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**Structural performance and damage evaluation of corroded infill  
slabs from the Champlain Bridge after 60 years in service**

By

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Thesis submitted to the University of Ottawa in partial fulfillment of the  
requirements for the degree of

**Doctor of Philosophy in Civil Engineering**

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## **Author's Declaration**

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

I understand that my thesis may be made electronically available to the public.

## Abstract

The rapid deterioration of aging civil infrastructure represents a critical challenge for global transportation networks. While extensive research has focused on the performance of reinforced and prestressed concrete (PC) members subjected to laboratory-accelerated corrosion, there remains a fundamental disconnect between these controlled simulations and the complex, long-term degradation observed in field-aged structures. This doctoral research bridges this gap through a comprehensive multi-scale forensic and structural evaluation of roadway infill slabs extracted from the former Champlain Bridge in Montreal, Canada, after 57 years of aggressive environmental exposure. The research program integrates high-resolution non-destructive testing (NDT), forensic material characterization of 975 individual prestressing wires, and destructive testing of 15 full-scale slab panels under realistic boundary conditions. Material-scale analysis identifies a robust ductility transition threshold at approximately 3% mass loss, beyond which the ultimate strain capacity of parallel-wire tendons significantly deteriorates, independent of gauge length or prior loading history. At the structural scale, the research reveals a fundamental divergence in performance that is independent of average material degradation. Experimental results demonstrate a significant "capacity spread" at low corrosion levels. Specifically, specimens with nearly identical average mass-loss values exhibited ultimate load capacities varying by over 140% (160.1 kN to 384.3 kN). This disparity is identified as a consequence of localized interface conditions rather than distributed section loss. The forensic investigation confirms that grout integrity is a governing factor influencing the ultimate limit state. While solid grout coverage ensures ductile flexural behavior, crumbled or powdery grout leads to a more variable structural response. Such deterioration can trigger a premature transition to brittle shear failure. However, it can also facilitate bond-slip mechanisms that redistribute stresses and activate arching action. As demonstrated in the numerical investigation, this shift in the load-carrying mechanism can result in higher ultimate load capacities.

The experimental findings are further validated through nonlinear finite element (FE) modelling, which successfully isolates the "ductility gap" caused by bond-slip mechanisms. The results prove that simplified evaluation metrics relying on average section loss, which is the current industry standard for large-scale bridge inventories, can lead to unconservative safety estimates. This thesis contributes a performance-based framework for the assessment of aging PC infrastructure,

advocating for the prioritization of grout integrity and localized bond-penalty factors in national design codes. By integrating the assessment of volumetric steel loss with the governing physics of the tendon-concrete interface, this work provides bridge owners with a data-driven methodology to ensure the continued safety and reliability of aging transportation networks.



## List of Publications Based on this Research

### Journals:

- I. **Tawil D.\***, Martín-Pérez B., Sanchez LFM, Noël M. *Detection of corrosion effects on prestressed concrete bridge deck slabs from the Champlain bridge through non-destructive testing techniques*. Structural Concrete. 2024. Open Access: <https://doi.org/10.1002/suco.202400591>

### Conferences:

- I. **Tawil, D.\***, Martin-Peréz, B., F.M. Sanchez, L., and Noël, N. (2023). *Current Strengths and Limitations of Non-Destructive Techniques for the Corrosion Assessment of Post-Tensioned Concrete*. Capacity Assessment of Corroded Reinforced Concrete Structures: from Research to Daily Engineering Evaluation, CACRCS 2023, September 13-15, 2023, Parma, Italy.
- II. **Tawil, D.\***, Martin-Peréz, B., F.M. Sanchez, L., and Noël, N. (2023). *Detailed Visual Inspection of Grouted Post-Tensioning Tendons Embedded in Concrete Deck Slabs of a Decommissioned Bridge*. Capacity Assessment of Corroded Reinforced Concrete Structures: from Research to Daily Engineering Evaluation, CACRCS 2023, September 13-15, 2023, Parma, Italy.
- III. **Tawil, D.\***, Martin-Peréz, B., F.M. Sanchez, L.F.M., Noël, N. (2023). *Condition Evaluation of Steel Reinforcement Embedded in a Post-Tensioned Concrete Deck Slab From a Canadian Bridge*. 2023 Canadian Society for Civil Engineering Annual Conference, CSCE 2023, May 24-27, 2023, Moncton, Canada.
- IV. **Tawil, D.\***, Kristufek, L., Martin-Perez, B., Sanchez, L.F.M., Noël, M. (2022). Condition assessment of two prestressed concrete slabs after 60 years in service. 6th International Conference on Concrete Repair, Rehabilitation and Retrofitting (ICCRRR 2022), MATEC Web of Conferences 364, 03004 (2022).

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Beyond the publications listed above, additional manuscripts derived from Chapters 4, 5, and 6 of this thesis are currently in preparation and are scheduled for submission to leading structural engineering journals in the near future.

## Acknowledgements

I wish to express my deepest and most profound gratitude to my supervisor, Dr. Beatriz Martín-Pérez, for her unwavering guidance, patience, and mentorship throughout this journey. I am incredibly fortunate to have had her as a mentor. Her expertise, high standards, and academic rigor were fundamental to the completion of this research. Beyond her technical brilliance, I am profoundly grateful for her extraordinary kindness and the steadfast support she provided during the most challenging periods of my doctoral studies. Her belief in my work and her constant encouragement provided the strength I needed to see this work to completion.

My sincere gratitude also goes to Dr. Martin Noël for his invaluable insights and technical contributions, which significantly strengthened the quality of this work. I would also like to thank Dr. Leandro Sanchez for his thoughtful guidance and the expertise he shared throughout the development of this research. Their support was essential to the success of this work.

I would like to extend a special thanks to Dr. Gamal Elnabelsya and Dr. Muslim Majeed for their exceptional technical assistance. Their willingness to go above and beyond in facilitating the experimental set-up and addressing technical challenges in the lab was pivotal. To my colleagues in the Champlain Bridge research team, Issa Fowai and Leah Kristufek: thank you for the shared journey and the collaborative environment we built together. My sincere gratitude goes to anyone I may have inadvertently omitted, but whose support was nonetheless vital to the completion of this work.

I am also thankful to the Department of Civil Engineering at the University of Ottawa for fostering such a supportive and enriching research environment.

This research was made possible through the generous financial assistance of the American Concrete Institute (ACI) Foundation. I am deeply grateful to the entire ACI team at the headquarters in Farmington Hills, Michigan, for their support and for the distinct honor of being named an ACI Foundation Fellow. Additionally, I am thankful for the professional growth and industry perspective I gained during my internship at the ACI headquarters. I would also like to extend my sincere thanks to the Natural Sciences and Engineering Research Council of Canada (NSERC) for the support provided through the Postgraduate Scholarships-Doctoral (PGSD) program. The funding provided by NSERC was essential in allowing me to focus entirely on the

scientific rigor and development of this research. Furthermore, I wish to express my sincere gratitude to Jacques Cartier and Champlain Bridges Inc. (JCCBI) for their essential cooperation throughout the forensic investigation. This research would not have been possible without their support and the unique opportunity to evaluate these field-aged structural specimens following their 57 years of service.

I would like to extend my sincere gratitude to Samer Jabbour, Ian Fuller, and Greg Woltman for the opportunity to join the WSP team during my doctoral studies. Their flexibility and professional support were instrumental in allowing me to balance my responsibilities at the firm with the completion of this dissertation. I would also like to thank my colleague, Adel Bugaldian, for his continued encouragement and professional support throughout our time working together at WSP.

A very special and heartfelt thank you goes to Yacine Gueye Correa for her exceptional assistance, tireless dedication and hard work in the laboratory. Beyond her technical contributions, I am profoundly grateful for her genuine friendship and the encouragement she provided as I navigated the challenges of my doctoral journey. Yacine, your presence and support, both within the lab and beyond, have been truly remarkable, and I am forever grateful for your kindness.

Furthermore, nothing would have been possible without the immense support of my family and my parents, who made profound sacrifices to provide me with the opportunities I have today. My heartfelt gratitude goes to my mother, Silva, and my brothers, Michel, Elie and Dany: you have always been my pillars of strength and courage. Your support allowed me to focus entirely on my research and encouraged me to push through regardless of the challenges I faced. I also wish to honor the memory of my father, Fouad. His unwavering belief and lifetime of sacrifice were a constant source of strength, and I am deeply grateful for the support he provided during the years he was by my side.

I am incredibly grateful to my now husband and soulmate, Eddy Haddad: thank you for being my rock, my anchor, and my "iron dome". Your constant encouragement kept me pushing forward, and I am deeply grateful for your endless patience, love, and understanding. I am especially thankful for the way you consistently challenged my perspective and helped me refine and organize my thoughts throughout the most complex stages of this work.

## Dedication

إلى أبي الحبيب... الحاضر في قلبي

إلى مَنْ رحَلَ وَبَقِيَ في القلب

إلى فؤادٍ سكنَ الفؤاد

إلى روح والدي الغالي

إلى فؤاد... يا فؤادي

To my beloved father... always in my heart

To the one who has passed, yet remains within

To Fouad... who lives in my heart

To the memory of my father

To Fouad... my heart

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## Chapter 1: Introduction

### 1.1 General

The integrity and resilience of civil infrastructure systems are of increasing concern to governments, engineers, and researchers worldwide. Transportation infrastructure, particularly bridges, plays a critical role in facilitating economic activity, social connectivity, and public safety. However, bridges are continuously subjected to environmental stressors, increased traffic demand, and material aging. These factors contribute to the progressive degradation of structural components and significantly reduce service life. As infrastructure continues to age, many jurisdictions are now facing mounting maintenance needs and growing funding gaps, collectively referred to as “the infrastructure deficit” (ASCE Infrastructure Report Card, 2025; World Economic Forum, 2016).

Numerous international reports indicate that a substantial portion of global infrastructure has either reached or exceeded its intended design life. The implications of this are far-reaching. In addition to direct repair and replacement costs, deterioration can lead to severe economic, environmental, and safety consequences, including transportation delays, increased emissions, public safety risks, and in worst cases, catastrophic failure. For major economies, the direct and indirect financial burden of inadequate infrastructure maintenance is estimated in the billions of dollars annually (OECD, 2017). In Canada, these concerns are mirrored at the national level. Between 2003 and 2013, the country doubled its investment in public infrastructure, rising from \$14.5 billion to \$29.5 billion, and by 2017, this figure had reached \$56.6 billion, with over 40% allocated to transportation infrastructure such as highways, roads, and bridges (Infrastructure Canada, 2018; Statistics Canada, 2022). Despite these increases, national assessments consistently highlight that further investment is needed to address the country’s infrastructure deficit and avoid long-term economic consequences. Several factors continue to widen this gap. These include the effects of harsh winter climates, increased exposure to freeze–thaw cycles, widespread use of de-icing salts, climate change impacts, growing traffic volumes, and legacy underinvestment in maintenance and rehabilitation (Fenn et al., 2019). In 2019, 12.4% of Canadian bridges were classified as being in poor to very poor condition, a substantial increase from 4% in 2016 (Infrastructure Canada, 2016, 2019). This alarming trend points to a deteriorating bridge stock and the urgent need for condition assessment and life extension strategies.

A notable example illustrating this challenge is the former Champlain Bridge in Montreal, Quebec (Figure 1-1). Constructed in 1962, the bridge served as a major crossing over the St. Lawrence River, connecting the Island of Montreal to the South Shore. Over its six decades of service, the bridge experienced severe and accelerated deterioration due to sustained environmental exposure and increased traffic loading. Extensive cracking, corrosion of reinforcement and prestressing elements, and concrete delamination prompted significant federal interventions. Despite repeated repairs and substantial investments in emergency maintenance, the structure was ultimately decommissioned and replaced in 2019.



Figure 1-1 Former Champlain Bridge (reproduced from Jada, 2020)

### 1.1.1 Champlain Bridge (1962 – 2019)

The Champlain Bridge was constructed between 1958 and 1962 and remained in operation until 2019. It was labelled as one of the busiest bridges in Canada, allowing more than 50 million annual vehicle crossings and \$20 billion annually in Canada – U.S trade (Carlin, 2017; COWI, 2017). The Champlain Bridge was an iconic steel truss cantilever bridge with two concrete approach spans that connected the city of Montreal to the South shore crossing the Saint Lawrence River, as shown in Figure 1-2. The bridge accommodated three traffic lanes in each direction and had a total length of 3.44 km that spanned over three sections:

- Section 5: spanned a total of 2.15 km and connected Nun's Island, located in the Saint Lawrence River, to the Seaway.
- Section 6: spanned a total of 763 m and crossed over a portion of the Seaway.
- Section 7: spanned a total of 528 m and connected the Seaway to the city of Brossard.



Figure 1-2 Layout of the former Champlain Bridge – Section 5, 6 and 7 (reproduced from COWI, 2017)

Section 5 of the bridge consisted of 40 concrete spans and a total roadway deck width ranging between 23.16 m to 26.52 m (JCCBI, 2014). Each span functioned as a simply-supported system whose cross-section consisted of seven concrete girders separated by concrete infill slabs and diaphragms. A typical cross-section of the Section 5 concrete deck system is shown in Figure 1-3.

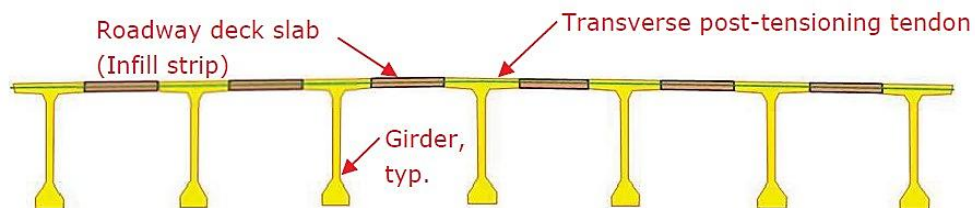


Figure 1-3 Typical cross-section of Section 5 concrete deck (reproduced from COWI, 2017)

The superstructure's novel design within Section 5, which included prestressed concrete girders connected to transversely post-tensioned cast-in-place infill slabs, made it challenging to replace deteriorated members or effectively monitor their structural health. To elaborate, the top flanges of the girders and the cast-in-place infill slabs constituted the deck and were interconnected by prestressing cables. The system also included a final asphalt riding surface. The resulting deck structure was a highly integrated design in both the transverse and longitudinal directions. It allowed the monolithic behaviour of the girders, intermediate slabs, and diaphragms, and was the most cost-efficient option among 28 alternatives considered for the original design (Delcan, 2011). Such design details and material characteristics of the roadway deck structure, along with poor drainage, contributed to the continuous and extensive deterioration experienced by the

superstructure components. As a result, rehabilitation measures were designed, at best, to mitigate the associated risks (Buckland & Taylor, 2013; COWI, 2017).

However, although multiple maintenance and restoration projects were completed throughout its lifespan, the severe level of deterioration continuously reduced the structure's functional service life. The Champlain Bridge was eventually closed and demolished after 57 years of service, as uncertainties in the nature of the deterioration remained. Although regular bridge inspections identified corrosion as the most prominent issue due to the heavy use of winter de-icing salts, other deterioration mechanisms may also have occurred at the microscopic scale, such as frost damage caused by freeze-thaw cycles, alkali-aggregate reaction, and carbonation-induced corrosion.

The Champlain Bridge was built during a crucial time in the evolution of bridge construction techniques and winter road maintenance practices. Winter runoff, which contains chlorides from de-icing salts, was not considered in the design of the bridge components. Moreover, the use of air entrainers to prevent frost damage from freezing and thawing cycles was still being developed at the time the bridge was constructed. This made concrete that was utilized during this time more susceptible to freeze-thaw cycles. In addition, the use of grouted prestressing tendons as concrete reinforcement was a new technology in the late 1950's. As a result, many concrete structures built between the 1960's and 1980's, similar to the Champlain Bridge, are showing major signs of distress at a quicker rate than expected. This is the case since distress mechanisms are generally associated with a reduction in the mechanical and durability properties of the affected concrete material, which may also bring about a decrease in the structure's functionality (Noël et al., 2018). In such cases, the structure is no longer capable of resisting the applied loads or even performing within its intended elastic range.

#### **1.1.1.1 Infill roadway deck slabs**

As part of the highly integrated superstructure, the infill slabs played a major role in the bridge's load-transfer system. The 200-mm deep deck slabs, topped with a 50-mm deep asphalt layer, were reinforced with grouted prestressing tendons as the primary reinforcement in the transverse direction, and regular steel reinforcement as the temperature reinforcement in the longitudinal direction. Each of the grouted post-tensioning tendons was composed of twelve 7-mm diameter strands located at an average spacing of 1,000 mm. The slabs functioned as a one-way system, spanning in the transverse direction a distance of 1.97 m between the girders (Buckland & Taylor,

2013), as illustrated in Figure 1-3. Figure 1-4 and Figure 1-5 show top and elevated views of 2 m x 3 m infill slab segments extracted from Section 5 of the Champlain Bridge. It should be noted that this study did not explicitly account for the prestress effects, as the tendons were cut and the slabs were extracted from the structure. In the actual bridge, a well-anchored tendon may rely less on bond development to maintain its intended behaviour. However, evaluating the specific loss of prestress due to both the extraction process and anchorage behaviour fell outside the scope of this study.



Figure 1-4 Infill slab extracted from Champlain Bridge



Figure 1-5 Infill slabs (stacked) extracted from Champlain Bridge

The continuous use of de-icing salts over the lifespan of the bridge, combined with the failure to incorporate appropriate mitigation measures into the original design, such as proper drainage



systems, contributed to the corrosion of the slabs' post-tensioned prestressing tendons (COWI, 2017; DESSAU, 2013, 2014). The close proximity of the infill slabs to traffic may have increased their exposure to carbon monoxide, leading to concrete carbonation and a gradual decrease in the pH level in the concrete pore solution. The combined effects of their exposure to carbonation and chloride ions from de-icing salt runoff may have accelerated the rate of corrosion of the tendons. In addition, the connection points between the infill slabs and the top flanges of the girders are highly dependent on the transverse prestressing tendons (Carlin et al., 1996). Hence, the possibility of water penetration introduces a vulnerability to the system.

### 1.1.1.2 Location of infill slabs in the structure

Eight concrete slabs, 2 m x 3 m in size, were received by the University of Ottawa in late 2021. The slabs were originally located between gridlines 44W and 43W of the Champlain Bridge, as highlighted in red in Figure 1-6.

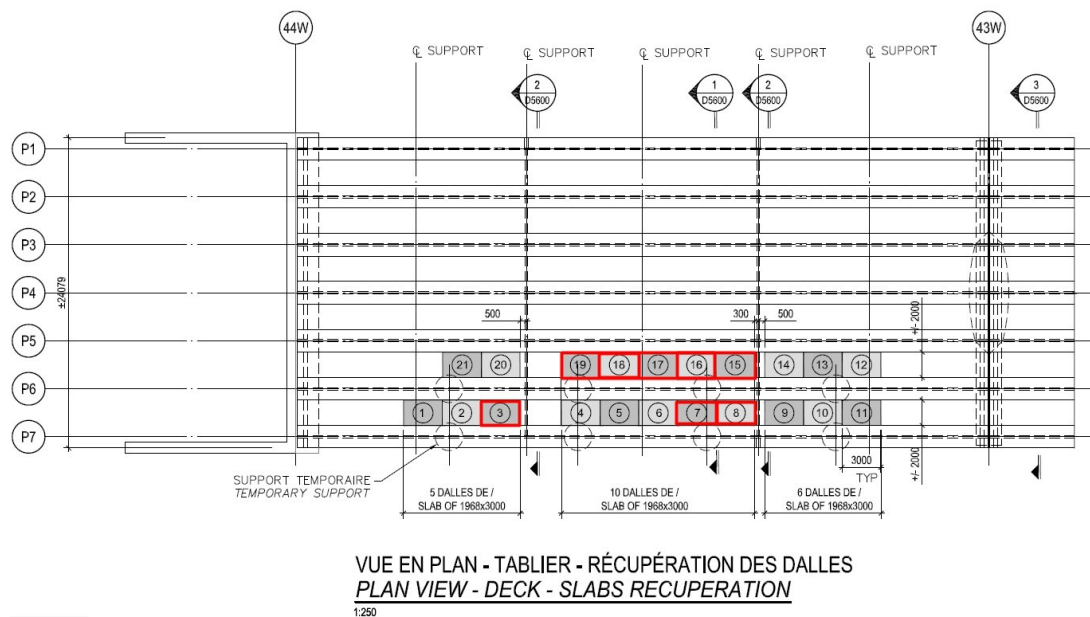


Figure 1-6 Location of infill slabs between piles 44W and 43W in the Champlain Bridge (reproduced from Mousseau, 2020)

Upon arrival at the University of Ottawa Structures Laboratory, the slabs were re-numbered 1 through 8. The new numbering system established by the University of Ottawa research team is shown in Table 1-1.



Table 1-1 Infill slabs current numbering system

| Original designation | Designation on plan drawing | New number as per University of Ottawa team |
|----------------------|-----------------------------|---------------------------------------------|
| D9-47.6              | 17                          | 1                                           |
| D9-47.2              | 7                           | 2                                           |
| D9-47.5              | 18                          | 3                                           |
| D9-47.1              | 3                           | 4                                           |
| D9-47.3              | 8                           | 5                                           |
| D9-47.4              | 19                          | 6                                           |
| D9-47.7              | 16                          | 7                                           |
| D9-47.8              | 15                          | 8                                           |

### 1.1.1.3 History of slabs

Following an inspection of the roadway deck slab completed between 2011 and 2013 of Section 5 of the Champlain Bridge, slab strengthening measures were recommended (AECOM, 2014). This included replacing concrete that had spalled off, and for cases of partial or total failure of prestressing strands, the work included the addition of external post-tensioning cables, or the addition of permanent structural steel slab supports. However, as per the structural drawings prepared by AECOM in 2011, the infill slabs allocated to the University of Ottawa did not undergo strengthening interventions (AECOM, 2014).

Design codes, including CSA A23-1942, the previous AASHO (now AASHTO) 1944, and ACI-ASCE “*Tentative Recommendations for Prestressed Concrete*”, which the Champlain Bridge was designed with, did not consider durability as a design parameter. However, the current CSA S6 standard, Canadian Highway Bridge Design Code (CSA Group, 2019a), incorporates the design for durability and entails a minimum service life of 75 years (The Jacques Cartier and Champlain Bridges Incorporated, 2019). As a result, the absence of a drainage system, combined with the extensive use of de-icing salts, allowed the salt-laden water to penetrate the infill slabs (The Jacques Cartier and Champlain Bridges Incorporated, 2019). In addition, as the bridge deck slopes towards the edge girders, it is expected that infill slabs located along the edge girders exhibit the most damage. A drainage system was eventually installed thirty years after the commissioning of the bridge.

### 1.1.1.4 In-situ condition evaluation

In 2008, an inspection of the infill slabs performed by AECOM determined that the roadway deck slab of Section 5 was suffering from significant signs of deterioration (AECOM, 2014). Several

infill slabs were experiencing fracture of their transverse prestressing tendons. Observations also included cracking along the soffit of several infill slabs, as well as traces of efflorescence. The infill slabs located along the edges of the span experienced concrete bursting to a depth of 70 mm. Signs of corroded reinforcement were visible at these locations. Figure 1-7 highlights the observed signs of deterioration along the deck slab soffit. It is evident that the transversely prestressed tendons had experienced corrosion, and the surrounding concrete had spalled off. This affected the transverse integrity of the deck slab, at the local level, as these tendons were crucial in ensuring proper load transfer within the deck system (AECOM, 2014). The resistance of the slab in the vicinity of this local failure was still provided through the bond between the tendon and grout (Buckland & Taylor, 2013).

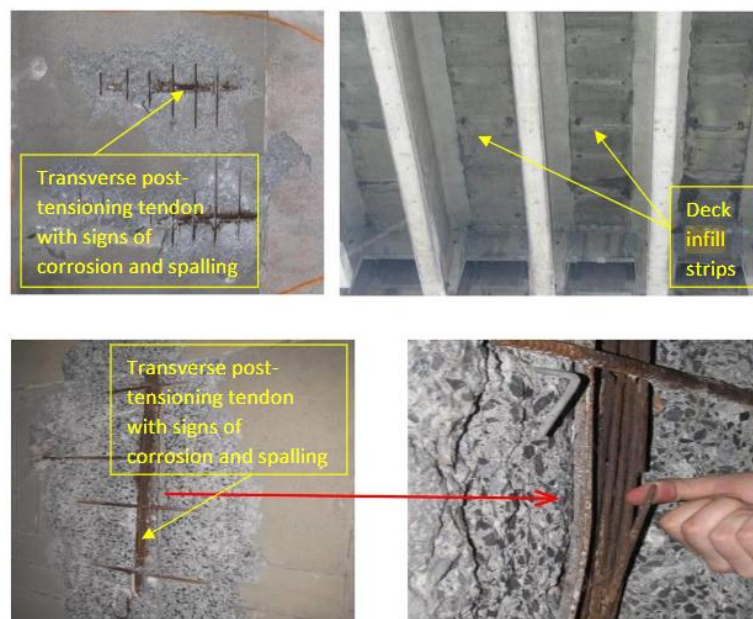


Figure 1-7 Observed signs of deterioration along the soffit of the deck slab (reproduced from COWI, 2017)

The results obtained from the electromagnetic inspection of the Section 5 infill slabs performed by Vector Corrosion Company Technologies in 2009 were analyzed by AECOM and compiled with their findings (AECOM, 2014). At the time, the results of the electromagnetic inspection showed that there existed three locations within span 44W-43W (Figure 1-6) that were characterized with a medium risk for tendon fracture. Subsequently, AECOM performed a study to establish a correlation between the various types of defects observed along the underside of the slab and the

fracture or loss of cross-sectional area of the transverse prestressing tendons. The intent of this study was to better understand the extent of damage of the prestressing cables by viewing their condition beyond the apparent concrete damage. Three types of defects were established: 1) concrete bursting with exposed wires; 2) concrete delamination without cracking; and 3) transverse cracking parallel to the prestressing cable without concrete delamination (AECOM, 2014). The results of this study are summarized in Table 1-2. It was found that a total of 3 locations within span 44W-43W of Section 5 experienced the various types of damages listed above.

Table 1-2 Type and number of prestressing cable defects within span 44W-43W

| Span           | Type of defect                           |                                                    |                                                                                      |                |
|----------------|------------------------------------------|----------------------------------------------------|--------------------------------------------------------------------------------------|----------------|
|                | 1 – concrete bursting with exposed wires | 2 – concrete delamination without contour cracking | 3 – transverse cracking parallel to prestressing cable without concrete delamination | Total per span |
| <b>44W-43W</b> | 1                                        | 1                                                  | 1                                                                                    | 3              |

Subsequently, an analysis was undertaken to determine the fracture or loss in cross-sectional area of the prestressing cables under the different types of damage observed. The results showed that with a Type 1 damage, the percentage of fracture of the prestressing tendons was 100% (i.e., all wires were fractured), while that under Type 2 damage was 50%. There was no fracture of tendons in slabs that experienced a Type 3 damage. Among all the spans in Section 5 of the Champlain Bridge, span 44W-43W was prioritized as the first span to undergo slab reinforcement and repair (AECOM, 2014).

Following this examination and study, AECOM completed a second inspection between 2011-2013, after which slab strengthening methods were recommended and implemented. However, further studies on the level of deterioration of the slab projected an increase in the number of localized deck repairs required per year. In case a serious problem was not addressed by the inspection team, the associated risk would have been a local deck failure (COWI, 2017).

## 1.2 Research gap

Although the Canadian Highway Bridge Design Code CSA S6:19 (CSA Group, 2019a) outlines general procedures for evaluating the ultimate and serviceability limit states of deteriorated reinforced concrete elements, it does not provide a quantitative framework for assessing the structural performance of ageing prestressed concrete infrastructure affected by deleterious

mechanisms such as reinforcement corrosion. Establishing robust correlations between reductions in mechanical properties, physical integrity, and durability, and their implications for member-level structural performance, is essential for predicting future deterioration and selecting appropriate management or rehabilitation strategies.

While considerable progress in this field has been achieved throughout the years, available research mainly focuses on determining the material deterioration and structural implications of reinforced or prestressed concrete beams or slabs subjected to accelerated corrosion by conducting laboratory experimental testing (Azad et al., 2007; Cousin & Martín-Pérez, 2010; Diao et al., 2012; Lu et al., 2016; Rinaldi et al., 2010; Zhang, Wang, Zhang, Ma, et al., 2017). However, laboratory test samples subjected to accelerated corrosion techniques do not typically sustain loads simultaneously, while field members undergo a variety of stress-state and confinement conditions while being exposed to fluctuating and aggressive environmental conditions for several years. Although findings from accelerated laboratory tests provide acceptable performance evaluations for specimens cured and tested in the laboratory, the results and conclusions extracted from these tests would not necessarily be directly applicable to the evaluation of ageing concrete members present in the field.

Currently, there is a notable gap in correlating material-scale deterioration observed in field-retrieved elements, including corrosion-induced section loss, concrete and prestressing strand damage, and concrete cracking, with their overall full-scale structural performance. This gap is particularly evident for prestressed concrete bridge deck slabs subjected to long-term chloride exposure and corrosion. Research on the structural behaviour of such slabs, especially those removed from service after decades of real environmental exposure, remains scarce. Given that reinforcement corrosion is a global challenge with significant economic implications, improving our understanding of the in-situ performance of corroded prestressed concrete slabs is critical.

By bridging this critical knowledge gap, this study aims to contribute meaningfully to the development of advanced assessment methodologies and predictive tools that support more sustainable, effective, and cost-efficient management strategies for aging bridge inventories. The ultimate goal is to safeguard public safety, extend the useful service life of critical infrastructure, and inform future policy and engineering practices. This study directly addresses this gap by examining full-scale deck slabs recovered from the former Champlain Bridge in Montreal after

approximately 60 years of service, linking their material condition to their structural capacity and identifying deterioration trends relevant to service-life prediction and asset management.

### **1.3 Research objectives**

This study forms part of a broader evaluation of the former Champlain Bridge, encompassing the diagnosis and prognosis of various extracted structural elements, and the development of management protocols for bridges in similar climates. The objectives of the present study are the following:

- Evaluate the effectiveness of existing non-destructive techniques (NDT) used for assessing the overall structural integrity of grouted post-tensioned flexural concrete members. The aim is to identify the most effective technique(s) by evaluating the strengths and limitations of current methods, and by comparing the results of each test with data from structural load testing field-retrieved members. This will also provide valuable insights into potential areas for further research and improvement.
- As there are currently no effective techniques available to evaluate grouted prestressing tendons within a member, recommendations for addressing these tendons will be developed.
- Establish correlations between the reduction in the mechanical properties of the prestressing tendons due to corrosion and both the extent and type of corrosion damage, as determined through gravimetric analysis. These correlations will be used to assess the structural implications of corrosion over time.
- Establish a correlation between the mechanical properties of extracted grouted post-tensioning tendons and the extent and degree of corrosion damage (gravimetric analysis) with the structural performance of the corresponding flexural field members, particularly in terms of load-carrying capacity and deflection.
- Evaluate bond quality of prestressing tendons within the structural field elements using numerical modelling techniques, with particular attention to its influence on observed failure modes and residual capacities. This technique was employed to investigate how variations in bond quality affect member behaviour, performance and load resistance. The models were calibrated and validated using the experimental data obtained from full-scale testing.

## 1.4 Scope of work

The scope of work for this study includes experimental and analytical components. Figure 1-8 presents an illustration of the different components of the research scope. The experimental program is divided into two phases and includes testing of eight concrete roadway deck slabs. It involves the assessment of the structural performance and integrity of the slabs and includes the following aspects:

- **Phase 1:** A detailed visual inspection combined with advanced NDTs was employed on all slabs to evaluate the extent of deterioration. The visual inspection helps in qualitatively identifying areas of concrete spalling, delamination, and cracking. Innovative NDT techniques, including rebound hammer (RH) testing, ultrasonic pulse velocity (UPV) testing, ground penetrating radar (GPR), ultrasonic pulse echo (UPE) testing, wireless impact echo (IE), novel corrosion monitoring devices (half-cell potential and corrosion rate measurements), and surface resistivity measurements were used to examine sub-surface defects and reinforcement corrosion activity. The RH test provides rapid quantitative measurements of concrete hardness, while the UPV method is a widely used rapid damage detection technique that produces quantitative readings to assess concrete uniformity and identify internal damage. GPR utilizes radio waves to profile prestressing tendons and detect areas of complete duct or tendon rupture. The UPE method and the novel wireless IE method, the most advanced and integrated wireless approach, detect discontinuities in grouted tendon ducts. These rapid testing techniques generate contour maps of sub-surface defects. Finally, novel corrosion monitoring devices, equipped with non-invasive and wireless technology, provide fast and accurate real-time data to detect and evaluate reinforcement corrosion within seconds. Specifically, half-cell potential measurements indicate the likelihood of active corrosion in reinforcement by assessing the electrical potential of steel embedded in concrete; corrosion rate measurements quantify the instantaneous rate at which steel is corroding; and surface resistivity measurements provide insight into the concrete's permeability and ability to resist ion penetration, which are key factors influencing corrosion risk.
- **Phase 2:** Material testing was conducted on cylindrical cores obtained from the elements to determine concrete mechanical properties through compressive strength testing and NDT techniques, including RH test, UPV and surface resistivity measurements. These tests provide

valuable information on the structural implications of corrosion-induced damage and the remaining capacity of the concrete in the affected slab elements.

- **Phase 3:** There currently exist several conventional testing techniques (Lu et al., 2016; Rinaldi et al., 2010) that evaluate the current condition of prestressed concrete elements and identify internal damage and corrosion. However, a correlation between data from these tests and from NDTs is currently missing. In addition, existing laboratory methods on small samples extracted from real structures are not practical for the condition assessment of existing structures. In this research, three destructive testing methods were employed on actual field structures to develop new correlations that cover a variety of mechanical and durability properties for corroded concrete structures. These include the classic four-point bending test, tendon tensile test, and gravimetric steel mass loss analysis. A four-point bending test set-up was used with attention to boundary conditions to approximate the actual conditions in the bridge deck, including the contribution of arching action. The test investigated the influence of corrosion on the structural capacity of the members, considering sufficient tendon development length. The experimental test program concluded with a tensile test and gravimetric steel mass loss analysis to correlate the actual corrosion degree with the data measured in Phase 1.

The numerical research program consisted of the following aspects:

- The data obtained from the NDT and destructive testing was used as input parameters to develop non-linear finite element models (FEM) that simulate realistic field conditions, where the material is expected to undergo plastic deformation and exceed linear elastic behaviour. The commercial 2D FEM package VecTor2 (Wong et al., 2013) was used in the simulations. In addition to capturing concrete behaviour, the models specifically incorporate the evaluation of prestressing tendon bond quality, enabling investigation into how bond degradation affects overall member performance and failure modes. The FEM was calibrated and validated against experimental test data and relevant literature.

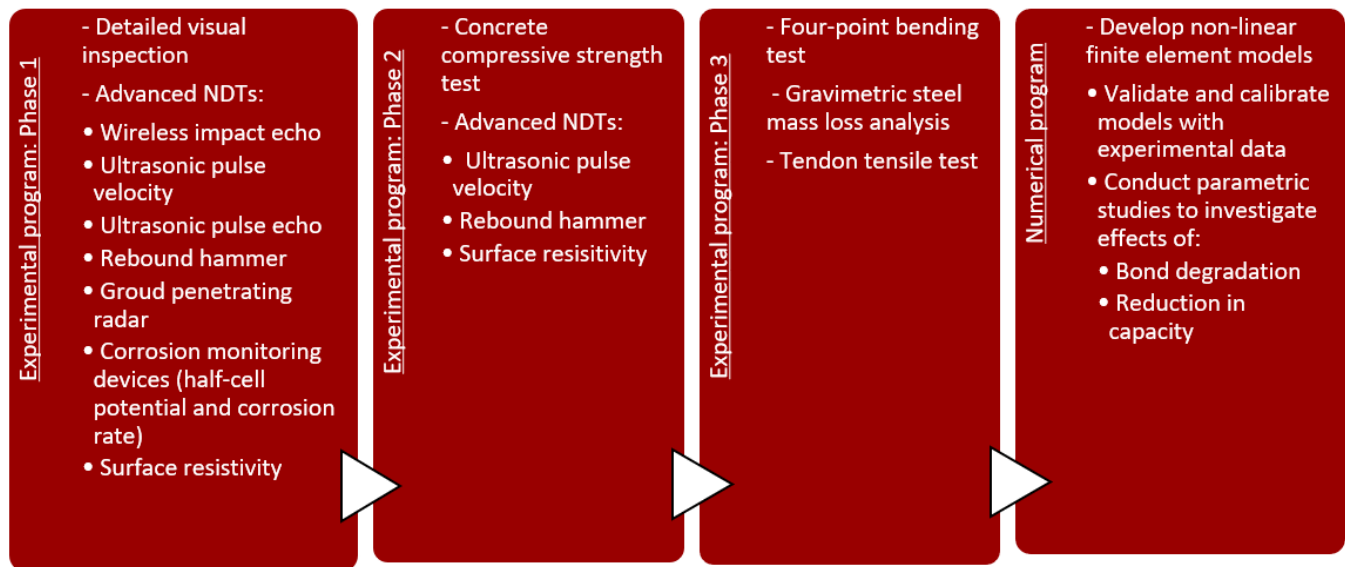


Figure 1-8 Components of research scope

## 1.5 Organization of thesis

This thesis is prepared in a “paper format,” including seven chapters. Each chapter is presented with its own bibliography. The chapters are as follows:

- Chapter 1 introduces the proposed thesis topic and provides the respective objectives and scope of work.
- Chapter 2 presents background information on the mechanism of corrosion and its adverse effects and summarizes the findings of previous studies on the flexural and shear behaviour of prestressed concrete beams. This chapter also discusses various methods that are currently employed to assess bridges and their components. Case studies on the condition assessment of bridges in similar conditions as the Champlain Bridge are also presented.
- Chapter 3 presents a comprehensive review of NDT techniques for evaluating corrosion-induced damage in prestressed concrete members. As aging infrastructure increasingly raises public safety concerns, assessing the condition and performance of corroded concrete elements through non-invasive means becomes critical, particularly since significant tendon cross-sectional loss can occur before visible signs appear. This chapter investigates a range of NDT methods applied to a concrete bridge deck, including visual inspection, Schmidt RH, UPV, corrosion detection techniques, GPR, UPE, and wireless IE. The study evaluates each method’s effectiveness in detecting internal defects and establishes correlations between test results to



improve the understanding of aging concrete infrastructure. This chapter has been published in *Structural Concrete* as Tawil et al. (2024).

- Chapter 4 focuses on evaluating the residual structural capacity of prestressed concrete bridge deck slabs. These slabs exhibited various levels of corrosion affecting their prestressing tendons. Testing under conditions simulating partial fixity allowed assessment of load capacity, crack development, energy dissipation, and failure mechanisms. Advanced analyses using sectional stress methods and digital image correlation provided deeper insights into the influence of pre-compression, tendon-to-concrete bond characteristics, and redistribution of internal forces. The findings emphasize the importance of tendon grouting quality and integrity in maintaining slab performance, showing that well-bonded tendons contribute to higher strength and ductility, while corrosion and bond deterioration can lead to brittle shear failures. This chapter will be submitted for publication in an international journal.
- Chapter 5 investigates the impact of corrosion on the mechanical behaviour of prestressing steel strands with a parallel-wire configuration, based on an extensive experimental program conducted on wires extracted from the decommissioned Champlain Bridge. The study evaluates the influence of corrosion on ultimate strength and ductility using a large dataset of individual wire tests. A logarithmic treatment of mass-loss data is adopted to capture the broad range of corrosion severities, and statistical analyses, including Welch's t-test, are used to identify significant changes in mechanical response across corrosion levels, while considering gauge length effects. The results demonstrate that mechanical degradation initiates at relatively low corrosion levels and becomes more pronounced with increasing mass loss. A consistent transition in ductility deterioration is identified near a 3% mass-loss threshold, a finding that is further validated through comparison with multiple independent datasets. The chapter highlights the robustness of this threshold and emphasizes the importance of mass-loss-based indicators for assessing the structural performance of corroded prestressing steel. This chapter will be submitted for publication in an international journal.
- Chapter 6 centers on the application of FEM techniques to evaluate the deterioration of bond between prestressing tendons and surrounding concrete, and its influence on the structural behaviour of bridge deck slabs. The chapter examines how varying degrees of bond degradation, caused by either corrosion or poor grouting conditions, affect key performance indicators such as load-carrying capacity, deflection patterns, and overall slab response. By

integrating experimental data on the extent of corrosion, the numerical models provide a detailed understanding of the interplay between bond quality and slab performance under realistic loading conditions. This chapter will be submitted for publication in an international journal.

- Chapter 7 synthesizes the multi-scale findings of this research to provide a set of comprehensive conclusions and establishes a performance-based framework for the assessment and management of corrosion-affected prestressed concrete bridge decks. This framework integrates forensic NDT protocols with structural evaluation strategies, providing actionable, data-driven guidance for infrastructure maintenance and rehabilitation.

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## **Chapter 2: Literature review**

### **2.1 Introduction**

Concrete durability may be affected by several deterioration mechanisms, including frost damage by freeze-thaw cycles, corrosion of reinforcement, sulphate attack, and internal swelling reactions. When considering the associated economic, technical, and social consequences, corrosion of steel reinforcement introduces the most severe impacts (François et al., 2018). Recent collapses of highway bridges, such as the Morandi bridge and Lowe's Motor Speedway pedestrian bridge, have highlighted the devastating effects resulting from progressive corrosion-induced deterioration (Domaneschi et al., 2020; Sly, 2001). These tragic events have proven that this mechanism poses a considerable durability problem in mild and severe climatic conditions (Bentur et al., 1997).

This chapter provides a comprehensive review of the mechanisms governing reinforcement corrosion and the resulting degradation of the mechanical properties of concrete and steel. The discussion extends to the structural performance of reinforced and prestressed concrete elements at both serviceability and ultimate limit states. Beyond material degradation, the chapter evaluates current global practices for the condition assessment of bridge elements, including a comparative review of Canadian and international inspection manuals and NDT techniques. Furthermore, the chapter examines the structural evaluation frameworks established by the Canadian Highway Bridge Design Code CSA S6 and the fib Bulletin 111.

### **2.2 Mechanism of reinforcement corrosion**

Under normal exposure conditions, concrete is an alkaline material (pH of 12 to 13). The concrete pore solution contains high concentrations of soluble calcium, sodium, and potassium oxides, which naturally protect the reinforcement from corrosion. This chemical compatibility between the reinforcing steel and concrete contributes to the composite behaviour of reinforced concrete, which in turn enhances its high durability. The state of high alkalinity results in the formation of a “passive” layer of ferric oxide,  $\text{Fe}_2\text{O}_3$ , around the steel surface, which is characterized by a dense and impenetrable film. The breakdown of this passive film, referred to as depassivation of steel, is triggered by either carbonation of the concrete cover or concrete contamination due to the ingress of chloride ions (Bertolini et al., 2013).

Unhydrated ferric oxide initially has a volume of about twice that of the steel it replaces. However, when it becomes hydrated in the presence of oxygen and moisture, it swells to an extent where the volume increase of the steel-concrete interface is up to six times, depending on the steel corrosion product (Poursaei, 2016). As depicted in Figure 2-1, various steel corrosion products occupy different volumes.

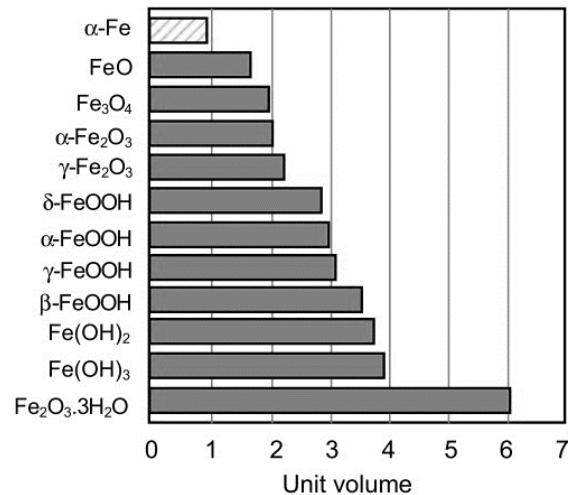


Figure 2-1 Volume of different corrosion products relative to iron (Fe) (reproduced from Poursaei, 2016)

This will inevitably lead to expansive forces in the surrounding concrete, which will cause cracking and spalling of the concrete, as well as the formation of red-brown brittle and flaky rust along the reinforcing steel bars. Figure 2-2 illustrates concrete cracking due to the expansion of oxide.

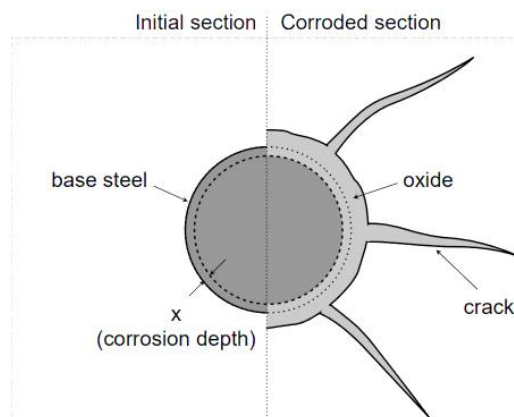


Figure 2-2 Cross-section of a corroding reinforcing bar with cracking due to expansion of oxide (reproduced from Andrade et al., 1993)

The following equations illustrate the formation of hydrated ferric oxide, known as rust (Broomfield, 2007; François et al., 2018):



Ferrous hydroxide



Ferric hydroxide



Hydrated ferric oxide (rust)

Concrete carbonation and concrete contamination by chlorides do not attack the integrity of concrete but rather render reinforcement vulnerable to corrosion provided there is sufficient humidity and oxygen in the concrete to sustain the associated electrochemical reactions.

### 2.2.1 Concrete carbonation

Carbonation of the concrete cover occurs as the carbon dioxide present in the atmosphere penetrates into the concrete pore solution to react with the alkaline hydroxides, as per the following reactions (Broomfield, 2007):



Carbonic acid



Calcium carbonate

The product of the first reaction between carbon dioxide and water is carbonic acid, a weak acid, which then reacts with the hydrates of the cement paste, such as Portlandite  $\text{Ca}(\text{OH})_2$ , to neutralize the alkalis in the pore water. The result is the formation of calcium carbonate ( $\text{CaCO}_3$ ). This reaction is also accompanied by a reduction in pH to a value between 8 and 9, which eventually leads to the destabilization and dissolution of the passive film. Figure 2-3 illustrates the quantity of rust formation as the pH level drops.

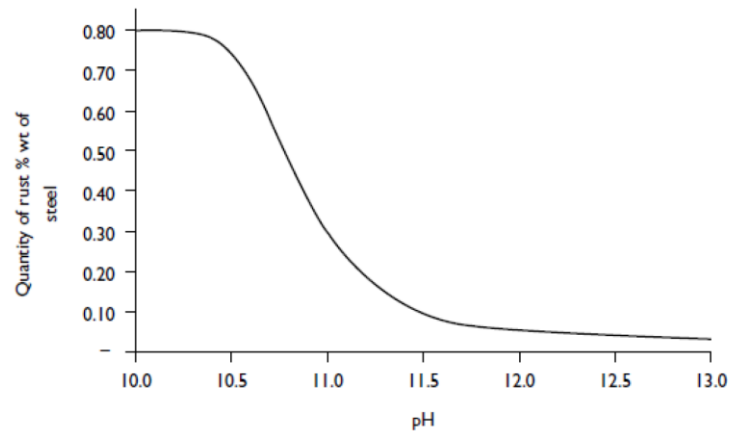


Figure 2-3 The variation of the quantity of rust formation as a function of pH (reproduced from Broomfield, 2007)

Although there exist debates on the type of corrosion induced by carbonation (T. T. Nguyen et al., 2022), it is generally believed that carbonation causes uniform damage along the steel reinforcement. Hence, this can be associated with damage at a “microcell” level, where continuous corrosion along the reinforcing steel is observed (Broomfield, 2007; François et al., 2018). Figure 2-4 illustrates the damage associated with concrete carbonation.

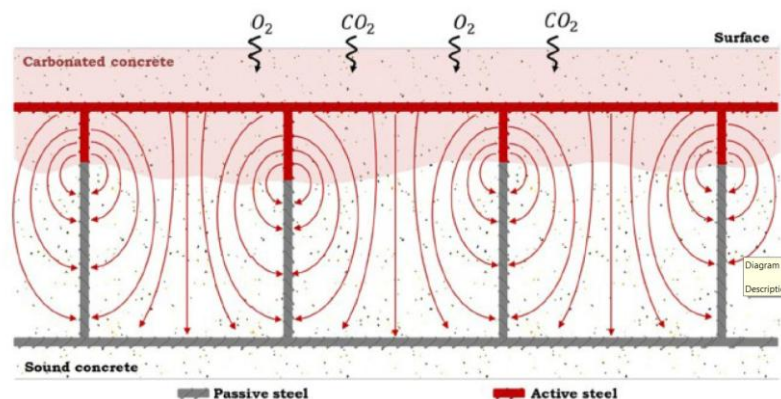


Figure 2-4 Uniform corrosion induced by concrete carbonation (reproduced from François et al., 2018)

The damage associated with concrete carbonation occurs at a faster rate when there is minimal concrete cover to the reinforcing steel. In addition, carbonation can also occur in the case when the concrete cover is thick enough, but the concrete is of poor quality; that is, the pores are well connected, allow the fast ingress of  $\text{CO}_2$ , and are characterized by a low level of alkalinity. The latter scenario occurs when the concrete mix solution has low cement content, high water-to-cement ratio and has undergone poor curing conditions.



The layer of carbonated concrete advances into the sound concrete according to Fick's law of diffusion. The law states that the rate of diffusion of a gas across a membrane is directly proportional to the concentration gradient of the substance on the two sides of the membrane and is also inversely proportional to the thickness of the membrane (Sundén, 2019). In the context of concrete carbonation, the diffusion of  $\text{CO}_2$  is significantly affected by the moisture content of the pores. Fully saturated concrete impedes gas diffusion, while dry concrete slows the reaction. Optimal carbonation occurs at approximately 80% pore saturation, balancing sufficient  $\text{CO}_2$  diffusion and water availability for the chemical reaction to proceed (Tuutti, 1982).

Many factors associated with reinforced concrete may affect the ability to resist carbonation-induced corrosion. From a design perspective, an adequate amount of concrete cover and a low water-to-cement ratio are essential to resist carbonation. In addition, a well-cured concrete that has also undergone good compaction makes the diffusion process difficult.

Concrete carbonation can be easily detected (Broomfield, 2007). A pH indicator, such as phenolphthalein mixed with water and alcohol, is used to detect the change in pH across a freshly exposed concrete surface. The test can be performed on concrete cores or fragments. The phenolphthalein solution changes from colourless at carbonated zones (low pH) to pink at uncarbonated zones (high pH).

### **2.2.2 Concrete Contamination by Chlorides**

Chlorides may originate in the concrete either from an external or internal source (Broomfield, 2007; François et al., 2018). External chlorides derive from direct contact of concrete with sea water or sea-salt spray, or from the de-icing salts used in extreme winter conditions. Internal chlorides, however, emerge from the inner concrete components, such as cement, water, additives, and aggregates. Examples of this include the intentional addition of chloride-set accelerators, such as sodium chloride ( $\text{NaCl}$ ) and the readily available calcium chloride ( $\text{CaCl}_2$ ), or the use of seawater or contaminated aggregates in the mix. However, in accordance with CSA A23.1, the use of  $\text{CaCl}_2$  as an accelerator for certain exposure classifications in structural concrete is prohibited (CSA Group, 2019b). This prohibition was introduced in the 1977 codes. In addition, current design standards have established stringent limits on chloride ion content by mass of the cementing material and the contributions from mixing water and aggregates used for concrete (CSA Group,

2019b). This ensures that the total chloride content in concrete remains below the permissible threshold.

The depassivation of the steel reinforcement by chloride attack is different than that by carbonation. To elaborate, the chloride ions attack and break down the steel passive layer. They function as catalysts to the corrosion process. However, unlike the carbonation mechanism, the concrete does not exhibit a drop in pH level. The chloride attack mechanism is illustrated in Figure 2-5. This mechanism is challenging to treat, as the ions undergo recycling and are difficult to eliminate.

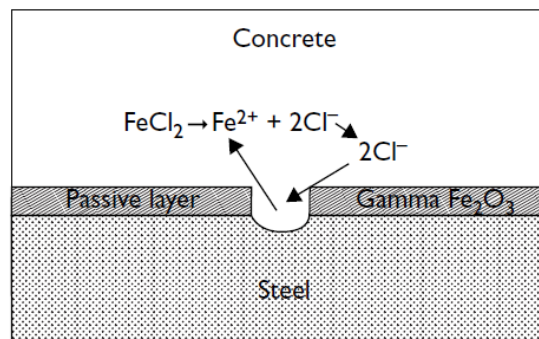


Figure 2-5 The break-down of the steel passive layer through chloride attack (reproduced from Broomfield, 2007)

Part of the chlorides that diffuse into concrete do not contribute to depassivation, as they become physically or chemically bound by the cement matrix. On the other hand, free chlorides dissolved in the concrete pore solution are the ions responsible for reinforcing steel depassivation.

Despite significant research efforts aimed at determining the critical chloride content for corrosion initiation, the scientific community remains engaged in an ongoing debate regarding the establishment of a clear threshold for this phenomenon (Angst et al., 2009). Although the critical chloride content is typically defined as the total chloride content relative to the weight of cement, other definitions include the free chloride content, either relative to the weight of cement or concrete, or as a concentration in the pore solution, and the chloride-to-hydroxyl ion ratio which relates the chloride ion activity to the pH of the pore solution.

Chloride-induced corrosion is characterized by the formation of areas of rust separated by areas of “clean” steel (Broomfield, 2007). This is commonly referred to as the “macrocell” phenomenon. It occurs as the presence of high amounts of water carry the chloride ions into the concrete. These

chlorides are characterized by their ability to absorb and retain moisture. The water located in the concrete pores enhance its electrical conductivity. As a result, this helps separate the anode and the cathode.

Chloride ions also induce localized corrosion, which is also referred to as pitting corrosion (François et al., 2018). The result of this mechanism is galvanic coupling between the new sites of active steel and the surrounding passive steel, as illustrated in Figure 2-6. Once the galvanic cell initiates, the reaction is self-maintained, as the electric field that is generated concentrates chloride ions in the active areas. Finally, the localized anodic sites exchange electrical current with the passive steel areas.

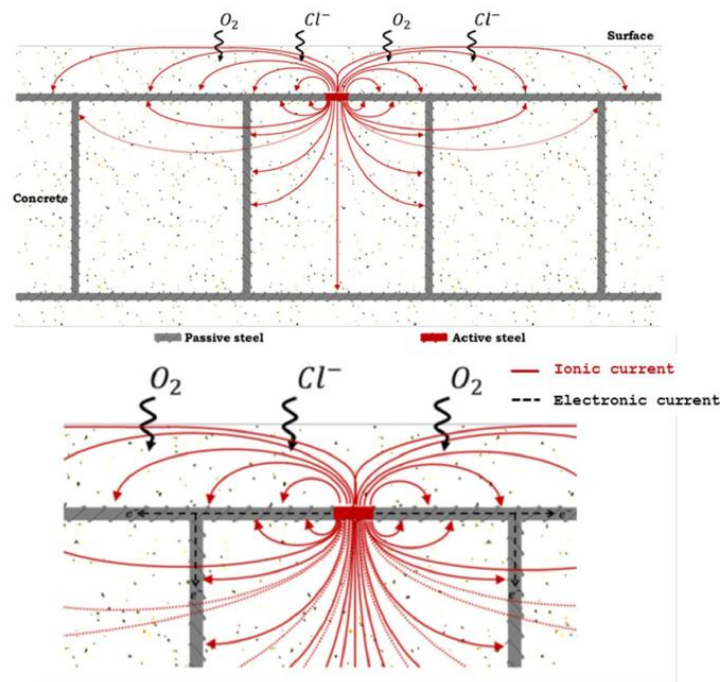


Figure 2-6 Illustration of chloride-induced corrosion (reproduced from François et al., 2018)

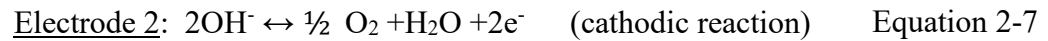
## 2.3 Types of corrosion

There are two types of corrosion observed in corroding reinforcement embedded in concrete. These are uniform corrosion and pitting corrosion.

### 2.3.1 Uniform corrosion

As previously mentioned, carbonation induces uniform corrosion, which is the electrical coupling of two infinitely close reversible electrodes. This allows the ohmic effects from the electrolyte's

(concrete) electrical resistivity to be disregarded. As a result, a potential gradient will not exist in such close space, which eventually gives rise to a uniform equilibrium potential of the coupled system. The following equations of equilibrium illustrate how steel corrosion in concrete results from bringing two reversible electrodes into electrolytic contact:



To maintain electrical neutrality, the two electrons created in the anodic reaction due to the dissolution of steel in the pore water must be consumed. This is done through the cathodic reaction, which also consumes water and oxygen. Figure 2-7 displays the full corrosion process of steel. The cathodic reaction generates hydroxyl ions, which then increase the alkalinity and strengthen the passive layer.

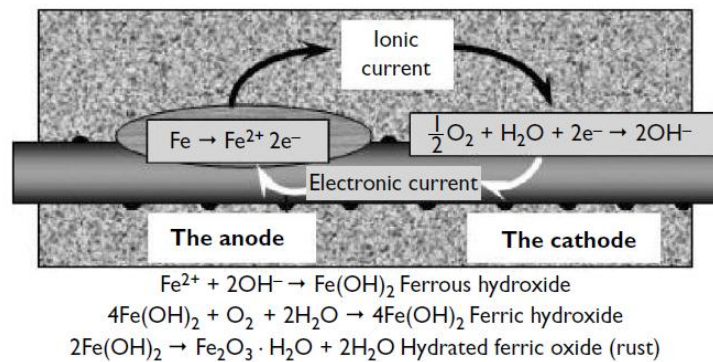


Figure 2-7 Diagram displaying the anodic, cathodic and oxidation/hydration reactions of corroding steel (reproduced from Broomfield, 2007)

The uniform property of corrosion, in this case, is due to the presence of Electrode 2 in dissolved form within the electrolyte, the concrete pore solution in this case, which is in contact with the steel (François et al., 2018). As a result, Electrode 2 exists across the entire surface of Electrode 1. The coupling of these two electrodes makes way for the direct transfer of electrons. This results in a new state of equilibrium, which is a function of the two coupled electrodes. The electrical coupling between the two electrodes also leads to their reciprocal polarization towards a common free potential, known as the corrosion potential,  $E_{\text{corr}}$ . This parameter consists of each of the electrode's reversible potential,  $E_{\text{rev}}$ . The reversible potential of Electrode 1 is lower than that of Electrode 2. This implies that Electrode 1 is anodically polarized and undergoes iron dissolution, whereas Electrode 2 is cathodically polarized and undergoes oxygen reduction. Corrosion

eventually occurs as a result of the anodic and cathodic reactions being no longer a function of the oxidized and reduced forms of the same electrode. The electrons freed through Electrode 1 are consumed to produce  $\text{OH}^-$  ions. The resulting corrosion potential of the coupling represents an equilibrium potential that is both anodic and cathodic and is defined by a unique  $E_{\text{corr}}$  solution. At equilibrium, the anodic current density defines the corrosion current density,  $i_{\text{corr}}$ . Once the corrosion current density is defined, it is then converted into metallic mass loss using Faraday's law.

### 2.3.2 Pitting corrosion

Pitting corrosion is a mechanism which occurs as a result of chloride attack. Pitting corrosion is the electrical coupling of two spatially separate electrochemical systems. Such systems may include two reversible electrodes, two distinct corrosion systems such as an active or passive system, or a reversible electrode coupled with a corrosion system. As a result, the large distance between the two electrodes allows the ohmic effects from the electrolyte's (concrete) electrical resistivity to stay in place. The same equations of equilibrium illustrated in Equations 2.6 and 2.7 apply to the case of pitting corrosion. In fact, the large distance between the anodic and cathodic sites results in a significant difference between the anodic and cathodic potentials, as well as a potential gradient in the volume and current (galvanic current or macrocell current). The difference between the anodic and cathodic potentials of a galvanic system introduces an additional variable, which eventually makes the system's state of equilibrium more difficult to define than the case of uniform corrosion. Similar to the case of uniform corrosion, the electrical coupling between the two corrosion systems leads to the active and passive sites being anodically and cathodically polarized, respectively. The result is an increase in the corrosion kinetics of the active steel and the cathodic protection of the passive steel (François et al., 2018). Figure 2-8 displays the typical pattern of pitting corrosion. Localized corrosion involves the growth of a corrosion spot, also referred to as an anodic site or pit.



Figure 2-8 Typical pattern of pitting corrosion (reproduced from François et al., 2018)

## 2.4 Corrosion-induced damage in post-tensioned tendon systems

The diagram illustrated in Figure 2-9 highlights and summarizes the effects of uniform (general) and pitting (localized) corrosion on a reinforced concrete member. While corrosion causes volumetric expansion, as well as reductions in reinforcement cross-sectional area and bond strength, the changes in reinforcement mechanical properties differ between uniform and pitting corrosion. Nevertheless, the decrease in the mechanical properties of the embedded reinforcement results in reductions in the load-carrying capacity of the member. Volumetric expansion causes cracking of the concrete cover, which then affects the effective depth of the member and its load-carrying capacity. In addition, the reduced cover thickness caused by corrosion-induced cover cracking can lead to a further reduction in the corrosion resistance of the reinforcement. This is so since the reinforcement is exposed to the surrounding environment, which may contain corrosive agents. In this case, a cycle of degradation initiates, where the reinforcement experiences further corrosion, additional cracking, and exposure. Reductions in bond strength due to low steel-to-concrete surface area cause reductions in the adhesion and mechanical interlock between the steel and the adjacent concrete, ultimately affecting the member's load-carrying capacity.

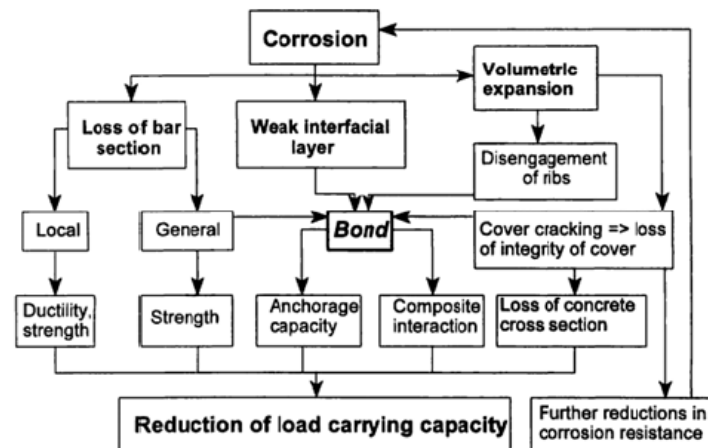


Figure 2-9 Summary of effects of uniform and pitting corrosion on a reinforced concrete member  
(reproduced from fib, 2000)

Bonded post-tensioned tendons are usually placed inside a polyethylene or galvanized steel duct embedded in concrete. Portland cement grout is then pumped into the duct and surrounds the tendons. A superior barrier against corrosion is offered when the duct is solidly and completely filled with grout. However, recent structural failures of corroded post-tensioned structures have

revealed that the main causes of corrosion are typically severe environmental conditions or inadequate detailing and construction practices, such as grout deficiencies (ACI Committee 222, 2014).

In post-tensioned systems, corrosion-induced damage manifests in ways similar to conventional reinforced concrete but with often more severe structural consequences due to the high stress levels in tendons. Such damage typically includes (Cairns et al., 2005):

- a) Reduction in area of steel reinforcement,
- b) Changes in the ductility of steel reinforcement due to pitting corrosion,
- c) Reduction in concrete area and mechanical properties, including corrosion-induced cracking, and
- d) Concrete-steel bond degradation.

#### 2.4.1 Reduction in area of steel reinforcement

The steel reinforcement cross-section is reduced in both chloride and carbonation-induced corrosion, although the pattern of loss differs. Uniform corrosion produces a relatively even reduction around the perimeter, whereas pitting corrosion leads to highly localized cross-section loss. Figure 2-10 illustrates how uniform corrosion occurs around the reinforcement perimeter, while pitting corrosion is localized.

The corrosion rate is a quantitative description of the corrosion propagation and is a measure of the loss of metal per unit of steel surface area per unit of time. The corrosion rate is estimated in terms of corrosion current density ' $i_{corr}$ '. For the case of uniform corrosion, the corrosion rate can be used to calculate the loss of metal according to Faraday's law of electrochemical equivalence (Val, 2007):

$$m = \frac{A \cdot I_{corr} t}{zF} \quad \text{Equation 2-8}$$

where  $m$  = metal mass loss (g),  $A$  = molar mass of the metal (55.85 g/mol for iron),  $I_{corr}$  = corrosion current (A),  $t$  = corrosion duration (s),  $F$  = Faraday's constant (96,494 C/(mol.e<sup>-</sup>) or A.s/(mol.e<sup>-</sup>)), and  $z$  = metal valency ( $z = 2$  for  $Fe \rightarrow Fe^{2+} + 2e^{-}$ ).

Knowing the change in mass of a corroded reinforcing bar allows to calculate the change in its cross-sectional area. This is based on the assumption that the corrosion current density is equal for

a group of ‘n’ reinforcing bars having the same diameter ‘D<sub>0</sub>’. Their cross-sectional area after ‘t’ years of uniform corrosion is calculated as follows:

$$A_s(t) = n \cdot \frac{\pi[D_0 - \Delta D(t)]^2}{4} \geq 0 \quad \text{Equation 2-9}$$

$$\Delta D(t) = 0.0232(i_{corr})(t) \quad \text{Equation 2-10}$$

$$i_{corr} = \frac{I_{corr}}{(\pi)(d_b)(L)} \quad \text{Equation 2-11}$$

where  $\Delta D$  = change in reinforcement diameter,  $i_{corr}$  = corrosion current density ( $\mu\text{A}/\text{cm}^2$ ), and L is the length of the reinforcing bar.

For the case of pitting corrosion, however, the corrosion current density cannot be converted directly into a loss of cross-sectional area of corroded steel. Studies have shown that the depth of a pit ‘p’, which is also referred to as the maximum penetration of pitting after ‘t’ years of corrosion initiation can be calculated as follows (Val, 2007):

$$p(t) = 0.0116 \cdot i_{corr} \cdot R \quad \text{Equation 2-12}$$

where R = ratio between the maximum penetration of pitting, ‘P<sub>max</sub>’ (mm), to the average corrosion penetration ‘P<sub>av</sub>’ (mm).

When estimating the loss of cross-sectional area of a reinforcing bar due to a pit formation, some assumptions about the geometry of the pit are required. The formulations suggested by Val and Melchers (1997) consider a hemispherical model of a pit, as shown in Figure 2-10.

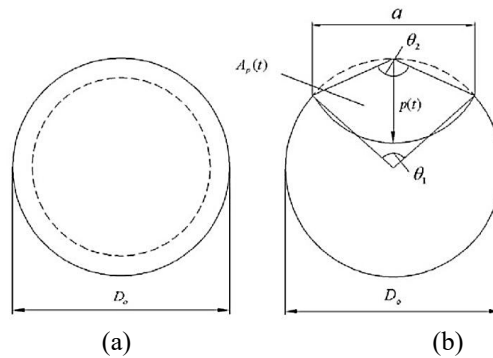


Figure 2-10 Reinforcement steel section loss due to: (a) uniform corrosion, (b) pit configuration  
(reproduced from Val, 2007)



Based on this assumption, the cross-sectional area of a pit, 'A<sub>p</sub>', in a corroded reinforcing bar with an initial diameter of 'D<sub>0</sub>' after 't' years of corrosion can be estimated as follows (Val, 2007):

$$A_p(t) = \begin{cases} A_1 + A_2 & p(t) \leq \frac{D_0}{\sqrt{2}} \\ \frac{\pi D_0^2}{4} - A_1 + A_2 & \frac{D_0}{\sqrt{2}} < p(t) \leq D_0 \\ \frac{\pi D_0^2}{4} & p(t) > D_0 \end{cases} \quad \text{Equation 2-13}$$

where:

$$A_1 = \frac{1}{2} \left[ \theta_1 \left( \frac{D_0}{2} \right) - a \left| \frac{D_0}{2} - \frac{p(t)^2}{D_0} \right| \right] \quad \text{Equation 2-14}$$

$$A_2 = \frac{1}{2} \left[ \theta_2 p(t)^2 - a \frac{p(t)^2}{D_0} \right] \quad \text{Equation 2-15}$$

$$a = 2p(t) \sqrt{1 - \left[ \frac{p(t)}{D_0} \right]^2} \quad \text{Equation 2-16}$$

$$\theta_1 = 2 \arcsin \left( \frac{a}{D_0} \right) \quad \text{Equation 2-17}$$

$$\theta_2 = 2 \arcsin \left( \frac{a}{2p(t)} \right) \quad \text{Equation 2-18}$$

### 2.4.2 Changes in steel ductility due to pitting corrosion

The formation of pits introduces large and highly localized strains in the reinforcement. To elaborate, since the length of the defect is small, the average strain over a finite length is smaller than the local strain at the pit location. As a result, the bar breaks at an average strain much lower than the ultimate strain of the virgin steel. This reduces the average ductility of the bars to a point where its behaviour becomes very brittle (when pitting reaches 50% of the sectional area) (Coronelli & Gambarova, 2004). The reduction in the bar ductility is implemented by enforcing lower ultimate strains for the corroded bar. Based on the experimental work produced by Cairns and Millard (1999) and Castel et al. (2000), the following formula for the percent reduction of the bar cross-section may be used:

$$\alpha_{pit} = \frac{\Delta A_{pit}}{A_{bar}} \times 100 (\%) \quad \text{Equation 2-19}$$

where  $\Delta A_{pit}$  = area reduction due to pitting, and  $A_{bar}$  = nominal bar cross sectional area.

The change in the ultimate strain as a function of the percent of section reduction can be observed as follows:

$$\epsilon'_{su} = \epsilon_{sy} + (\epsilon_{su} - \epsilon_{sy}) \cdot \left(1 - \frac{\alpha_{pit}}{\alpha_{pit}^{max}}\right) \quad \text{for } \alpha_{pit} < \alpha_{pit}^{max} \quad \text{Equation 2-20}$$

where values for  $\alpha_{pit}^{max}$  range between 0.1 – 0.5, as reported in the literature (Coronelli & Gambarova, 2004). Figure 2-11 (b) illustrates the change in the behaviour of the reinforcement as a result of corrosion.

