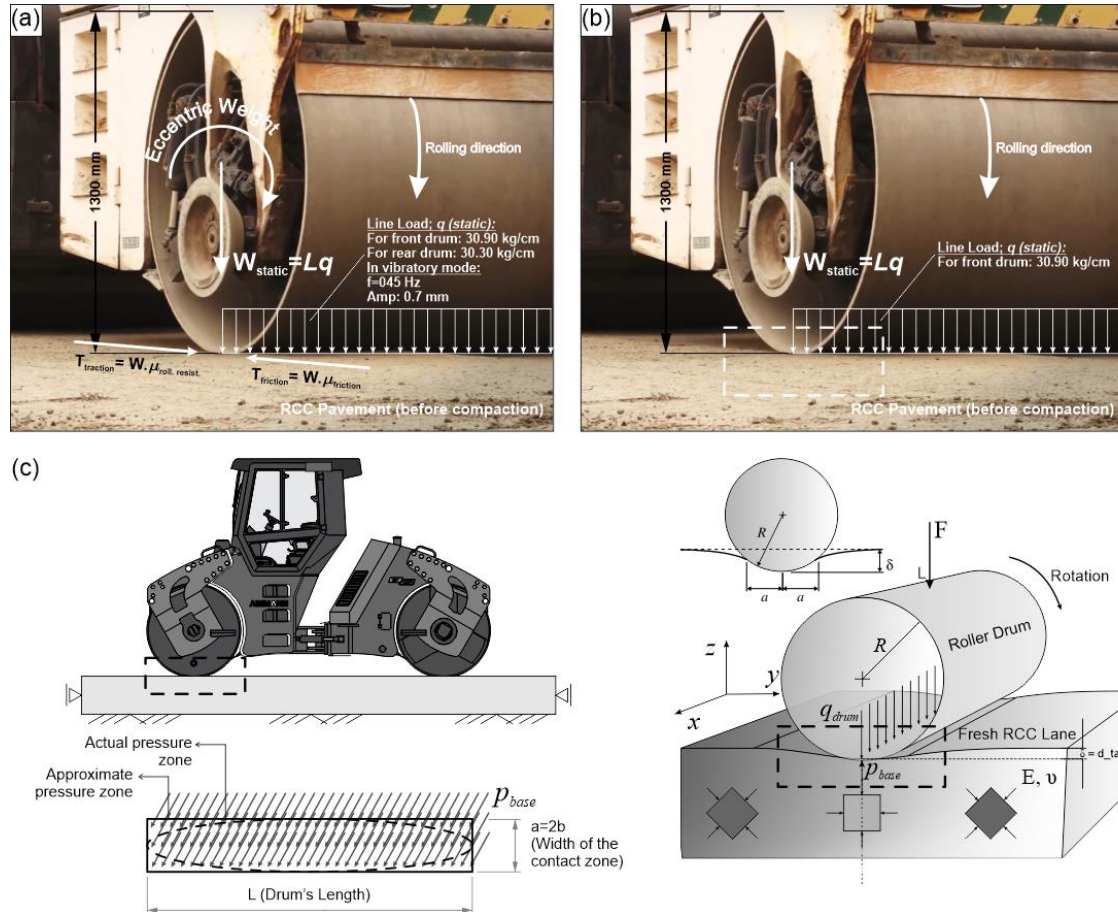


Figure 58. Details of the drum load and mechanical model interaction between fresh RCC medium and the roller: (a) Dynamic condition, (b) Static condition, (c) Load and base pressure

According to specs detailed in this table, the specified machine used in the modeling was the epitome of one of the best roller choices. Vibrational compaction is a technique that utilizes vibrations to densify compactable media, such as soils, artificial fills, and RCC. When an innate static force is transformed into vibrations, the weight and impact increase the energy produced. Vibrations from a vibrating shaft are transmitted to the ground through a contact surface, which can substantially vary depending on the physical and mechanical features of the medium. The vibration effect aims to induce soil particles to move in the direction of the vibration, leading to a more compact structure. This process is similar to

consolidation but without the need for water discharge. While vibrational compaction is commonly used for RCC, static compaction offers several advantages. Introducing static vibrations can help reduce segregation, improve workability, and increase the density of the concrete. In this study, the static mode of the roller was modeled using the load limits expressed in Table 7. The Hertzian contact theory is employed to model the pressure exerted on the soil or RCC during compaction. Based on this theory, the pressure based on the contact area between the vibrating shaft and the material was calculated. The low elasticity modulus of the RCC, which was found upon optimization, is attributed to the compactable RCC medium without incorporating its plastic and strain-hardening properties. These properties are often neglected in pure elasticity-based models and have been unknown to all of us to date. For this reason, the pressure to be applied was calculated analytically and applied as stress to the fresh material in MPa.

3.8.3. Material Definitions

In various roller-compacted concrete (RCC) pavement projects, different concrete consistencies were observed to be compacted under 10-ton rollers, particularly during the initial pass. Depending on the nominal thickness of the laid concrete, vertical deformations of 10-20% were observed in fresh RCC as the final compaction. A model was created to simulate this behavior, representing a 230 mm thick fresh concrete layer. The model aimed to simulate a 10 mm compaction after the first drum pass. The entire consecutive rolling process was not simulated due to the unavailability of a material model that could accurately capture fresh-state concrete's plasticity and hardening features. The primary objective of the modeling was to determine how the drum's initial position affects compaction performance in cold joints with varying geometries. It is notable that the initial compaction predominantly influences the subsequent compaction stages of the soil-like medium. The interface zone between hardened and fresh RCC, which constitutes the adjacent joint zone, was modeled using a 30° friction angle ($\mu=0.58$). This value is consistent with the overall range (0.50 - 0.60) suggested by Bowles (1996) to account for adhesion between the adjacent surfaces of the old and fresh concrete. The elasticity moduli for the base and subgrade layers were selected to exhibit characteristics indicative of well-compacted and highly stiff strata, respectively. This selection ensures that the RCC mat neither subsides into the base layer due to insufficient stiffness nor does the base layer manifest behavior indistinguishable from

that of the subgrade stratum. Such differentiation in mechanical properties is crucial for maintaining the structural integrity and performance of the pavement system while optimizing load distribution and minimizing differential settlement. The material properties employed in the finite element model of the pavement are shown in Table 8.

Table 8. Constitutive material properties of the material models employed in FEA

Layers	Modulus of Elasticity (MPa)	Poisson's Ratio	Natural Density (kN/m ³)	Thickness (mm)	Length (mm)
Hardened (Set) RCC	30,000	0.2	24.0	200	3,000
Fresh RCC	7.875	0.354	24.2	230	3,000
Base Course (Granular)	120	0.35	20.0	300	6,000
Subgrade Soil (Stiff)	100	0.35	17.0	1500	6,000

3.8.4. Element Types and Meshes

Single solid (continuum) element type was used in quasi-2D finite element modellings. Assigning the third dimension is 0.1 unit and employing strain confinement in that dimension, the fifteen-node wedge element (C3D15) was employed. The element-based details are given in Table 9.

Table 9. Element details of the finite element models

Finite Element Model	Node #	Element #	Element Type	Size of the FE Model (in-plane)	Max Element Size	Min Element Size
Fresh Joint Model	728,587	12,010	C3D15	12.09 m2	50 mm	5 mm
Cold Jointed Model, Tapered (90°)	718,401	10,924	C3D15	12.09 m2	50 mm	5 mm
Cold Jointed Model, Tapered (60°)	723,651	11,010	C3D15	12.09 m2	50 mm	5 mm
Cold Jointed Model, Tapered (45°)	728,070	11,077	C3D15	12.09 m2	50 mm	5 mm
Cold Jointed Model, Tapered (30°)	701,952	10,025	C3D15	12.09 m2	50 mm	5 mm
Cold Jointed Model, Notched (N1)	719,264	10,937	C3D15	12.09 m2	50 mm	5 mm
Cold Jointed Model, Notched (N2)	725,017	12,406	C3D15	12.09 m2	50 mm	5 mm

The C3D15 is a multipurpose wedge-shaped element, utilizing 9 integration points without any reduced integration. This technique essentially creates a quasi-2D model using 3D elements. It is an approach used when 3D elements are preferred for various reasons

(e.g., material model compatibility, specific output requirements, or software limitations). Throughout the modeling, it was preferred to keep the element sizes as small as possible. Seeing that coarse meshes potentially create relatively bulky mechanical bodies divided into finite elements less sensitive against heavy loads. The principal freshly jointed pavement model was tested many times using various element sizes to eliminate isochoric behavior and hourglass formation as much as possible. In doing so, the most favorable element size was thereupon determined. In this sense, the extrapolation of values belonging to the integration points (Barlow, 1976) was finally stabilized.

3.8.5. Parts, Partitions, and Assembly

Each pavement model was created from several parts roughly the actual size with different strengths and material properties. To this end, the actual dimensions of the parts were used without alteration. The geometries were chosen to be as regular as possible. Element geometries were partitioned to directly have prismatic shapes as much as possible, thereby avoiding irregular mesh shapes and node contacts. Each longitudinal layer is bonded and designed as frictionless hard contact. The old-to-fresh RCC interface was designed with friction contact, and no bonding (gluing) parameter was employed. In that sense, the adhesion between the hardened concrete and fresh concrete lane was modeled using a 30° friction angle. Instead of applying the single dead load by a 2D roller to the compactable layer, the pressure through the contact surface between the drum and layer was calculated and applied.

Essentially, the Hertz solutions of the 2D and 3D indentation problems differ. While the compacted layer has too low elasticity modulus compared to those for solid materials, the behaviors between the solid roller and fresh concrete yielded too many warpings of the shell, with strains diverging to infinity, thereby cutting off the run. In an actual manner, by employing this computation pattern, the Hertzian indentation problem, initially developed for contacted solid materials, was somehow tested to see if it could be used in nonsolid media. The 3D Hertzian solution was initially employed, and the results were tested using 2D finite elements to determine whether the displacement and Hertz pressure handled from FEA were identical to the initial solutions. When the Hertz and finite element solutions were found to be compatible, other variants of the pavement model with different types of cold longitudinal joints were devised. The static implicit method was employed in each analysis,

as the problem consisted of a mass-to-mass contact problem and steady-state deformation of an unbound medium. The bottom of the model was modeled as constrained with simple supports, and both sides of the theoretical stiff subgrade were confined with simple non-sliding bearings, which simulate the bounded subgrade layer. The granular base, hardened RCC, and fresh side were modeled with no lateral confinement, as the design assumed that the base course and surface courses were constructed on an embankment.

3.8.6. FEA Outputs and Validation

Finite element models were calibrated, particularly regarding the material and geometrical model, as no full-scale experiment was conducted. In essence, it was deemed that the material model employed in the FEAs would be acceptably valid if the Hertzian behavior calculated beforehand against the applied drum load were to be met with the created constitutive model. The only unavoidable source of uncertainty is the assumption that the model, which becomes more complex after adding the base and subbase course to the whole model, exhibits the same material behavior. The target displacement occurring on the top of the layered FEM was first checked against the displacement obtained from Hertz's solution for a single compactable half-plane to pre-validate the model. The representative data presented in the relevant sections were primarily obtained from these critical sections, visually depicted in Figure 59.

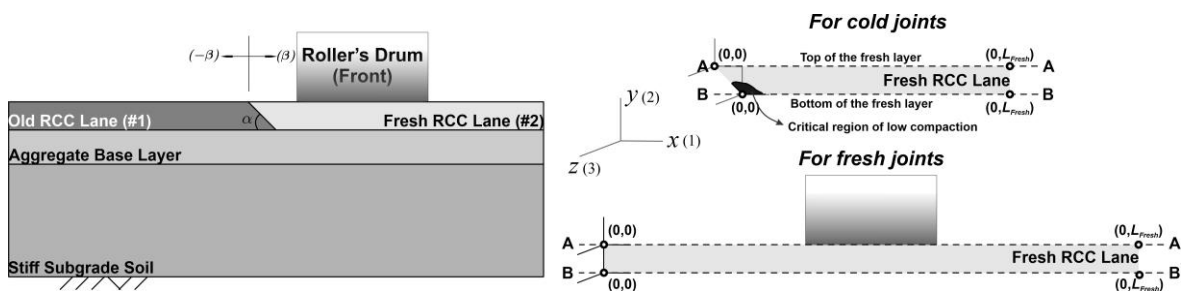


Figure 59. Illustration of critical sections in the finite element model used for comparing deformations across different configurations

The calculated Hertz contact pressure was applied as a line pressure in MPa on the freshly jointed fresh RCC model, and the maximum deformation at the center of the drum load was compared with that from the Hertzian solution, achieving complete agreement. The same approach was also applied to the maximum pressure values, which were successfully

matched. The remaining models were created using the same mechanical and physical properties. After performing analyses for each pavement model with distinct cold longitudinal joint shapes, the deformation values, distribution, and gradients in critical sections were examined.



4. RESULTS AND DISCUSSION

This section presents the results of several studies that have neither been previously conducted nor evaluated. These studies shed light on the solution to the main problem addressed in this thesis. I initially evaluated the validity of the analysis methods for subsequent analyses and then went further. I aimed to draw a clear demarcation line to indicate the engineering problem addressed. Furthermore, I determined the limits of the problem and the validity of the solution methods, thereby providing some apparent remarks and recommendations. All computer operations were performed in a computing environment with the Windows operating system, an Intel(R) Core (TM) i7-7700 CPU @ 3.60GHz processor, and 56 GB DDR4 RAM.

4.1. An Analytical Approach to Evaluating Vibrating Hammers for Molding Roller-Compacted Concrete (RCC) by Simulating Actual Roller Compaction Effects

The main purpose of this study is to analytically determine to what extent the use of electric vibrating hammers, a substitute compaction technique frequently used for molding fresh RCC concrete, represents actual roller compaction, determine the required compaction times, and conduct a scientific scrutinizing of the relevant standard, ASTM C1435 (2014), and make recommendations. It is possibly the first of its kind in this study that it is established to pave the way to compute the substitute compaction energy via an analytical approach. Two rollers, small-to-moderate and large size, and two electric vibratory hammers, one lighter than the other, were hypothetically employed in comparisons. Several transfer plates attached to the electric hammers, used in preparing the laboratory samples, were modeled, produced, and tested. The characteristic physical features of the tested roller and hammers were given in the pertinent chapter of this thesis. The sample computation and their reiterated pathways were also given for representation. First, the energy that the hammer can produce is mainly based on the total mass of the device, which can be defined by three components. The mass of the hammer itself, the mass of the bid used, and the mass of the transfer plate directly help act the dynamic force onto the sample being cast.

The central focus of rigid-body dynamics is the analysis of interdependent systems resisting outside forces. The analysis is made simpler by lowering the number of transport

and rotation parameters that characterize the system for each body under the assumption that the bodies are stiff and do not deform because of applied forces. In that sense, the mass is the core matter for producing the dynamic force with the cyclic (frequency-driven) motion concept. The cyclic motion or amplitude of the rollers was taken from their specification catalogs. Figure 60 illustrates the approaches used to measure the displacement characteristics of the vibrating electric hammer.

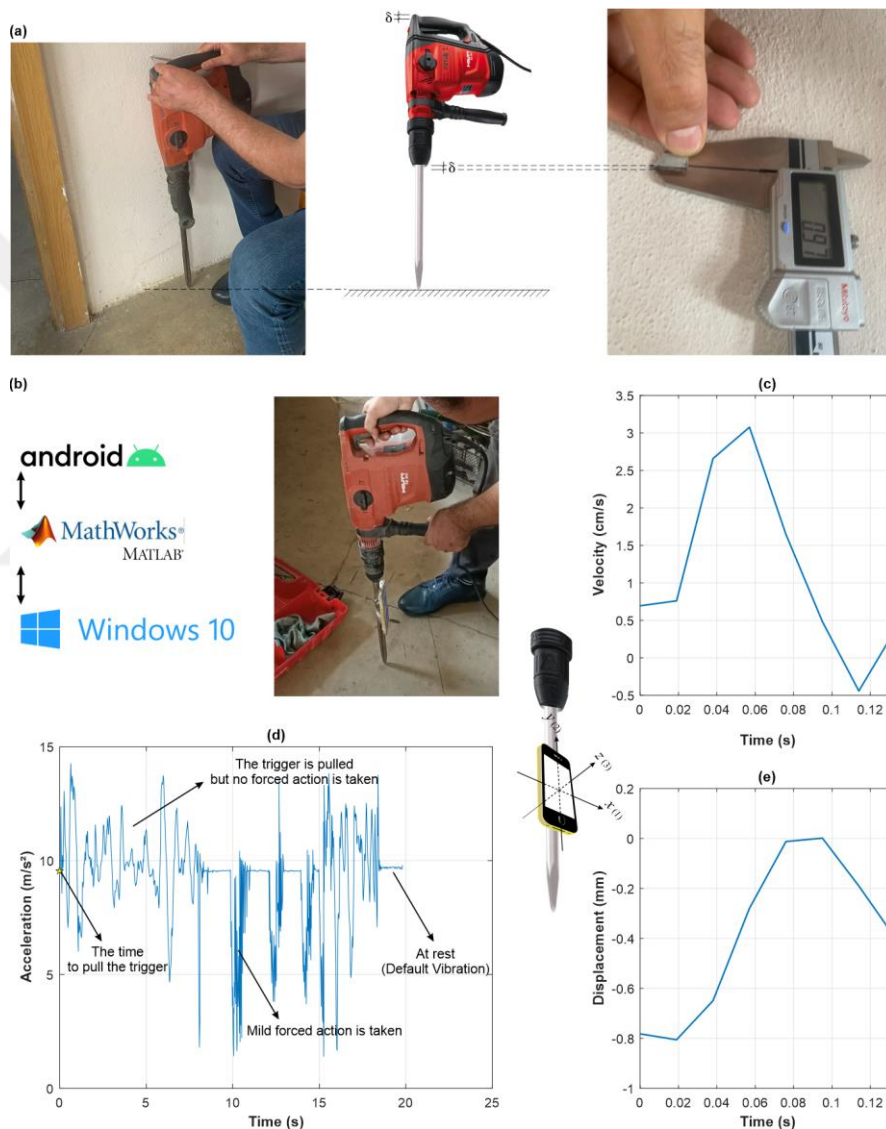


Figure 60. Vibration amplitude measurement when the electric hammer makes initial dynamic impact; (a) manual measurements using a caliper, (b-e) digital measurements using a cell phone's acceleration sensor

The amplitudes of the hammers were not given in the pertinent catalogs. As is known, the bids can move when the device is at rest, whose margins depend on the type, technology,

and brand of the devices. What matters is the amount of amplitude in dynamic conditions. This amount cannot exceed a few millimeters in terms of the human hand's ability to use the tool. For this, the device was placed on a rigid concrete floor in rest mode, and its movement at the moment the trigger was pressed was marked on the level axis (on the wall) and measured with a caliper. Here, approximately 1 mm dynamic amplitude was determined as expected. Essentially, the movement of the bid in the rest state of the instrument was too low to be marked with a pencil. However, in dynamic mode, an amount of amplitude was measured, which already provides the tool with the ability to be used as a demolition hammer and breaker.

Figure 60a denotes the amplitude measurement when the hammer is shifted to the states from the rest and active. Secondly, an Android mobile phone was used as a vibration meter with its built-in accelerometer. Vibration records were captured via MathWorks® Cloud from the MATLAB Mobile application. The phone was affixed to the breaking rig and operated for a specific period. Subsequently, the first vibration mode was determined, considering the hammer's frequency of 55 Hz. Given that the time derivative of the velocity function is acceleration and the time derivative of the position function is the velocity function, the displacement and displacement matrix were computed through time integrations. The relevant measurement steps and spectra are shown in Figure 60b–e.

The second issue that must be intrigued is the effect of masses of extras excluding the hammer's mass. The amplitude and frequency content of the hammer are known from specs. When the bid and tamping plates are attached to the device, the total mass of the entire system increases along with the ascendance of the dynamic force produced. Figure 61 expresses the dynamic force variation produced by the hammer excluded, including the extra masses of bid and varied tamping plates. It is observed from the figure that the incorporation of the extra masses into the bare mass of the hammer makes dramatic changes in the resultant dynamic force produced. It becomes apparent that the changing mass yields rather different force content, which contrastingly affects the substitute compaction time. When the substitute compaction is designed, another factor concerns the physical properties of the actual roller, whose compaction capabilities are to be replicated by the electric hammer. The utmost care was taken in selecting the roller compactor models, and there is no specific purpose in selecting the brands employed. The leading technical benchmark is their obsolete mechanical features. The practical reason is that the city municipality and local transportation department, where I am not off, have compactors and hammers. I wanted to

keep them on standby. Therefore, choices were made considering the opportunity to use these tools in the future in more comprehensive and project-oriented analyses.

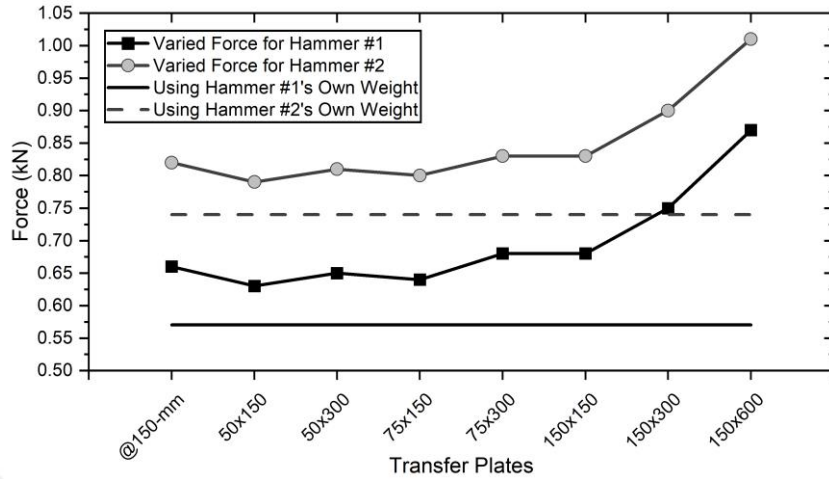


Figure 61. Variation of the dynamic force produced by the hammers in the case of including and excluding masses of the bid and tamping plate

Aside from the amplitude and frequency content of the drums, which are also known for each machine from its specs, the mass and rolling velocity directly change the output compaction performance. The mass of the front and rear drums is either fixed or tailored according to the machine's technical capabilities. In the simplest sense, the mass of the drum is to be known for further computations. I considered the two roller compactor models in the calculations, which are well-known and widely used in the pavement construction industry. Namely, the spec-oriented mass of the front from was employed to be 51,514.33 N (5.25 ton force-metric), while the g is 9.80665 m/s^2 . The two-dimensional Hertzian contact theory is employed for one critical factor: the contact zone between the roller and compactable medium. This study gave the relevant methods and relations within different theoretical contact perspectives. Upon solving the indentation problem of a fully rigid steel cylinder on an elastic half-space, the data shown in Figure 62 was obtained. Figure 62 (a) denotes the b values, which are the widths of the contact areas calculated through the physical properties of Rollers # 1 and 2. Figure 62 (b) shows the indentation amount obtained from the same calculations. The contact surface is essentially an ellipsoid whose second dimension is much larger than its first. This area was considered a rectangular zone for simplification, as is commonly employed. The margin of the load transferring toward the half-space was kept fixed, assuming that the static mode of the roller was active. Therefore, considering the

model, the stress occurring under the drum is nothing but a load distribution spread over the area. When we consider this load distribution as a linear load to model the situation in the middle of the entire section, we represent the condition at the drum's center.

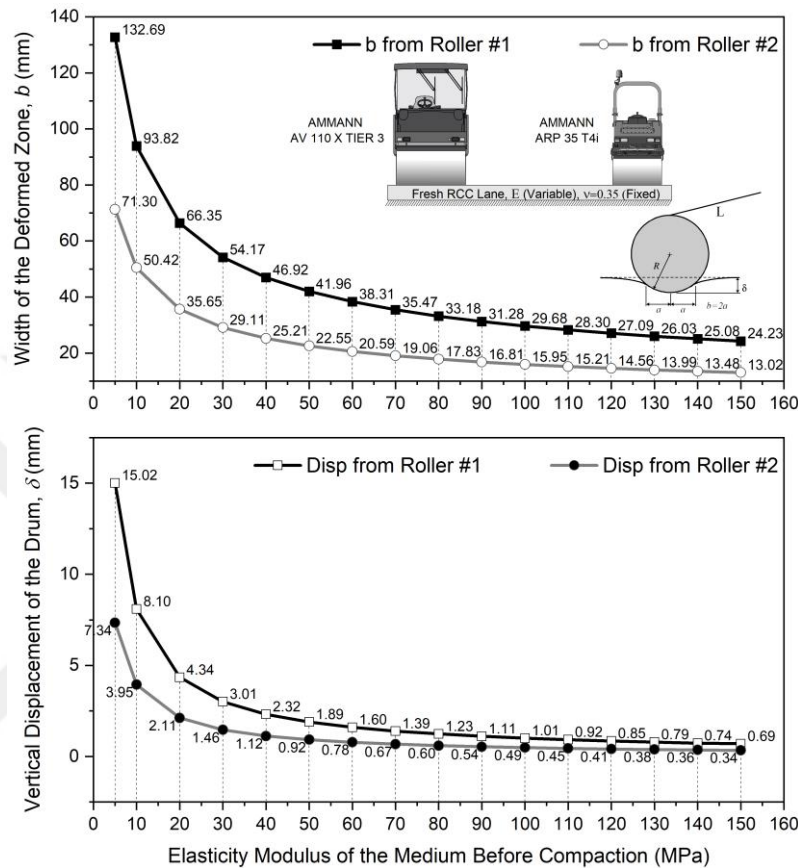


Figure 62. Variation of b and displacement values obtained from varied contact problem solved within Hertz theory

Another critical factor regarding the roller compactor is the speed of the roller's motion. Withrow (1988) noted that the maximum allowable roller speed for adequate compaction is approximately 3.2 km/h. If the roller is operated at a significantly higher speed than this threshold value, increasing the number of passes becomes demanded in order to obtain the desired result of uniform compaction. It is known that the roller drivers set the velocity margin up to the consistency of the layer being compacted. The main objective is to rapidly assess the consistency and indentation depth so that the adopted velocity does not produce ripples or undulations on the surface RCC mat. Namely, the roller speed is not the same in every project and is worth testing in simulations. In that sense, varying roller velocities of 1, 2, and 3 km/h were considered in iterative calculations. Now that all the

prerequisites are fulfilled, the substitute compaction times, which emulate the one-pass of the front drum of the roller compactor, were computed and plotted in Figure 63. Since the necessary point-by-point computations were made in the relevant section, only the graphical illustration and interpretation of the results are presented here. As shown in Figure 63, by adopting the model compactors (AV 110 X and ARP 35 T4i), it is possible to produce the same compaction energy per unit area as that created by the front (also by the rear) drum of the roller compactor on a fresh concrete layer laid by the paver, using electric vibration hammers (TE 60-AVR and HR 5201C). Figure 63 presents a comprehensive analysis of substitute compaction methods using different hammers to simulate roller compaction in fresh RCC. It compares the effectiveness of two different hammers (Hammer #1 and Hammer #2) in simulating two different rollers (Roller #1 and Roller #2) at various speeds and with different transfer plate configurations. The analysis is organized by roller speed (1 km/h, 2 km/h, and 3 km/h) and various transfer plate dimensions, ranging from 50x150 mm to 150x600 mm. A single pass of a single drum of the rollers was analyzed. The complete round-trip analysis is given separately below. Hammer #2 consistently shows higher values than Hammer #1 for all configurations, suggesting it may be more effective in simulating roller compaction. The simulated values for Roller #1 are significantly higher than those for Roller #2, indicating that Roller #1 is a more efficient compactor. As the speed of the simulated roller increases from 1 km/h to 3 km/h, the required compaction time values decrease, implying that slower roller speeds may result in better compaction. Larger transfer plate sizes result in higher required compaction times, with the 150x600 mm plate consistently showing the highest values across all configurations, as expected. Furthermore, the data variation presented in Figure 63 suggests that current standard methods, such as ASTM C1435, may need more comprehensive revision to account for these variables. This could lead to more accurate and reliable laboratory testing procedures that better reflect the compaction achieved by different roller types and operating conditions in the field. All things considered, the difference between the required compaction times for each hammer-roller coupling was rather significant. While the mass of Hammer #1 is less than that of Hammer #2, the required time for Hammer #1 to impart the same energy for each roller model tested is lower than that needed for Hammer #2. Under all circumstances, this study showed that equivalent energy could be transferred onto a mold using a transfer plate, though the required time for robust compaction was slightly longer than that of a model roller compactor.

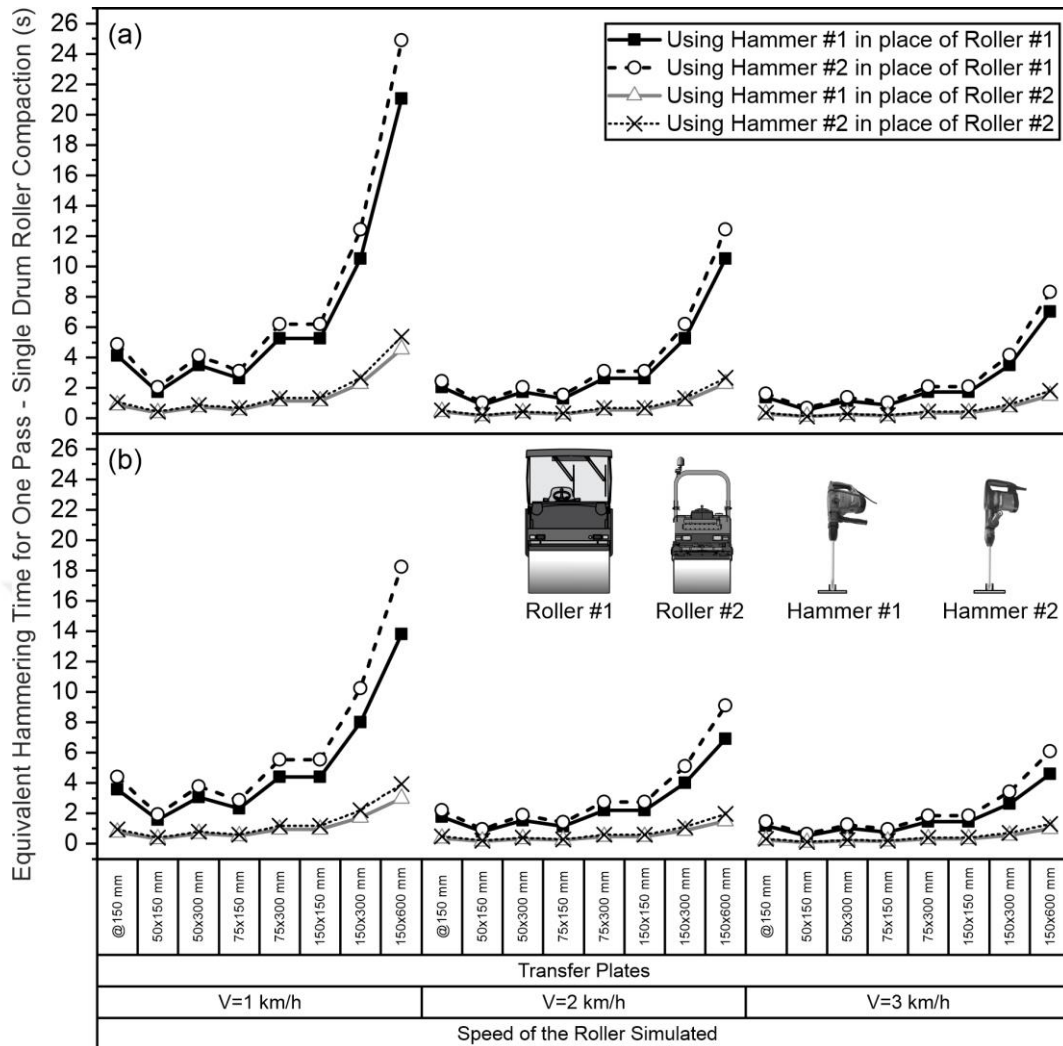


Figure 63. Substitute compaction times for different electric hammers simulating a single pass of the front drum of the roller compactors

For example, assuming a 1 km/h speed for each roller tested, a 4.41-second compaction effort can theoretically generate the same energy if a 150 x 150 mm steel transfer plate is used with Hammer #1 instead of Roller #1. The required compaction time for the same-sized mold is 5.55 seconds when using Hammer #2 in place of Roller #1. When all other circumstances are kept fixed, the required compaction time for the 150-mm diameter plate is 3.57 s when using Hammer #1 in place of Roller #1 and 4.41 s for Hammer #2 in place of Roller #1. It is apparent to the naked eye that Roller #2, which has almost one-third the mass of Roller #1, can be deputized by each hammer's effort in rather reduced times compared to those needed for Roller #1. When 150 x 600-mm plates are used, a 13.79-s compaction time is needed to operate Hammer #1 in place of Roller #1 and 18.23 s for Hammer #2 in place of Roller #1. When the same computation is performed for Roller #2, the required

compaction times are 3.99 s and 3.95 s for Hammer #1 and #2, respectively. As the speed of the roller is increased from 1 to 2 and 3 km/h, the required compaction time for each hammer decreases dramatically. This is consistent with the rolling mechanics: as rollers operate at higher speeds, the contact time is reduced, thereby reducing the amount of energy imparted to the compacted medium. The maximum compaction time for each layer when casting samples according to ASTM C1435 is 20 s, provided that the mortar ring does not appear over the plate during the tamping of the transfer plate. Three typical in-mold layers are cast and compacted, followed by one overtopping layer, which receives an extra compaction. ASTM C1435 aims to replicate in-field roller compaction, and it is known that several round trips are typically needed to finalize compaction in the field. In this context, the analysis made for a single drum replication given above should be generalized to states that can epitomize the round trip of the full tandem roller. What needs to be done is to first calculate the energy for a single drum of a two-drum machine and then calculate the hammering time required to produce the total energy transferred from each roller to the fresh RCC, assuming four drum passes as a complete round trip. The necessary calculations were made parametrically, and the equivalent compaction times obtained are given in Figure 64.