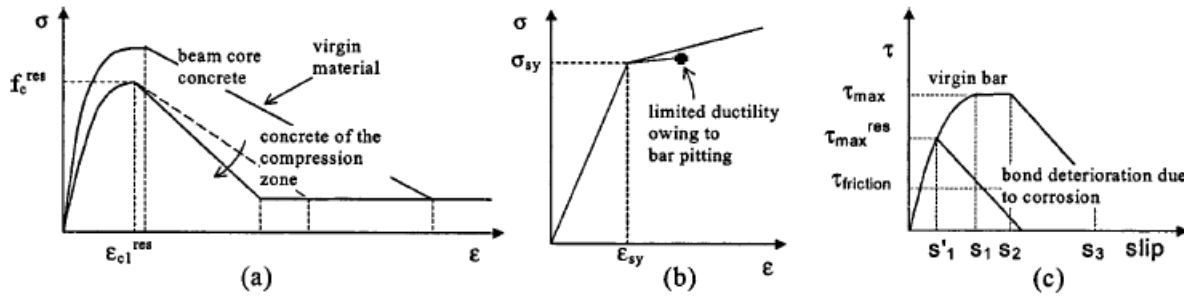


Figure 2-11 (a) Concrete behaviour in compression, (b) steel behaviour, (c) bond behaviour (reproduced from Coronelli & Gambarova, 2004)

Lu et al. (2016) examined the reduction in the mechanical performance of corroded prestressing strands and developed a damage model to explain their failure mechanisms. The findings demonstrated that the strength and ductility of corroded strands decrease with increasing corrosion levels. Furthermore, it was found that the hemispherical model for the pit shape, highlighted in Figure 2-10, accurately predicts the reduction in strength of corroded prestressing strands.

Franceschini et al. (2022) experimentally explored the tensile behaviour of twenty-four naturally corroded prestressing strands that were retrieved from full-scale prestressed concrete beams exposed to chlorides from sea water for 10 years. The beams were extracted from a refrigeration tower of a thermal power plant, where they were exposed to constant wet and dry cycles. The beams shared the same rectangular cross section of 150 mm x 300 mm and were reinforced with 2 seven-wire strands with an equivalent diameter of 12.9 mm located at the bottom of the cross-section. Details of the beam geometry and reinforcement can be seen in Figure 2-13. The six external wires and the core wire had a radius of 2.13 mm and 2.19 mm, respectively. Uncorroded strands were identified and measured for their cross-sectional area (100 mm<sup>2</sup>). The area of the inner and outer wires was 15 mm<sup>2</sup> and 14.22 mm<sup>2</sup>, respectively. The samples were divided into

two main groups according to their lengths: the first group consisted of twelve strands of 450 mm, while the second group had twelve strands 700 mm long.

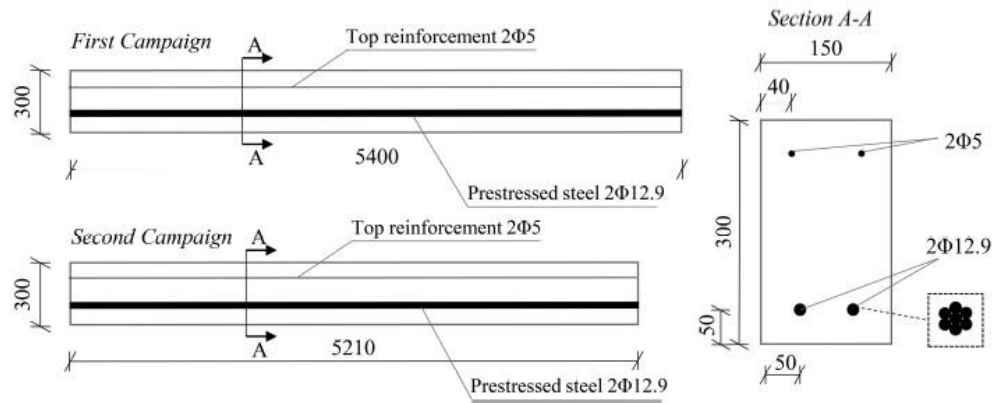


Figure 2-12 Reinforcement details and geometry of beams tested by Franceschini et al. (2022)

Prior to tensile testing, the mass loss due to corrosion for each strand was evaluated as per ASTM G1 (ASTM G1, 2011). Pit depths and maximum cross-sectional loss for each wire along the length of the samples were also measured. Subsequently, tensile tests on 18 corroded and 4 uncorroded prestressing strands were carried out using a universal testing machine. During the tests, the two ends of each strand were held by the grips through epoxy resin moulded into steel tubes to avoid damaging the end parts of the specimens. The samples were then loaded to failure using a displacement-control method, and the failure behaviour of the corroded and non-corroded strands were noted. Digital Image Correlation (DIC) was also used during the testing phase to estimate the strain field of each sample. Figure 2-13 and Figure 2-14 display the stress-strain responses for the uncorroded and corroded samples, respectively. The stress-strain curve for the uncorroded prestressing strands consists of an elastic phase, followed by a transient yielding phase and a hardening phase. The ultimate strength,  $f_{pu}$ , and strain,  $\epsilon_{pu}$ , of the uncorroded samples were found to be 1902 MPa and 0.0051, respectively. The results for the corroded strands are summarized in

Table 2-1, where the failure of a strand was assumed to correspond with the failure of its first most corroded wire.

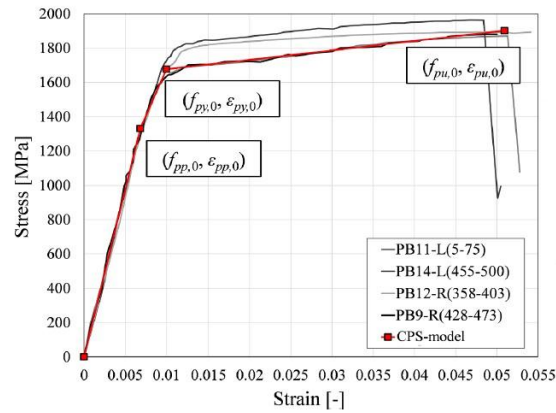


Figure 2-13 Stress-strain response of uncorroded samples (reproduced from Franceschini et al., 2022)

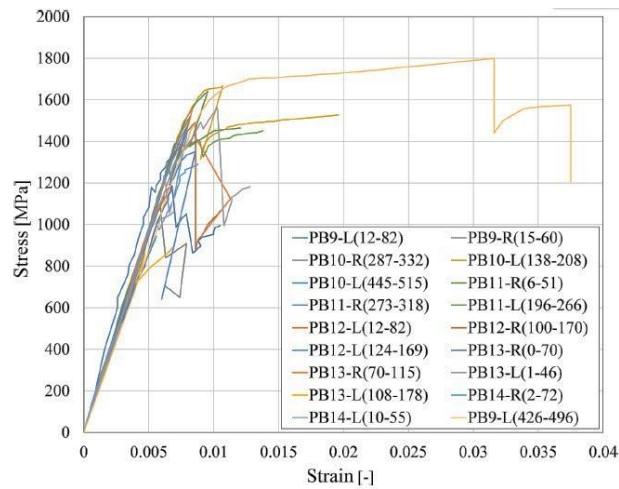


Figure 2-14 Stress-strain response of corroded samples (reproduced from Franceschini et al., 2022)

Table 2-1 Results of the experimental tensile test (reproduced from Franceschini et al., 2022)

| Sample ID       | Type of failure | $f_{pu,corr,exp}$ (MPa) | $\epsilon_{pu,corr,exp}$ |
|-----------------|-----------------|-------------------------|--------------------------|
| PB9-L(426-496)  | ELA/PLA         | 1799                    | 0.0316                   |
| PB-9R(15-60)    | ELA             | 1082                    | 0.0059                   |
| PB-9R(428-473)  | ELA/PLA         | 1880                    | 0.0500                   |
| PB10-L(138-208) | ELA/PLA         | 1667                    | 0.0100                   |
| PB10-L(445-515) | ELA             | 944                     | 0.0058                   |
| PB10-R(287-332) | ELA             | 1075                    | 0.0045                   |
| PB11-L(5-75)    | ELA/PLA         | 1871                    | 0.0513                   |
| PB11-L(196-266) | ELA             | 1516                    | 0.0082                   |
| PB11-R(6-51)    | ELA             | 1642                    | 0.0088                   |
| PB11-R(273-318) | ELA             | 1387                    | 0.0075                   |
| PB12-L(12-82)   | ELA             | 1185                    | 0.0069                   |
| PB12-L(124-169) | ELA             | 1491                    | 0.0081                   |
| PB12-R(100-170) | ELA             | 1492                    | 0.0086                   |
| PB12-R(358-403) | ELA/PLA         | 1893                    | 0.0541                   |
| PB13-L(1-46)    | ELA/PLA         | 1562                    | 0.0103                   |
| PB13-L(108-178) | ELA             | 800                     | 0.0041                   |
| PB13-R(0-70)    | ELA             | 1381                    | 0.0077                   |
| PB13-R(70-115)  | ELA             | 1562                    | 0.0089                   |
| PB14-L(10-55)   | ELA             | 1239                    | 0.0079                   |
| PB14-L(455-500) | ELA/PLA         | 1963                    | 0.0484                   |
| PB14-R(2-72)    | ELA             | 1452                    | 0.0075                   |

ELA/PLA: plastic behaviour characterized by the elastic, yielding and hardening phases;

ELA: brittle behaviour characterized by the elastic and yielding phases, or the elastic phase only.

It is clear that a significant decrease in the ultimate strength and strain values are observed with increases in corrosion level and cross-sectional loss. In addition, the most corroded wire experienced fracture at an earlier stage than the other external wires, which resulted in a sudden decrease in the strand resistance. As shown in Figure 2-14 and

Table 2-1, increases in corrosion levels results in decreases in the length of the hardening phase. Strands that experienced no hardening underwent significant brittle failures. The results of this study highlight the detrimental effects of corrosion on prestressing strands due to the shift in failure modes from ductile to brittle.

### 2.4.3 Concrete section reduction and material behaviour

As the corrosion products introduce expansive forces, the concrete cover experiences cracking and spalling. The thickness of the concrete cover may also be reduced when modelling the effects of corrosion. In the compression zone of a concrete element, cracking and spalling are considered by applying a brittle post-peak behaviour, as well as reducing the strength of the concrete cover. According to the Modified Compression Field Theory by Vecchio and Collins (1986), the reduction of the compressive strength depends on the magnitude of the average tensile strain in the transverse direction, which introduces longitudinal microcracks along the concrete element. The reduced concrete strength can be obtained as follows:

$$f_c^* = \frac{f_c'}{1 + K \cdot \frac{\epsilon_1}{\epsilon_{co}}} \quad \text{Equation 2-21}$$

where  $K$  = coefficient related to bar roughness and diameter;  $\epsilon_{co}$  = strain at the peak compressive stress  $f_c'$ ; and  $\epsilon_1$  = average (smeared) tensile strain in the cracked concrete at right angles to the direction of the applied compression. In the case of concrete tensile strains induced by the expansion of corrosion products, the value of  $\epsilon_1$  is calculated using the following equation:

$$\epsilon_1 = \frac{(b_f - b_o)}{b_o} \quad \text{Equation 2-22}$$

where  $b_o$  = section width in the virgin state (no corrosion-induced cracks); and  $b_f$  = increased beam width due to corrosion cracking. Equation 2-23 gives an approximation of the increase in the beam width:

$$b_f - b_o = n_{bars} \cdot w_{cr} \quad \text{Equation 2-23}$$

where  $n_{bars}$  = the number of bars in the compression zone, and  $w_{cr}$  = the total crack width for a given corrosion penetration depth ' $X(t)$ ', calculated using Equation 2.24.

$$X(t) = \frac{\Delta D(t)}{2} \quad \text{Equation 2-24}$$

The total crack width may also be calculated as proposed by Molina et al. (1993):

$$w_{cr} = \sum u_{i-corr} = 2\pi(v_{rs} - 1).X \quad \text{Equation 2-25}$$

where  $v_{rs}$  = ratio of volumetric expansion of the oxides with respect to the virgin material,  $X$  = depth of the corrosion attack (mm), and  $u_{i-corr}$  = opening of each single corrosion-induced crack (mm). As suggested by Molina et al. (1993), the value of  $v_{rs}$  is taken equal to 2. Equation 2-25 assumes that all the corrosion products accumulate around the corroded bar (Coronelli & Gambarova, 2004). Figure 2-15 illustrates the corrosion-product accumulation around a bar, as well as the crack width.

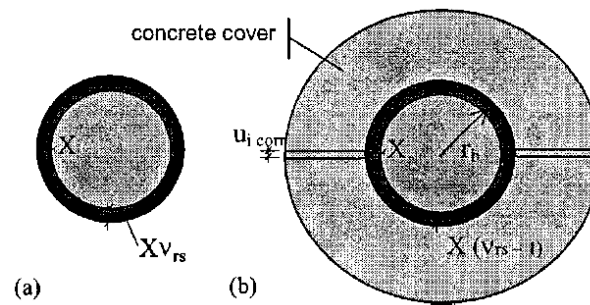


Figure 2-15 (a) Corrosion product accumulation around a bar, (b) total corrosion crack width  $w_{cr}$   
(reproduced from Coronelli & Gambarova, 2004)

#### 2.4.4 Bond Degradation

The composite action between the concrete and reinforcement is introduced through a bond stress-slip relation at the interface of the two materials (see Figure 2-11(c)). The transfer of bond stresses between the two materials occurs due to chemical adhesion, mechanical interlocking, and friction. However, corrosion of the reinforcement affects the bond strength in multiple ways. As depicted in Figure 2-16, an initial increase of the bond strength is exhibited at low corrosion levels (pre-cracking stage). This is due to the evolution of rust which enhances the bar surface roughness (Al-Sulaimani et al., 1990; Coronelli & Gambarova, 2004). The behaviour is then followed by a decrease in bond strength due to the cracking and splitting of the concrete cover. This is attributed to the reduction in the height of the ribs present along a deformed bar, the decrease in mechanical interlock between the ribs and concrete, the growth of corrosion products, as well as the reduction in bar confinement due to the longitudinal cracking of the concrete cover (Al-Sulaimani et al., 1990).

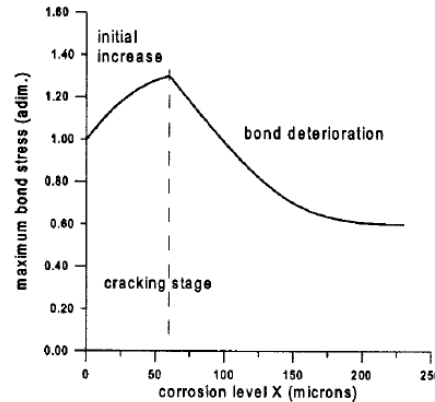


Figure 2-16 Variation of the bond strength as a function of corrosion level (reproduced from Coronelli & Gambarova, 2004)

Bond failure in corroded bars generally occurs by splitting (Coronelli & Gambarova, 2004). As a result, the parameters of the bond stress-slip relationship must be adjusted to reflect this brittle behaviour. As illustrated in Figure 2-11(c), depending on the corrosion level, the bond strength ' $\tau_{\max}^{\text{res}}$ ', the slip at the maximum bond stress ' $s_1$ ' and the softening branch are modified. In addition, cover splitting affects the post-peak behaviour by eliminating the plateau and the frictional strength.

Li & Yuan (2013) carried out an experimental program to investigate the bond behaviour and mechanism between prestressing strands and concrete using 12 pull-out specimens. Eight specimens underwent corrosion while the remaining four were the uncorroded control specimens. The specimens were mixed with chloride and were exposed to both indoor and outdoor natural climate to simulate actual corrosion effects. Concrete prisms of dimensions 300 mm x 150 mm x 150 mm reinforced with two bonded strands and one stirrup circuit were prepared, as illustrated in Figure 2-17.

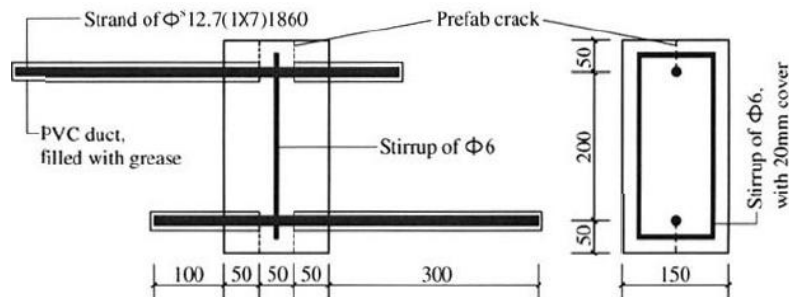


Figure 2-17 Reinforcement details and geometry of test specimens investigated by Li & Yuan (2013)



Prior to testing the specimens, the accelerated corrosion method was adopted by mixing salt in the concrete and exposing the specimens to indoor and outdoor natural climate conditions. Subsequently, pull-out tests on all 12 specimens were completed. The results showed that regardless of the location of corrosion along the span length, bond strength degradation and a decrease in global strength were observed.

## **2.5 Structural performance of corroded flexural members**

A number of experimental studies have been conducted to investigate the behaviour of reinforced and prestressed concrete elements subjected to reinforcement corrosion at serviceability states. Rodriguez et al. (1997) performed an experimental study where the serviceability behaviour of reinforced concrete beams with corroded reinforcement was investigated. The load-displacement responses showed a noticeable reduction in the stiffness at both the initial and post-cracking stages. This can be attributed to the formation and opening of the bending and bending-shear cracks, as well as the crushing of the concrete cover (reduction in concrete cross-section), yielding of the longitudinal reinforcement and failure of the concrete-steel bond. The extent of the above structural response depends on the corrosion level. In addition, as the corrosion level increases, the probability of having shear cracks at low load levels increases as well (Coronelli & Gambarova, 2004; Rodriguez et al., 1997).

A study performed by Castel et al. (2000) showed that the stiffness reduction depends on the deterioration of both the bar cross-section and the concrete-steel bond. The strain increase at the pit locations leads to an increase in the element displacement at serviceability levels.

Furthermore, El Menoufy (2015) investigated the fatigue properties of corroded and non-corroded pretensioned concrete beams with externally bonded Carbon Fibre Reinforcement Polymer (CFRP) sheets. The results showed significant reductions in the fatigue resistance of the corroded prestressed beams when compared to that of the non-corroded specimens.

Mirza et al. (2015) highlighted the effects of corrosion on the reduction of strand tensile strength and ductility by examining the effects of corrosion on eight prestressed concrete prisms under different prestressing levels. No correlation was found between the level of prestress and corrosion activity.

A review of published literature reveals fewer research on the deterioration of mechanical properties of prestressing strands than on reinforcing steel. The fundamental mechanisms for corrosion of prestressing steel in concrete are essentially identical to those for lower-grade reinforcing bars (ACI Committee 222, 2014). However, in addition to differences in the steel materials and properties (high-strength steel tendons and mild deformed steel reinforcement), the mechanical principles of prestressed concrete structures differ from that of reinforced concrete structures. The induced prestressed forces not only alter the cracking pattern of a prestressed element, but also enhance the shear capacity (Marí et al., 2016). Furthermore, there are differences in the degradation of prestressing steel strands and deformed bars under corrosion due to various reasons (ACI Committee 222, 2014). To begin with, throughout their service, prestressing tendons are continuously subjected to a stress ranging from 55 to 65 percent of their ultimate tensile strength. Therefore, any losses of cross-sectional areas will increase the net tension stress and can lead to local yielding and fracture. Furthermore, the strength of prestressing tendons is typically four to five times higher in magnitude than that of non-prestressed reinforcing steel. As such, prestressing steel is usually manufactured and utilized in the form of smaller diameter wires or individual strands. Smaller-diameter prestressing steel wires are prone to losing relative cross-sectional area more rapidly than larger non-prestressing bars at similar corrosion rates. As a result, corrosion-induced sectional reductions and notch effects in prestressing strands may result in structural collapse without warning.

Research on the structural behaviour of prestressed concrete beams is currently more abundant than that on prestressed one-way concrete slabs. However, the behaviour and design of one-way concrete slabs is generally similar to that of concrete beams. As a result, this section reviews some previous studies that have investigated the flexural and shear behaviour of corroded prestressed concrete beams tested under static loading.

### **2.5.1 Flexural behaviour**

Mircea et al. (1994) investigated the durability of 130 prestressed concrete beams and 130 reinforced concrete beams subjected to a long-term aggressive degradation process and a constant load for a period ranging between 10 and 12 years. Details of the prestressed elements are shown in Figure 2-18.

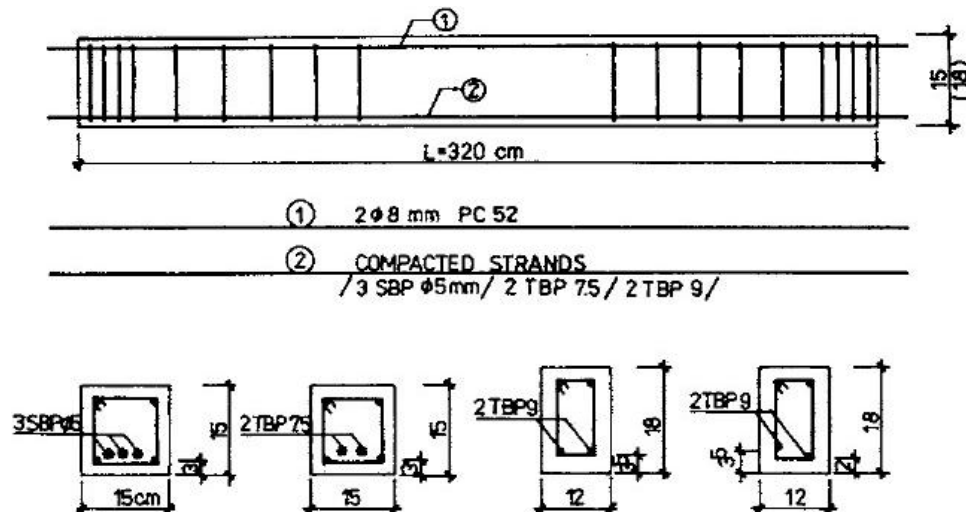


Figure 2-18 Reinforcement details and geometry of prestressed beams tested by Mircea et al. (1994)

After subjecting the beams to bending tests, it was reported that the prestressed beams exhibited smaller crack widths than the reinforced concrete beams. In addition, a reduction in ductility was observed for all beams. However, no data for the attained corrosion levels were documented.

The failure mode of non-corroded beams depends on the reinforcement ratio. To elaborate, lightly reinforced concrete beams exhibit failure at the ultimate strain, without experiencing concrete crushing (steel yielding precedes concrete crushing), while moderately reinforced beams may fail by concrete crushing. However, for the case of corroded beams, the order of events that lead to beam failure differs. Rodriguez et al. (1997) showed that lightly reinforced concrete beams exhibiting mild corrosion experienced steel yielding and concrete crushing at similar times. As the corrosion level increased for such beams, concrete crushing preceded steel yielding. This trend displays a progressive loss of ductility. The structural responses may even become brittle due to the loss of bar ductility as a result of pitting corrosion.

Rinaldi et al. (2010) investigated the effects of corrosion on prestressed members by performing experimental tests on nine pretensioned concrete beams. The beams had rectangular cross sections of dimensions 200 mm x 300 mm x 3,000 mm. The beams were first subjected to artificial corrosion using a sodium chloride solution to obtain different degrees of corrosion. The beams were then subjected to four-point bending tests. The beams were prestressed with 11.21-mm diameter seven-wire strands. Two prestressed tendons were placed at the bottom of each section, while one was placed at the top. Regular reinforcing steel consisting of 2-10 mm diameter bars

was placed at the top and bottom of each section. In addition, double-legged 8-mm-diameter stirrups were also placed along the length of the beams. The top, bottom and stirrup reinforcement were all epoxy-coated against corrosion. Details of the beam geometry and reinforcement are shown in Figure 2-19.

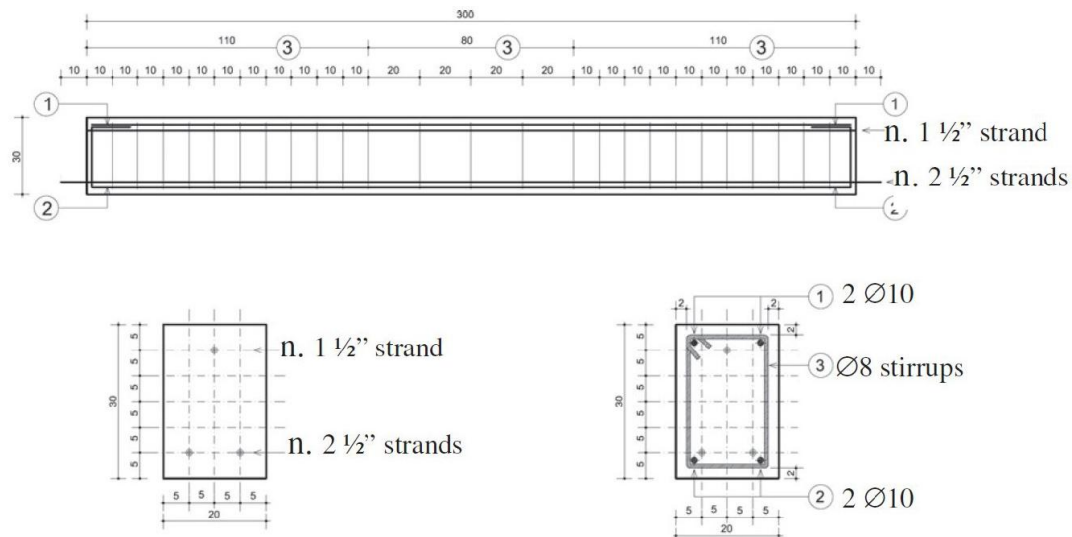


Figure 2-19 Reinforcement details and geometry of test specimens prepared by Rinaldi et al. (2010)

Corrosion was limited to the central zone of the beams, such that only the bottom two strands were to corrode. The specimens were then subjected to four-point bending tests. For each beam, the load was increased, and the vertical mid-span displacements were measured until failure. Once the testing was complete, the corroded strands were cut, cleaned, and weighed. The mass loss was then obtained.

The results showed that while the reference uncorroded beams collapsed due to concrete crushing, corroded beams with mild to severe corrosion levels (14% and 20% mass loss for each strand, respectively) experienced local wire rupture. Corrosion was found to have a significant effect on the ultimate capacity and failure mode of prestressed concrete beams, where the structural response changed from ductile to brittle. These findings are shown in Figure 2-20.

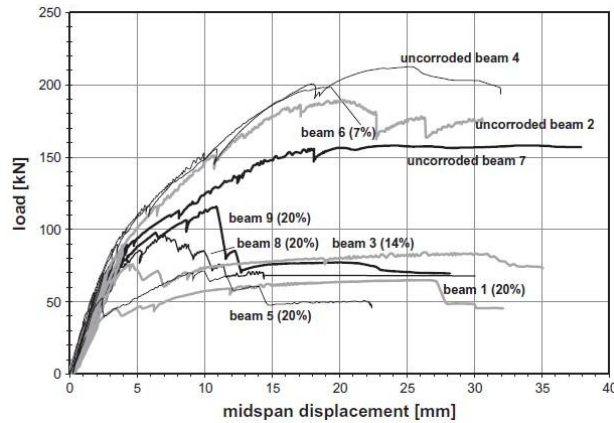


Figure 2-20 Load-midspan deflection curves for specimens tested by Rinaldi et al. (2010)

Rogers et al. (2012) investigated the behaviour of 19 pretensioned beams extracted from the Tiwai Point, a decommissioned bridge in New Zealand, after 42 years of service. The elements were exposed to an extremely aggressive coastal environment, which led to various degrees of corrosion and a loss of up to 60% of the cross-section of the bottom layer of pretensioned strands. Details of a typical beam are shown in Figure 2-21. Some tested beams had different reinforcement arrangements. Central beams contained 12-12.7 mm diameter strands, while edge beams contained 10-12.7 mm pretensioned strands.

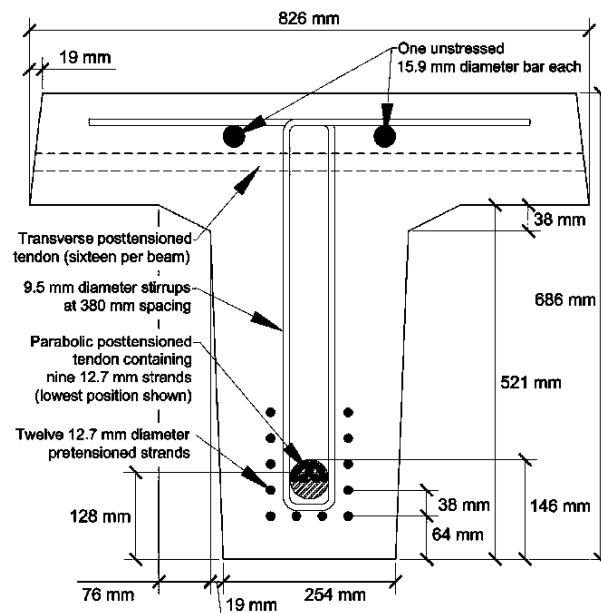
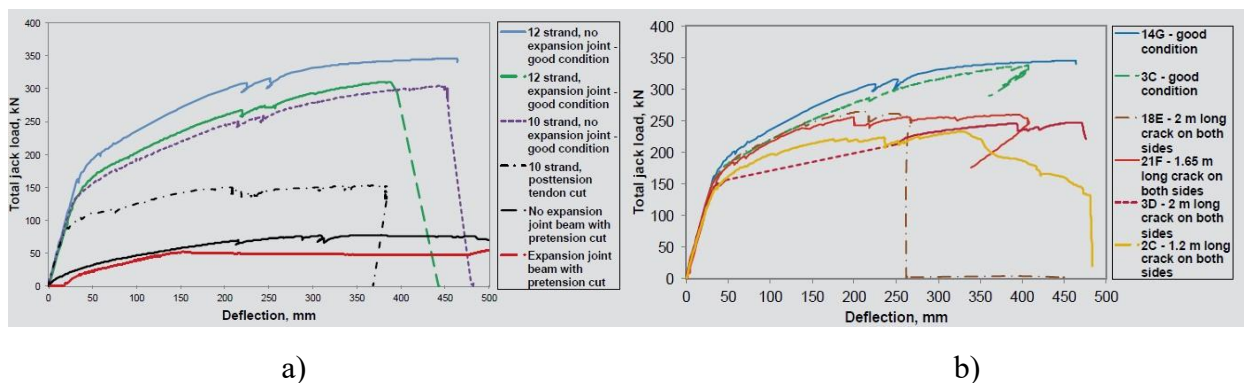


Figure 2-21 Reinforcement details and geometry of test specimens prepared by Rogers et al. (2012)

Prior to conducting destructive testing, a detailed visual inspection as well as NDT, including a cover depth survey and electro-potential mapping (EPM), were performed. It was found that the majority of the beams exhibited minor cracking and spalling along the sides of the web, which is consistent with corrosion of the stirrups. In addition, some beams experienced longitudinal cracking along the sides of the web and at the elevation of the pretensioned reinforcement. This was also accompanied by rust staining. A cover survey was then performed using an electromagnetic cover meter to determine the cover depth to the pretensioned reinforcement. The results showed no external evidence of strand corrosion in areas of the lowest cover depth measurements. In addition, EPM was performed on the sides of the webs and beam soffits to identify any areas of active corrosion that were not visible on the concrete surface. The results indicated a strong correlation between areas identified as corroding by EPM and areas of strand failure. The next stage of the experimental work was the destructive testing of the beams using a four-point bending test. Results of the load-deflection responses are summarized in Figure 2-22. It was found that all corroded beams displayed a reduction in their capacities, with strength losses being roughly proportional to the number of corroded strands. While the control specimens exhibited a mixed shear-flexural failure mode characterized by concrete crushing and flexural shear cracks, the corroded beams experienced pure flexural failure modes, accompanied by flexural cracks extending from the corrosion cracks. Strand failure was also identified in elements with severe corrosion. Following destructive testing of the beams, a corrosion damage survey was implemented at failure locations. The ducts making up the post-tensioned tendons did not suffer from any damage and were well grouted.



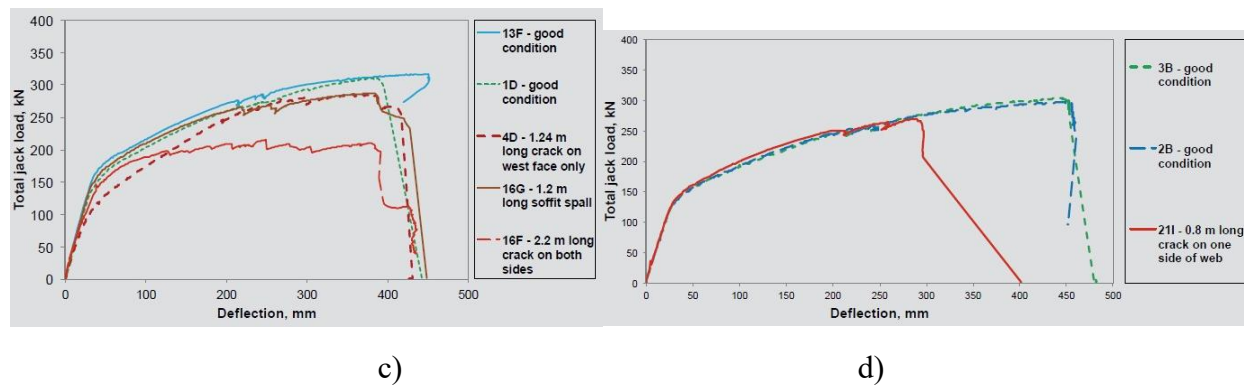


Figure 2-22 Experimental test results by Rogers et al. (2012): a) control samples, b) samples with 12 pretensioned strands with no expansion joint, c) samples with 12 pretensioned strands with expansion joint, d) samples with 10 pretensioned strands with no expansion joint

Pape & Melchers (2013) conducted flexural testing on three 45-year-old corroded prestressed concrete 'T'-beams extracted from a bridge structure, with the intent of investigating their structural performance. The beams were selected from the structure based on having superficially little or no cracking damage, medium observed damage, and the most severe observed damage. The beams were reinforced with two 18-5 mm wire high tensile strands, as shown in Figure 2-23.

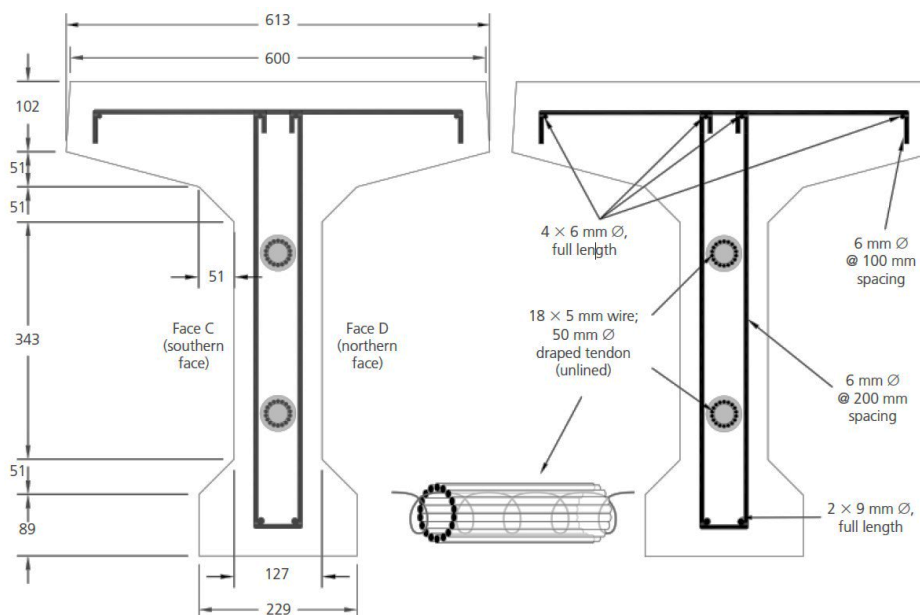


Figure 2-23 Reinforcement details and geometry of test specimens investigated by Pape & Melchers (2013)

The damage assessment was performed by a visual inspection, where beam A was found to be in good condition, while beam B was found to be in reasonable condition, having longitudinal cracks on both sides of the web, and beam C was in poor condition, as it suffered from severe longitudinal cracking on both sides of the web. Each beam was tested using a four-point bending setup, as shown in Figure 2-24.

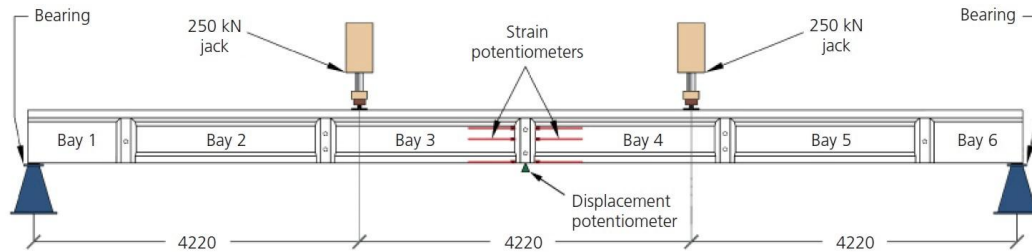


Figure 2-24 Four-point bending test set-up, as prepared by Pape & Melchers (2013)

The beams rested on steel supports at each end that closely replicated the original field bearing plates. The test result showed a significant loss of ultimate capacity in the corroded prestressed concrete beams, as shown in Figure 2-25. Note that the testing program for beam A occurred over a period of 2 days, and the test intermission is identified in the figure as “Period of unloading and loading”. The load to initiate progressive collapse, the maximum capacity and the deformation capacity at ultimate capacity all underwent a reduction, in an approximately linear fashion, as the degree of corrosion in the tendons increased.

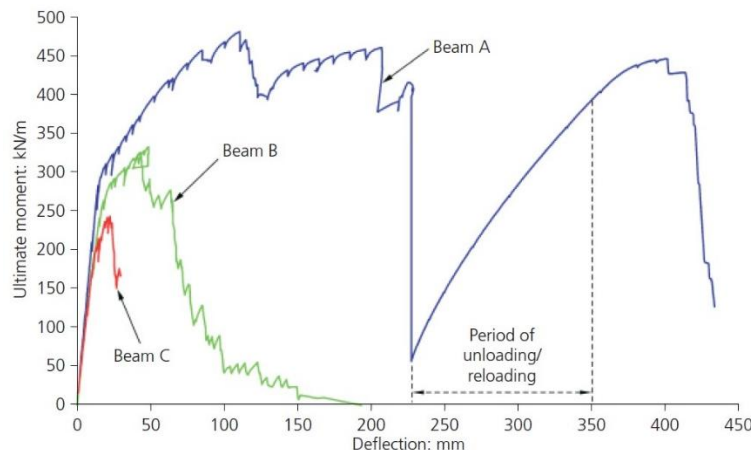


Figure 2-25 Moment-deflection curves for beams A, B and C, as tested by Pape & Melchers (2013)

Jeon et al. (2020) investigated the behaviour of four simply-supported prestressed concrete elements under four-point bending tests. The post-tensioned beams had a section of 150 mm x 220



mm x 2,000 mm and were reinforced with a single seven-wire-strand having a diameter of 15.2 mm. The dimensions and details of the four specimens are presented in Figure 2-26.

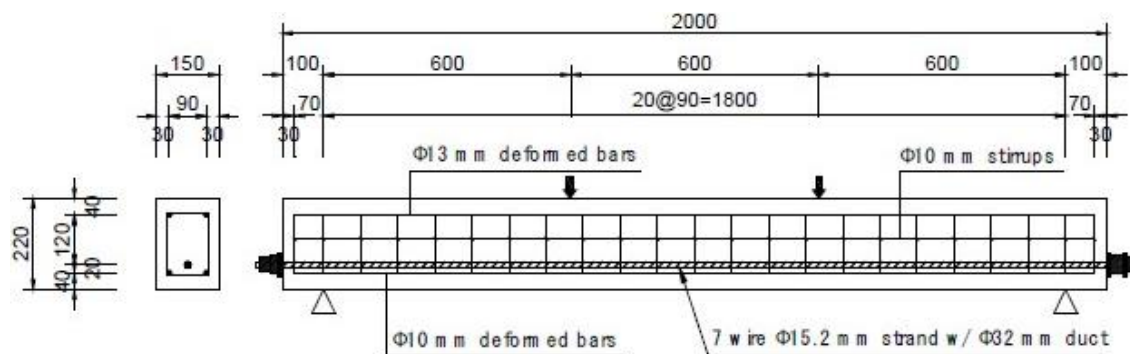


Figure 2-26 Prestressed concrete beam specimen reinforcement details and geometry tested by Jeon et al. (2020)

The prestressing strands were first artificially corroded through exposure to sodium chloride for several months and cyclic humid and dry conditions. Before placing the strands inside the concrete beams, the section loss for each strand was determined. Details on the location and extent of corrosion for each beam are highlighted in Table 2-2.

Table 2-2 Details of beams tested by Jeon et al. (2020)

| Beam ID | Location of corrosion | Section loss (strand) (%) | Section loss (most corroded wire) (%) | Effective prestressing force (kN) |
|---------|-----------------------|---------------------------|---------------------------------------|-----------------------------------|
| RB      | -                     | 0                         | 0                                     | 160                               |
| QB      | 1/4                   | 4.82                      | 21.56                                 |                                   |
| MB1     | 1/2                   | 6.67                      | 22.24                                 |                                   |
| MB2     | 1/2                   | 7.51                      | 18.89                                 |                                   |

For beams with localized corrosion near supports, the effects of corrosion showed limited influence on the strength of the beams. This is attributed to the adequate bond present between the strand and the surrounding grout. In addition, the elements failed in compressive concrete crushing after one-wire fracture. However, prestressed concrete beams with strand corrosion located at the midspan displayed strength decreases of 10 – 17%, and displacement decreases of 12 – 13%. The elements also failed in compressive concrete crushing after two-wire fractures. Hence, the corrosion of strands at the location of maximum moment, under the given test set-up, significantly influences the ultimate capacity and behaviour of a prestressed concrete beam. Figure 2-27 shows

the load-displacement curves for the control and corroded samples. Table 2-3 summarizes the findings from this study.

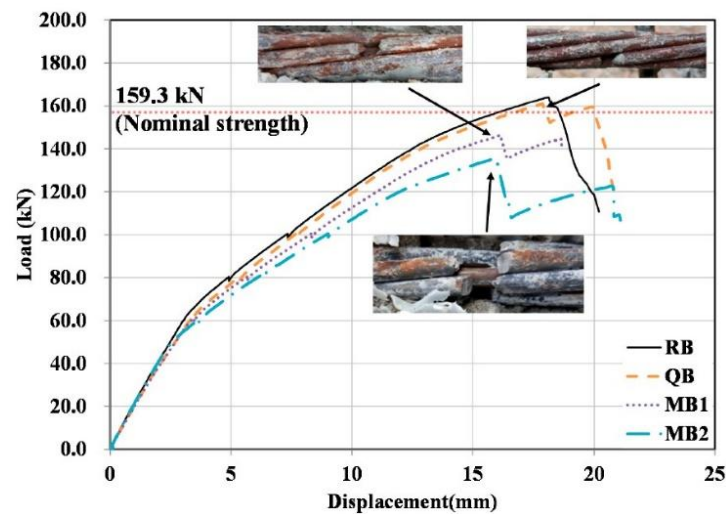


Figure 2-27 Load-displacement curves for samples tested by Jeon et al. (2020)

Table 2-3 Test results of experimental study by Jeon et al. (2020)

| Beam ID    | Maximum load (kN) | Maximum displacement (mm) | Failure mode                                           |
|------------|-------------------|---------------------------|--------------------------------------------------------|
| <b>RB</b>  | 163.9             | 18.40                     | Compressive concrete crushing                          |
| <b>QB</b>  | 161.0             | 17.89                     | Compressive concrete crushing after one wire fracture  |
| <b>MB1</b> | 146.4             | 16.14                     | Compressive concrete crushing after two wire fractures |
| <b>MB2</b> | 135.6             | 15.98                     | Compressive concrete crushing after two wire fractures |

Jeon & Shim (2020) studied the impact of corroded internal prestressing strands on the flexural behaviour of post-tensioned concrete beams by testing a total of five specimens with corroded internal strands having varying degrees of corrosion. The specimens were fabricated with dimensions 380 mm x 280 mm x 3,100 mm, as shown in Figure 2-28.

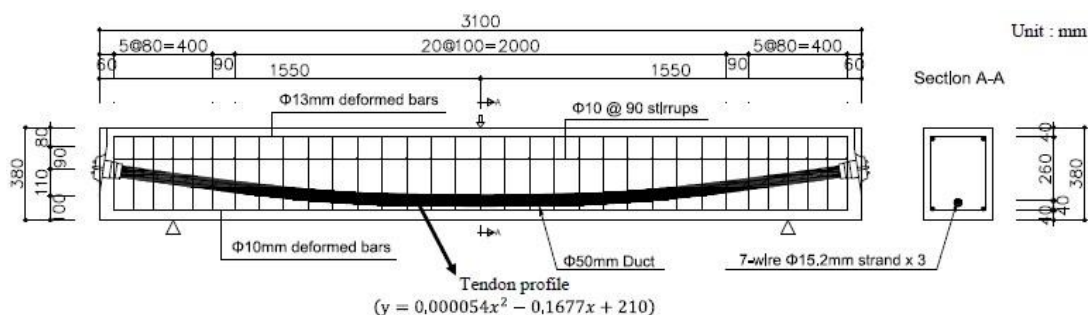


Figure 2-28 Prestressed concrete beam specimen details tested by Jeon & Shim (2020)

The beams were reinforced with 2-13 mm diameter top reinforcement, 2-10 mm diameter bottom reinforcement, and 3-15.2 mm diameter strands, each having a nominal area of 138.7 mm<sup>2</sup>. Once the beams were cast, the strands were artificially corroded at different locations along the strand length prior to placing them in the concrete elements. Different levels of corrosion were achieved by setting a random corrosion induction time. Subsequently, the cross-sectional losses of the tendons were measured, and the strands were then placed in the concrete elements. Finally, a prestressing force was applied to each tendon, and the grout was injected.

Table 2-4 highlights the corrosion details of the tested specimens.

Table 2-4 Details of beams tested by Jeon & Shim (2020)

| Beam ID | Number of corroded strands | Location of corrosion | Wire-unit section loss (%) | Strand-unit section loss (%) | Tendon-unit section loss (%) |
|---------|----------------------------|-----------------------|----------------------------|------------------------------|------------------------------|
| CB1     | 2                          | Midspan               | 30.22                      | 14.26                        | 9.49                         |
| CB2     | 3                          | N/A                   | 22.54                      | 9.51                         | 8.84                         |
| CB3     | 2                          | 300 mm from support   | 29.02                      | 10.71                        | 6.89                         |
| CB4     | 3                          | N/A                   | 19.55                      | 6.94                         | 6.80                         |
| RB5     | 0                          | -                     | -                          | -                            | -                            |

The simply-supported prestressed concrete beams were tested under a three-point bending test.

Figure 2-29 shows the load-deflection curves of the control and corroded specimens.

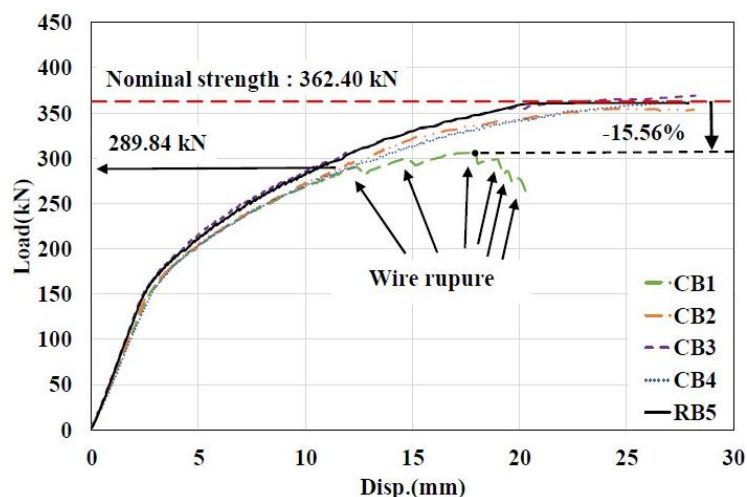


Figure 2-29 Load-displacement curves for samples tested by Jeon & Shim (2020)

The results from this study showed that the ultimate capacity decreased for beams that suffered from corrosion at midspan. Beams with a 10% tendon section loss at midspan exhibited a 16% reduction in ultimate strength after rupture of the third wire. However, there was no effect on the ultimate flexural load of beams with corroded strands located near the supports.

Table 2-5 summarizes the obtained results.

Table 2-5 Test results of experimental study by Jeon & Shim (2020)

| Beam ID | Cracking load (kN) | Maximum load (kN)               | Failure mode                                             |
|---------|--------------------|---------------------------------|----------------------------------------------------------|
| CB1     | 130                | 289.84 (when first wire failed) | Compressive concrete crushing after rupture of six wires |
| CB2     | 140                | 354.27                          | Compressive concrete crushing                            |
| CB3     | 140                | 367.75                          | Compressive concrete crushing                            |
| CB4     | 140                | 362.11                          | Compressive concrete crushing                            |
| RB5     | 140                | 361.13                          | Compressive concrete crushing                            |

## 2.5.2 Shear behaviour

Studies performed by Coronelli & Gambarova (2004) showed that more shear cracks develop in corroded members. This is due to the reduction in section width and stirrup cross-section. The experimental studies performed by Rodriguez et al. (1997) also showed a shift in the failure mode from flexural to shear due to the pitting effects in the corroded stirrups.

Wang et al. (2020) experimentally studied the shear behaviour of corroded post-tensioned prestressed concrete beams with full or insufficient grouting. The study consisted of 8 beams divided into two groups: 4 beams were fully grouted (Group F), while the remaining four beams were insufficiently grouted (Group I). All beams were 3,000 mm long and were designed with a cross section of 160 mm x 450 mm and a single 15.2 mm diameter seven-wire curved strand with a yield and ultimate strength of 1,412.4 and 1,876.3 MPa, respectively. Once the strand tensioning and grouting process were completed, the beams were subjected to electrochemically accelerated corrosion. Figure 2-30 displays the reinforcement details and geometry of the tested beams.

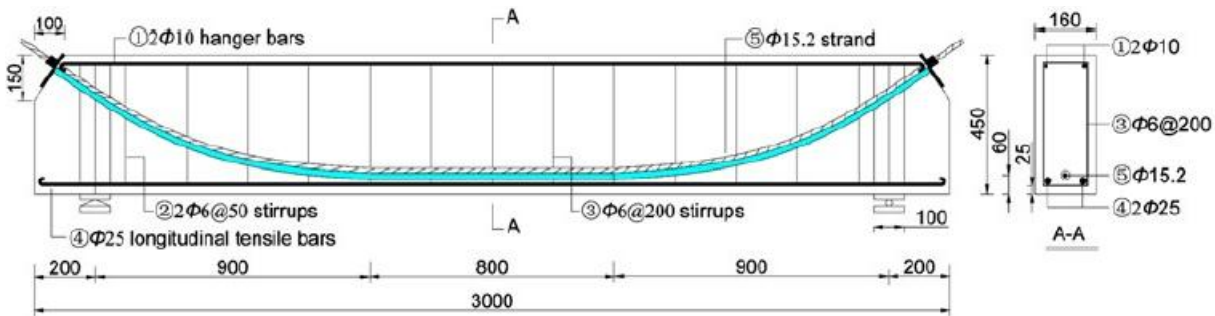


Figure 2-30 Reinforcement details and geometry of beams tested by Wang et al. (2020)

The simply-supported beams were then tested under four-point bending. Once the destructive testing was completed and the strands were exposed, the cross-sectional loss of the strands due to corrosion was measured. The load-midspan deflection and load-maximum diagonal crack width curves are highlighted in Figure 2-31 and Figure 2-32.

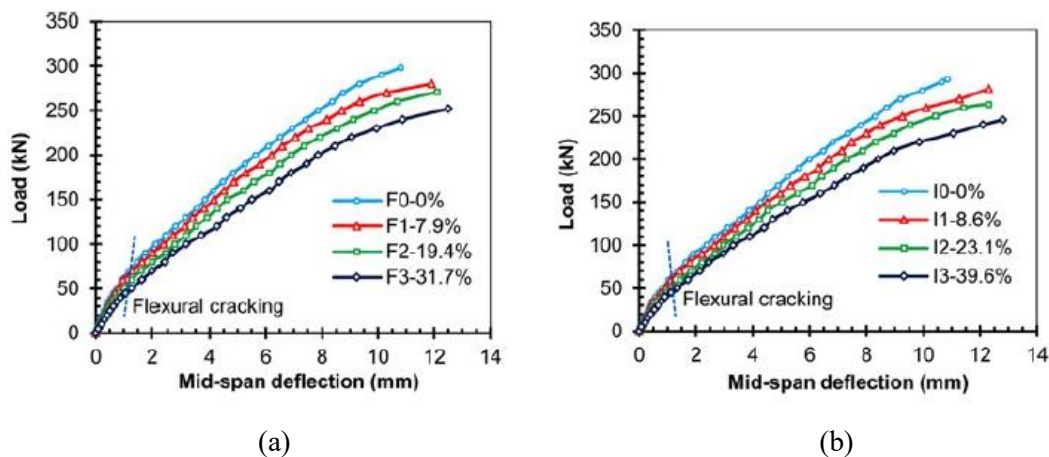


Figure 2-31 Load mid-span deflection response of: a) Group F and b) Group I with various degrees of corrosion tested by Wang et al. (2020)

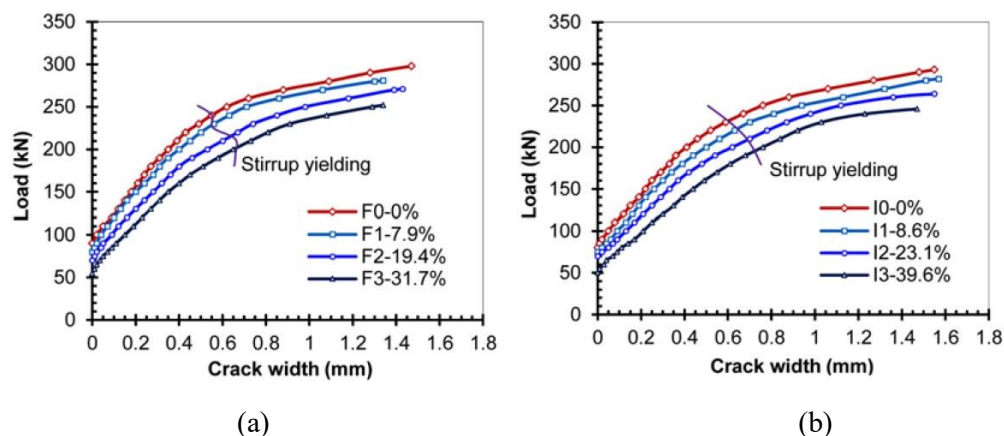


Figure 2-32 Load-maximum diagonal crack width curves of: a) Group F and b) Group I tested by Wang et al. (2020)

The results indicated that for both groups, the strand corrosion accelerated the development of diagonal cracking which was followed by the premature formation of a critical diagonal crack. For all beams, the load-maximum diagonal crack width curves contained an initial linear growth phase, due to the resistance of the stirrups to the propagation of the crack width, followed by a non-linear growth phase, as the stirrups began to experience yielding. The strand corrosion and grouting defects affected the propagation rate of both phases, as highlighted in Figure 2-32. To elaborate, it can be seen that the strand corrosion accelerated the development of diagonal crack widths, which ultimately resulted in the premature formation of a critical diagonal crack, regardless of whether the grouting was full or insufficient. As for the effects of insufficient grouting, when comparing the load-maximum diagonal crack width response of the uncorroded beams I0 and F0 shown in Figure 2-32, it is clear that the propagation rate of I0 was larger than that of F0. Furthermore, the load-maximum diagonal crack width of I0 was nearly the same as that of the corroded beam F1, which experienced 7.9% corrosion loss, while the diagonal crack propagation of I1 was faster than that of F1. These findings emphasize how grouting defects can impact the continued propagation of diagonal cracks.

The ultimate shear strength for both groups was also found to decrease with increasing corrosion degrees. The failure mode for all beams was a shear-compression failure mode, except for the most corroded beam with insufficient grouting, where the failure mode was characterized by wire rupture. In addition, the tests showed that grouting defects located within the shear span decrease the number of diagonal cracks formed prior to failure and accelerate the crack propagation.

Insufficient grouting can also decrease the beam post-cracking stiffness. A summary table of the obtained results is provided in Table 2-6.

Table 2-6 Test results of the experimental study by Wang et al. (2020)

| Beam number                         | Group I: beams with insufficient grout |     |      |      | Group F: beams with full grouting |     |      |      |
|-------------------------------------|----------------------------------------|-----|------|------|-----------------------------------|-----|------|------|
|                                     | I0                                     | I1  | I2   | I3   | F0                                | F1  | F2   | F3   |
| Corrosion duration (days)           | 0                                      | 3   | 6    | 10   | 0                                 | 3   | 6    | 10   |
| Corrosion loss (%)                  | 0                                      | 8.6 | 23.1 | 39.6 | 0                                 | 7.9 | 19.4 | 31.7 |
| Concrete compressive strength (MPa) | 42.2                                   |     |      |      |                                   |     |      |      |
| Flexural cracking strength (kN)     | 67                                     | 61  | 52   | 43   | 71                                | 63  | 54   | 42   |
| Diagonal cracking strength (kN)     | 86                                     | 79  | 68   | 52   | 90                                | 81  | 73   | 56   |
| Ultimate shear strength (kN)        | 293                                    | 282 | 264  | 246  | 298                               | 281 | 271  | 252  |
| Failure mode                        | SC                                     | SC  | SC   | RW   | SC                                | SC  | SC   | SC   |

SC = shear-compression failure; RW = rupture of wires.

Belletti et al. (2020) investigated the shear capacity of 7 naturally corroded prestressed concrete beams and one control beam reported in Franceschini et al. (2022). The mechanical properties of the strands included a yield strength and ultimate strain of 1580 MPa and 0.05, respectively. The beams were all designed without transverse reinforcement.

Following a visual inspection to detect areas damaged by corrosion, the specimens were subjected to four-point bending tests under a simply-supported test setup, followed by a mass loss analysis. During the testing, the Digital Image Correlation (DIC) technique and linear variable displacement transducer (LVDT) were used to measure the displacements and strains. The region of interest for the DIC method was the surface area of the beam located between the supports. Three different levels of corrosion were identified along all strand lengths: level 1 with mass loss < 2% (low level), level 2 with mass loss varying between 2 – 10 % (medium level) and level 3 with mass loss greater than 10% (high level). Figure 2-33 through Figure 2-39 show a) the geometry and details of the loading set-up of the corroded beams prior to testing, where areas of concrete spalling and visible longitudinal cracking are highlighted in red, b) locations of varying corrosion degrees along the beam length and c) an image of the beam post-failure. Note that areas highlighted in blue, green, and red, correspond to corrosion levels 1, 2 and 3, respectively.



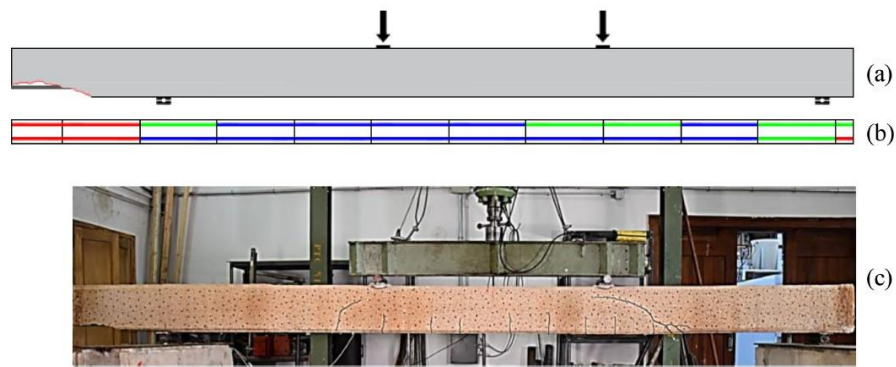


Figure 2-33 Beam PB4P5 tested by Belletti et al. (2020): a) crack pattern map prior to test, b) corrosion level across beam length and c) beam at failure

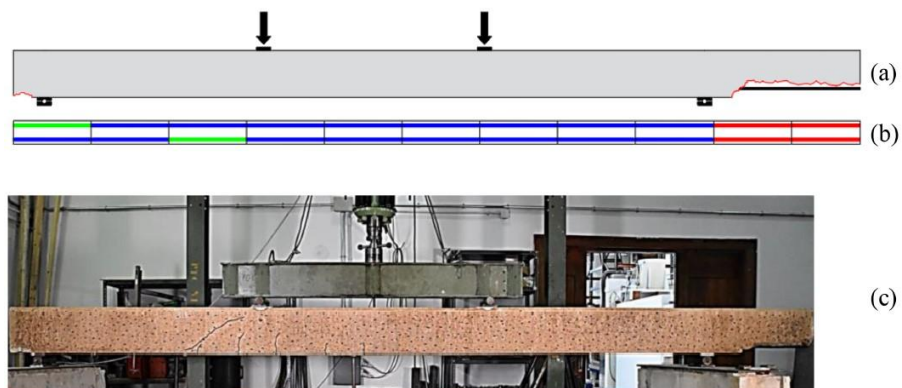


Figure 2-34 Beam PB4P6 tested by Belletti et al. (2020): a) crack pattern map prior to test, b) corrosion level across beam length and c) beam at failure

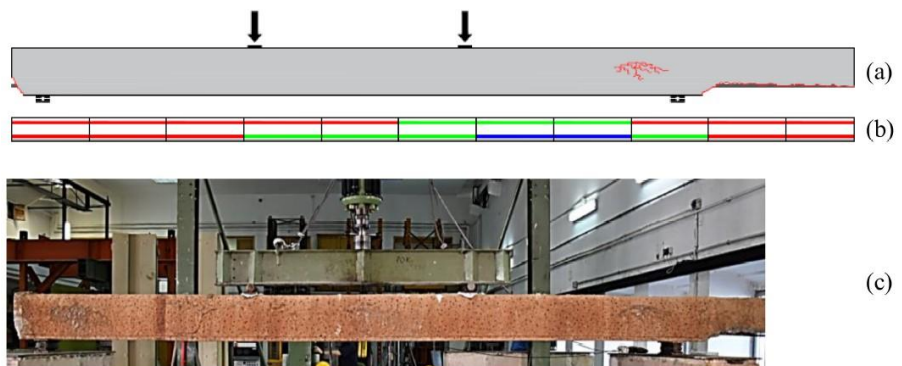


Figure 2-35 Beam PB4P7 tested by Belletti et al. (2020): a) crack pattern map prior to test, b) corrosion level across beam length and c) beam at failure



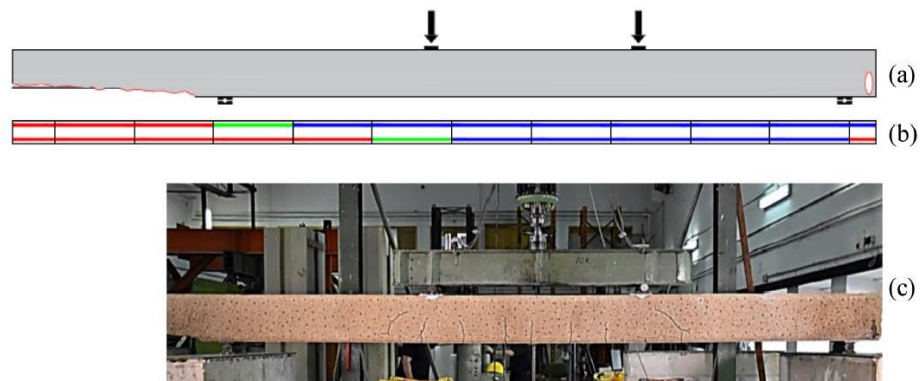


Figure 2-36 Beam PB4P8 tested by Belletti et al. (2020): a) crack pattern map prior to test, b) corrosion level across beam length and c) beam at failure

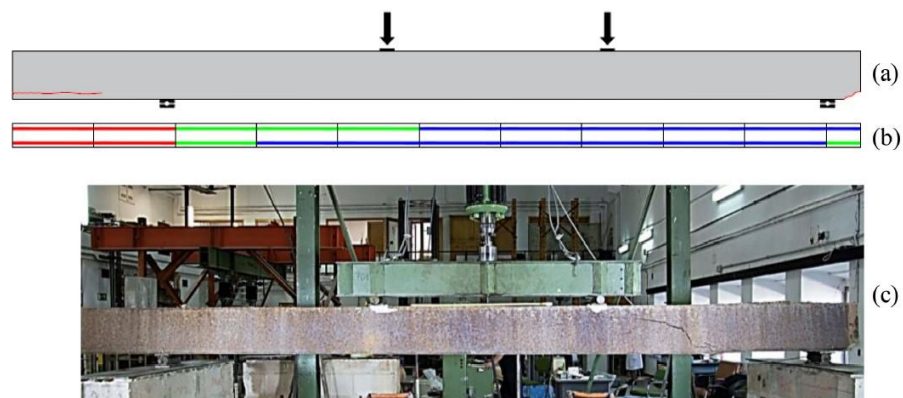


Figure 2-37 Beam PB4P9 tested by Belletti et al. (2020): a) crack pattern map prior to test, b) corrosion level across beam length and c) beam at failure

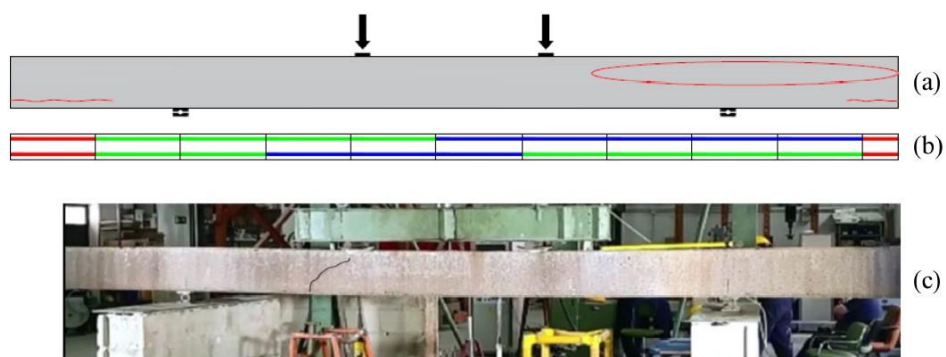


Figure 2-38 Beam PB4P13 tested by Belletti et al. (2020): a) crack pattern map prior to test, b) corrosion level across beam length and c) beam at failure

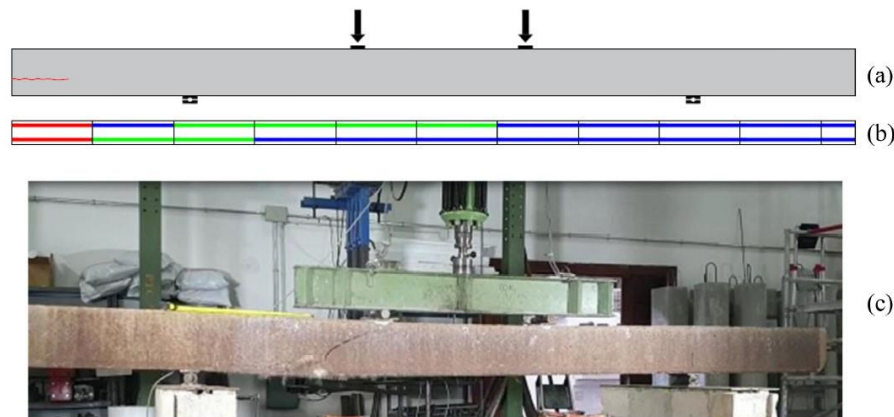


Figure 2-39 Beam PB4P14 tested by Belletti et al. (2020): a) crack pattern map prior to test, b) corrosion level across beam length and c) beam at failure

The four-point bending test results highlighted the effects of corrosion on the residual capacity of prestressed concrete beams, with shear resistance and ductility decreasing for beams affected by corrosion. Two failure modes were observed: shear I and shear II. Shear I failure corresponds to the formation of vertical bending cracks along the beam soffit in locations between the loading points (where a maximum constant moment region occurs). Additional diagonal shear cracks also developed from the bottom to the top of the beam in the shear spans, where regions of constant shear forces occur. Shear II failure corresponds to the formation of a single diagonal shear crack that occurs in the more damaged shear span. No vertical cracking is observed with this type of failure. Figure 2-40 and Table 2-7 provide the load-deflection curves and a summary of the experimental results, respectively. The load-deflection curves are marked to identify load values corresponding to the contour plots of maximum principal strains.

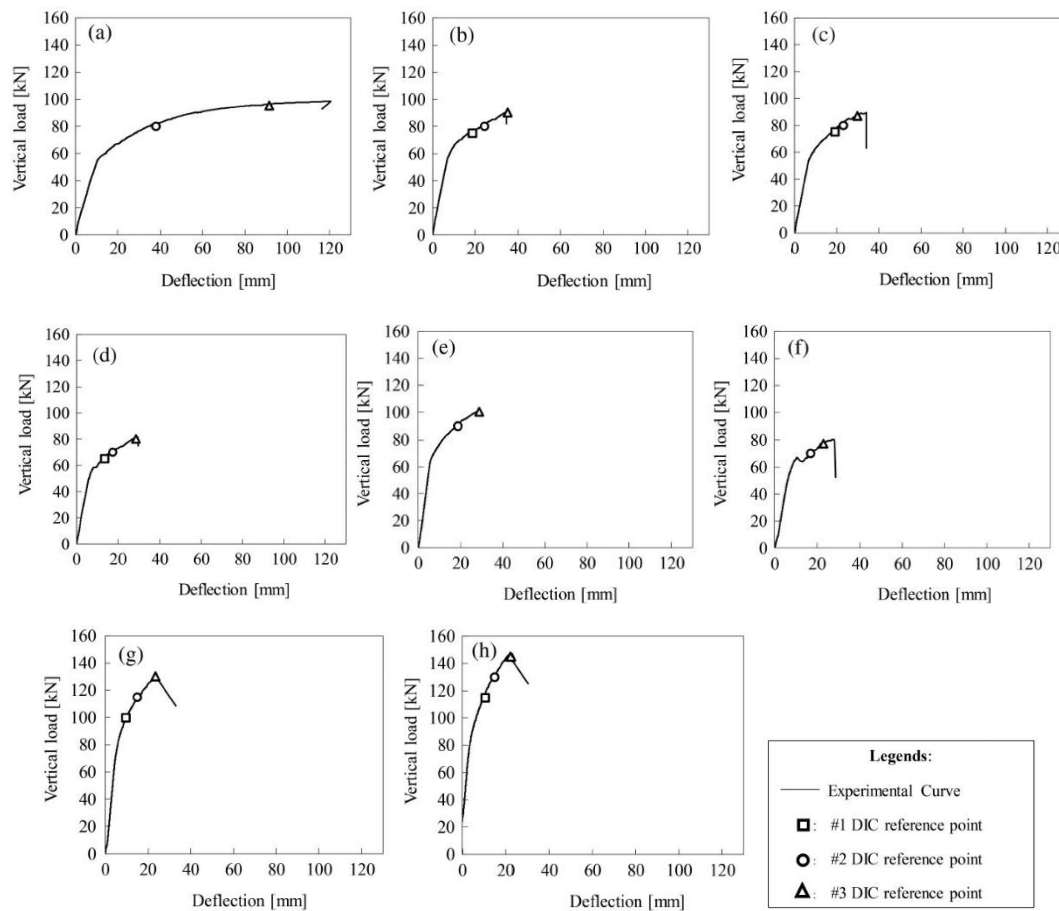


Figure 2-40 Load-deflection curves of beams a) PBN4P2, b) PB4P5, c) PB4P6, d) PB4P7, e) PB4P8, f) PB4P9, g) PB4P13 and h) PB4P14 tested by Belletti et al. (2020)

Table 2-7 Test results of experimental study by Belletti et al. (2020)

| Beam ID                 | Shear span-to-depth ratio | Ultimate load (kN) | Failure mode |
|-------------------------|---------------------------|--------------------|--------------|
| PBN4P2 (reference beam) | 6.7                       | 98.47              | Shear I      |
| PB4P5                   | 5.7                       | 90.21              | Shear I      |
| PB4P6                   | 5.7                       | 89.39              | Shear I      |
| PB4P7                   | 5.5                       | 81.74              | Shear II     |
| PB4P8                   | 5.2                       | 100.06             | Shear I      |
| PB4P9                   | 5.4                       | 80                 | Shear II     |
| PB4P13                  | 4.3                       | 127.45             | Shear I      |
| PB4P14                  | 4.1                       | 145.90             | Shear I      |

Based on the results, for beams with similar shear span-to-depth ratios, the shear resistance decreased as the corrosion levels increased. The corrosion also caused strain localizations and premature failure modes. It is also evident that shear I failure was more ductile than shear II, due

to strand yielding in shear I. Shear failures I and II were found to be independent of the shear span-to-depth ratios. For beams suffering from severe corrosion, higher diagonal crack opening widths were developed, which then negatively affected the aggregate interlock mechanism. Figure 2-40-a) shows that the ductility of the uncorroded beam is much higher than that of the corroded beams due to the absence of corrosion-induced stress localizations. The deformed parabolic shape of the beams under the applied loads were plotted using the vertical displacements obtained from the DIC analyses. The deformed shapes of specimens PBN4P2, PB4P6, PB4P8, PB4P13 and PB4P14 were generally parabolic and symmetric with respect to the mid-span, where maximum deflection was observed. However, specimens PB4P5, PB4P7 and PB4P9 had an unsymmetric profile and a strong displacement localized where diagonal shear cracks occurred, which could be attributed to localized strand debonding, leading to a localized stiffness reduction.

## **2.6 Assessment of bridges and bridge elements**

Routine visual inspections of bridges offer a convenient method for assessing their overall condition, safety, and performance. They also allow the identification of any major damage or failure of a structural component (Ministry of Transportation of Ontario, 2008). However, a visual inspection does not offer information on internal defects within a structural component, such as poor concrete cover, delamination, and reinforcement corrosion, which may eventually affect the integrity of the entire structure (Public Works and Government Services Canada, 2010). Traditionally, the condition assessment of concrete structures has been performed by coring hardened specimens for standard tests. However, this approach does not provide data on the in-situ properties of concrete, which is critical for the condition assessment of structures. In addition, cores may cause local damage and limit the obtained data to the core location.

### **2.6.1 Bridge inspection manuals in Canada and abroad**

There currently exists a limited number of standards and inspection manuals for the detailed and in-depth condition and structural assessment of prestressed concrete bridges. In Canada, the Ontario Structure Inspection Manual (OSIM) (Ministry of Transportation of Ontario, 2008), developed by the province of Ontario, is the most widely used bridge inspection manual in Canada (Ministry of Transportation of Ontario, 2008). Public Works and Government Services Canada, which owns many Canadian federal bridges, uses the Bridge Inspection Manual, derived from the OSIM, for the inspection of its bridges (Public Works and Government Services Canada, 2010).

The OSIM provides general details, requirements and qualitative descriptions for the inspection procedure and condition assessment of bridge structures and their components. General guidelines, procedures, and NDT methods are also provided for the case when there is a need to perform further structural investigations.

In 2013, the Federal Highway Administration of the U.S. Department of Transportation published a technical advisory to give guidance to bridge owners on assessing and managing the long-term performance of grouted post-tensioned concrete bridges that suffer from chloride-induced corrosion. This advisory came into effect after the 2010 discovery of post-tensioning grouts with elevated chloride levels in a post-tensioned concrete bridge in Texas. This document only considers the assessment of grouted ducts to determine their associated levels of chlorides. Guidance on determining the level of protection required for the tendons is also provided. The corrosion risk level can then be determined based on varying combinations of chloride concentration and post-tensioning protection levels. The document then recommends performing calculations to determine the residual ductility of the system and to ensure detectable cracking prior to strength loss and failure. The ductility check is paramount, as the post-tensioning force reduces due to corrosion. The bridge should also be able to transmit forces between its structural elements. Finally, based on all the determined parameters, recommended management follow-up actions are provided. In addition, a qualitative inspection procedure for post-tensioned bridges is provided (Federal Highway Administration - U.S. Department of Transportation, 2013).

Most recently, in 2021, DEKRA Industrial, with the assistance of the Norwegian Road Authorities, issued an inspection manual describing the various recommended methods for inspecting post-tensioned concrete bridges. The intent is to provide an in-depth tool for the planning, execution and evaluation of inspections using NDT methods. However, since multiple NDT methods for determining the material and strength characteristics of concrete have been well established, NDT methods described in the inspection manual primarily focus on the identification of hidden defects, such as air-filled or water-filled voids and poor grouting of mortar in tendon ducts, in order to evaluate the potential corrosion or wire rupture (DEKRA Industrial, 2021). The inspection manual presents a flowchart for the inspection of post-tensioned concrete structures and is shown in Figure 2-41.

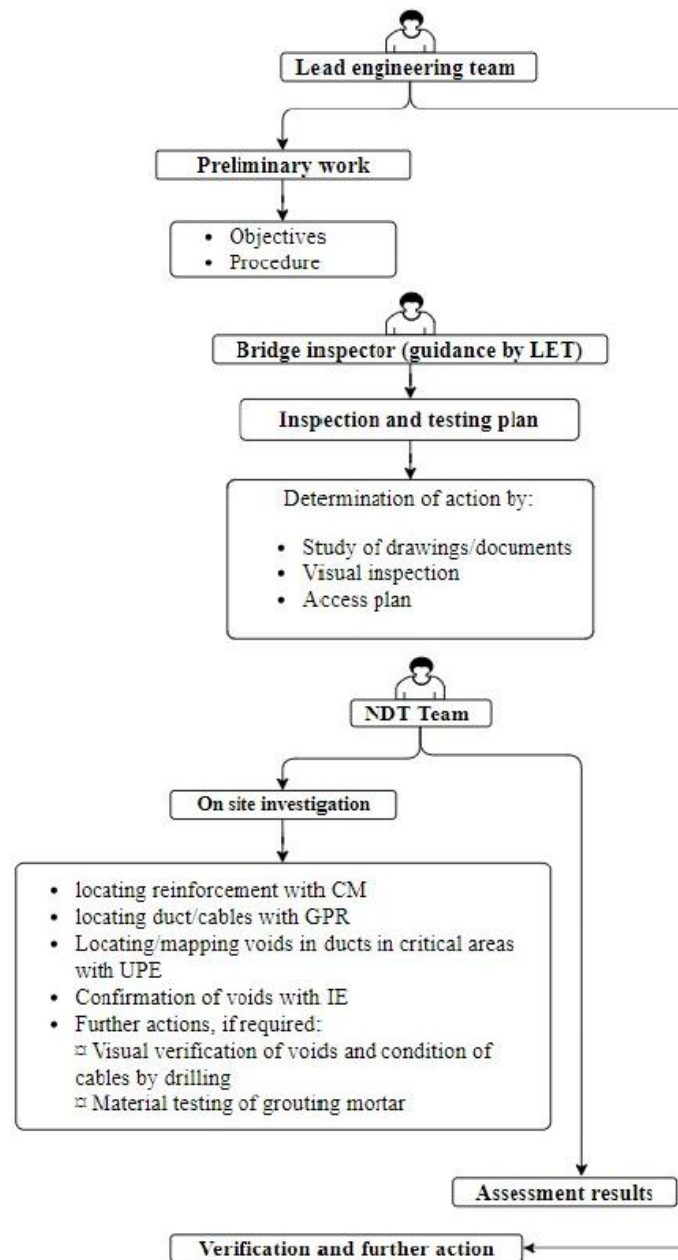


Figure 2-41 Flowchart for the inspection of post-tensioned concrete structures (reproduced from DEKRA Industrial, 2021)

The document also presents various common defects in post-tensioning tendons of bridge elements (Figure 2-42). These weak points may introduce chloride water into the tendon ducts and cause corrosion.

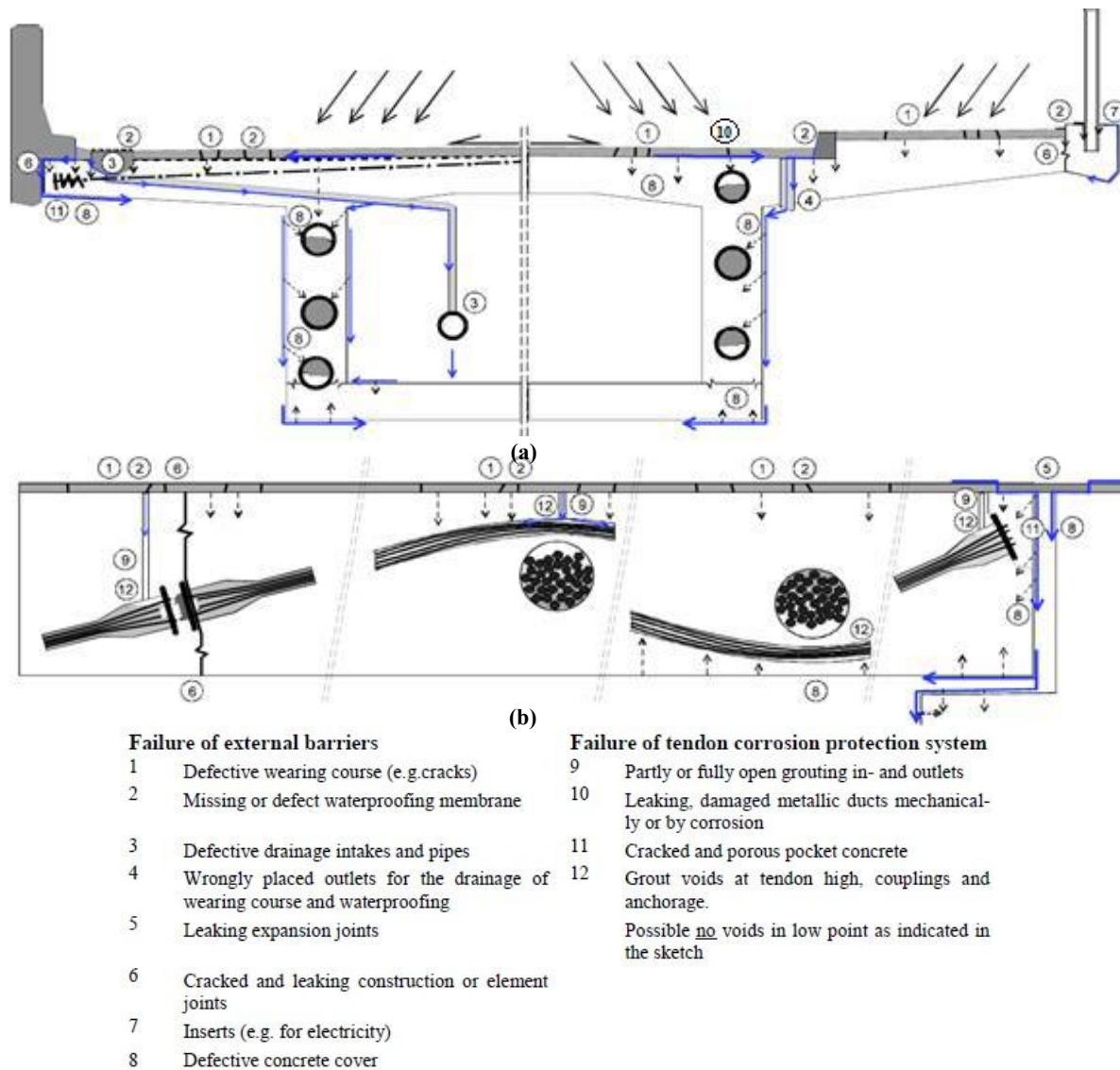


Figure 2-42 Hazard scenarios for post-tensioning tendons in a typical concrete bridge box girder. Locations of possible weak points that may introduce chloride water and cause corrosion are indicated in the a) cross-sectional view, b) longitudinal section view (reproduced from DEKRA Industrial, 2021)

The inspection manual provides detailed descriptions and testing procedures for the grout assessment using NDT methods. Capabilities and limitations for each technique are also presented.

### **2.6.2 Non-destructive testing methods**

Non-destructive testing (NDT) methods have become increasingly popular when examining materials and structures for defects without compromising their structural integrity. The growth in the application of NDT methods is attributed to the following factors (ACI Committee 228, 2013):

- Improvements in technological hardware and software used to collect and analyze data.
- Economic advantages when assessing multiple concrete structures.
- Rapid and thorough assessments of existing concrete structures.
- Quality assurance for existing structures and concrete repairs.

NDT methods have been proven to provide crucial information for the condition assessment and structural performance of prestressed concrete elements. Such information includes member dimensions, areas of concrete cracking and delamination, reinforcement corrosion activity, grouting defects within prestressing strands, extent, and degree of damage from deleterious mechanisms, and concrete strength. Tests for obtaining this data are typically conducted prior to rehabilitation to select the most effective method of rehabilitation. The following set of NDT methods has been proven to provide valuable information regarding the in-place properties and integrity of concrete and prestressing steel strands in PT members.

#### **2.6.2.1 Schmidt hammer**

The Schmidt hammer, also known as the rebound or Swiss hammer, is a testing apparatus that evaluates the hardness, or elastic properties, of concrete, as per ASTM C805 (ASTM C805, 2018). This device uses the energy absorbed during its interaction with the concrete test specimen to assess the element's strength and stiffness. Concrete with low strength and stiffness properties will absorb more energy and will thus yield low rebound numbers when compared to concrete with high strength and stiffness properties (American Concrete Institute (ACI 228), 2013). Figure 2-44 illustrates the operation of the Schmidt hammer. A typical curve showing the correlation between the concrete strength and rebound number is shown in Figure 2-43.



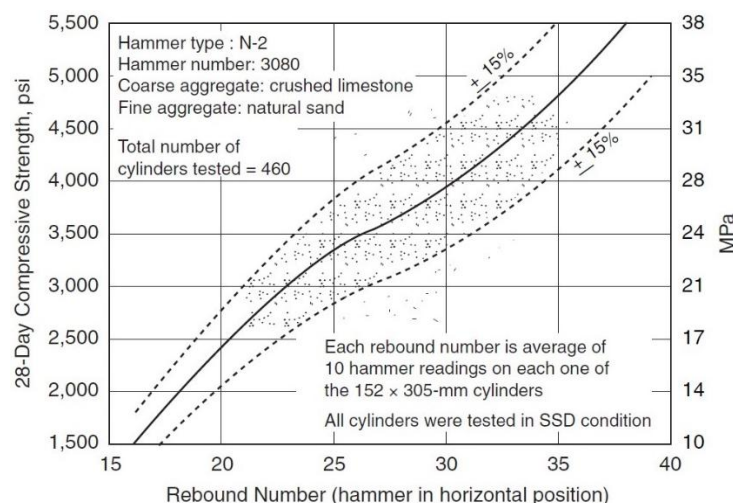


Figure 2-43 Relationship between 28-day compressive strength and rebound number using Type N-2 hammer on concrete with limestone aggregate (reproduced from Malhotra & Carino, 2003)

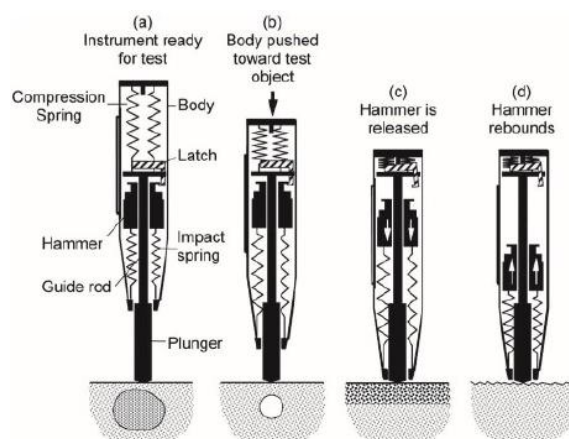


Figure 2-44 Illustration for the operation of the Schmidt hammer (reproduced from ACI 228, 2013)

### 2.6.2.2 Ultrasonic pulse velocity

The ultrasonic pulse velocity (UPV) method measures the propagation velocity of longitudinal stress-wave pulses over a known path length and through the concrete member according to ASTM C597 (ASTM C597, 2016). This technique determines the uniformity and quality of concrete by utilizing a pulser that sends a short-duration and high-voltage signal to the device's transducer placed at the opposite concrete surface. This will then cause the transducer to vibrate at its resonant frequency, generating a stress-wave pulse in the concrete member. The test requires a viscous fluid along the pulser-concrete and transducer-concrete interface, which helps transfer the vibrations to the concrete. The device measures the elapsed travel time of the signal from the start of the electric

pulse to the time the pulse is received. The pulse velocity through the concrete can then be obtained by dividing the direct path length by the travel time (ACI 228, 2013). Figure 2-45 displays a schematic of the UPV apparatus and highlights its operation.

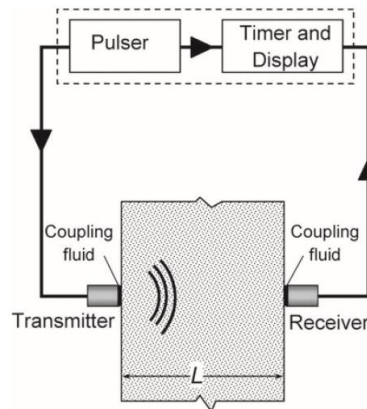


Figure 2-45 Schematic of the UPV apparatus (reproduced from ACI 228, 2013)

### 2.6.2.3 Combined sonic rebound method

The combined Sonic Rebound (SonReb) method determines the concrete compressive strength. This method combines the results from both the Schmidt rebound hammer and UPV and is the most common non-destructive approach for concrete strength assessment. It has resulted in strength relationships with higher correlation coefficients than when these methods are used alone (ACI 228, 2013). Throughout the years, many empirical formulations have been established to determine the concrete compressive strength with the SonReb method. Seventeen formulations, developed between 1979 and 2009 and discussed by Cristofaro et al. (2020), that represent an important sample of the predictive models proposed so far have been evaluated. The formulations are listed in Table 2-8. They were developed by utilizing different concrete cubes and cylinders prepared in the laboratory.

Table 2-8 Formulations to determine concrete compressive strength using the SonReb method (Cristofaro et al., 2020a)

| Author                 | Year | Formulation                                                | Units                    |
|------------------------|------|------------------------------------------------------------|--------------------------|
| Bellander              | 1979 | $R_{cub} = 0.00082(I_r)^3 + 11.03(V_{us}) - 32.7$          | MPa, km/s                |
| Meynink, Samarin       | 1979 | $f_{cyl} = -24.1 + 1.24(I_r) + 0.058(V_{us})^4$            | MPa, km/s                |
| Giacchetti, Lacquaniti | 1980 | $R_{cub} = 7.695(10^{-11})(I_r)^{1.4} (V_{us})^{2.6}$      | MPa, m/s                 |
| Bocca, Cianfrone       | 1983 | $R_{cub} = 2.765(10^{-9})(I_r)^{1.3114} (V_{us})^{2.487}$  | kg/cm <sup>2</sup> , m/s |
| Samarin, Dhir          | 1984 | $f_{cyl} = -12 + 0.76(I_r) + 0.1(V_{us})^4$                | MPa, km/s                |
| Gasparik               | 1992 | $R_{cub} = 0.0286(I_r)^{1.246} (V_{us})^{1.85}$            | MPa, km/s                |
| RILEM                  | 1993 | $R_{cub} = 9.27(10^{-11})(I_r)^{1.4} (V_{us})^{2.6}$       | MPa, m/s                 |
| Di Leo, Pascale        | 1994 | $R_{cub} = 1.2(10^{-9})(I_r)^{1.058} (V_{us})^{2.446}$     | MPa, m/s                 |
| Arioglu, Koyluglu      | 1996 | $R_{cub} = 0.00153(I_r^3 V_{us}^4)^{0.611}$                | MPa, km/s                |
| Ramyar Kol             | 1996 | $f_{cyl} = -39.57 + 1.532(I_r) + 5.0614(V_{us})$           | MPa, km/s                |
| Kheder                 | 1996 | $R_{cub} = 0.0158(I_r)^{1.1171} (V_{us})^{0.4254}$         | MPa, m/s                 |
| Benoncini, Formichi    | 2003 | $f_{cyl} = 5.9 + 2.712(10^{-15})(I_r)(V_{us})^4$           | MPa, m/s                 |
| Caiaro et al.          | 2003 | $R_{cub} = 1.74(10^{-7})(I_r)^{-0.0674} (V_{us})^{2.36}$   | MPa, m/s                 |
| Del Monte et al.       | 2004 | $R_{cub} = 4.40(10^{-7})(I_r^2 V_{us}^3)^{0.5634}$         | MPa, m/s                 |
| Menditto et al.        | 2004 | $R_{cub} = 4.0(10^{-5})(I_r^{1.88148})(V_{us}^{0.80840})$  | MPa, m/s                 |
| Faella et al.          | 2009 | $R_{cub} = 2.6199(10^{-8})(I_r^{0.5341})(V_{us}^{2.2878})$ | MPa, m/s                 |
| Faella et al.          | 2009 | $R_{cub} = 0.26511(I_r) + 0.01385(V_{us}) - 34.51583$      | MPa, m/s                 |

$R_{cub}$  = concrete cubic strength;  $I_r$  = rebound number;  $V_{us}$  = ultrasonic velocity;  $f_{cyl}$  = concrete cylindrical strength.

#### 2.6.2.4 Ultrasonic pulse echo

The Ultrasonic Pulse Echo (UPE) method, which is based on P-waves, is a technique originally designed for testing metals, requiring access to only one face of the member. According to ASTM E114 (ASTM E114, 2020), this technique is used to evaluate the integrity of the grout within the prestressing tendon ducts by locating areas of voids. In this method, a transmitter sends a stress pulse into an object, which then propagates into the object and reflects off flaws or interfaces. The reflected wave is then captured by the same transducer acting as a receiver. Recent advances in the

UPE technique have introduced the multiple pitch-catch method using an array of transducers. This technique makes use of separate receiving transducers located close to the transmitting transducer so that multiple pitch-catch measurements are recorded sequentially; that is, the transducers are designed to operate as transmitters and receivers in a predefined sequence. The resulting round-trip travel time measurements are then used to reconstruct an internal image portraying the reflecting interfaces (ACI Committee 228, 2013). A schematic illustrating this system is shown in Figure 2-46.

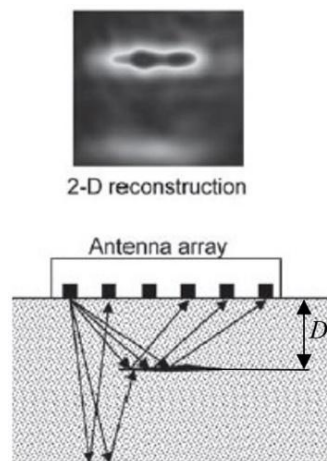


Figure 2-46 Schematic illustration of advanced UPE method (reproduced from ACI 228, 2013)

The advanced UPE utilizes dry point contact transducers that transmit short pulses of low-frequency (less than 150 kHz) S-waves into concrete (ACI Committee 228, 2013). This technology does not require any coupling fluid, and the transducers are designed with a spring system to ensure efficient coupling to irregular surfaces.

#### 2.6.2.5 Impact echo

The Impact Echo (IE) technology makes use of a mechanical impact to generate a stress pulse with greater penetration ability without the need of a bulky transmitting transducer according to ASTM C1383 (ASTM C1383, 2019). However, one disadvantage of the IE over the UPE is that the generated stress pulse is scattered over a certain area, whereas the UPE produces a more focused stress pulse at a point (ACI Committee 228, 2013). The IE technique is used to determine the thickness of concrete elements and may also be used as a supplementary method to determine voids within grouted ducts of post-tensioned concrete members. The IE method introduces waves

into a test object in all directions which results in reflections arriving from multiple directions. An illustration depicting the principle of the IE is shown in Figure 2-47.

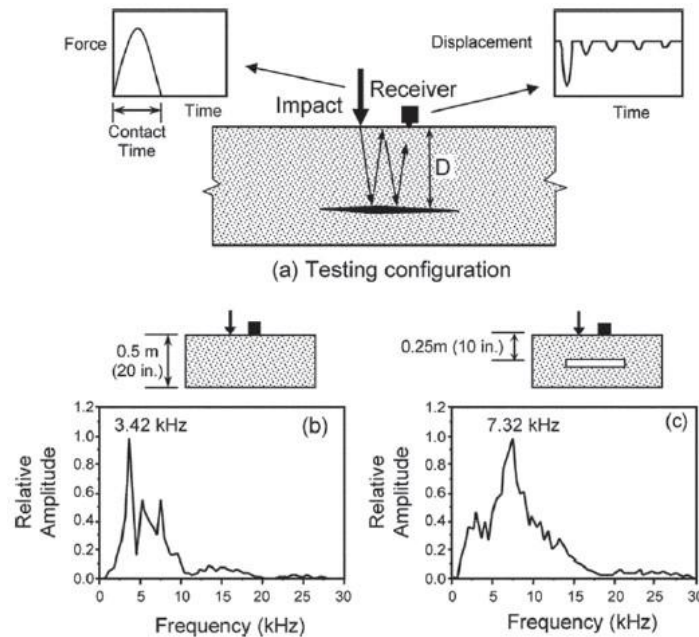


Figure 2-47 a) Schematic of IE method; b) amplitude spectrum for thickness determination of solid slab, c) amplitude spectrum for void detection in slab (reproduced from ACI 228, 2013)

The transient stress pulse introduced into the test object generates P-waves and S-waves that travel into the object along hemispherical wavefronts. A generated surface wave also propagates along the surface and away from the impact point. All waves are then reflected by internal or external interfaces or boundaries, which then arrive at the surface where the impact was introduced to generate displacements. A receiving transducer records these displacements using a data-acquisition device. The data is then analyzed in the frequency domain, which offers a much quicker and simpler approach when compared to a time-domain analysis (ACI Committee 228, 2013). Figure 2-47-b) highlights an example of the amplitude spectrum, where a peak indicates the thickness frequency of an element. Similarly, the peak in the amplitude spectrum shown in Figure 2-47-c) indicates that the reflections are due to the presence of a void within the element.

#### 2.6.2.6 Ground penetrating radar

The Ground Penetrating Radar (GPR) technique is similar to the UPE technique, except that the GPR method makes use of electromagnetic waves, as per ASTM D6432 (ASTM D6432, 2019), such as radio waves and microwaves, instead of stress waves. This technique has been used as a

supplementary method to evaluate the presence of voids in grouted post-tensioning metallic ducts. This technique is often referred to as short-pulse radar or impulse radar, due to the use of short pulses of electromagnetic waves (microwaves) when inspecting shallow media within structural elements. The operating principle of the GPR is highlighted in Figure 2-48. An antenna, attached to a device, transmits short pulses of electromagnetic energy within a certain frequency range. This energy then penetrates the surveyed material and travels through the medium. When an interface between mediums of different dielectric properties is encountered, part of the energy is reflected to the antenna. This then generates an output signal which contains information on the transmitted medium, distance of the reflector, pulse speed within the medium, and the portion of the pulse that was attenuated. A material's dielectric constant is the amount of electrostatic energy stored per unit volume for a unit potential gradient. The analysis of the measured time-domain waveforms allows the determination of the depth of the reflecting wave or interface. This is done through analyzing the measured round-trip travel time and the speed of the electromagnetic wave (ACI Committee 228, 2013).

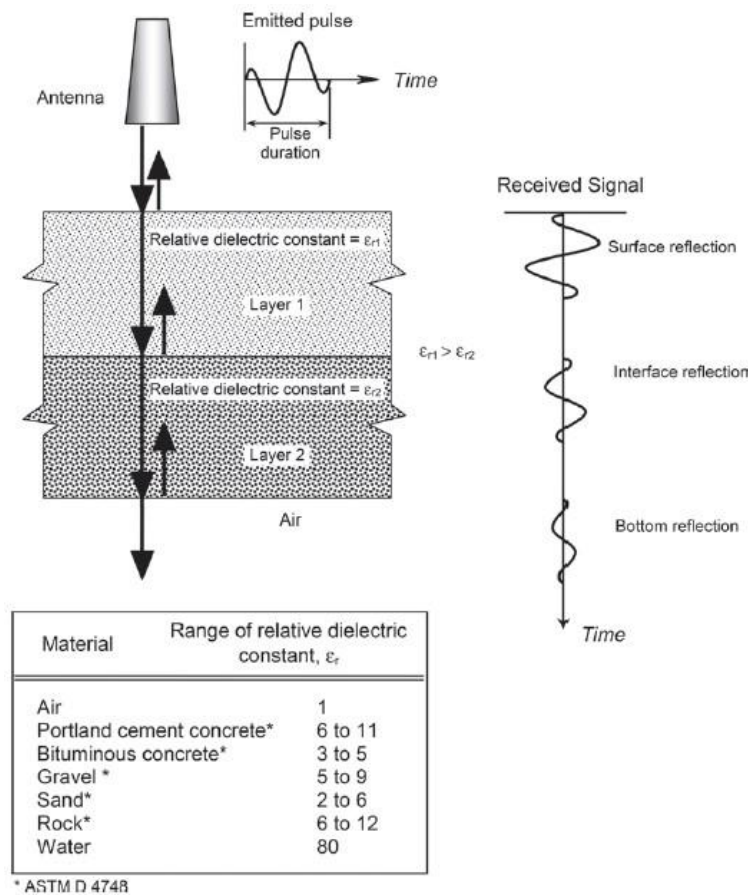


Figure 2-48 Operating principle of GPR highlighting reflections of electromagnetic radiation pulse at interfaces between different materials having various dielectric constants (reproduced from ACI 228, 2013)

### 2.6.2.7 Corrosion measurements

The detection and evaluation of corrosion in embedded steel reinforcement and prestressing tendons is performed by measuring the half-cell potential, corrosion rate, and the concrete surface resistivity. As shown in Figure 2-49, the different electrochemical techniques provide different types of information which can then be used to assess a given concrete element. While potential mapping provides critical information before corrosion becomes visible at the concrete surface, linear polarization and concrete resistivity provide quantitative information on the propagation stage.

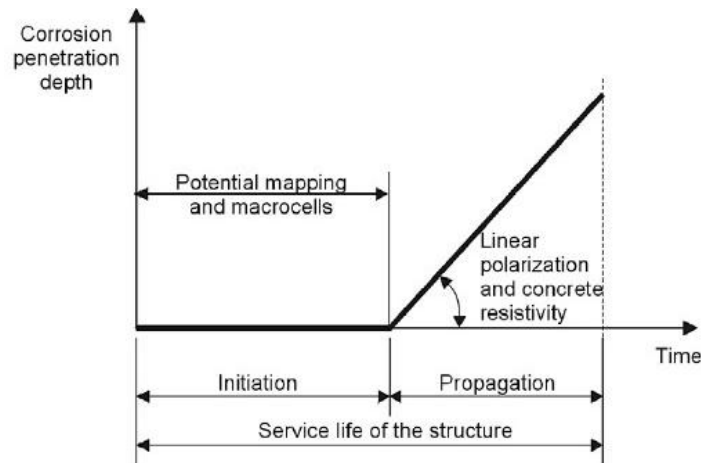


Figure 2-49 Different information obtained from the various electrochemical techniques (reproduced from (Bertolini et al., 2013))

#### 2.6.2.7.1 Half-cell potential

Measuring the electrochemical potentials on embedded prestressing tendon or reinforcing bar relative to a reference half-cell located on the concrete surface allows the detection of corrosion activity. ASTM C876 (ASTM C876, 2015) suggests conducting half-cell potential measurements as an initial evaluation of the probability of corrosion in embedded reinforcement before conducting additional testing. A continuous connection with the steel is crucial in order to yield good results (Bungey et al., 2006). As per ASTM C876 (ASTM C876, 2015), half-cell potential measurements greater than  $-200$  mV/CSE indicate a 10% likelihood of active corrosion, while measurements ranging between  $-200$  mV to  $-350$  mV/CSE are characterized with uncertain likelihood of corrosion, and measurements less than  $-350$  mV/CSE indicate a 90% likelihood of active corrosion. To date, potential mapping is the only standardized NDT for assessing the probability of corrosion of reinforcement embedded in concrete (Bertolini et al., 2013). However, factors such as concrete cover and resistivity affect the readings obtained at the concrete surface. As a result, the interpretation of half-cell potential readings is not a straightforward procedure. Once the corrosion sites are identified by potential mapping, corrosion rate measurements or other laboratory and destructive testing can be performed.

#### 2.6.2.7.2 Corrosion rate

Obtaining quantitative information on the corrosion rate of reinforcement is crucial for structural assessments, service-life predictions and evaluating repair methods for corroding structures. In



general, two different interpretations for “corrosion rate” of reinforcement in concrete exist: average corrosion rate and instantaneous corrosion rate (Bertolini et al., 2013). The average corrosion rate is required when conducting service-life modelling or prognosis studies, as shown by the slope of the propagation curve in Figure 2-49. Once the loss of steel mass or cross-section is determined, the average corrosion rate is then calculated over a long period of time. On the other hand, the instantaneous corrosion rate ( $i_{corr}$ ) may be determined by the polarization resistance (LPR) method or the CEPRA (Connectionless Electrical Pulse Response Analysis) method. As corrosion rate measurements are sensitive to factors such as moisture and temperature, corrections must be considered to allow for reasonable comparisons of measurements obtained at different times during the lifetime of a structure.

Andrade & Alonso (1996) classified corrosion rate measurements into three categories based on their values. Measurements lower than  $0.5 \mu\text{A}/\text{cm}^2$  are considered low, those between  $0.5$  and  $1.0 \mu\text{A}/\text{cm}^2$  are considered moderate, and those exceeding  $1.0 \mu\text{A}/\text{cm}^2$  are classified as high.

#### 2.6.2.7.2.1 Linear polarization resistance method

The Linear Polarization Resistance (LPR) method is based on the assumption of the linear behaviour of the polarization curve near the corrosion potential. It typically requires direct contact with the embedded steel reinforcement. As shown in Figure 2-50, the slope of the curve ( $\Delta E/\Delta I$ ), where  $\Delta E$  is a small variation in potential that is applied to the system and  $\Delta I$  is the resulting current required to achieve this variation, is known as the polarization resistance ‘ $R_p$ ’ of the reinforcement under investigation. The corrosion current density ‘ $i_{corr}$ ’ (instantaneous corrosion rate) can then be calculated using the following equation:

$$i_{corr} = \frac{B}{R_p} \quad \text{Equation 2-26}$$

where  $i_{corr}$  is measured in  $\text{mA}/\text{m}^2$ ,  $R_p$  is measured in  $\Omega \cdot \text{m}^2$ , and  $B$  is the Stern-Geary constant (mV) that is determined experimentally and is related to the anodic and cathodic Tafel slopes. The Tafel slope is a parameter that characterizes the rate of electrochemical reactions that occur at the steel/concrete interface. The value of  $B$  is usually taken as 26 mV for actively corroding reinforcement in concrete and 52 mV for passive reinforcement (Bertolini et al., 2013). The corrosion rate can then be expressed as a penetration rate using the following equation:

$$i_{corr \text{ (penetration rate)}} = K \cdot i_{corr} \quad \text{Equation 2-27}$$

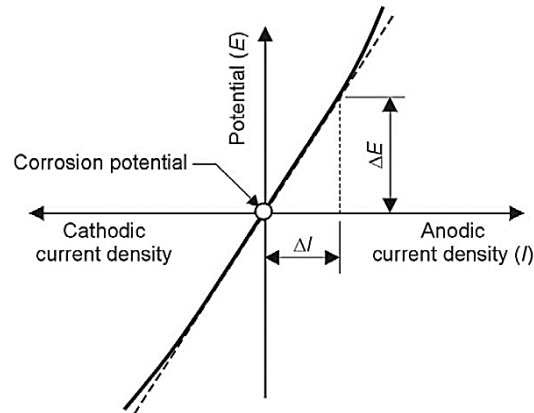


Figure 2-50 Polarization curve in the vicinity of the corrosion potential (reproduced from Bertolini et al., 2013)

Where  $i_{corr \text{ (penetration rate)}}$  is expressed in  $\mu\text{m/year}$  and  $K$  is the corrosion rate for iron with an anodic current density of  $1\text{mA/m}^2$  and has a value of  $1.17 \mu\text{m/year}$ . A schematic setup of the device that measures the polarization resistance using the LPR technique is shown in Figure 2-51.

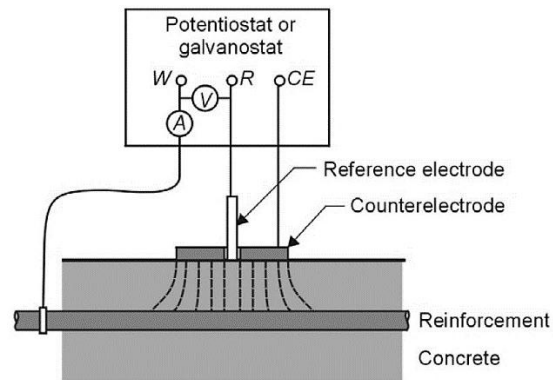


Figure 2-51 Setup for LPR technique (reproduced from Bertolini et al., 2013)

#### 2.6.2.7.2.2 CEPRA method

Measuring the low-frequency impedance of embedded reinforcement in a direct manner is a time-consuming process and can be easily disrupted by noise. Nonetheless, the recently developed CEPRA method involves the application of a narrow current pulse for a short period of time in order to measure the low-frequency behaviour of the embedded reinforcement by making use of the recorded voltage and applied current (Ghods et al., 2016). This method does not require a

direct connection to the embedded reinforcement, which allows for the measurement of corrosion rates without damaging the concrete cover.

The low-frequency impedance can then be used to determine the state of corrosion. The polarization resistance ' $R_p$ ' of the reinforcement is first calculated using the following equation (Ghods et al., 2016):

$$R_p = A \cdot R_{c4} \quad \text{Equation 2-28}$$

where  $R_p$  is the polarization resistance of the reinforcement under investigation,  $A$  is the area of reinforcing steel polarized by the applied current, and  $R_{c4}$  is the charge transfer resistance of reinforcing steel which is a measure of the resistance to electron transfer at the steel/concrete interface as a result of corrosion.

The reinforcement corrosion rate ( $i_{\text{corr}}$ ) is then determined by the device using the recorded voltage, the applied current, and the low-frequency impedance response by implementing Equation 2-26.

### 2.6.2.7.3 Concrete surface resistivity

The ability of the corrosion currents to flow through concrete can be determined by measuring the electrical resistivity of the concrete under investigation. The most widely used technique for resistivity measurements involves utilizing a four-point probe, called a Wenner probe, and placing it directly on the concrete surface. The device consists of four equally spaced point electrodes, as shown in Figure 2-52. The two outermost electrodes induce the AC current, while the two innermost electrodes measure the resulting potential drop of the electric field. The resistance ' $R$ ' and the concrete resistivity ' $\rho$ ' are then calculated using the following equations (Bertolini et al., 2013):

$$R = \frac{\Delta V}{\Delta I} \quad \text{Equation 2-29}$$

$$\rho = 2\pi(a)(R) \quad \text{Equation 2-30}$$

where  $\Delta V$  is the superimposed voltage,  $\Delta I$  is the resulting measured current, and ' $a$ ' is the spacing between the electrodes.

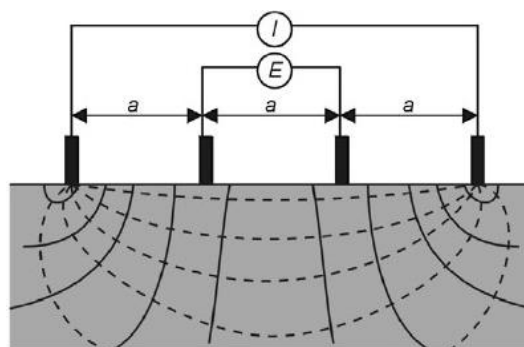


Figure 2-52 Schematic representation of Wenner probe for concrete electrical resistivity measurements (reproduced from Bertolini et al., 2013)

Concrete resistance against electrical current can then be correlated to the resistance against chloride ion movement. Resistivity measurements supplement half-cell potential measurements and affect corrosion rate measurements due to their influence on the flow of ions through concrete. For cases of active corrosion, regions of low resistivity may also be associated with high corrosion rate of reinforcement (ACI Committee 228, 2013). AASHTO T 358-15 (2015) provides a correlation between concrete surface resistivity determined from concrete cylinders and their corresponding resistance to chloride ion penetration (Table 2-9). High chloride ion penetration indicates lower concrete durability. Since AASHTO-T-358 utilizes cylinders for the tests, the ranges indicated in Table 2-9 should be compared to concrete elements with caution, as these elements have variable moisture exposure conditions.

Table 2-9 Correlation of chloride ion penetration to surface resistivity (AASHTO T 358-15, 2015)

| Chloride Ion Penetration | Surface Resistivity Test                                                 |                                                                           |
|--------------------------|--------------------------------------------------------------------------|---------------------------------------------------------------------------|
|                          | 100 mm x 200 mm (4" x 8")<br>Cylinder (kΩ.cm)<br>Probe tip spacing = 1.5 | 150 mm x 300 mm (6" x 12")<br>Cylinder (kΩ.cm)<br>Probe tip spacing = 1.5 |
| <b>High</b>              | < 12                                                                     | < 9.5                                                                     |
| <b>Moderate</b>          | 12 – 21                                                                  | 9.5 – 16.5                                                                |
| <b>Low</b>               | 21 – 37                                                                  | 16.5 – 29                                                                 |
| <b>Very Low</b>          | 37 – 254                                                                 | 29 – 199                                                                  |
| <b>Negligible</b>        | > 254                                                                    | > 199                                                                     |

In addition, resistivity measurements are also sensitive to temperature changes and concrete composition, such as the pore-water composition (alkali and chloride content) (Bertolini et al., 2013). Temperatures and resistivity values are inversely proportional, that is, higher temperatures cause low resistivity values. High resistivity values are generally obtained for dry concrete or

concrete with low water-to-cement ratio. Depending on the concrete composition, a non-universal relationship between the concrete resistivity and corrosion rate of reinforcement after depassivation was found through research (Hornbostel et al., 2013). By implementing a simplified approach, the corrosion rate was found to be inversely proportional to the resistivity.

RILEM (Polder et al., 2000) provides a detailed procedure for concrete resistivity measurements. When using the Wenner probe device, it is recommended to place the device diagonally inside any reinforcement mesh to avoid measurements near reinforcements. Studies of a chloride-contaminated bridge deck showed a correlation between half-cell potential mapping and resistivity measurements (Bertolini et al., 2013). Areas of positive potentials and high resistivities indicate dry and passive zones, while areas of negative potentials and low resistivities indicate wet corroding zones. Areas of risk that will corrode in the future are identified with low resistivities and passive potentials.

### **2.6.3 Material characterization**

The link between corrosion-induced damage at the microscopic and overall structural levels is key in facilitating the understanding of a global damage distress process. As such, material and mechanical testing of extracted core samples can aid in examining a wide range of concrete properties.

Sanchez et al. (2017, 2018) suggest the use of a comprehensive multi-level assessment protocol, consisting of mechanical and microscopic procedures for the diagnostic evaluation of the extent of damage in ageing infrastructure. The authors defined concrete damage as the reduction in stiffness and mechanical properties, in addition to the loss in physical integrity and/or durability. As such, microscopic and mechanical tools, including the SDT, DRI, and compressive and tensile strength tests, were found to be the most suitable when assessing damage in concrete. The SDT was initially developed to quantify the degree of distress in rock specimens by observing the crack density and the cycles of loading/unloading (stress-strain relationship). The technique was then proposed to assess the impact of alkali-silica reactions (ASR) on concrete cylinder samples by applying compressive cyclic loading. Chrisp et al. (1993) suggested using this technique to evaluate the effects of other distress mechanisms in concrete, including frost damage and corrosion. The DRI is a semi-quantitative petrographic procedure used to quantify the degree of damage in concrete affected by internal swelling reactions, specifically ASR (Sanchez et al., 2020).

It makes use of a stereomicroscope (16x magnification), in which crack features are counted within a 1 cm<sup>2</sup> grid that is drawn on the surface of a polished concrete cylinder core section. The counts attributed to each type of petrographic feature are subsequently multiplied by weighting factors aimed at balancing their relative importance to the overall deterioration of the material.

Sanchez et al. (2020) completed a condition assessment of the Robert-Bourassa/Charest (RBC) highway bridge structure, located in Quebec, Canada, through this comprehensive multi-level assessment protocol. The objective was to test the efficiency and reliability of the multi-level approach for the condition assessment of aging infrastructure that is affected by various distress mechanisms. The study consisted of microscopic and mechanical test tools, in addition to conventional mechanical test procedures, that aim at assessing the different material properties of concrete core elements as a function of different degrees of damage. Within this framework, SDT and DRI techniques were both implemented on multiple cores extracted from different samples. Additionally, compressive and tensile strength tests were conducted on cores extracted from different RBC bridge members with the aim of evaluating the structural implications of different damage mechanisms on the members over time. Both tests are commonly used for the condition assessment of ageing structural elements (Sanchez et al., 2017). In addition, they are fundamental in design practice, where compressive strength is used to define the concrete strength,  $f'_c$ , for capacity calculations, and estimate parameters such as modulus of rupture, while tensile strength provides information on cracking behaviour, and ultimate and serviceability limits (CSA Group, 2019c). Corrosion, coupled with other damage mechanisms such as freeze-thaw cycles and alkali-silica reaction, were found at different degrees for distinct concrete members. Strength tests yielded lower results than the design values, which raises concerns on the structural capacity of the bridge to withstand service loads. Furthermore, the results gathered from the study highlighted the disagreement between the external (visual) and internal conditions of damaged concrete structures. While the visual inspection showed little evidence of external damage, findings from the multi-level assessment revealed significant deterioration.

Chloride ingress is dependent on the permeability of concrete, thickness of concrete cover to reinforcement, and chloride exposure. Therefore, when assessing the probability of corrosion initiation, the concrete cover thickness and chloride content level at the reinforcement surface are critical parameters. Osmolska et al. (2020) completed a field investigation of the prestressed

girders from the Dalselv Bridge, located in the northern part of Norway near sea level. Chloride concentrations were measured at different locations for three girders using potentiometric titrations. Although very high chloride concentrations were found at the level of the strands, small corrosion spots were identified on the strands, which could be due to either macrocell corrosion (due to large variation in the surface area of steel reinforcement embedded in the element) that delays strand corrosion, or differences in the quality of the concrete-to-steel interface.

Investigating the crack patterns resulting from reinforcement corrosion may provide information on the crack propagation and extent of corrosion. Sanz Merino (2014) experimentally investigated the crack pattern of artificially corroded concrete specimens by impregnating slices of extracted cores with resin containing fluorescein. The low viscous fluid penetrated into the microcracks and pores of the concrete to allow the analysis of the cracking pattern under UV light. A single main crack was observed along the concrete cover and parallel to the axis of the reinforcement, along with secondary cracks that became visible only under UV light after impregnation (Figure 2-53). Additional tests proved that the impregnation technique does not introduce additional cracking in the specimen.

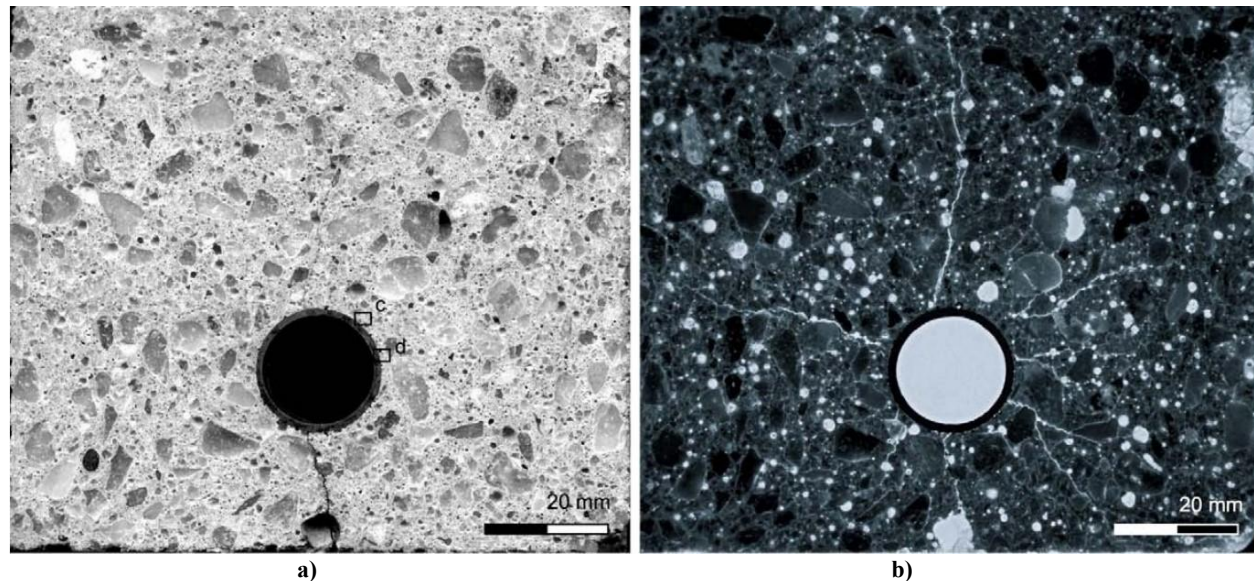


Figure 2-53 Crack pattern in accelerated corrosion tests: a) under naked eye and b) after impregnation with resin containing fluorescein and illuminated under UV light (reproduced from Sanz Merino, 2014)

#### 2.6.4 Case studies

The damage assessment of in-service concrete bridge elements is crucial in ensuring adequate capacity, ductility, and public safety. Various case studies address the condition and structural assessment of in-service concrete bridge elements by use of different NDT techniques.

Oh et al. (2013) evaluated the field application of three NDT methods on an in-service concrete bridge deck that passes over Interstate 66 near Gainesville, Virginia, USA. The concrete bridge deck was selected for the case study, as this element is among the most vulnerable within the transportation infrastructure. The IE, infrared thermography and chain-drag methods were all implemented to detect shallow delamination defects in the deck. The results showed that all tests were able to locate internal defects in the specimen with acceptable accuracy.

Sun et al. (2018) conducted an evaluation of a concrete bridge deck to assess the presence of delamination and reinforcement corrosion. Two NDT techniques, acoustic scanning and GPR, were employed for this purpose. While the acoustic scanning system was applied for rapid delamination mapping, the GPR scanning was utilized to characterize concrete deterioration and reinforcement corrosion. A comparison between the results shows that these two techniques provide complementary information about the conditions of bridge deck systems. While the acoustic system mainly identified shallow delamination, the GPR was able to identify more flaws and evaluate rebar corrosion at early stages.

Topczewski & Cieřła (2014) utilized NDT techniques and material examinations to perform an assessment of the load-bearing capacity of an existing concrete bridge over the Biala River in Bialystok, Poland. NDT methods including GPR, profometer, IE, and the pull-out method were implemented. The GPR and profometer were conducted to determine the amount of reinforcement embedded within the concrete bridge deck slab. The IE and pull-out methods were used to determine the thickness of the elements and their concrete strengths. Once the reinforcement layout was determined, the IE did not identify any structural defects, such as voids or internal cracks. The assessment revealed that the bridge condition at the time was adequate and did not cause a safety endangerment to its users. However, due to the progressive degradation, appropriate repairs were recommended.



Masoumi et al. (2020) conducted a condition assessment of prestressed concrete bridge girders of a concrete highway bridge structure in Ontario, Canada, after 47 years of service. The study incorporated various NDT methods, such as visual inspection, crack monitoring, UPV and GPR. UPV was used to measure the depth of the longitudinal and inclined cracks on the webs and flanges of the prestressed concrete girders, while GPR was primarily used to identify any delamination or debonding between the prestressing strands or rebar and the concrete. The test results showed no delamination or debonding between the prestressing strands or reinforcing bars and concrete. Such findings were confirmed by all conducted testing methods.

### **2.6.5 Structural evaluation**

The structural evaluation of existing concrete bridges, such as the bridge deck slabs of the Champlain Bridge, necessitates a shift from standard design practices to more refined assessment methodologies. While recent design codes for new structures prioritize conservatism to ensure a 75-year service life, the evaluation of existing, deteriorated structures requires a more accurate determination of capacity to make informed decisions regarding repair, rehabilitation, or closure (fib Task Group 3.2, 2024).

#### **2.6.5.1 Evaluation framework per Canadian Highway Bridge Design Code (CSA S6)**

The Canadian Highway Bridge Design Code: CSA S6:19 (CSA Group, 2019a) provides a general procedure for determining if an existing bridge can safely carry its current or proposed loads. The evaluation methodology relevant to the assessment of corroded post-tensioning tendons in deck slabs include a condition inspection, material strengths, target reliability and load testing.

##### **2.6.5.1.1 Condition inspection**

The evaluation must be based on current construction drawings, measurements and data, identifying specific defects and collecting sufficient data to properly consider these defects in the evaluation.

##### **2.6.5.1.2 Material strengths**

Rather than relying solely on original design values, Section 14 of CSA S6 allows for determining material strengths based on existing construction drawings, an estimation by considering the construction date or, more accurately, from the analysis of representative samples and tests.

#### **2.6.5.1.3 Target reliability index**

The CSA S6-19 utilizes a target reliability index,  $\beta$ , to determine appropriate load factors for structural evaluation. This index is determined based on system and element behaviours, the severity of existing deterioration, and the level of the inspection. System behaviour is categorized by the consequences of a component's failure, specifically whether it triggers a global collapse or remains a localized failure. This classification depends on the structural redundancy, the presence of multiple load paths, and whether the design incorporates structural continuity. Element behaviour is classified by the ductility and predictability of the failure mode. The code distinguishes between elements subject to sudden, brittle loss of capacity with little warning and those that exhibit gradual, ductile failure with post-yield capacity. Furthermore, the target reliability index is increased for structures where failure poses a high risk to life safety, or for bridges classified as essential to the local economy or designated as critical emergency routes.

#### **2.6.5.1.4 Load testing**

For complex cases where analytical models are insufficient for assessing the actual behaviour of a bridge, CSA S6 outlines procedures for static or dynamic load testing to verify structural behaviour and establish live load capacity factors.

#### **2.6.5.2 Advanced modelling per fib Bulletin 111**

fib Bulletin 111 (fib Task Group 3.2, 2024) complements traditional code-based evaluation by providing advanced state-of-the-art models for the structural performance of existing concrete structures. For bridge deck slabs with corroded tendons, it emphasizes the use of NDTs, residual capacity, corrosion effects of PT tendons, bond deterioration, and numerical simulation.

##### **2.6.5.2.1 Condition survey and NDT methodologies**

fib bulletin 111 outlines a rigorous procedure for acquiring information on a structure's current state through condition surveys. It specifically highlights several NDTs to identify internal defects and material properties without compromising structural integrity. These include UPV, IE, electromagnetic meters, GPR, radiography, and rebound hammer.

#### **2.6.5.2.2 Residual capacity**

The bulletin highlights that simplified design equations often ignore mechanisms such as compressive membrane action in slabs that can develop substantial residual capacity and are therefore worth considering in an assessment.

#### **2.6.5.2.3 Corrosion effects on PT tendons**

Evaluation models must account for the pitting nature of corrosion, which leads to localized cross-sectional loss that governs the tensile capacity and impairs the average ductility of the bar. In prestressed concrete, the high stress level in tendons adds the risk of stress corrosion cracking (SCC) and hydrogen embrittlement (HE), which can lead to brittle failures without visible warning.

#### **2.6.5.2.4 Bond deterioration**

Modelling the breakdown of bond between corroded tendons and the surrounding concrete is critical, as it can significantly alter the flexural and shear resistance of structural members. For unbonded tendons, brittle failure because of stress corrosion leads to a prestressing-force reduction directly proportional to the cross-section loss. For bonded tendons, a failure or fracture may lead to the development of a new anchorage on either side of the break, gradually building up the prestressing effect along a new transfer length.

#### **2.6.5.2.5 Numerical simulation**

The bulletin provides guidance on using non-linear finite element analysis (NLFEA) to simulate the actual state of a structure, including time-dependent effects of deterioration. This approach allows for separate modelling of concrete, steel bars, and the steel-concrete interface with dedicated constitutive models that consider the effect of corrosion on concrete strength, reinforcement ultimate strain, and bond stress-slip relations.

While CSA S6:19 provides a general procedure and target reliability indices for bridge evaluation in Canada, it remains a broad, high-level document that relies on the judgment of the practicing engineer. The code incorporates indices that account for factors such as damage severity and structural redundancy. However, it does not prescribe detailed methodologies for quantifying these effects. In contrast, fib Bulletin 111 provides a more rigorous technical basis by presenting advanced models that capture the electrochemical and mechanical behaviour of corroded post-

tensioning tendons. This enables a more refined assessment of structural performance, moving beyond the simplified and often conservative assumptions inherent in national design codes.

## **2.7 Gap in the state-of-the art**

It is clear that extensive studies have been conducted over the years with the aim of studying the effects of reinforcement corrosion on the structural capacity and ductility of reinforced and prestressed concrete elements. However, most of the studies have applied artificial corrosion regimes to small-scale laboratory specimens. Limited research has investigated the effects of corrosion on the behaviour of in-service field bridge structures. As a result, there is a need to conduct research on corroded field specimens extracted from actual bridge structures to better understand their performance, with the aim to correlate results obtained from laboratory studies to actual field structures, aiding in the evaluation of their remaining service lives and the improvement of repair and rehabilitative measures.

In addition, compared to regular reinforced concrete bridge elements, case studies on the in-service field performance of prestressed concrete elements are scarce. As a result, there is a need to investigate the performance of in-service corroded prestressed concrete field members to better understand their residual strengths and remaining service lives.

Previous studies demonstrate the ability to conduct different NDT techniques on in-service concrete bridge elements. However, there are limited studies showing the use of a combination of various non-destructive, semi-destructive, and destructive methods to determine the load capacity of an existing bridge structure, and to correlate their results. In particular, no such studies include critical corroded prestressed concrete bridge elements, such as bridge decks. Establishing correlations between various NDT techniques and destructive test results from a decommissioned prestressed bridge deck will provide a reliable basis for interpreting in-service inspection data. This approach enables engineers to develop more accurate condition assessments and optimize maintenance and asset management protocols for similar bridge systems. Furthermore, evaluating the structural response of corroded in-service bridge deck elements under progressive degradation, through the application of validated FEM, will allow for a deeper understanding of deformation patterns and cracking behaviour under realistic loading conditions.

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## **Chapter 3: Detection of corrosion effects on prestressed concrete bridge deck slabs from the Champlain Bridge through non-destructive testing techniques<sup>1</sup>**

### **Abstract**

As aging infrastructures raise public concerns, evaluating their performance is crucial for maintaining structural integrity, especially for corroding prestressed concrete members. These structures may experience substantial tendon cross-sectional area loss before any visible deterioration becomes detectable. While various non-destructive techniques (NDT) have proven effective in labs, correlating corrosion-induced damage in field members remains a challenge. Establishing these correlations is key for understanding the overall performance of aging structural concrete elements and ensuring their continued safe operation through non-invasive means. This paper investigates various NDTs on a concrete bridge deck, aiming to correlate results. Visual inspection, Schmidt rebound hammer, Ultrasonic Pulse Velocity (UPV), corrosion detection techniques, Ground Penetrating Radar (GPR), Ultrasonic Pulse Echo (UPE), and Impact Echo (IE) methods are evaluated for detecting concrete deck damage. Results show the methods' capabilities in detecting defects to a certain extent, highlighting their potential in assessing aging concrete infrastructures.

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<sup>1</sup>A version of this chapter has been published in Structural Concrete (Tawil D, Martín-Pérez B, Sanchez LFM, Noël M. Detection of corrosion effects on prestressed concrete bridge deck slabs from the Champlain bridge through non-destructive testing techniques. Structural Concrete. 2025;26(5):5761–5784. <https://doi.org/10.1002/suco.202400591>)

### 3.1 Introduction

Despite the growing interest and research in comprehending the implications of various damage mechanisms threatening the integrity of societal infrastructure, concerns about the safety and durability of transportation networks persist. The most recent Canadian infrastructure report card (Canada Infrastructure, 2019) highlighted an increase in the percentage of bridges with conditions categorized as fair, poor, or very poor, from 26% in 2016 (Canada Infrastructure, 2016) to 38% in 2019. The escalating number of bridges exhibiting compromised conditions emphasizes the need for a rigorous and comprehensive assessment methodology to ensure the safety of these vital components of our infrastructure.

The primary factor contributing to the degradation of concrete structures is the corrosion of embedded reinforcing steel, incurring a global cost estimated at 2.5 trillion US dollars in 2013, as reported by the National Association of Corrosion Engineers (NACE International) (Koch et al., 2016; Surahyo, 2019). Addressing this challenge is of paramount importance, particularly regarding the natural corrosion of prestressing tendons in concrete slabs, which poses significant structural vulnerabilities.

Non-destructive techniques (NDT) have become increasingly popular in providing crucial information when investigating material properties of field structures and ensuring their structural integrity (Concu et al., 2011; Popovics, 2003; Topczewski & Cieřła, 2014). The industry now relies on a variety of well-established methods that utilize various technologies to assess the concrete quality and tendon conditions (Borgerson & White, 2021; DMJM Harris, 2003; Wiss, Janney, Elstner Associates, Inc., 2017).

However, existing inspection standards and guidelines face challenges in accurately assessing these vulnerabilities. Common NDTs that measure corrosion likelihood and rate may encounter difficulties in assessing grouted multi-strand tendons due to the typical spiral arrangement of the tendons. Additionally, there remains a critical need to develop correlations between the results of various NDTs to enhance the assessment accuracy and establish optimized and effective maintenance and management strategies.

### **3.1.1 Assessment of concrete bridges and bridge elements**

A condition evaluation typically commences with a visual inspection to detect any apparent damage. However, it lacks the capability to detect internal defects, such as compromised concrete cover, delamination, and reinforcement corrosion. Therefore, the following step in the assessment often involves coring hardened specimens for standard tests. Despite its utility, this method fails to capture concrete in-situ properties that are essential for a comprehensive evaluation. Moreover, coring may induce localized damage and restrict data collection.

Current standards and inspection manuals for thoroughly assessing prestressed concrete bridges are limited. For instance, Canada's widely used Ontario Structure Inspection Manual (OSIM) (Ministry of Transportation of Ontario, 2008) provides general details and qualitative descriptions, while a 2013 technical advisory by the U.S. Federal Highway Administration focuses on managing chloride-induced corrosion in grouted post-tensioned concrete bridges (Federal Highway Administration - U.S. Department of Transportation, 2013). This document only considers the assessment of grouted ducts for chloride levels. A 2021 inspection manual by DEKRA Industrial and the Norwegian Road Authorities provides comprehensive recommendations for identifying hidden defects in tendon ducts (DEKRA Industrial, 2021). Despite these efforts, there remains a significant gap in assessment guidelines that integrate visual inspection with a comprehensive set of NDTs to thoroughly evaluate both concrete and prestressing strands.

Previous studies have shown the effectiveness of individual non-destructive testing (NDT) on in-service concrete bridge elements (Gucunski et al., 2014; Masoumi et al., 2020; Oh et al., 2013a; Pasculli et al., 2018; Sun et al., 2018). However, there is a notable gap in research focusing on integrating various non-destructive, semi-destructive, and destructive methods to comprehensively assess the condition and determine the load capacity of existing bridge structures through result correlation. In particular, no such studies include critical corroded prestressed concrete bridge elements, such as bridge decks.

### **3.1.2 Role of non-destructive testing**

Evaluating post-tensioned concrete bridges is challenging due to their complex systems. Visual inspections, which offer useful findings for reinforced concrete bridges, are inadequate for

assessing post-tensioned bridges (Mazzatura et al., 2023). Therefore, additional NDTs have proven crucial.

Despite recent advancements, directly measuring corrosion in grouted post-tensioned concrete systems remains challenging due to metal tendon ducts. While there is currently no direct method for assessing corrosion in these tendons, other modern NDTs can indirectly evaluate their state of corrosion, though they pose challenges (ACI Committee 228, 2013).

### **3.2 Description of structure under investigation**

This paper presents a condition evaluation of sections of the deck slab of the former Champlain Bridge in Montreal, Canada, using NDTs. The slabs were extracted during the bridge's deconstruction in 2019 after 57 years of service. The work reported in this paper is part of a broader study to evaluate the condition and performance of the corroded infill slabs that served for nearly six decades. The NDT results are detailed for one slab containing two embedded tendons.

Construction of the bridge over the Saint Lawrence River began in 1957, and it was opened to the public in 1962. Despite efforts to mitigate deterioration, the bridge underwent significant degradation over its lifespan. The original design, chosen from 28 alternatives for its cost-efficiency, featured a highly integrated deck structure that facilitated monolithic behaviour of girders, intermediate slabs, and diaphragms (Delcan, 2011). However, it lacked durability considerations, leaving it vulnerable to the effects of de-icing salts introduced after construction. Additionally, during the bridge's construction period, the development of air entrainers to mitigate freeze-thaw cycles was still ongoing, rendering the concrete used more susceptible to such environmental conditions. The superstructure's deck system was directly exposed to these harsh environmental conditions, with poor drainage exacerbating the deterioration of its components. Moreover, as one of Canada's busiest bridges, handling over 50 million vehicle crossings and supporting \$20 billion in annual Canada–U.S. trade (Carlin, 2017; COWI, 2017) the bridge faced increasing stresses from evolving vehicle loads, particularly heavier trucks, which further strained a design that was not originally intended to accommodate such demands.

The bridge's deck design featured seven prestressed concrete girders connected to six transversely post-tensioned cast-in-place infill slabs (Figure 3-1). The infill slab design relied on the transverse prestressing tendons to ensure continuity between the infill slabs and the top flanges of the girders,

facilitating effective load transfer. Hence, the possibility of water penetration into the tendons at the girder/slab joint introduced a vulnerability to the system.

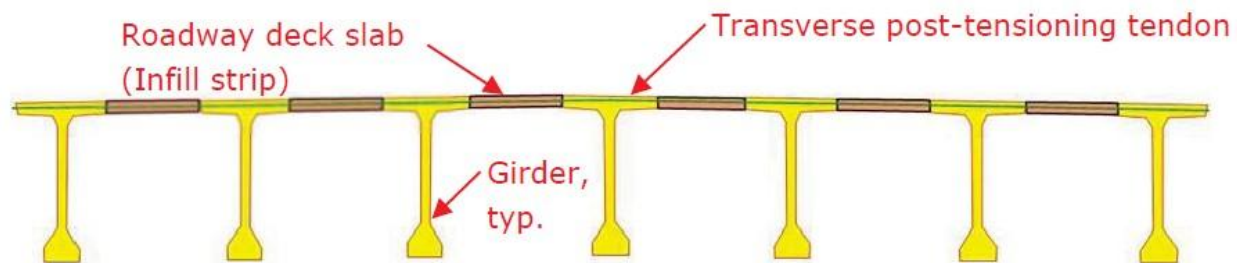


Figure 3-1 Typical cross-section of the concrete deck (COWI, 2017)

The slab design also included secondary mild deformed steel reinforcement for crack control, spanning longitudinally parallel to traffic flow. However, the absence of a waterproofing membrane, coupled with the extensive use of de-icing salts, allowed the salt-laden water to infiltrate, leading to reinforcement corrosion.

The obtained slab sample measured 225 mm deep, 2,000 mm wide and 3,000 mm long, with a 50 mm layer of asphalt topping. Grouted post-tensioning tendons consisted of twelve 7-mm diameter cables, spaced at an average of 1,000 mm. The slabs functioned as a one-way system, spanning 1,970 mm between the girders. Figure 3-2 and Figure 3-3 show the primary and the secondary reinforcement, respectively.





Figure 3-2 Stacked infill deck slab samples (longitudinal edge)



Figure 3-3 Stacked infill deck slab samples (transverse edge)

### 3.3 NDT methods

#### 3.3.1 Visual inspection

A visual inspection forms the basis for formulating an adequate testing program (International Atomic Energy Agency, 2002). It plays a key role in identifying and distinguishing between various signs of distress, including cracks, pop-outs, spalling, disintegration, efflorescence, and weathering. Nonetheless, a visual inspection survey is limited to visible external damage of the structure. It does not provide an assessment of the internal structural condition, which is influenced by numerous other parameters, including material properties, design considerations and environmental factors.

Qualitative, semi-qualitative, and quantitative visual inspection techniques were used on 2 m x 3 m slabs overlaid with asphalt pavement. A visual survey included documenting any unusual features or visual signs of deterioration. Slab soffit surfaces, as well as the cut side surfaces of each slab and tendon cross-sections, were all inspected and photographed. Available limited relevant structural drawings showing plan and section views of the roadway deck system, in addition to available documents on past inspection reports, were also examined. Crack widths were further inspected and measured using a crack comparator card and a 25x magnifier. Finally, the Cracking Index (CI) method was used to quantitatively assess the extent of damage. This technique averages the crack widths and frequencies along a defined square perimeter (Figure 3-4).

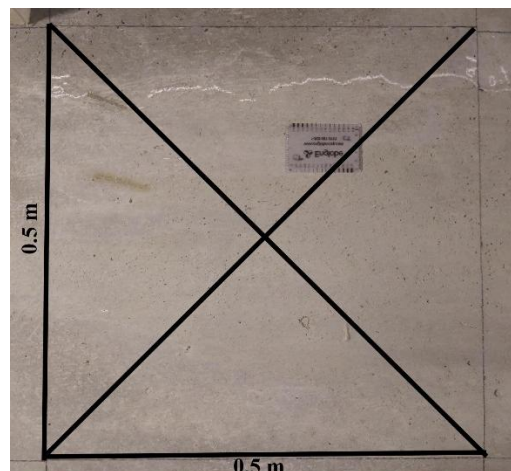


Figure 3-4 Measurement of the cracking index along slab soffit

The CI, expressed in mm/m, is calculated by summing the crack widths and then dividing the result by the base length of 0.5 m. The resulting value is categorized into levels ranging from negligible (0–0.5) to ultra-high (>10), providing a range of crack severity (Godart et al., 1992).

The slabs were labelled sequentially to indicate their respective numbers, with each label corresponding to a specific slab, such as 'S1' for Slab 1. Tendons were labelled to indicate the slab, the edge, and tendon number. For example, S1A1 and S1B1 refer to tendon 1 visible from edges A and B of slab 1, respectively. Detailed findings from the visual inspection are presented in Section 6.1.