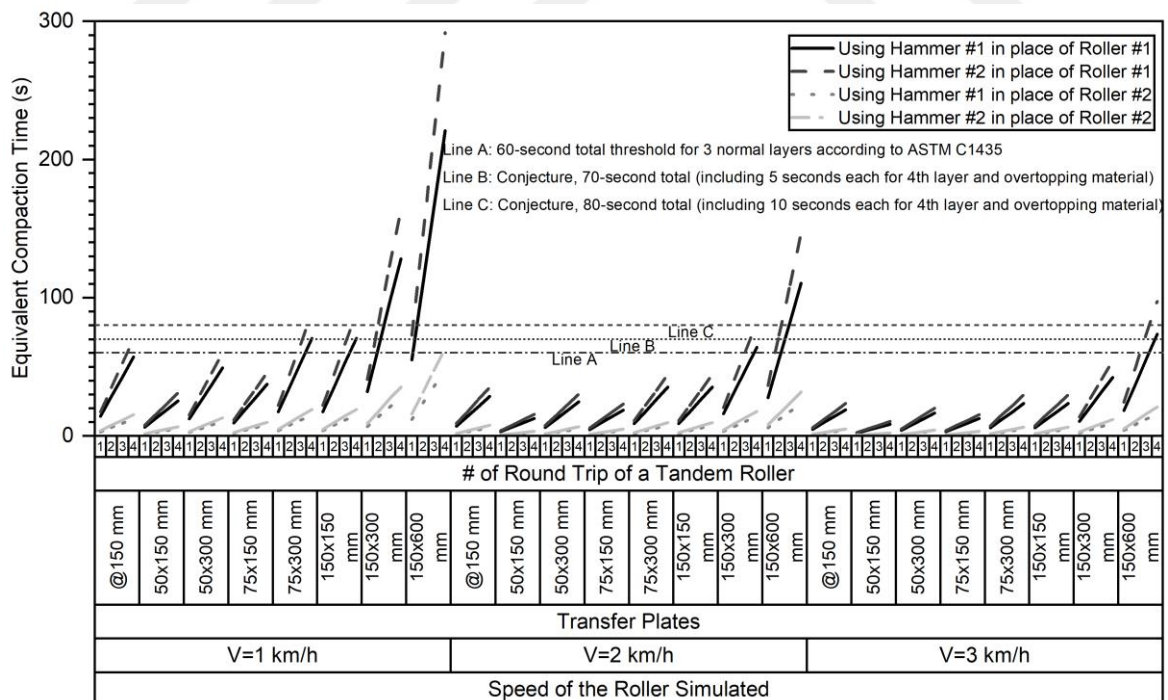


Figure 64. Required substitute compaction time using vibrating electric hammers in place of full-sized rollers, accounting for the number of trips of the tandem rollers

The dashed-dotted line shows the 60 s threshold value. This value is specified in ASTM C1435 as the maximum time allowed for the compression of a 150-mm diameter cylindrical sample for only 3 layers. No specific time limit is mentioned in ASTM C1435 for the fourth layer and overtopping. We can state that the 60 s compaction for the normal three layers is the maximum. The fourth and overtopping layers require less time to compact because the three-layer RCC is already molded and has created a stiff bed. We can infer from practical implementations that 5 s each for the fourth layer and overtopping material to consolidate is sufficient. An additional line is plotted as a conjecture using 10 seconds each for the 4th layer and overtopping material.

The size of the plates and the molds were considered identical in the calculations, and nominal tolerance was not used. This approach was chosen because dimensions given with tolerance (146 ± 3 mm [$5 \frac{3}{4}$ - $6 \frac{1}{8}$ in.]) might confuse the reader and obscure the subject's essence. When examining Figure 64, the 60-second line for the required total time (Line A) is achieved in 4 full turns for Roller #1 for a 150 mm diameter plate at a speed of 1 km/h (disregarding nominal tolerance). However, when the roller speed increases to 2 and 3 km/h for the same plate, the required substitute compaction time remains well below lines A, B, and C. With these characteristics, the standard is only somewhat effective in simulating a low-speed and high-weight machine, but it predicts excessive compaction time when the operation speed of the roller increases. It is observed that Hammer #1 can produce the same compaction energy in a shorter time than Hammer #2, despite its lower mass. This difference becomes especially significant as the surface area of the sample mold to be compacted (or the tamping plate area) increases, and the efficiency difference between Hammer #1 and Hammer #2 grows considerably. On the other hand, the required compaction times for beam molds, such as 150 x 300 mm and 150 x 600 mm, are quite high, and it is debatable how much of these times are used for compaction in the laboratory or the field. While beam samples are compacted with a large plate, it has been observed in various settings that compaction is generally stopped when the plate is not surrounded by mortar, resulting in shorter periods than those shown in the calculations. Conversely, the required compaction times for small-sized sample molds are generally much less than those recommended for the standard cylinder and calculated here. In this case, contrary to large-sized samples, excessive compaction occurs.

Forssblad (1980) demonstrated a correlation and interchangeability between compaction methods, although the potential effects vary depending on the material being

compacted. Schrader (1992) demonstrated that the interrelationship between percent air-free density and achievable strength at maximum density in RCC mixtures exists, and they have been strongly correlated, highlighting the importance of robust compaction in reducing air content in the compacted medium. This is why most specifications require a minimum density of at least 98 percent of the maximum density, often determined using a modified Proctor test. Chhorn & Lee (2018) emphasized the importance of attaining 98% of the maximum dry density of the RCC mixture during compaction to achieve the desired physical and mechanical properties. Kurita et al. (1988) conducted laboratory and field compaction studies on RCC. They observed that acceleration parallel to the roller's movement and in the vertical direction occurs within the concrete mat, contributing to its ultimate consolidation. Similar to laboratory tests, the surface settlement ratio followed a curve. They noted that a 7% surface settlement ratio was sufficient for dense concrete consolidation under the studied construction conditions. Using the original field data of Kurita et al. (1988), I developed two new nonlinear relationships presented in Figure 65 to better predict the RCC layer's settlement ratio based on its thickness and the number of roller compactor passes.

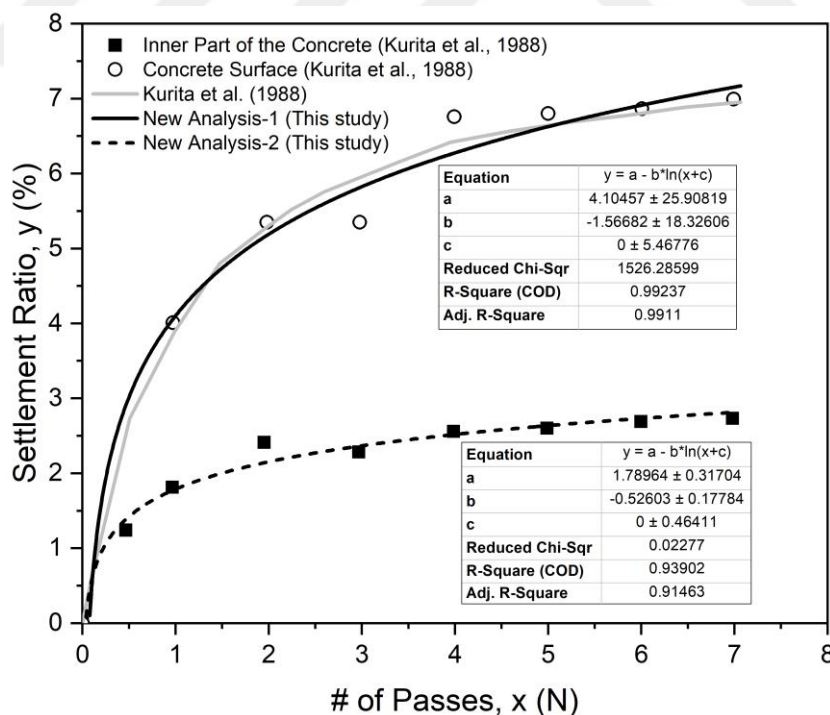


Figure 65. Relationships between # the roller pass against settlement ratio of the RCC mat (adapted from Kurita et al. (1988))

Casagrande (1936) demonstrated that one of the fundamental principles in soil dynamics is that initially loose and dense soils under the same vertical stress tend to approach the same relative density when subjected to shear stress. To put it simply, while dense soils exhibit loosening behavior under shear stress, loose soils show relatively tightening behavior. Consequently, it can be claimed that if the compaction process of fresh RCC continues beyond a certain number of passes, the previously dense matrix due to roller compaction may loosen, resulting in decreased overall strength.

Withrow (1988) identified key factors influencing the compaction of RCC using vibrating rollers. Primary factors include static axle loading and vibrating amplitude. Secondary considerations encompass vibrating mass, roller dimensions, frequency, and rolling speed. Material characteristics also affect compaction, including grain size and distribution, moisture content, and lift thickness. Similarly, Nanni et al. (1996) highlighted that paver speed, rolling techniques, and moving (rolling) patterns significantly impact load transfer efficiency. Schrader (2003) conducted a comprehensive investigation into the limitations of various compaction techniques (e.g., pneumatic tamper, Hilti Kango, Vebe vibration table, internal vibrator, and rodding) compared to 10-tonne vibratory rollers, examining their potential effects on optimum water content, compressive strengths, and unit weights of RCC mixes. Schrader (2003) concluded that the compaction technique significantly influences compaction performance. Efficient compaction is crucial, as excessive voids can reduce concrete strength and weaken the matrix (ACI, 2016). According to the American Concrete Institute (ACI, 2000), efficient compaction can reduce the total air content in the concrete matrix by one percent, potentially preventing a 5% decrease in compressive strength. Rollings (1988b) noted that improper compaction, particularly at the bottom of an RCC pavement, can reduce strength by 25%. Given that the bottom part of the RCC mat is critical for tensile stress resistance, ensuring efficient compaction from top to bottom is essential.

The choice of a roller is crucial for compaction performance, affecting the expected results in laying down the RCC under actual conditions. Evaluating how well molding and electric hammer compaction simulate the actual rolling procedure is essential, highlighting the relevance of the thesis problem of this study. Hirose & Yanagida (1984) established a relationship between vibratory drums' amplitude and frequency modes and their influence on density ratios. They found that contact area, drum speed, frequency, amplitude, and the number of passes significantly affect vibration energy. Figure 66 presents computed

theoretical vibration energy and density ratio for the roller compactor employed in this study, with overlaid original data and currently improved relationships.

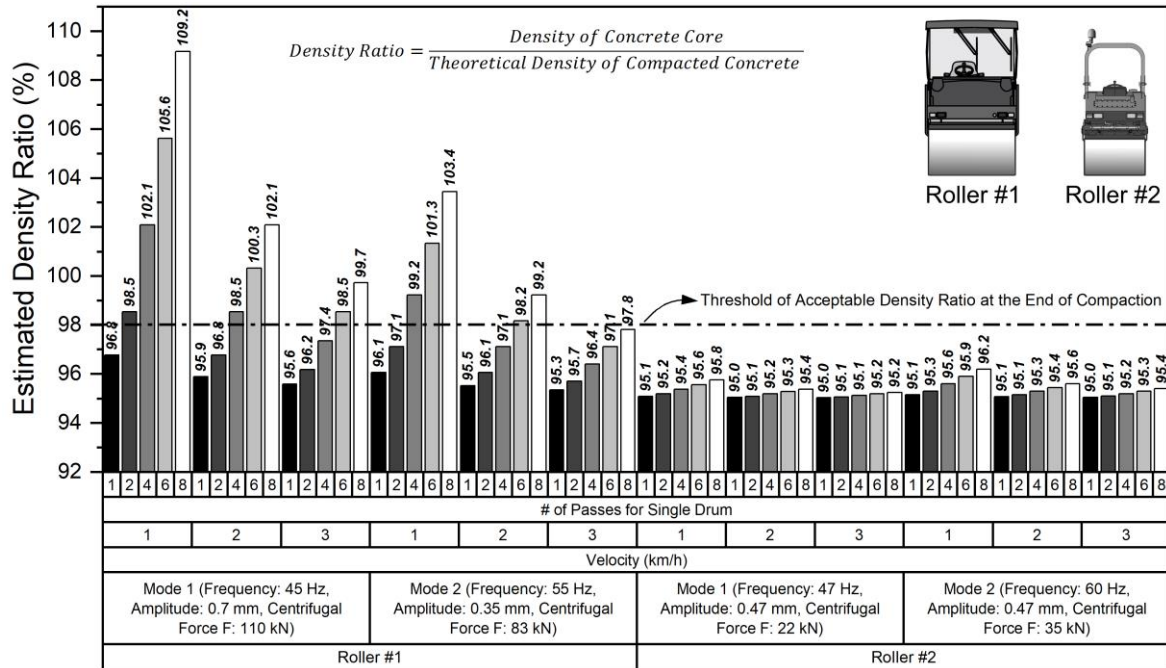


Figure 66. Estimated density ratios against dynamic compaction performance of the Roller #1 and 2

When 98% compaction performance, or the theoretical density ratio value, is regarded as a threshold, the performances of two different rollers theoretically tested in all variations are clearly shown in Figure 66, represented by a dashed-dotted line. Roller #1 is fundamentally a conventional compactor used in RCC pavements with its approximately 10.4-ton-force roller. Roller #2 is a light tandem roller with a 3.2-ton-force load in total. The effect created by a single drum was considered in the calculations. Therefore, two passes essentially represent the total effect of the machine's two drums at once, and eight passes are defined as two full tours when viewed as a round trip. Roller #2, being lighter, cannot provide the desired 98% compaction threshold. Although Roller #1 has a heavier mass than Roller #2, it cannot provide sufficient compression energy with a drum effect in every mode and at every speed to yield a 98% density ratio. For Roller #1, a sufficient density ratio can be achieved with just a single drum with two passes after one forward movement of the whole machine, only at the lowest speed of 1km/h. A single drum provides a sufficient density ratio value at 1 and 2 km/h speeds in Mode #1 with four passes (one complete tour of the entire machine), but not at 3 km/h speed. In Mode #2, the sufficient ratio is provided only at 1 km/h

speed with four passes, not at other speeds. The desired density ratio can be achieved in six passes with a single drum for Roller #1 in all alternatives in Mode #1, and in all alternatives except 3 km/h for Mode #2. With a single drum, in eight passes (two complete rounds of the whole machine), the desired density ratio is achieved at each speed value, as we round the value at 3 km/h (97.8%) to 98%. It is predictable that the roller's total mass is effective under static conditions. However, this analysis shows that both dead and centrifugal loads in vibrating mode also affect performance. There is a significant, though not dramatic, difference between the two operation modes of each machine. Mode #1, with low frequency and large amplitude, was found to be more effective, although Mode #2 also creates a sufficient effect. While it is well-known that the number of drum passes is rather effective, this analysis demonstrated that when a certain number of passes is achieved for any variation applied, a 98% density ratio can be attained by Roller #1. The machine's speed appears to be another critical factor. When 3.2 km/h is accepted as the maximum operating speed in RCC, upon the suggestion of Withrow (1988), a better effect can be created at speeds lower than this threshold. However, a sufficient density ratio can be achieved in all speed modes, including the maximum tested speed of 3 km/h, provided the conditions are met. It is evident that Roller #1 outperforms Roller #2 in achieving a better density ratio.

Another factor that should be discussed is the lift thickness or, in other words, the nominal thickness of the RCC lane paved by the screed. There is no shadow of a doubt that the thickness of the compacted medium affects the roller's compaction performance, regardless of whether it is heavy or somewhat lighter. Recommended lift thicknesses in RCCPs can vary, but a commonly adopted thickness for compacted RCC is around 200–250 mm (9–10 inches) (ACI, 1995; Harrington et al., 2010). The European Ready Mixed Concrete Organization (ERMCO, 2013) guide specifies that the RCC thickness tolerance should be within ± 15 mm of the pavement's design height to avoid water ponding. If the required RCC thickness exceeds the 250–300 mm limit, the layer can be divided into multiple layers, as Choi & Hansen (2005) and Zarrin & Shirazi (2015) recommended. Withrow (1988) suggested dividing compacted lifts into two sections: a static load of 20 kg/cm for lifts up to 150 mm and a minimum static load of 27 kg/cm for thicker lifts. This observation signifies that as the lift thickness increases, the required load capacity of the roller also increases.

Alternative techniques are also available for both RCC production and molding. However, some methods, particularly the use of electric vibrating hammers, stand out and

are widely preferred. Shabani et al. (2021) proposed using the Superpave Gyratory Compactor (SGC) for designing and molding the RCC. They noted that while the gyration level has minimal influence on mechanical capacity, the SGC significantly impacts the behavior of the fresh mixture. Selvam et al. (2023) compared the compaction performance of various laboratory methods (vibratory hammer, vibratory table, modified Proctor, and gyratory compactor) to actual infield compaction performed by rollers. Their research revealed that a vibrating hammer is the most efficient for replicating field compaction, leading to improved aggregate dispersion and higher strength values.

The aggregate properties in the fresh mix, such as maximum nominal aggregate diameter, coarse and fine aggregate angularity, gradation pattern, and potential behavior upon hydration, are often overlooked. Even though the amount is less than that of conventional plain concrete, the water/cement ratio and the amount of cement will also affect the consistency of the paste, thereby influencing the formation of the matrix. The use of fiber additives also affects the workability of fresh concrete, and future standards should consider these factors. While various parameters can be managed in each project, they should be robustly specified in a code. Future regulations should extend coverage to fully encompass the physics of the actual problem, especially regarding compaction.

The findings reported here shed fresh light on the use of substitute compaction in place of actual roller compaction. One of the more significant findings to emerge from this study is the apparent need for revising ASTM C1435 (2014) to include more comprehensive guidelines and reduce uncertainties. The issue of substitute compaction is intriguing and could be usefully explored in further research, highlighting the necessity for large-scale studies to support revisions on the standard. The original remarks given here have important implications for future practice. The information from this study can improve the relevant standards and develop targeted interventions. While the vibrating table technique governed by ASTM C1176 (2013) was not studied here, it is known that it can decrease overall strength and density (Tayabji & Okamoto, 1987). This study emphasizes the need for improved standards to ensure accurate simulation of field conditions and reduce measurement uncertainties.

In conclusion, this study has shown that it is possible to use electric hammers as a substitute for roller compactors in molding RCC samples. The compression time required for a one-time pass of a single drum of the roller compactor for various models used was calculated and provided. This supports the assertion that electric vibrating hammers can

deputize for roller compactors when roller compaction is not available. It is also important to note that the standard cylinder sample container is filled three times including an overtopping and compacted four times according to ASTM 1435, potentially leading to over-compaction. Various tools with different specifications can produce different amounts of compaction energy, increasing the measurement uncertainty of the molding and testing process. Additionally, it is difficult to compact a 300 mm high sample in one shift with equivalent loading of a vibration hammer in both theoretical and practical manners. Choi & Hansen (2005) noted that the vibratory hammer technique (ASTM C1435, 2014) provides fairly good compaction compared to effort conducted with a 10-ton vibratory roller under real-world conditions in the field. Based on the results of a comprehensive comparative analysis, the present study recommends revising ASTM C1435 to include more comprehensive guidelines and provide a universal scope. The standard should include closed analytical formulas regarding the instrument's properties, the sample mold's diversity, and the fresh concrete's fundamental physical properties. Lift thickness limits should be specified to help users understand the extent to which the actual roller is modeled. Further research is needed to support these revisions and ensure accurate and reliable results in RCC compaction and testing.

4.2. Optimization of Fresh-State Roller-Compactable Concrete (RCC) Strength Parameters: Enhancing Compaction Simulations with Diverse Rollers and Displacement Targets

This study provides a quantitative basis for understanding the role of roller weight in RCC pavement compaction. Elucidating the relationships between roller characteristics, material properties, and compaction outcomes aims to contribute to the optimization realm of RCC pavement construction practices. These insights can inform equipment selection, mix design, and construction planning, ultimately leading to a more efficient and effective RCC pavement compaction process. The comprehensive analysis of the two compaction scenarios, Case #1 (heavy roller) and Case #2 (light roller), reveals significant insights into the mechanics of RCC compaction.

A MATLAB code used in the computations and presented in the appendix section of this study is a comprehensive implementation of a single-objective optimization algorithm designed to investigate the elastic properties of materials under Hertzian contact conditions.

The primary goal is to determine the elastic modulus (E) and Poisson's ratio (ν) of a frictionless elastic half-space subjected to indentation by a cylindrical front drum under constant load. Two roller compactors (AMMANN AV 110 X and AMMANN ARP 35 T4i) with distinct mechanical characteristics were selected from real-world applications. These compactors were intentionally chosen due to their widespread use in asphalt and soil compaction tasks, regardless of the brand. Several other candidate rollers could perform the same functions as those selected. Moreover, given their availability in our region, these machines were considered for potential practical use in future iterations of this study. The computational approach is particularly relevant for characterizing materials in various engineering applications, including tire-road interactions, roller compaction processes, and material testing.

The optimization process is rooted in Hertzian contact theory, which provides a framework for understanding the deformation and stress distribution in elastic bodies under contact. The code systematically investigates a range of target displacements, from $d_{\text{target_lower}}$ (0.5 mm) to $d_{\text{target_upper}}$ (20 mm), with 500 discrete points (num_arrays). The code's design allows for the customization of target values and the number of discrete points to accommodate user preferences, enabling the creation of search spaces of any desired size for optimized parameter pairs. For each target displacement, the algorithm seeks to find the optimal combination of E and ν that would result in the observed deformation. The optimization process is primarily driven by MATLAB's "fmincon" function, a specialized solver for constrained nonlinear optimization problems. This is further enhanced by embedding it within a GlobalSearch framework, which helps to avoid local minima and find the global optimum. The optimization variables, E and ν , are constrained within realistic bounds: E is limited between 1 MPa and 120 MPa, while ν is constrained between 0.31 and 0.40, reflecting typical ranges for road foundation soil materials. The objective function, a crucial component of the optimization process, is the absolute difference between the calculated and target displacements. The calculated displacement is derived from a logarithmic formulation of the Hertzian contact equation. This formulation accounts for the cylindrical geometry of the contact and provides a more accurate representation of the deformation compared to simpler Hertzian models.

The code generates several visualizations to aid in interpreting results: (i) A panel plot showing the optimized E and ν values versus target displacement. (ii) A 2D colormap

illustrating the relationship between target displacement, optimized E , and the corresponding ν values. (iii) Two heatmaps, one displaying the distribution of ν values across the E -displacement space and another showing the distribution of E values across the ν -displacement space. These visualizations offer insights into the sensitivity of the optimized parameters to changes in target displacement and help identify trends or patterns in the material behavior. In addition to the primary optimization results, the code computes several related parameters of interest in contact mechanics: (j) The equivalent elastic modulus (E^*), which combines the elastic properties of both contacting bodies. (jj) The half-width of the contact zone (a), providing information about the extent of the contact area. (jjj) The maximum Hertzian pressure (P_{max}) in the contact zone. These supplementary parameters significantly augment the analytical framework, thereby facilitating a more exhaustive characterization of the contact scenario. The code also incorporates several practical features to facilitate further analysis and reporting: (k) Results are saved in multiple formats (Excel, CSV, and plain text file) for easy access and post-processing. (kk) All generated figures are saved in high-resolution formats (JPEG, EPS, and PDF) suitable for publication. (kkk) The code includes customizable parameters for figure formatting, such as printing resolution (dpi, Dots per Inch), font sizes, and axis labels. From a computational perspective, the code is optimized for performance, utilizing vectorized operations where possible and employing efficient data structures. The use of pre-allocation for result arrays ($E_{optimized}$ and $\nu_{optimized}$) ensures efficient memory usage, which is particularly important if a large number of optimization runs are performed.

Calculations were conducted for two distinct scenarios to evaluate compaction methods for RCC pavements. The first scenario involves using a 10-ton roller compactor, which is widely recommended and has become standard practice for RCC pavements. The second scenario examines the use of a smaller compactor with a mass of 3.2 tons. Including the smaller compactor as an alternative to the larger one analytically tests an assumption commonly held in practice. By solving and comparing both scenarios, I aim to evaluate the prevalent notion among engineers that lower-weight compactors can achieve comparable results to their heavier counterparts. Both scenarios were analyzed under the assumption that the compactors were operated in static mode. While using the vibratory mode is an everyday choice in RCC pavement construction, some engineers, drawing from their field experiences, posit that relatively loose fresh RCC material with high compression capacity may segregate

in vibratory mode. Although this specific issue falls outside the scope of this research, it is directly related to the consistency and compaction values of the laid and compacted material. It is worth noting that static mode is also employed in a method frequently used in HMA pavements, known as “rolling from the cold side.” In this technique, the roller operates on the cold lane, overlapping the fresh material by approximately 152 mm (6 inches). Figure 67 presents the initial optimization results generated by our computational model for both Case #1 and Case #2.

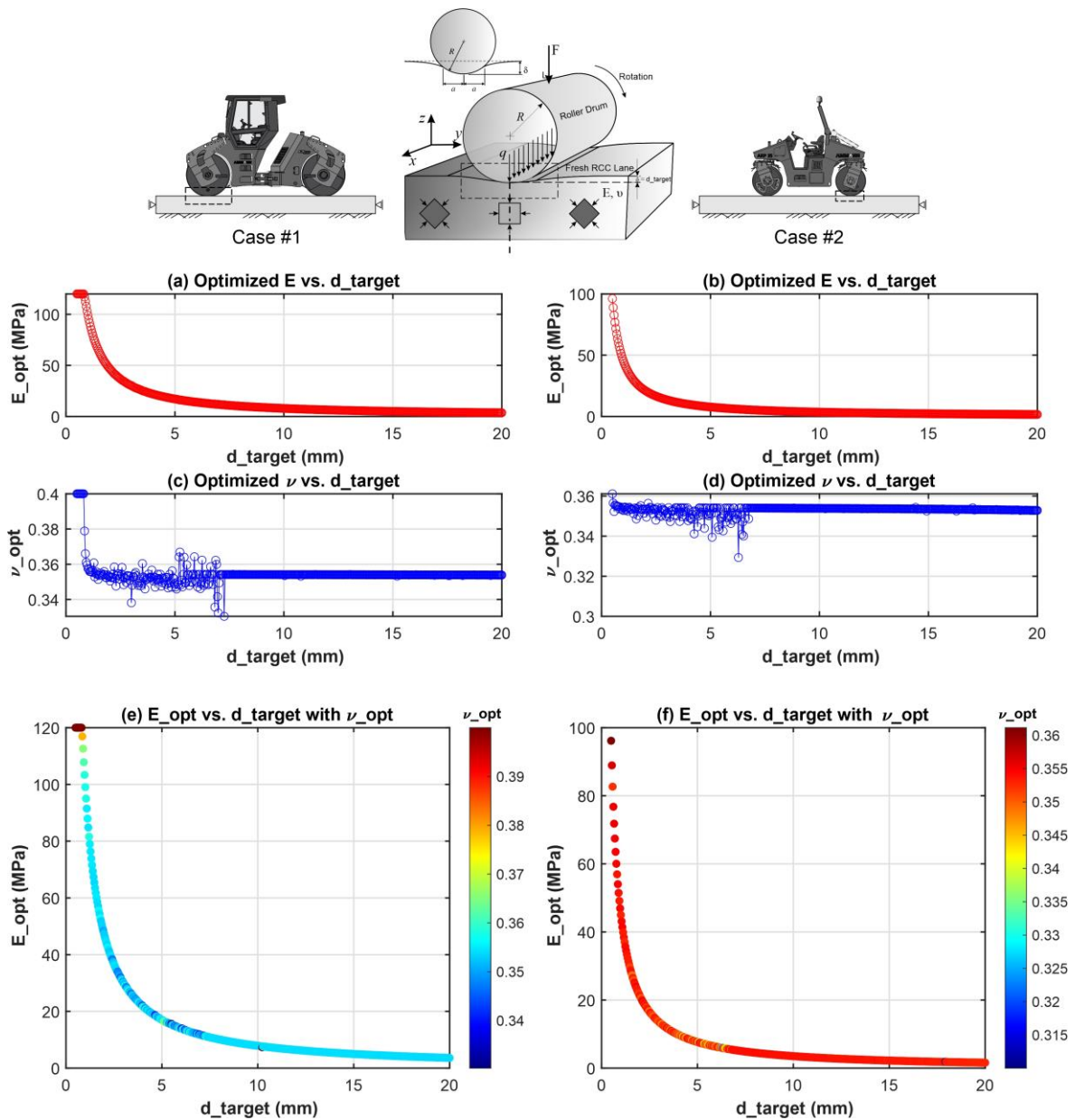


Figure 67. Initial results of the optimization algorithm applied for Case #1 and Case #2

Analysis of these results reveals several key observations. The elasticity modulus in both cases (panels a and b) shows no dispersion throughout the calculation range. A nonlinear pattern in the elasticity modulus is observed as deformation increases. Both cases exhibit intense dispersion in the Poisson's ratio within the search space up to a target deformation of approximately 7-7.5 mm. Beyond this point, the Poisson's ratio becomes saturated at higher deformation values. While the array size for both Poisson's ratio and elasticity modulus is identical, the tested values for Poisson's ratio are confined to a narrow range between 0.31 and 0.40, resulting in very close values being examined. When we visualize the Poisson's ratio using the optimized $E - \nu$ pairs (panels e and f), we observe distinct patterns for each case. For case #1 (large rollers), the optimum values are found across a broader range. For case #2 (smaller rollers), the pair values are determined within a more constrained range. This disparity in the optimization ranges between the two cases suggests that the choice of roller size may significantly influence the material's behavior during compaction. The computational model employed also generates heat maps for both cases to provide a more detailed visualization of the optimum values. A heat map is a two-dimensional visual representation of data where individual values within a matrix are encoded as colors. In Figures 68a and b, the maximum Poisson's ratio determined for Case #1 was 0.379, while it was 0.357 for Case #2 within the same deformation range. At the maximum deformation, the Poisson's ratio determined for Case #1 was 0.354, while it was 0.353 for Case #2. As we move towards higher deformation amounts, the Poisson's ratio values determined for the two cases converge. Figures 68c and d show grouping results for the elasticity modulus (E) from the optimized $E - \nu$ pairs. In the lowest deformation group, the elasticity modulus of the compactable medium that can be analyzed in Case #1 is determined as 116.94 MPa, while it is 96.16 MPa for Case #2. At the maximum deformation limits, the elasticity modulus of the medium being compacted in Case #1 is determined as 3.77 MPa, while it is 1.66 MPa for Case #2. This finding indicates that the large roller can effectively compact the layer with an elasticity modulus approximately three times larger than that which a small roller can compact. Furthermore, when the target deformation for the compacted layer approaches the upper limits, the heavy roller outperforms the light roller, demonstrating its ability to compact a layer with an elasticity modulus roughly two times larger than in Case #2.

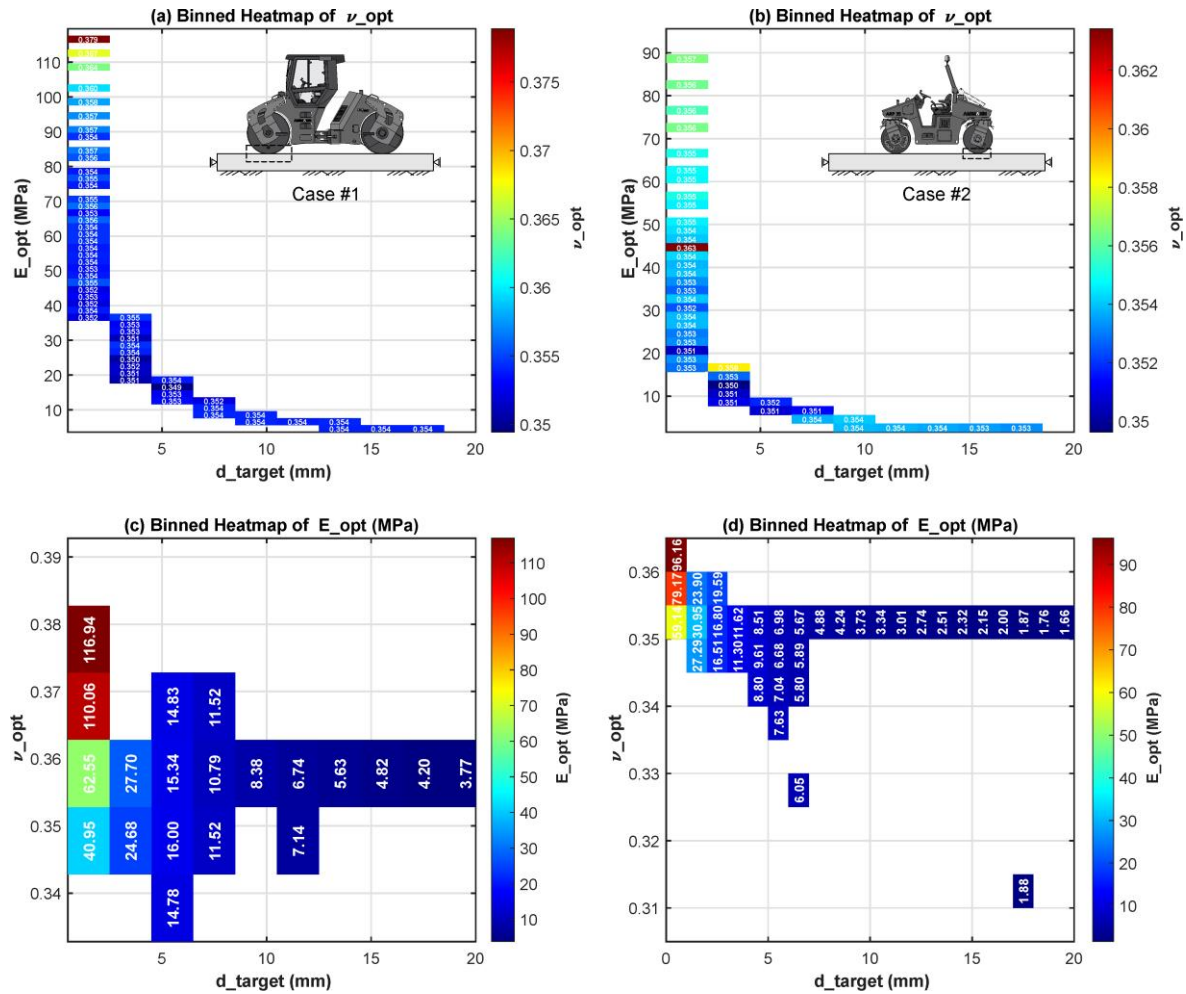


Figure 68. Heat maps generated for Case #1 and Case #2

The analysis has yielded pairs of elasticity modulus (E) and Poisson's ratio (ν) determined across a comprehensive search space for two rollers with similar characteristics but differing compaction capacities. By plotting the results obtained for Case #1 and Case #2 on the same figure, we can delineate the area encompassing data for intermediate cases; scenarios where specified rollers lighter than heavy rollers but heavier than light rollers are used for identical target compaction amounts.

Figure 69 illustrates these comparative results. Figure 69a plots the target deformation amounts against the maximum Hertzian stresses, clearly defining the region for intermediate cases. Case #1 consistently produces higher stress than Case #2 for equivalent deformation values. The difference in stress values between the two cases is approximately 50 percent for both low and high deformation targets.