### **3.3.2 Schmidt rebound hammer**

The Schmidt hammer, also known as the rebound or Swiss hammer, is a testing apparatus used to evaluate the hardness, or elastic properties, of concrete. This device uses the energy absorbed during its interaction with the concrete test specimen to assess the element's strength and stiffness. Concrete with low strength and stiffness properties will absorb more energy and will thus yield low rebound numbers when compared to concrete with high strength and stiffness properties (ACI Committee 228, 2019). However, despite its popularity, the Schmidt hammer provides limited insight into the condition of the internal structure.

Rebound numbers were measured along three transverse strips at 500 mm, 1,500 mm, and 2,500 mm intervals from the slab edge and along the soffit (Figure 3-5), following ASTM C805 standard (ASTM C805, 2018). The results from the Schmidt rebound hammer tests are thoroughly discussed in Section 3.4.3

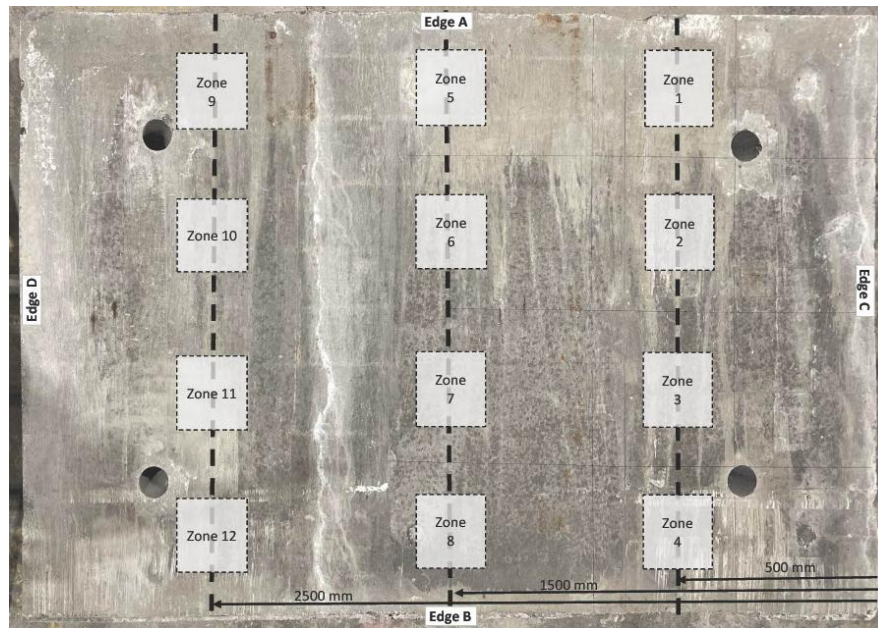


Figure 3-5 Typical rebound hammer test areas along slab soffits

### 3.3.3 Ultrasonic pulse velocity

The Ultrasonic Pulse Velocity (UPV) method measures the propagation velocity of longitudinal stress-wave pulses over a known path length and through the concrete member. This technique determines the uniformity and quality of concrete by utilizing a pulser that sends a short-duration and high-voltage signal to the device's transducer placed at the opposite concrete surface (ACI Committee 228, 2019). This will cause the transducer to vibrate at its resonant frequency, generating a stress-wave pulse in the concrete member. The test requires a viscous fluid along the pulser-concrete and transducer-concrete interface, which helps transfer the vibrations to the concrete. The device measures the elapsed travel time of the signal from the start of the electric pulse to the time the pulse is received. The pulse velocity through the concrete can then be obtained by dividing the direct path length by the travel time.

Indirect UPV measurements were obtained at 200 mm intervals between transducers and along 15×800 mm longitudinal strips, bounded by the reinforcement in both directions (Figure 3-6). Tests were completed as per ASTM C597 (ASTM C597, 2016), and results were analyzed according to Whitehurst (Whitehurst, 1996). The values are categorized into different levels, ranging from poor concrete quality (2.0 – 3.0 km/s) to excellent quality (> 4.5 km/s). The results of UPV measurements are comprehensively covered in Section 3.4.4 .



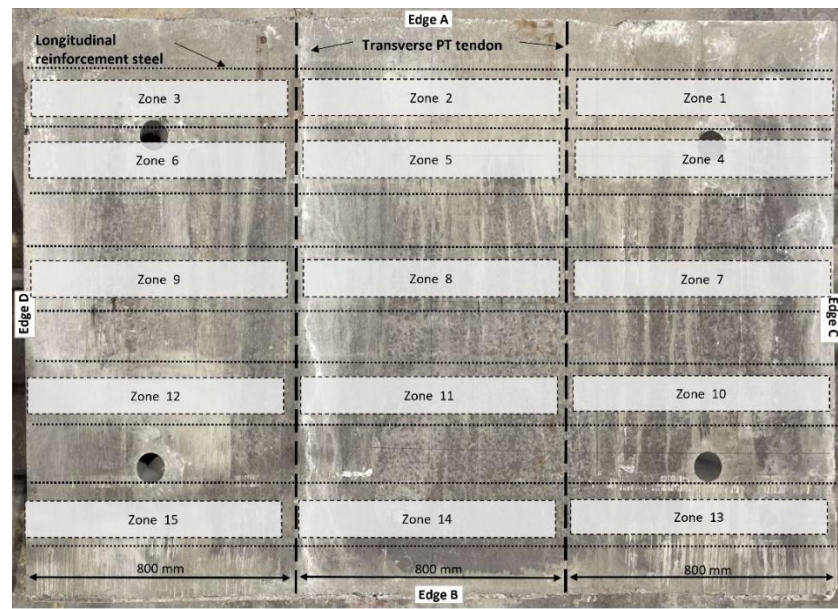


Figure 3-6 Typical UPV test areas along slab soffits

### 3.3.4 Combined sonic rebound method

The sonic rebound (SonReb) method integrates two widely used NDT techniques, namely the Schmidt Rebound Hammer and the UPV methods, to determine the mechanical strength of concrete. Since the inception of the SonReb method in the mid-1960s, there has been a continuous emphasis on extensive research dedicated to developing empirical formulations and calibrating them (Cristofaro et al., 2020b).

Seventeen formulations, discussed by Cristofaro et al. (2020), representing a significant sample of predictive models were evaluated (Table 3-1). For S1, the final average rebound number combined with each UPV reading estimated concrete compressive strengths using the established formulations. The findings from the Combined SonReb method are detailed in Section 3.4.5 .

Table 3-1 Formulations to determine concrete compressive strength using the SonReb method (Cristofaro et al. 2020)

| Author                        | Year | Formulation                                                 | Units                    |
|-------------------------------|------|-------------------------------------------------------------|--------------------------|
| <b>Bellander</b>              | 1979 | $R_{cub} = 0.00082(I_r)^3 + 11.03(V_{us}) - 32.7$           | MPa, km/s                |
| <b>Meynink, Samarin</b>       | 1979 | $f_{cyl} = -24.1 + 1.24(I_r) + 0.058(V_{us})^4$             | MPa, km/s                |
| <b>Giacchetti, Lacquaniti</b> | 1980 | $R_{cub} = 7.695(10^{-11})(I_r)^{1.4} (V_{us})^{2.6}$       | MPa, m/s                 |
| <b>Bocca, Cianfrone</b>       | 1983 | $R_{cub} = 2.765(10^{-9})(I_r)^{1.3114} (V_{us})^{2.487}$   | kg/cm <sup>2</sup> , m/s |
| <b>Samarin, Dhir</b>          | 1984 | $f_{cyl} = -12 + 0.76(I_r) + 0.1(V_{us})^4$                 | MPa, km/s                |
| <b>Gasparik</b>               | 1992 | $R_{cub} = 0.0286(I_r)^{1.246} (V_{us})^{1.85}$             | MPa, km/s                |
| <b>RILEM</b>                  | 1993 | $R_{cub} = 9.27(10^{-11})(I_r)^{1.4} (V_{us})^{2.6}$        | MPa, m/s                 |
| <b>Di Leo, Pascale</b>        | 1994 | $R_{cub} = 1.2(10^{-9})(I_r)^{1.058} (V_{us})^{2.446}$      | MPa, m/s                 |
| <b>Arioglu, Koyluglu</b>      | 1996 | $R_{cub} = 0.00153(I_r^3 V_{us}^4)^{0.611}$                 | MPa, km/s                |
| <b>Ramyar Kol</b>             | 1996 | $f_{cyl} = -39.57 + 1.532(I_r) + 5.0614(V_{us})$            | MPa, km/s                |
| <b>Kheder</b>                 | 1996 | $R_{cub} = 0.0158(I_r)^{1.1171} (V_{us})^{0.4254}$          | MPa, m/s                 |
| <b>Benoncini, Formichi</b>    | 2003 | $f_{cyl} = 5.9 + 2.712(10^{-15})(I_r)(V_{us})^4$            | MPa, m/s                 |
| <b>Caiaro et al.</b>          | 2003 | $R_{cub} = 1.74(10^{-7})(I_r)^{-0.0674} (V_{us})^{2.36}$    | MPa, m/s                 |
| <b>Del Monte et al.</b>       | 2004 | $R_{cub} = 4.40(10^{-7})(I_r^2 V_{us}^3)^{0.5634}$          | MPa, m/s                 |
| <b>Menditto et al.</b>        | 2004 | $R_{cub} = 4.0(10^{-5})(I_r)^{1.88148} (V_{us})^{0.80840}$  | MPa, m/s                 |
| <b>Faella et al.</b>          | 2009 | $R_{cub} = 2.6199(10^{-8})(I_r)^{0.5341} (V_{us})^{2.2878}$ | MPa, m/s                 |
| <b>Faella et al.</b>          | 2009 | $R_{cub} = 0.26511(I_r) + 0.01385(V_{us}) - 34.51583$       | MPa, m/s                 |

$R_{cub}$  = concrete cubic strength;  $I_r$  = rebound number;  $V_{us}$  = ultrasonic velocity;  $f_{cyl}$  = concrete cylindrical strength.

### 3.3.5 Corrosion measurements

The detection and evaluation of corrosion in embedded steel reinforcement is performed by measuring the half-cell potential, corrosion rate, and concrete surface resistivity values. As shown in Figure 3-7, the different electrochemical techniques provide different types of information that can then be used to assess the corrosion conditions of embedded reinforcement. While potential mapping provides critical information before corrosion becomes visible at the concrete surface, linear polarization resistance (LPR), Connectionless Electrical Pulse Response Analysis (CEPRA), and concrete resistivity methods provide quantitative information on the propagation stage.

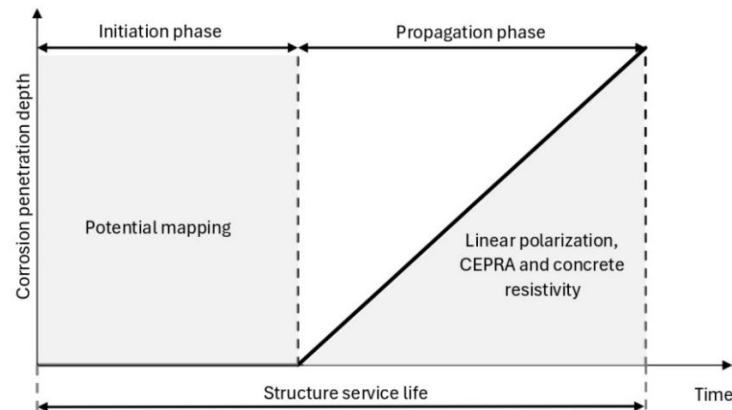


Figure 3-7 Different information obtained from the various electrochemical techniques

### 3.3.5.1 Half-cell potential

Measuring the electrochemical potentials on an embedded prestressing tendon or reinforcing bar relative to a reference half-cell located on the concrete surface allows the detection of corrosion activity. ASTM C876 (ASTM C876, 2015) suggests conducting half-cell potential measurements as an initial evaluation of the probability of corrosion in embedded reinforcement before conducting additional testing. A continuous connection with the steel is crucial to yield good results (Bungey & Grantham, 2006). To date, potential mapping is the only standardized NDT for assessing the probability of corrosion of reinforcement embedded in concrete. The readings taken at the concrete surface are influenced by factors such as concrete cover and resistivity, which make the interpretation process for half-cell potential readings complex.

### 3.3.5.2 Corrosion rate

Obtaining quantitative information on the corrosion rate of reinforcement is crucial for structural assessments, service-life predictions, and the evaluation of repair methods for corroding structures. In general, two different interpretations for “corrosion rate” of reinforcement in concrete exist: average corrosion rate and instantaneous corrosion rate (Bertolini et al., 2013). The average corrosion rate is required when conducting service-life modelling or prognosis studies. Once the loss of steel mass or cross-section is determined, the average corrosion rate is then calculated over a long period of time. On the other hand, the instantaneous corrosion rate may be determined by the LPR and CEPRA methods. As corrosion rate measurements are sensitive to factors such as

moisture and temperature, corrections must be taken into account to allow for reasonable comparisons of measurements obtained at different times during the lifetime of a structure.

### **3.3.6 Concrete surface resistivity**

The ability of corrosion currents to flow through concrete can be determined by measuring the concrete's electrical resistivity. The most widely used technique involves utilizing a four-point probe, called a Wenner probe, and placing it directly on the concrete surface. The device consists of four equally spaced point electrodes, where the two outermost electrodes induce the AC current, while the two innermost electrodes measure the resulting potential drop of the electric field. Concrete resistance against electrical current may be correlated to the resistance against chloride ion movement. Resistivity measurements supplement half-cell potential measurements and affect corrosion rate measurements due to their influence on the flow of ions through concrete.

Corrosion parameters including the electrical half-cell potential, corrosion rate using the CEPRA method (Ghods et al., 2016), and concrete surface electrical resistivity were measured at intervals halfway between consecutive longitudinal reinforcing steel bars. Given the time-dependent nature of concrete saturation and its impact on corrosion parameters, the slab underwent a conditioning phase, as per the procedure detailed in Tawil et al. (Tawil et al., 2023a) prior to measurement. Half-cell potential measurements were conducted following ASTM C876 (ASTM C876, 2015), corrosion rate measurements were assessed according to Andrade & Alonso (Andrade & Alonso, 1996), and surface resistivity was evaluated in accordance with AASHTO T-358-15 (AASHTO T 358-15, 2015). Measurements of corrosion parameters can be found in Section 3.4.6.3 .

### **3.3.7 Ground penetrating radar**

GPR, also called short-pulse radar or impulse radar technique, is an NDT that has extensive uses for concrete examination, such as identifying delamination, voids, air gaps, as well as reinforcing bars and duct assessment. Additionally, it is an excellent tool for measuring concrete cover and thickness, as well as the depths of tendon ducts and reinforcement (ACI Committee 228, 2013). Radio detection and ranging (RADAR) technology used in NDTs generate electromagnetic waves with frequencies that range between 400 MHz to 2,600 MHz and penetrate the concrete to detect any defects or anomalies located at a maximum depth of 750 mm (ACI Committee 228, 2013). This is done through the reflection of these waves at an interface between two media with different



dielectric properties (ACI Committee 228, 2013; Gaydeckp & Burdekin, 1998). The dielectric property, or impedance, of a medium is a constant that is determined by its electrical and magnetic properties. It is a measure of its resistance to the propagation of a wave through it (ACI Committee 228, 2013). If there is a large difference in the dielectric constants at an interface, a large amount of electromagnetic energy will be reflected at that interface (International Atomic Energy Agency, 2002).

Prior to conducting the GPR test method and all subsequent tests, the slabs were cut to 2,000 mm long x 1,000 mm. wide. This was made to accurately simulate field conditions for the structural testing phase (Chapter 4), where the bridge deck spans 2,000 mm and operates as a one-way slab system.

The GPR device collected data along the slab soffit using a 0.1 m x 0.1 m grid (Figure 3-8), following ASTM D6087 (ASTM D6087, 2022) . Note that the gridline spacing in the longitudinal direction and along the centerline of the slab appears tighter in Figure 3-8 due to additional gridlines that were incorporated for the UPE measurements. The results of GPR measurements can be found in Section 3.4.7 .

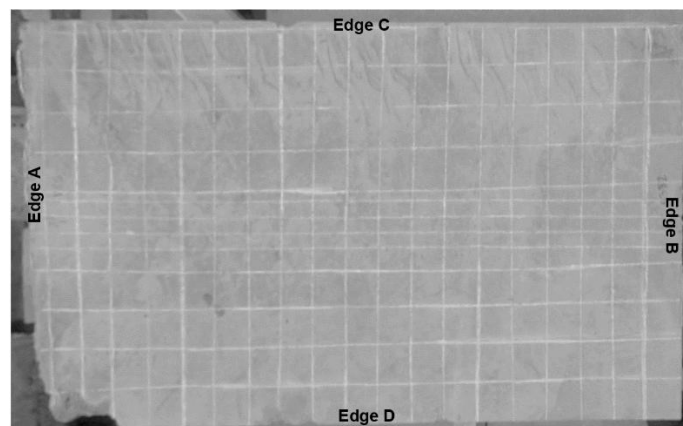


Figure 3-8 0.1 m x 0.1 m grid drawn on concrete soffit for GPR measurements

### 3.3.8 Ultrasonic pulse echo

The UPE is a stress-wave method which makes use of stress waves above the audible range (frequency > 20 kHz) when detecting internal flaws or measuring thickness (ACI Committee 228, 2013). In this technique, a transmitter unit induces a highly damped but short-duration stress pulse

into the concrete element from an accessible surface. As the pulse travels through the test object and reflects off flaws or different interfaces, a receiver records the reflected response.

The asphalt topping was first removed to ensure continuous ultrasonic wave propagation through the concrete. Measurements were then taken at 5 cm intervals along and parallel to the tendons on both the soffit and top surface of the slab, following ASTM E797/E797M (ASTM E797/E797M, 2021). UPE measurement results are presented in Section 3.4.8 .

### **3.3.9 Impact echo**

In the IE technique, a transient stress pulse with greater penetration ability is generated using a mechanical impact on the surface. The generated frequencies range from a few 100 Hz to several 10,000 kHz (Gaydeckp & Burdekin, 1998). Waves produced by the stress pulse reflect off internal or external interferences and travel back to the surface impact point. The surface will then reflect it back to the interference or void, and so on. This is known as the transient resonance condition. A receiving transducer then measures the displacements produced at the impact point due to the arrival of the waves.

The IE technique was employed as a supplementary test alongside UPE measurements to confirm the presence of voids within the grouted prestressing tendons. Measurements were taken along the centerline of the tendon and along both the top and soffit surfaces of the slab at 5 cm intervals. A high-frequency steel hammer (10-mm diameter) impacted the concrete slab, and the resulting signals were recorded and digitized. Post-processing identified voids and their depths across the slab cross-section based on peak frequency information. The results of IE measurements are comprehensively covered in Section 3.4.9 .

### **3.3.10 Summary of test methods**

Table 3-2 provides a comprehensive summary of the test methods utilized in the evaluation, detailing the specific variables or features measured along with their corresponding units, and the inherent precision and limitations of each technique.

Table 3-2 Overview of the implemented NDTs

| Technique                           | Measured variable /feature                              | Units                     | Precision                                                                                 | Strengths                                                                                                                           | Limitations                                                                                                                                                                                                                                        |
|-------------------------------------|---------------------------------------------------------|---------------------------|-------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>Visual Inspection</b>            | Damage features.<br>Cracking Index.                     | mm/m                      | N/A                                                                                       | Simple and cost-effective.<br>Immediate damage assessment.                                                                          | Limited to visible/surface damage.<br>Cannot detect fine cracks.                                                                                                                                                                                   |
| <b>Schmidt Rebound Hammer</b>       | Concrete hardness/elastic properties:<br>Rebound number | Unitless                  | Standard deviation of 2.5 units, as per ASTM C805                                         | Simple and cost-effective.<br>Surface hardness indicator.                                                                           | Affected by near-surface concrete layer (texture, and aggregate type/volume).<br>Does not provide information about the quality of deeper concrete layers.<br>The correlation between rebound number and compressive strength is not well-defined. |
| <b>UPV</b>                          | Pulse velocity:<br>UPV                                  | km/s                      | Variability of 2% for sound concrete, and 20% for deteriorated concrete, as per ASTM C597 | Capable of detecting internal defects, such as voids, cracks, and honeycombing.<br>Correlates with concrete quality and uniformity. | Influenced by surface conditions, concrete saturation degree, and aggregate type/gradation.<br>Limited path length.                                                                                                                                |
| <b>SonReb</b>                       | Concrete compressive strength                           | MPa                       | N/A                                                                                       | Improve accuracy for concrete strength assessment by integrating two methods.                                                       | Current formulations vary in accuracies. Further validation analyses are still required.                                                                                                                                                           |
| <b>Half-Cell Potential</b>          | Likelihood of corrosion                                 | mV/CSE                    | Standard deviation of 20 mV, as per ASTM C876                                             | Only standardized technique for corrosion assessment.                                                                               | Influenced by environmental conditions, saturation degree, surface conditions and concrete cracking.<br>No direct assessment on extent/rate of corrosion.                                                                                          |
| <b>Corrosion Rate by CEPR</b>       | Corrosion rate                                          | $\mu\text{A}/\text{cm}^2$ | N/A                                                                                       | Direct corrosion rate assessment.                                                                                                   | Influenced by environmental conditions, saturation degree, surface conditions and concrete cracking.<br>Challenges in accurately assessing tendons within metal ducts due to potential signal interference.                                        |
| <b>Concrete Surface Resistivity</b> | Resistance to chloride ion penetration                  | $\Omega\cdot\text{m}$     | N/A                                                                                       | Concrete resistivity is related to chloride permeability.                                                                           | Influenced by environmental conditions, saturation degree and surface conditions.                                                                                                                                                                  |
| <b>GPR</b>                          | Embedded steel reinforcement                            | Unitless                  | Variability of $\pm 11.2\%$ as per ASTM D6087                                             | Detects voids in plastic ducts.<br>Fast method.<br>Assess multi-layer systems.                                                      | Cannot detect voids in metallic ducts.<br>Limited function in highly reinforced areas.<br>Sensitive to concrete moisture content.<br>Cannot detect early signs of corrosion                                                                        |

|            |                                                       |          |     |                                                                                       |                                                                                                                                                                                                                |
|------------|-------------------------------------------------------|----------|-----|---------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>UPE</b> | Defects/voids within concrete or grout in tendon duct | Unitless | N/A | Detect voids in metallic ducts.<br>Detect objects behind beyond reinforcement layers. | Sensitive to size of delamination/void plane.<br>Cannot penetrate beyond concrete-air interface.<br>Cannot locate tendon duct with no voids.                                                                   |
| <b>IE</b>  | Defects/voids within concrete or grout in tendon duct | Unitless | N/A | Greater penetration ability than UPE.<br>Detect objects beyond reinforcement layers.  | Ambiguity due to uncertain wave paths.<br>Limited in detecting reinforcement/tendon ducts with small diameters.<br>Cannot penetrate beyond concrete-air interface.<br>Cannot locate tendon duct with no voids. |

### 3.4 Results

#### 3.4.1 Visual inspection

This section details the NDT results for slab S1. Results for the remaining slabs are presented in Appendix A. During the visual inspection of S1, a single transverse crack was detected along the soffit, aligning with tendon S1A1-S1B1, and accompanied by white efflorescence on the surrounding surface (Figure 3-9). No delamination or additional cracks were observed along the slab soffit. Additionally, the longitudinal temperature reinforcement showed no observable cracks nearby.

It was also observed that the top cover thickness of both tendons exhibited slight variation, measuring 10.1 cm near edge A and 11.5 cm near edge B for S1A1-S1B1 and 10.2 cm near edge A and 11.5 cm near edge B for S1A2-S1B2. Furthermore, the concrete cover to the top and bottom longitudinal steel reinforcement varied; the top cover thickness was between 6-7 cm for the top layer, and the bottom cover thickness varied between 3-5 cm for the bottom layer. As shown in Figure 3-9, four 100-mm diameter concrete cores were extracted from the sample to facilitate the removal of the deck slab from the bridge structure.

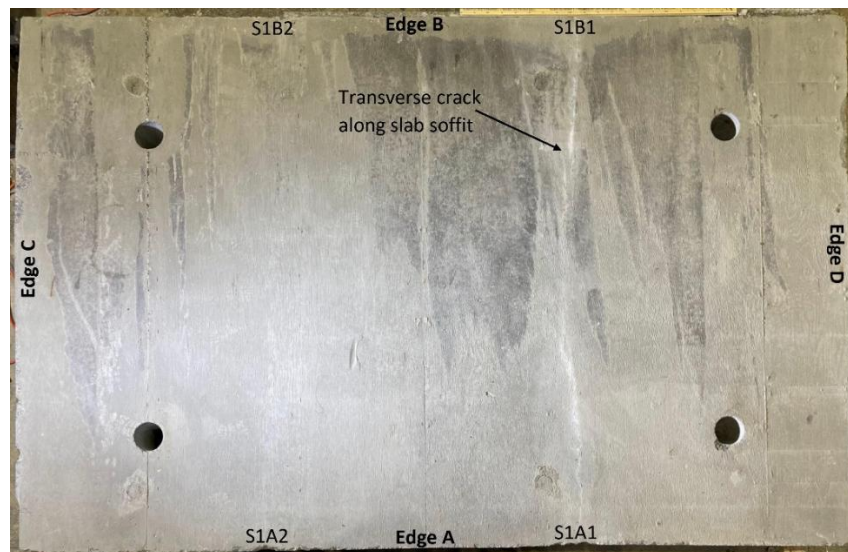


Figure 3-9 S1 soffit highlighting transverse crack with white efflorescence

The tendon cross-sections along the cut planes were inspected for the visual quality and application of grout, as well as the presence of voids (Figure 3-10). While S1A1 exhibits adequate grout surrounding the individual cables, S1A2, S1B1, and S1B2 appear to have concentrated grout along

one side of the metal duct, leaving the opposite side in direct contact with the cables due to insufficient grouting. This deficiency in grout coverage on one side of the duct may have exposed the cables, increasing the risk of corrosion. Small gaps were also observed between the individual cables.



Figure 3-10. Tendon cross-sections at cut surfaces

### 3.4.2 Crack measurements

The maximum opening of the transverse crack measured 0.1 mm, with a CI value less than 0.2 mm/m, suggesting negligible damage, as per Godart et al. (1992) (Godart et al., 1992). Given the limited extent of cracking observed on the slab soffit, it would have been unlikely to warrant further investigation if the slab had remained in service.

### 3.4.3 Schmidt rebound hammer

S1 exhibited an average rebound number of 54.84, accompanied by a coefficient of variation of 2.18%, as indicated in Table 3-3. While the rebound hammer is recognized for providing information solely into the surface quality of concrete and not its interior conditions, it can be inferred that the uniformity of the concrete surface quality is similar throughout the slab (Malhotra & Carino, 2003).

Table 3-3. Schmidt Rebound Hammer Numbers – S1

| Zone | Point |    |    |    |    |    |    |    |    |                   | Average      | Average of 4 zones | Standard deviation | Coefficient of variation (%) |
|------|-------|----|----|----|----|----|----|----|----|-------------------|--------------|--------------------|--------------------|------------------------------|
|      | 1     | 2  | 3  | 4  | 5  | 6  | 7  | 8  | 9  | 10                |              |                    |                    |                              |
| 1    | 56    | 52 | 55 | 57 | 56 | 53 | 56 | 58 | 59 | 53                | 55.50        | 55.45              | 1.75               | 3.15                         |
| 2    | 60    | 58 | 58 | 56 | 58 | 56 | 57 | 55 | 57 | 58                | 57.30        |                    |                    |                              |
| 3    | 50    | 56 | 57 | 57 | 54 | 60 | 59 | 57 | 54 | 55                | 55.90        |                    |                    |                              |
| 4    | 52    | 53 | 51 | 54 | 53 | 52 | 54 | 55 | 52 | 55                | 53.10        |                    |                    |                              |
| 5    | 55    | 58 | 55 | 52 | 55 | 49 | 54 | 58 | 54 | 51                | 54.10        | 54.90              | 0.88               | 1.59                         |
| 6    | 50    | 57 | 51 | 52 | 58 | 54 | 58 | 54 | 56 | 53                | 54.30        |                    |                    |                              |
| 7    | 55    | 58 | 57 | 58 | 58 | 52 | 52 | 56 | 60 | 54                | 56.00        |                    |                    |                              |
| 8    | 56    | 57 | 57 | 50 | 56 | 52 | 56 | 58 | 56 | 54                | 55.20        |                    |                    |                              |
| 9    | 54    | 56 | 53 | 52 | 52 | 56 | 52 | 56 | 54 | 52                | 53.70        | 54.18              | 0.59               | 1.09                         |
| 10   | 56    | 58 | 53 | 56 | 56 | 56 | 51 | 51 | 50 | 55                | 54.20        |                    |                    |                              |
| 11   | 50    | 53 | 56 | 57 | 53 | 56 | 52 | 56 | 55 | 50                | 53.80        |                    |                    |                              |
| 12   | 54    | 53 | 58 | 57 | 54 | 55 | 52 | 60 | 53 | 54                | 55.00        |                    |                    |                              |
|      |       |    |    |    |    |    |    |    |    | <b>All values</b> | <b>54.84</b> |                    | <b>1.20</b>        | <b>2.18</b>                  |

### 3.4.4 Ultrasonic pulse velocity

Figure 3-11 shows a contour map illustrating the distribution of ultrasonic pulse velocities across the soffit of S1. The colour map is based on reference criteria outlined by Whitehurst (Whitehurst, 1996) to aid visual interpretation. Locations highlighted in red correspond to areas of poor concrete quality (2.0 – 3.0 km/s), while locations highlighted in yellow, green, and dark green correspond to areas of doubtful (3.0 – 3.5 km/s), good (3.5 – 4.5 km/s), and excellent qualities (> 4.5 km/s), respectively.

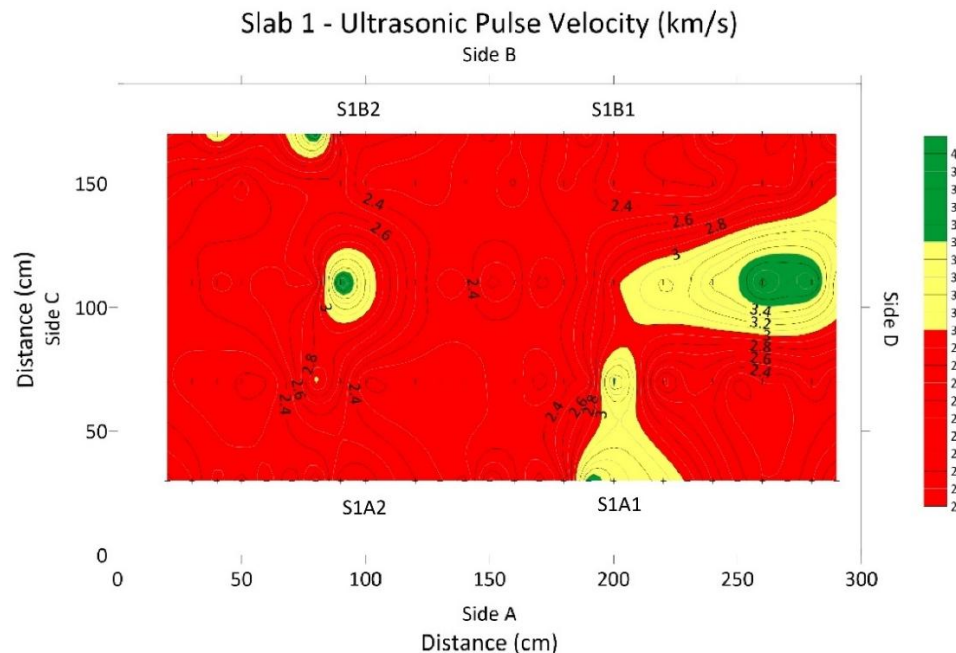


Figure 3-11 UPV mapping - S1

Most UPV measurements fell within the range 2.0 – 2.9 km/s, suggesting poor concrete quality. Potential damage to the slab edges might have occurred during specimen removal from the deck. Internal damage could also be present due to the corrosion of reinforcement and tendons. In other regions, such as the midsection of S1A2-S1B2 and half of S1A1-S1B1 (from side A to half of the tendon), measurements showed higher values ranging from 3.0 to 4.0 km/s, suggesting uncertain-to-satisfactory internal conditions.

### 3.4.5 Combined sonic rebound method

The reported values in Table 3-4 highlight the minimum, maximum, median, and average compressive strength estimates for S1 by implementing each of the formulations presented in Table 2-8.



Table 3-4. Estimated concrete compressive strengths (MPa) for slab 1 using the SonReb method

| Slab number | Value                                                     | Formulations by authors (Year) |                         |                               |                         |                      |                 |               |                        |
|-------------|-----------------------------------------------------------|--------------------------------|-------------------------|-------------------------------|-------------------------|----------------------|-----------------|---------------|------------------------|
|             |                                                           | Bellander (1979)               | Meynink, Samarin (1979) | Giacchetti, Lacquaniti (1980) | Bocca, Cianfrone (1983) | Samarin, Dhir (1984) | Gasparik (1992) | RILEM (1993)  | Di Leo, Pascale (1994) |
| 1           | Minimum                                                   | 156.50                         | 44.15                   | 10.72                         | 11.20                   | 31.46                | 19.87           | 12.92         | 13.13                  |
|             | Maximum                                                   | 182.26                         | 44.80                   | 57.66                         | 55.96                   | 53.33                | 65.78           | 69.46         | 63.93                  |
|             | <i>Median</i>                                             | <i>162.15</i>                  | <i>44.26</i>            | <i>17.21</i>                  | <i>17.60</i>            | <i>33.36</i>         | <i>27.82</i>    | <i>20.73</i>  | <i>20.49</i>           |
|             | <i>Average</i>                                            | <i>163.78</i>                  | <i>44.30</i>            | <i>20.57</i>                  | <i>20.80</i>            | <i>34.97</i>         | <i>30.98</i>    | <i>24.79</i>  | <i>24.11</i>           |
|             | <i>Percent diff. from design strength (%)</i>             | <i>374.74</i>                  | <i>28.41</i>            | <i>40.36</i>                  | <i>60.11</i>            | <i>1.35</i>          | <i>10.19</i>    | <i>28.16</i>  | <i>30.11</i>           |
|             | <i>Percent diff. from tested compressive strength (%)</i> | <i>174.34</i>                  | <i>-25.80</i>           | <i>-65.54</i>                 | <i>-65.16</i>           | <i>-41.42</i>        | <i>-48.11</i>   | <i>-58.48</i> | <i>-59.61</i>          |

Samarin and Dhir's formulation (Samarin & Dhir, 1984) approximated a concrete compressive strength to within 1.35% of the reported design strength of 34.5 MPa. Bellander's approach (Bellander, 1979) yielded notably higher values, while others varied around the design strength. Subsequently, compressive strength testing was conducted on the concrete slab, yielding an average strength of 59.7 MPa, with the detailed results presented in Chapter 4.

As concrete cures over time, its strength typically increases. However, damage due to the anticipated corrosion of the prestressing tendons, along with possible damage by freeze-thaw cycles resulting from the structure's exposure to harsh environmental conditions, may lead to a decrease in concrete strength. Therefore, it is challenging to discuss SonReb results without concrete core compressive strength tests.

### 3.4.6 Corrosion assessment

#### 3.4.6.1 Half-cell potential

Figure 3-12 presents a top slab view illustrating half-cell potential measurements along the length of the transverse prestressing tendons for S1. The orientation and location of the embedded tendons are indicated by a dashed line, accompanied by tendon edge labels. Some data points were challenging to obtain at certain locations along the tendon length. In this representation, the colour

map, based on reference criteria outlined by ASTM C876 (ASTM C876, 2015), indicates that green areas correspond to potential measurements greater than -200 mV CSE, suggesting a 10% likelihood of active corrosion; yellow areas correspond to measurements ranging between -200 mV and -350 mV CSE, with an unknown likelihood of corrosion; and red areas correspond to measurements less than -350 mV CSE, indicating a 90% likelihood of active corrosion.

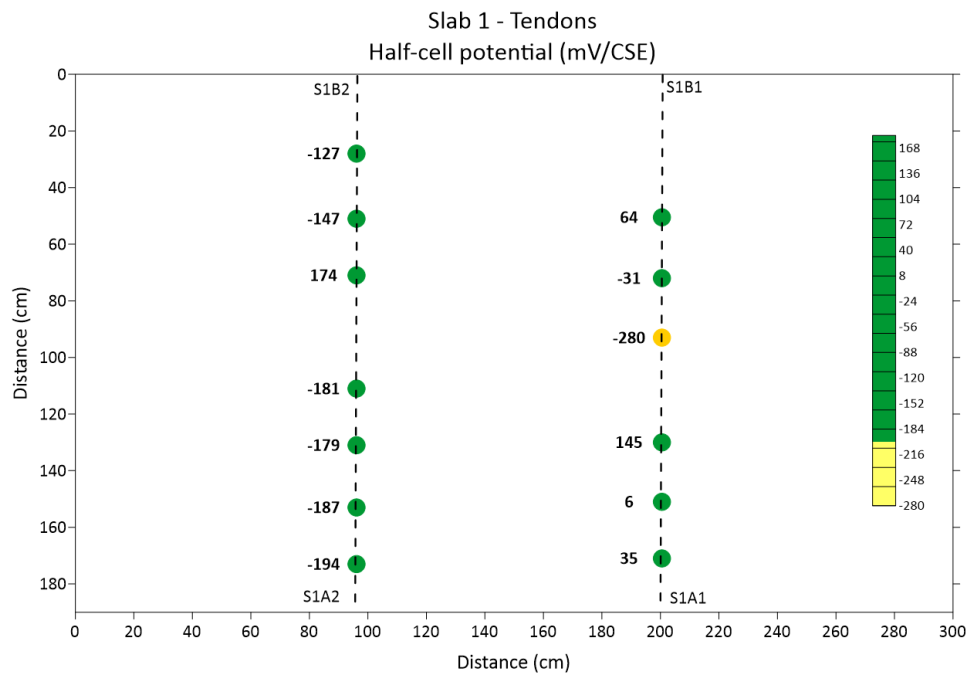


Figure 3-12. Half-cell potential measurements along prestressing tendons – S1

It can be seen that the probability of active corrosion for both tendons, S1A1-S1B1 and S1A2-S1B2, is generally less than 10% along their lengths.

### 3.4.6.2 Corrosion Rate

Figure 3-13 highlights corrosion rate measurements taken along the length of the transverse prestressing tendons for S1. The orientation and location of the embedded tendon are indicated by a dashed line, accompanied by tendon edge labels. Some data points were challenging to obtain at certain locations along the tendon length. In this representation, the colour map, based on reference criteria outlined by Andrade & Alonso (Andrade & Alonso, 1996), shows that green areas correspond to corrosion rate measurements less than  $0.5 \mu\text{A}/\text{cm}^2$  (low corrosion levels), yellow areas correspond to values between  $0.5$  and  $1.0 \mu\text{A}/\text{cm}^2$  (moderate corrosion levels), and red areas correspond to measurements exceeding  $1.0 \mu\text{A}/\text{cm}^2$  (high corrosion levels).

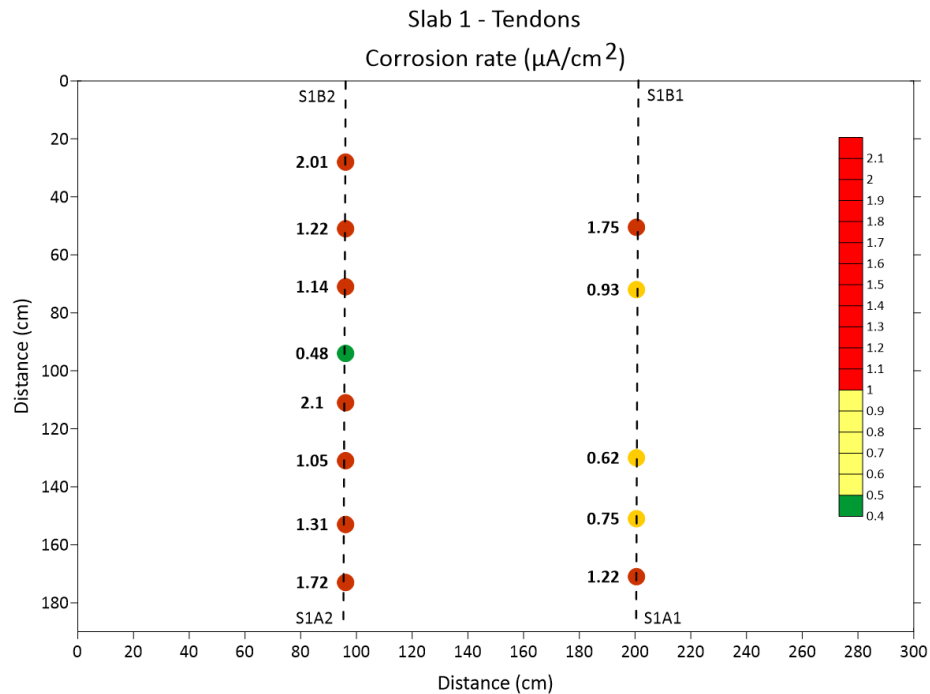


Figure 3-13. Corrosion rate measurements along prestressing tendons – S1

Corrosion rate measurements seem to deviate from the findings of half-cell potential assessments. Specifically, S1A1-S1B1 displays high corrosion rates near edges A and B, while the remaining sections exhibit moderate corrosion conditions. In the case of S1A2-S1B2, high corrosion rates are observed along most of its length.

### 3.4.6.3 Concrete Surface Resistivity

Figure 3-14 illustrates the concrete surface resistivity measurements acquired along the length of the transverse prestressing tendons for S1. The orientation and location of the embedded tendon are indicated by a dashed line, accompanied by tendon edge labels. Some data points were challenging to obtain at certain locations along the tendon length.

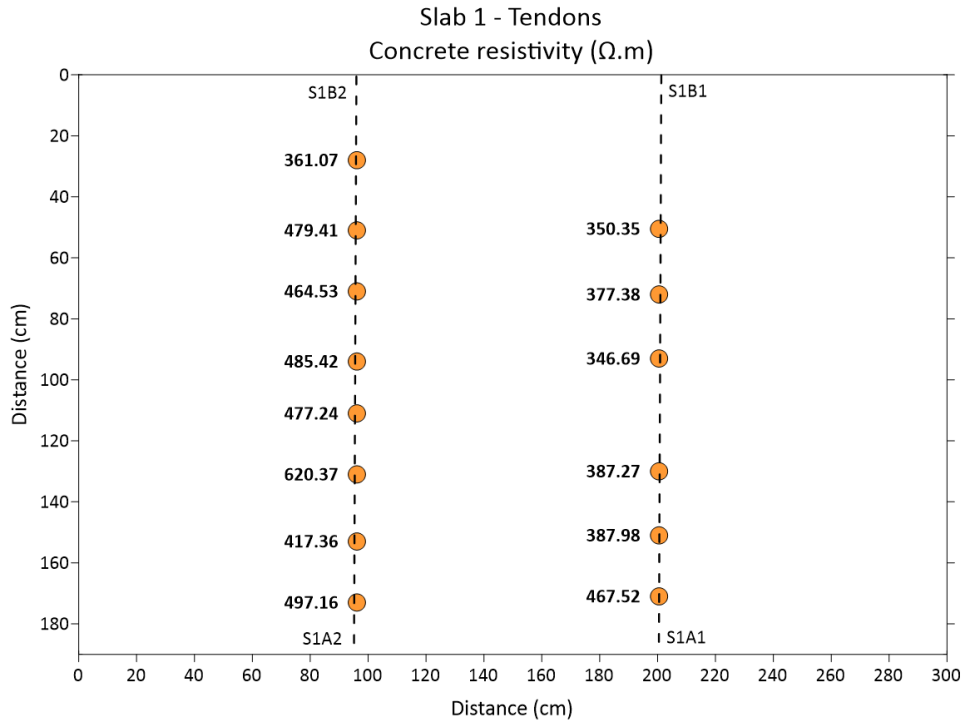


Figure 3-14. Concrete surface resistivity measurements along prestressing tendons – S1

Concrete resistivity values are expected to decrease in areas with high corrosion rates (Hornbostel et al., 2013). However, in the case of S1, no such trend was evident. Measurements conducted along the concrete surrounding both tendons consistently fell within the range of 346 – 620  $\Omega.m$ . This suggests low to very low chloride ion penetration abilities at locations experiencing high tendon corrosion rates, as per AASHTO TP-95 (AASHTO TP-95, 2014).

### 3.4.7 Ground penetrating radar

For the GPR, UPE, and IE tests, S1 was divided into two smaller slabs, each 1,000 mm wide and spanning 2,000 mm. Both slabs had one tendon from S1 centered along the width, as depicted in Figure 3-15. Each of the two slabs is designated as follows: S1-1 for the slab containing S1A1-S1B1, and S1-2 for the slab containing S1A2-S1B2. The side slab segments will be utilized in the future for extracting concrete cores and conducting material testing.

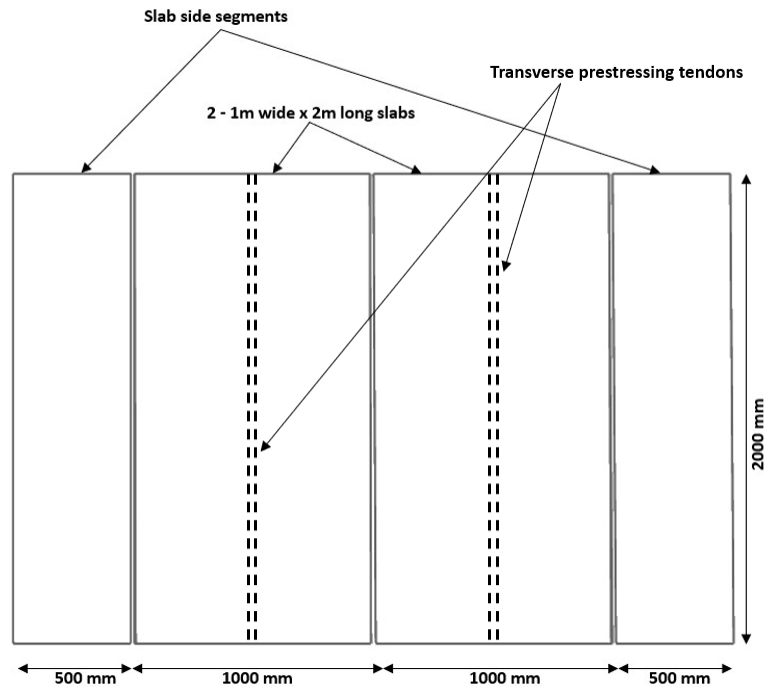
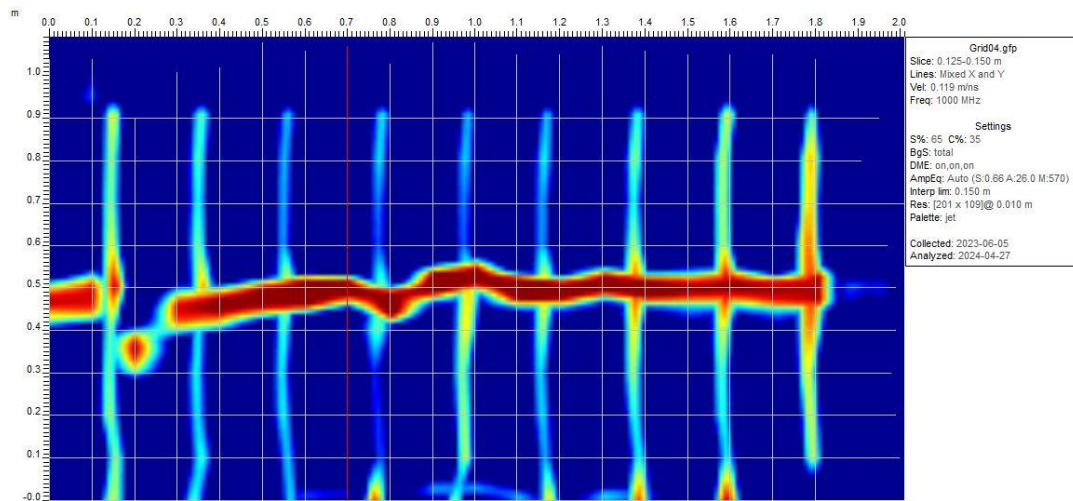
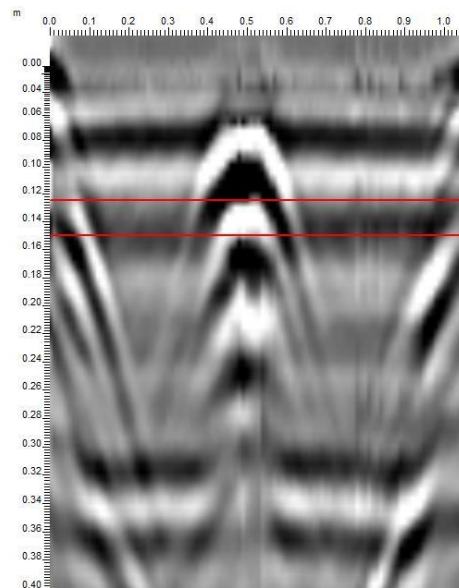


Figure 3-15. Plan view of S1 cut into two smaller slabs: S1-1 and S1-2

The resulting GPR soffit grid scans for S1-1 and S1-2, conducted at a slice depth of 0.125 – 0.150 m, are depicted in Figure 3-16 (a) and Figure 3-17 (a), respectively. Strong GPR signal responses are depicted by red areas, whereas intermediate and weak signal responses are represented by yellow and blue, respectively. Red signals typically denote high-intensity reflections, indicating a strong contrast in di-electric properties at an interface between two media, such as embedded reinforcement within the concrete. Yellow signals typically represent intermediate reflections, suggesting moderate contrasts in di-electric properties that point to less distinct features. Blue signals generally indicate low-intensity reflections, associated with more uniform concrete and less reflective areas.

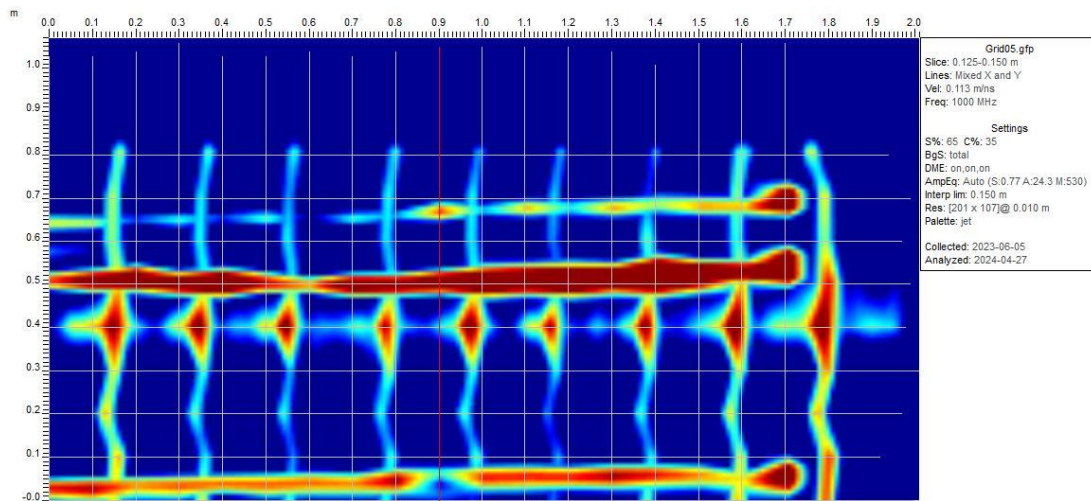


(a)

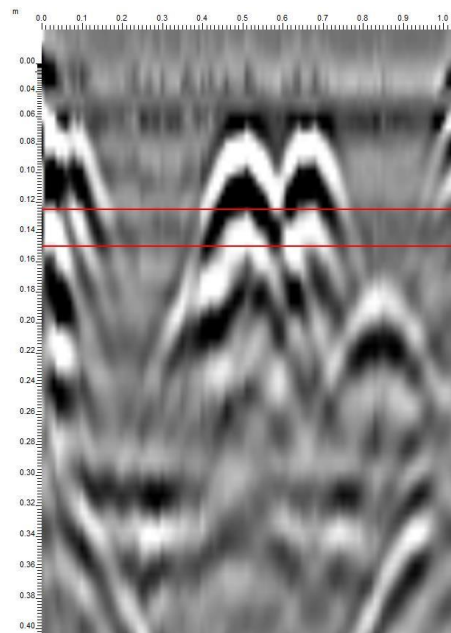


(b)

Figure 3-16. S1-1: GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 0.7 m slice view from edge A – Slab section view



(a)



(b)

Figure 3-17. S1-2: GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 0.9 m slice view from edge A – Slab section view

In Figure 3-16 (a) and Figure 3-17 (a) scans, the prestressing tendon is visibly situated approximately 0.5 m vertically from the 0.0 m reference point, spanning 2 m, and indicated by a red horizontal line. In the case of S1A1-S1B1, a discontinuity in the tendon is evident at a point 0.1 m from edge A, measuring 0.2 m in length. This discontinuity may be due to either a structural

defect, such as a rupture induced by corrosion at that specific location, or from interferences caused by nearby deeper secondary reinforcement spanning in the orthogonal direction and indicated by yellow and blue vertical lines.

For S1-1, Figure 3-16 (b) displays a slice view at 0.7 m from edge A, featuring a parabola with its peak representing the top surface of the tendon. This depiction reveals that the tendon is positioned 0.06 m below the slab soffit. Similar findings were observed for S1-2, as shown in Figure 3-17 (b), where a slice view at 0.9 m from edge A indicates a comparable depth of 0.06 m from the slab soffit to the top surface of the tendon. Additionally, Figure 3-17 (b) reveals two adjacent hyperbolas representing the tendon. The additional hyperbola is likely due to the presence of a thin metal duct layer, which permits the signal to penetrate through and reflect off the individual strands within the duct. Consequently, this phenomenon may potentially lead to misleading results, as it could be interpreted as an additional metal reinforcement parallel to the prestressing tendon.

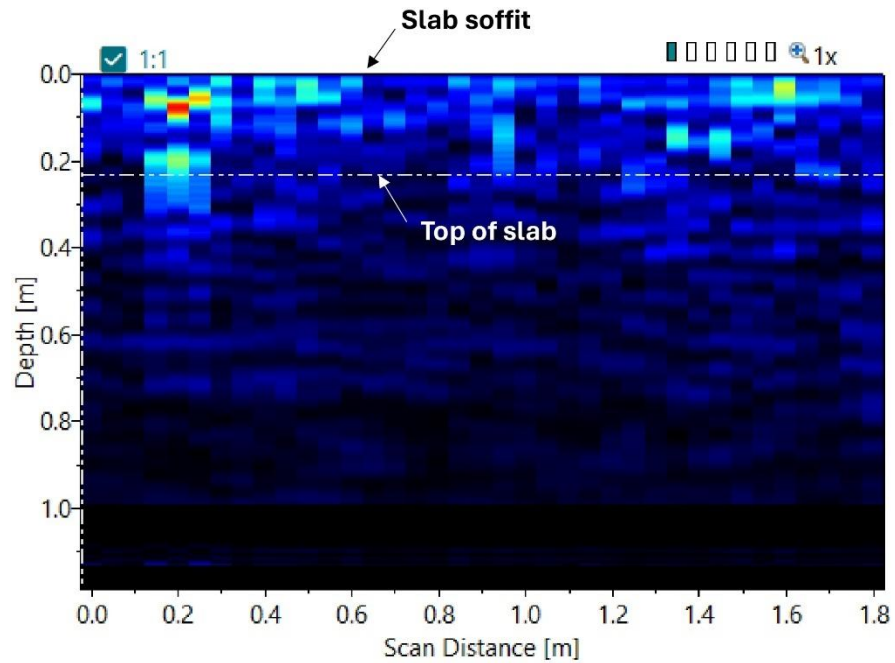
In both slice view scans, the slab depth (backwall) of 0.30 m is depicted as a horizontal dark line, aligning with the combined depth of the slab and asphalt surface. Furthermore, the GPR analysis successfully verified that the bottom concrete cover depth is approximately 0.06 m.

It is clear that the obtained results do not provide information on the presence of voids within the metal duct. It may also be worth noting that identifying areas of rupture caused by corrosion through the GPR grid scan may be challenging.

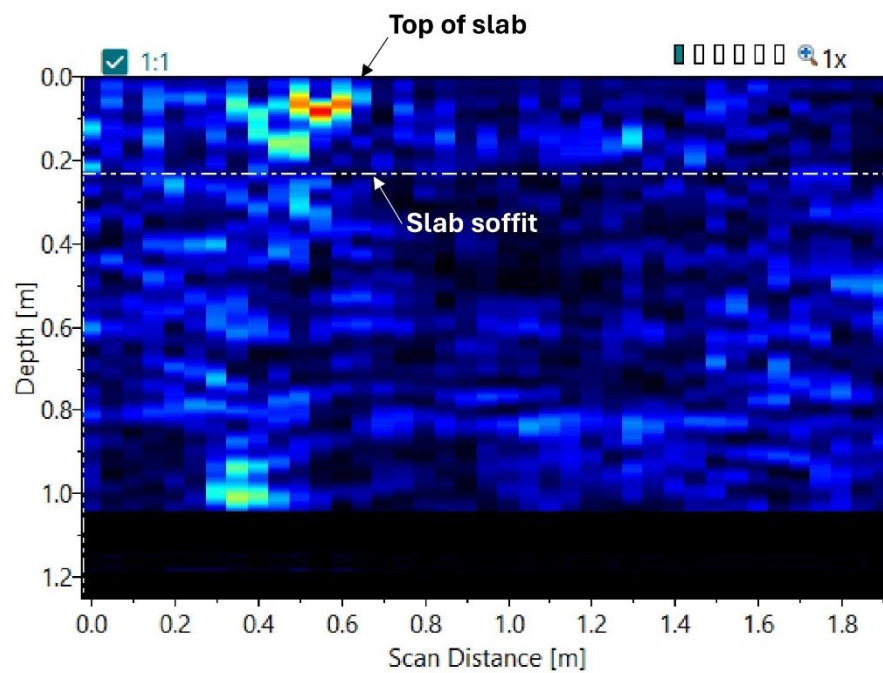
### **3.4.8 Ultrasonic pulse echo**

Before gathering UPE data, the upper asphalt layers for S1-1 and S1-2 were removed using a concrete jackhammer. Subsequently, UPE measurements were taken along the soffits and top concrete surfaces, covering the length of both tendons. Figure 3-18 and Figure 3-19 provide a sectional view of the slabs, where UPE strength signals along the tendons are displayed in colour format. Both sets of measurements were acquired along the centerline of the tendons and at 5 cm intervals, starting from edge A and progressing to edge B.



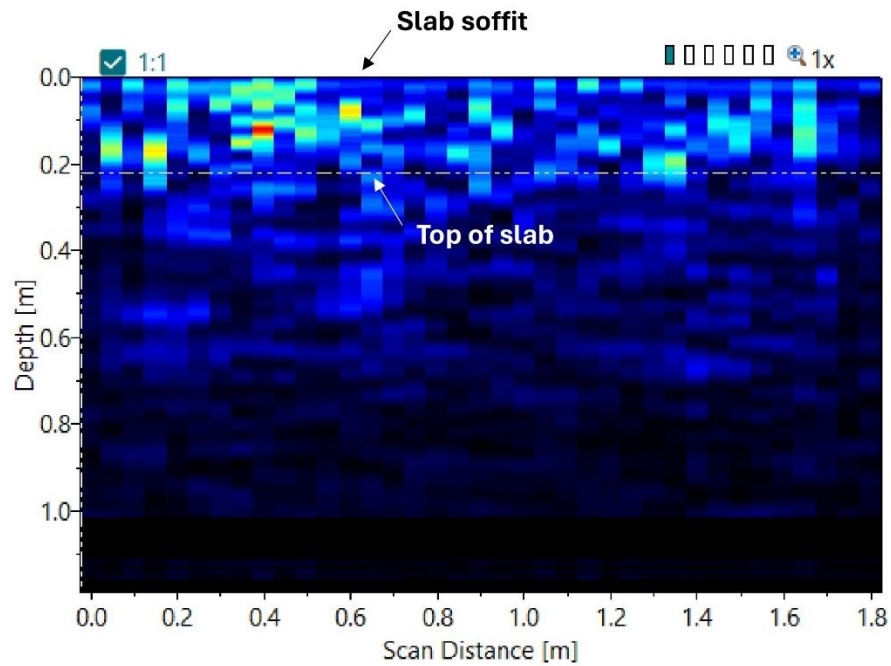


(a)

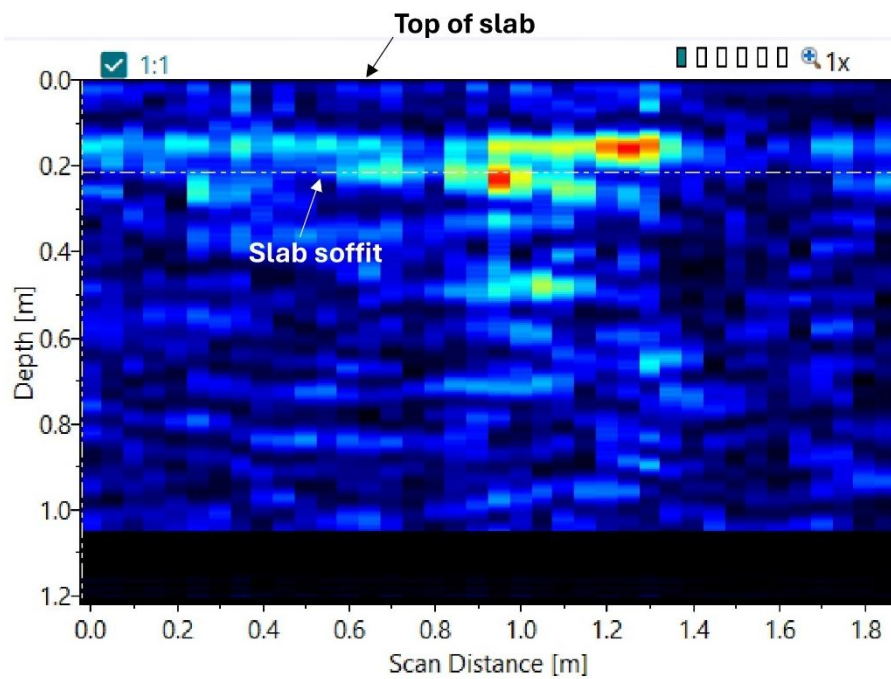


(b)

Figure 3-18. UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface of S1-1



(a)



(b)

Figure 3-19. UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface of S1-2

In UPE scans, colours are used to represent different signal strengths: red areas indicate strong UPE signal responses, yellow areas show intermediate responses, and blue areas represent weak responses. Red signals can either denote distinct and well-defined reflections from the back wall, where the signal reflects off the opposite concrete surface, suggesting good-quality concrete that is free from defects or internal flaws, or indicate scattered strong reflections from the internal flaws themselves, indicating lower concrete quality. Such concrete sections will lack a clear and well-defined back-wall reflection as a result of these internal reflectors (flaws) or surface degradation. Yellow signals suggest that the concrete in those regions may exhibit some level of deterioration or inconsistency, while blue signals suggest that the concrete is free from significant defects.

Strong and intermediate UPE signals obtained along S1A1 -S1B1 and S1A2-S1B2 and within the slab depths are represented by red and yellow squares in Figure 3-18 and Figure 3-19 and are detailed in

Table 3-5 and Table 3-6, respectively. The defects were numbered sequentially, starting from 1, with scanning originating at a distance of 0 m. For S1-1, given the concrete covers near edges A and B, and a nominal tendon diameter of 45 mm, it is possible that defects numbered 1 through 3 are situated within the grout for measurements taken along the slab soffit. However, defects 4 and 6 appear shallow, and defect 5 is located below the tendon, suggesting they could potentially indicate voids within the hardened concrete. Additionally, when measurements were taken from the top surface, all defects appeared shallow, indicating potential damage or voids near the concrete surface.

Regarding S1-2, for measurements taken from the slab soffit, and considering the concrete covers near edges A and B, it appears that defects 4 and 5 are within the tendon, while the remaining defects are situated deeper than the tendon and could indicate defects within the hardened concrete. When measurements were taken from the slab top surface, it was observed that defects 7 and 8 are within the tendon duct, whereas defects 9 through 12 are located deeper than the tendon and could suggest defects within the hardened concrete.

None of the measurements along both tendons showed a strong back-wall reflection, indicating poor concrete quality and internal defects. These issues were further confirmed by the strong and intermediate signals observed within the concrete depth.

Table 3-5. Summary of void locations along tendon length from UPE measurements – S1-1

| Surface measurement    | Clear concrete cover (mm) | Defect number | Depth (mm) | Distance from edge A (mm) |
|------------------------|---------------------------|---------------|------------|---------------------------|
| Along slab soffit      | 55 near edge A            | 1             | 80         | 200                       |
|                        | 78 near edge B            | 2             | 59         | 250                       |
|                        |                           | 3             | 57         | 500                       |
|                        |                           | 4             | 32         | 950                       |
|                        |                           | 5             | 141        | 1350                      |
|                        |                           | 6             | 43         | 1600                      |
| Along slab top surface | 101 (near edge A)         | 7             | 69         | 354                       |
|                        | 115 (near edge B)         | 8             | 53         | 500                       |
|                        |                           | 9             | 84         | 550                       |
|                        |                           | 10            | 59         | 600                       |

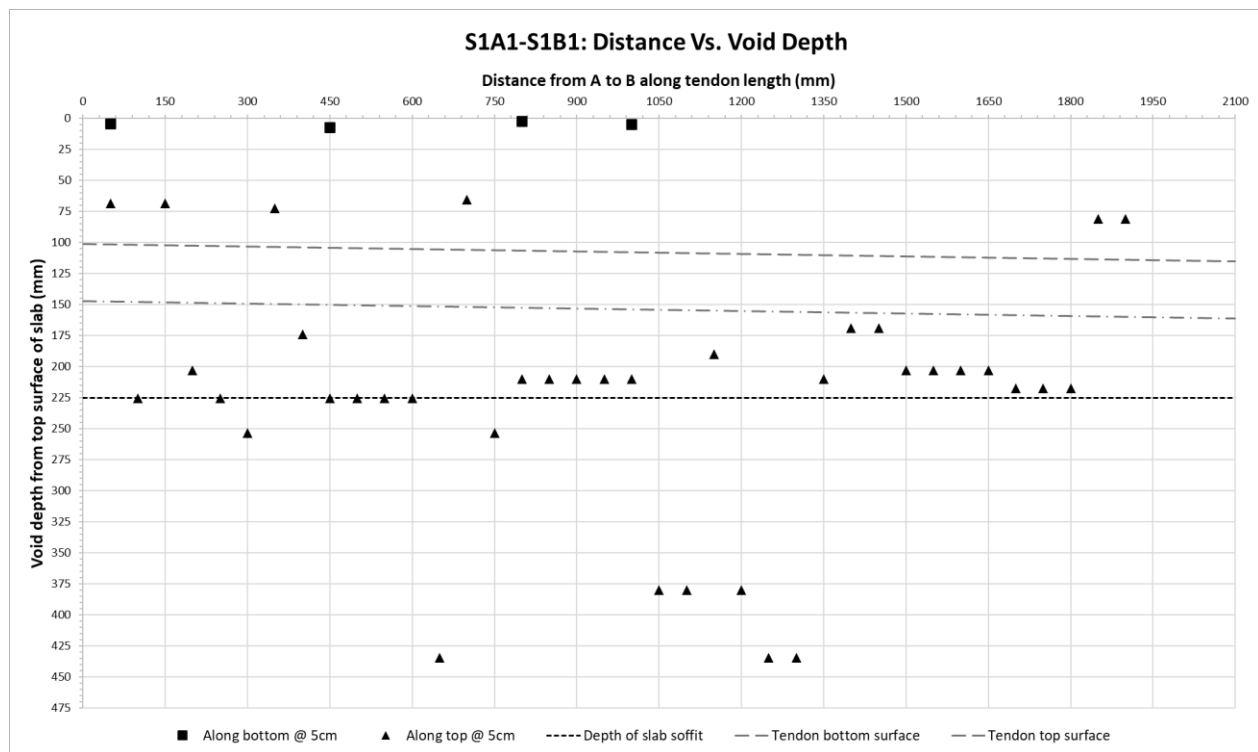
Table 3-6. Summary of void locations along tendon length from UPE measurements – S1-2

| Surface measurement    | Clear concrete cover (mm) | Defect number | Depth (mm) | Distance from edge A (mm) |
|------------------------|---------------------------|---------------|------------|---------------------------|
| Along slab soffit      | 67 near edge A            | 1             | 168        | 47                        |
|                        | 70 near edge B            | 2             | 176        | 151                       |
|                        |                           | 3             | 122        | 400                       |
|                        |                           | 4             | 81         | 596                       |
|                        |                           | 5             | 76         | 900                       |
|                        |                           | 6             | 197        | 1344                      |
| Along slab top surface | 102 (near edge A)         | 7             | 140        | 1646                      |
|                        | 115 (near edge B)         | 8             | 151        | 345                       |
|                        |                           | 9             | 215        | 673                       |
|                        |                           | 10            | 215        | 848                       |
|                        |                           | 11            | 257        | 947                       |
|                        |                           | 12            | 257        | 1249                      |

### 3.4.9 Impact echo

The graphs in Figure 3-20 and Figure 3-21 display the void locations along S1A1-S1B1 and S1A2-S1B2 detected during testing, represented as points along the distance axis. Each point on the distance axis represents a void identified through post-processing analysis of the device's data. Waves produced by the stress pulse reflect off internal or external interferences and travel back to the surface impact point. Post-processing techniques are then applied to determine the depth of discontinuities or establish the boundary of the member. Similarly to UPE scans, regions containing internal flaws exhibit either the absence of a back-wall reflection due to internal reflectors, surface

degradation, or both, or signal reflections indicating the depth of the flaw. The findings are also summarized in Table 3-7 and Table 3-8.



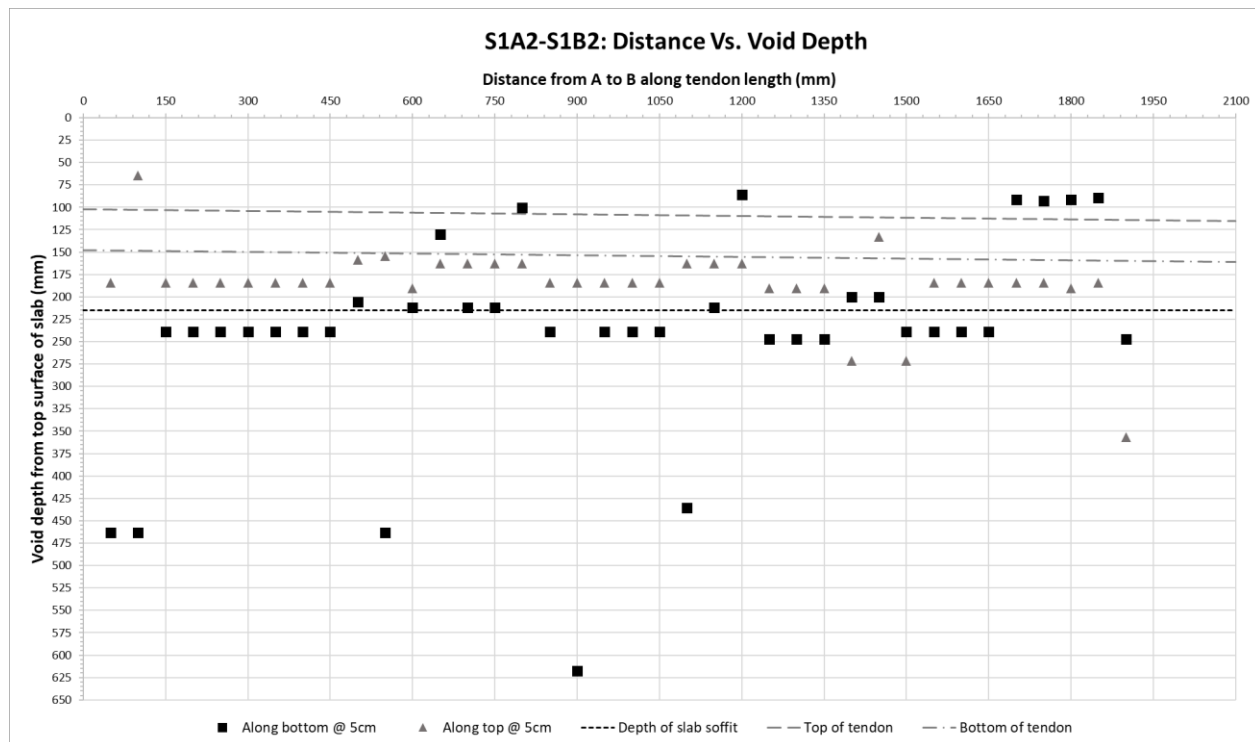


Figure 3-21. IE data at 5 cm intervals along S1A2-S1B2 from both slab surfaces

Table 3-7. Location of grouting void along tendon length from IE measurements – S1-1

| Surface measurement    | Clear concrete cover (mm) | Defect number | Depth (m) | Distance from edge A (m) |
|------------------------|---------------------------|---------------|-----------|--------------------------|
| Along slab top surface | 101 (near edge A)         | 1             | 0.168     | 1.400                    |
|                        | 95 (near edge B)          | 2             | 0.168     | 1.450                    |

Table 3-8. Location of grouting void along tendon length from IE measurements – S1-2

| Surface measurement    | Clear concrete cover (mm) | Defect number | Depth (m) | Distance from edge A (m) |
|------------------------|---------------------------|---------------|-----------|--------------------------|
| Along slab soffit      | 55 near edge A            | 1             | 0.130     | 0.650                    |
|                        | 78 near edge B            | 2             | 0.100     | 0.800                    |
| Along slab top surface | 102 (near edge A)         | 3             | 0.132     | 1.450                    |
|                        | 99 (near edge B)          |               |           |                          |

IE measurements for S1A1-S1B1 from the top surface revealed multiple shallow and deep defects, with only two defects (numbered 1 and 2) identified in the vicinity of the tendon duct, 1.400 m from edge A. The remaining measurements primarily showed a strong backwall signal, suggesting

minimal or no damage at those points. Conversely, measurements from the bottom surface only highlighted the backwall as the slab's top surface.

For S1A2-S1B2, IE measurements taken from the top concrete surface were concentrated near the slab soffit, displaying strong backwall signals. Only one defect was identified within the tendon ducts from the top measurements, located at 1.450 m from edge A. In contrast, measurements from the bottom surface mostly exceeded the depth of the slab, indicating possible surface degradation (Borgerson & White, 2021). Additionally, two measurements suggested grouting defects within the tendon, located at 0.650 m and 0.800 m from edge A, while the rest indicated concrete damage around the tendon.

### **3.5 Discussion**

#### **3.5.1 Damage features and internal concrete quality**

Despite its qualitative and somewhat subjective nature, visual inspection is crucial for decision-making and identifying areas for further assessment. In this study, visible damage observed along S1A1-S1B1 and the slab soffit, characterized by efflorescence and narrow cracking, is in line with UPV findings suggesting a poor to doubtful condition along the tendon length. However, CI results indicated negligible damage, which does not align with the visible deterioration observed. This discrepancy highlights the importance of comprehensive assessment techniques for accurately evaluating concrete structural integrity, especially since visual inspection is often the initial step in the assessment process. Concerns arise with accelerated cross-sectional area loss in smaller-diameter prestressing steel wires, particularly when visible damage is minimal. These findings emphasize the complexity of assessing prestressed concrete elements, as surface-level observations may not reveal underlying structural issues.

Additionally, while the rebound hammer indicates similar internal quality for both slabs, its inability to detect internal defects underscores the need for complementary assessment methods.

#### **3.5.2 Corrosion assessment techniques**

Half-cell potential measurements for both tendons indicate a low likelihood of corrosion in areas with high corrosion rates, while surface resistivity measurements demonstrate a poor correlation with corrosion rates, seen in increasing resistivity values as corrosion rates increase. As a result, ASTM C876 (ASTM C876, 2015) guidelines on recommending half-cell potential measurements

as an initial assessment of corrosion probability in reinforcement may potentially provide uncertain data on corrosion conditions. Therefore, integrating supplementary NDTs with half-cell potential readings becomes essential for a comprehensive evaluation during structural corrosion assessments in the field. Timely detection of corrosion activity in prestressed structures is crucial for implementing appropriate repair and rehabilitation measures. Corrosion measurements for individual cables within both tendons may reflect the condition of either the surrounding metal duct or the cables themselves. Further research will be conducted to verify this relationship.

### **3.5.3 Tendon grout assessment techniques**

The GPR findings do not provide clarity on voids within the metal duct due to potential signal reflection caused by the thin layer of metal duct and nearby secondary reinforcement. Hence, caution is necessary when interpreting GPR results. However, it is important to note that the GPR was successful in determining the cover thickness of the tendons.

UPE measurements from both top and bottom surfaces of the same slab yielded non-aligned results, as expected, due to complete wave reflection at defects, identifying different defects on each surface. However, this dual-surface approach offers a valuable method for comprehensive assessment of defects around or within tendon ducts.

IE measurements from opposite surfaces of the same slab revealed different defects, highlighting the importance of multi-surface assessments for identifying a broader range of anomalies. However, complexities and limitations of the method remain, particularly in distinguishing between different types of defects and accurately assessing their depth and extent.

Although the IE and UPE techniques are seen as complementary in identifying potential defects, the results from both tests on both tendons did not align as anticipated, as each technique has distinct limitations, as presented in Table 3-2 and discussed in a previous publication (Tawil et al., 2023b). This highlights the necessity for further research to develop a method to evaluate tendon grout and directly assess the level of corrosion.

### **3.5.4 Multi-technique assessment**

In the evaluation of both tendons, correlations were observed among the various NDTs, despite differences noted during visual inspections. Some tests, particularly those targeting concrete quality, aligned with corrosion rate measurements, highlighting the impact of corrosion on concrete



degradation. For instance, areas with doubtful concrete quality, as indicated by UPV values, corresponded to either low or moderate corrosion rates, while regions with poor UPV conditions were associated with high corrosion rates. Furthermore, areas exhibiting high corrosion rates or moderate rates nearing the high threshold were successfully identified as defective by both UPE and IE methods, offering a general indication of defect locations.

However, not all NDTs are complementary; some are designed to assess concrete quality and embedded tendons separately. Despite this, the combination of results from various tests remains crucial for providing a comprehensive evaluation of concrete elements. Each test contributes a vital piece to the overall assessment, underscoring the importance of a multi-technique approach in accurately evaluating structural integrity. These findings highlight the complexity of NDT assessments and the need for continued research to refine evaluation techniques and their interpretation for more precise assessments.

### **3.6 Conclusions**

The application of diverse NDTs in evaluating the corrosion effects of a slab from the former Champlain Bridge in Montréal, subjected to severe environmental conditions over several decades, has yielded significant insights. From the results obtained, the following conclusions can be drawn:

- Despite its qualitative nature, visual inspection remains crucial for initial assessments. However, it should be supplemented with NDTs.
- Concerns arise regarding the accuracy of corrosion assessments, with half-cell potential measurements potentially providing uncertain data on corrosion conditions. This is particularly concerning since guidelines such as ASTM C876 (ASTM C876, 2015) recommend half-cell potential measurements as a preliminary corrosion assessment. Therefore, it is important to supplement half-cell potential measurements with other corrosion-targeting NDTs during structural corrosion evaluations.
- Interpretation of GPR findings requires caution due to potential signal reflections, highlighting the need for careful analysis.
- The non-aligned results from UPE and IE measurements on both top and bottom surfaces highlight the value of a dual-surface approach.
- While a comprehensive assessment does not require utilizing every NDT method, relying on a single technique is insufficient due to the inherent uncertainties and limitations of individual

methods. Instead, a strategic selection of complementary NDT tools must be used to gather a complete dataset. Consequently, FEA, as a secondary step in the analysis, cannot be successfully executed or calibrated without incorporating these NDT results, as they provide the essential empirical data needed to capture the actual, in-situ structural conditions.

- A multi-technique assessment is essential for a comprehensive evaluation of corrosion activity and damage in prestressed structures, emphasizing the importance of timely detection for implementing appropriate repair measures. Defining correlations among the various NDTs employed in this assessment is crucial. Further research is needed to define these correlations more precisely.
- Complexities and limitations observed in current NDTs underscore the necessity for ongoing research to refine and improve evaluation techniques.

Further experimental research is underway to quantify the damage to both the concrete and tendons through destructive methods.

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## **Chapter 4: Experimental investigation of the residual structural performance of naturally corroded post-tensioned bridge deck slabs**

### **Abstract**

This study examines the residual structural performance of prestressed concrete bridge deck slabs extracted from the former Champlain Bridge after nearly 60 years in service. Fifteen one-way post-tensioned slabs, exhibiting varying degrees of tendon corrosion, were tested under four-point bending with partially fixed boundary conditions to evaluate their ultimate capacity, stiffness, ductility, and failure modes. Digital image correlation (DIC) and sectional stress analysis were used to assess pre-compression effects, bond behaviour, and crack propagation. Results showed that compressive membrane action enhanced load capacity, particularly in the inelastic range, but its influence diminished after cracking. Localized tendon corrosion and poor grout quality significantly reduced stiffness and bond, leading to brittle shear failures, while well-grouted tendons exhibited higher ductility and energy dissipation. The findings highlight the critical role of tendon integrity and grouting in maintaining the long-term performance of prestressed concrete slabs and provide valuable data for durability-based assessment and modelling of aging post-tensioned structures.

## 4.1 Introduction

Many of the bridges built during the post-World War II construction boom from the 1950s to the 1970s are now approaching the end of their effective service life. During this period, prestressed concrete (PC) emerged as a popular choice for bridge construction worldwide due to its versatility and ability to accommodate both short and long spans (V. S. Nguyen et al., 2013). However, as these PC bridges age, concerns regarding their long-term structural integrity and safety are increasing. Material degradation and potential construction deficiencies, especially in the post-tensioning tendons and grouting systems, can significantly compromise their load-carrying capacity. Due to the vital role of these bridges in transportation networks and the potential consequences of their failure, it is critical to regularly assess their condition and evaluate their structural performance. These issues highlight the urgent need for research—both experimental and numerical—to better understand these effects and guide future rehabilitation or replacement efforts.

The prevalence of corrosion in PC bridges is a global concern, though its severity varies across regions, leading to divergent approaches in post-tensioning systems. In the United Kingdom, for example, a moratorium on internally bonded post-tensioned (PT) structures was established in 1992 following several bridge failures, including the notable collapse of the Ynys-y-Gwas Bridge (Al-Mosawe et al., 2022; Moreton, 2001). This moratorium remains in effect for "internal, grouted tendons with discontinuous (poorly sealed) ducts" (Moreton, 2001). Similarly, countries such as Germany and Japan have expressed a preference for external prestressing systems (Herbrand & Hegger, 2013; Mutsuyoshi, 2001). These diverse approaches reflect the global concern regarding PT tendon corrosion and underscore the need for comprehensive studies to develop consistent and effective protection and maintenance strategies.

In both the United States and Canada, PC technology continues to be widely used, despite some notable cases of tendon corrosion. In the United States, Florida has been a hotspot for such incidents, including the Niles Channel Bridge (1999), the Mid-Bay Bridge (2000), and the Sunshine Skyway Bridge (2000) (Salas et al., 2004). In Canada, while there have been no reported bridge failures or collapses attributed to corrosion (Public Safety Canada, 2013), some PC bridges, such as the former Champlain Bridge in Montreal (1962) (Delcan, 2011) and the Frederick G. Gardiner



Expressway in Toronto (1966) (Doug et al., 2022), have exhibited signs of corrosion and required extensive rehabilitation.

These experiences have prompted transportation agencies and technical societies to develop emergency guidelines and technical reports (Fédération Internationale du béton (fib), 2023; Post-Tensioning Institute & (PTI), 2001). However, further research is still required to establish a comprehensive standard for durability-based design, construction, assessment, and monitoring of PC infrastructure.

PC bridge deck slabs act as the first line of defense for a bridge against the damaging effects of winter de-icing salts. Although this post-tensioning technology experienced rapid development in the post-war era, durability considerations were often overlooked during design. This has led to significant durability-related issues. When constructing bonded PC structures, it is crucial to fully grout the ducts following strand tensioning. This process achieves two primary purposes: establishing a bond between the strands and the surrounding concrete and protecting the prestressing steel wires with alkaline environments from exposure to water and chlorides, which can lead to reinforcement corrosion.

A major challenge faced by agencies today is the lack of effective monitoring and inspection techniques for bonded post-tensioned structures. Most condition surveys rely on visual inspections to identify surface-level issues such as cracking, spalling, and rust staining. However, these methods often fail to detect hidden deterioration in prestressing steel, leaving critical underlying issues unnoticed and increasing the risk of sudden and catastrophic collapses.

#### **4.2 Background: Champlain Bridge**

The former Champlain Bridge (1962 – 2019) in Canada was a vital infrastructure connecting the island of Montreal to Brossard (south shore) over the Saint Lawrence River. Opened in 1962, it served as one of Canada's busiest crossings, accommodating over 50 million vehicles annually and playing a crucial role in facilitating trade and transportation between Canada and the United States (Carlin, 2017; COWI, 2017).

The bridge, notable for its innovative design at the time of construction, featured two concrete approach spans and an iconic steel truss cantilever main span. However, prolonged exposure to chloride ingress from extensive application of de-icing salts during Quebec's harsh winters,

combined with increasingly heavy traffic, caused significant deterioration over the bridge's six decades of service, necessitating frequent and extensive maintenance and rehabilitation efforts. The superstructure's deck system was directly exposed to these environmental stresses, with inadequate drainage exacerbating the deterioration of its components. In 2019, it was replaced by the Samuel De Champlain Bridge, and the original structure was decommissioned and dismantled.

The deck design of the bridge's south approach span consisted of seven prestressed concrete I-girders connected by six transversely post-tensioned cast-in-place infill slabs (Figure 4-1). This configuration relied on transverse prestressing tendons to establish continuity between the infill slabs and the top flanges of the girders, ensuring efficient load transfer. However, the design also introduced a potential vulnerability, as water infiltration at the girder-slab joints could compromise the tendons and weaken the system.

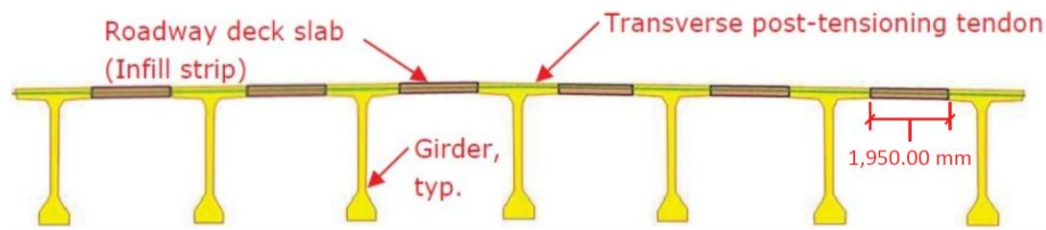


Figure 4-1 Typical deck cross-section at Brossard approach span (Carlin, 2017)

The resulting deck structure featured a highly integrated design in both transverse and longitudinal directions, enabling monolithic behaviour between girders, infill slabs, and diaphragms. This approach was the most cost-effective among 28 alternatives considered during the original design (Delcan, 2011). However, this integration, combined with poor drainage, contributed to persistent and widespread deterioration of the superstructure. Rehabilitation efforts were largely focused on risk mitigation rather than full restoration (Buckland & Taylor, 2013; COWI, 2017).

Despite repeated maintenance throughout its 57-year service life, the bridge was eventually closed and demolished due to unresolved deterioration concerns. While corrosion from de-icing salts was the primary issue observed during inspections, other mechanisms such as freeze-thaw damage and alkali-aggregate reaction may have also contributed to damage of various structural elements at the microstructural level.

The bridge was constructed during a transitional period in bridge design and winter maintenance practices. At the time, chloride-laden runoff was not accounted for in deck design, air entrainment for freeze-thaw resistance was still evolving, and grouted post-tensioned tendons were relatively new. These factors made structures from this era, including the Champlain Bridge, particularly vulnerable to premature deterioration, often resulting in reduced mechanical performance and structural functionality over time (Noël et al., 2018).

### **4.3 Materials and methods**

Fifteen naturally corroded PT concrete slabs were tested at the University of Ottawa's Structures Laboratory. The slabs were extracted in 2019 during the deconstruction of the bridge, which had been in service for 57 years. This study is part of a larger effort to comprehensively evaluate the condition and performance of the corroded deck slabs. This paper presents the results of structural testing on fifteen corroded PT slabs and the findings from a corrosion assessment of the PT tendons.

All obtained slabs had similar geometric properties and identical reinforcement layouts. The slab depths ranged from 200 to 215 mm deep, with dimensions of 1,950 mm (perpendicular to traffic direction) by 3,000 mm (parallel to traffic direction), topped by a 50 mm asphalt layer. The grouted post-tensioning tendons included twelve 7-mm diameter wires placed within a metal duct and spaced at an average of 1,000 mm apart. The slabs functioned as a one-way system, spanning 1,950 mm between the girders. The design also incorporated secondary mild deformed steel reinforcement for crack control in the direction perpendicular to the tendons. This reinforcement consisted of 10M bars spaced at 200 mm on center along the top of the slab, and 15M bars with an average spacing of 150 mm on center along the bottom. It is also worth noting that the cutting of the transverse tendons at the time the slabs were extracted may have contributed to localized grout damage near the joints and slab ends, as the sudden release of prestress allowed the steel to recover its elastic deformation and slippage inward. Consequently, with the effective prestress lost, the member behaves essentially as a conventionally reinforced concrete slab.

#### **4.3.1 Design of test specimens**

The fifteen PT concrete slab specimens were tested experimentally to assess the effects of post-tensioning tendon corrosion and bond behaviour on the structural capacity of the slabs. To simulate field conditions for the structural testing phase, where the individual infill slabs span 1,950 mm and function as a one-way slab system, the specimens were cut to 1,950 mm in length and 1,000

mm in width, with the tendons centered in each slab and spanning the full slab length. A sequential labeling system was used to identify the original eight slabs obtained from the bridge (e.g., 'S1' for Slab 1) and a second number is used to indicate the two test specimens (and their tendons) that were cut from those slabs. For instance, 'S1A1' denotes tendon 1 on edge A of slab 1, while 'S1B1' refers to tendon 1 on edge B of the same slab. Edge A corresponds to the side of the slab facing the lowest point of the deck profile, near the edge girders, as the bridge deck slopes transversely. Figure 6-2 presents a schematic diagram of the typical slab geometry, including key dimensions and reinforcement details. Based on experimental testing, the prestressing tendons, in an uncorroded state, exhibited a yield strength of 1,420 MPa and a nominal ultimate strain,  $\epsilon_u$ , of 6.76%. Figure 4-3 illustrates the typical tendon cross-section details, showing that the wires are grouped closely together, with some grout present between them, all encapsulated by grout within the duct. Figure 4-4 shows a photograph of the slab specimens prior to testing, positioned with the soffit facing upwards to facilitate inspection.

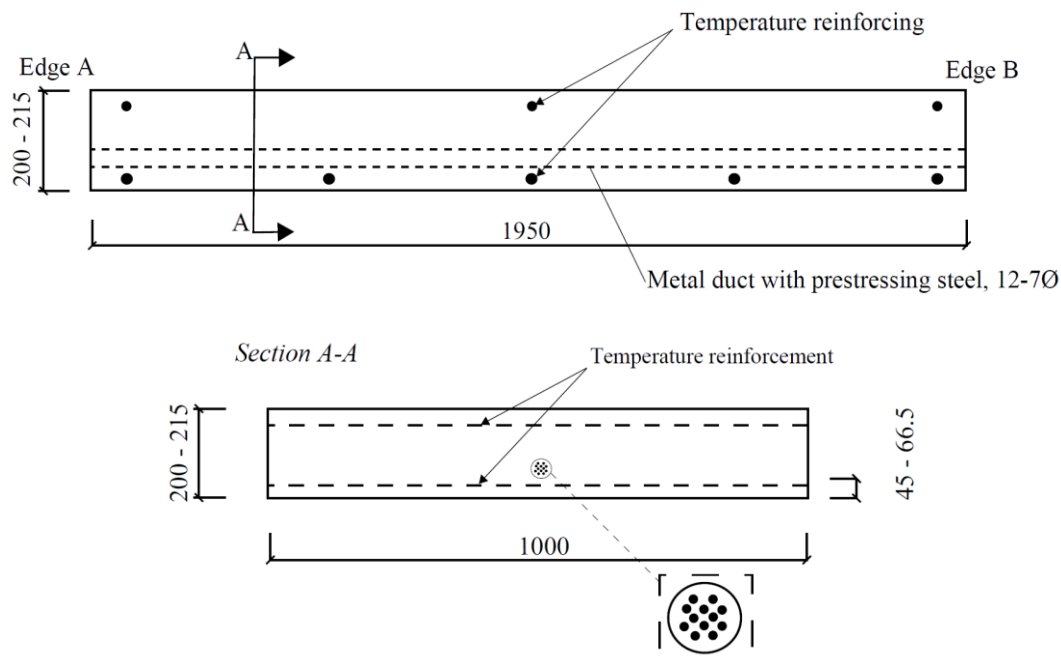


Figure 4-2 Slab geometry and reinforcement details (all dimensions are in mm)



Figure 4-3 Typical tendon cross-section



Figure 4-4 Full-scale prestressed concrete slab specimens prior to testing (inverted)

Prior to testing, the slabs underwent an extensive and rigorous non-destructive testing (NDT) program to evaluate the condition of the concrete, tendons, and the integrity of the grout within the metal ducts. The results from these NDT assessments on two slabs are discussed in Chapter 3.

#### 4.3.2 Test setup

Each slab was subjected to a quasi-static displacement-controlled four-point bending test to quantify its residual capacity. This testing method was chosen to investigate the behaviour of the slabs in constant moment and shear regions. The procedure adhered to the guidelines of ASTM C78 (ASTM C78/C78M, 2022). The boundary conditions for the test setup, shown in Figure 4-5, were designed to replicate real-world conditions, where the slabs were laterally constrained by stiff supporting girders. Two symmetrical loads were applied via a steel beam positioned at one-third of the span from each support. A servo-hydraulic actuator with a nominal maximum capacity of

2,000 kN was used to apply the load incrementally under displacement control at a rate of 0.025 mm/min until failure.

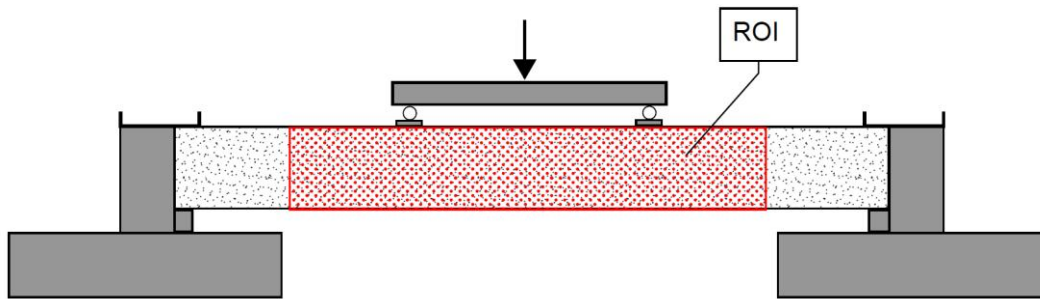


Figure 4-5 Test setup configuration: four-point bending test and region of interest (ROI) for DIC analysis



(a)



(b)

Figure 4-6 Test setup (a) without and (b) with slab, front rod and instrumentation



The steel supports were rigidly anchored to the strong floor of the Structures Laboratory to generate support fixity (Figure 4-6). The boundary conditions consisted of structural steel HSS (Hollow Structural Section) components that were welded together and securely connected to the strong floor using high-strength anchors. The slab rested on a 100 mm section of the top flange of an HSS 254x254 member and was clamped at the top using a C-section, which was attached to the HSS assembly through anchor rods and bolts. High-strength grout was also placed at both ends of the slab to fill the gap between the slab and the adjacent HSS sections. Additionally, two steel rods were positioned longitudinally at the front and back faces of the slabs, bolted to the HSS assembly. An actuator, equipped with a load cell, applied an increasing force to a steel spreader beam positioned at the slab's midspan. The load was then transferred through the spreader beam to the slab by two plate-and-roller assemblies. This setup ensured that the applied load was evenly distributed across the width of the slab.

This test configuration induces arching action in the slabs, similar to when vehicle loads are applied to the bridge deck. The arching phenomenon in concrete elements arises due to the significant disparity between its tensile and compressive strengths, as illustrated in Figure 4-7. Under loading, cracks form beneath the applied load due to the material's relatively low tensile strength. When the slab edges are restrained against lateral movement by a stiff supporting boundary, an internal arching behaviour, known as compressive membrane action, develops as the slab deflects.

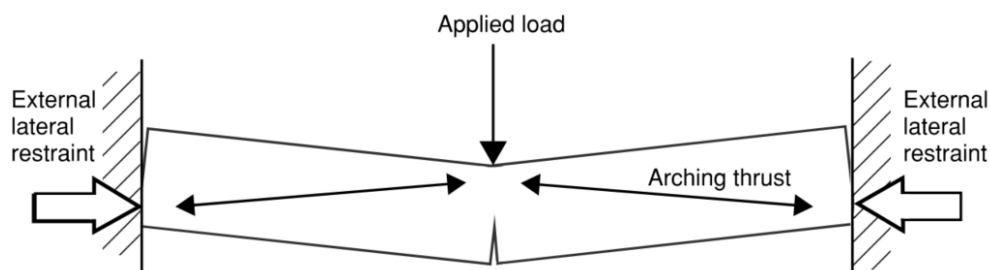


Figure 4-7 Arching behaviour in a laterally restrained reinforced concrete slab (reproduced from Taylor et al., 2001)

To monitor the nonlinear behaviour of the PC slabs throughout the loading process, digital image correlation (DIC) was employed. A high-contrast speckle pattern was applied to the front surface of each slab within a region of interest (ROI, selected as either the shear span or constant moment region), highlighted in Figure 4-5. Constant illumination was maintained to prevent any

interference from shadow effects. A single high-resolution digital camera, integrated into a DIC system produced by Zeiss Industrial Quality Solutions, captured images of the slab surface during testing. The images were post-processed using the two-dimensional software package Zeiss Correlate (GOM Company, 2015). The post-processing phase consisted of a frame-by-frame analysis, where the initial undeformed image of the beam served as a baseline for comparison with subsequent images captured at 5-second intervals throughout the loading process. This allowed for the determination of full-field displacements and strains.

In addition, Linear Variable Differential Transformers (LVDTs) were used to measure the vertical displacements of the slabs throughout the load test. Two LVDTs were placed at the midspan of the slab, one on the front face and one on the back face. Electronic strain gauges were utilized to measure the concrete strains during the flexural tests. The 60-mm-long strain gauges were strategically positioned on the slab surfaces at locations where maximum compressive strains were anticipated. These positions included the midspan of the slab (on the top surface) and areas near the slab supports (on the slab soffits). At each of these critical locations, three strain gauges were arranged across the width of the slab to ensure comprehensive strain measurement, resulting in a total of nine strain gauges used across the entire setup. Additionally, each rod positioned at the front and back faces of the slab was equipped with a strain gauge at its midspan as an indicator of the compression force applied to the system during the tests. Real-time data was collected using a data acquisition system linked to each of the strain gauges and LVDTs during the load test.

## **4.4 Results and discussion**

### **4.4.1 Load-deflection responses and energy-dissipating capacities**

The load-deflection and energy-dissipating capacity curves of the 15 specimens, categorized by their failure modes, are presented in Figure 4-14 for comparative analysis. Examination of the load-displacement curves and crack patterns revealed three distinct failure modes: flexural failure (observed in slabs S1A1-S1B1, S1A2-S1B2, S2A2-S2B2, S3A1-S3B1, S3A2-S3B2, S4A2-S4B2, S4A3-S4B3, S5A2-S5B2, S5A3-S5B3, S6A2-S6B2, and S7A2-S7B2), shear failure (observed in slabs S2A3-S2B3, S6A3-S6B3, and S8A3-S8B3), and a combined flexural-shear failure (observed in slab S7A3-S7B3). Representative images of the three failure modes are shown in Figure 4-8 to Figure 4-10.



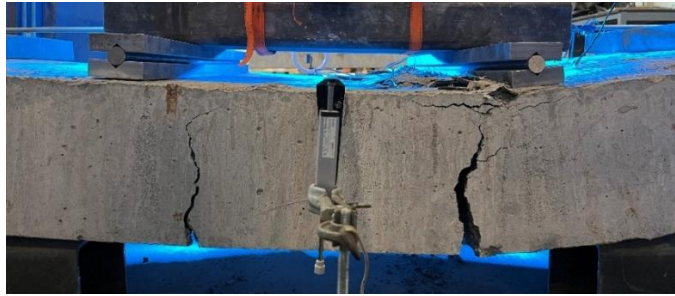


Figure 4-8 Flexural failure observed in slabs (S1A2-S1B2)

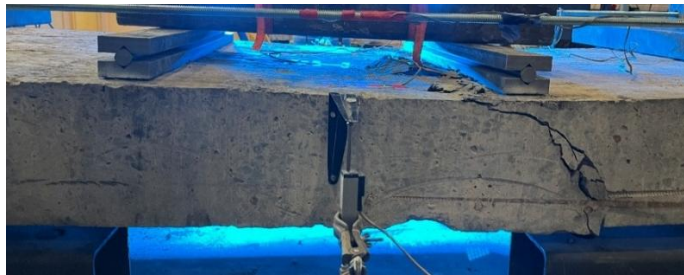


Figure 4-9 Shear failure observed in slabs (S2A3-S2B3)



Figure 4-10 Flexural-shear failure observed in slabs (S7A3-S7B3)

To determine the compressive strength of the concrete, two cylindrical cores were extracted from undamaged side segments of each slab after they were cut to 1 m widths from their original 3 m width. These cores were then tested in compression.

The key experimental results, including shear span-to-depth ratio, cracking load, ultimate load, deflection at ultimate and failure/end of test, energy-dissipating capacity, initial stiffness, deflection ratio, and failure mode are summarized in Table 4-1 for all tested specimens and are categorized by slab failure modes. Furthermore, Table 4-2 provides a summary of the load-deflection curve for each slab, where 'a' denotes the shear span of the specimen, accompanied by an image illustrating the slab's condition at failure.

Table 4-1 Summary of experimental test results

| Specimen                      | $f'_c$<br>(MPa) | a/d | $P_{crack}$<br>(kN) | $P_{ult}$<br>(kN) | $\Delta_{ult}$<br>(mm) | $\Delta_{failure}$ or<br>$\Delta_{end\ of\ test}$<br>(mm) | Energy-<br>dissipating<br>capacity<br>(kN.mm) | Initial<br>stiffness<br>(kN/mm) | Deflection<br>ratio<br>(mm/mm) | Failure mode   |
|-------------------------------|-----------------|-----|---------------------|-------------------|------------------------|-----------------------------------------------------------|-----------------------------------------------|---------------------------------|--------------------------------|----------------|
| <b>Flexural failure</b>       |                 |     |                     |                   |                        |                                                           |                                               |                                 |                                |                |
| S1A1-S1B1                     | 59.7            | 5.5 | 70.1                | 160.1             | 50.4                   | 50.4                                                      | 6583                                          | 118.4                           | 85.1                           | Flexure        |
| S1A2-S1B2                     | 59.7            | 5.1 | 100.5               | 356.8             | 33                     | 49.8                                                      | 12738                                         | 79.7                            | 42.2                           | Flexure        |
| S2A2-S2B2                     | 65.8            | 5.0 | 103.8               | 255.7             | 25.2                   | 49.8                                                      | 10121                                         | 103.8                           | 57.6                           | Flexure        |
| S3A1-S3B1                     | 64.3            | 5.2 | 80.3                | 172.8             | 34.9                   | 50.0                                                      | 7098                                          | 80.3                            | 31.3                           | Flexure        |
| S3A2-S3B2                     | 64.3            | 5.2 | 90.5                | 340.6             | 30.6                   | 49.4                                                      | 11281                                         | 320.4                           | 174.9                          | Flexure        |
| S4A2-S4B2                     | 63.8            | 4.7 | 81.6                | 282.3             | 34.1                   | 49.7                                                      | 9857                                          | 54.1                            | 33.0                           | Flexure        |
| S4A3-S4B3                     | 63.8            | 4.6 | 83.9                | 292.2             | 35.0                   | 50.0                                                      | 10307                                         | 419.5                           | 250.3                          | Flexure        |
| S5A2-S5B2                     | 56.5            | 4.8 | 129.3               | 384.3             | 41.7                   | 45.6                                                      | 13559                                         | 71.7                            | 25.3                           | Flexure        |
| S5A3-S5B3                     | 56.5            | 4.8 | 78.7                | 192.2             | 50.2                   | 50.2                                                      | 5617                                          | 78.7                            | 53.7                           | Flexure        |
| S6A2-S6B2                     | 61.2            | 5.4 | 69.6                | 187.2             | 38.7                   | 50.1                                                      | 6907                                          | 89.2                            | 63.9                           | Flexure        |
| S7A2-S7B2                     | 65.7            | 5.4 | 80.2                | 245.5             | 19.5                   | 49.9                                                      | 9729                                          | 61.1                            | 38.0                           | Flexure        |
| <b>Shear failure</b>          |                 |     |                     |                   |                        |                                                           |                                               |                                 |                                |                |
| S2A3-S2B3                     | 65.8            | 4.8 | 186.9               | 213.5             | 3.5                    | 18.8                                                      | 3271                                          | 108.9                           | 10.9                           | Shear          |
| S6A3-S6B3                     | 61.2            | 5.2 | 91.9                | 263.4             | 30.8                   | 42.5                                                      | 8711                                          | 114.4                           | 52.8                           | Shear          |
| S8A3-S8B3                     | 66.1            | 4.9 | 102.6               | 233.9             | 23.1                   | 24.59                                                     | 1859                                          | 90.3                            | 21.7                           | Shear          |
| <b>Flexural-shear failure</b> |                 |     |                     |                   |                        |                                                           |                                               |                                 |                                |                |
| S7A3-S7B3                     | 65.7            | 4.9 | 78.98               | 198.4             | 55.5                   | 55.5                                                      | 9195                                          | 53.2                            | 37.4                           | Flexural-shear |

The failure mode of each slab was determined by analyzing the load-deflection responses along with the observed cracking behaviour during testing. DIC analyses could not be performed for slabs S3A2-S3B2 & S8A3-S8B3 due to technical issues with the DIC device during testing.

Slabs that failed in flexure exhibited an initial linear elastic response, followed by a nonlinear stage as flexural cracks formed in the constant moment region between the two loading points, eventually leading to concrete crushing in the compression zone between the two loading points. The post-peak behaviour varied among slabs, with some exhibiting a gradual decrease in load, indicating ductile failure, while others showed a more sudden drop, suggesting a more brittle response. The more brittle response observed in some slabs may be attributed to the presence of microcracks in the concrete, which promote local crack propagation under flexural loading. The number of cracks in the constant moment region played a significant role in post-peak behaviour.

Multiple cracks in the constant moment region are generally associated with good bond between the tendons and the surrounding concrete. At lower corrosion levels, bond strength may increase due to the formation of corrosion products, which roughen the surface of the steel and enhance mechanical interlock during the pre-cracking stage (Al-Sulaimani et al., 1990; Coronelli & Gambarova, 2004). This initial improvement is typically followed by a decline in bond strength as corrosion progresses and causes cracking and spalling of the concrete cover.

In the case of slabs S1A2-S1B2, S3A2-S3B2, and S4A2-S4B2, two flexural cracks developed in the constant moment region, indicating good bond conditions with localized stress concentrations at crack locations. Moreover, no cracks were observed along the slab soffit parallel to the tendon in these slabs, suggesting that despite the presence of corrosion observed in the tendons during post-test analysis, the corrosion level and distribution may have been low enough to enhance rather than degrade the bond. This improved bond may have helped reduce crack widths and allowed greater stress transfer from the tendons to the surrounding concrete. It is also possible that better bond contributed to retaining more of the prestressing force after slab removal, resulting in higher residual compressive stress in the concrete.

Slab S1A1-S1B1 exhibited a distinct post-cracking behaviour, with the applied load continuing to increase gradually despite significant cracking and deformation. The test was stopped once crack widths became substantial and midspan deflection exceeded 50 mm, beyond the measurement range of the LVDTs. It is also noted that this slab experienced unequal loading at the two-point load locations due to equipment issues, which may have contributed to the observed behaviour.

Slab S1A1-S1B1, with consistently low corrosion levels ( $< 1\%$ ) across all wires, presents an interesting case. Despite the minimal corrosion, this slab exhibited the lowest ultimate load (160.1 kN) and energy dissipation, suggesting that factors beyond corrosion contributed to its poor performance. Furthermore, a crack running along the tendon length was observed on the soffit of this slab, hinting at potential bond deficiencies. Considering the crumbled grout observed within this slab's tendon duct, whose quality and integrity were assessed and reported in Chapter 3, it is plausible that poor grouting practices during construction, rather than corrosion, played a significant role in its reduced capacity. This highlights the importance of proper grouting techniques in ensuring the long-term performance of prestressed concrete structures. These

findings also indicate that corrosion level alone is not a reliable predictor of performance; grouting quality and bond condition are equally critical factors in maintaining structural integrity.

Slabs with a single flexural crack, such as S2A2-S2B2, S3A1-S3B1, S4A3-S4B3, S5A2-S5B2, S5A3-S5B3 S6A2-S6B2, and S7A2-S7B2, can be associated with compromised bond conditions. This is supported by the lower number of flexural cracks and the wider crack widths observed in these slabs (Mousa & Mousa, 2016). These slabs demonstrated a more stable post-peak response, characterized by a plateau-like behaviour in their load–deflection curves. This behaviour indicates a more uniform distribution of stress along the reinforcement and lower reinforcement strains. As a result, curvature becomes concentrated at the single crack location, which lead to localized crushing of the concrete in the compression zone. These slabs also deflected to the point where they were resting on the legs of the steel supports, necessitating the termination of the test. Despite this, signs of concrete crushing were still observed in the top compression zone due to the high stresses.

For slabs that failed in shear, the failure was characterized by a single dominant crack forming in the shear span, which is the region between the supports and the loading points. Unlike flexural failures, where cracks develop progressively within the constant moment region, shear failures tend to be more abrupt due to the brittle nature of shear resistance. This behaviour was reflected in the load-deflection curves of slabs S2A3-S2B3, S6A3-S6B3 and S8A3-S8B3, where most slabs exhibited a sudden drop in load at ultimate capacity, followed by a gradual decline in the post-peak region. The sharp drop in load indicates the rapid propagation of the shear crack, which led to a sudden loss of stiffness and strength. Notably, these same slabs exhibited soffit cracking and elevated localized corrosion, suggesting a potential correlation between tendon condition and the observed shear failure mode.

For slab S7A3-S7B3, which experienced a combined flexural-shear failure, two dominant cracks were observed: a diagonal crack in the shear span and a vertical crack in the constant moment region. These cracks widened simultaneously, accompanied by concentrated deformation in their respective regions (Figure 4-10). This allowed for a more distributed failure mechanism, leading to a smoother load-deflection curve.

The energy-dissipating capacity, or flexural toughness, of a structural element reflects its ability to absorb energy before failure. This study quantifies the flexural toughness of the PC slabs by

calculating the area under their respective load-deflection curves, as reported and presented in Table 4-1 and Figure 4-14. While alternative methodologies for determining flexural toughness exist, as discussed in Aslani and Samali (Aslani & Samali, 2014), this approach provides a direct measure of the total energy absorbed by each slab prior to failure.

Previous research has established a link between decreased toughness and increased corrosion levels in reinforced concrete (Vehovar et al., 1998; Yang et al., 2024), primarily due to mass loss and bond degradation between the reinforcement and surrounding concrete. Interestingly, observations during tendon extraction confirmed these findings. Several slabs exhibited longitudinal cracks along the tendon length, indicating potential bond deficiencies. Slabs S2A3-S2B3, S6A3-S6B3, and S8A3-S8B3, all of which failed in shear, displayed such cracks along the soffit. This suggests that anchorage failure may have contributed to the shear failure mode by hindering effective load transfer and stress distribution.

Slab sample S2A3-S2B3, which exhibited the lowest energy-dissipating capacity, lowest deflection at ultimate load and a shear failure mode, also showed a weak bond between the tendon and the surrounding concrete during the tendon removal process. In fact, a jackhammer was not needed to separate the tendon from this slab, unlike the remaining slabs where the tendons were firmly bonded within their ducts. This lack of bond in S2A3-S2B3 likely contributed to its poor performance, as it hindered proper load transfer between the tendon and the concrete. Furthermore, when cutting slab S2A3 into a slender beam to facilitate tendon removal, a large crack was observed running along the tendon length, as highlighted in Figure 4-11. This crack provides further evidence of bond failure between the tendon and the surrounding concrete.



Figure 4-11 Evidence of bond failure in slab S2A3-S2B3

Slab S1A1-S1B1, which had the lowest energy-dissipating capacity from the slabs that failed in flexure, also exhibited a crack along the tendon length. Although it had low corrosion levels, the

presence of the crack and the crumbled grout within its duct suggests that poor initial bond quality, possibly due to inadequate grouting, may have compromised its performance.

Similarly, slab S7A3-S7B3, which failed in flexural-shear and had the third-lowest ultimate capacity, also showed a crack along the tendon. This case highlights that even with some level of bond degradation, slabs can still exhibit a degree of flexural behaviour before ultimate failure. The combined flexural-shear failure mode suggests that both flexural and shear mechanisms contributed to its failure, likely influenced by a combination of factors such as bond degradation, concrete properties, and loading conditions.

Conversely, slab S5A2-S5B2, which had the highest ultimate load and energy-dissipating capacity, also demonstrated excellent bond characteristics. Notably, the grout within this slab's duct was solid throughout the tendon length, unlike the crumbled and powdery grout observed in the other slabs, as shown in Figure 4-12. This solid grout likely contributed to the proper bond and overall performance of S5A2-S5B2.



Figure 4-12 Grout conditions in tendon ducts: (a) solid in S5A2-S5B2, and (b) crumbled in remaining tendons

The extent of energy dissipation before failure can be considered an indicator of ductility (Reyes-Salazar et al., 2018). Based on this, S5A2-S5B2, with its extensive energy dissipation, is expected to exhibit the greatest ductility, while S2A3-S2B3 is expected to have the lowest. However, this expectation was not fully supported by the observed deflection ratios.

The ductility index is defined as the ability of a structural member to resist a large deformation after the elastic stage without a sudden drop in strength. To examine the ductility, the deflection

ductility index was estimated as the ratio between the ultimate deflection and the deflection at the initial concrete cracking (Figure 4-13). Although it is common to use the deflection at reinforcement yielding for such calculations, this was not feasible in the present study due to the absence of clearly defined yielding in the post-tensioned tendons and the variability introduced by corrosion, which made it difficult to establish a clear yield point. A clear positive correlation emerged, indicating that slabs with higher initial stiffness tend to have a higher deflection ratio. This trend is primarily attributed to the smaller initial deflection in stiffer slabs, which results in a larger deflection ratio.

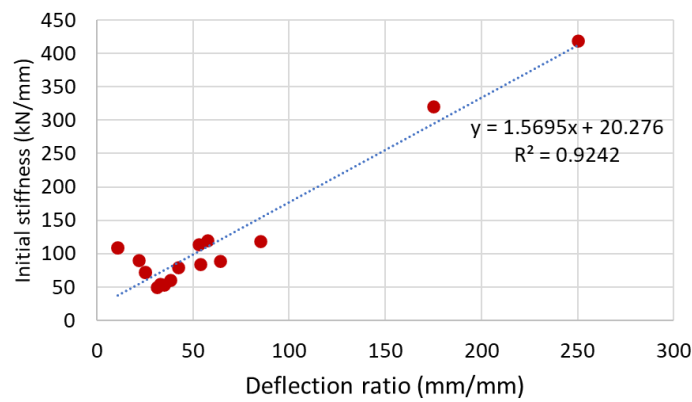
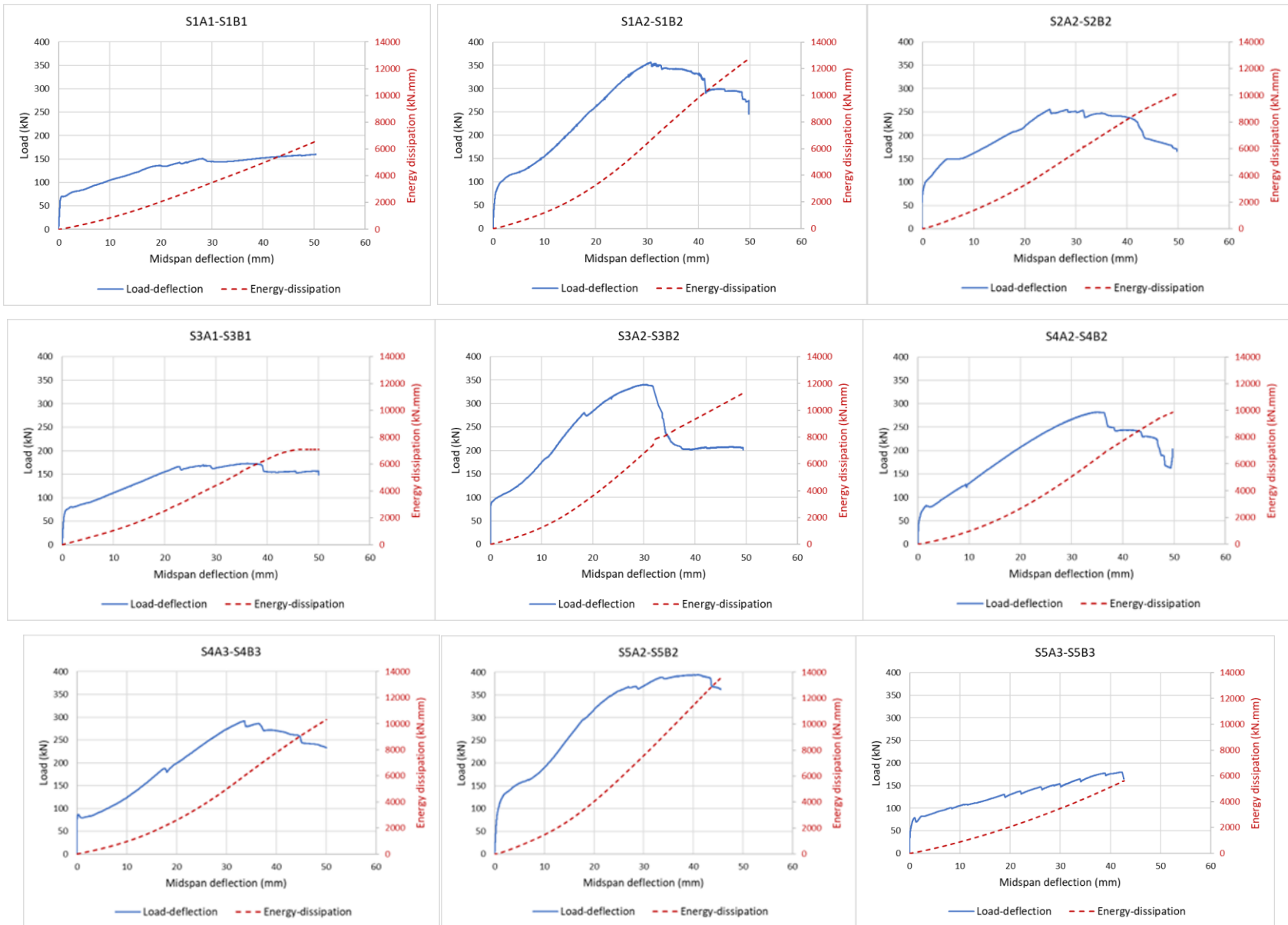
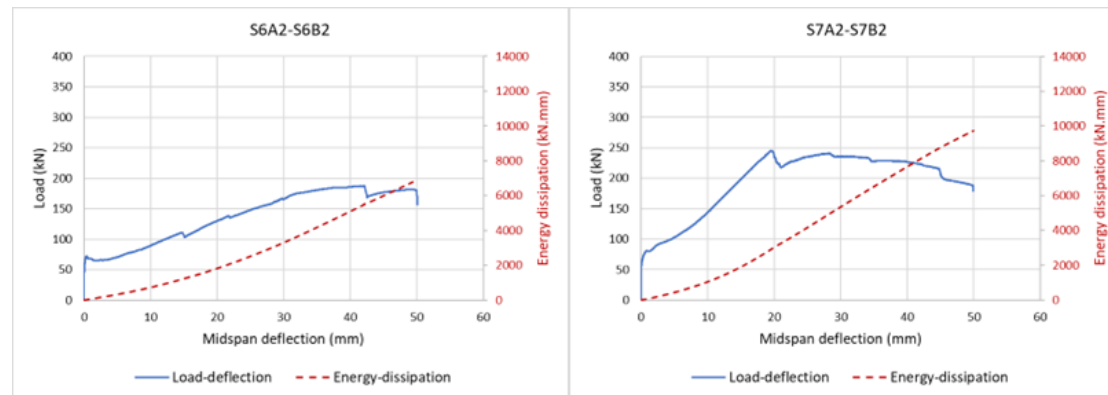
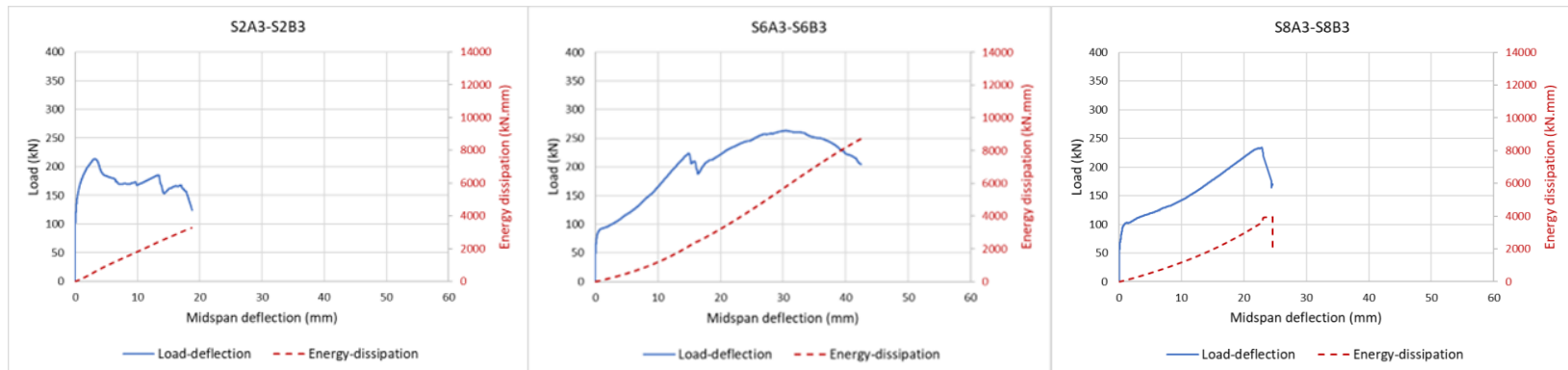
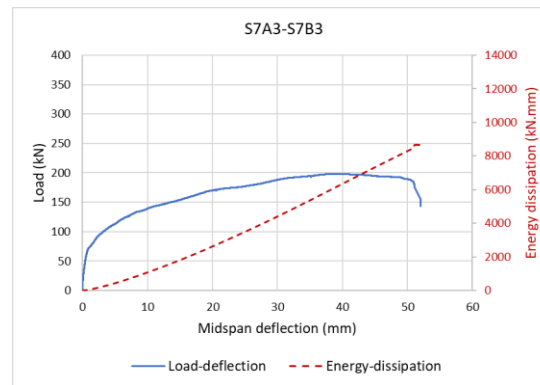


Figure 4-13 Deflection ratio versus initial stiffness





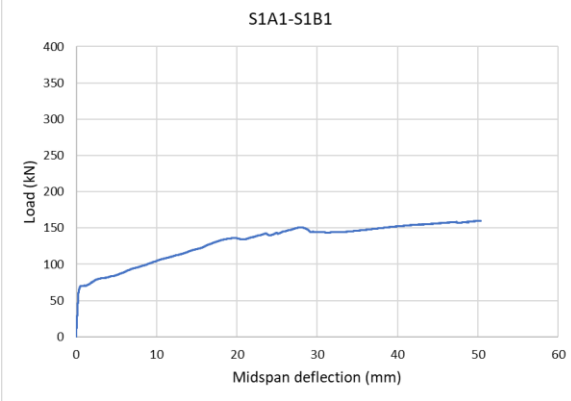
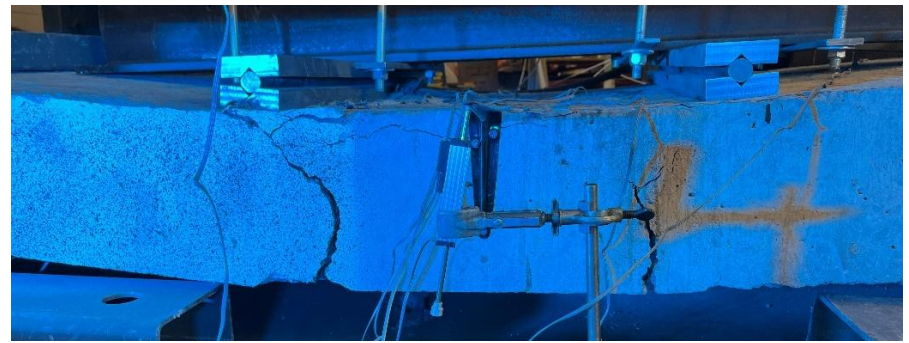
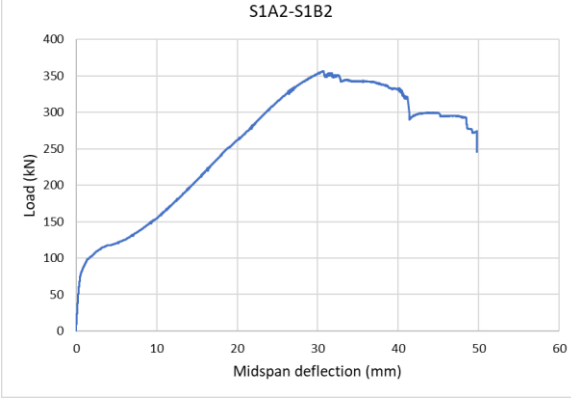
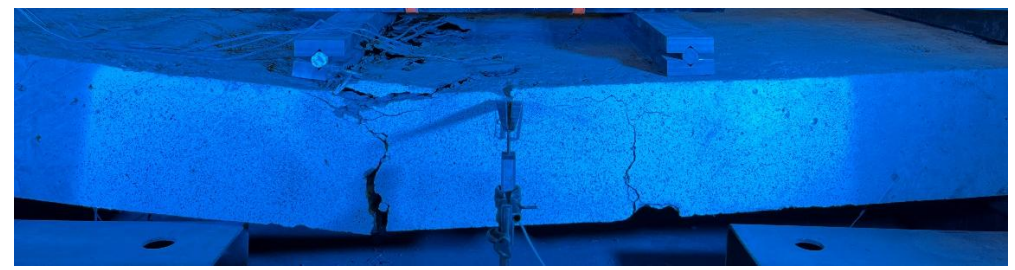
**(a) Flexural failure****(b) Shear failure**

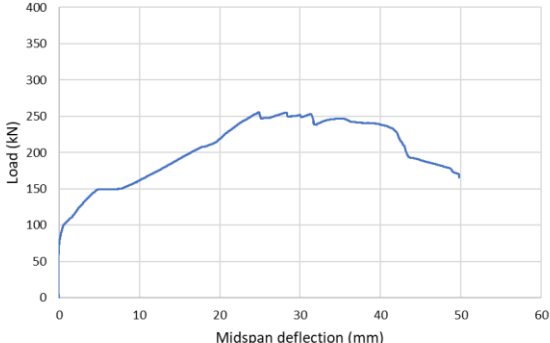
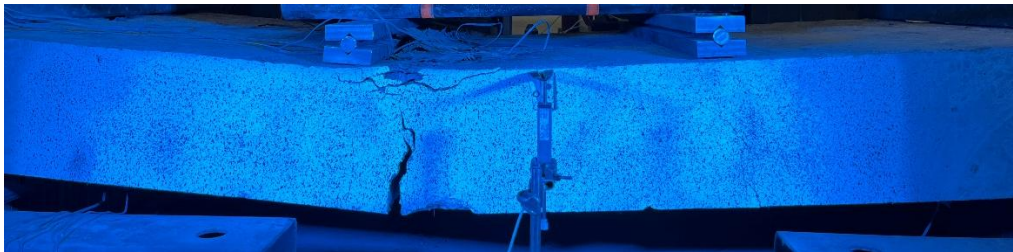
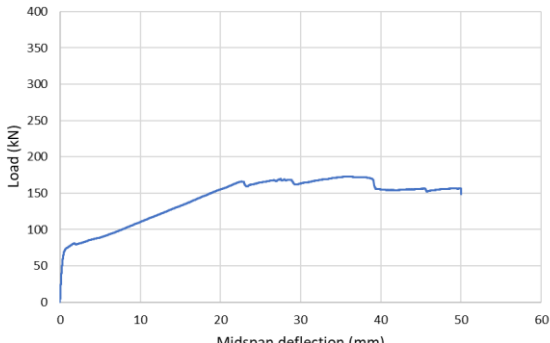
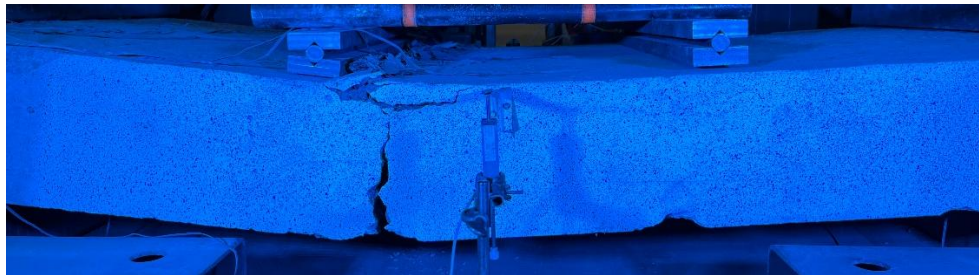


**(c) Combined flexural-shear failure**

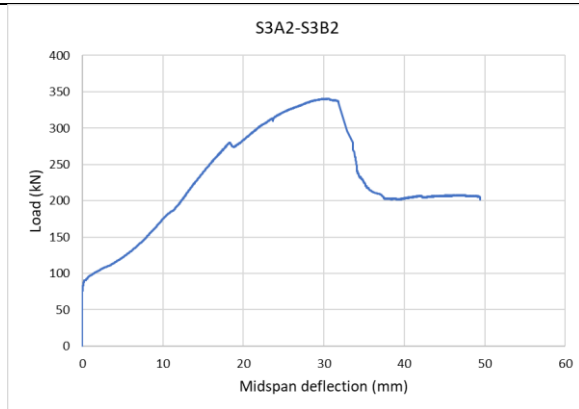
Figure 4-14 Load-deflection and energy dissipation curves of slab specimens, categorized by failure modes

Table 4-2 Summary of bending test results: load-deflection behaviour and failure mode

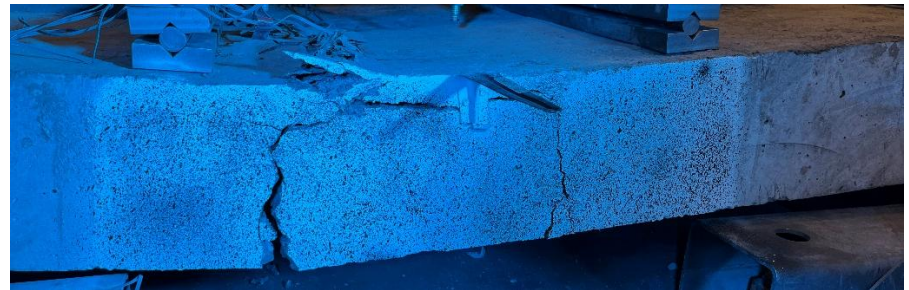
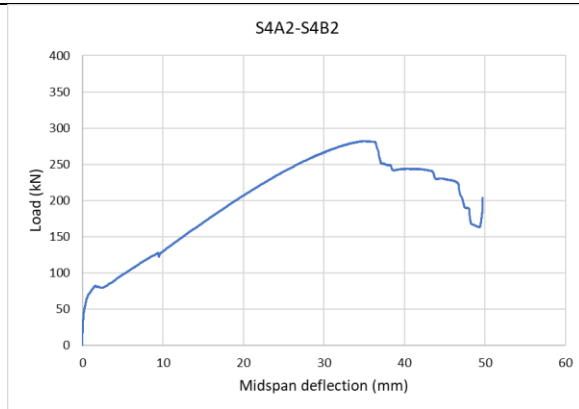
| Slab                    | Load-deflection curve                                                                               | At failure                                                                          |
|-------------------------|-----------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------|
| <b>Flexural failure</b> |                                                                                                     |                                                                                     |
| S1A1-S1B1               |  <p>S1A1-S1B1</p>  |  |
| S1A2-S1B2               |  <p>S1A2-S1B2</p> |  |

| S2A2-S2B2               | <div><p>S2A2-S2B2</p><p>Load (kN)</p><p>Midspan deflection (mm)</p><table><caption>Approximate data for S2A2-S2B2 graph</caption><tr><th>Midspan deflection (mm)</th><th>Load (kN)</th></tr><tr><td>0</td><td>0</td></tr><tr><td>5</td><td>150</td></tr><tr><td>10</td><td>160</td></tr><tr><td>20</td><td>210</td></tr><tr><td>25</td><td>250</td></tr><tr><td>30</td><td>255</td></tr><tr><td>35</td><td>245</td></tr><tr><td>40</td><td>240</td></tr><tr><td>45</td><td>190</td></tr><tr><td>50</td><td>170</td></tr></table></div> | Midspan deflection (mm) | Load (kN) | 0 | 0 | 5 | 150 | 10 | 160 | 20 | 210 | 25 | 250 | 30 | 255 | 35 | 245 | 40 | 240 | 45 | 190 | 50 | 170 |   |
|-------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------|-----------|---|---|---|-----|----|-----|----|-----|----|-----|----|-----|----|-----|----|-----|----|-----|----|-----|-------------------------------------------------------------------------------------|
| Midspan deflection (mm) | Load (kN)                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 0                       | 0                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 5                       | 150                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 10                      | 160                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 20                      | 210                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 25                      | 250                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 30                      | 255                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 35                      | 245                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 40                      | 240                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 45                      | 190                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 50                      | 170                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| S3A1-S3B1               | <div><p>S3A1-S3B1</p><p>Load (kN)</p><p>Midspan deflection (mm)</p><table><caption>Approximate data for S3A1-S3B1 graph</caption><tr><th>Midspan deflection (mm)</th><th>Load (kN)</th></tr><tr><td>0</td><td>0</td></tr><tr><td>5</td><td>80</td></tr><tr><td>10</td><td>110</td></tr><tr><td>20</td><td>160</td></tr><tr><td>25</td><td>165</td></tr><tr><td>30</td><td>165</td></tr><tr><td>35</td><td>170</td></tr><tr><td>40</td><td>160</td></tr><tr><td>45</td><td>155</td></tr><tr><td>50</td><td>155</td></tr></table></div>  | Midspan deflection (mm) | Load (kN) | 0 | 0 | 5 | 80  | 10 | 110 | 20 | 160 | 25 | 165 | 30 | 165 | 35 | 170 | 40 | 160 | 45 | 155 | 50 | 155 |  |
| Midspan deflection (mm) | Load (kN)                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 0                       | 0                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 5                       | 80                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                      |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 10                      | 110                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 20                      | 160                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 25                      | 165                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 30                      | 165                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 35                      | 170                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 40                      | 160                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 45                      | 155                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |
| 50                      | 155                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |    |     |                                                                                     |

S3A2-S3B2



S4A2-S4B2

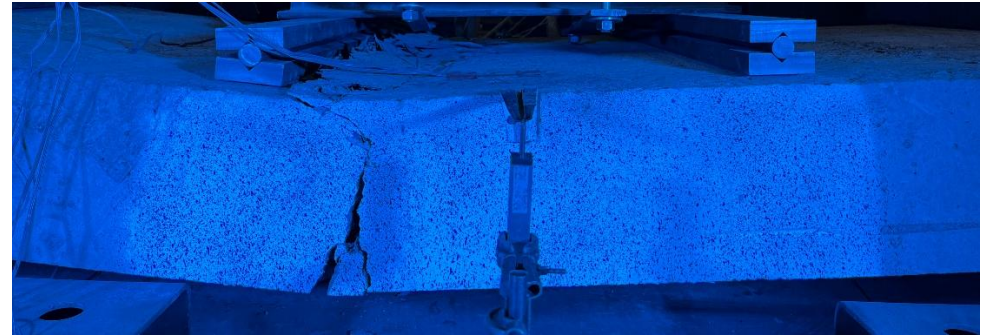
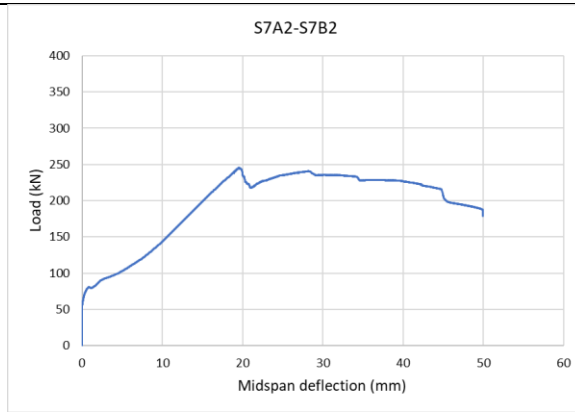




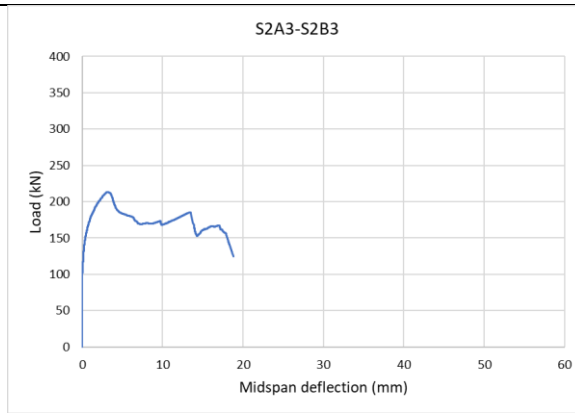
| S4A3-S4B3               | <p>S4A3-S4B3</p> <p>A line graph showing the relationship between Load (kN) on the y-axis and Midspan deflection (mm) on the x-axis for specimen S4A3-S4B3. The y-axis ranges from 0 to 400 kN in increments of 50. The x-axis ranges from 0 to 60 mm in increments of 10. The curve starts at approximately 80 kN at 0 mm deflection, rises to a peak of about 290 kN at 33 mm deflection, and then gradually declines to approximately 230 kN at 50 mm deflection.</p> <table><tr><th>Midspan deflection (mm)</th><th>Load (kN)</th></tr><tr><td>0</td><td>80</td></tr><tr><td>10</td><td>120</td></tr><tr><td>20</td><td>190</td></tr><tr><td>30</td><td>270</td></tr><tr><td>33</td><td>290</td></tr><tr><td>40</td><td>270</td></tr><tr><td>50</td><td>230</td></tr></table> | Midspan deflection (mm) | Load (kN) | 0 | 80  | 10 | 120 | 20 | 190 | 30 | 270 | 33 | 290 | 40 | 270 | 50                                                                                                                                                                                                                                                          | 230 | <p>A photograph of a concrete beam specimen S4A3-S4B3 under a three-point bending test. The beam is supported at two points and loaded at its center. A vertical crack is visible on the top surface of the beam, indicating failure under compression.</p> |
|-------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------|-----------|---|-----|----|-----|----|-----|----|-----|----|-----|----|-----|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Midspan deflection (mm) | Load (kN)                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| 0                       | 80                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| 10                      | 120                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| 20                      | 190                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| 30                      | 270                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| 33                      | 290                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| 40                      | 270                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| 50                      | 230                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| S5A2-S5B2               | <p>S5A2-S5B2</p> <p>A line graph showing the relationship between Load (kN) on the y-axis and Midspan deflection (mm) on the x-axis for specimen S5A2-S5B2. The y-axis ranges from 0 to 400 kN in increments of 50. The x-axis ranges from 0 to 60 mm in increments of 10. The curve starts at approximately 100 kN at 0 mm deflection, rises to a peak of about 390 kN at 40 mm deflection, and then slightly declines to approximately 360 kN at 45 mm deflection.</p> <table><tr><th>Midspan deflection (mm)</th><th>Load (kN)</th></tr><tr><td>0</td><td>100</td></tr><tr><td>10</td><td>180</td></tr><tr><td>20</td><td>300</td></tr><tr><td>30</td><td>370</td></tr><tr><td>40</td><td>390</td></tr><tr><td>45</td><td>360</td></tr></table>                                | Midspan deflection (mm) | Load (kN) | 0 | 100 | 10 | 180 | 20 | 300 | 30 | 370 | 40 | 390 | 45 | 360 | <p>A photograph of a concrete beam specimen S5A2-S5B2 under a three-point bending test. The beam is supported at two points and loaded at its center. A vertical crack is visible on the top surface of the beam, indicating failure under compression.</p> |     |                                                                                                                                                                                                                                                             |
| Midspan deflection (mm) | Load (kN)                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| 0                       | 100                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| 10                      | 180                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| 20                      | 300                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| 30                      | 370                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| 40                      | 390                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |
| 45                      | 360                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |                         |           |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                             |     |                                                                                                                                                                                                                                                             |

| S5A3-S5B3               | <div><p>S5A3-S5B3</p><p>Load (kN)</p><p>Midspan deflection (mm)</p><p>This graph shows the load-deflection behavior for specimen S5A3-S5B3. The load increases from 0 to approximately 175 kN as the midspan deflection increases from 0 to 42 mm. The curve is non-linear and exhibits some fluctuations.</p><table><tr><th>Midspan deflection (mm)</th><th>Load (kN)</th></tr><tr><td>0</td><td>0</td></tr><tr><td>10</td><td>100</td></tr><tr><td>20</td><td>130</td></tr><tr><td>30</td><td>150</td></tr><tr><td>40</td><td>170</td></tr><tr><td>42</td><td>175</td></tr></table></div> | Midspan deflection (mm) | Load (kN) | 0 | 0 | 10 | 100 | 20 | 130 | 30 | 150 | 40 | 170 | 42 | 175 | <p>A photograph of the physical specimen S5A3-S5B3, a concrete beam with a central crack, supported by a testing apparatus.</p> |
|-------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------|-----------|---|---|----|-----|----|-----|----|-----|----|-----|----|-----|---------------------------------------------------------------------------------------------------------------------------------|
| Midspan deflection (mm) | Load (kN)                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |
| 0                       | 0                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                           |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |
| 10                      | 100                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |
| 20                      | 130                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |
| 30                      | 150                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |
| 40                      | 170                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |
| 42                      | 175                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |
| S6A2-S6B2               | <div><p>S6A2-S6B2</p><p>Load (kN)</p><p>Midspan deflection (mm)</p><p>This graph shows the load-deflection behavior for specimen S6A2-S6B2. The load increases from 0 to approximately 185 kN as the midspan deflection increases from 0 to 48 mm. The curve is non-linear and exhibits some fluctuations.</p><table><tr><th>Midspan deflection (mm)</th><th>Load (kN)</th></tr><tr><td>0</td><td>0</td></tr><tr><td>10</td><td>80</td></tr><tr><td>20</td><td>130</td></tr><tr><td>30</td><td>160</td></tr><tr><td>40</td><td>180</td></tr><tr><td>48</td><td>185</td></tr></table></div>  | Midspan deflection (mm) | Load (kN) | 0 | 0 | 10 | 80  | 20 | 130 | 30 | 160 | 40 | 180 | 48 | 185 | <p>A photograph of the physical specimen S6A2-S6B2, a concrete beam with a central crack, supported by a testing apparatus.</p> |
| Midspan deflection (mm) | Load (kN)                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |
| 0                       | 0                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                           |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |
| 10                      | 80                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                          |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |
| 20                      | 130                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |
| 30                      | 160                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |
| 40                      | 180                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |
| 48                      | 185                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |                         |           |   |   |    |     |    |     |    |     |    |     |    |     |                                                                                                                                 |

S7A2-S7B2

**Shear failure**

S2A3-S2B3

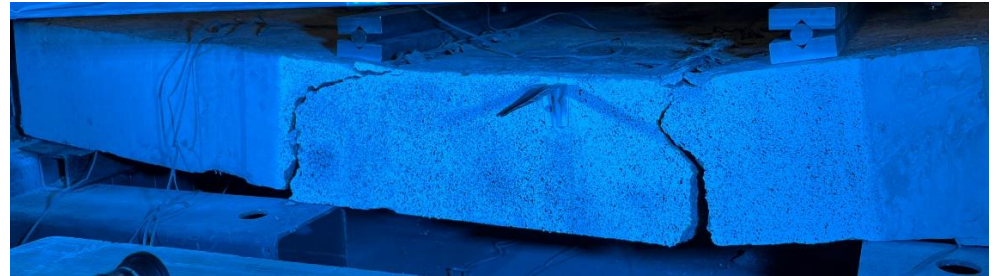
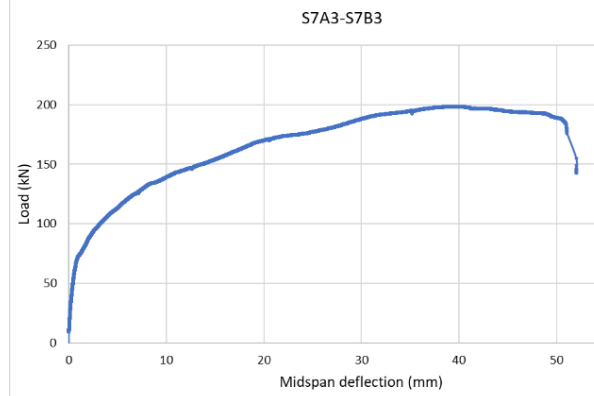




| S6A3-S6B3               | <p>S6A3-S6B3</p> <p>A line graph showing the relationship between Load (kN) on the y-axis and Midspan deflection (mm) on the x-axis for specimen S6A3-S6B3. The y-axis ranges from 0 to 400 kN in increments of 50. The x-axis ranges from 0 to 60 mm in increments of 10. The curve starts at (0,0), rises to a peak of approximately 260 kN at 30 mm deflection, and then decreases to about 210 kN at 42 mm deflection. There is a small local peak at 15 mm deflection.</p> <table><tr><th>Midspan deflection (mm)</th><th>Load (kN)</th></tr><tr><td>0</td><td>0</td></tr><tr><td>5</td><td>80</td></tr><tr><td>10</td><td>150</td></tr><tr><td>15</td><td>220</td></tr><tr><td>20</td><td>230</td></tr><tr><td>25</td><td>250</td></tr><tr><td>30</td><td>260</td></tr><tr><td>35</td><td>250</td></tr><tr><td>40</td><td>230</td></tr><tr><td>42</td><td>210</td></tr></table> | Midspan deflection (mm) | Load (kN) | 0 | 0 | 5 | 80  | 10 | 150 | 15 | 220 | 20 | 230 | 25 | 250 | 30 | 260 | 35                                                                                                                                                                                                                                                                                                     | 250 | 40 | 230 | 42 | 210 | <p>A photograph of a concrete beam specimen S6A3-S6B3 under test. The beam is supported at two points and has a large, dark, diagonal crack running from the bottom left towards the center. The crack is approximately 10 cm long. The beam is surrounded by a network of blue wires and sensors.</p> |
|-------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------|-----------|---|---|---|-----|----|-----|----|-----|----|-----|----|-----|----|-----|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----|----|-----|----|-----|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Midspan deflection (mm) | Load (kN)                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                             |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 0                       | 0                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 5                       | 80                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                    |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 10                      | 150                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 15                      | 220                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 20                      | 230                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 25                      | 250                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 30                      | 260                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 35                      | 250                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 40                      | 230                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 42                      | 210                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| S8A3-S8B3               | <p>S8A3-S8B3</p> <p>A line graph showing the relationship between Load (kN) on the y-axis and Midspan deflection (mm) on the x-axis for specimen S8A3-S8B3. The y-axis ranges from 0 to 400 kN in increments of 50. The x-axis ranges from 0 to 60 mm in increments of 10. The curve starts at (0,0), rises to a peak of approximately 230 kN at 22 mm deflection, and then drops sharply to about 160 kN at 25 mm deflection.</p> <table><tr><th>Midspan deflection (mm)</th><th>Load (kN)</th></tr><tr><td>0</td><td>0</td></tr><tr><td>5</td><td>100</td></tr><tr><td>10</td><td>140</td></tr><tr><td>15</td><td>180</td></tr><tr><td>20</td><td>220</td></tr><tr><td>22</td><td>230</td></tr><tr><td>25</td><td>160</td></tr></table>                                                                                                                                             | Midspan deflection (mm) | Load (kN) | 0 | 0 | 5 | 100 | 10 | 140 | 15 | 180 | 20 | 220 | 22 | 230 | 25 | 160 | <p>A photograph of a concrete beam specimen S8A3-S8B3 under test. The beam is supported at two points and has a large, dark, diagonal crack running from the bottom left towards the center. The crack is approximately 10 cm long. The beam is surrounded by a network of blue wires and sensors.</p> |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| Midspan deflection (mm) | Load (kN)                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                             |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 0                       | 0                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 5                       | 100                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 10                      | 140                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 15                      | 180                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 20                      | 220                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 22                      | 230                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |
| 25                      | 160                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                   |                         |           |   |   |   |     |    |     |    |     |    |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |     |    |     |    |     |                                                                                                                                                                                                                                                                                                        |

**Flexural-shear failure**

S7A3-S7B3



To determine the degree of fixity at the supports, a comparative analysis was conducted using both DIC and LVDT data. First, a specific load level below the cracking load was identified using the DIC vertical deflection measurements. The corresponding load at this same deflection was then cross-referenced from the LVDT readings. Next, the strain profile across the mid-section of the slab under this target load was extracted from the DIC data. This strain profile was subsequently used in a stress analysis to calculate the applied load on the slab under two extreme boundary conditions: simply supported and fixed-fixed.. The calculated loads consistently fell between the values predicted for these two idealized conditions, indicating that the slabs were partially fixed throughout the testing phase.