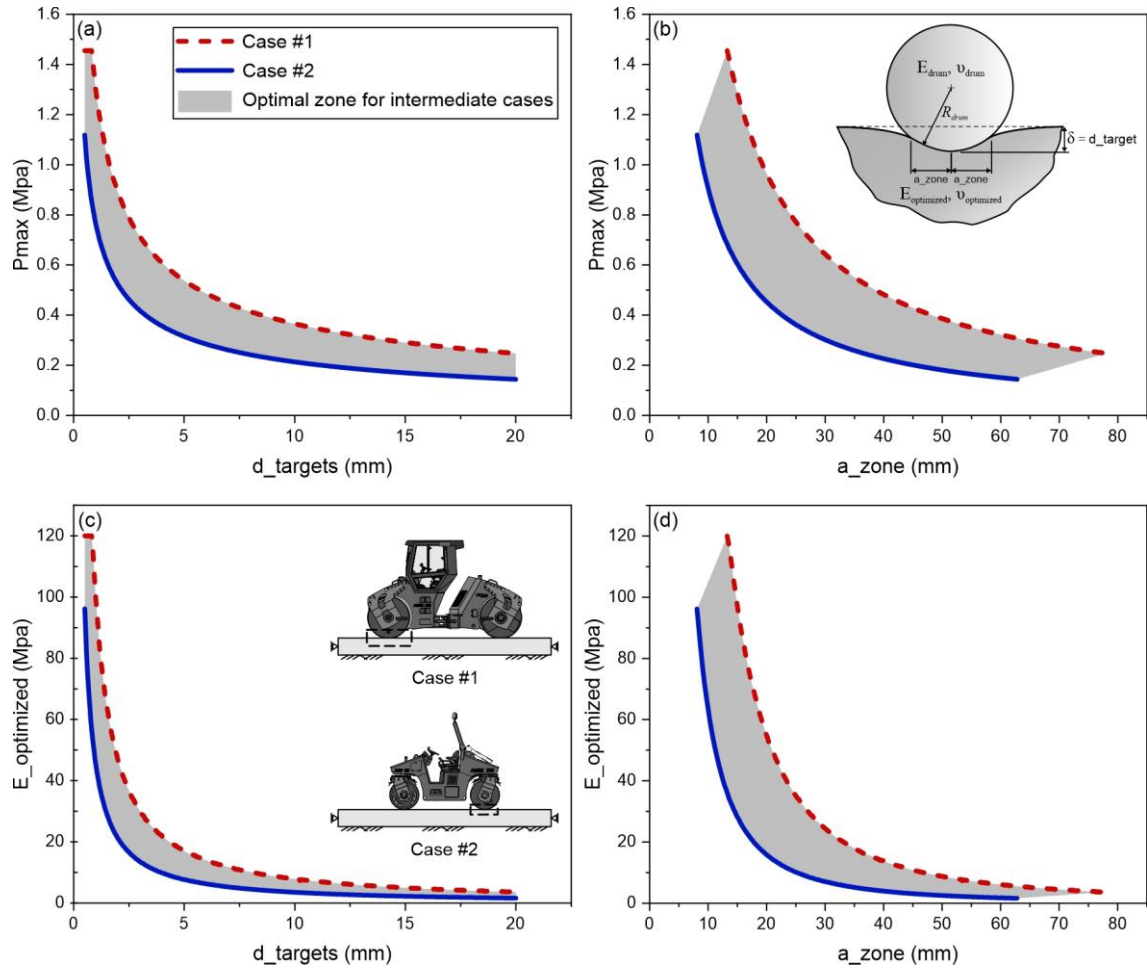


Figure 69. Intermediate optimum regions bounded by Case #1 and Case #2.

Figure 69b presents the stress values formed in both cases and their distribution according to the value of a , half of the short side of the ellipsoid contact area formed at the drum's bottom (which can be approximated as a rectangular area, with the long side corresponding to the drum's length). The disparity between the formed contact surfaces is substantial, with the difference in values alone being roughly 25%. This indicates that when compressing a layer with low elasticity modulus, the heavy roller creates a contact area considerably more significant than that formed under light rollers. Conversely, heavy rollers tend to sink slightly more for layers with very low elasticity moduli than light rollers. Figure 69c illustrates the distribution of the target deformation amount against the optimized elasticity modulus. A noteworthy observation is that the area delineated for the intermediate cases between the two primary scenarios is relatively narrow. However, it is crucial to highlight that when comparing the maximum elasticity moduli of the medium to be compacted, Case #1 demonstrates values approximately 25% higher than Case #2. Figure

69d presents the values plotted against the elasticity modulus. A significant disparity between the two cases becomes apparent upon examination of these nonlinear curves, mainly when drawing a horizontal line from the elasticity axis. In relatively high-elasticity scenarios, the difference in values between the two cases approaches one hundred percent. This rate of difference gradually decreases as the elasticity diminishes; nevertheless, it consistently remains around 50%. This persistent substantial difference underscores the significant impact of roller weight on the compaction process across various material properties.

The heat map analysis demonstrated that while both rollers show similar trends in Poisson's ratio as deformation increases, the heavy roller consistently achieves higher elasticity modulus values. This suggests a superior ability to compact layers with higher initial stiffness. The comparative analysis further elucidated the differences between the two cases. The heavy roller consistently produced higher Hertzian stresses for equivalent deformation values, with the stress difference between the two cases remaining around 50% across the deformation range. Additionally, the contact area formed by the heavy roller was approximately 25% larger than that of the light roller, indicating a more efficient distribution of compaction force. Perhaps most notably, the analysis of elasticity modulus distribution revealed that the heavy roller could effectively compact materials with up to 20% higher maximum elasticity moduli than the light roller. In scenarios with high material elasticity, the performance gap between the two roller types approached 82%, gradually decreasing to about 44% for materials with lower elasticity. These findings collectively underscore the significant impact of roller weight on the RCC compaction process. The heavy roller demonstrates superior performance across various material properties, particularly in compacting stiffer materials and achieving higher degrees of compaction. However, it is noteworthy that the choice between heavy and light rollers may depend on specific project requirements, material characteristics, and economic considerations.

In conclusion, this study sought to provide valuable insights for optimizing RCC pavement construction practices. By establishing a quantitative framework for roller selection based on desired material properties and compaction outcomes, this research offers a practical tool for engineers. Future studies could delve into the potential implications of these findings on long-term pavement performance and durability.

4.3. Investigating the Impact of Initial Roller Position on Longitudinal Joint Compaction in Roller-Compacted Concrete (RCC) Pavements: A Full-Scale Finite Element Analysis

This discussion comprehensively analyzes the joint compaction data obtained from several finite element analyses in the context of longitudinally cold-jointed RCC pavements. It aims to address the effects of different tapered cold longitudinal joint types on deformation patterns, including those of notched wedge joints. The discussion also covers the implications of these findings for practice, the limitations of the study, and potential directions for future research.

Throughout this investigation, the primary focus is on the characteristics of fresh-state dry concrete. This type of concrete is typically laid using a screed and is in a preparatory state, awaiting the consecutive application of roller compaction. Upon completion of the compaction process, the resulting material is referred to as roller-compacted concrete (RCC). Consequently, within the context of this study, fresh-state concrete is effectively synonymous with fresh RCC. This distinction emphasizes that the concrete has not as yet been subjected to the compaction process but is in a suitable state for compaction. Consequently, the abbreviation RCC is accurate when referring to this material as roller-compactable concrete.

In this chapter, the simulations focus on rolling a granular yet soil-like material (fresh RCC) to reduce its thickness by transferring the load onto the contact area beneath the roller. Creating full-sized 3D pavement models with dense finite element grids can be overly demanding for standard computers and time-consuming to solve. Therefore, I have reduced the 3D model to a full-sized 2D model within a plane strain condition, a common technique in conventional FEM to simplify the analysis. The problem was solved using static implicit instead of dynamic explicit methods. Some breakdowns are discussed here to explain the method employed. The benchmark for the plane stress condition is only fulfilled by confining the strain to sectionally designed planes. With this simplification, the domination of the third dimension of the model (width) in the computation overload can be eliminated. The plane's width (mostly deformable shell elements) is conceptually pointless in 2D analysis; however, in computer modeling, the thickness is set to an infinitesimal margin relative to the total size of the problem and is not allowed to deform during the solution. This approach, which involves attributing a value of 1.0 to the third dimension of the model, is well-known and commonly employed when solving one-plane sections of structures, such

as in designing retaining structures or predicting foundation settlements, mainly using closed formulas. As the total size of the finite element model approaches the actual size of the problem encountered in practice, the number of boundary conditions needed to confine the model diminishes. For this very reason, when the roller exerts its load onto a plate or any planar stratum, the length of the shell plane tends to increase abruptly in direct proportion to the decrease in thickness, causing plastic deformation without affecting volume. Consequently, the usability limits of elasticity theory tend to vanish due to the excessive strains occurring right under the roller as the layer has an elasticity modulus that is too low against the solid materials. If otherwise happens, the strains remain within the elastic portion of the stress-strain plot for materials with very large elasticity moduli (e.g., solid steel). However, for materials with relatively small moduli, like loose sand layers, the strains quickly tend to reach the elastic limits and shift towards a plastic state. At this juncture, the deformations rapidly increase and urge to diverge to infinity without incorporating plasticity features, if any, in the material model. However, dealing with concrete or steel-like materials in finite element modeling using only elasticity theory creates a great comfort zone for the designer, unlike using materials with very low elasticity moduli like soft clay and sand. This does not mean that materials with low-elasticity modulus cannot be modeled in finite element analysis. Two distinct but interrelated approaches could be employed to solve this problem. The first approach employs elastoplastic constitutive models within the domain of strain hardening using small-step based time-dependent solving when the solver engine attempts to solve each step. This approach directly taps down the strain behavior, trying to go to the infinity. This method is not overly complex to set up if the constitutive model is established beforehand. However, the solver PC should be sufficiently capable and beyond conventional PCs. As noted, the most crucial benchmark that should be handled beforehand is the constitutive model that provides the material with elastoplastic behavior. From basic soil mechanics knowledge, if the material does not exhibit true cohesion, even though it shows apparent cohesion, undisturbed sampling is not possible. While plasticity-oriented soil modeling can be derived from triaxial compaction tests on mostly cylindrical undisturbed and remolded soil samples, it becomes impossible for fresh-state RCC. If we delve into the exact problem of fresh RCC compaction, we face the challenge of defining the precise pre-consolidated features of the fresh-state RCC laid down by the paver, as the compaction effort is to be made after the paver lays down the fresh material. All these restrictive and obstructive features make it impossible to take undisturbed samples from the

fresh RCC immediately after the paver has laid it, that is, from the pre-consolidated fresh but dry concrete. Since the undisturbed samples should exactly reflect the state when the paver laid the material, samples could theoretically be taken by compacting the fresh RCC in molds in the laboratory using an electric hammer or a vibration table, leveraging relative or substitute compaction. Despite attempts to utilize this method in experiments with various mold sizes, fresh samples could not be extracted from the molds without incurring minimal damage. Additionally, the initial compaction, which could potentially reflect the impact of the paver, could not be adjusted using an electric hammer. Instead, it was deemed that constructing a theoretical model based on elasticity theory and Hertz contact theory would be the most reasonable and applicable path under possible conditions for future use in the modeling. The second approach I employ regarding using relatively low-modulus materials is creating pavement models with layered orientations, as with exact pavement modeling in practice. If the subsequent strata's elasticity properties are more significant than the low-modulus top layer to be compacted, the subsequent layers help suppress the divergence of the initial displacement of the low-modulus layer, behaving just as hardening zones.

Initially, data on a stress-strain relationship for the base or foundation layer in the models was sought from the literature, but significantly systemized data was not found. Sarkar & Dawson (2017) provided several elastic material models, and Al-Khateeb et al. (2011) presented a Drucker-Prager model with relatively few yet sufficient parameters for the base and subbase layers. Combining the given literature, I created layered orientations and mechanical properties, detailed in this study's pertinent section. The benchmark here was to input the results from the Hertz solution into the finite element model and obtain the same deformation. When the three-layered orientation is employed to model the pavement using the actual pavement size, the compaction amount obtained from the Hertz solution matches that from the finite element analysis. This section aimed not to characterize the plasticity of the concrete subjected to pre-consolidation by the paver but to determine the model that gives the targeted deformation amount and perform parametric analyses using this model. From this perspective, when the physical and load properties of the compactor are taken into account in the calculations, and the contact area and stresses obtained from the Hertz solution are considered, the desired target deformation amount is fully achieved, and the primary model is obtained for use in parametric calculations. Thus, the benchmark in the modeling phase was achieved, and the calculations were continued on models with different geometries. The longitudinal joint models were subjected to distributed static loads

with fixed amplitude ranges. The following sections provide details of the developed models. When solving a finite element problem, a common question is whether to use an implicit or explicit method. The choice depends on the algorithm used by the software. The explicit method computes the system's current state based on its previous state and is well-suited for problems with significant inertial effects and high complexity, such as crash simulations. However, it becomes computationally expensive as model complexity increases. Conversely, the implicit method is advantageous for problems without time dependence or inertial effects where the system is in equilibrium. This study's problem involves a mass-based equilibrium rather than force-based, making the implicit method more appropriate for its efficiency and computational economy. Although the explicit method could be used with parameter adjustments, the implicit method was preferred for its better alignment with the system's characteristics and resource efficiency.

Some outputs from the fresh jointed model, namely, not having any cold longitudinal model, are shown in Figure 70. In each panel of Figure 70, an apparent axial symmetry is observed in the fresh joint condition. This reveals that regardless of which axis of deformation we consider, there is a deformation of the same value in symmetry. While the signs of the symmetry values (+ or -) are the same for vertical deformation, in the panels showing horizontal deformation, the deformation magnitude is the same, but the values are in opposite directions, as expected from theory. As seen in Figure 70, panels c and d, the maximum vertical deformation values on the surface are observed precisely in the middle of the drum and approach zero just after the outer edge of the drum. At a certain distance, a small amount of deformation occurs in the positive direction as relief. This situation is also evident at the base of the fresh concrete, where the deformation changes direction at points far from the drum but towards the end of the pavement width. This fact underscores the importance of a constant pattern and sequential compaction because loosened areas can remain in addition to compressed points in random compaction. Meanwhile, when the vertical deformation is at its highest level, a deformation of approximately 10.5 mm has been determined, with the exact value being -10.473 mm. At the base of the fresh concrete, a vertical deformation value of -3.9 mm has been determined, with the exact value being -3.931 mm. These values will be used as benchmark values to evaluate the deformation amounts of cold joints.

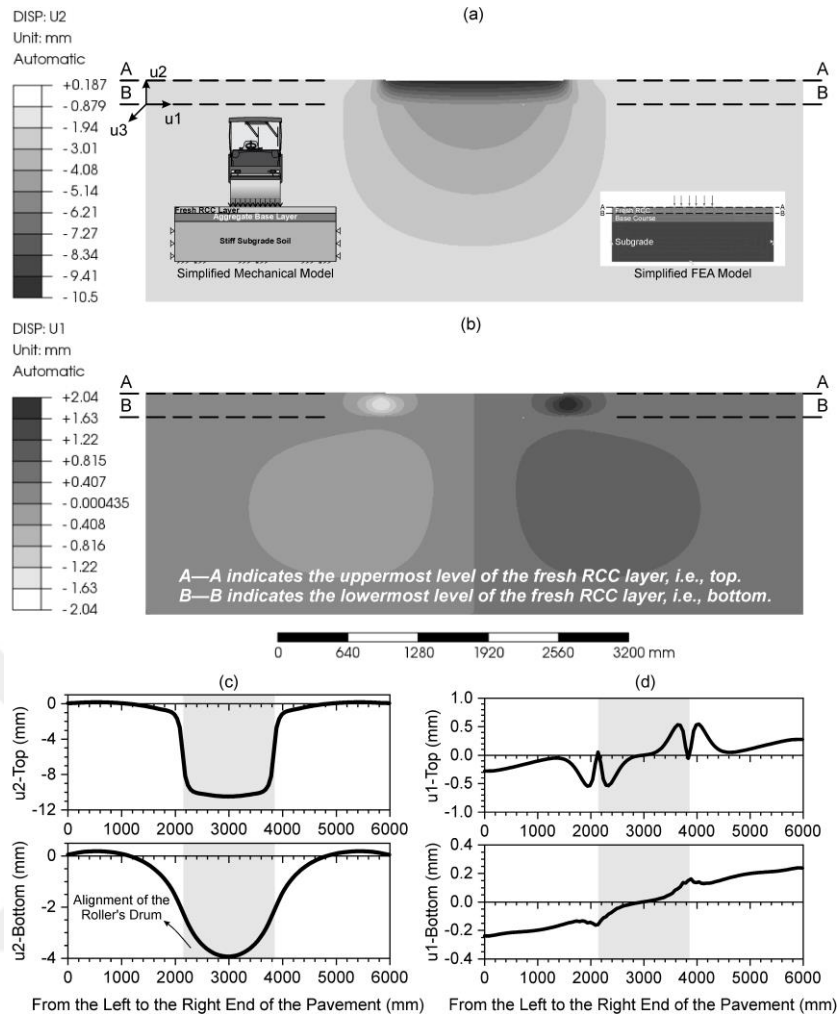


Figure 70. Vertical and horizontal deformations of the model under drum load: (a) Vertical deformation distribution (U2), (b) Horizontal deformation distribution (U1), (c) Vertical deformation distribution versus distance at the top and bottom sections (A-A and B-B) of the fresh RCC, (d) Horizontal deformation distribution versus distance at the top and bottom sections (A-A and B-B) of the fresh RCC.

While all the details regarding the finite element analyses are given in the appendix, a sample model (45-degree inclined cold joint with the drum in position 3) and the deformation distributions related to this model are shown in Figure 71. The figure reveals that even when the drum is compressing from the edge of the joint, the amount of compaction on the joint is considerably less than in those areas far from the joint due to the joint's geometry. This situation alone demonstrates the importance of joint geometry and the effect of the initial roller position. The inclined joint surface, especially in joints with slopes, somewhat restricts the compaction behavior along the vertical axis. Deformations show a distribution, particularly from the bottom longitudinal of the cold joint. Therefore, it provides no benefit regarding vertical compaction to the part below the old concrete side. Additionally, when

examining the horizontal deformation distribution, it is observed that deformations occur in opposite directions to the right and left under the drum. Especially on the fresh concrete surface and immediately below it, different deformations occur at distances that are not symmetrical to each other. While the main aim is not to determine displacement on the old side of the RCC pavement but on the fresh side, substituting the basic model for hardened (old) RCC would still help observe plastic hinges where the concrete is partially under the drum's load. However, in all tested loading conditions, plastic hinges could not be observed on the old RCC side due to the robust elastic properties assigned to the hardened concrete side.

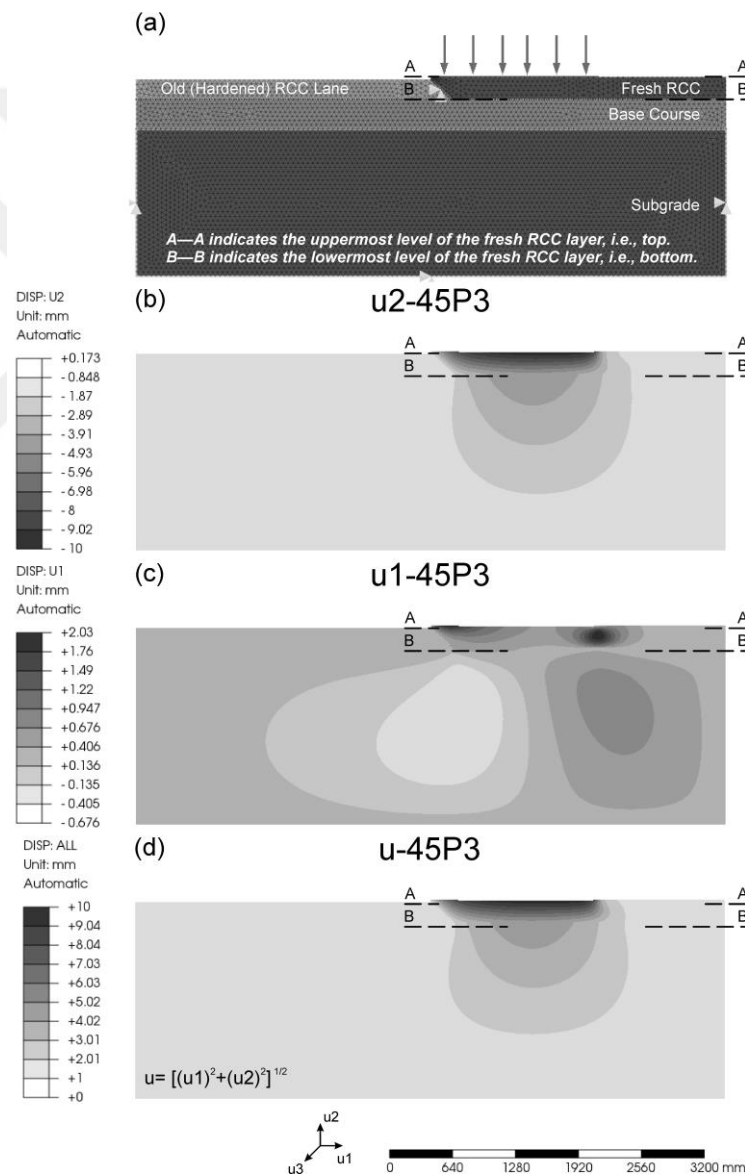


Figure 71. Details of the sample model (45° joint): vertical, horizontal, and resultant deformations of the pavement model under drum load at position 3.

Instead of providing all the distribution characteristics related to each model in this section, these details are included in the appendix. Figure 72 shows the distribution of vertical deformations along a section taken from the top and bottom of the fresh concrete. The deformation values for each position and joint model are presented in these distributions. Similarly, horizontal deformation distributions are plotted using the same approach, with the corresponding figure provided in the appendix. Naturally, extracting all the critical details from the distributions in Figure 72 at once would be visually challenging. However, it should be noted that different compaction patterns are observed near the joint for each drum position specific to each joint model tested. As the distance from the joint increases (for example, beyond 2.5 m, as represented in the figure), the deformations tend to approach zero and equalize.

Simplifying the deformation data, consisting of hundreds of columns and rows, will help understand the analysis results related to the examined problem. First, the distribution of maximum deformation values in sections taken from each model's top and bottom of the fresh layer will be compared with the level deformation pair (-10.5, -3.9) obtained from the model with fresh joints. Figure 73, shown below, illustrates this comparison. In each tested model, the maximum deformation values at the top and bottom of the fresh RCC are not significantly different from the benchmark line representing the fresh jointed RCC lane compacted with the same roller while on the symmetry axis. This pattern is evident not only at the top of the layer but also at the bottom. As the drum's initial position shifts from P1 to P5, the maximum displacement values observed at the top of the fresh RCC layer approach the benchmark line. This indicates how far the drum is from the joint and how much deformation occurs at the top layer without affecting compaction in the problematic zone near the joint. From a first impression, which mostly comes up wrong in real life, the data suggests little difference between the tested methods. It is promising that no significant difference exists between the maximum vertical deformation amounts observed between the positions and models, with values close to those seen in the fresh joint. Considering the variation in deformation patterns near the joint, examining reference points at specific distances from the joint will initially help determine which model allows for higher compaction levels.

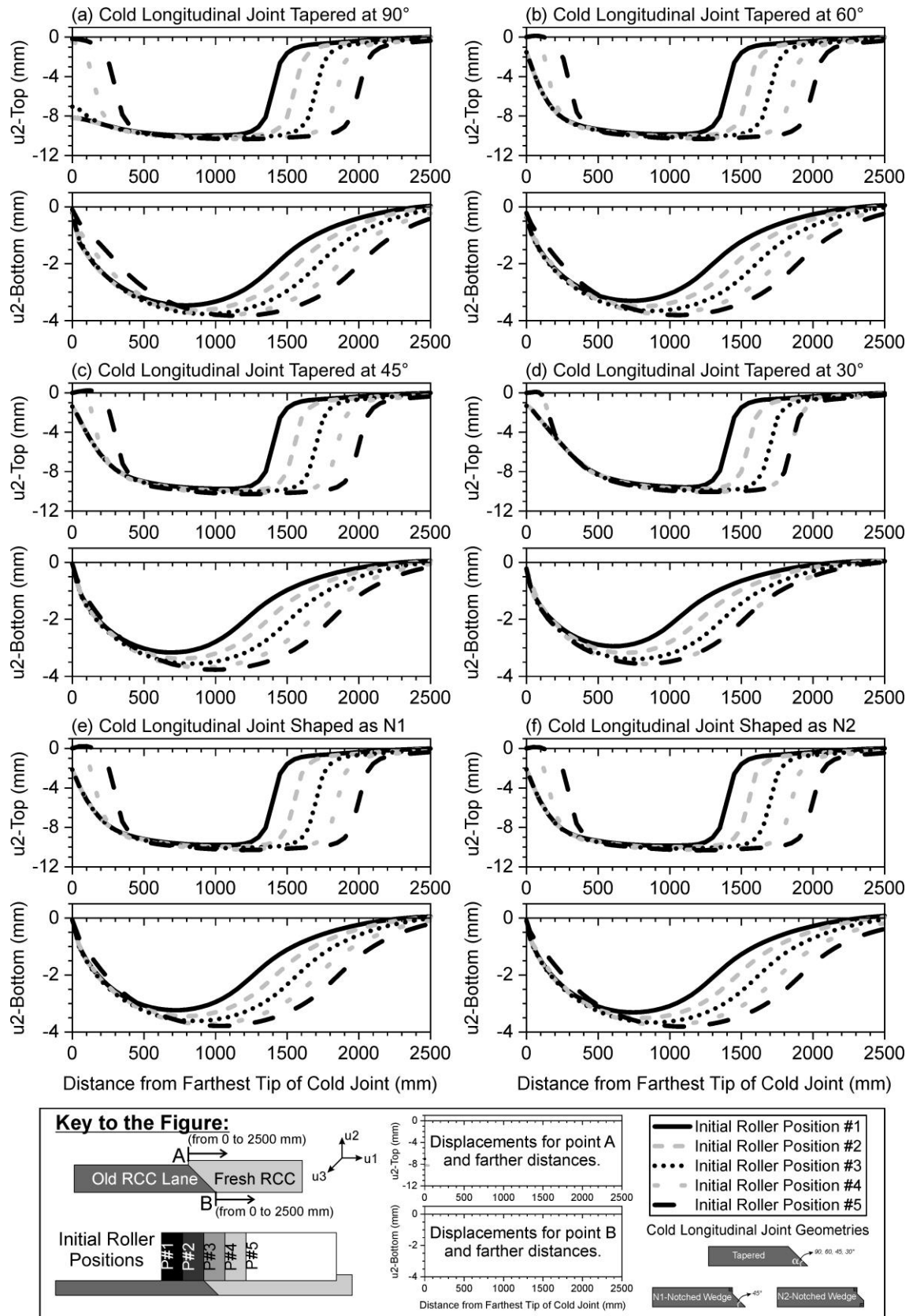


Figure 72. Variation in vertical deformation at the top and bottom faces of the fresh RCC layer versus distance from the joint face.

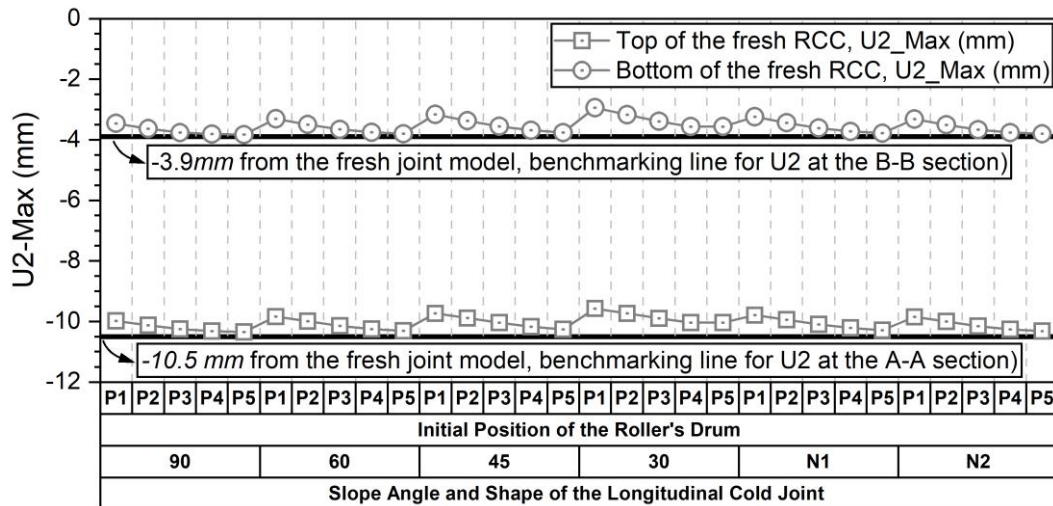


Figure 73. Maximum top and bottom deformations in the tested model.

Figure 73 shows a distribution using deformation data by picking from nodes at specific distances to the joint as reference points. By examining the upper and lower points directly above the joint and at distances of 150 mm and 300 mm away, the study evaluates which model achieves the best compaction behavior in areas close to the joint, considered the problematic cold-jointing zone. For the top measurements, data collection starts from the visible joint line on the surface. The bottom measurements begin where the joint originates along the horizontal axis. In the case of a 90° joint, the starting points for both the top and bottom measurements are innately aligned due to symmetric geometry. However, the bottom measurements have a horizontal offset for other angles. Despite this offset, the top and bottom data consistently start from (0.00) using the same reference logic. The first attempt focuses on the deformations at the top of the layer for each joint inclination at the specified distances to further the interpretations.

The following observations can be made based on the refined data used in plotting Figure 74. For deformations near the joint (0.0 mm), the 90° joint exhibited the highest deformation at the top, significantly much larger than the deformations observed for other angles. While P1, P2, and P3 yielded better results at all three examined distances, particularly in the 90° joint, P1 and P2 produced the highest compaction at points close to the joint. Position P3 can be considered the classical position where loading is mostly applied from the joint edge in practice, followed by P1 and P2. P4 and P5, respectively, created the most minor deformation around the joint in the 90° model. Regarding deformations in the

base part, P1, P2, and P3 caused similar vertical deformations, while P4 and P5 again resulted in the worst deformations.

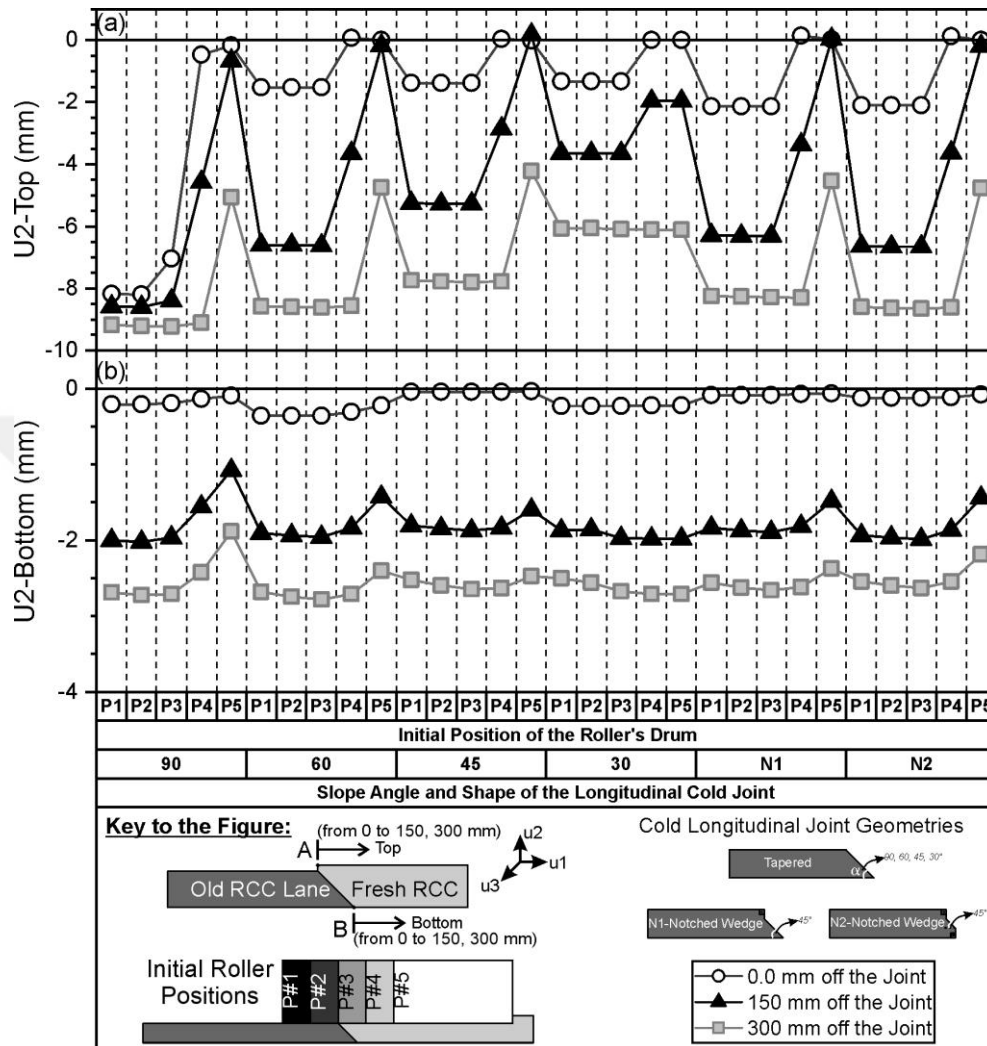


Figure 74. Vertical deformation values for each model at varying distances from the joints.

In models other than the 90° joint, specifically those angled to the horizontal due to the joint's geometry, the effect of position is more pronounced than in the 90° joint. In these models, deformations in both the base and surface parts showed similar results according to the varying drum positions specific to each joint model. At the surface level, in the 60°, 45°, N1, and N2 models, P1, P2, and P3 positions yielded very similar results at points 0 and 150 mm off the joint. At the 300 mm distance, P4 also aligns with these positions, resulting in similar deformations. In these models, P5 consistently yielded the lowest deformation values. This outcome is expected from one perspective, as P5 is at the position where the drum is farthest from the joint. A similar trend is observed at the base level: while P1, P2,

P3, and P4 yielded comparable results for the nodes at all three distances, P5 stands out by delivering the lowest performance. Interestingly, the 30° model presents a more unique case. It exhibited the lowest surface deformations among all models.

From above mentioned remarks, the general observations can be made as follows: (i) the 90° joint consistently showed the highest deformation across all distances. (ii) the 30° joint consistently exhibited the lowest deformation across all distances. (iii) deformations increased with distance from the joint for all angles and joint types. (iv) the notched wedge joints (N1 and N2) behaved similarly to each other but generally showed higher deformations than simply angled joints, except for the 90° joint. These observations suggest that the joint angle significantly impacts compaction performance, with steeper angles leading to higher deformations.

Additionally, the notched wedge design tends to result in more significant deformations than angled joints with the same inclination (e.g., 45°). The P1 and P2 positions achieve the highest success, particularly in the joint with a 90° inclination. P3 follows closely in joints with the same angle, showing only a minimal difference. However, the most significant disparity occurs with P3, where it lags behind P1 and P2 by approximately 1 mm on the joint surface (at the 0 mm distance) in the 90° joint. These findings have practical implications, including the recommendation to prefer steeper joint angles where possible and to pay careful attention to compaction for joints, particularly with gradual angles and complex geometries.

The observed trend of increasing deformation with steeper joint angles (particularly from 90° to other angles) could be attributed to better compaction and reduced stress concentration. Interestingly, the notched wedge joints (N1 and N2) yielded higher compaction than tapered joints despite the potential benefits of increased contact area and improved compaction opportunities. However, these notched wedge joints still lag behind the 90° and 60° joints in performance. The hierarchical order for displacement-led performance of the joint models is as follows: 90° > 60° > N2 > N1 > 45° > 30°. The high compaction observed in the 90° joint is worth discussing and can be attributed to several factors. Vertical joints create sharp discontinuities in the pavement structure, leading to stress concentration at the joint interface. The abrupt change in adjacent materials at a 90° joint could provide minimal lateral support, much less than those from the inclined surface support of tapered joints, potentially causing increased deformation under loading. In

contrast, achieving adequate compaction at 45° and 30° joints is challenging due to the limited area available for deformation, resulting in a weaker joint area likely prone to premature cracking. It could be contemplated that the notched wedge models, such as N1 and N2, theoretically offer a larger contact area than other counterparts between the old and new RCC material, potentially enhancing load transfer across the joint. This intuitive assumption is disproved here. However, the complex geometry of these notched wedges led to non-uniform stress distribution, resulting in inadequate and uneven compaction within the notches. Besides, it could initially be thought that more gradual angles could help distribute stresses over a larger joint area, potentially reducing the likelihood of localized deformation. However, this hypothesis was also disproven by the findings of this study.

Withrow (1988) noted that rolling patterns for RCC pavements closely mirror those used for HMA pavements. For the first fresh lane, which is not adjacent to any other lane, he recommended making the first roller pass 30-40 cm inside unconfined edges to prevent crumbling. Edge compaction can then be achieved using a heavy plate compactor or a walk-behind roller. The first static pass was recommended to overlap 25% onto the new RCC, while the first vibratory pass should overlap 10% onto the compacted material to maximize energy absorption by the fresh RCC. In addition, the old RCC joint compaction techniques suggested by Withrow are similar, requiring a 20-30 cm overlap of fresh material onto the hardened RCC, with any excess material pushed back using a lute. Through analysis results, the present study has confirmed that the method suggested by Withrow (1988), perhaps based primarily on his observations and experience, is indeed useful. By developing variants of this method, I have also pioneered an investigation into the types of deformations that occur under different conditions.

As with any fundamental study, several limitations and drawbacks are present in this study. This study is based on idealized conditions that may not fully capture the complexities of real-world pavement environments. The study employed specific loading conditions and performed static loading, but it did not consider other potential loading scenarios, such as static versus dynamic, pure dynamic, or temperature-dependent dynamic effects, which could likely influence deformation patterns. Additionally, variations in fresh RCC mixture properties, such as cement type and fineness, water-to-cement (w/c) ratio, and aggregate gradation, could affect actual compaction behavior. This analysis did not account for these factors, so a fixed material model was used. In that sense, the data derived from the finite

element analyses represents a snapshot in time and does not account for long-term effects such as fatigue, aging, and environmental factors. The finite element model was designed under plane strain conditions, focusing on two-dimensional deformation. This approach overlooks three-dimensional stress states in pavements by definition. Nonetheless, the two-dimensional model employed provides valuable insights into the yet unresolved aspects of the problem.

Based on these findings, several practical recommendations can be made. A key takeaway from this study is that steeper joint angles, ideally 90° , should be preferred over gradual angles to maximize vertical deformation, especially near the cold joint's face and bottom. Additionally, this study emphasizes the importance of adequate compaction near cold joints, particularly for mild taper angles and complex irregular geometries, by noting that sufficient compaction cannot be achieved near mildly tapered cold joints. While notched wedge joints offer theoretical advantages in terms of interlocking, our findings suggest that they should not be the primary choice. If notched wedge joints are considered, careful attention must be paid to their construction to ensure proper compaction and performance in the vicinity of these cold joints.

Some issues related to Turkish cold joint construction practices are highlighted by coupling them with the results of this study. Commonly used cold joint types in RCC pavements include freely arranged joints (as laid by a paver) with a roughly 60 or 45-degree slope and vertical joints created with saw cutting. While joint shapers are sometimes used, their application is often limited by the insufficient thickness of available shapers designed for HMA pavements. However, manufacturers have long produced drum-mounted joint shapers with sufficient dimensions for RCC pavements. These shapers can provide initial shaping and even consolidation, and a simple plate, as an edge restriction device, attached to the paver's edge can be used for additional shaping.

4.4. Experimental Investigation of Bond Strength Enhancement Materials for Fresh-to-Hardened Roller-Compacted Concrete Interfaces

The split-prism and bi-surface shear test were employed on concrete specimens to evaluate the bonding strength of cold joints treated with various interface methods. This research represents a pioneering effort in the field and holds significant implications for several reasons. Primarily, by investigating an array of bonding treatments for cold joints in RCC pavements, the study aims to enhance the longevity and durability of concrete structures. Cold joints, a common occurrence in RCC pavement construction, can be fortified against structural failures resulting from everyday distress by applying effective bonding treatments. Furthermore, this study provides a comprehensive comparative analysis of various bonding treatments and their efficacy in augmenting the bonding strength of cold joints. This critical information is reasonably instrumental in selecting optimal treatments for RCC pavement construction projects. Additionally, the study explores the impact of fiber reinforcement on the bonding strength of cold joints, potentially guiding engineers in developing more resilient concrete mixtures. The findings presented herein contribute to the knowledge base necessary for developing guidelines and best practices for RCC pavement construction, an area currently lacking in comprehensive research. By identifying effective treatments, this research enables engineers to make informed decisions, ensuring the structural integrity of concrete pavements. Consequently, this study advances the understanding of how different treatments and materials can enhance the performance of cold joints in RCC pavements, contributing to safer, more resilient infrastructure.

In the procedure of split prism test, each specimen was bisected, and diverse bonding agents were applied to the adjacent interfaces before reconnecting the old RCC block to the fresh RCC material. The bonding agents were applied to the face of the old RCC block that would adjoin the new RCC block's surface. After 28 days of hardening and curing, the specimens were subjected to pure shear forces to measure the bonding strength at the joint interface. The abbreviated sample names are explained before presenting the detailed findings of this study to facilitate understanding. P denotes plain RCC, while F represents fiber-reinforced RCC. The PP configuration indicates the rejoining of plain old RCC to plain new concrete, with FF following the same pattern for fiber-reinforced specimens. Suffixes appended to each designation indicate specific treatments: NO for untreated joints, W for water treatment (spray application), S for cement slurry treatment, and E for epoxy mortar

treatment (utilizing epoxy bonding adhesive MasterBrace ADH 1406, formerly known as Concretive® 1406). PRef denotes the plain concrete block as a reference specimen without a cold joint, and FRef refers to a fiber-reinforced concrete reference specimen without a cold joint. In addition to the split shear test, a bi-surface shear test was designed to assess the bonding strength of cold joints treated with various bonding agents between narrow and wide concrete blocks. The narrow block (representing the later-cast, new RCC) measured 50x150x150 mm, while the wide block (simulating the first-cast, old RCC) had dimensions of 100x150x150 mm. Following a protocol employed in the split prism test samples, the new side was cast after applying the bonding agent to the face of the old RCC block, and the two blocks were joined. After a 28-day curing period, the assembly was subjected to shear forces using rectangular steel supports. All specimens were produced as cubes, cast in 150x150x150 mm molds. Initial tests were conducted on samples at 1 and 3 days of curing. However, nearly all samples, excluding those treated with epoxy, experienced sudden rupture at the cold joints upon initial loading. Consequently, the definitive evaluation was performed on 28-day samples. Failure loads were measured in Newtons, with a consistent loading rate of 0.25 MPa/s. A minimum of three samples were utilized to ensure statistical reliability for each testing mode and sample type. Bonding strength values were recorded in MPa, accompanied by corresponding standard deviations to assess result variability. Experimental results were compared with allowable minimum bond strength limits for concrete-to-concrete bonding given in the manual called Guide for the Selection of Materials for the Repair of Concrete from American Concrete Institute (ACI, 2006) to validate the effectiveness of the treatments in meeting industry standards. This benchmarking is crucial for future practical applications and could inform regulatory compliance.

Figure 75 collectively shows some of the fracture modes obtained from split prism tests performed on cube samples. The most ductile fractures were observed in freshly jointed samples (PRef and FRef). While PRef samples exhibited splitting and abrupt breakups, FRef samples generally did not show detachment or complete splitting in their failure mode. This phenomenon was later observed in the cross-sections of samples manually split with a steel scoop. Generally, during splitting, the old concrete block remained between the supports, while a rapid rupture separated the new concrete part from the adjacent section. In PPE and FFE samples, most of the epoxy layer remained on the stable block.



Figure 75. Failure modes observed in the split prism tests conducted on jointed RCC

Figure 76 illustrates the states of the adjacent surfaces after testing. Sandy-grainy surfaces were observed in the intermediate sections of NO and W-modified samples.

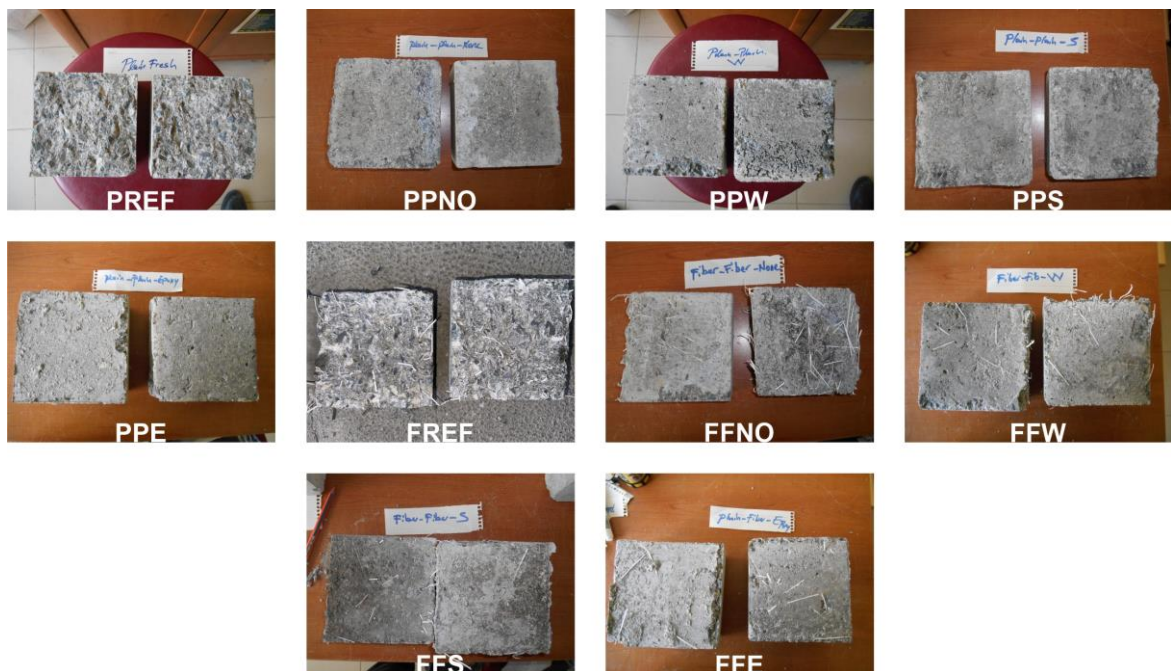


Figure 76. Fracture surfaces of the bonded specimens in split prism test

This indicates that even when an insufficient bond forms in the cross-section, an abrasion effect occurs on the adjacent concrete surfaces due to the relative movement of the blocks in hard contact, with a certain coefficient of friction. In fiber-reinforced concrete fracture modes where fibers break apart, most fibers were found to break with sharp ends, while those that did not break were stripped from the concrete matrix with minimal force. However, in areas where fibers that had not yet broken off but showed some degree of adherence remained stable in the matrix, the surrounding concrete crumbled to some extent. Epoxy-reinforced sections exhibited a very characteristic behavior, warranting additional photographs for these samples. Figure 77 demonstrates that the epoxy cross-section layer generally remains on the old concrete after collapse. This layer either remained entirely on one side, with visible sand protrusion on the opposite surface, or showed partial or block-like separation. The observed characteristics of partial or all-out detachment of the epoxy adhesive was consistent with what signified by Gungor et al. (2024). Off the remarks to be made on the results, much as it would be an early onset, contrary to widespread belief amongst concrete practitioners, the use of epoxy bonding agent tends to lead to brittle fracture rather than providing an overall ductile behavior in the cold RCC joints.

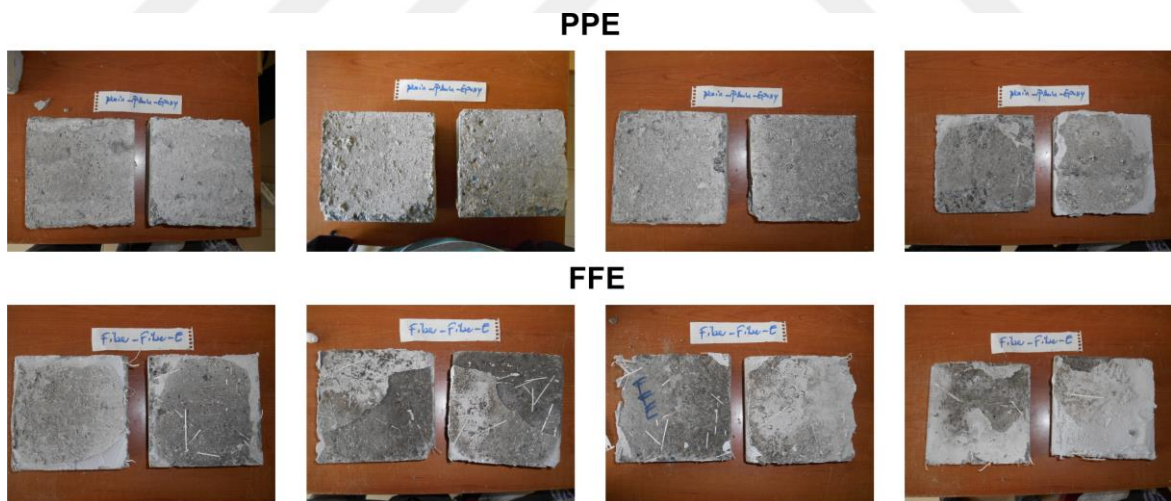


Figure 77. Fracture surfaces of the bonded specimens with epoxy mortar in split prism test

The split-prism test results, presented in Figure 78a, yield several critical observations. The plain reference (PRef) and fiber-reinforced reference (FRef) specimens, without cold joints, exhibited the highest bonding strengths of 4.11 MPa and 4.26 MPa, respectively.

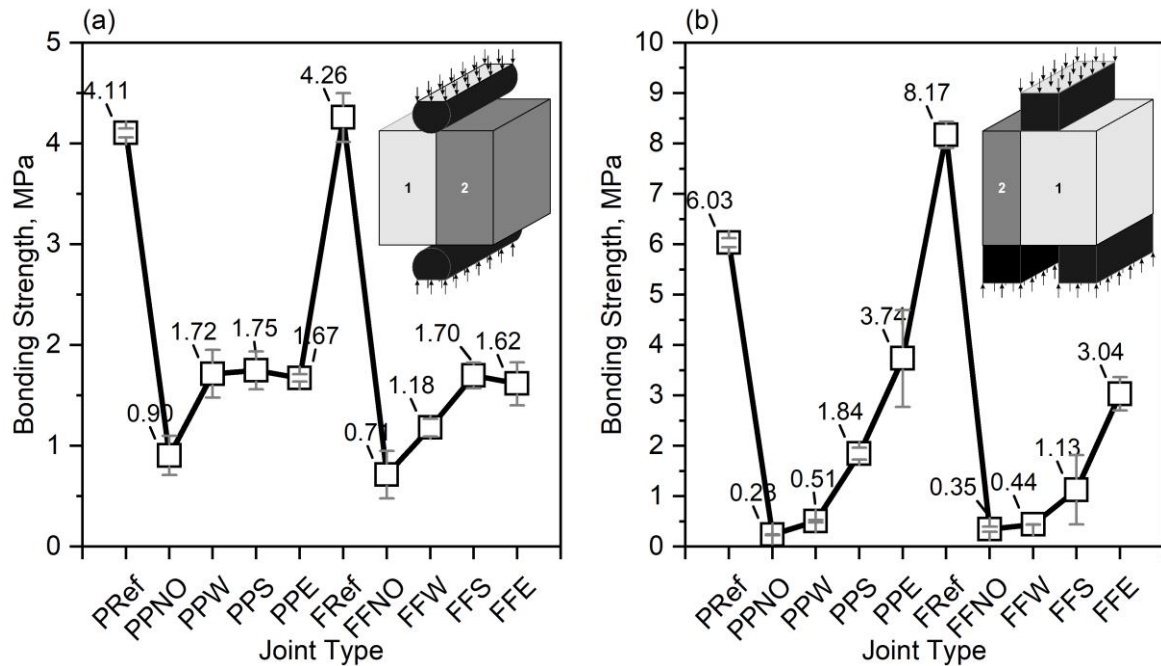


Figure 78. Bonding strengths obtained from split prism and bi-surface shear tests

These values establish a benchmark for assessing the effectiveness of different bonding treatments applied to cold joints. The results indicate that fiber reinforcement marginally improves the bonding strength of reference specimens. Bonding strength varied significantly among specimens with cold joints, depending on the bonding treatment. Untreated specimens (PPNO and FFNO) showed the lowest bonding strengths of 0.90 MPa and 0.71 MPa, respectively, highlighting the detrimental effect of cold joints on bonding strength. Water treatment (PPW and FFW) improved bonding strengths to 1.72 MPa and 1.18 MPa, respectively, suggesting that even simplistic treatment with sprayed water positively impacts cold joint strength. Cement slurry treatment (PPS and FFS) resulted in similar bonding strengths of 1.75 MPa and 1.70 MPa, respectively, showing that cement slurry is slightly more effective than water treatment. Epoxy mortar treatment (PPE and FFE) achieved bonding strengths of 1.67 MPa and 1.62 MPa, respectively, indicating that while epoxy mortar treatment provides some improvement over untreated joints, it is less effective than cement slurry treatment in this context. The situation in which the treatment with epoxy mortar lagged behind the treated joints with water and cement slurry was not expected in the early stages of the study planning. Nevertheless, when I examined the procedure, theory, failure modes, and bonding strength values collectively, they brought a possible explanation to mind.

When the specifications of MasterBrace ADH 1406 (formerly known as Concrecive® 1406) are examined, it is known that the compressive strength of the mortar is 30 MPa and 70 MPa, and the flexural strength is 17 MPa and 25 MPa at 1-day and 7-day age, respectively. Our lab tests show that the 28-day compressive strength of plain RCC is 55 MPa and 45 MPa for fiber-reinforced RCC, and the modulus of rupture is 7 MPa and 8MPa, respectively. Namely, under any circumstance, the thin epoxy mortar film implemented onto the cold joint surface is more resilient than the RCC itself. When examining the failure modes of the jointed prisms, the epoxy-reinforced joints exhibited somewhat different characteristics. In a nutshell, the results of the split-prism tests highlighted the superior performance of fiber-reinforced concrete in enhancing the bonding strength across all treatment types, albeit only slightly.

Figure 79 illustrates the failure modes in bi-surface shear tests. The failure modes of PRef and FRef samples differ significantly from other cold-jointed samples. The fresh-side blocks showed sudden separation from the old concrete block in cold joints, with a shear-driven failure on the joint surface. For reference samples with hot or fresh joints, failure occurred in an almost vertical direction near the symmetry axis of the sample. PRef samples exhibited a split with a sharp, vertical, and wide-open crack, while FRef samples showed cracks with a smaller width spread over a larger area. Neither PRef nor FRef samples could be split with a scoop or similar tool after the experiment, preventing observation of the inner surfaces. Namely, these samples maintained their block structure upon testing. However, almost all samples displayed sudden failure on adjacent cold joint surfaces, resulting in block-like separation. In fiber-incorporated samples, some fibers continued to grip the opposite block, but similar to split prism tests, the concrete at the bottom course of the distressed fibers on the old concrete showed apparent wear and chiseling. PPE and FFE samples exhibited characteristic behavior that distinguished them from other reinforced samples. Cracking occurred not where the epoxy film contacted the old concrete but in the fresh concrete part of the epoxy. This cracking caused separation from the remaining surface and spread in both plain and fiber samples, leading to partial failure on the fresh side.



Figure 79. Failure modes observed in the bi-surface shear tests conducted on jointed RCC

Figure 80 shows the separated surfaces of some samples after the bi-surface shear test. These samples displayed the same physical appearance as in split prism tests. However, unlike the split prism test, interface surfaces were less worn and eroded, resulting in a less sandy-grainy surface. Both split prism and bi-surface shear test results strikingly revealed that interface reinforcement with epoxy adhesive, particularly in fiber-reinforced samples, fully covering some protruding fibers from the old concrete side with epoxy film, hampered the fiber adhesion typically observed on the new concrete side in other samples. While the high-strength bonding benefit from epoxy adhesive was immediately noticeable, the contribution of the fibers to overall bonding strength was found to be almost eliminated due to the epoxy film covering the fibers.



Figure 80. Fracture surfaces of the bonded specimens in a bi-surface shear test

Regarding the bi-surface shear test results, the overall picture showed some differences and departures from what was observed in the splitting prism test. The bi-surface shear test results are presented in Figure 78b. In the bi-surface shear tests, the reference specimens (PRef and FRef) again showed the highest bonding strengths, with 6.03 MPa and 8.17 MPa, respectively. The fiber-reinforced reference (FRef) exhibited a significantly higher bonding strength than the plain reference (PRef), demonstrating the beneficial impact of fiber reinforcement in enhancing the shear strength of the joints. The untreated specimens (PPNO and FFNO) exhibited very low bonding strengths of 0.23 MPa and 0.35 MPa, respectively, confirming the adverse effects of cold joints. Water treatment (PPW and FFW) improved bonding strengths of 0.51 MPa and 0.44 MPa, respectively, which were relatively low but still better than untreated specimens. Cement slurry treatment (PPS and FFS) significantly improved the bonding strengths to 1.84 MPa and 1.13 MPa, respectively, indicating that cement slurry was an effective treatment for enhancing the shear strength of cold joints. Epoxy mortar treatment (PPE and FFE) yielded bonding strengths of 3.74 MPa and 3.04 MPa, respectively, demonstrating the highest improvement among the treatments applied, particularly in the plain concrete specimens (PPE). Comparing these results with the allowable minimum bond strength limits for concrete-to-concrete bonding given by ACI (2006) could be beneficial to track to what extent the joint treatment was effective thereby

checking where the obtained bonding values were. For the split shear tests, only the reference specimens (PRef and FRef) exceed the upper limit of the direct tensile test bonding strength after 14 days (2.1 MPa), with bonding strengths of 4.11 MPa and 4.26 MPa, respectively. The treated cold joints, however, fell short of these values, indicating the need for more effective bonding treatments to meet or exceed the allowable limits. In the bi-surface shear tests, the reference specimens (PRef and FRef) again showed bonding strengths well above the upper limit of the direct tensile test after 14 days. Notably, the epoxy mortar treatment (PPE and FFE) also exceeded the upper limit of 2.1 MPa, with bonding strengths of 3.74 MPa and 3.04 MPa, respectively, highlighting its effectiveness in improving joint performance.

With the samples produced in this study, simultaneous identification of bond strength and friction angle would have been possible only if a ready-to-use direct concrete shear device similar to that used in soil mechanics was available. In our case, I could not calculate the friction coefficients because no slanted surface against the loading direction was employed. Namely, while the slanted surfaces were not employed, the normal stress coupled with the pure shear could not emerge and be computed. Much as direct calculation of friction coefficients was not performed in this study, it is worth noting that various studies have reported friction coefficients for concrete-to-concrete surfaces. McLean & Pierce (1988) investigated RCC in dam construction, finding that bonded joints, treated or untreated, exhibited shear strength parameters similar to the parent materials. Fully bonded joints demonstrated friction angles of $24-73^\circ$ with bond strengths of 62-4,288 kPa upon break-bond failure and $34-52^\circ$ with 96.5-1,489 kPa upon sliding failure. They concluded that the commonly used parameters to that date of $\phi=45^\circ$ and $c=0.1f_c$ for a broad range of RCC were unconservative, particularly for all but the cement-richest mixes. The pair of $\phi=45^\circ$ and $c=0$ was not universally conservative for unbonded joints. Comparing strength envelopes of $\phi=45^\circ$, $c=0$, and $\phi=40^\circ$, $c=152\text{kPa}$ revealed a shift from conservative to unconservative values at a normal joint stress of about 945kPa. They noted some apparent cohesion in unbonded joints under normal loads, suggesting that using a lower ϕ and $c=0$ might be overly conservative. The friction coefficients range from 0.47 for smooth surfaces to 1.44 for rough surfaces (Hussein et al., 2016). Dolen & Tayabji (1988) conducted extensive experiments on fresh and partially bonded RCC joints, observing bond strengths ranging from 724 to 3000 kPa. They noted a decrease in tensile strength and cohesion of bonded joints and sliding friction of nonbonded joints between 8-hour and 72-hour untreated joints. They found that

increasing the cementitious materials content in the mix improved joint bonding strength. Although the prime mover of split prisms and bi-surface shear tests remain unknown, these methods have garnered critical research interest for their scientific rigor and contributions to understanding bond strength in concrete. Drawing upon the seminal works of Hansen & Reinhardt (1991), Hansen et al. (1999), and ACI (1999), Choi & Hansen (2005) proffered characteristic strength parameters for RCC. These comprehensive studies elucidate that the cohesion values for RCC mass range from 0.83 to 4.14 MPa, while for mortared RCC lift joints, the range spans 0.48 to 4.140 MPa. Unmortared RCC lift joints exhibit a broader spectrum, from 0 to 4.14 MPa. Concomitantly, the friction angle for all RCC types falls within the 35° to 70° range.

Geissert et al. (1999) employed the split prism test using a universal compressive testing machine to simulate pure shear, providing a versatile alternative. Momayez et al. (2004) utilized the bi-surface shear test on cubic samples to evaluate bond strength, highlighting the impact of sample age, silica fume content, and surface roughness on bonding effectiveness. Daneshvar et al. (2022) comprehensively reviewed various bonding testing methods, offering a thorough overview of alternative approaches. Bettencourt Ribeiro et al. (2001) emphasized the importance of wet curing the lower layer in two-layer RCC pavements to prevent premature hardening, noting the necessity of timely compaction of the top layer of the two-layer concrete and the use of retarders. Azizmohammadi et al. (2021) further explored RCC bond conditions, noting the influence of joint age and bedding layer, emphasizing that increased cement content in bedding mortar improves shear and tensile strength while reducing permeability. Mohammed et al. (2019) underscored the suitability of well-bonded two-layer RCC pavements for mild to medium traffic densities. Gungor et al. (2024) found that polymer-modified cement-based adhesive mortar achieved superior bonding in two-layer RCC, while cationic bituminous emulsion proved ineffective as a bonding agent. Liu et al. (2018) observed that RCC interlayer shear strength decreases over time but can be enhanced with treatments like expansion agents and nano silica that are more beneficial than mortar alone, improving friction coefficients and cohesive stress. They noted that as cohesive stress decreases, the friction coefficient tends to keep steady with the increase in the time interval. Moreover, they found that the expansion agent increased the friction coefficient as the Nano silica improved the cohesive stress. Hussain et al. (2020) conducted a study on concrete pavements and found that steel fibers provided the best flexural performance, significantly reducing the design thickness compared to

polypropylene and glass fibers. However, their cost-benefit analysis revealed that steel fiber-reinforced concrete is less economical than glass and polypropylene fiber-reinforced concrete for the same load-carrying capacity. They finally concluded that glass and polypropylene fiber-reinforced concretes can offer more cost-effective pavements than steel fiber-reinforced and conventional plain concrete. Shunmuga Vembu & Ammasi (2023) also provided insights into variables affecting bond strength in conventional concrete, noting fiber reinforcement's ability to enhance bond strength and durability. Feng et al. (2022) highlighted that applying a cement slurry or epoxy on a conventional concrete surface significantly improves the bond strength further by adding that the epoxy mortar improved more than the cement slurry in terms of bonding strength. Liu et al. (2024) extensively reviewed the interface bonding properties of new and old concrete interfaces. When examined thoroughly, the results drawn from this present study are entirely consistent with the early use of the bonding agents but employed in conventional concrete.

When panels a and b in Figure 78 are examined, the bond strength values obtained from the two experiments differed in samples treated with the same method but with different geometric configurations. It would be helpful to bring together some practical and theoretical evaluations on this subject and examine it for future research. The bi-surface shear test typically yields higher bonding strengths than the split prism test, which generally produces lower bonding strengths. This discrepancy arises from the fundamental properties of concrete and its constituent components. In the bi-surface shear test, the shear bond strength is measured, which benefits from the mechanical interlocking of aggregates and the frictional resistance at the interface. These mechanisms contribute to a higher measured bond strength. Conversely, the split prism test measures tensile bond strength, which is inherently lower due to the intrinsic weaknesses in the cement paste, especially at the interfaces between the cement paste and aggregates. Tensile strength is more susceptible to failure because of micro-cracks propagation and defects within the concrete matrix. Thus, the observed higher interface bonding strength in the bi-surface shear test is primarily due to the robust frictional and interlocking forces that resist shear stresses, compared to the weaker tensile forces that govern the failure in the split prism test. The detailed interpretations regarding the other type of testing procedures of the bonding strength can be found in Liu et al. (2024).

In summary, the experimental results indicate that while fiber reinforcement improves bonding strength to some extent, bonding treatment plays a crucial role in

determining the effectiveness of cold joint repairs. Epoxy mortar treatment shows the most promise, especially in bi-surface shear tests, where it significantly enhances the bonding strength to levels exceeding the allowable limits for direct tensile tests. Cement slurry also proves to be a feasible option, particularly in split prism tests. These findings underscore the importance of selecting appropriate bonding treatments to ensure the resilience and durability of cold joints in roller-compacted concrete pavements.

Furthermore, some of the cube models were simulated for the split prism test using Calculix/PrePoMax. The aim was to conduct a retrospective analysis using the failure load and related bond strength values obtained from experiments. The hypothesis was that if an ultimate vertical displacement (0.09 mm was employed here) is defined under the same conditions but using the load and strength relationship pair for each model, the required joint stiffness modulus could be calculated and assigned to the intersection section to achieve the targeted displacement for each model.

Two methods could be considered for this analysis. The first is the cohesive zone method, which is well-known but has limited applicability to our problem due to its requirement for numerous onset parameters. Given the available information, the second, more effective, and much more straightforward approach is to apply Mohr or Mohr-Coulomb behavior to the interface surface elements only using friction and cohesion couples. This approach involves defining a separate stiffness matrix for the nodes in the intersection zone, with failure mode occurring due to stress at the defined nodes. A domain-specific stiffness ($\text{Stiffness} = \text{Force} / \text{Displacement}$) approach was used, with a target displacement of 0.09 mm. The interface area was calculated, and the total force at failure was determined by multiplying the bond stress by the area, which is the opposite of what was done to determine the bond strength upon the splitting. Stiffness for the entire interface was calculated as force/target displacement in N/mm, and then stiffness per unit area was calculated by dividing stiffness by the interface area in N/mm/mm². When these calculated stiffness values were defined for each model, similar deformation patterns were anticipated in the analysis.

Calculations were performed only for plain concrete samples, using a dummy elasticity modulus of 35 GPa and a Poisson's ratio of 0.2. The pins were modeled as rigid solid elements. As shown in Figure 81 a-c, PPE, PPS, and PPW models exhibited almost identical behavior, approaching the target displacement. However, the PPNO model (Figure 81d) achieved only half of the expected deformation. This suggests overall that in the NO extension models, despite the absence of interface reinforcement treatment and thus low

bond strength, the elements are likely influenced by friction to an extent, indicating that a cohesive model coupled with a friction model (MC Model) might be more appropriate than a simple Mohr model.

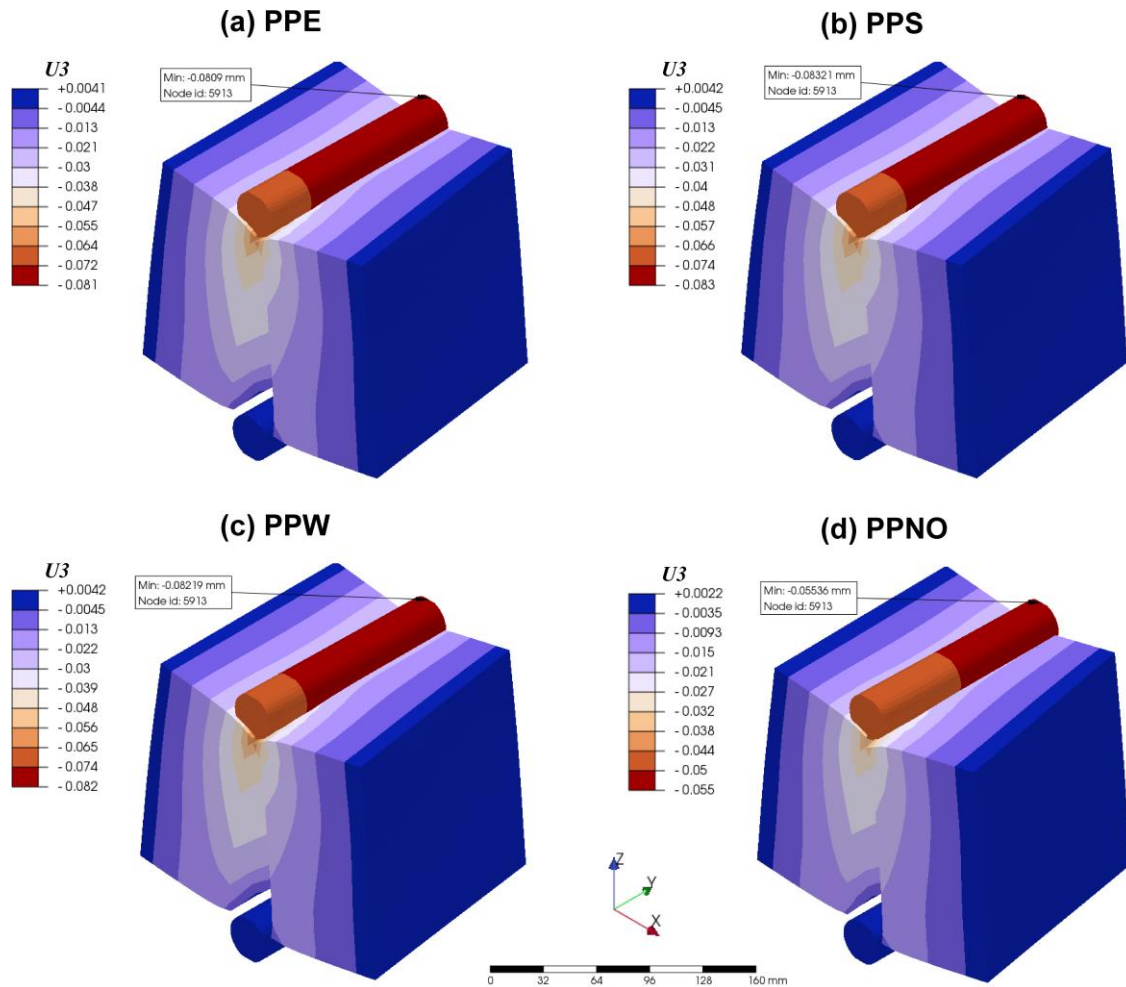


Figure 81. Finite element analysis results of some split prism tests

Figure 82 illustrates the lateral movement simulation of the finite element model for the PPW sample in the split prism experiment. The model accurately replicated experimental observations, where one adjacent block typically remained fixed on the supports while the other showed sudden separation and fell from the platform. In the finite element model, as shear force is transferred to the interface section and the concrete right begins to crush due to pressure at the upper part, the increasing shear force overcomes the bond strength at an instantaneous rate. Consequently, the separated block slides on the pin and falls. Using hard contact between elements and cohesion provided by relative stiffness spread across the areal

element at the interface, the finite element analysis produced highly realistic results that closely matched experimental observations.