Figure 4-15 presents the ratios of experimental load to the theoretical loads predicted for simply supported and fully fixed boundary conditions for all tested slabs. As shown, the ratios with respect to the simply supported case are consistently greater than one, while those with respect to the fully fixed case remain below one. This clearly confirms that the actual slab behaviour lies between the two idealized support conditions, indicative of partial fixity. This partial fixity highlights the challenge in achieving perfectly fixed-fixed boundary conditions in experimental settings. Nevertheless, the observed behaviour can still be effectively interpreted using traditional sectional stress analysis methods and further refined through accurate finite element modelling (FEM) techniques. This combined approach allows for a comprehensive understanding of the structural response, accounting for the actual support conditions.

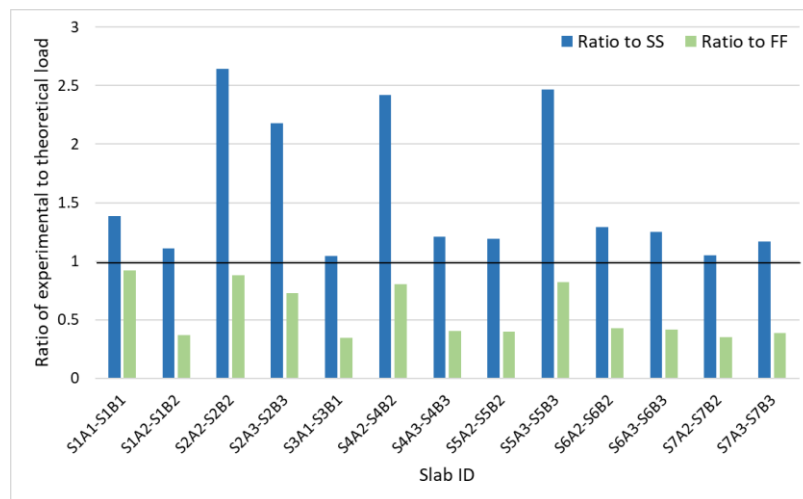


Figure 4-15 Comparison of experimental load to theoretical predictions for simply supported and fully fixed boundary conditions across tested slabs

The tested slabs exhibited a range of ultimate load capacities from 160.1 kN to 384.3 kN. To investigate the influence of the pre-compression force, a sectional analysis was performed at midspan for each slab under both ultimate load and cracking load conditions. This analysis considered the axial load imposed by the rods and its eccentricity relative to the slab's centroid, as detailed in Figure 4-16, assuming that plane sections remain plane at ultimate conditions and that the entire tensile force in the rods is transferred to the slab as a compressive force. A concrete crushing strain of 0.0035 was used at ultimate conditions, and no material strength reduction factors were applied in the analysis. Initial prestressing forces in the tendons were neglected, as the tendons had been cut, and any remaining residual force was considered negligible.

Figure 4-17 illustrates the relationship between the slab capacities at (a) cracking and (b) ultimate loads as a function of the applied compressive load. It can be seen that the compression induced by the steel rods generated arching action, effectively enhancing the capacity of the slabs by introducing a beneficial compressive force.

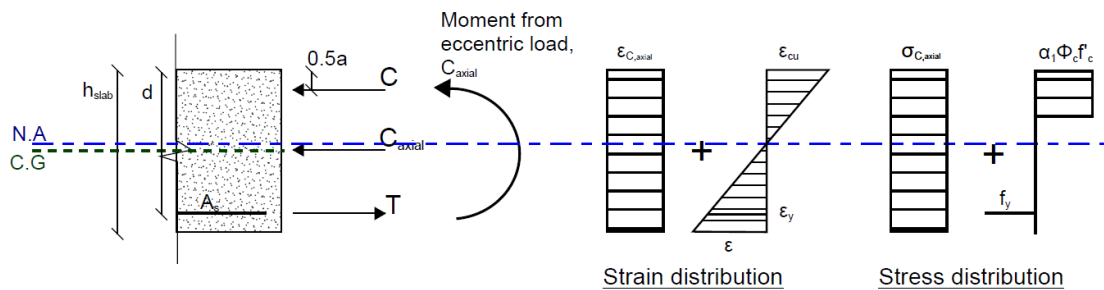


Figure 4-16 Sectional analysis to determine theoretical moment assuming perfect bond conditions

A positive correlation with a clear increasing trend between compressive stress and load-carrying capacity was observed for both loading cases. However, the trend exhibited a stronger correlation at ultimate load, with an  $R^2$  value of 65%, whereas an  $R^2$  value of 14% was observed at cracking load.

This difference in correlation can be attributed to the behaviour of the concrete. At ultimate load, the concrete is cracked, and the tensile resistance is primarily provided by the tendons. As the cracks widen, the slab undergoes deformation, leading to an average elongation that induces tension in the rods, indicating that the slab is in compression. Since it is well known that compression enhances concrete behaviour, this contributes to the slab's overall response under loading.

Conversely, at cracking load, the tensile force in the rods is minimal because the concrete remains uncracked, and deformation is limited at this stage. As a result, the concrete itself plays a primary role in resisting the applied load before cracking occurs. Additionally, since the compressive load at this stage is small, the data points used to establish this relationship are limited, and in some cases, the compressive loads at cracking for various slabs were quite low.

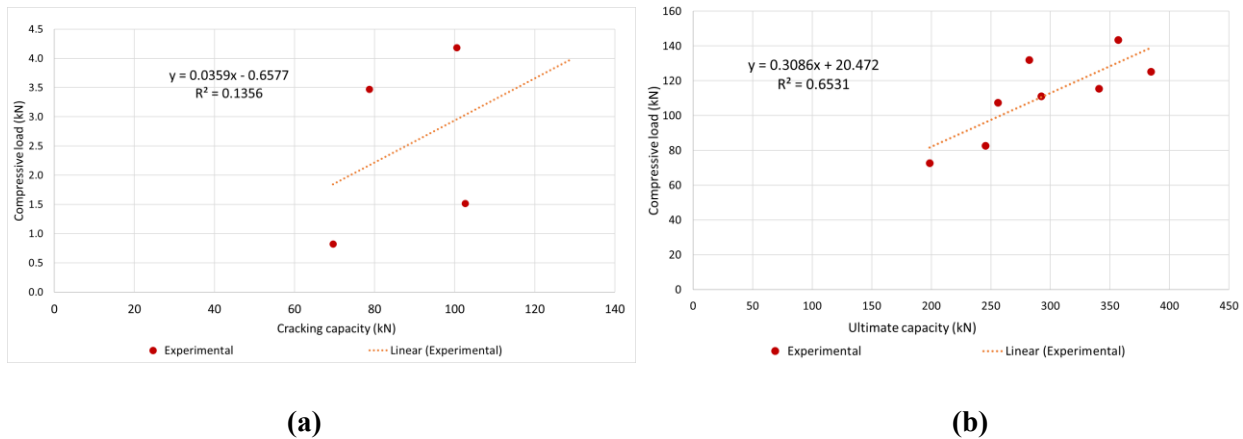


Figure 4-17 Slab capacities as a function of compressive load at: (a) cracking load, and (b) ultimate load

#### 4.4.2 Stiffness

The initial stiffness of the tested samples can be defined as the slope of the load-deflection curve and is reported in Table 4-1. The values were obtained using the slope of the secant line connecting the origin to the cracking load, and range from as low as 53.2 kN/mm to as high as 419.5 kN/mm. Slabs with the lowest initial stiffness values (54.1 kN/mm for S4A2-S4B2 and 53.2 kN/mm for S7A3-S7B3) both failed in flexure and exhibited similar values for energy-dissipating capacity, cracking load, and ultimate load. In contrast, slabs with the highest initial stiffness values (320.4 kN/mm for S3A2-S3B2 and 419.5 kN/mm for S4A3-S4B3) displayed similar energy-dissipating capacity, cracking load, ultimate load, and deflection at ultimate load. Both slabs failed in flexure.

The high degree of variability in initial stiffness, despite a consistent concrete compressive strength among the slabs, may be primarily driven by the presence of pre-existing micro-cracks and localized defects within the concrete matrix prior to testing.

Studies by Said & Hussein (Said & Hussein, 2019) and Aoude et al. (Aoude et al., 2014) found that a 50% reduction in reinforcement mass due to corrosion can lead to a 66% to 75% loss in

stiffness, while a 15% mass loss can cause a 56.7% drop in stiffness. However, directly comparing these findings to the current study requires caution, as those studies employed accelerated corrosion techniques resulting in uniform corrosion, while this study examines naturally occurring uniform and localized corrosion. It is also important to note that under corrosion conditions, post-tensioning cables behave differently from regular reinforcing steel.

Despite this difference, a similar trend can be observed. Tendons with localized areas of high corrosion, such as S7A2-S7B2 with a maximum corrosion level of 19% and other localized areas approaching 6%, experienced an 85% decrease in stiffness compared to S4A3-S4B3, which had the highest stiffness and a maximum localized corrosion level of only 1.24% in one of its cables. Similarly, S2A2-S2B2, with a maximum corrosion level of 14% and other localized areas approaching 3%, showed a 75% reduction in stiffness. These observations suggest a trend where slabs with tendons with multiple strands experiencing high localized corrosion exhibit a significant reduction in stiffness.

It is important to note that S1A1-S1B1, despite minimal corrosion ( $<1\%$ ), exhibited lower stiffness than S4A3-S4B3. This suggests that factors beyond corrosion, such as potential poor bond grouting, may have influenced the stiffness of S1A1-S1B1. Therefore, S4A3-S4B3 serves as a more appropriate baseline for comparison when assessing the impact of corrosion on stiffness.

In the presence of corrosion, a reduced initial stiffness of post-tensioned concrete members may result from concrete microcracking, loss of confinement, or bond deterioration due to expansive corrosion products. After cracking, the impact of defective grouting conditions becomes more evident, leading to increased flexibility, or tendon movement, in unbonded or partially bonded specimens compared to bonded ones (Losanno et al., 2024). Additionally, fully and partially bonded specimens exhibit similar peak bending moment capacities, whereas unbonded specimens reach a lower peak bending moment capacity. This reduction is due to the limited tensile force within the unbonded tendons, as the tendons are unable to effectively transfer stress, leading to a weaker overall bending resistance.

However, it is important to consider that all slabs in this study were extracted from a field structure and as demonstrated in Section 4.4.4 through sectional analysis, exhibited some degree of bond deficiency. This inherent variability in bond conditions makes it challenging to directly correlate

initial stiffness with grouting defects, as all specimens are already affected by varying degrees of bond degradation.

#### 4.4.3 Bond analysis

To assess the impact of grout-steel bond degradation on moment capacity, a sectional analysis, as detailed in Figure 4-16 was performed on each tested slab. This analysis aimed to determine the theoretical moment capacity assuming perfect bond conditions. Strain gauge data collected from the steel rods provided the axial compressive force acting along the center of the slabs at ultimate load, assuming that the load is completely resisted by the slabs. This theoretical capacity was then compared to the experimentally observed ultimate moment capacity of the slabs, which was calculated using the maximum load applied during the four-point bending test, under two boundary conditions: simply supported and fixed-fixed. This comparison was deemed necessary as the actual behaviour of the slabs fell within the range defined by these two idealized boundary conditions.

Figure 4-18 presents the ratios of experimental to theoretical moment capacities for the tested slabs that failed in flexural for both simply supported and fixed boundary conditions, respectively. The ideal ratio of 1.0 is indicated by the solid black line.

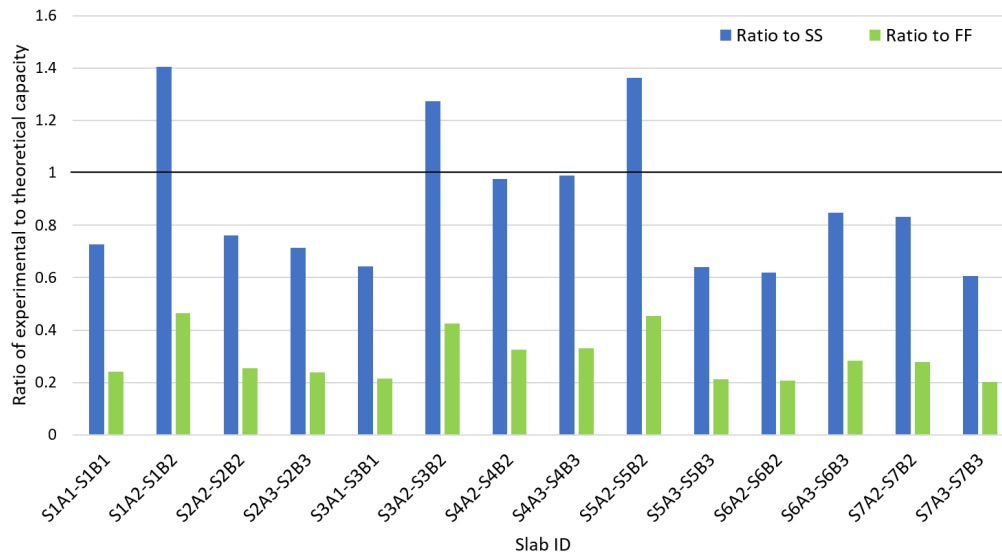


Figure 4-18 Experimental to theoretical moment capacity ratios for slabs that failed in flexure

The experimental moment capacities of the tested slabs, calculated using the maximum applied load during the four-point bending test, consistently fell below the theoretical moment capacities

derived from sectional analysis assuming perfect bond conditions, with some slabs failing in modes other than flexure. This discrepancy points to a bond deficiency between the prestressing steel and the surrounding concrete. This reduced bond strength results in lower tensile forces being developed in the tendons and, consequently, lower compressive forces in the concrete.

Furthermore, the actual boundary conditions of the tested slabs lie somewhere between the idealized cases of simply supported and fixed-fixed conditions. Therefore, bond degradation and less-than-ideal boundary conditions lead to lower observed moment capacities compared to theoretical predictions.

The experimental shear capacities were notably lower than the theoretical predictions, with ratios ranging from 0.84 to 0.56, depending on the assumed boundary condition, as shown in Figure 4-19. This behaviour may be attributed to localized effects. The four-point bending test configuration generates high shear forces near the supports. If bond degradation is more pronounced in these regions, it can lead to a localized reduction in shear capacity, potentially triggering premature shear failure. Furthermore, bond deterioration in the constant moment region, under the applied loads, may prevent force transfer between the tendon and surrounding concrete. This can cause stress concentrations to shift toward the shear span, further increasing the demand in these regions and contributing to shear failure. Additionally, the presence of micro-cracks, which are likely in slabs extracted from a field structure with nearly 60 years of service life, could further contribute to localized shear weakness. These micro-cracks may act as stress concentrators, reducing the shear resistance of the concrete and promoting shear failure even under reduced moment demands. The shear capacity was calculated using the general method outlined in CSA A23.3, without accounting for any prestressing effects.

Further analysis, including detailed finite element Modelling, could help to further explain the complex interaction of these factors and provide a more comprehensive understanding of the shear failure mechanism in these slabs.



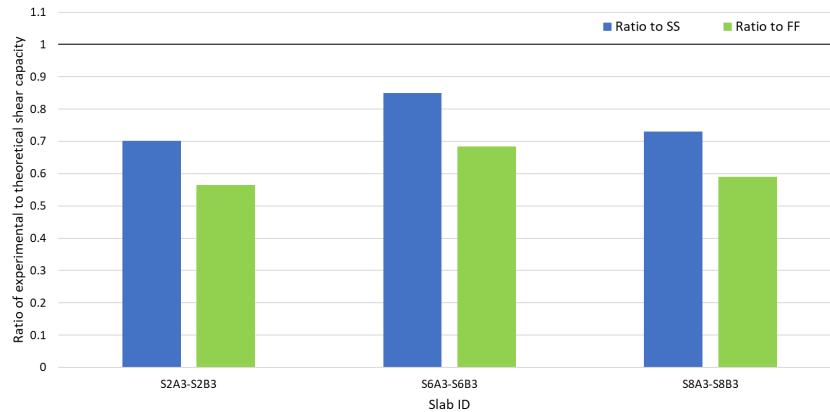


Figure 4-19 Experimental to theoretical shear capacity ratios for slabs that failed in shear

#### 4.4.4 DIC analysis

The horizontal 2D strain distributions across the front face of the slabs at ultimate loading conditions, along with their crack propagation, were analyzed using DIC techniques and are presented in Figure 4-20. The region of interest varied between slabs, focusing either on a single shear span or the pure flexural zone. It is important to note that DIC analyses could not be performed for slabs S3A2-S3B2 and S8A3-S8B3 due to technical issues with the DIC equipment during testing.

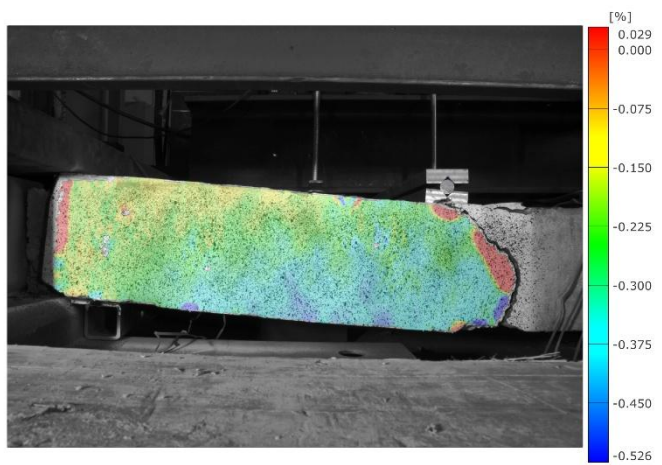
The legend accompanying the strain distribution maps provides key insights into the strain values across the slabs. The colour scale indicates the level of tensile strain, with red representing areas of high tensile strain and blue indicating regions with compressive strain. These strain maps help identify the locations where cracks initiated and propagated.

The distribution, orientation, and density of cracks varied depending on whether the slab failed in flexure, shear, or flexural-shear. In flexural failures, strain concentrations were primarily localized along vertical cracks in the constant moment region, with strain gradually increasing as the cracks widened. In shear failures, strain localization was more pronounced along diagonal cracks, with a rapid increase in deformation near ultimate load, leading to abrupt failure. In flexural-shear failures, strain concentrations were observed at both the shear-span border and within the constant moment region, indicating a combined failure mode. The DIC results confirm that the cracking patterns and their evolution directly influenced the post-peak response of each slab. The presence of multiple flexural cracks allowed for a more gradual reduction in load capacity, whereas a single shear crack

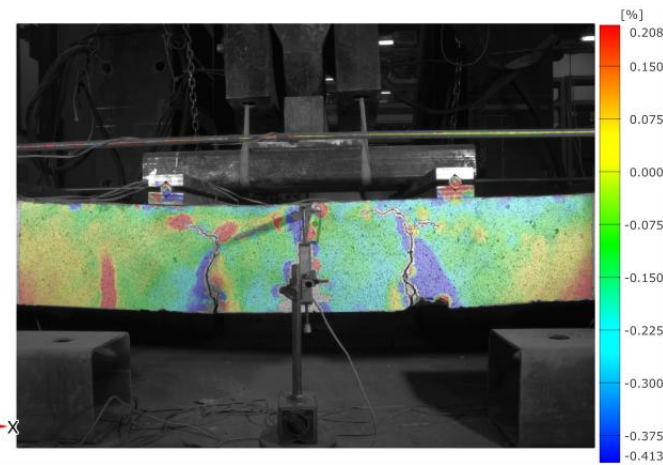
resulted in a sudden failure. The interaction between shear and flexural cracks in flexural-shear failures led to intermediate post-peak behaviours, depending on the number and distribution of cracks.

For S2A3-S2B3, a crack initially formed outside the ROI and later propagated into it, which prevented the DIC from capturing it at ultimate load. The condition of slab S2A3-S2B3 at failure is shown in Table 4-2.

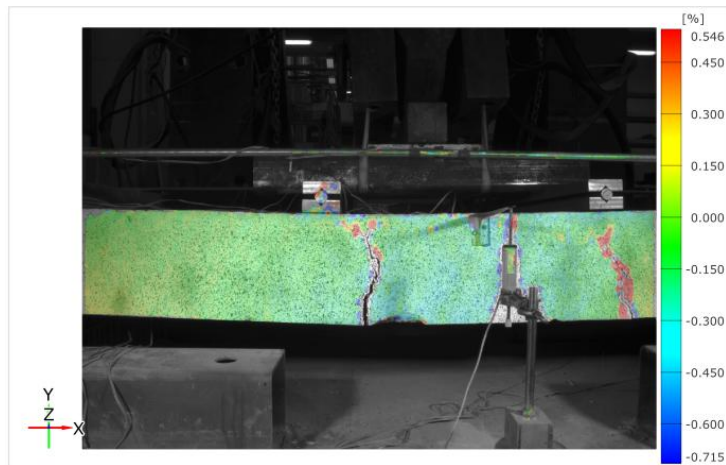
For slab S3A2-S3B2, two cracks occurred in the flexural region, as shown in Table 4-2. For slab S8A3-S8B3, shear failure is observed through the diagonal crack that developed in the shear zone and propagated, as shown in Table 4-2.



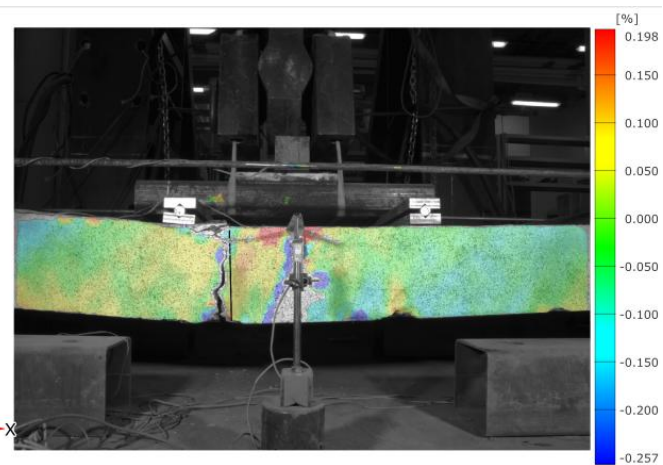
**(1) S1A1-S1B1**



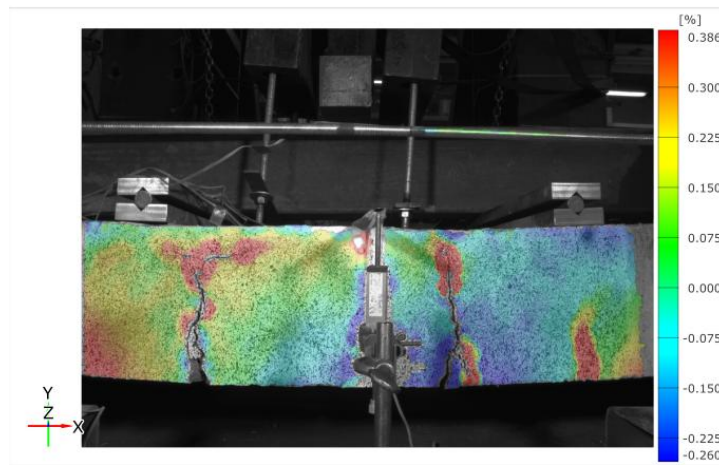
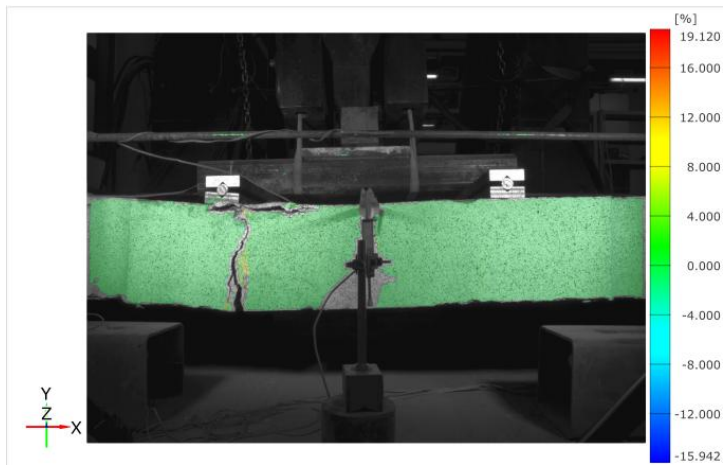
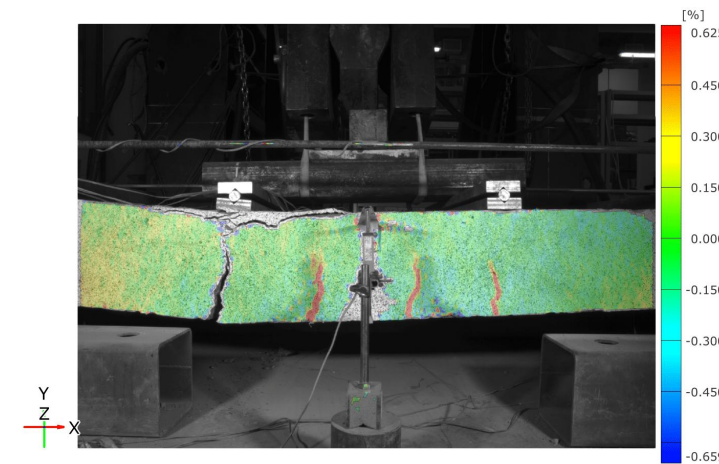
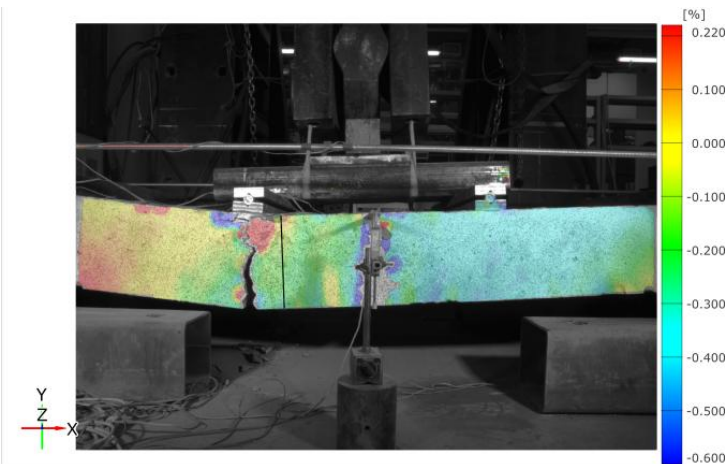
**(2) S1A2-S1B2**

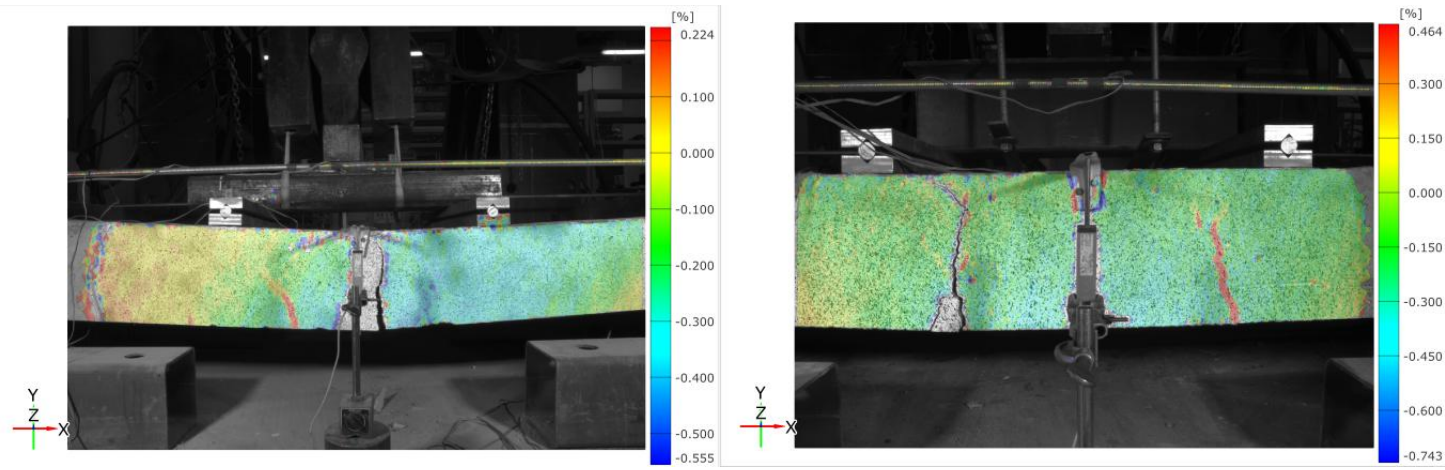


**(3) S2A2-S2B2**



**(4) S3A1-S3B1**

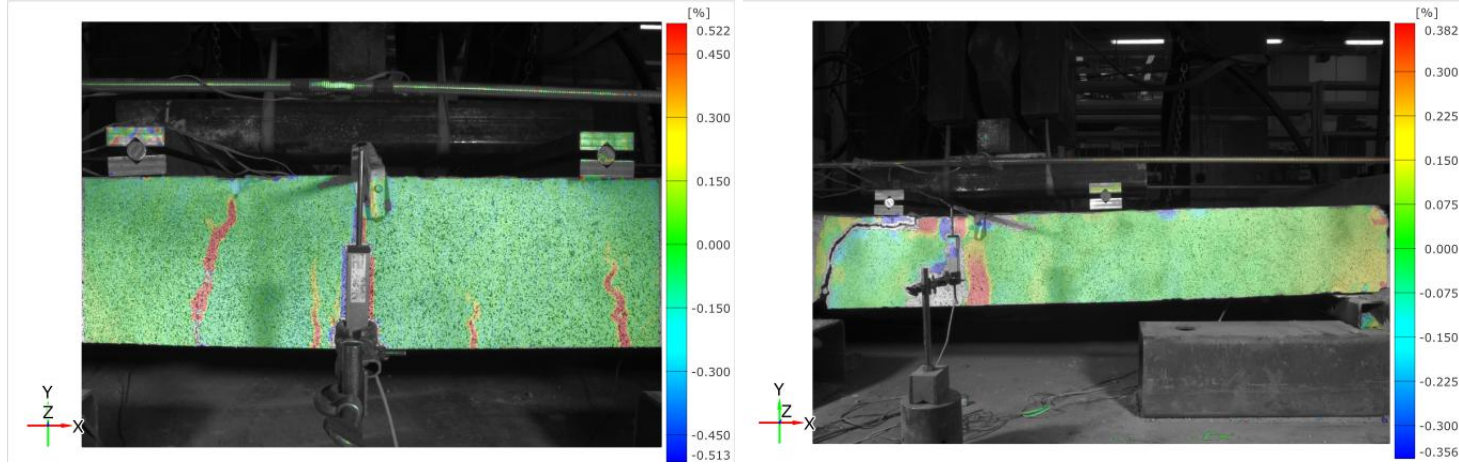
**(5) S4A2-S4B2****(6) S4A3-S4B3****(7) S5A2-S5B2****(8) S5A3-S5B3**



(9) S6A2-S6B2

(10) S7A2-S7B2

**(a) Flexural failure**

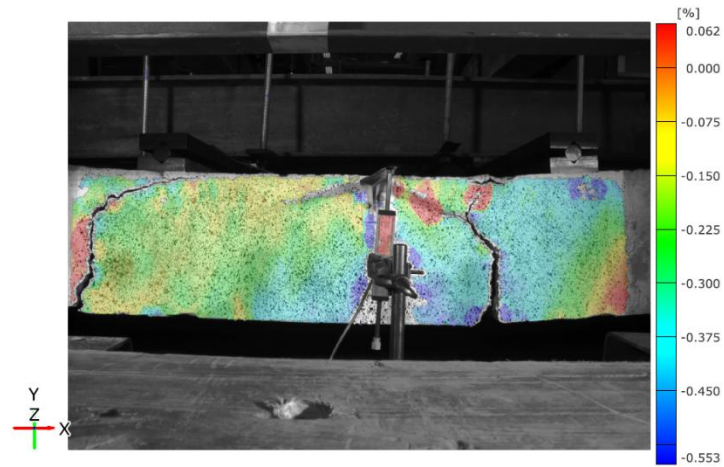


(11) S2A3-S2B3

(12) S6A3-S6B3

**(b) Shear failure**





**(13) S7A3-S7B3**

**(c) Flexural-shear failure**

Figure 4-20 DIC analyses of slabs at ultimate loading conditions, categorized by failure modes

## 4.5 Conclusions

This paper investigates the influence of corrosion on the ultimate capacity and behaviour of PC slabs. Fifteen naturally corroded one-way PC slabs were subjected to four-point bending tests until failure. The observed failure modes and crack patterns were then analyzed and correlated with visual inspection findings, including crack pattern maps and mass loss measurements. Based on the experimental results, key conclusions are drawn regarding the impact of corrosion on the structural performance of PC slabs:

- The slabs exhibited partial fixity during testing, with calculated loads consistently falling between simply supported and fixed-fixed boundary conditions. Despite challenges in achieving perfectly fixed conditions, traditional stress analysis methods proved effective for interpreting structural responses.
- A positive correlation was observed between compressive stress and load-carrying capacity, particularly at ultimate loads, which is expected. The pre-compression force introduced arching action, enhancing slab capacities by generating a beneficial compressive force.
- Although most tendons exhibited low overall corrosion ( $<1\%$ ), localized severe corrosion (up to  $19\%$ ) was observed, underscoring the need to consider localized damage in performance assessments.
- High localized corrosion likely influenced bond strength between tendons and grout, affecting failure modes and overall performance. However, minimal corrosion levels ( $<1\%$ ) did not necessarily ensure high performance, as poor grouting practices also significantly impacted slab behaviour.
- The interplay between corrosion, bond strength, and failure mechanisms revealed varied outcomes, with some tendons failing in flexure and others in shear, depending on bond conditions and loading. Localized bond degradation contributed to reduced stiffness and load-carrying capacity, especially in slabs with multiple strands exhibiting high corrosion levels.
- The energy-dissipating capacity correlated with failure modes, with slabs exhibiting shear or flexural-shear failures showing rapid energy accumulation. Slabs with solid grout and

good bonding, such as S5A2-S5B2, displayed higher ultimate loads, energy dissipation, and ductility, emphasizing the importance of proper grouting.

- Slabs with localized high corrosion showed significant stiffness reductions, aligning with trends observed in previous studies on accelerated corrosion.
- Further research is needed to refine the understanding of the complex relationship between corrosion, bond strength, and failure mechanisms in naturally corroded PC structures.

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## **Chapter 5: Mechanical properties of corroded prestressing tendons recovered from the Champlain Bridge**

### **Abstract**

This study investigates the impact of corrosion on the mechanical properties of prestressing steel strands with a parallel-wire configuration, extracted from the former Champlain Bridge in Montreal, Canada, after nearly six decades in service. Throughout its lifespan, the bridge was exposed to a harsh northern climate and aggressive environmental conditions, including heavy use of de-icing salts, which accelerated corrosion of the embedded prestressing steel. A comprehensive experimental program was conducted on 975 individual wire samples to evaluate the effects of corrosion on the ultimate strength and ductility. The analysis first examined the experimental results using a logarithmic transformation of mass-loss data to capture the wide range of corrosion severities observed. This log-scale evaluation enabled a consistent comparison between uncorroded to lightly corroded samples (<1%) and progressively damaged wires (1–10%). In this stage, gauge lengths were explicitly considered to ensure that deformation values were grouped and modeled consistently across specimens with different measurement bases. Welch's t-test was then applied across these corrosion ranges, confirming statistically significant differences in ultimate strain and demonstrating that mechanical degradation begins at low corrosion levels and intensifies as mass loss increases. A second evaluation incorporated all available data from the present study and two external datasets, grouping observations into mass-loss categories below and above 3%. Despite differences in testing procedures, gauge lengths, corrosion exposure conditions, and baseline material properties, the combined dataset exhibited a statistically significant separation at this 3% threshold. This consistent behaviour across all data indicates that the transition in ductility deterioration near 3% mass loss is a robust and reproducible phenomenon across independent experimental programs. Finally, a cumulative mass-loss analysis was performed by combining all available data points from the present experimental program and three additional datasets reported in the literature. This combined dataset allowed the corrosion severity observed in different experimental programs to be compared on a common basis. The resulting cumulative distributions show that the progression of corrosion varies depending on exposure conditions and testing methodologies, producing different trajectories of mass-loss development. Nevertheless, when

these datasets are evaluated together, they consistently exhibit a transition in ductility deterioration near 3% mass loss. This convergence across independent studies reinforces the robustness of the identified corrosion–ductility threshold and highlights the usefulness of mass-loss-based indicators for evaluating the structural performance of corroded prestressing steel.

## 5.1 Introduction

In recent years, the principles of sustainability and resilience have become increasingly prevalent in the engineering community (ACI Committee 130, 2019; Wallace, 2024). This shift has been driven by the urgent need to reduce the environmental footprint of the construction industry, which is a major contributor to carbon emissions and resource depletion. While new infrastructure is sometimes necessary to meet evolving societal demands, extending the service life of existing structures remains a cornerstone of sustainable development. The preservation of aging infrastructure not only minimizes the environmental impact but also reduces the economic and social costs associated with demolition and reconstruction. This focus on extending service life is further underscored by the escalating challenges of global warming, which contributes to an increase in the frequency and intensity of major weather events such as storms, heatwaves, hurricanes, and floods, placing unprecedented stress on existing infrastructure that is designed for historical weather patterns and leading to premature failures and reduced service lives (Intergovernmental Panel on Climate Change (IPCC), 2022). The "Global Assessment Report" by the UNDRR (United Nations Office for Disaster Risk Reduction (UNDRR), 2022) stresses the need for urgent proactive measures to maintain and adapt existing infrastructure while designing new systems that account for future climate conditions. Climate change acts as a "threat multiplier", exacerbating existing vulnerabilities while introducing new ones. This underscores the critical importance of taking comprehensive action to build resilient infrastructure and ensure the long-term sustainability of our built environment.

A critical challenge in maintaining existing infrastructure lies in addressing the deterioration of structural components caused by environmental exposure and aging. Among these, corrosion of reinforcement in prestressed concrete structures is a predominant issue, particularly in regions exposed to harsh environments, such as marine areas or where de-icing salts are heavily used. Corrosion-induced damage compromises the mechanical properties of reinforcement, leading to reduced load-carrying capacity and an increased risk of failure. This vulnerability, as described by

Angst (Angst, 2023), can be compared to the Achilles' heel of reinforced concrete: while the material exhibits exceptional strength and durability under most conditions, corrosion undermines its integrity, exposing a critical weakness.

In addition to these challenges, the broader context of climate change further complicates the durability of concrete structures. Rising global temperatures, increased humidity, and more severe environmental conditions accelerate corrosion processes (Intergovernmental Panel on Climate Change (IPCC), 2022). At the same time, the construction industry faces mounting pressure to reduce greenhouse gas (GHG) emissions, with concrete production alone contributing between 5-10% of total anthropogenic CO<sub>2</sub> emissions (Scrivener et al., 2018). This dual challenge of maintaining durability while minimizing environmental impact has made prioritizing sustainability and resilience essential for the industry, requiring innovative strategies to ensure that concrete structures remain both durable and environmentally responsible throughout their lifecycle.

In Canada, the Champlain Bridge in Montreal serves as an example of infrastructure that has been subjected to decades of environmental exposure. After only 57 years of service, the bridge experienced such significant deterioration, including chloride-induced corrosion of its prestressing tendons, that repairs were no longer viable and a replacement structure was completed in 2019. This highlights the importance of understanding the mechanical properties of naturally corroded prestressing steel from aging structures. Such knowledge is critical for assessing the structural capacity of affected structures and developing effective maintenance strategies that ensure structural safety and prolong service life. Furthermore, as infrastructure management agencies and engineers face the challenge of balancing economic constraints with sustainability goals, this insight becomes invaluable in guiding decisions that prioritize rehabilitation over reconstruction.

While the detrimental effects of corrosion are well-recognized, empirical data on naturally corroded post-tensioning (PT) tendons remains scarce. Most existing models rely on laboratory-induced damage, which fails to capture the long-term deterioration seen in the field. This study addresses this critical gap by evaluating tendons recovered from Montreal's former Champlain Bridge, a major Canadian infrastructure asset decommissioned after nearly six decades of harsh environmental exposure. By analyzing naturally corroded tendon samples, this work provides a rare, large-scale assessment of real-world degradation, identifying a robust threshold where

corrosion-induced mass loss triggers a significant transition in structural ductility goals, this insight becomes invaluable in guiding decisions that prioritize rehabilitation over reconstruction.

## **5.2 Background**

### **5.2.1 Previous studies**

Despite the widespread recognition of the detrimental effects of corrosion, most studies on steel reinforcement have primarily focused on conventional reinforcing bars. Extensive research has been conducted to understand their mechanical and durability performance under various corrosive environments (Almusallam, 2001; Fernandez et al., 2015; Fernandez & Berrocal, 2019; Haris & Gadve, 2021; Liu et al., 2024; Papadopoulos et al., 2007). In contrast, research on the corrosion behaviour of prestressing steel, which plays a critical role in the structural integrity of prestressed concrete elements, remains comparatively limited (Bhagwat et al., 2024; Franceschini et al., 2022, 2023; Jeon et al., 2020; Lopes & Simoes, 1999; Lu et al., 2016; Peng et al., 2024). Additionally, a significant proportion of studies on prestressing steel rely on laboratory specimens subjected to accelerated corrosion regimes, in which electro-chemical procedures are used to induce rapid deterioration. While these laboratory investigations provide valuable insights, they often fail to fully replicate the complexities and inherent variability of natural corrosion processes that develop gradually over decades under fluctuating environmental conditions.

Field studies on naturally corroded prestressing steel strands extracted from real structures and exposed to natural environmental conditions over extended periods are scarce (Castel et al., 2000; Jeon et al., 2020), limiting the understanding of real-world degradation mechanisms and their impacts on mechanical properties. This gap is particularly critical because pitting corrosion, a form of localized corrosion commonly observed in naturally corroding structures, has a significant detrimental effect on the mechanical performance of reinforcement. It has been reported that pitting reduces both the tensile strength and deformation capacity of the steel, impairing its overall mechanical behaviour (Jeon et al., 2019; J. Li et al., 2023). Assessing the structural safety of corroded prestressed concrete structures requires a clear understanding of the degraded mechanical behaviour of corroded prestressing steel strands. However, there is currently no established model that quantifies the relationship between corrosion levels and the remaining mechanical properties, nor has a consensus been achieved on how to define such correlations.

The reduction in strength due to corrosion is primarily attributed to the loss of cross-sectional area of the steel tendons, a phenomenon that has been well-documented in the literature (Franceschini et al., 2022; Jeon et al., 2019, 2020; Peng et al., 2024). Several studies have additionally investigated the correlation between corrosion levels and the reduction in the ultimate strain of prestressing steel strands under natural and artificial corrosion conditions, with researchers identifying different regression trends based on their experimental observations (Franceschini et al., 2022, 2023; Jeon et al., 2019; Lu et al., 2016; Wang, Li, et al., 2020; Zhang et al., 2019; Zhang, Wang, Zhang, & Liu, 2017). These studies can be categorized into those focusing on prestressing strands and those examining individual wires.

### **5.2.2 Test parameters and mechanical behaviour of uncorroded steel**

To contextualize the mechanical response of corroded prestressing steel, it is essential to first establish the baseline behaviour and testing conditions of uncorroded (control) steel. Table 5-2 summarizes the key mechanical properties, test setups, and stress–strain characteristics reported in previous studies for uncorroded strands and wires (Franceschini et al., 2022, 2023; Jeon et al., 2019; Lu et al., 2016; Wang, Li, et al., 2020; Zhang et al., 2019; Zhang, Wang, Zhang, & Liu, 2017). Although those studies evaluate similar high-strength steels, their experimental conditions differ in terms of loading rate and gauge length. These parameters influence the measured strain response, particularly ductility-related quantities such as yield strain, ultimate strain, and percent elongation (Calliser & Rethwisch, 2010).

Percent elongation (% EL), in particular, is sensitive to the gauge length used during testing. Therefore, when comparing ductility metrics across studies, the original gauge length ( $l_0$ ) must be explicitly stated to ensure consistency and avoid misinterpretation. In contrast, the elastic modulus is considered a microstructure-insensitive property, as its value is dominated by the stiffness of atomic bonds (Askeland & Wright, 2017), which for previous studies consistently ranges between 195–200 GPa (Franceschini et al., 2022, 2023; Jeon et al., 2019; Lu et al., 2016; Wang, Li, et al., 2020; Zhang et al., 2019; Zhang, Wang, Zhang, & Liu, 2017).

The uncorroded reference steels consistently exhibit high tensile capacities, with ultimate strengths typically between 1,865 and 1,963 MPa and yield strains on the order of 0.008–0.012. Ultimate strain values vary more widely, ranging from 0.03 to 0.075, which may be due to differences in gauge lengths and/or post-elastic deformation behaviour as shown in Figure 5-1. Establishing these

baseline parameters is essential for evaluating corrosion-induced degradation, as observed reductions in strength and ductility must be interpreted considering both the inherent material behaviour and the variability introduced by testing conditions. A bilinear idealization was selected for the steel stress-strain relationship instead of a continuous model like the Ramberg-Osgood relationship to remain strictly consistent with the analytical approach embedded within the finite element analysis software, VecTor2, detailed in Chapter 6. Because VecTor2 utilizes a distinct, well-defined yield point followed by a linear post-yield branch for this material type, adopting the bilinear model ensured perfect compatibility with the finite element program's constitutive formulations.

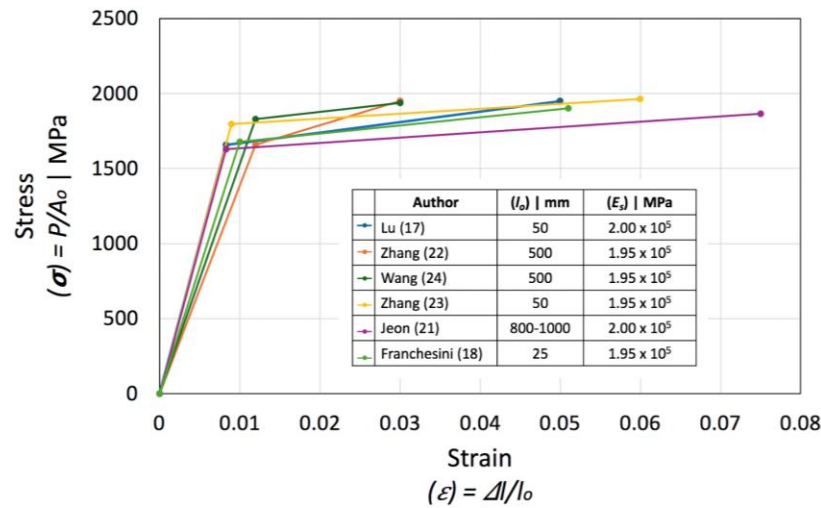


Figure 5-1 Bilinear mechanical model representation of the stress–strain behaviour of uncorroded (reference) steel for studies (Franceschini et al., 2022; Jeon et al., 2019; J. Li et al., 2023; Lu et al., 2016; Zhang et al., 2019; Zhang, Wang, Zhang, & Liu, 2017)<sup>§</sup>

<sup>§</sup> The difference in slopes reported in Franceschini et al. (2022), Jeon et al. (2019) and W. ping Zhang et al. (2019) compared with the other studies, persists even though the modulus of elasticity is consistently stated as  $1.95 \times 10^5$ . This variation arises from author-specific adjustments made when reporting simplified models. For example, Franceschini et al. (2022) define the yield strength as the average stress measured at an imposed strain of  $\varepsilon = 1\%$ . This type of methodological adjustment influences the reported slope of the idealized stress–strain relationship, leading to differences among studies despite the use of similar modulus of elasticity this is in accordance with nonlinear behaviour, as Calliser & Rethwisch (2010) call states, for nonlinear elastic materials



either the tangent modulus or the secant modulus is typically used to describe stiffness. The tangent modulus is defined as the slope of the stress–strain curve at a specific stress level, whereas the secant modulus represents the slope of a line drawn from the origin to a given point on the stress–strain curve.

### **5.2.3 Modelling approaches for the mechanical behaviour of corroded steel**

Lu et al. (Lu et al., 2016) found that the relationship between the strand section loss due to corrosion and the ultimate strain followed an exponential regression trend. This suggests that at lower corrosion levels, the rate of reduction in ultimate strain is more pronounced, but as corrosion progresses, the rate of decrease stabilizes. The critical corrosion threshold in this study was reported as 8%. On the other hand, Zhang et al. (Zhang, Wang, Zhang, & Liu, 2017) and Wang et al. (Wang, Li, et al., 2020) reported a linear regression trend divided into distinct intervals. Their findings indicate that the rate of decrease in ultimate strain does not follow a smooth exponential pattern with increasing strand corrosion, but rather progresses in linear stages, with each interval exhibiting a different slope. The threshold of critical cross-sectional loss is reported as 11% for Zhang et al. and 10.4% for Wang et al.

Zhang et al. (Zhang et al., 2019) investigated the section loss of individual wires and found a linear regression trend divided into intervals. Their study highlighted that the reduction in wire ultimate strain does not follow a single trend but is segmented into stages, with different rates of decrease observed. The threshold of critical cross-sectional loss in this study was reported as 8%.

Meanwhile, Jeon et al. (Jeon et al., 2019) and Franceschini et al. (Franceschini et al., 2022) found that the degradation of the wire's ultimate strain does not follow a single continuous trend, but instead exhibits distinct regimes separated by identifiable corrosion thresholds. Jeon et al. (Jeon et al., 2019) identified a transition threshold beyond which the degradation of ultimate strain accelerates significantly and follows a different exponential trend. The critical cross-sectional loss associated with this transition was 8.1% for pit type 1 (hemispherical), 10.7% for pit type 2 (wide-shallow), and 5.4% for pit type 3 (planar). The different pit types signify the geometric footprint (shape and morphology) of corrosion. These shapes affect how stress concentrates locally and how quickly the ultimate strain degrades, and is described by a “pitting factor” (Stewart, 2009).

The pit types are used to convert a field-measured pitting depth into a quantitative cross-sectional area loss. Pit type 1, as shown in Jeon et al. (Jeon et al., 2019), represents a localized loss of wire cross-section where a circular segment of loss is created. Pit type 2 represents corrosion that has spread laterally across the wire surface more than type 1. Finally, type 3 represents a flattening of the corroded wire, where the corroded wire has a flat section. Franceschini et al. (Franceschini et al., 2022) showed that the degradation of ultimate strain follows different exponential trends as a function of corrosion level, depending on pit morphology. Their results indicate transitions between these exponential behaviours at corrosion levels of approximately 8.1% for pit type 1, 10.7% for pit type 2, and 5.4% for pit type 3, separating distinct degradation regimes.

In addition, Franceschini et al. (Franceschini et al., 2022) introduced a new exponential relationship for assessing ultimate strength, considering different wire pit morphology types. Their study suggests that the degradation mechanism of individual wires is strongly influenced by localized corrosion effects, reinforcing the importance of pit characterization when evaluating the mechanical performance of corroded prestressing wires. In a subsequent study, they developed a simplified model that only requires the maximum pit depth of the most corroded wire to estimate the tensile force and strain curve of corroded strands. This approach eliminates the need for detailed in situ inspections involving pit morphology configuration, cross-sectional loss, or mass loss (Franceschini et al., 2023).

Understanding these variations is crucial for accurately modelling the mechanical behaviour of corroded prestressing strands and assessing the structural integrity of aging infrastructure. The challenges posed by pre-existing damage, uneven corrosion distributions, and exposure to varying service loads add layers of complexity, making field data essential for the development of more reliable models and strategies to evaluate and mitigate corrosion in prestressed structures. This lack of comprehensive understanding presents a critical challenge for engineers tasked with assessing and retrofitting aging prestressed concrete infrastructure. To address these challenges, this study investigates the mechanical properties of naturally corroded prestressing steel extracted from the post-tensioning tendons of the roadway deck slabs from the Champlain Bridge, a structure that had been exposed to 57 years of harsh environmental conditions. Analyzing the tensile behaviour of the corroded post-tensioning wires provides valuable insights into the long-term effects of natural

corrosion on pre-stressing steel. The findings aim to contribute to the development of more accurate predictive models for evaluating the capacity of corroded reinforcement.

Table 5-1 Summary of experimental studies on corroded prestressing strands

| Experiment al study                                              | Lu et al. (Lu et al., 2016)       | Zhang et al. (Zhang, Wang, Zhang, & Liu, 2017)                                          | Wang et al. (Wang, Li, et al., 2020)          | Zhang et al. (Zhang et al., 2019) | Jeon et al. (Jeon et al., 2019) | Franceschini et al. (Franceschini et al., 2022) |
|------------------------------------------------------------------|-----------------------------------|-----------------------------------------------------------------------------------------|-----------------------------------------------|-----------------------------------|---------------------------------|-------------------------------------------------|
| <b>1.1 Experimental program</b>                                  |                                   |                                                                                         |                                               |                                   |                                 |                                                 |
| Type of strand                                                   | Helical                           | Helical                                                                                 | Helical                                       | Helical                           | Helical                         | Helical                                         |
| Type of corrosion                                                | Artificial: Impressed current     | Artificial: Climate box                                                                 | Artificial: Climate box                       | Artificial: Impressed current     | Natural: Chloride               | Natural: Chloride                               |
| Reference Samples                                                | 2                                 | 3                                                                                       | 1                                             | Test on wires                     | 3                               | 4                                               |
| Corroded Samples                                                 | 12                                | 7                                                                                       | 18                                            |                                   | 13                              | 19                                              |
| Equivalent diameter of strands (mm)                              | 15.2                              | 15.2                                                                                    | 15.2                                          | 15.2                              | 15.2                            | 12.9                                            |
| Characteristics of artificial corrosion                          | 100 ( $\mu\text{A}/\text{cm}^2$ ) | Salt fog (5% salt solution)                                                             | By using a wet sponge: 120 min on 120 min off | 100 ( $\mu\text{A}/\text{cm}^2$ ) | N/A                             | N/A                                             |
| Maximum strand/wire section loss (%)                             | Strand: 20.4%                     | Strand: 27.5%                                                                           | Strand: 27.5%                                 | Wire: 31.4%                       | Wire: 20.3%                     | Wire: 69.2%                                     |
| <b>1.2 Mechanical Properties of uncorroded (reference) steel</b> |                                   |                                                                                         |                                               |                                   |                                 |                                                 |
| Loading rate                                                     | 0.2 kN/s                          | 1.0 mm/min (until failure 1 <sup>st</sup> . wire)<br>2.0 mm/min (for residual capacity) | 1 mm/min 5.5% (elongation rate)               | Not indicated                     | 5.0 mm/min                      | Displacement control method                     |
| Gauge length ( $l_0$ ) mm                                        | 50                                | 500                                                                                     | 500                                           | 50                                | 800-1000                        | 25                                              |
| Yield Strength ( $f_y$ ) MPa                                     | 1657.6                            | 1830                                                                                    | 1830                                          | 1795.6                            | 1628                            | 1677.25                                         |
| Yield Strain ( $\epsilon_y$ )                                    | 0.0083                            | 0.012                                                                                   | 0.012                                         | 0.009                             | 0.0083                          | 0.01                                            |

|                                                     |                                                                                                                                          |                                                                                                                                                                                                                                                                                    |                                                                                                                                                                                                                                                                |                                                                                                                                                                                                                                                                                                                                                |                                                                                                                                                                                                                                                                                                                           |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                        |
|-----------------------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>Ultimate Strength (<math>f_u</math>) MPa</b>     | 1950                                                                                                                                     | 1938                                                                                                                                                                                                                                                                               | 1938                                                                                                                                                                                                                                                           | 1963                                                                                                                                                                                                                                                                                                                                           | 1865                                                                                                                                                                                                                                                                                                                      | 1901.75                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                |
| <b>Ultimate Strain (<math>\epsilon_u</math>)</b>    | 0.05                                                                                                                                     | 0.03                                                                                                                                                                                                                                                                               | 0.03                                                                                                                                                                                                                                                           | 0.06                                                                                                                                                                                                                                                                                                                                           | 0.075                                                                                                                                                                                                                                                                                                                     | 0.051                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                  |
| <b>Modulus of Elasticity (<math>E_s</math>) MPa</b> | $2.00 \times 10^5$                                                                                                                       | $1.95 \times 10^5$                                                                                                                                                                                                                                                                 | $1.95 \times 10^5$                                                                                                                                                                                                                                             | $2.00 \times 10^5$                                                                                                                                                                                                                                                                                                                             | $1.95 \times 10^5$                                                                                                                                                                                                                                                                                                        | $1.95 \times 10^5$                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |
| <b>1.3 Stress-strain relationship</b>               |                                                                                                                                          |                                                                                                                                                                                                                                                                                    |                                                                                                                                                                                                                                                                |                                                                                                                                                                                                                                                                                                                                                |                                                                                                                                                                                                                                                                                                                           |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                        |
| <b>Main model parameter</b>                         | Section loss (strand)                                                                                                                    | Section loss (strand)                                                                                                                                                                                                                                                              | Section loss (strand)                                                                                                                                                                                                                                          | Section loss (wire)                                                                                                                                                                                                                                                                                                                            | Section loss (wire)                                                                                                                                                                                                                                                                                                       | Section loss (wire)                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                    |
| <b>Regression trend: ultimate strain</b>            | Exponential                                                                                                                              | Linear in intervals                                                                                                                                                                                                                                                                | Linear in intervals                                                                                                                                                                                                                                            | Linear in intervals                                                                                                                                                                                                                                                                                                                            | Exponential in intervals                                                                                                                                                                                                                                                                                                  | Exponential in intervals                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                               |
| <b>Equation</b>                                     | $\epsilon_{corr.} = \epsilon_{control} \cdot [0.1 + 0.9e^{(-20f(D))}]$ <p>where f(D) represents a function of the area damage factor</p> | $\epsilon_{u,corr} = \begin{cases} \epsilon_{py} + (\rho_c - \rho) \frac{(\epsilon_{pu} - \epsilon_{py})}{\rho_c} \end{cases}$ <p>when:<br/> <math>\begin{cases} \rho \leq \rho_c \\ \rho &gt; \rho_c \end{cases}</math></p> <p>where <math>\rho</math> is the corrosion level</p> | $\epsilon_{u,corr} = \begin{cases} \epsilon_u(1 - 6.2\rho) \\ \frac{\sigma_u}{E} \approx 0.35 \end{cases}$ <p>when:<br/> <math>\begin{cases} \rho \leq \rho_c \\ \rho &gt; \rho_c \end{cases}</math></p> <p>where <math>\rho</math> is the corrosion level</p> | $\epsilon_{u,corr} = \begin{cases} \epsilon_y + \epsilon_{shb,0}(1 - 12.5g_s) \\ \epsilon_y \end{cases}$ <p>when:<br/> <math>\begin{cases} g_s \leq 0.08 \\ g_s &gt; 0.08 \end{cases}</math></p> <p>where <math>g_s</math> is the corrosion level and <math>\epsilon_{shb,0}</math> is the hardening branch length for the uncorroded wire</p> | $\epsilon_{u,corr} = \begin{cases} c\eta^2 + d\eta + e \\ f\eta^g \end{cases}$ <p>when:<br/> <math>\begin{cases} d_p &lt; 0.5 \\ d_p \geq 0.5 \end{cases}</math></p> <p>where <math>\eta</math> is the cross-section loss, and c, d, e, f and g are defined from regression analysis and depend on pit configurations</p> | $\epsilon_{u,corr} = \begin{cases} \epsilon_{py} + \frac{(f_{pu,corr} - f_{py,0})}{E_{s,0}''} \\ \epsilon_{pp,0} + \frac{(f_{pu,corr} - f_{pp,0})}{E_{s,0}'} \\ \frac{f_{pu,corr}}{E_{s,0}} \end{cases}$ <p>when:<br/> <math>\begin{cases} \mu &lt; \mu_{lim}, f_{py,0} &lt; f_{pu,corr} \leq f_{pu,0} \\ \mu \geq \mu_{lim}, \begin{cases} f_{pp,0} &lt; f_{pu,corr} \leq f_{py,0} \\ f_{pu,corr} &lt; f_{pp,0} \end{cases} \end{cases}</math></p> <p>where <math>\mu_{loss}</math> is the cross-sectional loss, <math>\mu_{lim}</math> is the critical value of cross-sectional loss, <math>f_{pp,0}</math> is the stress at the end of the elastic stage, and <math>E_{s,0}'</math> and <math>E_{s,0}''</math> are the uncorroded yielding and hardening modulus, respectively.</p> |
| <b>Regression trend: ultimate strength</b>          | Linear                                                                                                                                   | Linear in intervals                                                                                                                                                                                                                                                                | Linear in intervals                                                                                                                                                                                                                                            | Linear                                                                                                                                                                                                                                                                                                                                         | Linear                                                                                                                                                                                                                                                                                                                    | Exponential                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                            |

|                                                                                                                                  |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                            |                                                                                                                                                                                                                                                                                                                                                                                                                                                                       |                                                                                              |                                                                                                                                                                                            |                                                                                                                                                                                                                                                                                                                                                                                       |
|----------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| $f_{u,corr.} = f_{u,control} \cdot [1 - 2.8f(D)]$ <p>where <math>f(D)</math> represents a function of the area damage factor</p> | $\sigma_u = \begin{cases} E_{control} \cdot \varepsilon \\ f_{py} + (\varepsilon - \varepsilon_{py}) \frac{(f_{pu} - f_{py})}{(\varepsilon_{pu} - \varepsilon_{py})} \end{cases}$ <p>Failure</p> <p>when:</p> $\begin{cases} \varepsilon \leq \varepsilon_{py}, \rho \leq \rho_c \\ \varepsilon_{py} < \varepsilon \leq \varepsilon_{pu} - \frac{\rho}{\rho_c} (\varepsilon_{pu} - \varepsilon_{py}), \\ \text{and } \rho \leq \rho_c \\ \varepsilon > \varepsilon_{py}, \rho > \rho_c \end{cases}$ <p>where <math>\rho_c</math> is the critical corrosion level = 11%</p> | $\sigma_u = \begin{cases} E_{control} \cdot \varepsilon \\ f_{py,corr} + E_{pp}(\varepsilon - \varepsilon_{py,corr}) \end{cases}$ <p>Failure</p> <p>when:</p> $\begin{cases} \varepsilon \leq \varepsilon_{py,corr}, \rho \leq \rho_c \\ \varepsilon_{py,corr} < \varepsilon \leq \varepsilon_{pu,corr}, \rho \leq \rho_c \\ \varepsilon > \varepsilon_{py,corr}, \rho > \rho_c \end{cases}$ <p>where <math>\rho_c</math> is the critical corrosion level = 10.4%</p> | $\sigma_u = \sigma_{u,0}(1 - 1.209g_s)$ <p>where <math>g_s</math> is the corrosion level</p> | $\sigma_u = a_n + b$ <p>where <math>\eta</math> is the cross-section loss, and <math>a</math> and <math>b</math> are defined from regression analysis and depend on pit configurations</p> | $\sigma_u = \begin{cases} f_{pu,0}(e^{-1.588 \cdot \mu_{loss}}) \\ f_{pu,0}(e^{-1.377 \cdot \mu_{loss}}) \\ f_{pu,0}(e^{-1.035 \cdot \mu_{loss}}) \end{cases}$ <p>for: <math>\begin{cases} \text{Pit type 1: hemispherical} \\ \text{Pit type 2: wide - shallow} \\ \text{Pit type 3: planar} \end{cases}</math></p> <p>where <math>\mu_{loss}</math> is the cross-sectional loss</p> |
|----------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|

Table 5-2 Summary of experimental studies on corroded prestressing strands

| Experimental study                      | Lu et al. (Lu et al., 2016)       | Zhang et al. (Zhang, Wang, Zhang, & Liu, 2017) | Wang et al. (Wang, Li, et al., 2020)          | Zhang et al. (Zhang et al., 2019) | Jeon et al. (Jeon et al., 2019) | Franceschini et al. (Franceschini et al., 2022) |
|-----------------------------------------|-----------------------------------|------------------------------------------------|-----------------------------------------------|-----------------------------------|---------------------------------|-------------------------------------------------|
| <b>Experimental program</b>             |                                   |                                                |                                               |                                   |                                 |                                                 |
| Type of strand                          | Helical                           | Helical                                        | Helical                                       | Helical                           | Helical                         | Helical                                         |
| Type of corrosion                       | Artificial: Impressed current     | Artificial: Climate box                        | Artificial: Climate box                       | Artificial: Impressed current     | Natural: Chloride               | Natural: Chloride                               |
| Characteristics of artificial corrosion | 100 ( $\mu\text{A}/\text{cm}^2$ ) | Salt fog (5% salt solution)                    | By using a wet sponge: 120 min on 120 min off | 100 ( $\mu\text{A}/\text{cm}^2$ ) | N/A                             | N/A                                             |
| Maximum strand/wire section loss (%)    | Strand: 20.4%                     | Strand: 27.5%                                  | Strand: 27.5%                                 | Wire: 31.4%                       | Wire: 20.3%                     | Wire: 69.2%                                     |
| <b>Stress-strain relationship</b>       |                                   |                                                |                                               |                                   |                                 |                                                 |
| Main model parameter                    | Section loss (strand)             | Section loss (strand)                          | Section loss (strand)                         | Section loss (wire)               | Section loss (wire)             | Section loss (wire)                             |
| Regression trend: ultimate strain       | Exponential                       | Linear in intervals                            | Linear in intervals                           | Linear in intervals               | Exponential in intervals        | Exponential in intervals                        |
| Regression trend: ultimate strength     | Linear                            | Linear in intervals                            | Linear in intervals                           | Linear                            | Linear                          | Exponential                                     |

## 5.3 Materials and methods

### 5.3.1 Champlain Bridge

#### 5.3.1.1 Overview of the bridge and its environmental exposure conditions

The original Champlain Bridge (1962–2019) in Canada, which connected Montreal to Brossard over the Saint Lawrence River, was exposed to harsh environmental conditions and was decommissioned in 2019 after only 57 years of service, a lifespan falling significantly short of modern design standards. Frequent application of de-icing salts during Quebec’s severe winters resulted in significant chloride ingress, particularly affecting the bridge’s deck system, as this element was the first barrier against external elements. Exposure to a chloride environment was further exacerbated by inadequate drainage systems that allowed water infiltration at critical locations, especially the joints between the girders and the cast-in-place infill slabs.