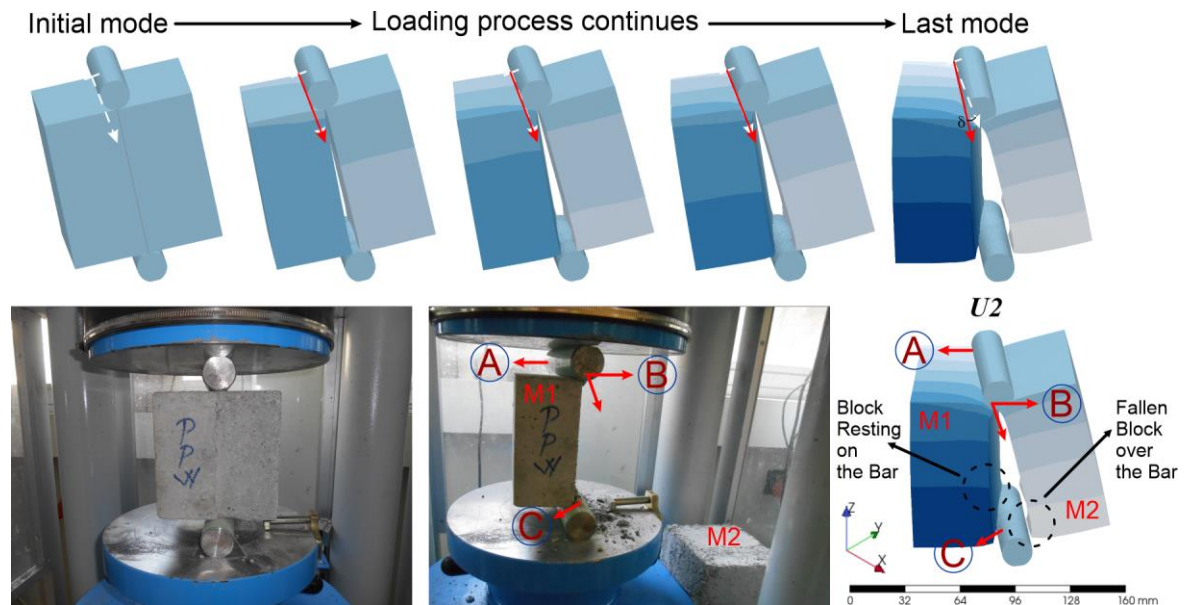


Figure 82. Lateral movement of the PPW model up to the actual test and FEM

While several different descriptions can be made, the simplistic one is used here from an engineering standpoint. The process of weighing the expected or predicted costs and benefits peculiar to an engineering project or just a choice to be made on it to see if it makes sense from a business standpoint is known as a cost-benefit analysis (CBA). Quite broad explanations and in-depth variations of the CBA can be found in Boardman et al. (2018) and Mishan & Quah (2021).

This part of the analysis aims to conduct a cost-benefit analysis of the interface reinforcement methods employed, considering material costs and determining effectiveness rankings, $E > S > W > NO$. This analysis aims to delineate the most economically viable solutions for concrete-to-concrete interface treatment methods to increase bonding in various applications. The methodology was kept as straightforward as possible by employing a cost-benefit analysis approach, focusing on material costs per square meter of application. The computational pathway is as follows. First, material costs for each bonding method per m^2 were calculated. The test-driven mechanical effectiveness hierarchy ($E > S > W > NO$) was turned into scores. The method is as follows: the numerical values obtained from bisurface tests on plain concrete samples were calculated and scored according to the strength value obtained from the jointless (fresh joint) model out of a 100% (treated model strength x 100

/ fresh model). Computation of a cost-benefit score by multiplying the cost per m² by the benefit score was done. Finally, the ranking of methods based on their cost-benefit scores was determined. Besides, material costs were derived from current market prices in Istanbul, Turkey. The application surface area was standardized into 1 m², and the workmanship and machinery costs were kept fixed for comparative purposes. Table 10 presents the calculated material costs, hypothetical benefit scores, and resulting cost-benefit scores for each treatment method. The economic conditions on which the calculation is based are the market conditions as of September 2024.

Table 10. Cost-benefit analysis of concrete-to-concrete bonding agents

Treatment Agent	Cost (\$/m ²)	Benefit Score (Out of 100%)	Cost-Benefit Score	Cost-Benefit Score (Out of 100%)
Epoxy bonding mortar-E	17.624	62.023	0.284	100.000
Cement slurry -S	0.235	30.514	0.008	2.715
Sprayed water-W	0.001	8.458	0.000	0.030
No treatment- NO	0.000	4.643	0.000	0.000

The efficacy of a cost-benefit analysis is dependent primarily on how important or urgent the current construction project is. From this point on, any project's criticality innately affects the CBA analysis results. Namely, some projects deemed critical by decision-makers, such as airport runways, military bases, and multimodal transportation hubs, generally need intensive examination due to their economic consequences and safety issues. On the other hand, despite their importance in facilitating community connectedness, less vital infrastructures, such as local roads, might not require the same level of inspection. This difference in given significance calls for a more sophisticated approach to asset management, in which the maintenance plans and resource allocations are matched to the functional requirements and possible hazards of individual structures. The analysis reveals an apparent inverse relationship between material cost and cost-effectiveness. The (NO) option emerges as the most cost-effective, followed by sprayed water (W), cement slurry (S), and epoxy bonding mortar (E). However, it is crucial to interpret these results within the context of structural requirements and long-term performance. While NO incurs no material cost, it provides minimal bonding strength benefit, potentially compromising structural integrity. Conversely, despite its high cost, E may be indispensable, particularly for critical structural applications or those exposed to harsh environments due to its superior bonding properties.

The W method demonstrates remarkable cost-effectiveness and may be suitable for dramatically less or nondemanding applications. S offers a balanced solution, combining moderate cost with potentially good bonding strength, making it a financially viable option for many pavement construction alternatives.

In summary, even these simple comparisons suggest that cost-effective solutions exist for various application requirements. In that sense, this simple cost-benefit analysis manifestly suggests that methods S and W are vastly preferable to E. However, It is imperative to consider that this analysis does not account for labor costs, equipment requirements, and costs, long-term durability, or any further project-specific needs, thereby not providing a universal benefit hierarchy amongst the tested methods. In that sense, it is advised that more holistic analyses should elaborately be undertaken if the specific constraints and project goals are to be fulfilled.

4.5. Evaluation of Mechanical Properties of Roller-Compacted Concrete Beams with Cold Longitudinal Joints: Influence of Joint Taper Angles and Interface Treatment Materials

Several samples were molded, cured, and tested in the laboratory to evaluate the effectiveness of various bond layer types and joint configurations in RCC pavements. The main goal was to identify the optimal method for ensuring strong adhesion between adjacent layers with respect to the shape of the cold joint and interface treatment method, thereby yielding a better modulus of rupture. Before analyzing the test results and going further, it is crucial to clarify the terms "tack coat" and "bond coat." These terms mostly come from asphalt pavement technology. While often used interchangeably, they serve distinct purposes in asphalt pavements. In this study, we'll focus on the "bond coat," a term more commonly used in concrete pavement technology. Tack coats are primarily used to enhance adhesion between asphalt adjoining (side by side) layers, while bond coats serve a broader role in ensuring the structural integrity of the pavement (in between the stacked asphalt layers). I begin by discussing the fundamental properties of the materials used in the samples, including their size, concrete quality, and joint configurations. Next, the results of various physical, mechanical, and non-destructive tests conducted on these samples are analyzed. By comparing the performance of different bond layer types and joint configurations, the study aims to determine the most effective technique for creating durable and long-lasting RCC pavements with cold longitudinal joints.

It is incidentally noteworthy that the differentiation between tack coat and bond coat becomes more apparent when considering HMA pavements. Tack coat usage in HMA pavements is mandated in 16 of the 50 states in the USA (McDaniel et al., 2012; Williams, 2011). Wu et al. (2020) prominently demonstrated that the quantity of tack coat significantly influences the bonding performance of layered HMA under traffic load excitations. Their study revealed that insufficient and sparse applications of tack coat result in meager interface bonding, leading to debonding. Conversely, the presence of thick bond layers, while aiming to enhance bonding, can paradoxically yield insufficient strength in joint areas. Considering that a bond coat enhances the initiation and continuity of bonding between old and fresh sides, it can be anticipated that there might be a discernible difference in the overall mechanical performance of various longitudinal models based on the application or absence of a bond coat. In essence, a bond coat is conventionally employed to enhance the interconnection of two adjacent sides of pavement slabs (old and fresh) as the fresh RCC hardens. It can be asserted that the use of a bond coat does not directly correlate with the initial compaction performance conducted through roller compactors. However, expecting a benefit from the use of bond coat is eminently reasonable. First off, several fundamental properties of plain and fiber reinforced RCC delineated through laboratory experiment are given below. Fundamental physical properties are shown in Figure 83.

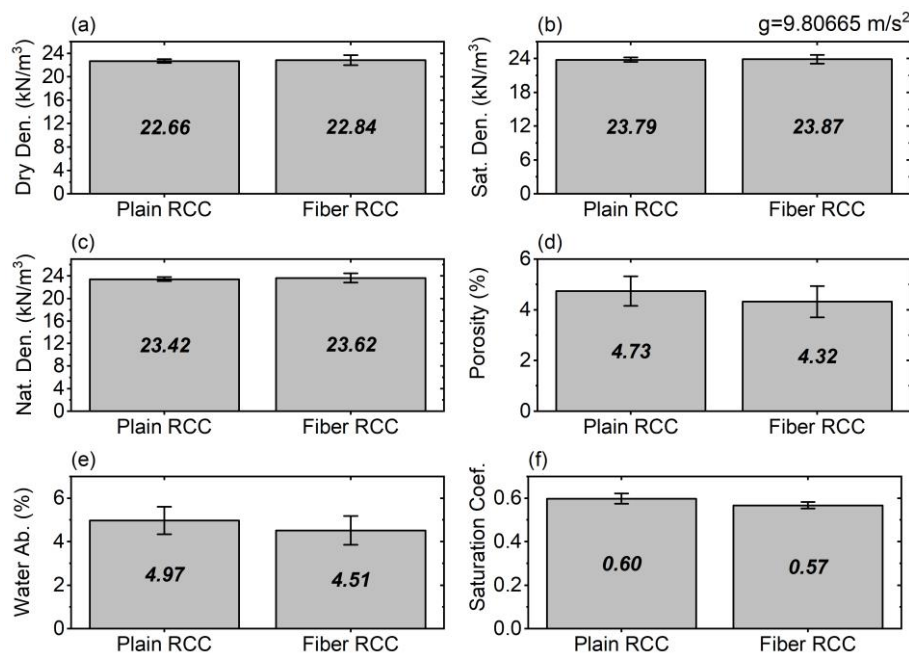


Figure 83. Fundamental material properties of plain and fiber-reinforced RCC

The data collection given in Figure 83 shows comparisons between plain RCC and fiber-reinforced RCC across various physical properties. Plain RCC exhibited a dry density of 22.66 kN/m^3 , while fiber RCC had a slightly higher dry density of 22.84 kN/m^3 . Saturated and natural densities followed a similar trend, with fiber RCC consistently showing higher values. Comparing these results to published studies, the observed densities align with typical values for RCC, which range from 22 to 24 kN/m^3 (Harrington et al., 2010). This consistency suggests that the produced RCC samples aligned with industry standards.

Shafigh et al. (2019) found that maximum dry density was achieved at 5.7% moisture content. However, they also noted that 5% moisture content yielded the lowest porosity and water absorption values. This finding is particularly relevant when compared to the results of the present study, where fiber-reinforced RCC showed lower porosity (4.32% vs. 4.73%) and water absorption (4.51% vs. 4.97%) compared to plain RCC. This can be attributed to the improved durability characteristics. The presented data also indicates that fiber-reinforced RCC has improved characteristics in terms of porosity and water absorption compared to plain RCC. This aligns with the common trend observed in the literature, where optimizing moisture content (as noted by Shafigh et al. (2019)) can lead to improved porosity and water absorption properties.

The lower saturation coefficient observed in fiber-reinforced RCC (0.567 vs. 0.597) further supports the potential for enhanced resistance to water penetration. This improvement in water intrusion-related properties could contribute to better durability. While the present data does not directly address air void distribution, it is worth noting the industry's emphasis on this aspect. Foster et al. (1964), who can be considered as the starting point of the subject, a few decades ago highlighted the importance of evenly distributed air voids for RCC durability. The observation of lower porosity in fiber-reinforced RCC might indirectly relate to air void distribution, potentially contributing to improved durability. Khayat & Libre (2014) discussed air-entrained RCC, noting its potential for increased frost resistance when properly achieved. However, they also mentioned that non-air-entrained RCC can show good frost resistance if the ingredients are of high quality and well-proportioned. The present data particularly showing improved density and reduced water absorption in fiber-reinforced RCC could potentially contribute to better frost resistance, even with somewhat extent of air entrainment.

Fiber RCC showed higher variability in density measurements, as evidenced by more significant standard deviations and coefficients of variation (COV), as shown in the relevant

table in the appendix. This increased variability could be attributed to the presence of fibers affecting the uniformity of the mix. In contrast, plain RCC exhibited more consistent density values. Overall, the findings suggest that the addition of fibers to RCC can improve its physical properties, particularly in terms of density, porosity, and water absorption. This may lead to enhanced durability and resistance to water-related damage. However, the increased variability in density measurements associated with fiber-reinforced RCC should be considered in design and construction practices. Further research could explore the long-term effects of fiber reinforcement on the durability and performance of RCC pavements under various environmental conditions and loading scenarios. Additionally, investigating methods to improve the consistency of fiber distribution in the mix could help reduce variability in density measurements.

The compressive strength test results for RCC samples reveal several notable trends and they are given in Figure 84. Plain RCC consistently outperformed fiber-reinforced RCC in terms of strength, with the difference being more pronounced at 7 days than at 28 days. Cube samples exhibited higher compressive strength compared to cylinder samples for both plain and fiber-reinforced RCC, likely due to the shape effect commonly observed in concrete testing. All samples demonstrated significant strength gain between 7 and 28 days, with fiber-reinforced cubes showing the most dramatic increase of 68.2%.

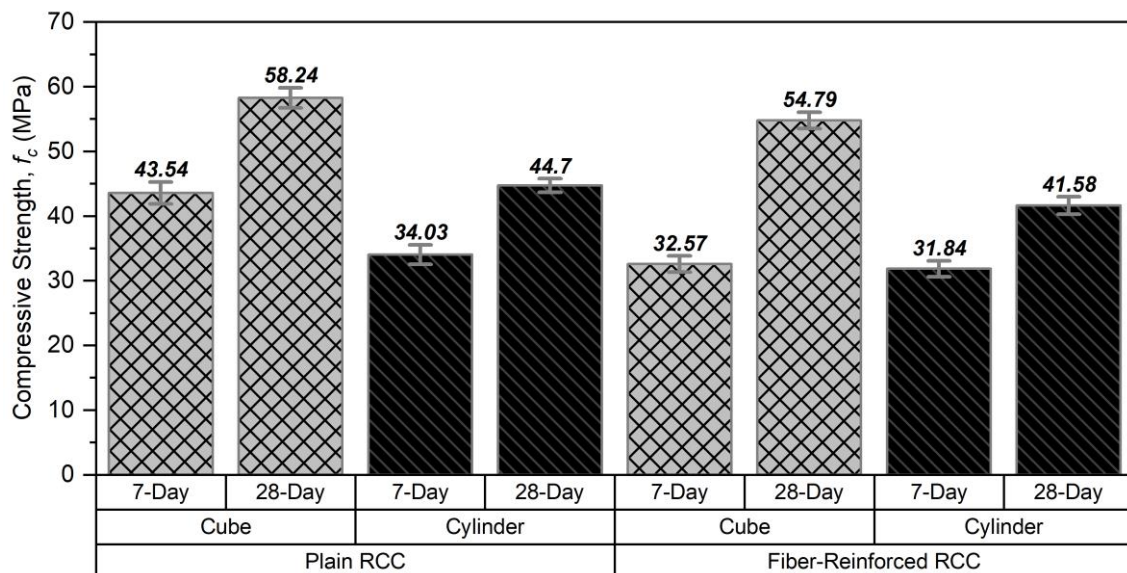


Figure 84. Compressive strength test results for RCC samples

The addition of fibers appeared to reduce early-age strength but allowed for substantial strength development by 28 days, particularly in cube samples. Low coefficient of variation (COV) values across all sample types ($<5\%$) indicated good consistency in test results and quality control in sample preparation. While plain RCC achieved higher overall strengths, the narrowing gap between plain and fiber-reinforced RCC at 28 days suggests that fiber addition may not significantly compromise long-term strength while conceptually offering other potential benefits such as improved ductility or crack resistance.

When contextualized within the existing literature, the experimental results of the RCC compressive strength tests reveal a complex interplay of factors influencing RCC performance. The plain RCC samples, with 7-day compressive strengths ranging from 34.03 MPa (cylinders) to 43.54 MPa (cubes), not only surpass the minimum requirements (24 MPa) stipulated by IDOT (2014) and specified in ACI 325.10R (ACI, 1995) as 27.6 MPa but also meet the high-performance RCC criteria (40 MPa) delineated by Reid & Marchand (1998). The plain and fiber-reinforced RCC samples met the 7-day compressive strength range noted in Harrington et al. (2010), surpassing the reported 28-day compressive strength values. This superior early-age strength, particularly in cube specimens, suggests enhanced potential for abrasion resistance, aligning with the ACI Committee 201 (ACI, 2008) recommendations. Conversely, while exhibiting lower initial strengths, the fiber-reinforced RCC samples demonstrated a remarkable strength gain of 68.2% between 7 and 28 days in cube specimens, indicating a subtle strength development pattern that deserves further investigation. Furthermore, the high compressive strengths achieved, particularly in plain RCC samples, suggest rather favorable conditions for joint strength, although as Xie et al. (2012) emphasize, the critical nature of shear strength parameters in joint zones necessitates specific testing of cohesion and friction angles to assess structural integrity comprehensively. The disparity in performance between plain and fiber-reinforced RCC, especially in early-age strength, underscores the need for a careful approach in RCC mix design, balancing immediate strength requirements with anticipated long-term performance considerations in design phase such as ductility and crack resistance.

The incorporation of experimental bending strength data obtained from no-jointed (aka fresh joint) RCC beam samples given in Figure 85 provides crucial insights into the flexural performance of both plain and fiber-reinforced RCC, offering a more comprehensive understanding of their mechanical properties against possible distress that is faced up with the time history of pavement in use. The fiber-reinforced RCC beams exhibited superior

bending strength, with mean values of 8.73 MPa and 7.18 MPa for 50x50x300-mm and 150x180x600-mm specimens, respectively. These values notably exceed the empirical estimates derived from various models, including those proposed by Chhorn et al. (2018), Tayabji & Okamoto (1987), and ACI 325 (ACI, 1995). The plain RCC beams still surpassed most empirical predictions while demonstrating lower bending strengths of 7.71 MPa and 7.13 MPa for the corresponding specimen sizes. This discrepancy between measured and estimated values underscores the limitations of existing empirical models and highlights the potential underestimation of flexural capacity in RCC design.

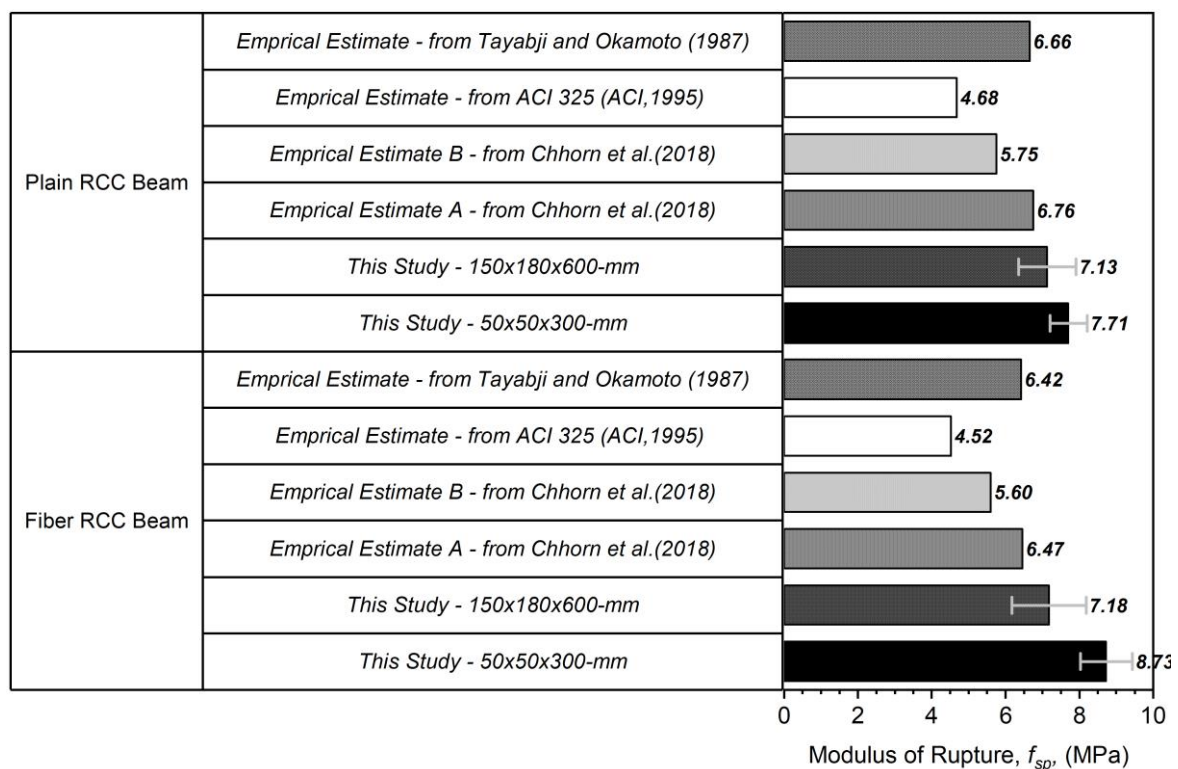


Figure 85. Modulus of rupture data from no-jointed RCC beam samples and empirical estimates

The observed size effect, manifested in the reduced bending strength of larger specimens, aligns with established fracture mechanics principles (Bažant, 1999; Zhao et al., 2016) and emphasizes the importance of scale considerations in RCC applications. Notably, the fiber-reinforced RCC beams exhibited a size effect nearly identical to that of plain RCC (17.75% reduction versus 7.53% reduction), without suggesting a pronounced complex interaction between fiber reinforcement and specimen geometry. The coefficient of variation (COV) for bending strength ranged from 6.50% to 14.00%, indicating reasonable

consistency in flexural performance, though with slightly higher variability in larger specimens and fiber-reinforced samples. The relevant table of data is in the appendix section.

Comparing the experimental rupture modulus to the 28-day compressive strengths reveals ratios significantly higher than those typically assumed in conventional concrete design. The fiber-reinforced RCC demonstrated bending strength to compressive strength ratios of 21.0% and 17.27% for small and large specimens, respectively, while plain RCC showed ratios of 17.25% and 15.95%. These ratios not only exceed the 12.5-17% range suggested by Harrington et al. (2010), 3.5-7.0 MPa but also surpass and outperform the estimates from all considered empirical models. This enhanced flexural performance, particularly in fiber-reinforced RCC, suggests potential advantages in pavement structures, where flexural capacity is a critical benchmark. Despite its lower compressive strength, the superior bending strength of fiber-reinforced RCC underscores the complex relationship between these properties and highlights the potential benefits of fiber inclusion in enhancing flexural performance. However, this observation challenges the conventional wisdom of prioritizing compressive strength and suggests that fiber reinforcement may offer a more balanced approach to RCC design, potentially improving crack resistance and durability in addition to flexural capacity. These findings not only validate earlier extrapolations of flexural performance but also necessitate a reevaluation of current design approaches and empirical models for RCC. The demonstrated enhancement in flexural capacity, particularly in fiber-reinforced specimens, along with the observed size effects, underscores the need for more detailed design methodologies that specifically account for fiber type and size, specimen geometry, and the non-linear relationship between compressive and flexural strengths in RCC. Furthermore, these findings highlight the importance of conducting direct experiments when designing RCC. Relying solely on theoretical calculations can lead to overly conservative designs, which might not fully utilize the material's potential.

The experimental studies conducted on large-scale beam samples revealed critical insights into their performance and durability. While these samples generally exhibited strength values compatible with far much smaller beam specimens, significant issues were observed during handling and testing, particularly at cold joint zones of the samples lacking interface strength. Large beam samples showed vulnerability to sudden separations or detachments during transportation and placement on the track of the loading machine. This phenomenon was not observed in smaller samples or cube specimens. This susceptibility to sudden loading raises concerns about the common practice of opening RCC pavements to

traffic shortly after construction. The findings underscore the importance of proper foundation and sub-base construction, as inadequate support can exacerbate cold joint weaknesses, especially under heavy vehicle loads. While the small and large samples were exposed to the same curing in the laboratory, some issues with infield curing could help solve the vulnerability of the longitudinal cold joint zones. It is recommended to extend curing periods before traffic exposure to address this issue, thereby considering enhanced reinforcement strategies at cold joints, improving quality control in foundation construction, and conducting comprehensive load testing, including sudden load scenarios (e.g., abrupt loading due to a very heavy vehicle with sudden braking or excessive loading during the lane change across a cold longitudinal joint whose one edge has elevated due to differential settlement) implementing rigorous monitoring programs, and updating design guidelines to account for observed vulnerabilities. These measures aim to improve RCC pavements' long-term performance and safety, particularly in addressing the challenges posed by cold joints and foundation interactions. The findings also underscore the importance of proper foundation and sub-base construction, as inadequate support can exacerbate cold joint weaknesses under heavy loads. Recommendations could include extending curing periods, enhancing reinforcement at cold joints, improving foundation quality control, conducting comprehensive load testing (including sudden load scenarios), implementing rigorous monitoring, and updating design guidelines.

All large-scale samples were tested using the Proceq Silver Schmidt Type L Concrete Test Hammer, which measures the Q-value with less energy and is suitable for concretes with strengths up to 100 MPa. The Proceq Silver Schmidt was chosen over the original (mechanical) hammer because its coil-spring mechanism is more durable for numerous sequential measurements (195 tests per beam with intermittent pauses). Measurements were taken on the side surface of the $150 \times 180 \times 600$ mm beam samples, perpendicular to the joint area rather than in the casting direction. An orthogonal grid was created on the beam using a pointwise cardboard template to mark equally spaced points. Three rebound measurements were taken at each grid point, and the average was recorded. In essence, the lateral surface areas of the beams were mapped as grids within an orthogonal coordinate system. An average Q-value for each beam model was determined by calculating the mean of all measurements. Besides, the Q-values for each grid point were averaged and assigned to that point; then, all gridded Q-values were mapped to visualize the spatial variations throughout the beam. The overall performance of each beam sample model, represented by

a single average Q-value, is shown in Figure 86. This average-based approach is commonly used to portray the overall performance in rebound tests. Additionally, the mapped Q-values are presented in Figure 87. As seen in Figure 86, no significant variations were observed among the models. Although with minor differences, four categorical groups were identified based on the Q-values assigned to each beam model. The plain and fiber reference models (P-REF and F-REF) exhibited nearly identical Q-value responses, while the PP60, PPN1, and FFN2 models appeared to perform better, potentially indicating higher strength values.

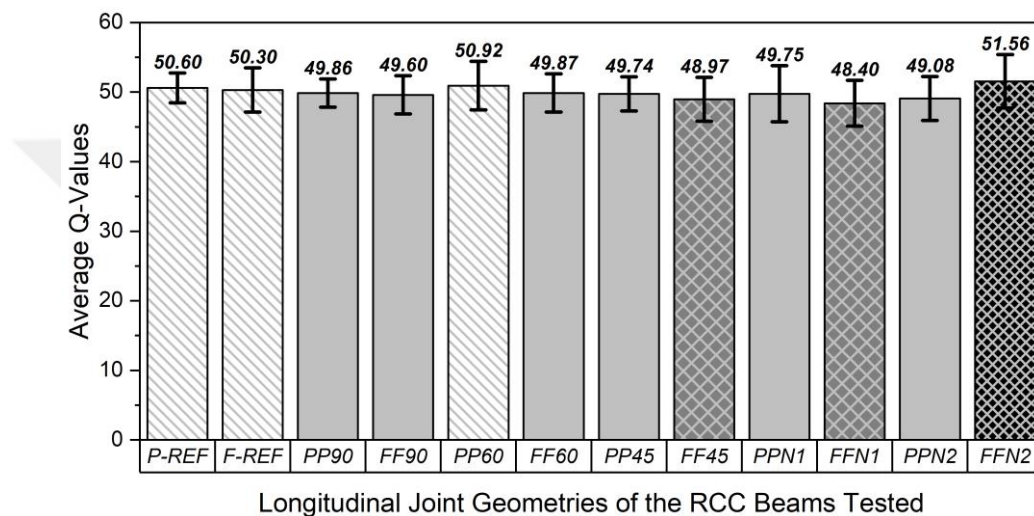


Figure 86. Average Q-values of beam samples with various cold joint models

Upon closer examination of Figure 87, noticeable differences between the beam models with varying cold longitudinal joint designs become apparent even upon initial inspection. Lower Q-values were observed along the frame boundaries corresponding to the sample edges for almost every specimen compared to those in neighboring areas. Potential measurement deviations at the edges and the restrictive effect of the mold on these areas likely contributed to this feature. However, considering that this effect is nearly absent in the reference samples, it can be inferred that the different joint geometries employed in the samples with cold longitudinal joints have implications for compaction behavior. In the fresh jointed (no-joint) samples, although P-REF and F-REF exhibited very similar overall Q-values, P-REF demonstrated a more uniform distribution of values. Conversely, F-REF displayed higher and clustered values than neighboring points both in the middle and on the right and left sides, while lower Q-values were observed in the immediate proximity of these areas, albeit with somewhat diminishing intensity.

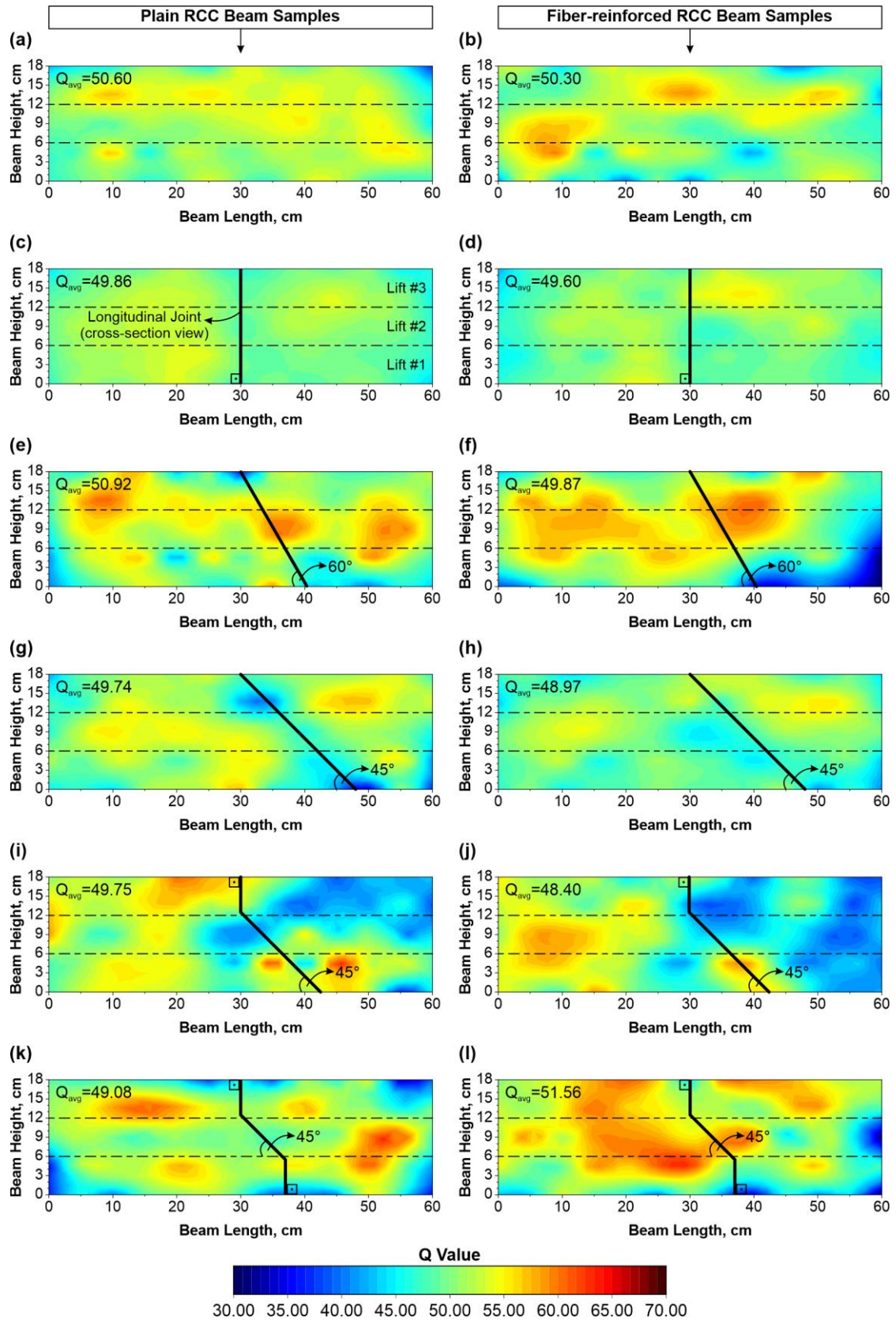


Figure 87. Spatial distribution of Schmidt hammer rebound values (Q) obtained for the tested RCC beams with different joint geometry

For the cold joint models formed with a 90° angle, both PP90 and FF90 generally exhibited homogeneous distributions similar to those of the reference samples. Although their average Q-values were slightly below those of the reference samples, the PP90 and FF90 models showed the least significant areas of low Q-values. In cold joint models formed with a 60° taper angle, both PP60 and FF60 models exhibited significantly higher and lower Q-values compared to the 90° model. On average, the PP60 model yielded slightly higher values (2%) than the FF60 model. However, as mentioned above, it would be misleading to base an evaluation solely on these averages.

For instance, low Q-values were observed at the lower end of the joint and towards the mold edge, particularly in the RCC cast as the second part. This phenomenon was especially pronounced in the FF60 model, where low Q-values were much more evident, starting from the cold joint bottom tip. High Q-value areas were also more prevalent in the FF60 model than in the PP60 model. In cold joint models formed with a 45° angle (PP45 and FF45), the distribution appeared similar to that of the 90° model in terms of homogeneity. However, a situation similar to that observed in PP60 and FF60 was also present regarding the decrease in Q-value at the taper end. Notably, in the PP45 model, areas of low Q-values were observed at the tapered end and in the upper parts of it. The N1 (single-notched wedge) and N2 (doubly-notched wedge) models exhibited distinct characteristics compared to all other models. While the N1 and N2 models are more similar to each other than to other models, they still exhibit significant differences between themselves. Specifically, the PPN1 and FFN1 models showed similarities to each other. The difference between their average Q-values was relatively small and comparable to the averages observed in other models. However, the spatial distribution of Q-values took on a markedly different form. In both the PPN1 and FFN1 models, relatively concentrated high Q-value areas were observed on the side where the first RCC side cast was made, thus defining the shape of the cold joint. Particularly in PPN1, these high-density areas were concentrated in the upper region of the beam, while in FFN1, they were predominantly located in the middle part of the beam. Interestingly, a high Q-value point is observed at the low-end tip of the wedge section. However, this high-density value transitioned to low-density areas as moving towards the notch section. In the PPN1 model, the low-value distribution on the second pouring side remained on the upper side of the mold, while in the FFN1 model, it appeared as clusters at both the upper and lower points.

In the N2 model, in terms of average values, the FFN2 model yielded higher values than the PPN2 model, contrary to the N1 model, albeit with a minimal difference. The N2 models exhibited fewer areas with low Q-values compared to the N1 models, and more areas with high Q-values were observed. The PPN2 model generally showed fewer high focal points, with the remaining areas displaying a homogeneous distribution. In contrast, the FFN2 model exhibited intense areas of high Q-values, particularly on the side of the first cast RCC. Unlike the N1 models, neither N2 model showed an increase in Q-value at the lower end tip of the wedge. On the contrary, a very rapid decrease in Q-value was observed at these points.

As another nondestructive test, the ultrasound pulse velocity features of the beam samples were aimed to be tested. This experiment investigates the ultrasound pulse velocity measurements on RCC beam samples with various configurations. The purpose is to determine whether differences in velocities can be observed amongst RCC beam models with different cold joint forms and concrete assortment and whether these velocities indicate which models are superior. The reference samples are without joints; other samples with cold joints are created in different geometries. As is known, elastic waves constitute a broad spectrum, encompassing all mechanical waves that propagate through a medium due to elasticity. Elastic waves propagate through a material as either longitudinal waves or transverse waves. Longitudinal waves, also known as P-waves or compression waves, compress and expand the medium through which direction that the wave travels. Transverse waves, or S-waves, cause particles to oscillate side-to-side, perpendicular to the wave's propagation direction. V_p denotes the velocity of the P-waves, and V_s denotes the velocity of the S-waves.

Four measurement techniques were employed in harmony with ASTM C597 (2016) using the Proceq Pundit Lab(+) and Proceq Pundit 200 (Proceq, 2021) comparatively: direct V_p (head-to-head), indirect $V_{p\#1}$ (head-to-side from old to new concrete), indirect $V_{p\#2}$ (head-to-side from new to old concrete), and direct V_s (top-to-top). The samples varied in terms of reinforcement (plain RCC or fiber-reinforced RCC), joint angle (90° , 60° , or 45°), and notch type (single-notched (N1), doubly-notched (N2)), and bonding agents (none, water, cement slurry, or epoxy-based mortar). The measurement results of ultrasound pulse velocities are shown in Figure 88.

The ultrasound pulse velocity measurements on RCC beam samples revealed interesting patterns across different joint configurations and treatments. The direct V_p

measurements showed a relatively consistent range across all samples, typically falling between 4700-5100 m/s. This consistency suggests that the overall integrity of the concrete remains largely similar despite various joint configurations and treatments. Indirect Vp measurements (both #1 and #2) generally showed slightly higher values than the direct Vp, ranging from about 4400-5300 m/s. This slight increase in indirect measurements could be attributed to the wave path potentially traveling through higher-quality surface layers of the concrete.

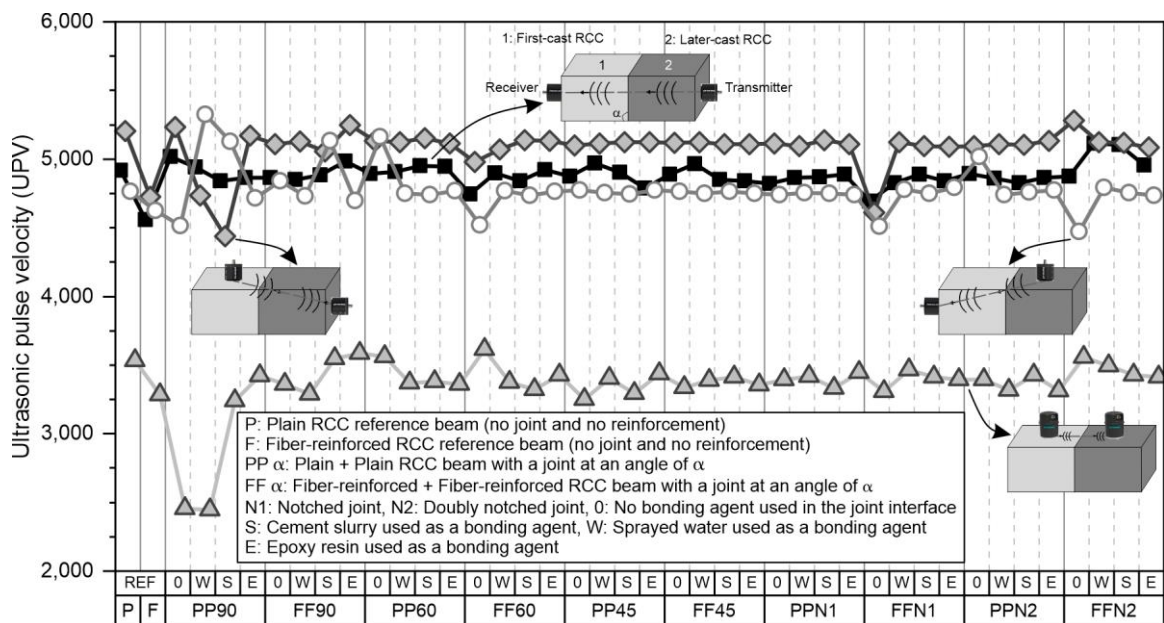


Figure 88. Ultrasonic pulse velocity measurements in RCC beam samples with various joint configurations and joint interface treatments

Interestingly, there is roughly no consistent pattern of indirect Vp#1 being higher or lower than indirect Vp#2, indicating that the direction of measurement (from old to new concrete or vice versa) did not predominantly affect the results. The direct Vs measurements presented the lowest velocities, spanning from approximately 2400 m/s to 3600 m/s. This pattern is consistent with what is expected due to the different nature of shear waves compared to compression waves. Notably, the lowest Vs values are observed in the PP90 samples with no bonding agent or water treatment, suggesting that these joint configurations might significantly impact shear wave propagation. These remarks were drawn as follows upon analyzing the effect of different treatments and joint configurations on velocities. It is noteworthy that no clear trend indicating that one joint angle (90°, 60°, or 45°) consistently performs better than others across all measurement types was observed. Fiber-reinforced

samples (FF series) often showed relatively slightly lower direct V_p values compared to their plain counterparts (PP series), but this trend was rather less pronounced in indirect measurements. The use of bonding agents (water, cement slurry, or epoxy) did not show a steady improvement in velocity measurements across all sample types. In some cases, bonded joints showed higher velocities, while unbonded joints performed similarly in others. The notched joint samples (N1 and N2) demonstrated velocity values comparable to the simply tapered joint samples, indicating that the specific geometry of the joint was of less influence on wave propagation than anticipated.

In brief, while the velocity measurements do reveal some tractable differences between various joint configurations and concrete arrangements, these differences are often subtle nevertheless. Overall examination of the data indicates that joints and different treatments have an admittedly modest impact on wave velocities in RCC samples. However, the effects of specific joint geometries, reinforcement, and employed bonding agents were not consistently pronounced across all the measurement types. This striking result itself underscores the complexity of RCC behavior, particularly at cold joints, and clearly highlights the need for a comprehensive assessment beyond just ultrasonic testing to evaluate joint performance.

A comprehensive dataset on the three-point bending test performed on RCC beam model samples with or without cold joints was compiled. Figure 89 collectively shows the rupture modulus of the tested beams. The following comparative analysis discusses the efficiency of the taper angle, the shape of the longitudinal joint, and the interface treatment methods employed.

The results demonstrated the significant impact of fiber reinforcement on the flexural strength of RCC beams. For both small (50x50x300-mm) and large (150x180x600-mm) beam samples, fiber-reinforced specimens (F-REF) exhibited higher rupture modulus values compared to plain concrete specimens (P-REF). The F-REF samples showed an average strength of 8.73 MPa for small beams and 7.18 MPa for large beams, while P-REF samples had slightly lower values of 7.71 MPa and 7.13 MPa, respectively. The improvement in flexural strength can directly be attributed to the fibers' ability to abruptly bridge cracks and then redistribute stresses, enhancing the overall ductility and load-bearing capacity of the RCC. For fiber-reinforced samples, the results suggest that different taper angles were performed variably depending on the beam size and interface treatment method.

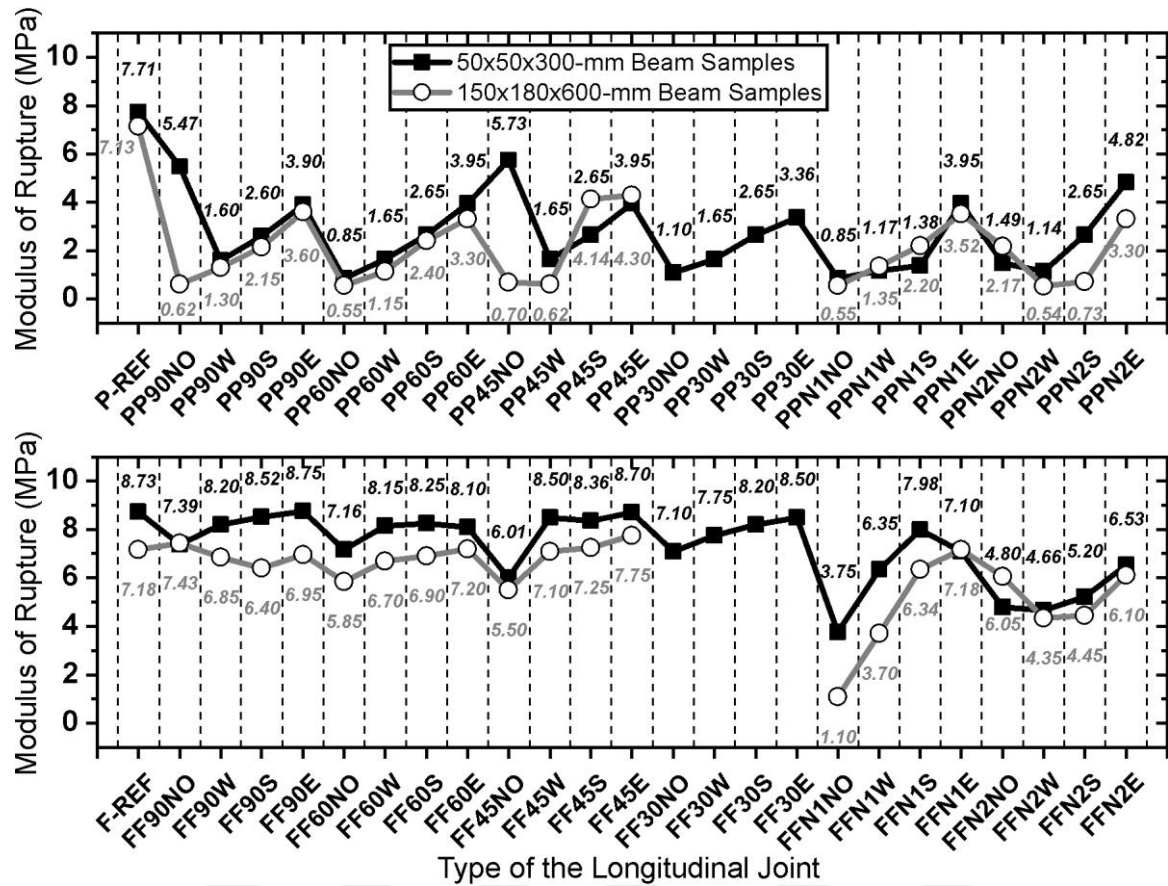


Figure 89. Rupture modulus of longitudinal fresh and cold jointed beam specimens considering interface treatment techniques

The hierarchical order of performance for epoxy-treated (E) fiber-reinforced samples was for small beams: $90^\circ > 45^\circ > 30^\circ > 60^\circ$, and for large beams: $45^\circ > 60^\circ > 90^\circ$. This pattern indicates that the 45° taper angle consistently performed well across both beam sizes, while the 90° angle showed high performance in small beams but appeared relatively lower in large beams. Tapered joints generally outperformed notched joints in fiber-reinforced samples. The N1 and N2 configurations showed lower strength values than tapered joints across different interface treatments. For instance, with epoxy bonding, FFN1E achieved 7.10 MPa and 7.18 MPa for small and large beams, respectively, compared to 8.75 MPa and 6.95 MPa for FF90E. This behavior also suggests that the tapered joints may provide better stress distribution along the joint interface compared to notched configurations. Agent E consistently outperformed other treatment methods in strengthening fiber-reinforced and plain concrete samples. This was true regardless of joint configuration or taper angle. For example, in fiber-reinforced concrete with a 90-degree taper, epoxy bonding increased strength by an average of 8.52 MPa in small beams and 6.95 MPa in large beams. Other

treatments, such as cement slurry application and water spraying, also improved but were less effective than epoxy bonding. The results reveal a marked contrast in joint performance between plain concrete and fiber-reinforced concrete specimens. Plain concrete joints consistently exhibited significantly lower strength values, particularly with no treatment method. Even with epoxy bonding, plain concrete joints failed to match the performance of untreated fiber-reinforced joints. This behavior underscores the critical role of fiber reinforcement in enhancing joint strength and overall structural integrity. Larger beam samples consistently exhibited lower rupture modulus values than smaller ones, although this trend was not always observed. This effect is likely due to the increased likelihood of flaws and stress concentrations in larger specimens, which can lead to earlier failure. However, the performance of some configurations, such as FF45E, was better in large beams, suggesting that careful joint design can mitigate size effects. The results strongly support the use of fiber reinforcement in RCC pavements, particularly in longitudinal cold joint regions. The 45° taper angle with epoxy treatment (FF45E) consistently performed well across both beam sizes, suggesting it might be an optimal configuration for cold joints when they are unavoidable. The study categorically demonstrates the significant performance gap between fiber-reinforced and plain concrete joints, with fiber-reinforced joints consistently outperforming their plain counterparts across all configurations. When cold joints are unavoidable, fiber reinforcement and appropriate interface treatments, particularly epoxy bonding, are crucial and are strongly recommended to enhance joint strength and overall pavement performance. The data overall shows that the order of the highest MR values for fiber-reinforced samples with epoxy treatment is: $90^\circ \geq 45^\circ > 60^\circ > N1 > 30^\circ > N2$, with 30° being pending due to lack of data for large beams. Epoxy treatment (E) consistently improved MR results for plain and fiber-reinforced RCC samples. The relative improvement was more visible in plain RCC tests than in fiber-reinforced, where epoxy treatment regularly expanded the MR values by 2-6 times. In fiber-reinforced samples, while epoxy treatment still provided benefits, the improvement was less striking due to the already higher baseline strength provided by the fiber reinforcement. Abut et al. (2022) conducted a study tracking the performance of RCCs for pavement construction, comparing samples cast in the laboratory to those laid in the field. They noted that almost all performance parameters obtained from the lab-cast RCC samples were found to be better than those from the field. This phenomenon was attributed to better compaction and curing conditions achieved in the laboratory. It can be stated that samples produced under laboratory conditions and in limited

sizes may introduce bias when compared to actual field conditions. However, although in-field samples are known to perform lower than laboratory-produced samples, the produced beam samples give quite good results. This generally depends on the strength properties of the produced RCC. Harrington et al. (2010) noted that the typical flexural strength of hardened RCC varies between 3.5 and 7 MPa. Our results generally support this information. This study critically explores the problem, providing valuable insights and potential solutions. While it does not provide a comprehensive solution as expected, it serves as a robust starting point for future research. This study proposes practical solutions for specific design and implementation needs by addressing industry challenges.



5. CONCLUSIONS

The present chapter of this thesis draws upon the entire study, tying up the various theoretical and empirical strands to explain the current conundrum beneath the construction of resilient cold longitudinal joints in RCC pavements and paves the way for the concluding remarks on the implications of the findings for future research in this area. Returning to the question posed at the beginning of this study, it is now possible to state that RCC pavements offer a viable and competitive alternative to HMA pavements, particularly regarding the strength and durability of longitudinal cold joints.

Upon examining the counterpart of the thesis problem in this study and potential solutions for HMA pavements, pavements, the motivation for this research is based on the premise that there should be definitive distinctions in strength and durability characteristics at and around the cold longitudinal joints of RCC pavements. These distinctions are expected to vary based on the positioning of initial roller compaction, alternative joint geometries, and the application of various new-to-old bonding agents. In that sense, the main scope of this study is to decide on the most suitable longitudinal joint construction method for RCC pavements. By understanding longitudinal joints' behavior with respect to different construction techniques, including possible alternatives in rolling operations, we aimed to shed fresh light on the issue. Before this study, there was no way of knowing which key parameters for better compaction and resilience had effectively driven the behaviors of cold longitudinal joints of RCC pavements against both compaction effort and load.

The findings below indicate that RCC's malleability helps shape the longitudinal cold joints with the desired taper angle, creating various joints in different geometries. However, how well this geometrically viable feature commingles with the compactability was one of the core subjects of this thesis. In the scope of this study, possible longitudinal joint shapes that could be applied in RCC pavement applications were first defined and then named according to those common conceptual techniques currently utilized for the longitudinal joints of HMA pavements. The physical parameters and material properties that affect the mechanical behavior of longitudinal joints were then determined and examined. Possible initial roller positions were employed upon examining available rolling techniques implemented both in RCC and HMA pavements. Based on observed mechanical behavior characteristics throughout the laboratory tests, the finite element model attributed to each

technique examined was created and analyzed through the FEA software called PrePoMax, an advanced open-source pre- and post-processor for the Calculix FEM solver, by taking into account different roller positions coupled with different cold joint models. The determined and employed joint construction techniques and initial roller positions were matched, and each model was then subjected to various destructive and nondestructive laboratory tests. Simultaneously with laboratory tests, advanced see-through scanning and digital image processing techniques were employed to determine the stress-strain behavior of the produced and modeled concrete. Potential candidate cold joint geometric arrangements (tapered at 90°, 60°, 45°, 30°, notched wedge-N1, doubly-notched wedge-N2) for RCC pavements were designed and investigated and compared with the bonding characteristics of new-to-old RCC. The reinforcement materials (water spray-W, cement slurry-S, epoxy-based repair mortar-E) were also analyzed to determine which yields the best results for adjacent joint surface reinforcement. Based on the results of this study and limited observations, the following conclusions can be drawn:

5.1. Usability of Electric Hammer in Place of Actual-Size Roller Compaction

Based on the comprehensive analysis presented, this study demonstrates the feasibility of using electric hammers as a substitute for roller compactors in molding RCC samples. The research provides required compaction times for various roller models and hammer configurations through detailed calculations and comparisons, supporting the use of electric vibrating hammers when roller compaction is unavailable. The study reveals significant variations in compaction energy produced by different tools and configurations, highlighting the potential for over-compaction in standard cylinder samples and the challenges in uniformly compacting large specimens. The findings underscore the need for a thorough revision of ASTM C1435 (2014) to include more comprehensive guidelines and reduce uncertainties in the RCC molding and testing process. Recommendations include incorporating analytical formulas that account for instrument properties, sample mold diversity, and the physical properties of fresh concrete. Additionally, specifying lift thickness limits would help users better understand how accurately the surrogate method simulates and represents actual roller compaction. These revisions, supported by further large-scale studies, would lead to more accurate simulation of field conditions, reduce

measurement uncertainties, and ultimately improve the reliability of RCC testing and quality control procedures.

5.2. Fresh RCC Roller Compaction: Strength Parameter Optimization

This study investigated the impact of roller weight on the compaction process of Roller-Compacted Concrete (RCC) pavements, comparing a heavy 10-ton roller (Case 1) with a lighter 3.2-ton roller (Case 2). Through comprehensive computational modeling and analysis, we have gained significant insights into the mechanics of RCC compaction and the relative performance of different roller types. Key findings that can be drawn from this study within its limits are as follows. Both roller types showed similar trends in Poisson's ratio as deformation increased. The heavy roller consistently achieved higher elasticity modulus values, indicating superior compaction of stiffer materials. The heavy roller produced approximately 50% higher Hertzian stresses compared to the light roller across all deformation levels. The contact area formed by the heavy roller was about 25% larger, suggesting more efficient force distribution. The heavy roller effectively compacted materials with up to 25% higher maximum elasticity moduli. Performance differences between roller types were most pronounced in high-elasticity scenarios, approaching 82%, and decreased to about 44% for lower-elasticity materials.

These findings are based on a confined but typical problem with significant implications for RCC pavement construction practices. For example, the choice between heavy and light rollers should be based on target material properties and desired compaction outcomes. Understanding the relationship between roller weight and material properties can inform the mix design decisions for optimal compaction. The superior performance of heavy rollers in specific scenarios may lead to faster achievement of target compaction levels, potentially reducing construction time and costs.

While this study provides valuable insights, it is essential to acknowledge its limitations, as is the case with any research. The study relies on computational modeling, which may not capture all-out real-world variables affecting compaction. Only two specific roller weights were examined, which may not represent the full spectrum of available equipment. The analysis primarily considered the static mode operation, while the vibratory mode is standard practice. The study focused on a specific range of material properties, which may not encompass all RCC mix designs used in practice.

Some future research could be built upon this work. Conducting field trials to verify the computational findings and assess real-world applicability could be groundbreaking. Expanding the study to include vibratory compaction modes and their effects on material behavior is worth testing. Investigating the relationship between initial compaction characteristics and long-term pavement performance and durability could be challenging but supply crucial knowledge. Evaluating the cost-benefit trade-offs of using different roller types in various project scenarios sounds exciting and can be linked to the current trend called intelligent compaction. Incorporating more complex material models to account for time-dependent behavior and heterogeneity in RCC mixes could create another dimension to this problem and would be revolutionary.

In conclusion, this study aims to provide empirical evidence to support or challenge the effectiveness of different compaction methods in RCC pavement construction, potentially influencing future industry practices and equipment selection.

5.3. Initial Roller Position on Longitudinal Cold Joint Compaction in RCC Pavements

This study offers valuable insights into the behavior of various joint configurations in RCC pavements. The hierarchical order for displacement-led performance of the cold joint models is as follows: $90^\circ > 60^\circ > N2 > N1 > 45^\circ > 30^\circ$. The clear trend of increased deformation with steeper joint angles reinforces current empirical and intuitive best practices in RCC pavement engineering. However, the complex and insufficient compaction behavior of notched wedge joints underscores the need to carefully consider their design and construction. The highest compaction achievement patterns were obtained by almost equally shifting the drum position 300 (P1) or 150 (P2) mm towards the hardened concrete side for the initial compaction. The position where the drum's outer edge is exactly parallel and touches the joint surface projection (P3) was determined as the second-best position. The initial drum positions remaining 150 (P4) and 300 mm (P5) off the joint and on the fresh side were determined as the least successful positions, respectively. As pavement engineering continues to evolve, ongoing research and field validation will be essential for optimizing joint design in terms of durability, performance, and cost-effectiveness. Investigating the behavior of cold and fresh joints under dynamic loading conditions to better simulate real-world traffic could be a valuable research topic. Additionally, longitudinal studies to understand how different joint configurations perform under normal and excessive

distress over time could provide further insights. Finally, examining the impact of temperature variations and moisture on joint deformation patterns is a worthwhile area of exploration.

5.4. Effects of Bond Strength Enhancement Materials for Cold-Jointed RCC Interfaces

This study investigates bond strength enhancement materials for fresh-to-hardened RCC interfaces, focusing on cold joint treatments in concrete pavements. In that sense, it is expected to yield valuable insights into the effectiveness of various treatments for cold joints in RCC pavements that could be used in industry. The study employed split-prism and bi-surface shear tests on 150x150x150 mm cubic specimens, revealing distinct bonding strength aspects through each method. Epoxy mortar emerged as the most effective treatment agent, particularly in bi-surface shear tests, where it substantially improved bonding strength, exceeding ACI (2006) standards. Cement slurry also proved viable, especially in split prism tests. Surprisingly, epoxy treatment underperformed in split-prism tests compared to water and cement slurry treatments, possibly due to failure occurring adjacent to the epoxy film rather than within itself. Water treatment showed a slightly modest improvement over the untreated joints, while fiber reinforcement enhanced the overall performance of concrete joints to a moderate extent across all treatment types. Distinct failure modes were observed for various treatments, with epoxy-treated joints exhibiting characteristic behavior apart from the others. Bi-surface shear tests generally yielded higher bonding strengths than split prism tests, attributed to mechanical interlocking and frictional resistance at the interface. While friction coefficients were not directly measured, their importance was discussed, referencing critical findings from previous research. The original findings presented herein align with and expand upon existing literature on the RCC-to-RCC bonding phenomenon, fiber incorporation, and joint interface treatment with bonding agents.

The finite element analysis using Calculix/PrePoMax provided valuable insights into the behavior of RCC interfaces under shear stress. While the PPE, PPS, and PPW models yielded realistic results closely matching the experimental observations, the PPNO model showed somewhat deviation. This discrepancy suggests that for untreated joints (PPNO), a Mohr-Coulomb model might be more appropriate than a simple Mohr model, indicating that friction plays a substantial role in the apparent bond strength of untreated interfaces. These numerical findings not only complement the experimental results but also highlight the

complex nature of bonding mechanisms in untreated RCC cold joints. This study demonstrates that bonding mechanisms can be effectively replicated or modeled when basic tests are performed, and fundamental knowledge is available. This promising outcome underscores the potential for developing more accurate predictive models for RCC joint behavior, which may significantly enhance the design and performance of RCC pavements in the future.

However, innate limitations of the present study directly include a focus on specific treatments and short-term bonding strength, highlighting the need for long-term durability research. All in all, the results of this study are crucial for developing best practices that the RCC pavement construction industry has long demanded. By comparing experimental results with industry standards, this research underscores the importance of selecting appropriate bonding treatments to initially meet and potentially exceed the allowable bond strength limits, ultimately contributing to safer, long-lasting, and more resilient RCC pavements. To the best of my knowledge, future research directions should focus on several key areas. Firstly, exploring a broader range of bonding treatments is essential. Investigating long-term durability under various everyday and extreme traffic distress and variable environmental conditions is also crucial. Experiments should be conducted on larger specimens or, better yet, actual pavement sections. Additionally, analyzing friction coefficients and their impact on bond strength would provide valuable insights into the design of RCC pavements. Developing numerical models to predict bonded interface behavior is another vital thoroughfare. Implementing pilot projects would help validate these findings in real-world conditions. In a nutshell, this study attempted to advance the understanding of how different interface treatments and materials can enhance the bonding performance of cold joints in RCC pavements, providing valuable guidance for engineers and contributing to developing more durable and efficient concrete infrastructure.

5.5. Influence of Joint Taper Angles and Interface Treatment for Cold RCC Joints

This study investigated the efficacy of various bonding treatments and joint configurations in RCC beam samples with cold joints. Fiber reinforcement was significantly improved flexural strength and durability. Tapered joints outperformed the notched joints in terms of strength and stress distribution. Epoxy treatment was found to be the most effective interface treatment for enhancing joint strength. Cold joint weaknesses were established as

susceptible to sudden separations and detachments, emphasizing the importance of proper foundation and curing in-field constructions. Of all non-destructive testing, ultrasonic pulse velocity measurements provided insights into joint performance, but results were often subtle. The 45° and 90° taper angles with epoxy treatment consistently performed well across both beam sizes, suggesting it might be an optimal configuration for cold joints when unavoidable. Fiber reinforcement and appropriate interface treatments, particularly epoxy bonding, were found to be rather crucial factors for enhancing joint strength and overall pavement performance, especially in specific cases where cold joints cannot be entirely avoided. The results categorically demonstrated the significant performance gap between fiber-reinforced and plain concrete joints. Fiber-reinforced joints consistently outperformed their plain counterparts across all configurations, highlighting the importance of fiber reinforcement in improving joint strength and durability. The results overall show that the order of the highest MR values for fiber-reinforced samples with epoxy treatment is $90^\circ \geq 45^\circ > 60^\circ > N1 > 30^\circ > N2$, with 30° being pending due to lack of data for large beams. Epoxy treatment (E) consistently improved MR results for plain and fiber-reinforced RCC samples. The relative improvement was more visible in plain RCC tests than in fiber-reinforced, where the E regularly increased the MR values by 2-6 times. In fiber-reinforced samples, while epoxy treatment still provided benefits, the improvement was less striking due to the already higher baseline strength provided by the fiber reinforcement. While lab-produced samples might exhibit some bias, they still provide valuable insights and are a robust starting point for future research. The findings offer practical guidance for engineers and contractors involved in designing and constructing RCC pavements, leading to more durable and resilient infrastructure. Considering that this study is a fundamental experimental study, further research should focus on actual-scale testing under field conditions to validate the findings and assess the performance of the proposed joint configurations and interface treatments in real-world applications.

5.6. Concluding Remarks and Prospects

This chapter presents some closing thoughts on the concepts and findings discussed in the previous chapters of this thesis. Considering the genuine benefits expected from any pavement structure constructed with even the highest quality building materials, no type of pavement is unbeatable and perfect, nor does it forever stay resilient and solid. Asphalt

pavements' demise seems unlikely until the oil reserves run out on the Earth. However, the fierce rivalry between roller-compacted concrete pavements and asphalt pavements will continue for more sustainable, eco-friendly, and economical pavement alternatives. With this in mind, it seems reasonably assertable and rationally desirable that the use of RCC in pavements will increase at an incremental speed, not only in Türkiye but in most parts of the world. A significant advantage of using RCC in pavement construction is its ability to expedite traffic reopening, especially when an entire lane is cast within a single work shift. Ease of installment, combined with the material's durability and the ability to utilize domestic resources, makes RCC a compelling choice for future pavement projects in Türkiye. As the global trend shifts towards more sustainable and cost-effective pavement materials, RCC is poised to play an increasingly significant role in constructing resilient infrastructure. While it is feasible to construct the majority of new road pavements using concrete and its specialized variants produced from Türkiye's domestic resources, the current preference for the use of asphalt, a largely imported material of which Türkiye is not a leading producer, is likely to shift soon. Successful plain concrete and RCC pavement applications may reasonably drive this change. Overall, the stereotyped trend in surface course material selection in Türkiye, which has shifted slightly towards RCC in some cities, is no longer economically and technically untenable for the Turkish engineering community. To the best of my knowledge, it is no longer surefire that the realm of HMA pavements appears unbeatable due to the outstanding merits of RCC.

Flexible pavement design experiences offer a superabundance of lesson resources. However, there are still relatively complex yet untouched undercurrents beneath the smooth pictures of RCC pavement design practices and particularly attempts to forecast the economic life of the pavements. For asphalt pavements, the reasons for the change and differences in mechanical and durability properties of the longitudinal joints and the pavement mat have been elucidated by several studies; a great majority of them were subsidized by the National Center for Asphalt Technology (NCAT) at Auburn University, US. However, all the efforts made to date have not yet yielded unequivocal answers to questions regarding the universally best joint construction technique and its long-term performance. In the realm of roller-compacted concrete pavements, the research on the more pertinent technical details for longitudinal joint construction has not been made clear to date. While very little semi-methodically derived information was available on this issue, this study could open a new discussion on the main topic.

In this study, several engineering parameters were utilized to assess the soundness, integrity, and quality of potential longitudinal joint types deemed suitable for RCC pavements, along with several evidence-based recommendations for future practices. Analysis-based recommendations and suggestions were introduced to understand better the differences in behavior characteristics of the longitudinal joints of RCC pavements investigated and to improve their quality.

1. It is shown that the surrogate RCC compaction method of ASTM C1435 is effective for its designed 150 mm diameter cylinders but proves excessive for smaller samples and insufficient for larger ones. The study reveals that when simulating rollers with diverse properties, the method often yields over-compaction even in standard-sized samples, only providing appropriate compaction for rollers within a specific energy range. This research defines the standard's practical limits and proposes directions for improvement in a more universally applicable surrogate compaction method.
2. Incorporating 1% polypropylene fiber into RCC increased the rupture modulus (MR) by approximately 15% compared to plain RCC, although it did not significantly enhance compressive strength. Plain RCC exhibited a lower void ratio and water absorption when compared to fiber-reinforced RCC.
3. A cost-benefit ratio analysis suggests that, although less beneficial than E, methods S and W are preferable due to their cost-effectiveness. The S technique generally resulted in better strength values, while leaving joints untreated was the least effective; a thin water film applied by spraying was found to be beneficial. The hierarchical benefit of joint interface reinforcement methods, compared to samples without reinforcement (NO), was $E > S > W > NO$, respectively.
4. From the strength tests, models with N2 and 30° geometries were the least favorable, while 90° and 45° offered similar benefits. The N1 technique performed comparably to the 60° option, and method S provided results as good as, or sometimes better than, water spraying. The order of the highest modulus of rupture values was obtained as $90^\circ > 45^\circ > 60^\circ > N1 > N2 > 30^\circ$.
5. Finite element analyses revealed that the cold longitudinal joint models achieved optimal compaction performance in the following order: $90^\circ > 60^\circ > N2 > N1 > 45^\circ > 30^\circ$. The most effective initial roller positions were found to be in the order of $P1 \geq P2 > P3 > P4 \geq P5$.

This research has made several significant contributions to the field of pavement engineering.

1. With this study, it was introduced and tested innovative joint construction techniques for RCC pavements, providing a framework for future research and practical applications.
2. A comprehensive analysis of the mechanical behavior of longitudinal joints in RCC pavements was provided, filling a critical gap in the existing literature.
3. It offered practical recommendations for constructing resilient cold longitudinal joints in RCC pavements, contributing to developing more durable and sustainable pavement systems.
4. The understanding of the interplay between joint geometry, compaction methods, and material properties was enhanced, leading to more effective design and construction practices.

Upon completing this study, testing the recommended techniques for longitudinal joint construction in practice became possible. However, more research is needed to evaluate the effect of several parameters, including aggregate gradation, mixture properties, and geometrical features, that were kept constant to simplify the problem in this study. I firmly believe that the findings drawn upon this study are rather rewarding and elucidatory unless massively countermanded by a well-rounded study of real-life design, construction, and traffic experiences.