The deck from the south approach span consisted of seven prestressed concrete I-girders connected by six transversely PT cast-in-place infill slabs, as shown in Figure 5-2. Straight transverse prestressing tendons embedded within the deck provided continuity between the slabs and the girder top flanges, facilitating efficient load transfer. However, this design made the tendons especially susceptible to chloride-laden moisture infiltrating through the joints. Over decades of exposure, this harsh environment compromised the tendons' integrity, resulting in progressive deterioration and a reduction in the slabs' structural capacity.

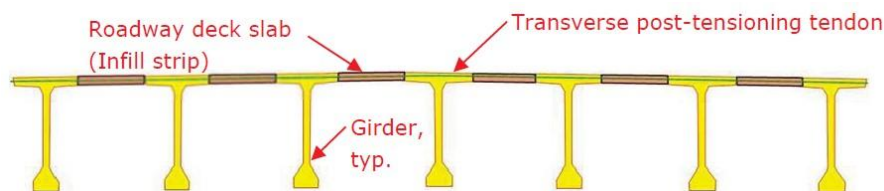


Figure 5-2 Typical deck cross-section at Brossard approach span (Carlin, 2017)

The slabs functioned as a one-way system, spanning transversely across the bridge deck width and transferring the traffic load to the supporting girders. The slab depths ranged from 200 to 215 mm, with embedded PT tendons spaced at an average of 1,000 mm, serving as the primary reinforcement. The tendons were made of high-strength steel, each comprising twelve 7-mm diameter wires bundled within a metal duct and having an initial tensile capacity of 1,690 MPa. The arrangement of the 12 wires within the metal duct, along with the surrounding grout, is shown



in Figure 5-3. The deck system of the superstructure was subjected to prolonged exposure to harsh environmental conditions, where inadequate drainage accelerated the degradation of its structural components. The lack of a waterproofing membrane, combined with heavy application of de-icing salts, enabled salt-laden water to enter the slabs. This infiltration ultimately led to the corrosion of the embedded reinforcement. Additional information about the history and exposure of the bridge can be found in Chapter 1.



Figure 5-3. Typical details of twelve-wire post-tensioning tendons (parallel wire arrangement)

### 5.3.2 Details of post-tensioning strand samples

This study utilized 180 smooth post-tensioning wires, each measuring 1,950 mm in length and having an original diameter of 7 mm (i.e., uncorroded cross-sectional area of 38.48 mm<sup>2</sup>). The collected samples were recovered from fifteen PT strands extracted from the eight deck slabs and exhibited a range of corrosion levels. To establish a baseline for comparison, uncorroded control samples were also collected from diaphragm elements in the same bridge section, providing reference data on the geometrical and mechanical properties of undamaged strands (Figure 5-4). These diaphragms did not experience any corrosion damage to their PT tendons, nor did they undergo any prior structural load testing. A total of 981 strand samples were tested in tension, comprising six uncorroded specimens and 975 corroded specimens.



Figure 5-4 Control wire samples obtained from diaphragm segments

Figure 5-5 presents the isolated smooth wires from a single post-tensioning tendon. Each tendon's wires were cut into segments ranging from 200 mm to 500 mm, with the cutting locations selected based on the distribution of damage features visually identified along the wire lengths. Areas with extensive pitting corrosion were avoided near the ends to minimize the risk of premature failure at the grips during testing. This process yielded 975 individual samples for analysis, in addition to 6 control samples representing the uncorroded condition. The 975 samples displayed varying levels of corrosion. The tendons were identified according to a labeling system corresponding to their location within the deck structure. Each label indicates the slab, edge, and tendon number. For example, S1A1-S1B1 - W1 (0-30) refers to tendon 1, visible from edges A and B of slab 1, respectively. W1 denotes wire 1 out of the 12 wires composing the tendon, and the range 0–30 indicates the 30 cm segment of the wire examined, starting from the exposed edge ( $x = 0$  cm) and extending to 30 cm along the tendon length.



Figure 5-5 Isolated wires from a single post-tensioning tendon

### 5.3.3 Laboratory test methods

#### 5.3.3.1 Monotonic tensile tests on uncorroded and corroded tendons

To evaluate the stress–strain behaviour of both corroded and uncorroded prestressing wires, mechanical tensile tests were conducted using a universal testing machine with a capacity of 100 kN, as shown in Figure 5-6. The direct monotonic tests were performed under displacement control, with loading rates varying between 2 and 15 mm/min. selected based on specimen length to ensure compliance with the strain-rate limits prescribed by ASTM A370 (ASTM A370, 2024). Each wire specimen was secured at both ends using grips, ensuring that the load was directly transferred to the bar. The gripping pressure was closely monitored and adjusted throughout the tests to prevent premature failure of the wires at the grips. During testing, the applied load and the total machine displacement were continuously recorded using an automatic data acquisition system until the specimen reached failure.



Figure 5-6 Tensile test set-up

The ultimate steel strain was calculated using the measured displacements and the original length of the wire, while the steel stress was determined using the nominal cross-sectional area of the uncorroded tendon.

$$\epsilon_{ult} = \frac{\Delta_{displacement}}{L_{wire}} \quad \text{Equation 5-1}$$

$$\sigma_{ult} = \frac{F}{A_{nominal}} \quad \text{Equation 5-2}$$

### 5.3.4 Measurements of mass loss induced by corrosion

Prior to tendon extraction, the PT concrete slabs were subjected to structural testing to evaluate their flexural performance after decades of service. Specifically, four-point bending tests were conducted to simulate service-level loading conditions and assess the structural response of the corroded slabs. The detailed findings of the structural testing program are reported in Chapter 4.

Following the four-point bending tests of the slabs, the prestressing strands were extracted from each slab for mass loss analysis. To prepare the segments for analysis, rust products were removed according to the ASTM G1 standard (ASTM G1, 2017). This cleaning process involved immersing the segments in a solution of 500 mL of 37% hydrochloric acid, 3.5 g of hexamethylene tetramine, and 500 mL of reagent water. The mass loss due to corrosion,  $m_{corr}$ , was then calculated for each segment using Equation 5-3:

$$m_{corr} = \frac{(m_0 - m)}{m_0} \times 100 \quad (\%) \quad \text{Equation 5-3}$$

where  $m_0$  is the initial mass of the uncorroded wire sample obtained from a diaphragm section, and  $m$  is the mass of the corroded strand segment after cleaning.

Six of the 981 samples, labeled C-1 through C-6 and shown in Figure 5-4, were uncorroded and were used as reference specimens for the mass-loss calculations.

## 5.4 Results and discussion

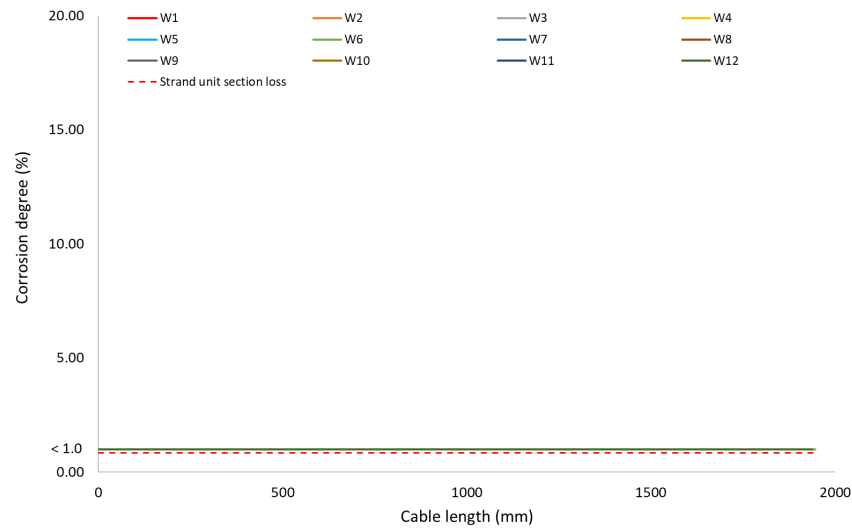
### 5.4.1 Section loss quantification

Figure 5-7 presents the mass loss profiles for the 12 prestressing wires within each tendon, labeled W1 to W12, as well as each strand's unit section loss. The strand-unit section loss is calculated as the sum of the individual wire-unit section losses within the strand, divided by the original cross-sectional area of the strand. This quantity does not represent an average of the individual wire losses, but rather an integrated strand-level measure of section loss governing the tendon's effective load-carrying capacity. As a result, the strand-unit section loss may plot below the individual wire

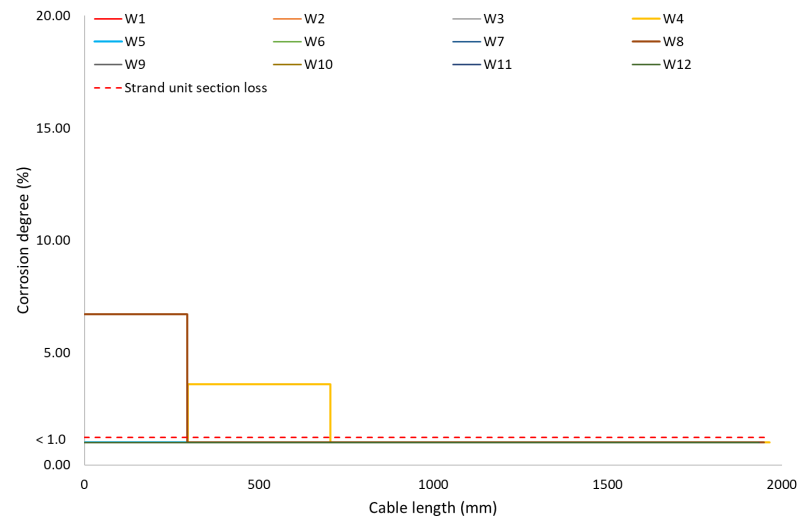
loss curves when corrosion is localized to only a subset of wires. Including the strand-unit section loss alongside the individual wire losses enables a direct comparison between localized corrosion effects and the overall cross-sectional degradation of the prestressing tendon. While wire-level measurements capture the nonuniform and localized nature of corrosion along individual wires, the strand-unit section loss provides an integrated measure of section loss at different locations.

Since the majority of measurements exhibited corrosion levels below 1%, a value labeled “< 1.0” was introduced on the y-axis to improve visualization. This value represents corrosion degrees lower than 1% and does not correspond to a discrete numerical measurement. Corrosion levels exceeding 1% are shown using the actual percentage values indicated on the y-axis. The mass loss analysis of the 12 prestressing wires revealed variations in corrosion levels along the wire lengths. However, higher degrees of corrosion were generally observed near the wire ends. For example, W5 from specimen S7A2 was significantly corroded. These ends correspond to the connection between the slab and the top flange of the girder, where a construction joint existed. It is likely that saltwater ingress occurred at these joints, leading to more severe corrosion in these areas. These findings are consistent with the NDT tendon corrosion assessments, which indicated higher corrosion rate measurements at the slab/girder connection, as further detailed in Chapter 3.

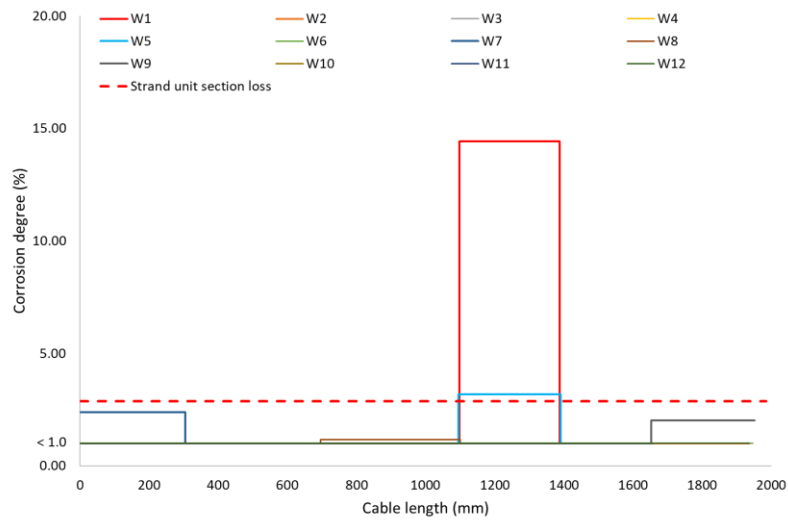
Reliance on the strand-unit section loss alone may be misleading when evaluating corrosion damage in prestressing steel. Because this parameter represents a combined measure normalized by the original strand area, it can mask highly localized corrosion affecting individual wires. As observed in Figure 5-7, locations where one or more wires exhibit corrosion levels exceeding 1% may still correspond to relatively low strand-unit section loss values, particularly when corrosion is concentrated in a limited number of wires. Since failure of prestressing strands is often governed by the most severely corroded wire rather than the average or cumulative section loss, assessment based solely on strand-level metrics may underestimate the structural implications of localized corrosion. Accordingly, evaluation of corrosion in prestressing tendons should consider both strand-level section loss and the maximum wire-level corrosion to capture the combined effects of global capacity reduction and localized damage, as demonstrated in Section 5.4.4 .



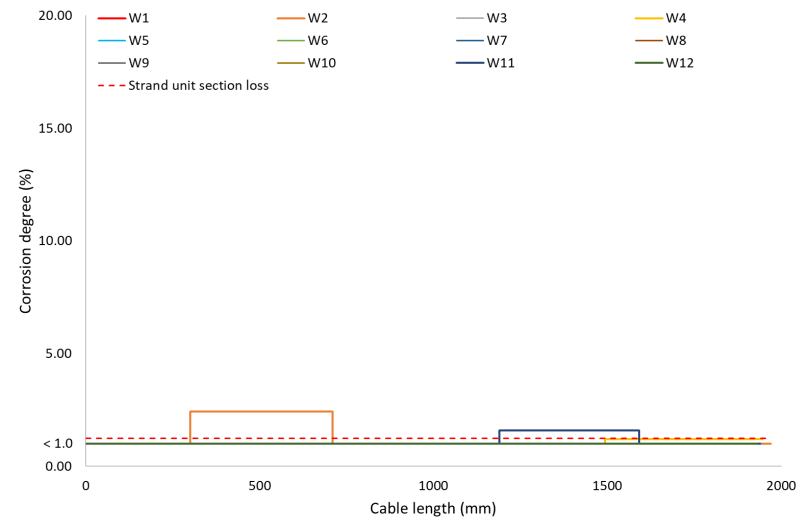
(a) S1A1-S1B1



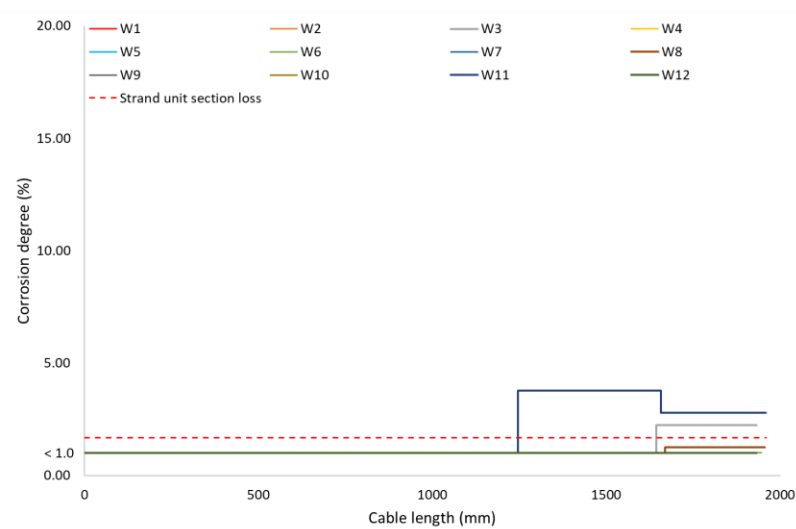
(b) S1A2-S1B2



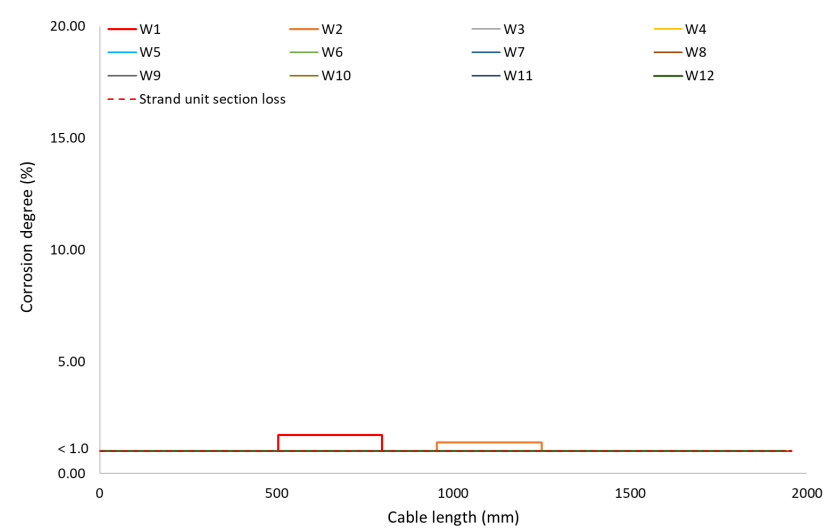
(c) S2A2-S2B2



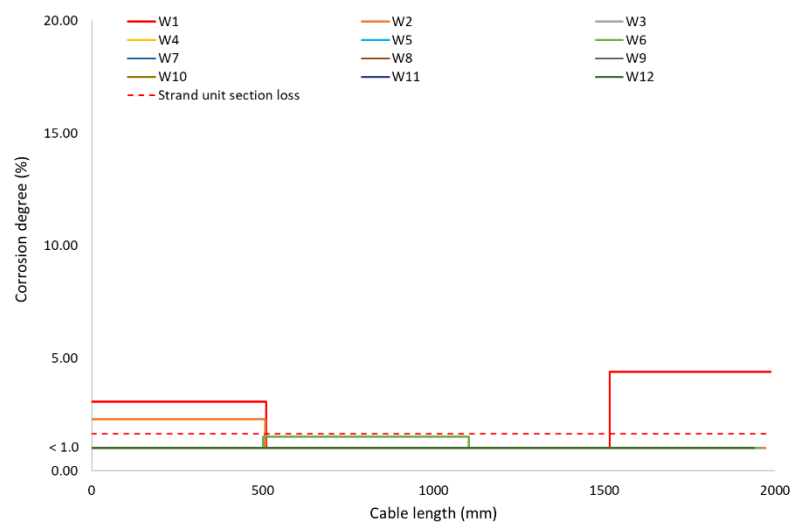
(d) S2A3-S2B3



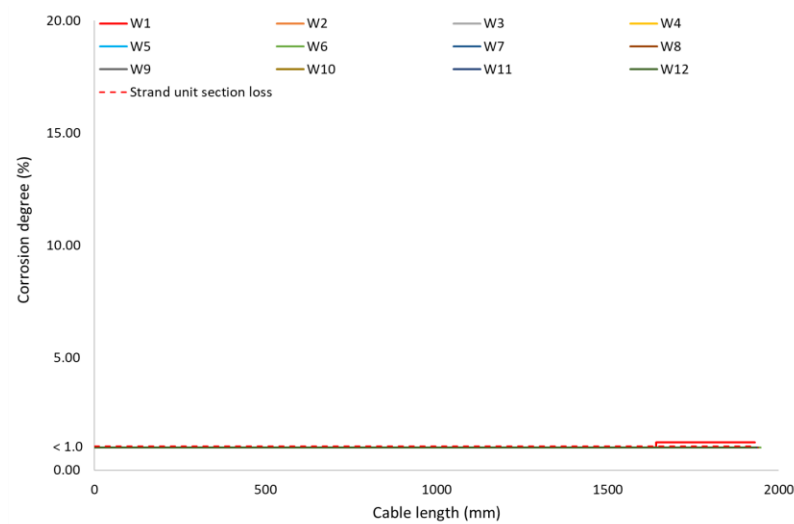
(e) S3A1-S3B1



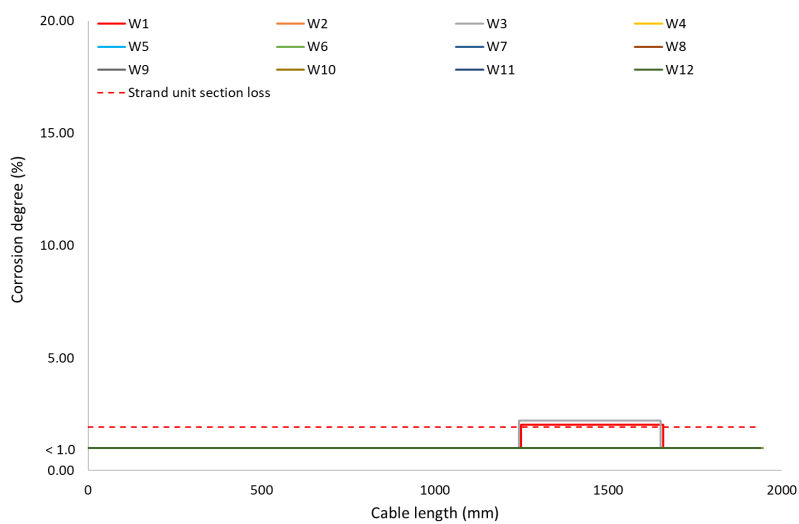
(f) S3A2-S3B2



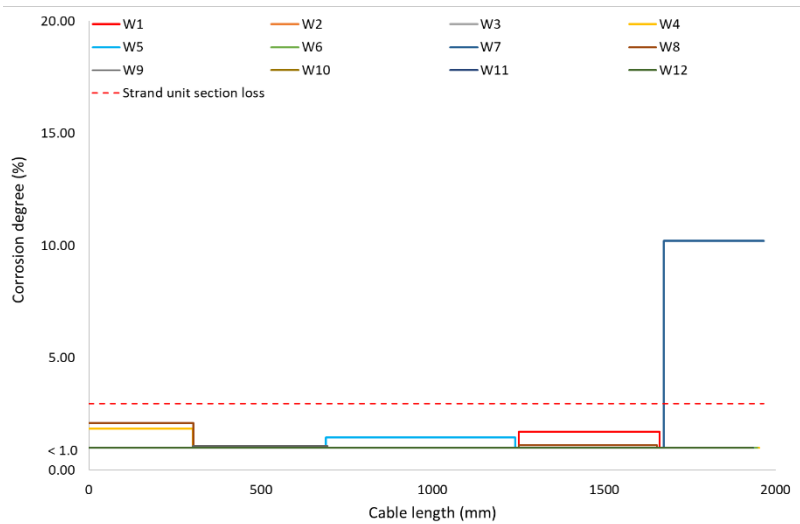
(g) S4A2-S4B2



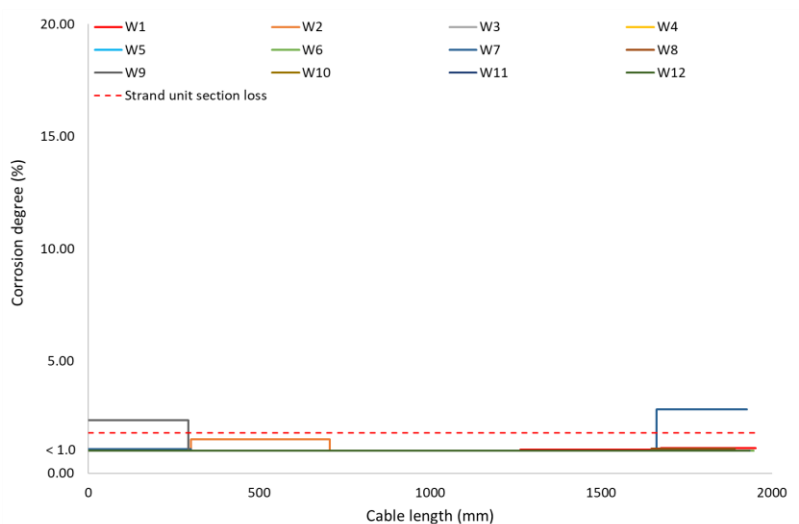
(h) S4A3-S4B3



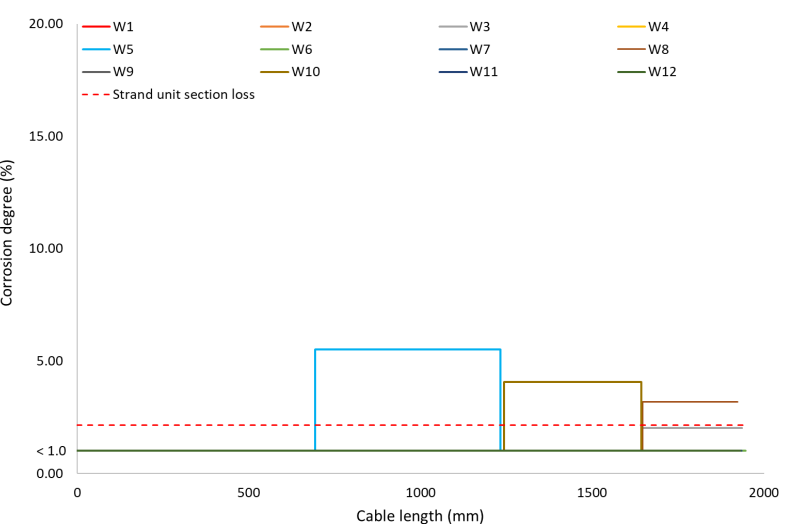
(i) S5A2-S5B2



(j) S5A3-S5B3

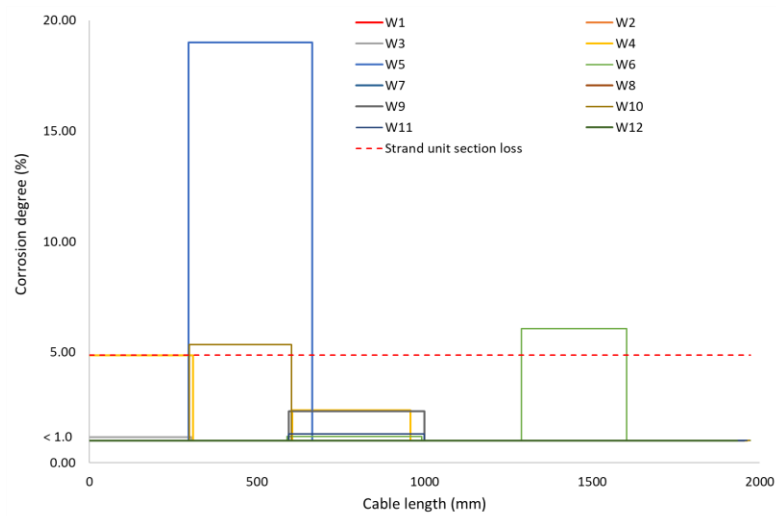


(k) S6A2-S6B2

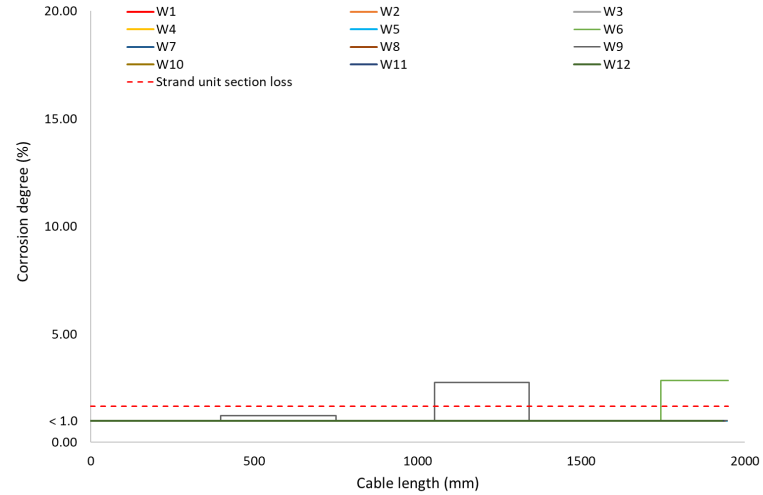


(l) S6A3-S6B3

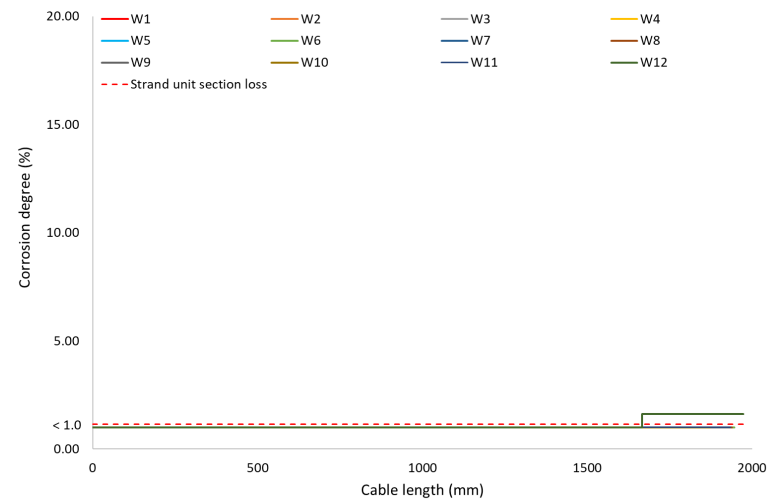




(m) S7A2-S7B2



(n) S7A3-S7B3



(o) S8A3-S8B3

Figure 5-7 Fifteen average mass loss profiles due to corrosion for the 12 prestressing wires within each tendon

### 5.4.2 Mechanical characterization of uncorroded strands

Establishing a baseline for the mechanical performance of uncorroded reinforcement bars is essential for evaluating the effects of corrosion. Figure 5-8 presents the stress-strain behaviour of the uncorroded control prestressing wires. The six tested wires exhibited typical bi-linear elasto-plastic stress-strain curves for high-strength steel, characterized by an initial linear-elastic region extending to approximately 1,400 MPa, followed by a yield point and subsequent linear-plastic deformation. The average bi-linear stress-strain curve, bounded by the 5<sup>th</sup> and 95<sup>th</sup> percentile curves, demonstrates relatively low variability in the mechanical properties among the control wires, indicating a consistent material. The 95<sup>th</sup> and 5<sup>th</sup> percentile confidence intervals were obtained using a normal distribution, which highlights the natural dispersion in the material properties. The observed consistency in the control wires allows for greater confidence when attributing any observed degradation in the corroded samples to the effects of corrosion, rather than inherent material variability.

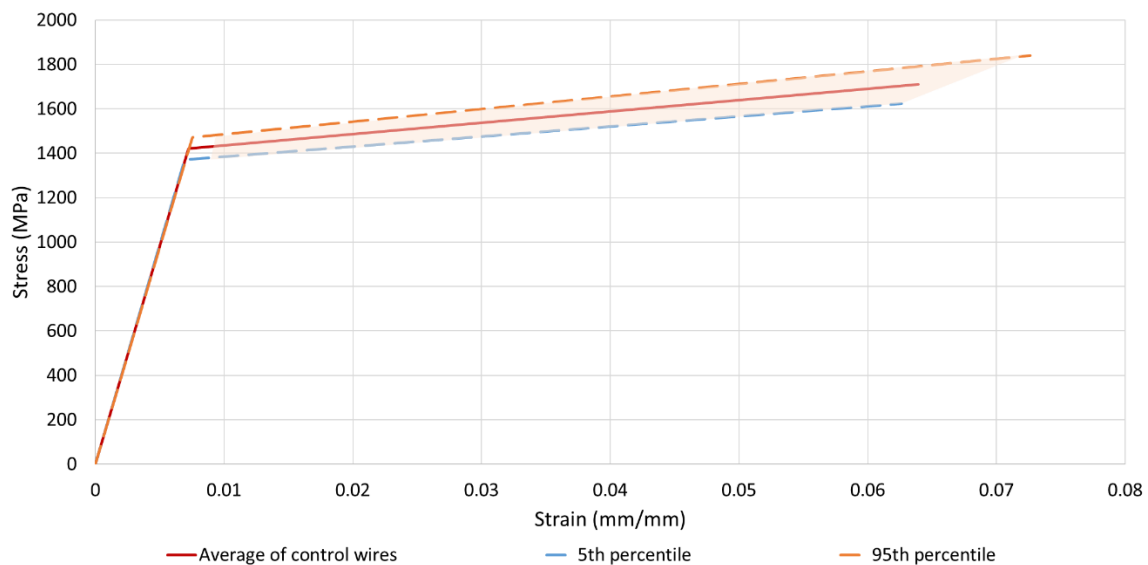


Figure 5-8 Tensile test results of uncorroded wires

Table 5-3 presents a summary of the key mechanical properties obtained from the tensile tests on the six uncorroded control wires (C1 to C6). Due to limitations in the availability of suitable uncorroded material, only six control wires could be extracted and tested. The properties reported include yield stress (MPa), yield strain (mm/mm), ultimate tensile strength (MPa), and ultimate strain (mm/mm). The yield stress and yield strain were defined using the 0.2% offset method, where

the yield point is identified as the intersection between the stress-strain curve and a line parallel to the linear-elastic region shifted by 0.002 mm/mm. Summary statistics, including the average, standard deviation, coefficient of variation (COV), 5<sup>th</sup> percentile, and 95<sup>th</sup> percentile values, are also provided.

Table 5-3 Summary of the mechanical properties of six uncorroded wires

| Wire                     | Length (mm) | Yield stress, $f_y$<br>(MPa) | Yield Strain,<br>$\epsilon_y$ (mm/mm) | Ultimate stress, $f_u$<br>(MPa) | Ultimate strain,<br>$\epsilon_u$ (mm/mm) |
|--------------------------|-------------|------------------------------|---------------------------------------|---------------------------------|------------------------------------------|
| <b>C1</b>                | 430         | 1431                         | 0.0073                                | 1706                            | 0.0697                                   |
| <b>C2</b>                | 430         | 1372                         | 0.0069                                | 1641                            | 0.0682                                   |
| <b>C3</b>                | 430         | 1415                         | 0.0072                                | 1673                            | 0.0626                                   |
| <b>C4</b>                | 431         | 1414                         | 0.0072                                | 1693                            | 0.0702                                   |
| <b>C5</b>                | 431         | 1466                         | 0.0075                                | 1730                            | 0.0652                                   |
| <b>C6</b>                | 431         | 1430                         | 0.0073                                | 1700                            | 0.0696                                   |
| <b>Average</b>           | 430.5       | 1421                         | 0.0072                                | 1691                            | 0.0676                                   |
| <b>Std. dev.</b>         | 0.55        | 30.63                        | 0.00018                               | 30.43                           | 0.00302                                  |
| <b>COV (%)</b>           | 0.13        | 2.16                         | 2.54                                  | 1.80                            | 4.47                                     |
| <b>5<sup>th</sup> %</b>  | 429.60      | 1371                         | 0.0069                                | 1641                            | 0.0626                                   |
| <b>95<sup>th</sup> %</b> | 431.40      | 1472                         | 0.0075                                | 1741                            | 0.0726                                   |

The average yield stress of the six control wires was 1,421 MPa, with a standard deviation of 30.63 MPa. The coefficient of variation (COV) for yield stress was 2.16%, indicating relatively low variability in this property. Similarly, the average ultimate tensile strength was 1,691 MPa, with a standard deviation of 30.43 MPa and a COV of 1.80%. The average yield strain was 0.0072 mm/mm (standard deviation = 0.00018 mm/mm, COV = 2.54%), and the average ultimate strain was 0.0676 mm/mm (standard deviation = 0.00302 mm/mm, COV = 4.47%).

The relatively low coefficients of variation for all measured properties, particularly for the yield stress and ultimate tensile strength (all below 5%), indicate good consistency and homogeneity in the mechanical properties of the control wires. The 5<sup>th</sup> and 95<sup>th</sup> percentile values provide a reasonable range of expected performance for uncorroded wires of this type. The higher COV for ultimate strain (4.47%) compared to yield stress and ultimate strength suggests that ductility is

slightly more variable than strength. The reported 5<sup>th</sup> and 95<sup>th</sup> percentile values for ultimate strain (0.0626 mm/mm and 0.0726 mm/mm, respectively) reflect appropriate precision.

### 5.4.3 Mechanical performance of corroded strands

During the slab tests, yielding of the prestressing tendons was identified in specimens S4A2-S4B2, S3A2-S3B2, S7A2-S7B2, S6A3-S6B3, S2A2-S2B2, S8A3-S8B3, S1A2-S1B2, S5A2-S5B2, and S4A3 by converting the measured ultimate slab loads to tendon stresses based on the corresponding bending moments and internal lever arm, and comparing these stresses with the yield stress obtained from the control tendon tests. Following the structural testing, strands were extracted from the slabs and individual wires were tested in direct tension. Therefore, for slabs in which tendon yielding occurred, the extracted wires may have experienced prior plastic deformation. As a result, the tensile response of these wires represents the residual mechanical behaviour of prestressing steel after combined corrosion exposure and structural-level loading. As the objective of the tensile testing was to characterize the post-loading mechanical response of corroded prestressing steel, no attempt was made to correct the measured properties for prior yielding during slab testing. Table 5-4 summarizes the estimated tendon stresses at slab ultimate loads under simply-supported conditions, showing the specimens that exceeded the control tendon yield stress.

Table 5-4 Estimated tendon stresses at ultimate loads

| Slab ID   | Estimated tendon stress at ultimate conditions (MPa) | Average control yield stress (MPa) |
|-----------|------------------------------------------------------|------------------------------------|
| S1A1-S1B1 | 1118                                                 | 1421                               |
| S1A2-S1B2 | 2334                                                 |                                    |
| S2A2-S2B2 | 1615                                                 |                                    |
| S2A3-S2B3 | 1309                                                 |                                    |
| S3A1-S3B1 | 1144                                                 |                                    |
| S3A2-S3B2 | 2246                                                 |                                    |
| S4A2-S4B2 | 1699                                                 |                                    |
| S4A3-S4B3 | 1727                                                 |                                    |
| S5A2-S5B2 | 2347                                                 |                                    |
| S5A3-S5B3 | 1183                                                 |                                    |
| S6A2-S6B2 | 1280                                                 |                                    |
| S6A3-S6B3 | 1737                                                 |                                    |
| S7A2-S7B2 | 1679                                                 |                                    |
| S7A3-S7B3 | 1239                                                 |                                    |
| S8A3-S8B3 | 1451                                                 |                                    |

A total of 975 tensile tests were conducted on corroded specimens to evaluate the effects of corrosion on ultimate strain and ductility. For brevity, only one representative stress-strain curve, S4A2-S4B2 W2 (0-50), is presented in Figure 5-9 to illustrate the general trend observed across all tests. Based on the experimental tests, no change in elastic modulus or yield strength is observed in the corroded specimens. This observation is consistent with the discussion presented in Section 3.2, where the modulus of elasticity is described as unaffected by corrosion.

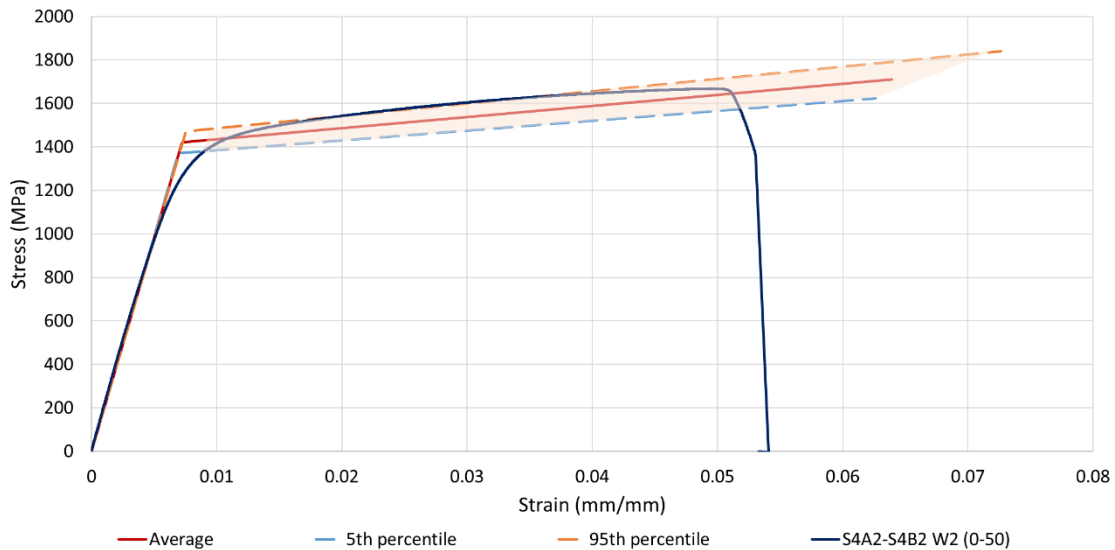


Figure 5-9 Typical tensile test results of uncorroded and corroded S4A2-S4B2 W2(0-50) wires

However, deterioration is observed in the experimental results, showing that corrosion adversely affects key mechanical properties, particularly ultimate strain and ductility. Corroded bars, such as sample S4A2-S4B2 W2 with a 2.29% corrosion level shown in Figure 10, display reduced total displacement and ductility compared to their uncorroded counterparts, with the extent of this reduction generally corresponding to the severity of corrosion.

It must be explicitly acknowledged that the tested wires had already experienced loading and possibly plastic yielding during the initial structural testing, which inherently reduced their remaining available strain capacity and resulted in a lower apparent ductility during subsequent tensile testing. Accounting for this prior plastic strain history fell outside the scope of our analysis and is noted as a limitation of this study, as it likely leads to a conservative underestimation of the wires' true baseline ultimate strain.

#### 5.4.4 Relationship between wire and strand cross-section loss

Figure 5-10 presents the relationship between the strand-unit section loss and the wire-unit section loss of the most severely corroded wire within each strand. The strand-unit section loss is calculated as the sum of the individual wire-unit section losses within the strand, divided by the original cross-sectional area of the strand. A clear positive correlation is observed, indicating that the corrosion-induced section loss of the most corroded wire increases as the overall strand-unit section loss increases. Inspection of the experimental data revealed a curvilinear relationship between strand-unit section loss and wire-unit section loss, indicating that the increase was not proportional. To capture this non-linear trend observed in the scatter plot, a second-order polynomial regression was fitted to the data, yielding the equation:  $y = 0.5223x^2 + 1.6273x$ , with an  $R^2$  value of 0.8368. This polynomial fit indicates a progressively increasing wire-unit section loss as strand-unit section loss increases, suggesting an accelerating deterioration mechanism. Most of the tested strands exhibited relatively low levels of corrosion, with strand-unit section losses primarily concentrated below 3%.

Despite this comprehensive measurement of the strand's overall area loss, which assumes uniform corrosion along the strand, the wire-unit section loss of the most corroded wire still consistently exceeded the calculated strand-unit section loss. This observation highlights the highly localized nature of corrosion within the strands. Even though the average corrosion across all wires might be relatively low (as reflected in the low strand-unit section loss values), individual wires can experience significantly higher localized corrosion. This non-uniform corrosion distribution is likely attributable to variations in local grout and environmental conditions within the wires. While all wires within a tendon are exposed to generally similar harsh external conditions, differences in grouting quality and the presence of voids or cracks can create localized micro-environments with varying levels of chloride exposure, moisture, and oxygen availability, leading to different corrosion conditions along the wires within a single tendon.

The scatter observed in the data, particularly at higher wire-unit section losses, may be related to the increased variability in the degree of pitting corrosion. For instance, the two strands with similar overall strand-unit section losses of around 3% have significantly different maximum wire-unit section loss of their most corroded wire. This is likely because the current analysis assumes a uniform cross-sectional area reduction along the length of each individual wire, which may not

accurately reflect the reality of localized pitting corrosion, where significant material loss can occur in concentrated areas.

This finding reinforces the importance of considering the condition of the most severely affected wires when assessing the potential impact of corrosion on the overall tensile capacity of the strand, as localized damage can disproportionately affect performance. The 'weakest link' principle must remain relevant, even when a full cross-sectional area measurement is available. Indeed, a tendon is only as strong as its weakest point. Even if the overall strand cross-section loss is relatively low, a few wires could experience significant localized section loss, leading to premature wire failure. This failure would then redistribute the load to the remaining wires, potentially initiating a cascade of progressive failures and ultimately leading to complete tendon failure. Therefore, a comprehensive assessment of tendon conditions may require considering not only the maximum wire-unit section loss but also the section loss distribution across all remaining wires within the strand.

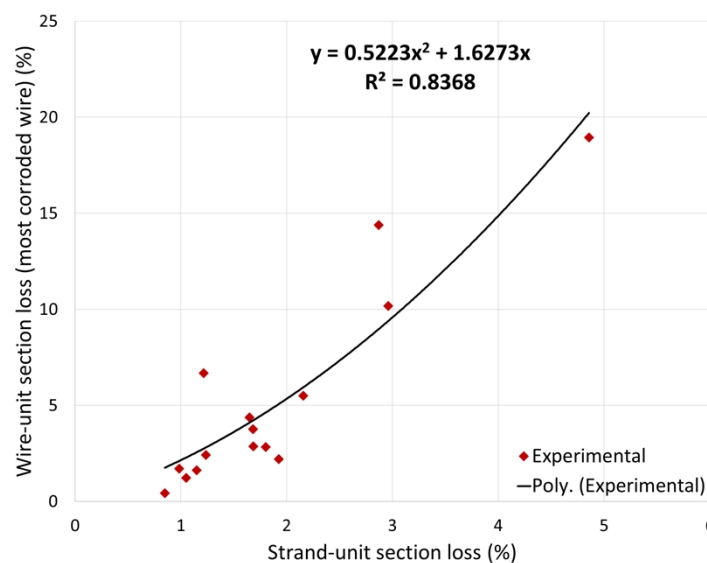


Figure 5-10 Relation between strand-unit section loss and wire-unit section loss of the most corroded wire

#### 5.4.5 Influence of tendon location on corrosion severity

The slabs tested at the University of Ottawa Structures Lab appeared undamaged and did not undergo the repair measures implemented elsewhere on the bridge throughout its service life. This condition helps explain why the overall corrosion levels in these specimens remained relatively low, as shown in Figure 5-7, with significant deterioration occurring only at specific locations along

certain tendons where moisture accumulation, chloride exposure, or grout voids created localized corrosive environments. To further understand the origin of these localized corrosion zones, a detailed examination of the strand cross sections was conducted.

Inspection of the strand cross sections revealed that the distribution of corrosion along the tendons was strongly influenced by the slab geometry and long-term exposure conditions. In particular, strand S2A2-S2B2 exhibited a localized grout void between several wires, as shown in Figure 5-11, and this location corresponded to one of the highest cross-sectional losses of approximately 15% in one of the strand wires at a distance ranging from 1,000 mm-1,500 mm from edge A. As shown in Figure 1-6, the bridge deck slopes downward from side A toward side B, and slab 2, which is reinforced with S2A2-S2B2, is positioned along the outer edge of the superstructure. Because the deck had no functioning drainage system for over 30 years, salt-laden runoff accumulated at the low points of the deck, specifically near the B-edges of slabs 4, 2, and 5. The combination of persistent moisture exposure at these low-slope regions and the presence of grout voids, which reduced the level of strand protection, created highly favorable conditions for active corrosion. The elevated corrosion observed in Slab 2, therefore, can be attributed to both its position as an edge slab within the sloping deck profile and the existence of localized voids that permitted direct ingress of chlorides to the prestressing wires.



Figure 5-11 Cross section of S2A2-S2B2 at side A

A similar pattern was observed for tendon S5A3-S5B3. One of its wires exhibited a localized cross-sectional loss of approximately 10% within 500 mm from edge B, indicating significant corrosion activity in this region. As shown in Figure 5-12, the cross-sectional inspection revealed distinct grout voids surrounding portions of the wires. In addition, in certain locations, the wires appear to



be in direct contact with the inner surface of the duct rather than being fully encapsulated by grout. This lack of grout coverage eliminates the protective alkaline environment typically provided by a well-grouted tendon and allows moisture and chlorides to penetrate the duct and reach the steel surface more readily. Consequently, the combination of the tendon's proximity to the deck's low-slope edge at Side B, where water accumulation was most severe, and the presence of voids that removed the intended corrosion protection created a localized environment highly conducive to corrosion.



Figure 5-12 Cross section of S5A3-S5B3 at side B

For tendon S7A2–S7B2, the most severe corrosion among all examined wires was measured. One of its wires exhibited a cross-sectional loss of approximately 19%, located between 1,500 mm and 2,000 mm from edge A. This tendon belongs to slab 7, which is positioned near the outer boundary of the deck and extends towards an edge girder. Given the deck's transverse slope toward the exterior and the absence of drainage over the structure's first 30-year service life, salt-laden runoff consistently migrated toward and accumulated at these edge regions. The persistent exposure to moisture and chlorides at the slab's low-point location would have contributed to a highly aggressive environment at the tendon elevation. The high corrosion level observed in S7A2–S7B2 is therefore consistent with its location vulnerability as an edge-adjacent slab directly connected to an exterior girder, where water ponding and chloride deposition were most pronounced.

## 5.5 Statistical analysis

The statistical analysis aims to evaluate the relationship between corrosion severity, expressed as mass loss, and the corresponding reduction in ultimate strain. Given that the experimental datasets

available in the literature differ in loading rate, gauge length, and baseline mechanical properties of the uncorroded specimens, the analysis focuses on identifying whether a corrosion threshold can be detected both within the present dataset and when incorporating results from other studies (Franceschini et al., 2022; Jeon et al., 2020; J. Li et al., 2023). The objective is to determine whether, beyond a certain level of mass loss, the reduction in ultimate strain stabilizes or transitions into a different degradation stage, indicating a consistent response despite variations in testing conditions.

To enable direct comparison across studies with different reference strains, all results are expressed as the ratio of the measured ultimate strain to the ultimate strain of the corresponding uncorroded specimen. This normalization allows the ductility reduction to be evaluated on a common basis and supports the identification of potential cross-study thresholds in corrosion–strain behaviour. The ultimate strain normalized by the uncorroded reference ultimate strain is given by Equation 5-4:

$$\varepsilon_{u,norm} = \frac{\varepsilon_u}{\varepsilon_{u,\mu}} \quad \text{Equation 5-4}$$

where  $\varepsilon_{u,norm}$  is the normalized ultimate strain,  $\varepsilon_u$  is the measured ultimate strain, and  $\varepsilon_{u,\mu}$  is the mean ultimate strain from the uncorroded reference.

### 5.5.1 Analysis of experimental data

A single integrated analysis was performed using grouped combinations of mass-loss intervals and gauge lengths. Because corrosion-induced mass loss follows a reduction process, the raw mass-loss values exhibit skewed or one tail behaviour; therefore, mass loss was transformed into a logarithmic scale to approximate normal distribution.

For each of the six gauge-length categories, specimens were then divided into five corresponding mass-loss groups on the log scale, as shown in Table 5-5, allowing comparisons among specimens sharing the same length measurement. This structure ensures that the statistical evaluation reflects both corrosion severity and gauge-length effects, enabling deformation trends to be assessed consistently within each gauge-level group.

Table 5-5 Mass loss groups for statistical analysis<sup>s</sup>

| Group                                   | Mass Loss            | Data Points |
|-----------------------------------------|----------------------|-------------|
|                                         | [ML] %               | [n]         |
| Gauge [L = 430] mm                      |                      |             |
| <b>G0</b>                               | Uncorroded Reference | 6           |
| Gauge range [L < 200] mm                |                      |             |
| <b>G1</b>                               | < 0.01               | 0           |
| <b>G2</b>                               | 0.01-0.10            | 2           |
| <b>G3</b>                               | 0.10-1.0             | 8           |
| <b>G4</b>                               | 1.0-10.0             | 0           |
| <b>G5</b>                               | 10.0-100.0           | 0           |
| Gauge range [200 ≤ L < 300] mm          |                      |             |
| <b>G6</b>                               | < 0.01               | 2           |
| <b>G7</b>                               | 0.01-0.10            | 71          |
| <b>G8</b>                               | 0.10-1.0             | 208         |
| <b>G9</b>                               | 1.0-10.0             | 19          |
| <b>G10</b>                              | 10.0-100.0           | 2           |
| Gauge range [300 ≤ L < 400] mm          |                      |             |
| <b>G11</b>                              | < 0.01               | 7           |
| <b>G12</b>                              | 0.01-0.10            | 87          |
| <b>G13</b>                              | 0.10-1.0             | 215         |
| <b>G14</b>                              | 1.0-10.0             | 12          |
| <b>G15</b>                              | 10.0-100.0           | 1           |
| Gauge range [400 ≤ L < 500] mm          |                      |             |
| <b>G16</b>                              | < 0.01               | 2           |
| <b>G17</b>                              | 0.01-0.10            | 70          |
| <b>G18</b>                              | 0.10-1.0             | 127         |
| <b>G19</b>                              | 1.0-10.0             | 16          |
| <b>G20</b>                              | 10.0-100.0           | 0           |
| Gauge range [500 ≤ L < 600] mm          |                      |             |
| <b>G21</b>                              | < 0.01               | 3           |
| <b>G22</b>                              | 0.01-0.10            | 42          |
| <b>G23</b>                              | 0.10-1.0             | 49          |
| <b>G24</b>                              | 1.0-10.0             | 5           |
| <b>G25</b>                              | 10.0-100.0           | 0           |
| Gauge range [L > 600] mm                |                      |             |
| <b>G26</b>                              | < 0.01               | 0           |
| <b>G27</b>                              | 0.01-0.10            | 0           |
| <b>G28</b>                              | 0.10-1.0             | 1           |
| <b>G29</b>                              | 1.0-10.0             | 1           |
| <b>G30</b>                              | 10.0-100.0           | 0           |
| <b>Total (without considering G0) =</b> |                      | <b>950</b>  |

<sup>8</sup>The difference between the 950 valid samples and the total of 975 is due to samples that experienced grip failure or were lost during the experimental process.

The statistical test evaluated whether ductility becomes significantly different beyond a defined mass-loss threshold. Welch's t-test was selected because it accommodates unequal variances between groups and remains appropriate under the assumption of normality following the logarithmic transformation of mass-loss data (Navidi, 2011). In contrast, the ANOVA analysis requires the additional assumption of equal variances across the test groups. The comparison was performed between the reference group (0.01–0.10% mass loss) and the test group (1.0–10.0% mass loss) for three gauge length ranges (2.2, 2.3 and 2.4 as listed in Table 5-5). The null hypothesis ( $H_0$ ) states that the mean ultimate strain of the test group is not statistically different from that of the reference group, while the alternative hypothesis ( $H_1$ ) states that the mean deformation of the test group is statistically different.

These two groups were selected because they contain sufficient data points to support a valid t-test, and the difference in sample size between them is smaller than that observed when including the intermediate 0.1–1% mass-loss group. This selection tries to minimize imbalance between groups and reduces the influence of sparsely populated intervals, ensuring a more reliable statistical comparison. The statistical test for Welch's t-statistic is given by Equation 5-5:

$$t = \frac{\bar{X}_1 - \bar{X}_2}{\sqrt{\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2}}} \quad \text{Equation 5-5}$$

where  $\bar{X}_1, \bar{X}_2$  are the mass loss means of groups 1 and 2, respectively,  $s_1^2, s_2^2$  are the sample variances, and  $n_1, n_2$  are the sample sizes.

### 5.5.2 Comparison with other studies

The second part of the analysis incorporates results from three external studies (Franceschini et al., 2022; Jeon et al., 2020; J. Li et al., 2023) to assess whether a similar threshold is observed across different experimental conditions. Because these studies differ in loading rate, gauge length, corrosion exposure method, and baseline mechanical properties, the comparison focuses on whether they exhibit a comparable transition region in the relationship between mass loss and strain

degradation. The objective is to determine whether this threshold converges across datasets despite methodological differences. While the external studies compared in this work explicitly investigate the specific corrosion morphology and its localized effects on tendon geometry, such microscopic morphological analysis was not conducted in this study. Instead, this study relies on idealized uniform cross-sectional area reductions. Furthermore, it should be noted that these external studies typically evaluate a helical wire configuration, whereas this study focuses strictly on a parallel wire configuration. These geometric and structural differences should be considered when directly comparing the performance and localized degradation results across the literature.

This cross-study evaluation is used to identify patterns that persist beyond individual testing conditions, thereby supporting or challenging the existence of a general corrosion–ductility threshold. Establishing such convergence enhances the applicability of the modelling approaches discussed in Section 5.2.3 and contributes to the development of generalized predictive tools for evaluating the capacity of corroded prestressing steel. As in the previous analysis, the Welch’s t-test was selected to conduct the statistical comparison.

#### **5.5.2.1 Statistical analysis of experimental results**

This subsection presents the results of the statistical evaluation performed on the grouped mass-loss and gauge-length data. The analysis summarizes the descriptive statistic characteristics of each group, identifies which groups meet the minimum data requirements for statistical testing, and reports the results of the Welch’s t-test used to assess whether corrosion beyond a certain mass loss threshold produces a statistically significant reduction in normalized ultimate strain.

Figure 5-13 shows the histograms for all mass-loss and gauge-length groups, illustrating the distribution and spread of the normalized ultimate strain values. Figure 5-14 (a) presents the box-and-whisker plots for all groups with at least five data points, ensuring sufficient representation for meaningful visualization. Figure 5-14 (b) displays the box-and-whisker plots for the subset of groups selected for inferential testing, those with sample sizes adequate for a Welch’s t-test, allowing for a direct comparison of mean strain responses across the defined mass-loss intervals.

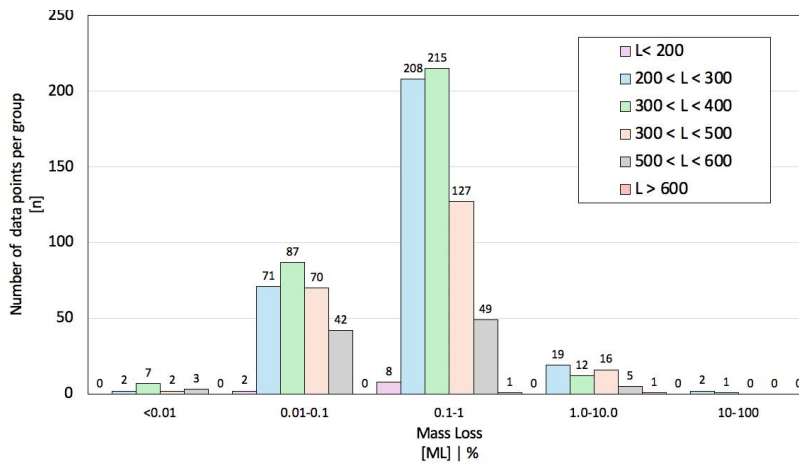


Figure 5-13 Histogram for groups G1-G30

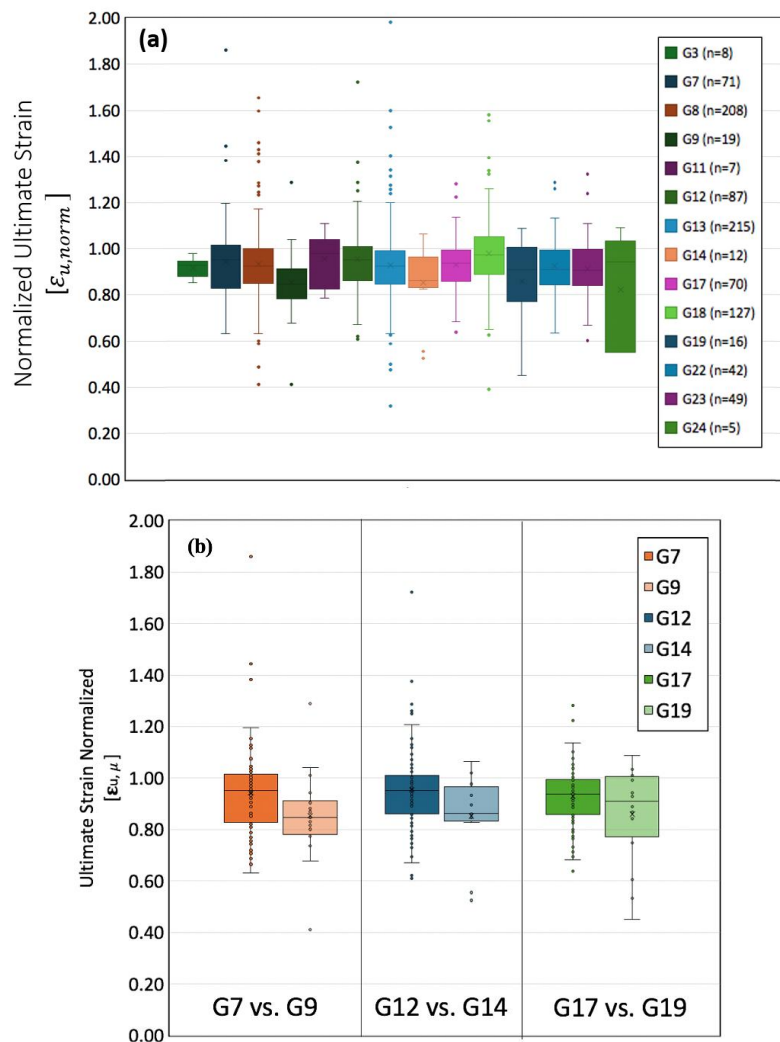


Figure 5-14 (a) Box-and-whisker plots for all groups containing at least five data points, and (b) box-and-whisker plots for the subset of groups meeting the criteria for Welch's t-test

Based on Welch's t-test results from Table 5-6, the comparison between G7 vs. G9 and G12 vs. G14 shows a statistically significant difference in their mean values. Because the p-value is below the 0.05 significance level, the null hypothesis ( $H_0$ ), which states that both groups have equal means, is rejected. In contrast, the comparison involving G17 and G19 leads to a failure to reject  $H_0$ , as the corresponding p-value exceeds 0.05. However, this outcome is driven not by similarity in performance, but by the characteristics of the dataset itself. Upon inspection, the strain data for G19 corresponds to specimens exhibiting mass losses close to 1.5%, resulting in a strong clustering of values near the lower bound of the deterioration range. Instead of spanning the expected distribution across 1–10% mass loss, the data are compressed into a narrow region with very limited variability. This lack of dispersion shown in Figure 5-15 reduces the statistical power of the test, making it unlikely to detect differences between groups even when such differences may exist physically.

Table 5-6 Summary of Welch's t-test results

| <b>Welch's Statistical Test Results</b> | <b>G7</b> | <b>G9</b> | <b>G12</b> | <b>G14</b> | <b>G17</b> | <b>G19</b> |
|-----------------------------------------|-----------|-----------|------------|------------|------------|------------|
| <b>Mean</b>                             | 0.9433    | 0.8537    | 0.9549     | 0.8521     | 0.9299     | 0.8593     |
| <b>Variance</b>                         | 0.0349    | 0.0290    | 0.0263     | 0.0268     | 0.0145     | 0.0345     |
| <b>Observations (n)</b>                 | 710       | 19        | 87         | 12         | 70         | 16         |
| <b>Degrees of Freedom</b>               | 31        |           | 14         |            | 18         |            |
| <b>t-statistic</b>                      | 1.9965    |           | 2.0443     |            | 1.4538     |            |
| <b>p-value (one tailed)</b>             | 0.0274    |           | 0.0301     |            | 0.0816     |            |
| <b>Critical t value (one tailed)</b>    | 1.6955    |           | 1.7613     |            | 1.7341     |            |
| <b>p-value (two tailed)</b>             | 0.0547    |           | 0.0602     |            | 0.1632     |            |
| <b>Critical t value (two tailed)</b>    | 2.0395    |           | 2.1448     |            | 2.1009     |            |

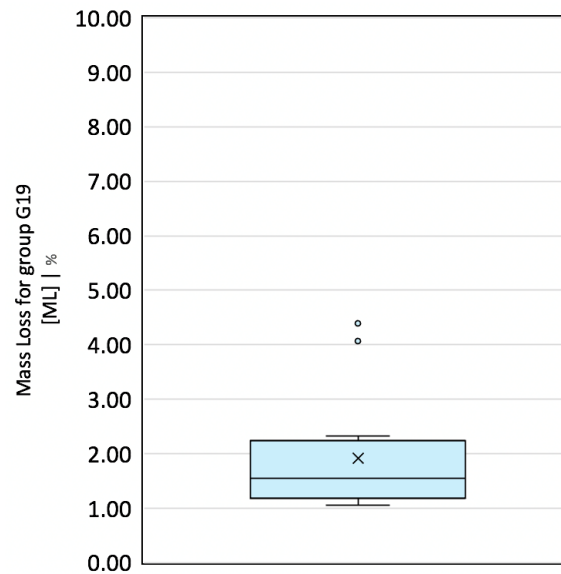


Figure 5-15 Box-and-whisker plots G19 mass loss

### 5.5.3 Comparison of results

Three variables are considered: bar length, mass loss, and normalized strain reduction. Due to data scatter, groupings were defined to enable statistical comparison (Table 5-3). Thresholds of 0.5%, 1%, 1.5%, and 2.5% mass loss were evaluated but did not show differences in mean normalized strain reduction. A threshold of 3% mass loss was the lowest level at which a statistically significant separation was observed. For mass losses greater than 3%, the data indicate reduced strain capacity; however, the number of observations is insufficient for statistical testing. Therefore, 3% mass loss is adopted as the minimum threshold for comparison across datasets.

Figure 5-16 presents a box-and-whisker comparison between the experimental results and those reported by two additional authors (Franceschini et al., 2022; C.-H. Jeon et al., 2020). The datasets were divided into two groups, mass loss  $< 3\%$  and  $\geq 3\%$ , to evaluate the influence of loading rate, gauge length, corrosion exposure conditions, and baseline mechanical properties on the threshold in ultimate strain reduction. Across all datasets, the distributions show a statistical separation between the two mass-loss groups, indicating that the transition in ductility reduction occurs independently of these factors. Table 5-7 summarizes the statistical results. Welch's t-test was applied to account for unequal variances and sample sizes. For each study, the mean ultimate strain of the  $\geq 3\%$  mass loss group differs from that of the reference group, supporting the presence of a common threshold in corrosion-related ductility degradation.