6. RECOMMENDATION FOR FUTURE WORKS

The RCC technology is enjoying a boom in both capabilities and utilization in Türkiye, albeit relatively recently. Despite the increasing frequency and scale of RCC pavement constructions, it is evident that several challenges persist, particularly in the realm of cold longitudinal joint integrity, calling for further advancements in construction practices. While there have been notable improvements in various aspects of RCC pavement technology, the prevention of cracking in cold longitudinal joints remains a significant hurdle.

This study brings forth novel insights into an original and yet unresolved subject requiring much more comprehensive exploration. The issue addressed in this thesis is not exclusive to RCC pavements but is also encountered in HMA pavements, where a clear consensus has yet to be reached. The findings presented here strongly advocate for future research endeavors to delve deeper into this underexamined problem within the domain of RCC pavement technology.

Tackling such a prolonged and intricate engineering problem necessitates more than a single, limited study. While this study has leveraged available resources to address the problem, inherent uncertainties are unavoidable in experimental and analytical scientific endeavors. Therefore, the presented findings are acknowledged with some approximation to the underlying reality. I firmly believe that refining and universalizing these findings could be achieved with reduced financial and physical constraints on the scope of this study. In summary, the results of this study propose several promising avenues for future research to elevate the performance and reliability of RCC pavement models.

Outlined below are some suggestions for future studies that I bear in mind.

1. An area of great promise lies in determining a robust and functional yielding theory for the mechanistic analysis of RCC pavements. There is a noticeable absence of a comprehensive and well-defined mechanical yielding theory suitable for fresh and hardened RCC. Despite this gap, certain valuable similarities between conventional concrete and hardened RCC provide a foundation for finite element analyses. Moreover, this study has demonstrated the effectiveness of the theoretical soil-like material analogy in modeling fresh RCC. Nevertheless, further methodological work is imperative to gain a deeper understanding of the mechanical model both for fresh and hardened RCC.

2. Developing new fatigue relations tailored to RCC could offer valuable insights into the diverse mechanical behaviors exhibited by longitudinal joints employing various construction techniques. This initiative would enhance our ability to estimate the long-term performance of longitudinal joints subjected to repetitive and high-frequency loadings.
3. Pioneering a time-dependent mechanical yielding model for RCC, especially during the transition from the fresh to the hardened states without compromising resilience, would be groundbreaking. This innovation could prove particularly beneficial in urgent scenarios, such as the immediate opening of military airport terminals.
4. An extensive study on RCC-to-RCC bonding efficiency under diverse physical conditions, whether ordinary or harsh, would be valuable. Investigating how RCC's mechanical resilience responds to various climatic cycles, considering different water-to-cement (w/c) ratios and curing conditions, could provide useful insights.
5. Utilizing the tested models and findings from this study for a comparative investigation into the effectiveness of entirely fresh joint construction techniques with different initial rolling positions would be innovative indeed.
6. Developing an easy-to-use laying technique without the need for saw-cutting, potentially incorporating new edge restraint devices and apparatus, would be practical and patentable.
7. Exploring the effective use of rumble strip grooves (milling or tooling) in RCC pavements, both fresh and hardened, is a novel and unexplored topic. Rumble strips can conveniently be installed on various sections of roadways, i.e., at the centerline or the edge of two-lane pavements, or most of the time on the freeway shoulders. To the best of my knowledge, for the time being, it seems impossible to install rumble strips on a surface course of a finished and just deployed RCCP using the same methods ordinarily implemented in HMA pavements. From this point on, investigating the installation of rumble strips on fresh RCC surfaces, especially in comparison to methods used in HMA pavements, could yield valuable insights.
8. Assessing the impact of rumble strips on the resilience and economic life of longitudinal joints in RCC pavements would be an exciting avenue for further investigation.

9. Identifying cross-related or interdependent factors influencing the lifelong performance and economic lifespan of RCC pavements would provide valuable insights.
10. Determining the advantageous or adverse effects of joint construction techniques in two-layer and inverse-type RCC pavements would be groundbreaking and instrumental for practical construction practices.
11. Conducting full-scale comprehensive studies that consider the aging of RCC pavements and the time history of traffic loads on the overall performance of longitudinal joints would be highly beneficial.



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8. APPENDIXES

APPENDIX 1. A MATLAB code snippet to parametrically compute the theoretical mechanical properties of a compactable half-space using nonlinear optimization based on Hertzian contact theory (Page 1/5)

```

clear
close all
format longG
tic
% This code was written by me (Murat NAS, 06/2024) and it is just for
% calculating the elasticity modulus and Poisson's ratio pairs with nonlinear
% optimization based on the Hertzian contact theory, considering the given
% indentation distances of a single cylindrical drum under constant load
% resting on a frictionless elastic half-space.
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% I set the display option off for each iteration, so you will see only the
% prompts that say, "GlobalSearch stopped because it analyzed all the trial
% points. All # of local solver runs converged with a positive local solver
% exit flag." This states that the algorithm is running in the background,
% letting you know it is still flowing.
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% I am setting the resolution (in DPI) for saving all the figures later on
dpi = 300; % this is for all the plots that can also be adjusted as needed
% first step is to give the data for the pointwise Hertzian problem
num_arrays = 500; % this allows us to run recursive optimization,
% If you trust your computer, you can increase this value as much as you want.
d_target_lower = 0.5; % I preset the limits of the target displacement, all in mm
d_target_upper = 20;
% Producing a linear array of target displacements in mm
d_targets = linspace(d_target_lower, d_target_upper, num_arrays);
R = 650; % Radius of the roller in mm
E_roller = 210 * 10^9; % Fixed modulus of elasticity for the roller (MPa),
% it is already roughly nondeformable
v_roller = 0.30; % Fixed Poisson ratio for the roller, it comes directly from steel
F = 50000; % the drum's load is in Newton either fixed or as is below
linear_load_of_drum = 30.9; % in kg/cm just as given in the catalogue.
length_of_the_drum = 1700; % in mm, it is mainly given as mm in the catalogs
% Now, the initial force in N from the features of roller employed
F = linear_load_of_drum * length_of_the_drum / 10 * 9.80665;
% by the way, let us define font size variables for the all figures
axisFontSize = 12;
titleFontSize = 12;
colorbarFontSize = 12;
heatmapCellFontSize = 12; % If your spacing value is small, it will be
% necessary to reduce the font size. The settings of the spacing at the
% beginning of the figure 3.
% I initialize arrays to later store optimized values easily,
% it is a well-known matlab prerequisite
E_optimized = zeros(num_arrays, 1);
v_optimized = zeros(num_arrays, 1);
% I now create an optimization problem and perform optimization for each d_target
% The bounds and methods are crucial and finely tuned.
% this is the most crucial issue in this code.
% the rest is made automatically. note that I have included a few modification
% opportunities for figures 3 and 4 below if you wish to use it.
for i = 1:num_arrays
    d_target = d_targets(i);
    options = optimset('Display', 'off', 'MaxIter', 1e5);
    problem = createOptimProblem('fmincon', 'objective', ...
        @(x) objectiveFunction(x, F, R, E_roller, v_roller, d_target,
            length_of_the_drum), ...
        'x0', [1, 0.30], 'lb', [1, 0.31], 'ub', [120, 0.40], ...
        'options', options);
    % here the codes perform optimization using global search,
    % using default settings
    gs = GlobalSearch;
    x = run(gs, problem);
    % I want to extract and store optimized values to use in figures later
    E_optimized(i) = x(1);
    v_optimized(i) = x(2);
end
% For ease of use, I save the results to a text file,
results = [d_targets', E_optimized, v_optimized];
for i=1:length(results)
    % Using the results we can compute the Estar for each pair
    Estar_backwards = 1 ./ (((1 - v_optimized.^2) ./ E_optimized) + ((1 - v_roller.^2)
        / E_roller));

```

APPENDIX 1. (Continued, Page 2/5)

```

72 end
73 for i=1:length(results)
74     % Half width of contact zone
75     a_zone = ((2 * F * 2 * R)/(pi * length_of_the_drum .* Estar_backwards)).^(1/2);
76     % Maximum Hertzian pressure in the contact zone
77     Pmax = ((2 * F .* Estar_backwards)/(pi * length_of_the_drum * 2 * R)).^(1/2);
78 end
79 % I better added the a_zone and pmax data to the results
80 results = [results, a_zone, Pmax];
81 % then I wanted to add the headers beacuse it became somewhat complicated matrix
82 original_headers = {'d_targets', 'E_optimized', 'v_optimized', 'a_zone', 'Pmax'};
83 results_with_headers = array2table(results, 'VariableNames', original_headers);
84 % providing several alternatives for the user's preference
85 writetable(results_with_headers, 'optimization_results.xlsx');
86 writetable(results_with_headers, 'optimization_results.csv');
87 writetable(results_with_headers, 'optimization_results.txt', 'Delimiter', '\t');
88 % Everthing already is OK and we are ready to plot the results
89 % Just look at the command window to see the thread of the optimization
90 % flow and see the final remark that denotes the code termination
91 % % %
92 % a simple panel plot
93 figure(1);
94 subplot(2, 1, 1);
95 plot(d_targets, E_optimized, '-o', 'Color', [1 0 0]);
96 xlabel('\bf d_target (mm)', 'FontSize', axisFontSize);
97 ylabel('\bf E_opt (MPa)', 'FontSize', axisFontSize);
98 title('(a) Optimized E vs. d_target', 'FontSize', titleFontSize);
99 grid on;
100 ax1 = gca;
101 ax1.LineWidth = 1.15;
102 ax1.FontSize = axisFontSize;
103 subplot(2, 1, 2);
104 plot(d_targets, v_optimized, '-o', 'Color', [0 0 1]);
105 xlabel('\bf d_target (mm)', 'FontSize', axisFontSize);
106 % ylabel('\bf Poisson Ratio (\nu)');
107 ylabel('\bf \nu_opt', 'FontSize', axisFontSize);
108 title('(b) Optimized \nu vs. d_target', 'FontSize', titleFontSize);
109 grid on;
110 ax1 = gca;
111 ax1.LineWidth = 1.15;
112 ax1.FontSize = axisFontSize;
113 % I save the plots as JPEG, EPS, and vector PDF with given DPI
114 print(gcf, 'Fig1_optimized_plot.jpg', '-djpeg', ['-r' num2str(dpi)]);
115 print(gcf, 'Fig1_optimized_plot.eps', '-depsc', ['-r' num2str(dpi)]);
116 print(gcf, 'Fig1_optimized_plot.pdf', '-dpdf', ['-r' num2str(dpi)]);
117 % % % fig 1 is done
118 % a 2D colormap using a color scale, it is set being the jet
119 figure(2);
120 scatter(d_targets, E_optimized, [], v_optimized, 'filled');
121 colormap(jet);
122 colorbar;
123 xlabel('\bf d_target (mm)');
124 ylabel('\bf E_opt (MPa)');
125 title('(c) E_opt vs. d_target with \nu_opt');
126 grid on;
127 box on;
128 h = colorbar;
129 ax2 = gca;
130 ax2.LineWidth = 1.15;
131 ax2.FontSize = axisFontSize;
132 % title(h, 'Poisson Ratio (\nu)'); % non-bold version
133 title(h, '\bf \nu_opt');
134 print(gcf, 'Fig2_optimized_plot_filled_dots.jpg', '-djpeg', ['-r' num2str(dpi)]);
135 print(gcf, 'Fig2_optimized_plot_filled_dots.eps', '-depsc', ['-r' num2str(dpi)]);
136 print(gcf, 'Fig2_optimized_plot_filled_dots.pdf', '-dpdf', ['-r' num2str(dpi)]);
137 % % %
138 % The below part is an advanced exercise for plotting the heatmaps.
139 % They are given here because it is hard to explain at the beginning of
140 % the code how the parameters are to be set without seeing the flow of the below.
141 %
142 % remembering that the results is a data matrix called "results",
143 % which was in the form of [d_targets', E_optimized, v_optimized]
144 % Extract the computed data from results matrix using the current shape of them

```

APPENDIX 1. (Continued, Page 3/5)

```

145 d_targets = results(:,1);
146 E_optimized = results(:,2);
147 v_optimized = results(:,3);
148 % now let us define bin edges. Namely I am setting the ranges of the bins
149 % depending on what I want to see in increments on the x and y axis values
150 x_axis_incl = 2.0;
151 y_axis_incl = 2.0;
152 x_edges = min(d_targets):x_axis_incl:max(d_targets);
153 y_edges = min(E_optimized):y_axis_incl:max(E_optimized);
154 % simple hist that we need indeed
155 [N, Xedges, Yedges] = histcounts2(d_targets, E_optimized, x_edges, y_edges);
156 % this is a bit tricky for calculating the mean v_optimized for each bin
157 v_binned = zeros(size(N));
158 for i = 1:size(N, 1)
159     for j = 1:size(N, 2)
160         bin_indices = d_targets >= Xedges(i) & d_targets < Xedges(i+1) & ...
161             E_optimized >= Yedges(j) & E_optimized < Yedges(j+1);
162         if any(bin_indices)
163             v_binned(i,j) = mean(v_optimized(bin_indices));
164         else
165             v_binned(i,j) = NaN;
166         end
167     end
168 end
169 % now we can create the heatmap
170 figure(3);
171 % Add an extra row and column to v_binned
172 v_binned_modified = [v_binned, v_binned(:,end)];
173 v_binned_modified = [v_binned_modified; v_binned_modified(end,:)];
174 h = pcolor(Xedges, Yedges, v_binned_modified);
175 set(h, 'EdgeColor', 'none');
176 colormap(jet);
177 c = colorbar;
178 c.Label.String = '\bf \nu\_{opt}';
179 c.Label.FontSize = colorbarFontSize;
180 c.FontSize = colorbarFontSize;
181 xlabel('\bf d\_{target} (mm)');
182 ylabel('\bf E\_{opt} (MPa)');
183 title('(d) Binned Heatmap of \bf \nu\_{opt}');
184 ax3 = gca;
185 ax3.LineWidth = 1.15;
186 ax3.FontSize = axisFontSize;
187 set(gca, 'YDir', 'normal');
188 grid on;
189 set(gcf, 'Position', [100, 100, 600, 500]);
190 for i = 1:size(v_binned, 1)
191     for j = 1:size(v_binned, 2)
192         x_pos = (Xedges(i) + Xedges(i+1)) / 2;
193         y_pos = (Yedges(j) + Yedges(j+1)) / 2;
194         if ~isnan(v_binned(i,j))
195             text(x_pos, y_pos, sprintf('%.3f', v_binned(i,j)), ...
196                 'HorizontalAlignment', 'center', ...
197                 'VerticalAlignment', 'middle', ...
198                 'Color', 'w', ...
199                 'FontWeight', 'bold', ...
200                 'Rotation', 0, ...
201                 'FontSize', heatmapCellFontSize-6);
202         end
203     end
204 end
205 xlim([min(d_targets) max(d_targets)])
206 print(gcf, 'Fig3_Heatmapfor_optimized_plot_for_nu.jpg', '-djpeg', ['-r' num2str(dpi)]);
207 print(gcf, 'Fig3_Heatmapfor_optimized_plot_for_nu.eps', '-depsc', ['-r' num2str(dpi)]);
208 print(gcf, 'Fig3_Heatmapfor_optimized_plot_for_nu.pdf', '-dpdf', ['-r' num2str(dpi)]);
209 % below code is nothing the same with above figure-producing code
210 % but using the different values
211 % let me define bin edges again with the indices of 2
212 % note that bin sizes directly affect the heat map. Some cogitation is needed.
213 x_axis_inc2 = 2.0;
214 y_axis_inc2 = 0.01;
215 x_edges_2 = min(d_targets):x_axis_inc2:max(d_targets)+x_axis_inc2;

```

APPENDIX 1. (Continued, Page 4/5)

```

216 y_edges_2 = min(v_optimized):y_axis_inc2:max(v_optimized);
217 % hist again
218 [N, Xedges_2, Yedges_2] = histcounts2(d_targets, v_optimized, x_edges_2, y_edges_2);
219 % [N, Xedges_2, Yedges_2] = histcounts2(d_targets, E_optimized, x_edges_2, y_edges_2);
220 E_binned_2 = zeros(size(N));
221 for i = 1:size(N, 1)
222     for j = 1:size(N, 2)
223         bin_indices_2 = d_targets >= Xedges_2(i) & d_targets < Xedges_2(i+1) & ...
224             v_optimized >= Yedges_2(j) & v_optimized < Yedges_2(j+1);
225         if any(bin_indices_2)
226             E_binned_2(i,j) = mean(E_optimized(bin_indices_2));
227         else
228             E_binned_2(i,j) = NaN;
229         end
230     end
231 end
232 % % %
233 figure(4);
234 % Add an extra row and column to E_binned_2
235 E_binned_2_modified = [E_binned_2, E_binned_2(:,end)];
236 E_binned_2_modified = [E_binned_2_modified; E_binned_2_modified(end,:)];
237 h = pcolor(Xedges_2, Yedges_2, E_binned_2_modified);
238 set(h, 'EdgeColor', 'none');
239 colormap(jet);
240 c = colorbar;
241 c.Label.String = '\bf E\_{opt} (MPa)';
242 c.Label.FontSize = colorbarFontSize;
243 c.FontSize = colorbarFontSize;
244 xlabel('\bf d\_{target} (mm)');
245 ylabel('\bf \nu\_{opt}');
246 title('(e) Binned Heatmap of \bf E\_{opt} (MPa)');
247 ax4 = gca;
248 ax4.LineWidth = 1.15;
249 ax4.FontSize = axisFontSize;
250 set(gca, 'YDir', 'normal');
251 grid on;
252 set(gcf, 'Position', [100, 100, 600, 500]);
253 for i = 1:size(E_binned_2, 1)
254     for j = 1:size(E_binned_2, 2)
255         x_pos_2 = (Xedges_2(i) + Xedges_2(i+1)) / 2;
256         y_pos_2 = (Yedges_2(j) + Yedges_2(j+1)) / 2;
257         if ~isnan(E_binned_2(i,j))
258             text(x_pos_2, y_pos_2, sprintf('%0.2f', E_binned_2(i,j)), ...
259                 'HorizontalAlignment', 'center', ...
260                 'VerticalAlignment', 'middle', ...
261                 'Color', 'w', ...
262                 'FontWeight', 'bold', ...
263                 'Rotation', 90, ...
264                 'FontSize', heatmapCellFontSize);
265         end
266     end
267 end
268 xlim([min(d_targets) max(d_targets)])
269
270 print(gcf, 'Fig4_Heatmapfor_optimized_plot_for_E.jpg', '-djpeg', ['-r' num2str(dpi)]);
271 print(gcf, 'Fig4_Heatmapfor_optimized_plot_for_E.eps', '-depsc', ['-r' num2str(dpi)]);
272 print(gcf, 'Fig4_Heatmapfor_optimized_plot_for_E.pdf', '-dpdf', ['-r' num2str(dpi)]);
273 % % % % %
274 clc;
275 disp(" ");
276 disp('Note that all generated products are saved where this code snippet is located
277 by you. ');
277 disp(
278     "
279 ");
278 disp('The operations have successfully been completed. ');
279 % that is all
280 toc
281 % Below function is also a crucial part of the entire code and should not be
282 % discarded.
282 function obj = objectiveFunction(x, F, R, E_roller, v_roller, d_target,
283     length_of_the_drum)
283     E = x(1);

```

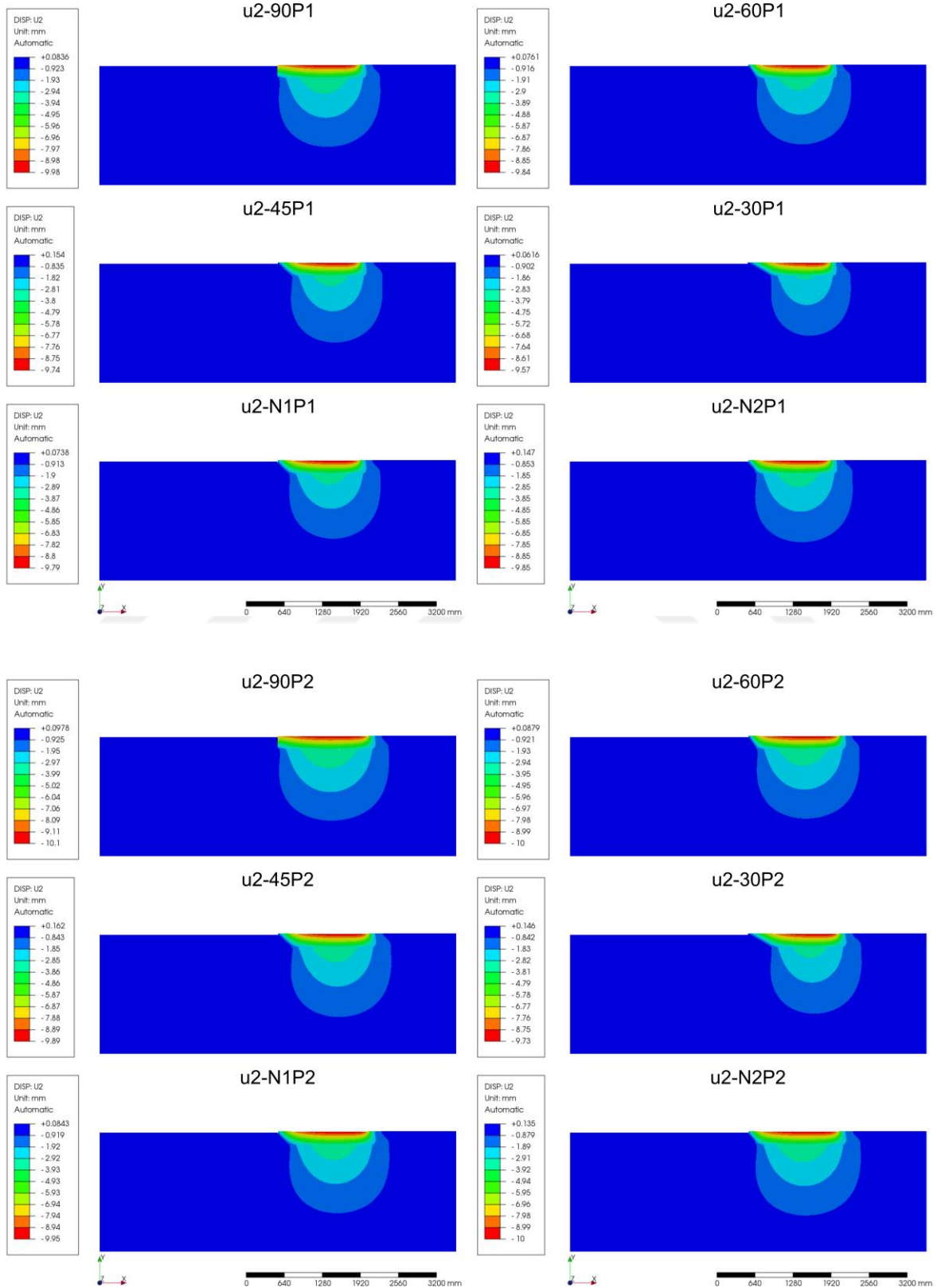
APPENDIX 1. (Continued, Page 5/5)

```

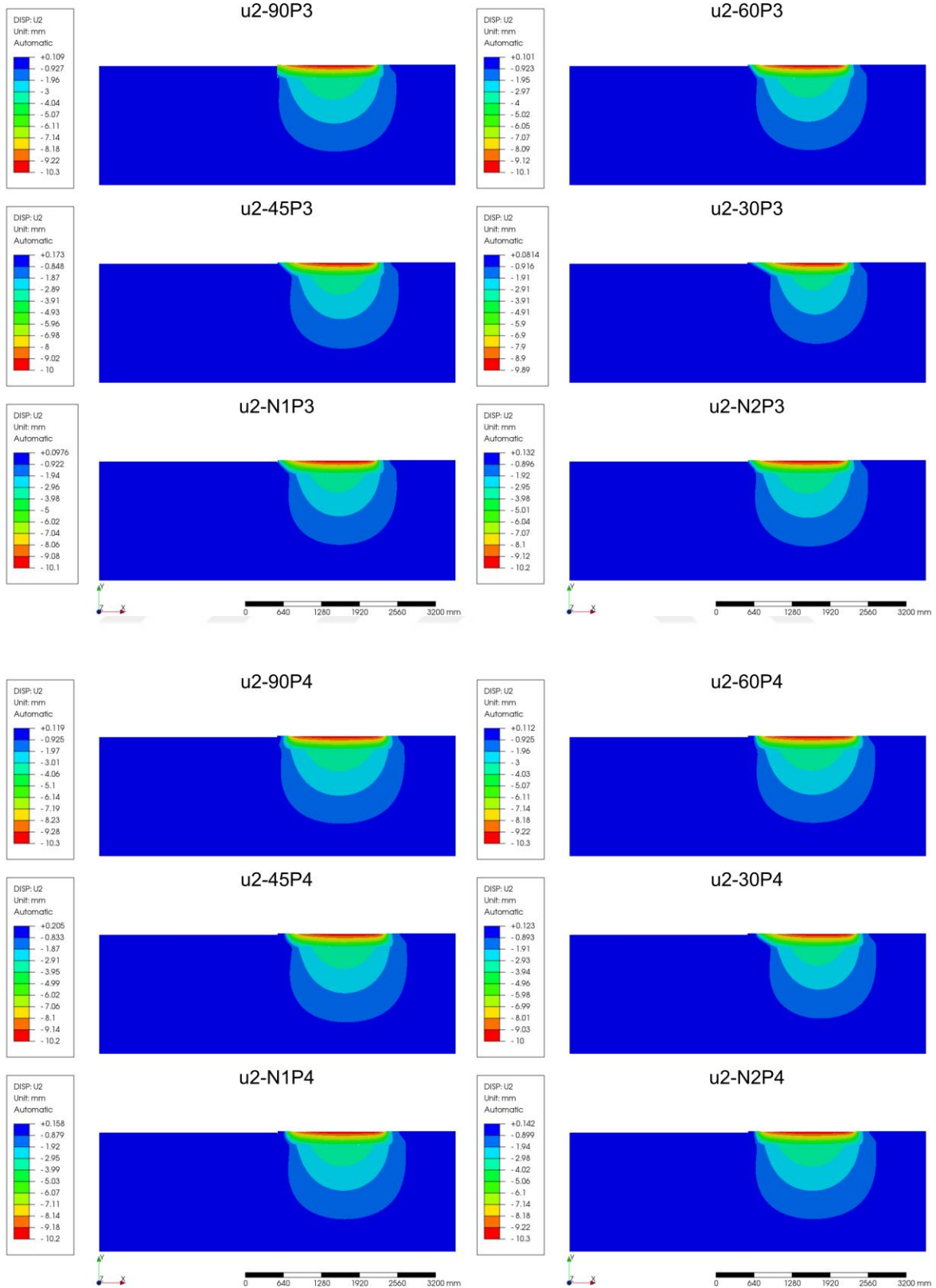
284     v = x(2);
285
286     % Calculate Estar which is crutial to the theory
287     myinverse = (1 - v^2) / E + (1 - v_roller^2) / E_roller;
288     Estar = 1 / myinverse;
289
290     % Calculate the displacement for the given E and v,
291     % they are from their arrays
292     % d_calculated = ((9 * F^2) / (16 * R * Estar^2))^(1/3);
293     d_calculated = (F / (pi * length_of_the_drum * Estar)) ...
294     * (1 + log((2 * pi * (length_of_the_drum^3 * Estar)/(F * 2 * R))));
295     % % % % % alternate relations for approximation
296     % % % % % aa = ((4 * R * F) / (pi * Estar));
297     % % % % % d_calculated = ((F / (pi * length_of_the_drum * Estar)) * (1.884 +
298     log(length_of_the_drum/((4 * R * F) / (pi * Estar))));
299     % My objective is to minimize the difference between calculated and target
300     displacement
301     % So we need an objective function to minimize, It could also be in a
302     % power form but this is just enough.
303     obj = abs(d_calculated - d_target);
304 end
305
306

```

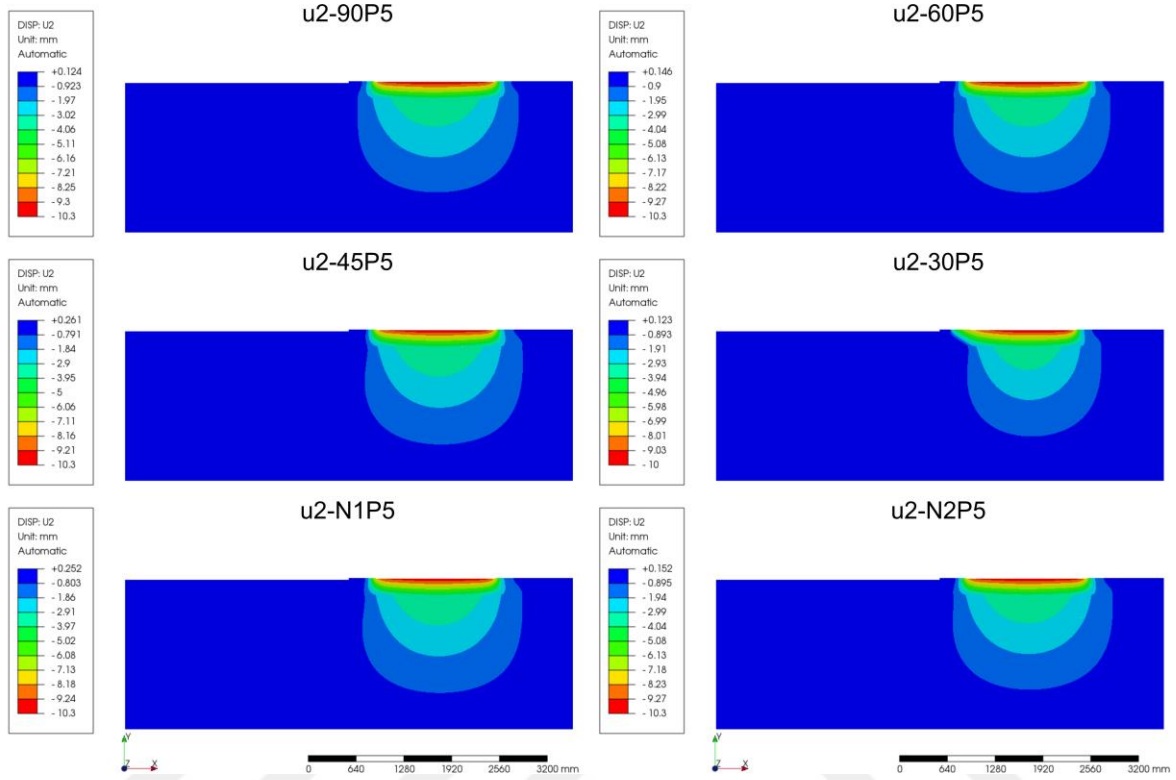
APPENDIX 2. FEA solutions of the pavement models showing vertical deformations (U2) with the roller drum in positions P1 and P2



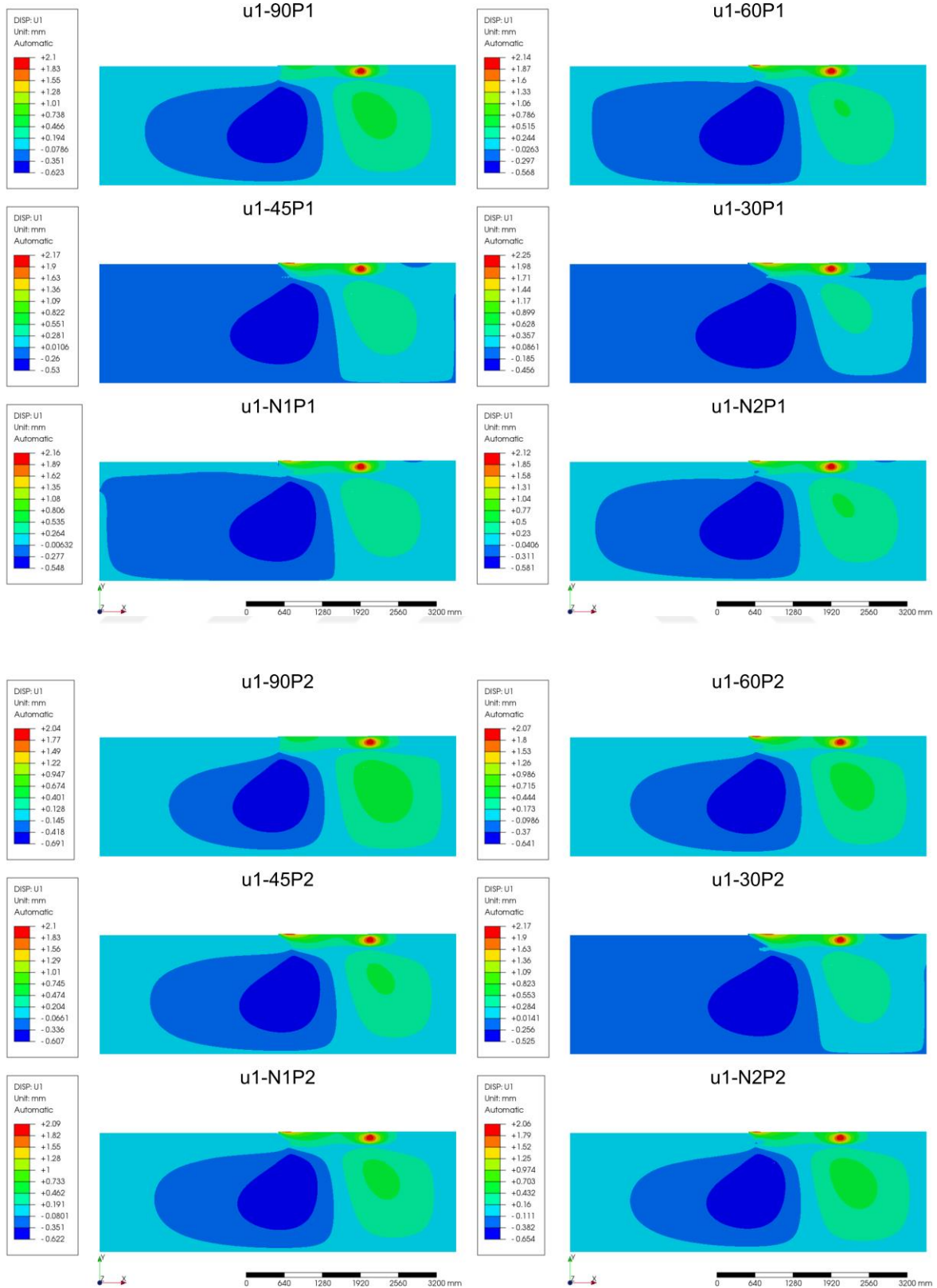
APPENDIX 3. FEA solutions of the pavement models showing vertical deformations (U2) with the roller drum in positions P3 and P4



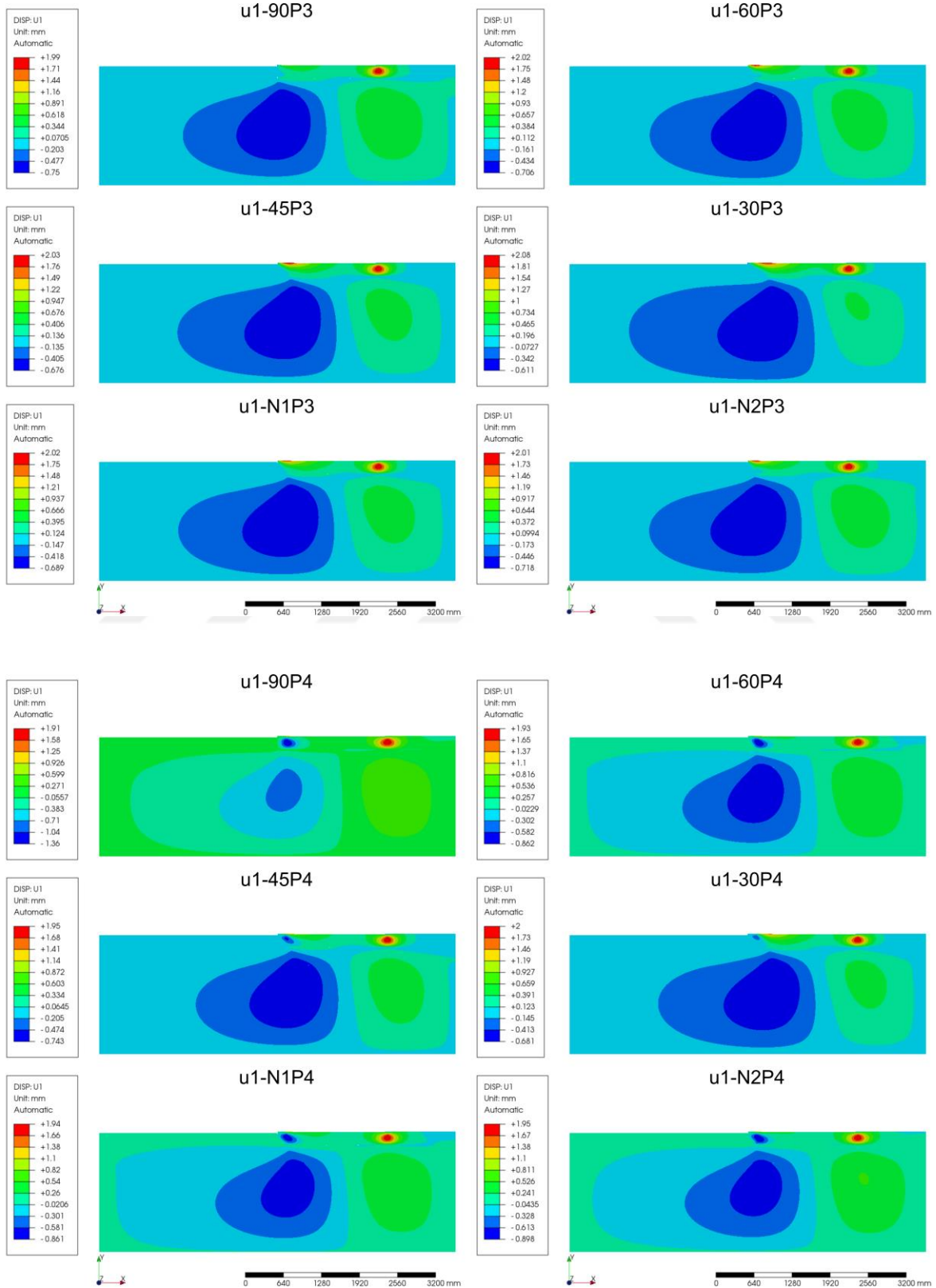
APPENDIX 4. FEA solutions of the pavement models showing vertical deformations (U2) with the roller drum in position P5



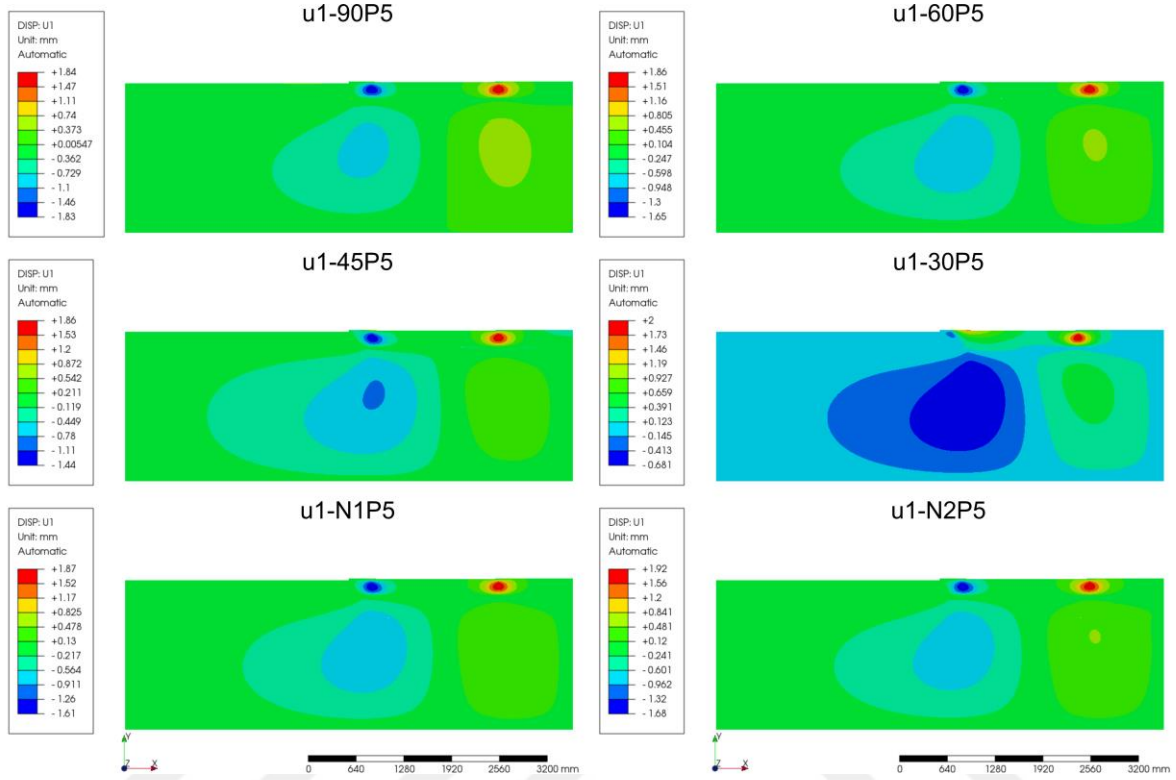
APPENDIX 5. FEA solutions of the pavement models showing horizontal deformations (U1) as the roller drum is in positions P1 and P2



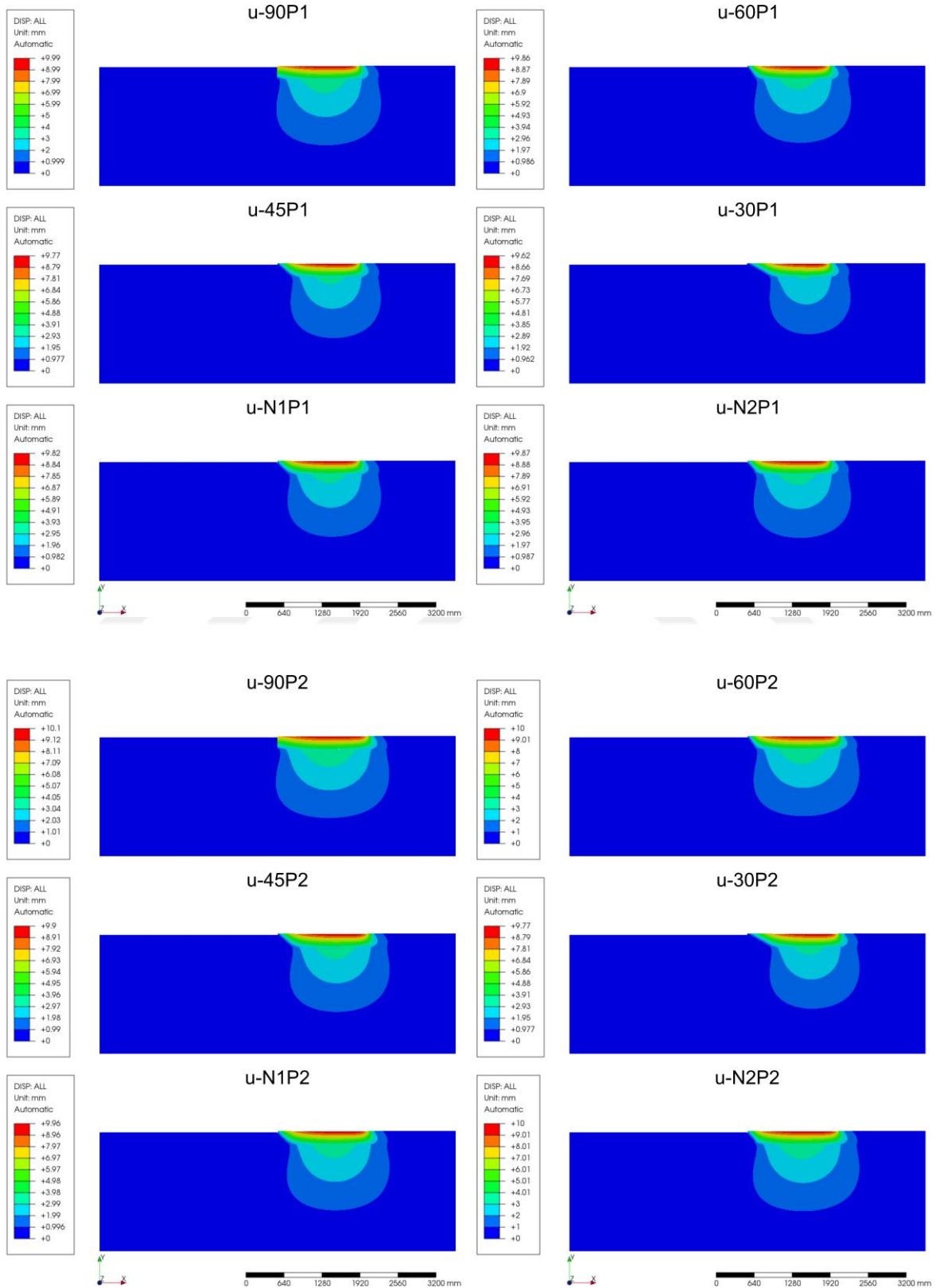
APPENDIX 6. FEA solutions of the pavement models showing horizontal deformations (U1) as the roller drum is in positions P3 and P4



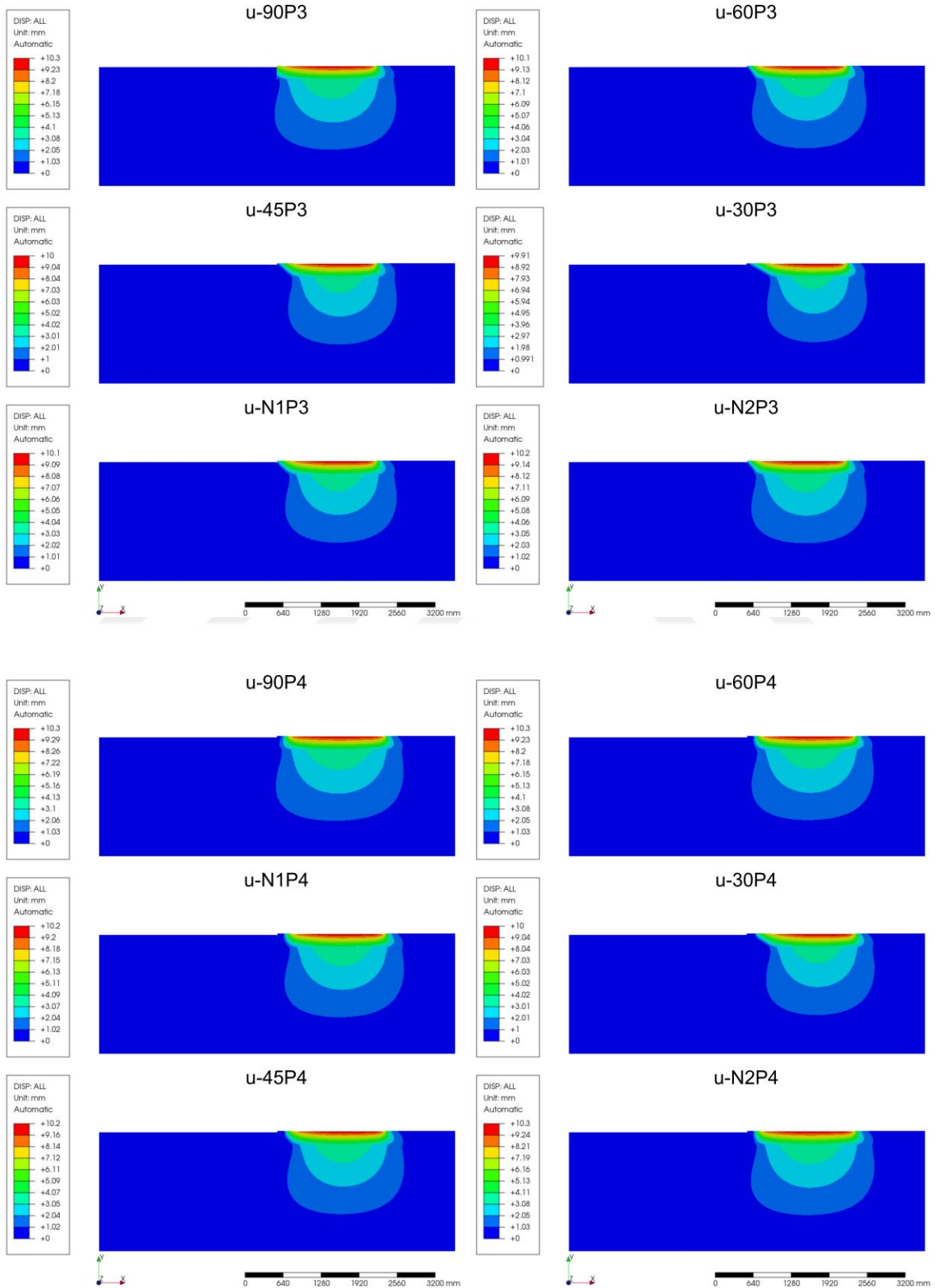
APPENDIX 7. FEA solutions of the pavement models showing horizontal deformations (U1) as the roller drum is in position P5



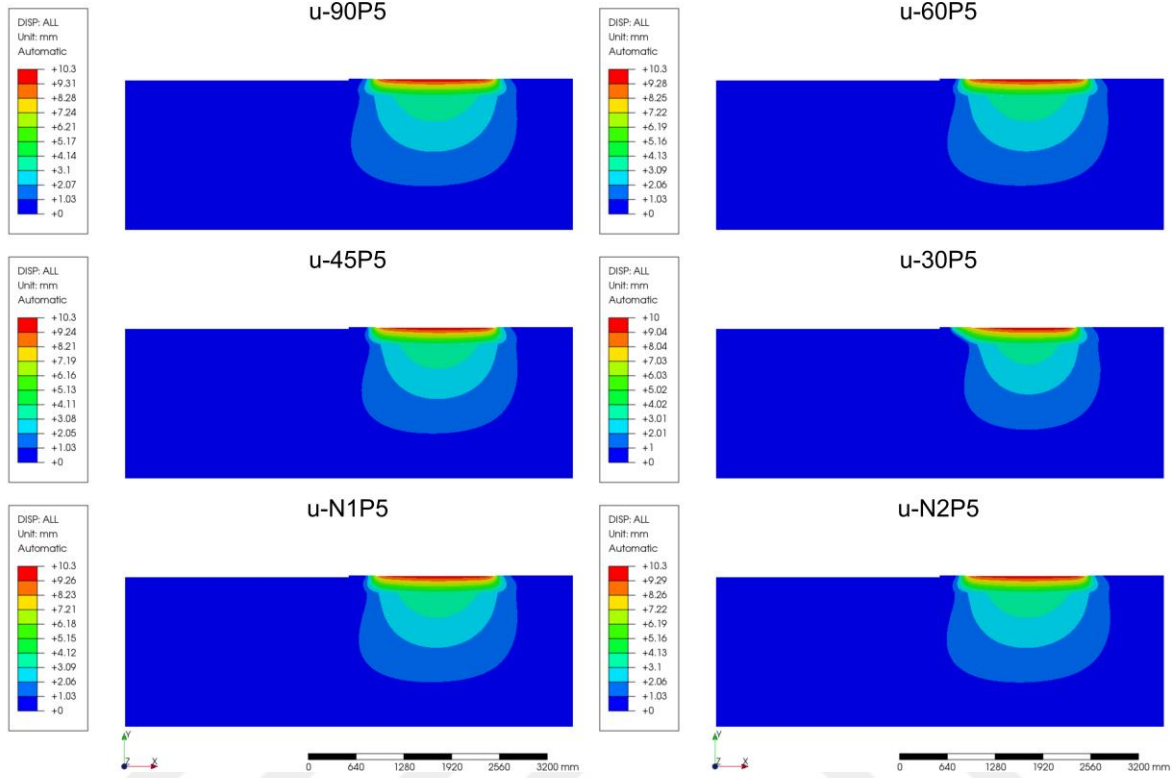
APPENDIX 8. FEA solutions of the pavement models showing resultant deformations (U) when the roller drum is in positions P1 and P2



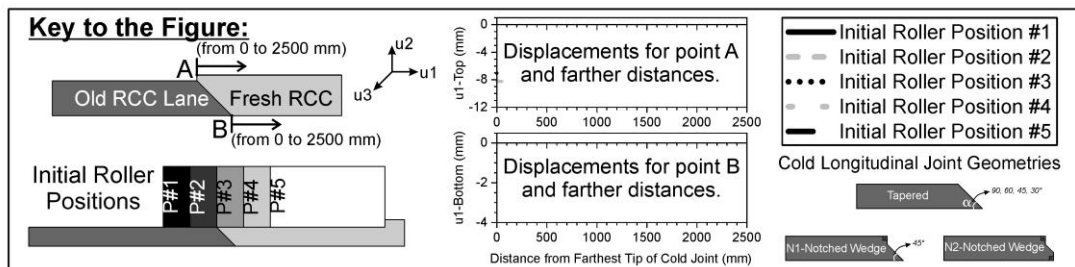
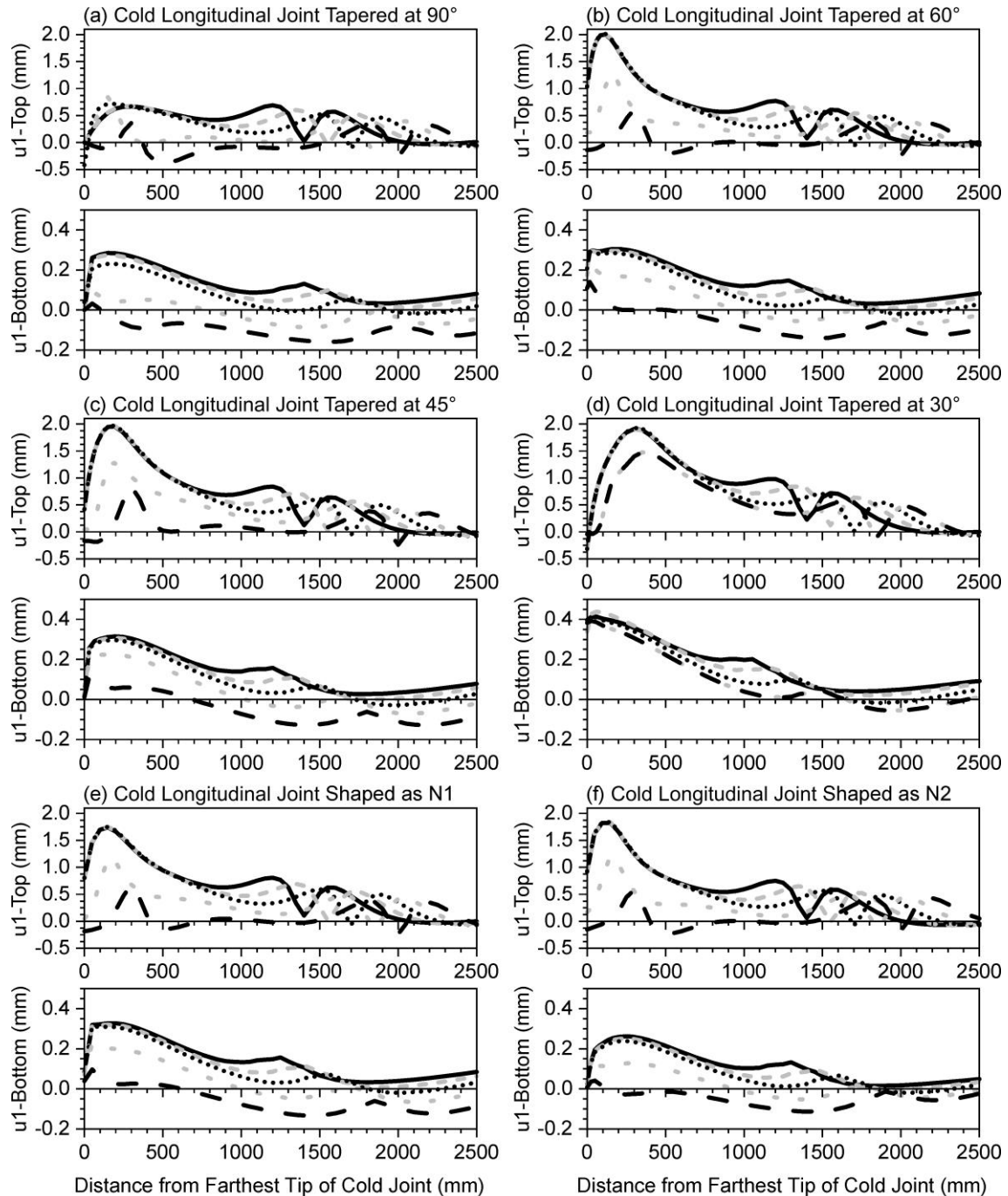
APPENDIX 9. FEA solutions of the pavement models showing resultant deformations (U) when the roller drum is in positions P3 and P4



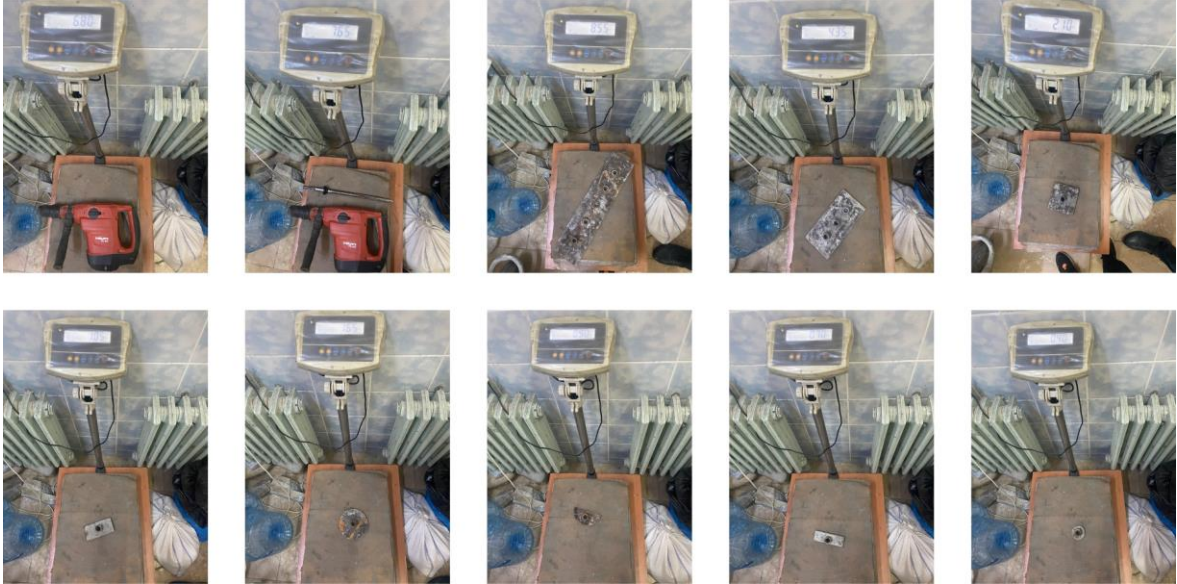
APPENDIX 10. FEA solutions of the pavement models showing resultant deformations (U) when the roller drum is in position P5



APPENDIX 11. Horizontal deformation variations at the top and bottom of the fresh RCC layer against the distance from the cold joint, determined by FEAs



APPENDIX 12. Determination of the masses of the transfer plates, vibrating hammer and bid employed in compacting the fresh-state RCC in molds



APPENDIX 13. Basic physical properties of plain and fiber-reinforced RCC, compressive strength test results and modulus of bending strength test data from no-jointed beam samples

Type	Plain RCC	Fiber RCC	Type	Plain RCC	Fiber RCC
Dry Den. (kN/m ³)	22.6618	22.8393	Porosity (%)	4.732	4.316
std	0.3181	0.8549	std	0.579	0.611
COV	0.0140	0.0374	COV	0.122	0.141
Sat. Den. (kN/m ³)	23.7883	23.8673	Water Ab. (%)	4.970	4.514
std	0.3889	0.7852	std	0.636	0.669
COV	0.0163	0.0329	COV	0.128	0.148
Nat. Den. (kN/m ³)	23.4175	23.6211	Saturation Coef.	0.597	0.567
std	0.3397	0.8119	std	0.024	0.015
COV	0.0145	0.0344	COV	0.039	0.027

Sample Name	Compressive Strength (MPa)		Compressive Strength -Average (MPa)		STD (MPa)		COV	
	for 7-day	for 28-day	for 7-day	for 28-day	for 7-day	for 28-day	for 7-day	for 28-day
Plain RCC-Cube-1	44.973	59.701	43.54	58.24	1.70	1.55	0.04	0.03
Plain RCC-Cube-2	41.664	56.605						
Plain RCC-Cube-3	43.981	58.407						
Fiber RCC-Cube-1	31.615	55.074	32.57	54.79	1.25	1.27	0.04	0.02
Fiber RCC-Cube-2	33.985	53.404						
Fiber RCC-Cube-3	32.101	55.906						
Plain RCC-Cylinder-1	34.504	45.104	34.03	44.70	1.52	1.05	0.04	0.02
Plain RCC-Cylinder-2	32.325	43.506						
Plain RCC-Cylinder-3	35.251	45.484						
Fiber RCC-Cylinder-1	31.535	41.627	31.84	41.58	1.22	1.35	0.04	0.03
Fiber RCC-Cylinder-2	30.807	40.204						
Fiber RCC-Cylinder-3	33.181	42.909						

Type of the Longitudinal Joint	Size of the Beams	Bending Strength (modulus of rupture)	Std Dev	COV	28-day Compressive strength	Empirical Estimate A-from Chhorn et al. (2018)	Empirical Estimate B-from Chhorn et al. (2018)	Empirical Estimate-from Tyabji and Okamoto (1987)	Empirical Estimate-from ACI 325 (ACI,1995)
Fiber RCC Beam	50x50x300-mm	8.73	0.703	0.081	41.58	6.47	5.60	6.42	4.52
Fiber RCC Beam	150x180x600-mm	7.18	1.003	0.140					
Plain RCC Beam	50x50x300-mm	7.71	0.499	0.065	44.7	6.76	5.75	6.66	4.68
Plain RCC Beam	150x180x600-mm	7.13	0.773	0.108					

APPENDIX 14. Average SilverSchmidt's rebound values (Q-value) for each beam sample, ultrasonic pulse velocity measurements in RCC beam samples with various joint configurations and joint interface treatments

Model	Average Q	Std	COV
P-REF	50.60	2.160	0.042688
F-REF	50.30	3.168	0.062982
PP90	49.86	2.016	0.040433
FF90	49.60	2.736	0.055161
PP60	50.92	3.484	0.068421
FF60	49.87	2.736	0.054863
PP45	49.74	2.448	0.049216
FF45	48.97	3.168	0.064693
PPN1	49.75	4.032	0.081045
FFN1	48.40	3.304	0.068264
PPN2	49.08	3.168	0.064548
FFN2	51.56	3.848	0.074631

Joint Type	Treatment	Direct Vp (m/S)	Indirect Vp#1 (m/S)	Indirect Vp#2 (m/S)	Direct Vs (m/S)	Joint Type	Treatment	Direct Vp (m/S)	Indirect Vp#1 (m/S)	Indirect Vp#2 (m/S)	Direct Vs (m/S)
P	Fresh-None	4,918	5,206	4,766	3,534	PP45	E	4,790	5,124	4,775	3,437
F	Fresh-None	4,559	4,724	4,627	3,283	FF45	0	4,891	5,118	4,766	3,338
PP90	0	5,017	5,235	4,516	2,457	FF45	W	4,965	5,123	4,751	3,390
PP90	W	4,942	4,733	5,323	2,447	FF45	S	4,849	5,109	4,762	3,412
PP90	S	4,843	4,437	5,127	3,240	FF45	E	4,841	5,110	4,751	3,354
PP90	E	4,866	5,172	4,714	3,421	PPN1	0	4,821	5,112	4,740	3,393
FF90	0	4,866	5,111	4,839	3,360	PPN1	W	4,863	5,097	4,753	3,420
FF90	W	4,849	5,127	4,730	3,291	PPN1	S	4,868	5,136	4,748	3,333
FF90	S	4,882	5,056	5,133	3,546	PPN1	E	4,890	5,111	4,741	3,448
FF90	E	4,983	5,246	4,696	3,585	FFN1	0	4,691	4,609	4,512	3,308
PP60	0	4,894	5,133	5,161	3,563	FFN1	W	4,824	5,123	4,778	3,465
PP60	W	4,908	5,123	4,751	3,369	FFN1	S	4,887	5,096	4,751	3,414
PP60	S	4,951	5,151	4,741	3,379	FFN1	E	4,842	5,092	4,795	3,394
PP60	E	4,944	5,114	4,770	3,359	PPN2	0	4,894	5,094	5,018	3,394
FF60	0	4,743	4,982	4,521	3,615	PPN2	W	4,858	5,109	4,742	3,318
FF60	W	4,899	5,073	4,771	3,376	PPN2	S	4,827	5,106	4,757	3,426
FF60	S	4,840	5,140	4,737	3,322	PPN2	E	4,863	5,132	4,775	3,314
FF60	E	4,922	5,133	4,762	3,429	FFN2	0	4,874	5,282	4,474	3,559
PP45	0	4,876	5,101	4,772	3,252	FFN2	W	5,114	5,121	4,795	3,497
PP45	W	4,971	5,115	4,753	3,403	FFN2	S	5,103	5,120	4,753	3,428
PP45	S	4,902	5,121	4,747	3,293	FFN2	E	4,956	5,086	4,733	3,411

APPENDIX 15. Split prism test results

Model	No	Failure Load (FL) (N)	Average FL	Bond Strength, ftb (MPa)	Average ftb (MPa)	std (MPa)	COV
P_Fr	1	91.838	92.39	4.082	4.11	0.045	0.010946
	2	91.780		4.079			
	3	93.560		4.158			
PPNO	1	24.725	20.34	1.099	0.90	0.194	0.21451
	2	16.000		0.711			
	3	20.289		0.902			
PPW	1	38.605	38.61	1.716	1.72	0.237	0.138013
	2	43.933		1.953			
	3	33.277		1.479			
PPS	1	40.81	39.34	1.814	1.75	0.188	0.107679
	2	42.644		1.895			
	3	34.564		1.536			
PPE	1	38.605	37.67	1.716	1.67	0.038	0.022622
	2	37.461		1.665			
	3	36.939		1.642			
F_Fr	1	101.722	95.83	4.521	4.26	0.242	0.056771
	2	91.001		4.044			
	3	94.755		4.211			
FFNO	1	10.704	16.05	0.476	0.71	0.235	0.3301
	2	16.140		0.717			
	3	21.297		0.947			
FFW	1	24.510	26.59	1.089	1.18	0.089	0.075084
	2	26.767		1.190			
	3	28.491		1.266			
FFS	1	38.8835	38.22	1.728	1.70	0.129	0.076122
	2	40.7345		1.810			
	3	35.032		1.557			
FFE	1	39.017	36.40	1.734	1.62	0.214	0.132096
	2	39.323		1.748			
	3	30.847		1.371			

APPENDIX 16. Bi-surface shear test results

Model	No	Failure Load (FL) (N)	Average FL (MPa)	Bond Strength, ftb (MPa)	Average ftb (MPa)	std (MPa)	COV
P_Fr	1	272.800	271.40	6.062	6.03	0.089	0.014718
	2	274.500		6.100			
	3	266.890		5.931			
PPNO	1	10.200	10.57	0.227	0.23	0.009	0.038247
	2	11.000		0.244			
	3	10.500		0.233			
PPW	1	21.800	22.77	0.484	0.51	0.025	0.04937
	2	24.000		0.533			
	3	22.500		0.500			
PPS	1	80.000	83.00	1.778	1.84	0.115	0.062604
	2	89.000		1.978			
	3	80.000		1.778			
PPE	1	118.900	168.23	2.642	3.74	0.963	0.257637
	2	200.200		4.449			
	3	185.600		4.124			
F_Fr	1	368.600	367.71	8.191	8.17	0.264	0.032348
	2	355.400		7.898			
	3	379.140		8.425			
FFNO	1	14.450	15.57	0.321	0.35	0.050	0.144497
	2	18.160		0.404			
	3	14.100		0.313			
FFW	1	20.000	19.87	0.444	0.44	0.003	0.007689
	2	19.700		0.438			
	3	19.900		0.442			
FFS	1	86.000	50.85	1.911	1.13	0.691	0.611195
	2	27.000		0.600			
	3	39.554		0.879			
FFE	1	122.400	136.59	2.720	3.04	0.327	0.107823
	2	151.800		3.373			
	3	135.556		3.012			

CURRICULUM VITAE

I, Murat NAS, am a research civil engineer currently working for the Karadeniz Technical University (KTU), Trabzon/Tukey. I earned my B.Sc. in civil engineering with an emphasis on Geotechnical Engineering and M.Sc. in Geotechnical Earthquake Engineering from the KTU with an emphasis on Earthquake-Induced Soil Liquefaction Hazard.

I have thus far worked as a research assistant, project engineer, and inspector. Further, as a practitioner, I have more than twenty years of experience in multi-component research projects and consulting businesses. My published articles and ongoing studies related to my fields of expertise can be viewed from internet resources or using my Web of Science Researcher ID (AAR-5473-2020) or TÜBİTAK ID (TBTk-0012-5958) or my ORCID ID (0000-0002-5726-4096).

I am married, with three children. I am a native speaker of Turkish and proficient in English (at the C1 level on the Common European Framework of Reference for Languages (CEFR) scale).