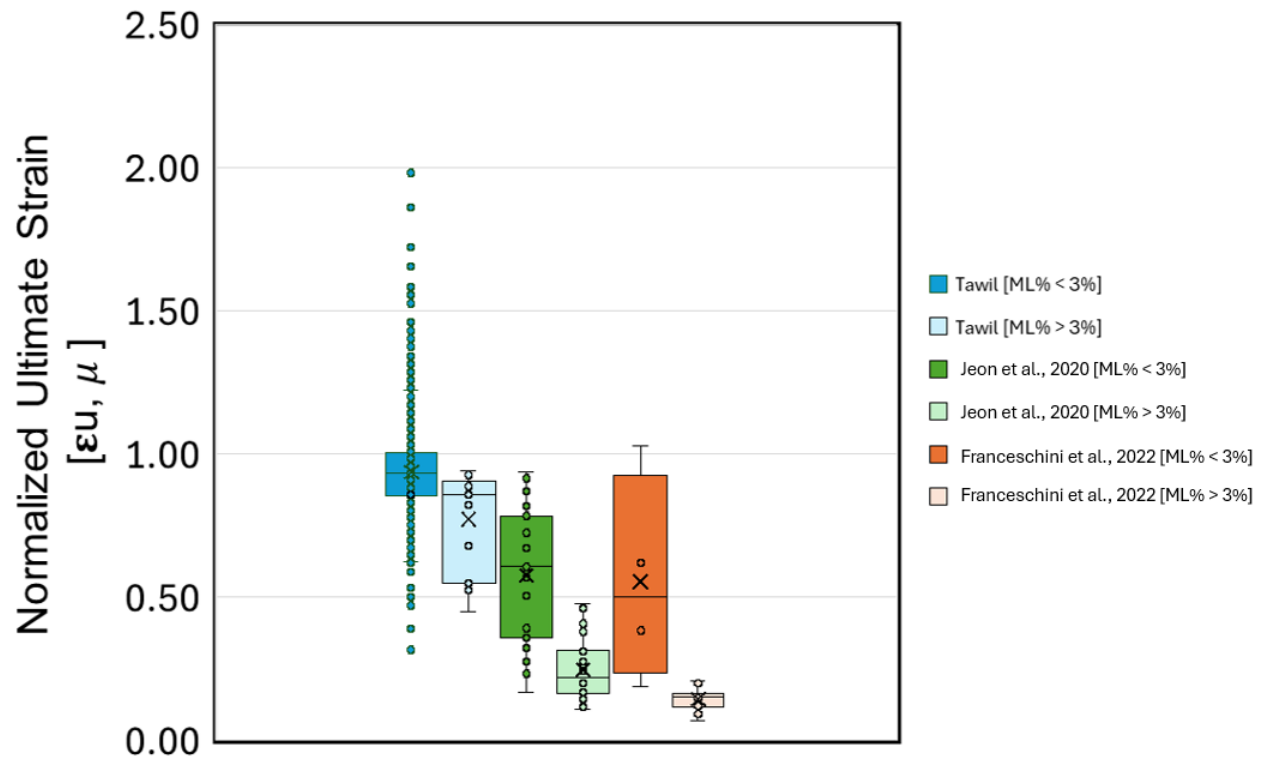


Figure 5-16 Box-and-whisker comparison between the experimental results and results from previous studies for two groups: mass loss < 3% and  $\geq 3\%$

Table 5-7 Welch's Test result for experimental data and study by Jeon et al. (2020)<sup>§</sup>

| Welch's Statistical Test Results | Experimental Results |        | Jeon et al. (2020) |        |
|----------------------------------|----------------------|--------|--------------------|--------|
| Mean                             | 0.9356               | 0.7725 | 0.5771             | 0.2482 |
| Variance                         | 0.0270               | 0.0303 | 0.0538             | 0.0120 |
| Observations (n)                 | 935                  | 15     | 27                 | 38     |
| Degrees of Freedom               | 14                   |        | 34                 |        |
| t-statistic                      | 3.6219               |        | 6.8489             |        |
| p-value (one tailed)             | 0.0014               |        | 0                  |        |
| Critical t value (one tailed)    | 1.7613               |        | 1.6909             |        |
| p-value (two tailed)             | 0.0018               |        | 0                  |        |
| Critical t value (two tailed)    | 2.1448               |        | 2.0322             |        |

<sup>§</sup>The results from Franceschini et al. (2022) were excluded from the statistical comparison because one of the groups did not have enough data points to meet the minimum requirements for performing the test.

The statistical test revealed a significant reduction in the experimental results and study by Jeon (Jeon et al., 2020). For the experimental results, the group with a mass loss of less than 3% exhibited a mean value of 0.9356 with a variance of 0.0270, whereas the group with a mass loss of more or equal 3% showed a lower mean of 0.7725 with a variance of 0.7725. The difference between group means was statistically significant with a t-value of 1.7613 for one tailed and 2.1448 for two tails. Similarly, the results from Jeon (Jeon et al., 2020) show that the group with a mass loss of less than 3% exhibited a mean value of 0.5771 with a variance of 0.0538, whereas the group with a mass loss equal or greater than 3% showed a lower mean of 0.2482 with a variance of 0.0120. The difference between group means was statistically significant with a t-value of 1.6909 for one tailed and 2.0322 for two tails.

Figure 5-17 provides a representation of all cumulative data points plotted against mass loss for the experimental results reported herein and those reported by other studies (Franceschini et al., 2022; Jeon et al., 2020; J. Li et al., 2023). The combined trend reveals that the increase in mass loss progresses along different stages, where some datasets exhibit a gradual reduction consistent with a low-decay (potential-type) relationship, while others follow a rapid-decay (exponential-type) trend. This combined visualization highlights the convergence of different datasets toward a multi-stage degradation pattern, reinforcing the interpretation that corrosion behaviour does not follow a single uniform regression form.

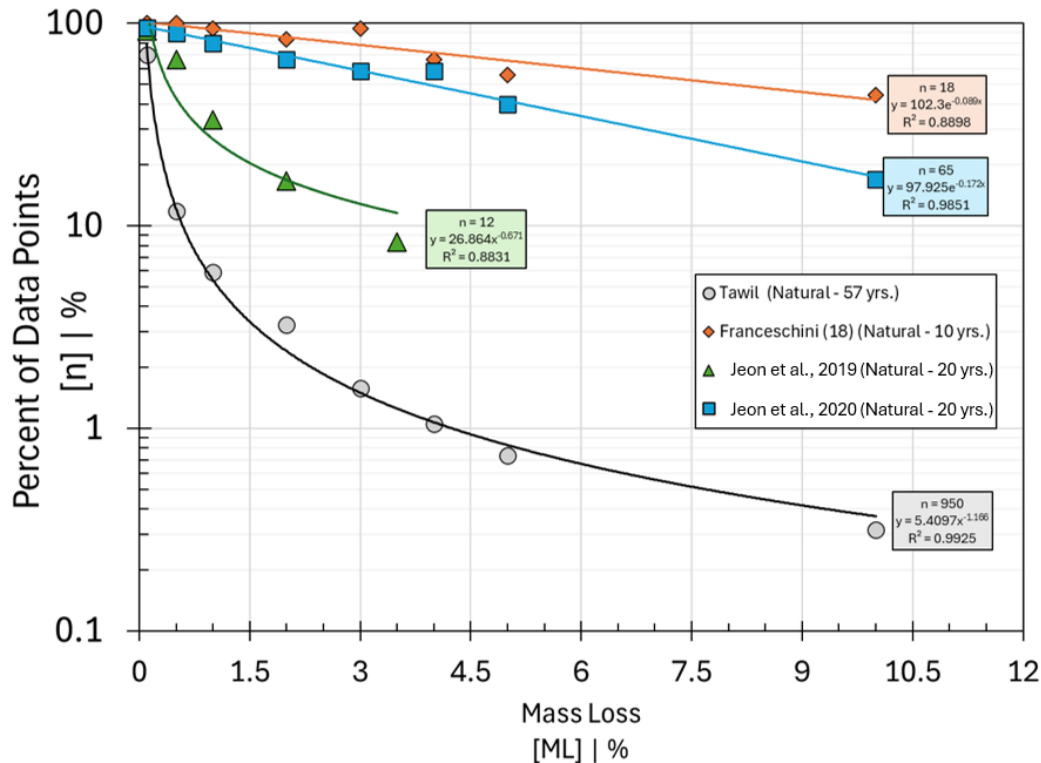


Figure 5-17 Cumulative data points plotted against mass loss for this study's experimental results and references by (Franceschini et al., 2022; Jeon et al., 2020; J. Li et al., 2023)

It is important to note that the discrepancy in the distribution of higher corrosion levels (>3%) between the experimental results and the three external studies (Franceschini et al., 2022; Jeon et al., 2020; J. Li et al., 2023) may be attributed to differences in tendon configuration. The helical seven-wire strands used by other researchers provided a greater number of data points exceeding 3% corrosion (Figure 5-18). In contrast, although the present study includes a larger overall dataset, only approximately 2% of the specimens exhibited corrosion levels above 3%. This difference may be related to the parallel wire configuration adopted in the design of Champlain Bridge deck slabs, highlighted in Figure 5-18, which allows for a greater volume of surrounding grout. As grout serves as the primary protective barrier against corrosion, the increased grout coverage in parallel wire systems may have contributed to the lower incidence of higher corrosion levels.

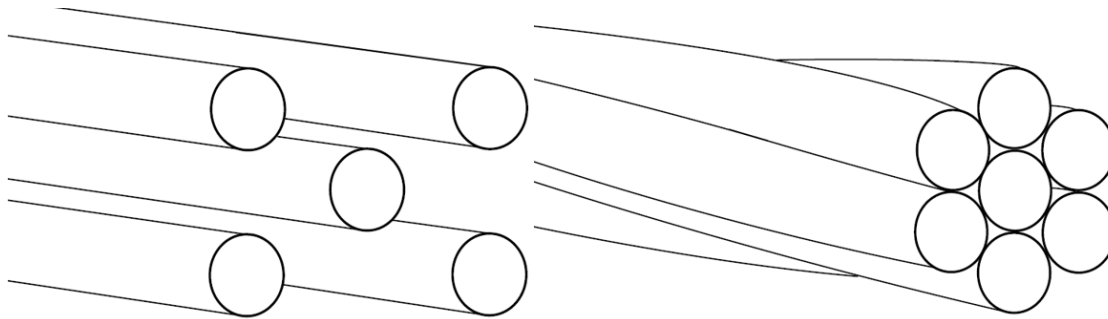


Figure 5-18 3D section profile of parallel wires (left) and helically wound wires (right)

## 5.6 Conclusions

This study investigated the impact of corrosion on the mechanical properties of prestressing steel strands with parallel wire configuration extracted from the Champlain Bridge in Montreal, Canada, after 57 years of service. Through a comprehensive experimental program, the effect of corrosion on ultimate strength and ductility was evaluated. The main conclusions of the study are as follows:

- Corrosion reduces the ultimate strain and ductility of prestressing wires. As expected, corroded prestressing tendons exhibited lower total displacement and ductility compared to uncorroded wires, with the severity of degradation increasing with greater section loss. The effect of corrosion on the elastic modulus and yield strength was observed not to be significant in the current study.
- A strong correlation was observed between strand-unit section loss and the most corroded wire-unit section loss, following a non-linear trend ( $R^2 = 0.8368$ ). This highlights that while overall strand corrosion may be low, individual wires can experience severe localized damage, impacting structural integrity and leading to premature failure and load redistribution. This emphasizes the need for a thorough assessment of corrosion damage, not only in terms of average section loss but also by considering the condition of the most severely affected wires.
- Localized corrosion in the tendons was primarily governed by the combined effects of slab location within the sloped deck, long-term water accumulation at edge regions due to the absence of drainage, and the presence of grout voids that exposed prestressing wires to chlorides. Although the tested slabs were generally undamaged and did not undergo repairs

during service, significant corrosion still developed at specific tendon locations where these unfavorable conditions aligned.

- Statistical analysis revealed a consistent threshold in the reduction of ductility. Welch's t-tests indicated that ultimate strain values become statistically different beyond approximately 1%-10% mass loss. This threshold was identified in mass-loss–gauge-length groupings. The results suggest that, past this corrosion level, ductility enters a distinct degradation stage, reflecting a transition in mechanical response.
- Cross-study comparison confirmed the existence of a corrosion–ductility transition region across different experimental conditions. When results from other studies were incorporated, the box-and-whisker distributions and statistical tests consistently showed significant differences between the  $<3\%$  and  $\geq 3\%$  mass-loss groups.
- The findings of this study can be used to improve the assessment and rehabilitation of existing structures by providing a better understanding of the impact of corrosion on the mechanical properties of prestressing steel. The results can be used to develop more accurate models for predicting the long-term performance of prestressed concrete structures under varying levels of corrosion damage.
- Further research is needed to enhance the predictive capability of mechanical properties in corroded wires by expanding the dataset and refining models for more reliable performance assessments.

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## **Chapter 6: Finite element modelling of corroded prestressed concrete roadway deck slabs recovered from the Champlain Bridge: investigating bond degradation and structural performance**

### **Abstract**

This paper presents a finite element (FE) investigation calibrated against experimental results from full-scale, naturally corroded post-tensioned (PT) concrete deck slabs recovered from the decommissioned Champlain Bridge in Montréal, Canada. The slabs, subjected to 57 years of service in a harsh environment with heavy de-icing salt application, exhibited variable and, in some cases, compromised grout quality, which influenced the tendon-concrete interaction and motivated the study's focus on bond integrity. Four representative slabs, which were nominally identical in terms of geometry but exhibited a range of behaviours and failure modes, were tested to failure under four-point bending. Two-dimensional nonlinear FE models were developed using VecTor2 to replicate the observed structural behaviour, while focusing on investigating the tendon-concrete bond integrity. The models incorporated corrosion damage by reducing the wires' cross-sectional area and selectively applying empirical models for bond degradation and cover cracking to investigate the governing failure mechanisms. For the three slabs that failed in a ductile flexural mode, the models accurately predicted the ultimate capacity by incorporating an imperfect bond within the shear span, with predicted-to-experimental load ratios consistently near 0.95. In contrast, for the slab that failed in a brittle shear mode, a different calibration strategy was necessary, involving a 73% and 14% reduction in the concrete's elastic modulus and tensile strength, respectively, as well as modelling the tendon as unbonded in the flexural zone. While this model successfully captured the correct failure mode, it under-predicted ultimate deformation by 26%. The strong agreement between experimental and numerical crack patterns further validated the models' ability to capture the underlying physical failure mechanisms. This study demonstrates that tailored calibration and modelling strategies are required to identify and characterize different failure modes, including the localization of bond loss, and provides a validated framework for assessing the residual capacity and tendon-concrete bond condition in aging, naturally corroded PT concrete infrastructure.

## 6.1 Introduction

The long-term durability of structural components is a cornerstone of modern civil engineering, ensuring the safety, serviceability, and economic viability of critical infrastructure. Prestressed concrete, in particular, has been widely adopted for its superior strength, efficiency, and inherent durability, making it a preferred material for bridge construction (fib (International Federation for Structural Concrete), 2012). The compressive stresses induced by prestressing can delay or prevent cracking under service loads, thereby limiting the ingress of aggressive agents through these cracks and protecting the reinforcing steel from corrosion. However, the mechanisms governing durability differ between pre-tensioned and post-tensioned (PT) systems. In pre-tensioned elements, the steel tendons are tensioned before casting the concrete and remain in direct bond with the surrounding concrete after release. This continuous bond provides effective protection against corrosion, as the dense concrete cover limits the ingress of chlorides and moisture. In contrast, PT systems use tendons housed within ducts that are subsequently grouted after stressing. The integrity of this grout is critical for corrosion protection, since any voids, segregation, or poor-quality grout can allow the ingress of chlorides and moisture along the duct, particularly near anchorage zones, leading to localized corrosion of the prestressing steel (ACI Committee 222, 2014).

Corrosion of prestressing steel is a complex electrochemical process with severe consequences, typically initiated by the ingress of chloride ions from de-icing salts or marine environments (Broomfield, 2007). Unlike the more uniform corrosion often seen in conventional reinforcing bars, the corrosion of high-strength prestressing steel can be localized and aggressive, leading to pitting and a significant reduction in the cross-sectional area of the strands (Schupack & Suarez, 1982). This is due to the highly stressed and fine microstructure of prestressing steel, making it more susceptible to stress corrosion cracking and hydrogen embrittlement, and localized exposure conditions such as incomplete grouting or trapped moisture that promote concentrated attack at a specific location. This loss of steel area directly impairs the load-carrying capacity of the member. Furthermore, the accumulation of corrosion products, which may occupy a larger volume than the original steel, generates expansive pressures at the steel-concrete interface. These pressures can cause cracking and spalling of the concrete cover, further accelerating the deterioration process.

A critical, yet often overlooked, consequence of this process is the degradation of the bond between the prestressing strands and the surrounding concrete or grout (Coronelli & Gambarova, 2004).

This bond is essential for the composite action of the member and for the effective transfer of forces from the steel to the concrete. A reduction in bond strength can lead to premature anchorage failure, increased deflections, and a significant reduction in the overall structural performance, potentially resulting in a sudden and less ductile failure (Cairns & Abdullah, 1996). To accurately predict the residual capacity and remaining service life of corroded prestressed concrete structures, it is imperative to understand the complex interplay between corrosion, bond degradation, and structural performance. While numerous studies have investigated these phenomena using accelerated corrosion techniques in laboratory settings, there is a growing recognition that such methods may not fully replicate the characteristics of natural corrosion. Therefore, studying full-scale beams and slabs recovered from decommissioned structures is essential, as it provides invaluable data reflecting the long-term effects of real-world exposure conditions.

Several notable studies have paved the way in this area. Pape and Melchers (Pape & Melchers, 2013) tested 45-year-old corroded PT concrete beams, which were shear reinforced, from a bridge in an aggressive marine environment. A key finding was that all tested beams, including one that appeared superficially sound, exhibited progressive and premature failure of prestressing wires and failed to reach their expected design capacity. This highlighted the unreliability of visual inspection for assessing the structural integrity of such members. Expanding on this, Belletti et al. (Belletti et al., 2020) conducted an extensive experimental campaign on full-scale pretensioned concrete beams without transverse reinforcement, recovered from a thermal power plant after 10 years of exposure to wetting cycles with marine water. It was found that the beneficial effect of prestressing on the shear capacity was significantly diminished by corrosion. This led to an unfavorable transition from the expected ductile flexural failure to a more dangerous, brittle shear failure, which occurred even in regions not typically critical for shear. Similarly, destructive tests by Rinaldi et al. (Rinaldi et al., 2010) on 50-year-old pretensioned concrete girders with shear reinforcement revealed substantial reductions in peak load, reaching up to 55% for moderate corrosion and 65% for severe corrosion. However, the impact on ductility was inconsistent, ranging from moderate decreases to no change at all.

More recently, Murray et al. (Murray et al., 2019) tested 45-year-old, naturally corroded pretensioned AASHTO girders with shear reinforcement and found that their ultimate shear capacities generally exceeded the predictions of modern design codes (AASHTO LRFD Bridge

Design Specifications (American Association of State Highway and Transportation Officials (AASHTO), 2015), even when strand slip due to corrosion-induced bond loss was observed. A key finding, however, was that while the ultimate capacity was not compromised by the observed corrosion levels, the deterioration created significant serviceability issues. Corrosion-induced cracking at the girder ends caused bearing issues, such as concrete cracking and spalling under service-level loads, indicating that even moderate corrosion can accelerate durability issues long before the ultimate strength is affected.

Beyond the overall structural response, some studies have also focused on the degradation of the bond between the prestressing strand and the surrounding concrete. The integrity of this bond is paramount for the composite action of prestressed concrete.

Li & Yuan (2013) conducted pull-out tests on specimens containing seven-wire strands subjected to 17 months of accelerated corrosion. A notable finding was that, within the tested levels of corrosion (quantified by concrete crack widths that reached 0.60 mm) the ultimate bond strength did not degenerate and often exceeded that of the uncorroded control specimens. The authors attributed this to a change in the governing mechanism: for uncorroded strands, a "screwing effect" along the helical wires dominated, but for corroded strands, the roughened surface and damaged ribs enhanced the mechanical "gear action," leading to higher resistance before failure. The study concluded that for pre-tensioned concrete structures, the degradation of the strand's tensile strength is a more critical factor for structural failure than the degradation of bond strength, which remains robust at low-to-moderate corrosion levels.

Further investigating the unique behaviour of prestressing strands, Robuschi et al. (2021) conducted an extensive series of pullout tests on 156 plain reinforcing bar specimens sourced from an 81-year-old decommissioned bridge. Their investigation revealed a complex relationship between corrosion and bond that was highly dependent on the bar's casting position. For top-cast bars, which are typically surrounded by less dense concrete, low levels of corrosion led to an increase in peak bond stress. This was attributed to the confinement effect of corrosion products filling voids before cracking the concrete cover. Conversely, for bottom-cast bars in denser concrete, the bond capacity showed a consistent decrease with increasing corrosion, as splitting cracks formed at much lower corrosion levels. This work critically demonstrated that the effect of

corrosion on bond is not monotonic and is significantly influenced by local concrete properties and the resulting damage mechanisms, such as cracking and spalling.

The shear behaviour of corroded PT prestressed concrete members, particularly those with grouting defects, was experimentally investigated by Wang, Hu, et al. (2020). Their study involved testing eight PT beams with shear reinforcement, which were divided into groups with full and insufficient grouting. Beams in each group were subjected to varying levels of accelerated corrosion before being tested to failure under four-point loading. The results demonstrated that strand corrosion significantly compromised shear performance by accelerating the formation and propagation of diagonal cracks and degrading both the post-cracking stiffness and the ultimate shear strength. For instance, in a fully grouted beam, a corrosion loss of 31.7% resulted in a 15.44% reduction in shear strength. Insufficient grouting was found to further aggravate the propagation of diagonal cracks and the degradation of post-cracking stiffness. A critical finding was the influence on failure modes; while fully grouted beams maintained a shear-compression failure mode up to 31.7% corrosion loss, beams with insufficient grouting exhibited a dangerous transition from shear-compression failure to a sudden rupture of corroded wires when corrosion loss exceeded 39.6%.

The complexity of these interacting phenomena has spurred the development of numerical modelling techniques to simulate corrosion-induced damage. A foundational study in this area was presented by Coronelli & Gambarova (2004), who developed a nonlinear finite element (FE) procedure to assess the performance of corroded reinforced concrete beams. Their approach modeled the effects of corrosion by systematically degrading the properties of each structural component. This included reducing the cross-sectional area of steel elements to account for corrosion and pitting, modifying the concrete's constitutive behaviour to reflect cracking and reduced compressive strength from rust expansion, and, critically, incorporating a specific model for the deterioration of the bond-slip relationship at the steel-concrete interface. The validated model successfully captured complex behaviours, such as the transition from ductile, tension-controlled failures to more brittle, compression-triggered failures as corrosion progressed. The study underscored that accurately predicting the residual ductility of a corroded member is of paramount importance and necessitates the inclusion of a robust bond-deterioration model.

A more recent and comprehensive numerical investigation by Losanno et al. (2024) explored the flexural response of PT concrete bridge girders. The study developed 3D FE models, first

calibrating them against experimental data on uncorroded beams with various grouting conditions (bonded, unbonded, and partially bonded). The validated models were then extended to simulate three levels of corrosion severity by implementing a degraded stress-strain relationship for the prestressing tendons. The study revealed that the interaction between grouting defects and corrosion is critical, with local defects reducing the load-carrying capacity more than global defects due to localized strain concentrations. Furthermore, the simulations showed that corrosion could induce a transition from a ductile to a brittle flexural failure, sometimes causing the premature rupture of even the uncorroded tendons due to stress redistribution.

These studies collectively underscore a critical point: corrosion in prestressed concrete is a multi-faceted damage mechanism that degrades material properties, compromises bond, reduces load capacity, and alters failure modes from ductile to brittle. While experimental tests on naturally corroded members have been invaluable, they are expensive and resource-intensive. Conversely, while numerical models offer a powerful predictive tool, they often rely on simplified assumptions or data from accelerated corrosion tests, which may not fully capture the intricacies of long-term, natural deterioration. A significant gap remains in validating high-fidelity numerical models against data from full-scale, naturally corroded structural elements that have been in service in harsh environments for decades.

Building upon this body of work, this paper presents a FE investigation calibrated against experimental results from naturally corroded post-tensioned concrete deck slabs recovered from the original Champlain Bridge. Details of the full-scale experimental testing of these slab specimens under four-point bending, as well as the damage characterization of the corroded strands, are presented in Chapter 3. The experimental results highlighted highly inconsistent grout quality among the recovered slabs, leading to a considerable variation in observed structural behaviour. This study aims to numerically replicate the experimentally observed responses using FE models. By accurately simulating the load-deflection behaviour and failure modes observed in the lab, the validated models are then used as a tool to perform a detailed investigation into the mechanisms that are difficult to measure directly, specifically the effects of corrosion damage and the integrity of the bond at the strand-grout interface. This approach allows for a comprehensive analysis of the bond-slip relationship and its influence on the ultimate capacity of the deteriorated slabs, providing a robust methodology for assessing aging prestressed concrete infrastructure.

## 6.2 Experimental program and material characterization

### 6.2.1 Description of recovered Champlain Bridge slabs

The specimens tested in this program were full-scale, naturally corroded PT concrete deck slabs recovered from the original Champlain Bridge, the history and service conditions of which are detailed in Section 3. The bridge's South approach span utilized a deck system composed of seven PT concrete I-girders with six cast-in-place transverse post-tensioned infill slabs positioned between them (Figure 6-2). Continuity across the slabs and the tops of the girders was achieved using straight transverse tendons embedded within the deck, enabling effective load transfer between components. However, this arrangement left the tendons vulnerable to the intrusion of chloride-contaminated moisture through the slab joints. Prolonged exposure to these conditions gradually deteriorated the tendons, ultimately diminishing their effectiveness and reducing the overall capacity of the infill slab system.

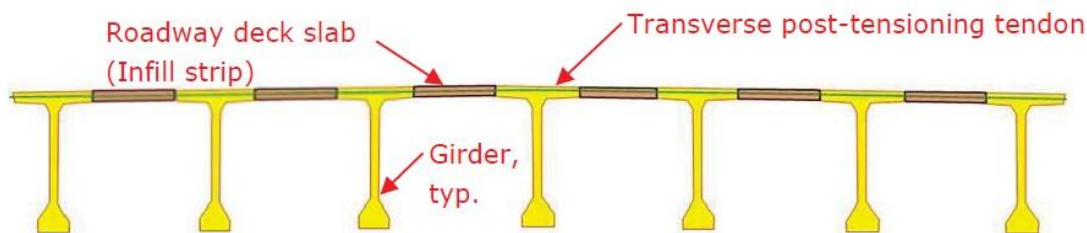


Figure 6-1 Typical deck cross-section at Brossard approach span (Carlin, 2017)

The as-built infill slabs were 200 to 215 mm deep, spanning 1,950 mm between girders, and were topped with a 50 mm asphalt layer. As detailed in Figure 6-2, the primary reinforcement consisted of grouted PT tendons containing twelve 7-mm diameter wires within a metal duct, spaced at an average of 1,000 mm. Secondary mild deformed steel reinforcement for temperature and crack control was also present in the non-load carrying direction, consisting of 10M bars at 200 mm on center in the top layer and 15M bars at an average spacing of 150 mm in the bottom layer.

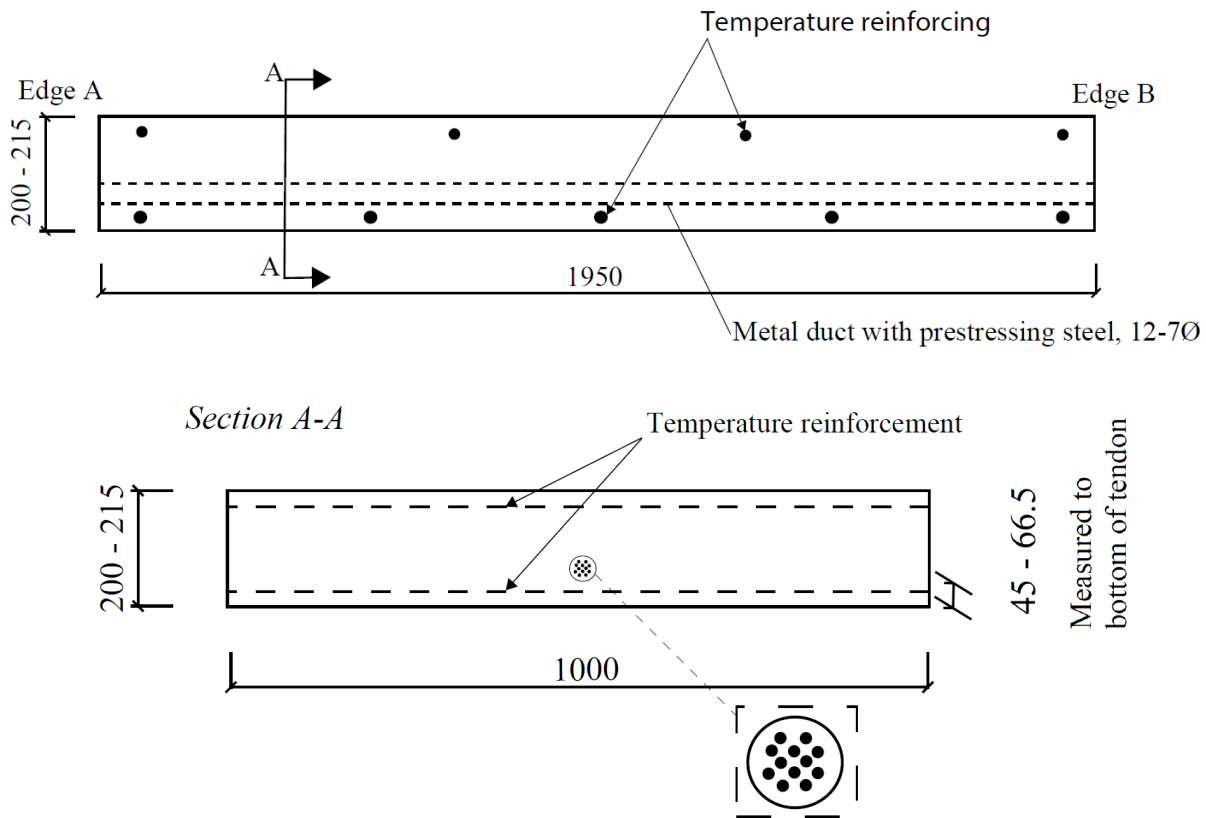


Figure 6-2 Slab geometry and reinforcement details (all dimensions are in mm)

For the structural testing phase, the recovered slabs were cut into 1,000 mm wide specimens, each containing a single, centered PT tendon spanning the full 1,950 mm length. This was done to isolate the behaviour of individual tendons and simulate the one-way spanning action of the deck system under laboratory conditions. A sequential labeling system was adopted to identify each test specimen, where, for example, 'S1A2' refers to the test specimen containing tendon 2 from the original Slab 1, cut from the side corresponding to Edge A. Although each slab specimen contained two tendons available for testing, some extracted slabs contained up to four embedded tendons. In those cases, tendons 1 and 4 were positioned very close to the cut edges of the extracted slab and could not be removed or tested. Therefore, only tendons 2 and 3 were tested, which is why the tendon numbering extends beyond a simple two-tendon system. A comprehensive characterization of the in-situ condition of the slabs prior to cutting, along with details on the slab geometry and reinforcement, is detailed in Chapter 3.



## 6.2.2 Material properties

### 6.2.2.1 Concrete compressive strength

To determine the compressive strength,  $f'_c$ , of the concrete, two cylindrical cores were extracted from undamaged side segments of each slab after they were cut to 1,000 mm widths. These cores were then tested in compression. The measured compressive strengths ranged from 59.7 MPa to 66.1 MPa. The average  $f'_c$  value for each individual slab is reported alongside its test results in Table 6-2.

### 6.2.2.2 Uncorroded prestressing steel properties

Tensile tests were performed on six uncorroded control wires collected from diaphragm elements in the same bridge section. These diaphragms did not experience any corrosion damage to their tendons. The wires exhibited typical bi-linear elasto-plastic behaviour for high-strength steel, characterized by an initial linear-elastic region extending to approximately 1,400 MPa, followed by a well-defined yield point and subsequent linear-plastic deformation.

A summary of the key mechanical properties is presented in Table 6-1. The reported 5<sup>th</sup> and 95<sup>th</sup> percentile material properties were calculated with the assumption that the test data follow a normal distribution. Further details on the tensile testing program and results are presented in Chapter 3.

Table 6-1 Summary of mechanical properties of six uncorroded wires

| Cable                    | Yield stress, $f_y$<br>(MPa) | Yield Strain, $\epsilon_y$<br>(mm/mm) | Ultimate stress, $f_u$<br>(MPa) | Ultimate strain, $\epsilon_u$<br>(mm/mm) |
|--------------------------|------------------------------|---------------------------------------|---------------------------------|------------------------------------------|
| C1                       | 1431                         | 0.0073                                | 1706                            | 0.0697                                   |
| C2                       | 1372                         | 0.0069                                | 1641                            | 0.0682                                   |
| C3                       | 1415                         | 0.0072                                | 1673                            | 0.0626                                   |
| C4                       | 1414                         | 0.0072                                | 1693                            | 0.0702                                   |
| C5                       | 1466                         | 0.0075                                | 1730                            | 0.0652                                   |
| C6                       | 1430                         | 0.0073                                | 1700                            | 0.0696                                   |
| <b>Average</b>           | 1421                         | 0.0072                                | 1691                            | 0.0676                                   |
| <b>Std. dev.</b>         | 30.63                        | 0.00018                               | 30.43                           | 0.00302                                  |
| <b>COV (%)</b>           | 2.16                         | 2.54                                  | 1.80                            | 4.47                                     |
| <b>5<sup>th</sup> %</b>  | 1371                         | 0.0069                                | 1641                            | 0.0626                                   |
| <b>95<sup>th</sup> %</b> | 1472                         | 0.0075                                | 1741                            | 0.0726                                   |

### 6.2.2.3 Mild steel reinforcement

The secondary reinforcement, consisting of 10M and 15M bars for temperature and crack control, was not subjected to material testing as part of this program, as they were oriented perpendicular to the main load-carrying direction of the slabs. Based on typical design specifications from the construction era, this reinforcement was assumed to be Grade 400 steel with a nominal yield strength of 400 MPa.

### 6.2.3 Four point bending test

Each slab specimen was subjected to a quasi-static, displacement-controlled four-point bending test until failure, adhering to the guidelines of ASTM C78 (ASTM C78/C78M, 2022). The test setup, shown in Figure 6-3, was designed to replicate the in-situ lateral restraint provided by the bridge girders. This was achieved using a stiff testing frame with HSS sections and restraining rods rigidly anchored to the laboratory strong floor, a configuration that was found to provide partial end fixity to the specimens. Digital image correlation (DIC) system was also employed to capture full-field deformation and strain measurements throughout the test.

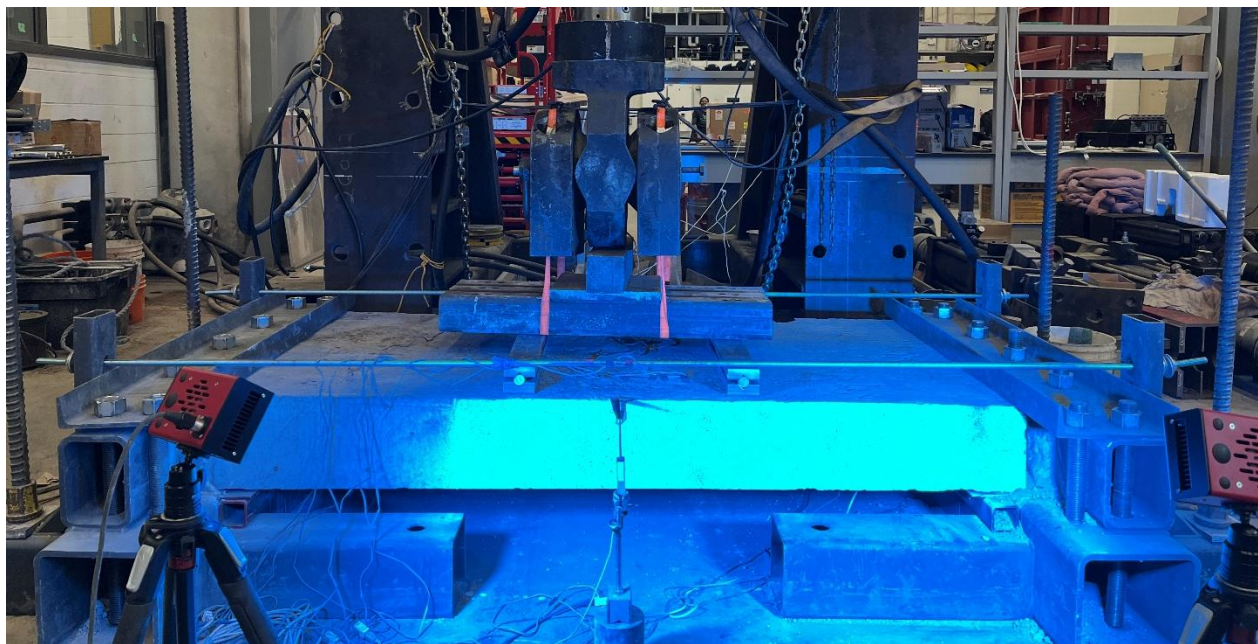


Figure 6-3 Test setup with slab, front/back rods and instrumentation

The key experimental results for four out of the fifteen tested slabs are summarized in Table 6-2. The data includes the concrete compressive strength ( $f'_c$ ), peak load ( $P_{ult}$ ), ultimate mid-span

deflection ( $\Delta_{ult}$ ), and the observed failure mode for each specimen. The slabs exhibited different governing failure mechanisms, including ductile flexure and brittle shear failures. Chapter 4 presents a detailed description of the experimental program and a comprehensive discussion of the detailed structural testing results.

Table 6-2 Summary of experimental bending test results

| Specimen                | Depth (mm) | Compressive strength, $f'_c$ (MPa) | a/d | $P_{crack}$ (kN) | $P_{ult}$ (kN) | $\Delta_{ult}$ (mm) | $\Delta_{failure}$ or $\Delta_{end}$ of test (mm) | Initial stiffness (kN/mm) |
|-------------------------|------------|------------------------------------|-----|------------------|----------------|---------------------|---------------------------------------------------|---------------------------|
| <b>Flexural failure</b> |            |                                    |     |                  |                |                     |                                                   |                           |
| <b>S1A2-S1B2</b>        | 215        | 59.7                               | 5.1 | 100.5            | 356.8          | 33                  | 49.8                                              | 79.7                      |
| <b>S3A2-S3B2</b>        | 210        | 64.3                               | 5.2 | 90.5             | 340.6          | 30.6                | 49.4                                              | 320.4                     |
| <b>S7A2-S7B2</b>        | 207.5      | 65.7                               | 5.4 | 80.2             | 245.5          | 19.5                | 49.9                                              | 61.1                      |
| <b>Shear failure</b>    |            |                                    |     |                  |                |                     |                                                   |                           |
| <b>S8A3-S8B3</b>        | 212.5      | 66.1                               | 4.9 | 102.6            | 233.9          | 23.1                | 24.59                                             | 90.3                      |

#### 6.2.4 Corrosion damage assessment

Following the four-point bending tests, the prestressing strands were extracted from each slab for mass loss analysis (Figure 6-4). Each tendon's cables were cut into segments ranging from 250 mm to 500 mm, with the cutting locations selected based on the distribution of damage features along the cable lengths. Areas with extensive pitting corrosion were avoided near the ends to minimize the risk of premature failure at the grips during testing. To prepare the segments for analysis, rust products were removed according to the ASTM G1 standard (ASTM G1, 2017).

The mass loss analysis of the 12 prestressing strands revealed variations in corrosion levels along the cable lengths. However, higher degrees of corrosion were generally observed near the cable ends. These ends correspond to the connection between the slab and the top flange of the girder, where a construction joint existed. It is likely that saltwater ingress occurred at these joints, leading to more severe corrosion in these areas. These findings are consistent with the NDT tendon corrosion assessments, which indicated higher corrosion rate measurements at the slab/girder connection, as further detailed in Chapter 3. A comprehensive discussion of the corrosion damage assessment, including the full methodology for the mass loss analysis and detailed findings on the distribution of corrosion along the prestressing tendons can be found in Chapter 5.



Figure 6-4 Extracted prestressing cables

### 6.3 Modelling approach for corrosion-induced damage

The FE simulations in this study were conducted using VecTor2, a nonlinear analysis program for two-dimensional reinforced concrete structures developed at the University of Toronto (Wong et al., 2013). The program's framework is based on the Distributed Stress Field Model (DSFM), an extension of the Modified Compression Field Theory (MCFT), which treats cracked concrete as an orthotropic material using a smeared, rotating crack approach. This investigation focuses specifically on modelling the effects of uniform corrosion, which is justified by the experimental observations: localized pitting was primarily confined to the tendon ends near the slab cut locations, regions that were away from the critical flexural sections, and none of the slabs exhibited tendon rupture. The primary experimental input for corrosion was the average mass loss, which was determined gravimetrically according to the ASTM G1 standard. This method provides a value that represents a uniform reduction in the steel cross-section over the length of the reinforcement. The analytical framework within VecTor2 incorporates the degrading effects of corrosion through several key formulations. These models account for the reduction of the steel cross-sectional area, the deterioration of bond strength, and the induction of tensile strains in the cover concrete due to the expansive nature of corrosion products.

#### 6.3.1 Reduction in reinforcement cross-sectional area

The program models uniform corrosion by reducing the area of the discrete truss elements representing the reinforcement. The key input for this model, the uniform corrosion rate ( $i_{\text{corr}}$ ), was derived experimentally from gravimetric analysis. Following the procedures outlined in the ASTM G1 standard, the mass loss was measured for individual wires along the length of the prestressing

tendon. For the analysis, the modelling focused on the most deteriorated region of the prestressing tendon. The representative input was determined by averaging the measured mass loss within the specific wire segment that contained the highest localized corrosion. This approach was adopted because the location of maximum wire corrosion is expected to govern the structural response and initiate failure, thus representing a critical case and conservative approach for assessing residual capacity. The selected mass loss percentage was then converted into an equivalent reduction in the wire's diameter to estimate a uniform corrosion rate in mm/year. This rate was applied over the active corrosion period, which was assumed to begin after a corrosion initiation time of 20 years of service life. The corrosion initiation time of 20 years was assumed based on historical observations of the Champlain Bridge concrete members. Inspection reports indicate that signs of deterioration, including reinforcement corrosion, freeze-thaw damage, and alkali-silica reaction (ASR), were already evident in the 1980s and 1990s, approximately 20-30 years after the bridge's construction (Carlin et al., 1996). Although detailed inspection records from the early service life are limited, a condition assessment conducted in 2008, after about 51 years of service, revealed advanced deterioration of the deck slabs, including concrete spalling and significant corrosion damage (AECOM, 2014). In addition, several slabs near those tested in this study experienced prestressing tendon fractures, indicating severe corrosion progression. The installation of drainage systems in the 1980s, prompted by already observed corrosion issues, further supports this timeline. Because such severe deterioration corresponds to the end of the corrosion propagation phase, which may last for decades, the initiation phase, during which chlorides penetrate the concrete cover and reach the reinforcement, must have occurred much earlier. Therefore, assuming a 20-year corrosion initiation period is considered a reasonable and conservative estimate for this study.

### **6.3.2 Deterioration of bond strength**

The crucial interaction between the prestressing strand and the surrounding concrete is modelled through bond-link elements, which simulate the bond stress-slip relationship. To account for corrosion-induced damage, this study implements the empirical bond strength reduction model proposed by Feng et al. (2016), which was specifically developed for reinforcing bars embedded in concrete and has been adopted in this study as an approximation for prestressing strands. This model describes the evolution of bond strength in three distinct stages. Initially, for low levels of corrosion, the bond strength increases due to the confining effect of rust products accumulating around the reinforcement before the cover concrete cracks. After reaching a peak level of corrosion

( $C_{pk}$ ), the bond strength undergoes a linear reduction. Finally, at high corrosion levels (typically over 10% weight loss), the relationship transitions to a residual strength phase. The bond strength reduction factor,  $R$ , is calculated as shown in Equation 6-1.

$$R(t) = \begin{cases} \left(-0.0320 \frac{c}{d_b} + 0.576\right) C(t) + 1 & \text{for } C(t) \leq C_{pk} \\ \left(-0.0137 \frac{c}{d_b} - 0.0247\right) C(t) + 1.42 + 0.0475 \frac{c}{d_b} & \text{for } C_{pk} < C(t) \leq C_{2-3} \\ -0.0016C(t) + 0.224 & \text{for } C(t) > C_{2-3} \end{cases} \quad \text{Equation 6-1}$$

where  $R$  is the ratio of corroded to uncorroded bond strength,  $C(t)$  is the corrosion mass loss (%),  $c$  is the concrete cover (mm),  $d_b$  is the bar diameter (mm),  $C_{pk}$  is the peak corrosion level which marks the threshold where beneficial rust confinement ends and cracking initiates, and  $C_{2-3}$  is the residual transition point, after which severe cracking has occurred and the bond strength reduces to a long-term residual friction phase.

The peak corrosion level,  $C_{pk}$ , is defined in Equation 6-2 as:

$$C_{pk} = 0.0864 \frac{c}{d_b} + 0.516 \quad \text{Equation 6-2}$$

Figure 6-5b illustrates the bond strength reduction factor,  $R$ , as a function of the corrosion mass loss percentage,  $C(t)$ , derived from the trilinear model proposed by Feng et al. (Feng et al., 2016). For this study, the model utilizes an equivalent bar diameter calculated based on the total area of the 12-wire strand. An average value for the concrete cover was used to establish the representative cover-to-bar diameter ratio,  $c/d_b$ , of 4.67 for the test slabs. This curve effectively demonstrates the three stages of bond degradation relevant to the current study: the initial bond enhancement, the rapid decline following cracking of the concrete cover, and the final residual strength phase for corroded bars with more than 10% weight loss. The trilinear nature of the bond strength degradation curve is governed by two critical transition points:  $C_{pk}$  and  $C_{2-3}$ . The  $C_{pk}$  value correlates to the onset of cracking in the concrete cover, as calculated through models based on the thick-walled cylinder analogy subjected to uniform expansive pressure.

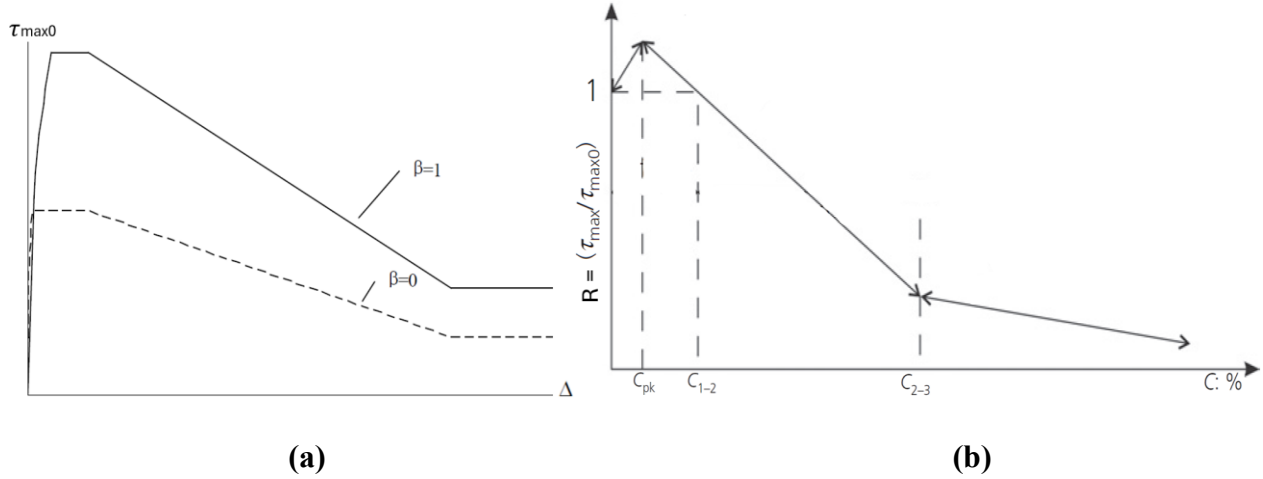


Figure 6-5 Bond reduction models: (a) Eligehausen bond stress-slip model and (b) Feng et al. (2016) model for bond strength reduction as a function of corrosion mass loss

### 6.3.3 Corrosion-induced cracking of concrete cover

The expansive pressure from rust formation is simulated by imposing an initial strain field on the concrete elements surrounding the corroded reinforcement. This study utilizes the model developed by Pantazopoulou and Papoulia (Pantazopoulou & Papoulia, 2001), which was formulated for reinforcing bars, as implemented in VecTor2. To apply this model to the prestressing strands, this study employs an equivalent strand diameter calculated based on the net cross-sectional area of the combined 12 wires that form the strand. The model idealizes the concrete cover as a thick-walled cylinder subjected to uniform internal pressure from the expanding corrosion products. A finite difference scheme is employed to solve the governing differential equation for radial displacement,  $u_r$ , through the cover thickness  $c$ , as shown in Equation 6-3 :

$$\frac{d^2 u_r}{dr^2} + \frac{1}{r} \cdot \frac{du_r}{dr} - \frac{u_r}{r^2} \cdot \frac{E_\theta}{E_r} = 0 \quad \text{Equation 6-3}$$

where:  $\frac{d_b}{2} \leq r \leq \frac{d_b}{2} + c$

The model uses an iterative procedure where the tangential stiffness ( $E_\theta$ ) and radial stiffness ( $E_r$ ) of the concrete are updated at each step to account for cracking and tension softening until the average change of the secant stiffness in two subsequent iterations is negligible (Habibi, 2017). The primary input is the volume of rust, which is determined using a volumetric expansion ratio of 3.0 in this study. This value is consistent with the initial theoretical and experimental studies on

rust expansion, where the ratio of corrosion product volume to consumed iron volume was shown to range from 2.0 to 4.0 (Andrade et al., 1993; Pantazopoulou & Papoulia, 2001). The analysis solves for the radial displacements and the resulting hoop strains ( $\epsilon_\theta = u_r/r$ ) throughout the cover. These calculated hoop strains are then transformed into Cartesian coordinates and applied as an initial strain offset to the concrete elements in the main FE model, simulating the damage caused by corrosion before the application of external loads.

## **6.4 FE modelling methodology**

### **6.4.1 Model geometry**

To simulate the structural response of the recovered Champlain Bridge slabs, two-dimensional (2D) nonlinear FE models were developed using the software VecTor2. The geometry of each model was based on the as-tested dimensions of the slab specimens, with a constant length of 1,950 mm and a variable depth corresponding to the measured thickness of each slab. Taking advantage of the symmetry in both the geometry and the four-point bending test setup, only half of the slab was modelled to improve computational efficiency.

The concrete slab was discretized using four-node plane stress rectangular elements. A mesh sensitivity analysis was conducted to determine an appropriate element size that balances computational accuracy and efficiency, resulting in the selection of a uniform 10 mm x 10 mm mesh (Figure 6-6). The out-of-plane thickness of the model was set to 1,000 mm to represent the full width of the tested slab specimens. Each model was assigned its respective concrete compressive strength ( $f'_c$ ) as determined from the experimental cylinder tests. Other fundamental concrete properties, including the tensile strength ( $f'_t$ ), strain at peak stress ( $\epsilon_o$ ), and the initial tangent modulus ( $E_c$ ), were calculated by VecTor2 using its default formulations based on the provided compressive strength (Wong et al., 2013). Unless otherwise stated, all other concrete material properties were maintained as the default values generated by VecTor2. The specific constitutive models governing the nonlinear behaviour of the concrete are detailed in Section 6.4.3

The reinforcement was modelled using two different approaches. The primary tendon was modeled using two-node discrete truss elements. For the uncorroded regions of the model, these elements were assigned the average mechanical properties of the control wires determined from the experimental program (Chapter 5). It should be noted that no initial prestress was applied to the tendons in the numerical model, as the extracted slabs had their tendons cut during removal and



were no longer expected to retain significant prestressing effects. The modelling of the degraded properties in the corroded sections is discussed in Section 6.4.4 . In contrast, the secondary mild steel reinforcement (10M and 15M bars) was modelled as smeared, out-of-plane reinforcement using the "Ductile Steel Reinforcement" material type, with the appropriate reinforcement ratios assigned to the concrete elements. The interaction and bond between the prestressing tendon and the concrete are detailed in Section 6.4.5 , while the boundary conditions and loading are described in Section 6.4.2 .

#### **6.4.2 Boundary conditions and loading**

The boundary conditions and loading in the FE models were configured to replicate the experimental four-point bending tests as closely as possible. To leverage the symmetrical nature of the test setup, only half of each slab was modeled, with symmetry enforced along the centerline. The boundary conditions were idealized as simply supported. A roller support was assigned to the nodes at the left end of the half-model, restraining them in the vertical direction. To maintain symmetry, a vertical roller was applied at the midspan, restraining the nodes in the horizontal direction only.

It is important to note that the experimental results, as discussed in Chapter 4, determined that the stiff HSS testing frame provided a degree of partial fixity to the slabs. Accurately modelling this partial fixity within a 2D FEA is challenging. As part of the preliminary analyses, two bounding support conditions were investigated, representing perfectly simply supported and fully fixed behaviour. While the fully fixed condition resulted in predicted capacities that were significantly higher than the experimental results, primarily due to the pronounced contribution of membrane action under full restraint, these bounding cases nonetheless provide useful insight into the range of structural response and potential in-situ performance, particularly for shear-critical slabs, where enhanced flexural capacity under increased restraint may shift the governing failure mode toward shear. However, for the purpose of model validation against the experimental results, the fully fixed condition was not considered representative of the test setup and was therefore not adopted in the primary analyses. Instead, simply supported boundary conditions were implemented in the finite element model. Because only half of the slab was modeled, a symmetry boundary condition was applied along the plane of symmetry, restraining displacement normal to the plane while allowing vertical translation and rotation. At the exterior support, a roller boundary condition was used,

restraining vertical displacement while allowing horizontal translation and rotation. This approach avoids introducing artificial in-plane restraint that could generate membrane forces while maintaining consistency with the intended simply supported behaviour. Accordingly, the simply supported model provides a consistent and well-defined baseline for comparison. On this basis, the finite element predictions are expected to underestimate the experimentally measured capacities, particularly for shear-critical slabs, as a result of the partial fixity present in the test configuration.

To prevent unrealistic stress concentrations at the points of support and load application, steel bearing and loading plates were explicitly modeled using rectangular elements with a specified yield strength of 400 MPa. Furthermore, to account for the three-dimensional confining effects of the plates and prevent premature crushing failures, out-of-plane reinforcement was also defined for the concrete elements in the support and loading regions. This modelling approach has been shown by Vecchio and Shim (F. J. Vecchio & Shim, 2004) to be critical for accurately capturing the enhanced strength and ductility in these zones. Consistent with the displacement-controlled nature of the experiments, the load was applied as an imposed support displacement in VecTor2. This vertical displacement was applied to the nodes corresponding to the loading plate located at one-third of the slab full span. Figure 6-6 shows the FE model for a representative slab, illustrating the mesh, boundary conditions, loading configuration, and the location of the prestressing tendon at its specified depth.

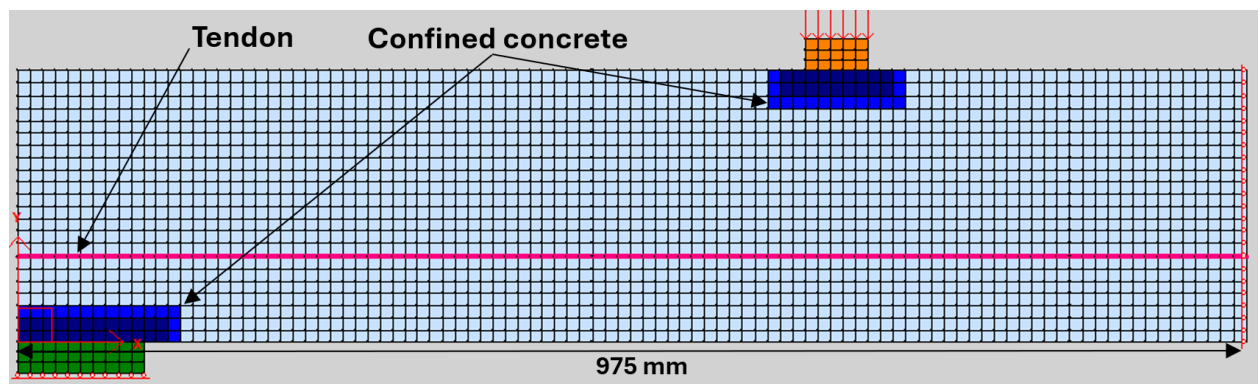


Figure 6-6 2D FE model of a prestressed concrete slab

### **6.4.3 Material constitutive models**

#### **6.4.3.1 Concrete models**

The nonlinear behaviour of the concrete was defined in VecTor2 primarily using the program's default constitutive models, which have been well-validated for a wide range of structures. However, one modification was made to better represent the material behaviour observed in the recovered slabs. The default pre-peak compression response model, the Hognestad Parabola, was changed to the Popovics (HSC) model for high strength concrete. This adjustment was made because the Popovics (HSC) model provides a more accurate representation of the stress-strain curve for concretes with compressive strengths greater than 40 MPa, which is consistent with the strengths measured from the slab core samples. Additionally, the default Nonlinear (Hordjik) tension softening model was replaced with the Bilinear model, which is adapted from the CEB-FIP model code and provides a more refined representation of concrete's post-cracking fracture energy dissipation (Wong et al., 2013).

The other key default models were retained for the analysis. The post-peak compressive behaviour was governed by the Modified Park-Kent model, which accounts for the ductility provided by confinement from the reinforcement. Compression softening, which reduces concrete's compressive strength in the presence of transverse cracking, was handled by the default Vecchio 1992-A model. For the tensile response, the Modified Bentz 2003 model was used for tension stiffening, and a Bilinear model governed tension softening. The initiation of cracking under combined stresses was determined using the Mohr-Coulomb (Stress) criterion, while shear slip along crack surfaces was calculated with the Walraven (Monotonic) model.

#### **6.4.3.2 Reinforcement models**

The behaviour of the steel reinforcement, including both the prestressing tendons and secondary mild steel bars, was defined using the default constitutive models provided in VecTor2. These models were selected to provide a comprehensive simulation of reinforcement behaviour, accounting for the elasto-plastic stress-strain response. The default monotonic stress-strain relationship is governed by the curve of the Seckin model with Bauschinger Effect, which provides a trilinear response consisting of an elastic region, a yield plateau, and a strain-hardening phase. The shear resistance of reinforcement crossing a crack was included using the default Tassios (Crack Slip) model for dowel action. It is also worth noting that the steel behaviour terminates at

the ultimate failure point with no descending branch, as localized strain reductions from corrosion pits were not included since specific morphology was not studied.

#### 6.4.3.3 Bond model

The interaction between the prestressing tendons and the surrounding concrete was simulated using the default bond model in VecTor2, the Eligehausen model (Eligehausen & Popov, E., and Bertero, 1983). This model was applied to the link elements connecting the concrete and the discrete truss elements representing the prestressing tendons, allowing for the simulation of bond-slip behaviour. The Eligehausen model defines the bond stress-slip relationship by establishing two bounding conditions: a ductile pullout failure for well-confined reinforcement and a brittle splitting failure for unconfined reinforcement (Habibi, 2017). The model's stress-slip curve is characterized by four distinct regions: an ascending non-linear branch, a constant bond stress plateau, a linearly declining branch, and a final residual stress branch (Wong et al., 2013). The specific bond behaviour activated in a simulation is controlled by the user-defined confinement pressure factor,  $\beta$ , which ranges from 0 (unconfined splitting) to 1 (fully confined pullout). Figure 6-5 illustrates the interactions in both bond models: (a) Eligehausen and (b) Feng et al. (2016). The resulting actual corroded bond stress,  $\tau_{\max\text{corr}}$ , used in the simulation is therefore the product of the original ultimate bond stress,  $\tau_{\max0}$ , from the Eligehausen model and the degradation factor,  $R$ , from the Feng et al. model.

#### 6.4.4 Simulation of corrosion damage

The simulation of corrosion damage was a two-stage process involving the application of known experimental data followed by a calibration of secondary deterioration mechanisms. First, the primary effect of corrosion, the reduction in the prestressing wire's cross-sectional area, was directly modeled based on experimental measurements. The analysis focused on the most deteriorated segments along the tendon's length, with the corrosion level for these segments being defined by averaging the measured mass loss within that specific zone, as illustrated in the chapter on effect of corrosion on mechanical properties (Chapter 5). Uniform corrosion was selected, as this aligns with the average mass loss data obtained for each wire using the ASTM G1 gravimetric method (ASTM G1, 2017). For slabs S3A2-S3B2, and S7A2-S7B2, the extent of the corrosion zone was defined directly from the experimental corrosion measurements taken along the wires of each tendon. These modelled segments represented the most corroded regions of the wires. A

specific modelling adjustment was necessary for slab S1A2-S1B2 and S8A3-S8B3. For these specimens, the corroded segment was defined from 90 mm to 285 mm from the left edge along the tendon, rather than from 0 mm. This was because initial simulations with corrosion starting at the end of the tendon resulted in a premature debonding failure along the entire length, likely due to simulated anchorage issues. By shifting the corroded zone 90 mm away from the anchorage, this numerical effect was avoided, allowing for a more accurate investigation of the slab's flexural behaviour. For all models, the calculated mass loss was converted into a corrosion rate (mm/year) that served as a fixed input for each respective slab model. Based on the bridge's known history, corrosion was assumed to initiate after 20 years of service, as discussed in Section 6.3.1. Within the VecTor2 software, these parameters are used to calculate the attack penetration which then determines the reduction in the diameter of the reinforcement, bond-strength, and the extent of cover cracking (Habibi, 2017).

Second, a key objective of the numerical study was to investigate the extent to which secondary mechanisms, such as bond degradation and cover cracking, influenced the observed structural behaviour. To this end, the models were calibrated against the experimental load-deflection curves. This was an iterative process where the empirical model for bond strength degradation (Feng et al. (Feng et al., 2016)) and the analytical model for cover cracking (Pantazopoulou and Papoulia (Pantazopoulou & Papoulia, 2001)) were selectively activated. For each slab, the combination of these models was adjusted until the numerical response accurately replicated the behaviour observed in the four-point bending test. This calibration served as an inverse analysis, using the model to diagnose the dominant failure mechanisms. The final modelling approach for each slab, indicating whether the bond and/or cracking models were required to achieve a match, is summarized in Table 6-3. Finally, although the average mass loss for slab S8A3-S8B3 was low at 1.63%, the cover cracking model was activated for this slab due to the physical observation of a crack running along its bottom soffit and parallel to its tendon, accompanied by efflorescence.

Although asymmetric corrosion was observed experimentally along several tendons, its influence on the global structural response of the slabs is expected to be minimal at the studied corrosion levels (<10%). The overall response is governed by the bond and cross-sectional properties at the most severely degraded sections. Therefore, modelling the measured worst-case corrosion level

symmetrically is a conservative simplification that can capture the critical bond-slip and failure mechanisms identified in this study.

Table 6-3 Summary of corrosion input parameters and calibration for corrosion damage models

| Slab      | Avg. mass loss utilized in model | Length of corroded tendon segment (mm) | Corrosion rate (mm/year) | Bond degradation model (Feng et al.) required? | Cover cracking model (Pantazopoulou and Papoulia) required? |
|-----------|----------------------------------|----------------------------------------|--------------------------|------------------------------------------------|-------------------------------------------------------------|
| S1A2-S1B2 | 6.7%                             | 90-294                                 | 0.022                    | Yes                                            | Yes                                                         |
| S3A2-S3B2 | 1.71%                            | 500-700                                | 0.0055                   | Yes                                            | Yes                                                         |
| S7A2-S7B2 | 9.01%                            | 294-664                                | 0.0297                   | Yes                                            | Yes                                                         |
| S8A3-S8B3 | 1.63%                            | 90-285                                 | 0.00533                  | No                                             | Yes                                                         |

#### 6.4.5 Modelling of the strand-concrete bond interface

To simulate the complex interaction and potential slip at the strand-concrete interface, the bond behaviour was explicitly modeled. The prestressing tendon, represented by discrete truss elements, was connected to the surrounding concrete elements using two-node link elements. These non-dimensional link elements act as springs that simulate the bond stress-slip relationship, allowing for an imperfect bond (Wong et al., 2013). This modelling approach also aims to capture the loss of bond associated with poor grout quality, which was identified during the experimental investigation (Chapter 5). For the slabs that failed in flexure, this imperfect bond modelling was applied specifically within segments of the shear span, where corrosion was also modelled, as detailed in Table 6-3, while a perfect bond was considered in the constant moment region. As for slab S8A3-S8B3, which failed in shear, a complete loss of bond was modeled in the flexure zone (constant moment region) by treating the tendon as unbonded in this region, while a perfect bond was considered in the shear span, combined with corrosion of the reinforcement. The lower midspan deflections observed at both ultimate load and at the end of testing, as displayed in Chapter 4, combined with the formation of pronounced diagonal shear cracks within the shear span, indicate that the prestressing tendon in slab S8A3-S8B3 did not develop significant stress in the constant moment region. This behaviour is consistent with loss of bond near midspan, resulting in reduced flexural participation of the tendon and a premature shear-controlled failure.

A parametric study was initially conducted to investigate the sensitivity of the models to the mode of bond failure. The constitutive behaviour of the link elements was governed by the default

Eligehausen Model, which can simulate two bounding conditions: a brittle splitting failure (characteristic of unconfined bars) and a ductile pullout failure (characteristic of well-confined bars). These two extremes are controlled by the  $\beta$  factor, where  $\beta=0$  represents an unconfined condition and  $\beta=1$  represents a fully confined condition.

However, the simulations revealed that the global load-deflection response of the slabs was largely insensitive to this parameter. For both the flexural-dominant and shear-dominant slabs, the results obtained using  $\beta=0$  (splitting failure) were nearly identical to those obtained using  $\beta=1$  (pullout failure). This important finding suggests that the structural behaviour of these restrained slabs is governed by other, more dominant mechanisms. Once a state of imperfect bond is introduced locally, the ultimate capacity is dictated primarily by the yielding of the reinforcement rather than the specific post-peak characteristics (i.e., brittle vs. ductile) of the local bond-slip relationship. Therefore, for the final analyses, a single imperfect bond with unconfined condition was used.

Figure 6-7 illustrates the FE meshes for the various tendon bond conditions in the four analyzed slabs, which were determined through the model calibration process. The bond between the prestressing tendon and the surrounding concrete was simulated using discrete link elements, which allowed for the definition of specific bond properties along the tendon's length. In the figure, the uncorroded portion of the tendon is shown in pink, while the segments where corrosion was applied are highlighted in black. For slabs S1A2-S1B2, S3A2-S3B2, and S7A2-S7B2, which exhibited flexural failures in the experimental tests, the calibration process required modelling a region of imperfect bond to accurately replicate their behaviour. As shown in Figure 6-7 (a-c), this imperfect bond zone was implemented within the shear span for these three slabs. In contrast, for slab S8A3-S8B3, the model calibration necessitated the definition of a fully unbonded tendon segment located directly within the high-flexure zone, as depicted in Figure 6-7 (d). Modelling bond conditions in a targeted manner was essential because the location and extent of bond degradation directly govern the mechanisms of force transfer between the prestressing tendon and the surrounding concrete, thereby controlling stiffness development, stress redistribution, and the dominant failure mode. It should also be noted that visual inspections prior to testing, as well as the retrieval of tendons following the structural tests, indicated that grout quality was inconsistent in several regions along the tendons. Although the exact locations of deficient grout could not be determined with certainty, these observations suggest that bond deterioration was also affected by grout deficiencies.

For the flexure-controlled slabs, localized bond degradation in the shear span promoted longitudinal strain redistribution along the tendon. In such cases, sufficient bond was maintained within the constant-moment region to allow the tendon to develop near-ultimate tensile stresses, causing flexural cracking and yielding to remain the primary failure mechanism.

Conversely, the shear-controlled behaviour of S8A3-S8B3 indicated that the tendon was not effectively developing tension in the constant-moment region. Representing this loss of bond within the flexure zone was therefore essential to capture the reduced flexural participation of the tendon and the resulting shift toward a shear-dominated response. The absence of bond in the high-moment region limited strain compatibility between the tendon and concrete, suppressing flexural resistance and increasing shear demand on the degraded concrete, which ultimately governed failure.

To accurately simulate the shear failure observed experimentally in slab S8A3-S8B3, a calibration of the concrete's tensile properties was required in the numerical model. While post-cracking parameters such as tension softening and tension stiffening models were also investigated, modifying the tensile strength and modulus of elasticity proved to be the most effective means of capturing the correct failure mode. The elastic modulus of the concrete in these regions was reduced by 73% from 44,729 MPa to 11,900 MPa to represent existing stiffness degradation associated with service-induced damage. Visual inspection revealed a longitudinal crack with efflorescence along the slab soffit, indicating prior moisture ingress and damage. In addition, the slab experienced localized damage during the cutting phase, where a small portion of concrete detached along the longitudinal edge near the region where the shear crack later propagated. These factors likely reduced the effective structural stiffness of the specimen by introducing internal cracking and local damage along critical regions. In addition, the concrete's tensile strength was ultimately reduced by 14 %, from 2.68 MPa to 2.30 MPa, for the modified material, represented by the brown-coloured elements in the FE mesh shown in Figure 6-7 (d). Consistent with the damage models proposed by Coronelli and Gambarova (Coronelli & Gambarova, 2004), the reduction in the static modulus of elasticity was assumed to be more severe than the reduction in the tensile strength. This discrepancy is attributed to the high sensitivity of concrete stiffness to internal micro-cracking caused by the expansion of rust products, which degrades the elastic properties of the matrix more rapidly than its ultimate tensile capacity (Zeng et al., 2022), in addition to secondary service-induced



mechanisms such as freeze-thaw cycles due to the bridge's exposure conditions and prior microcracking from long-term loading.

The modulus of elasticity was estimated from the dynamic modulus ( $E_d$ ) calculated from ultrasonic pulse velocity (UPV) measurements, as part of the initial non-destructive testing explained in Chapter 3, and by using the fundamental relationship between wave velocity and elastic constants (Malhotra & Carino, 2003), as shown in Equation 6-4:

$$E_d = \frac{V_p^2 \cdot \rho \cdot (1 + \nu)(1 - 2\nu)}{(1 - \nu)} \quad \text{Equation 6-4}$$

where  $V_p$  is the pulse velocity (m/s),  $\rho$  is the density (kg/m<sup>3</sup>), and  $\nu$  is Poisson's ratio.

For slab S8A3–S8B3, UPV measurements varied between 2.2 and 3.5 km/s. A conservative UPV value of 2.5 km/s was therefore selected for the model. Assuming a concrete density of 2,400 kg/m<sup>3</sup> and a Poisson's ratio of 0.2, the dynamic modulus was derived. Following the correlation established by Lydon and Balendran (Lydon & Balendran, 1986), the static modulus was taken as 83% of the dynamic modulus.

To determine the tensile strength of the degraded concrete elements in the FE mesh, a range of compressive and tensile strengths was estimated using combined NDT methods, as proposed by Kheder (Kheder, 1999) and shown in Equation 6-5. These formulations integrate NDT parameters including UPV and Rebound Number (RN) to account for both internal matrix integrity and surface hardness. RN measurements for S8A3-S8B3 were obtained as part of the NDT studies on the slabs, detailed in Chapter 3. Following the model for air-dry concrete, the relationship is defined as shown in Equation 6-5:

$$f'_c = 0.0158(V_p^{0.4254}) \cdot (RN^{1.1171}) \quad \text{Equation 6-5}$$

where  $V_p$  is the pulse velocity (m/s) and  $f'_c$  is the compressive strength (MPa).

For the measured UPV ranging between 2,300 m/s and 4,000 m/s, the calculated tensile strength, using Equation 6-6, which corresponds to the direct tensile strength relationship adopted in the MCFT and used in the constitutive models implemented in VecTor2 (Wong et al., 2013), varied from 2.04 MPa to 2.30 MPa.

$$f_t = 0.33\sqrt{f'_c} \quad \text{Equation 6-6}$$

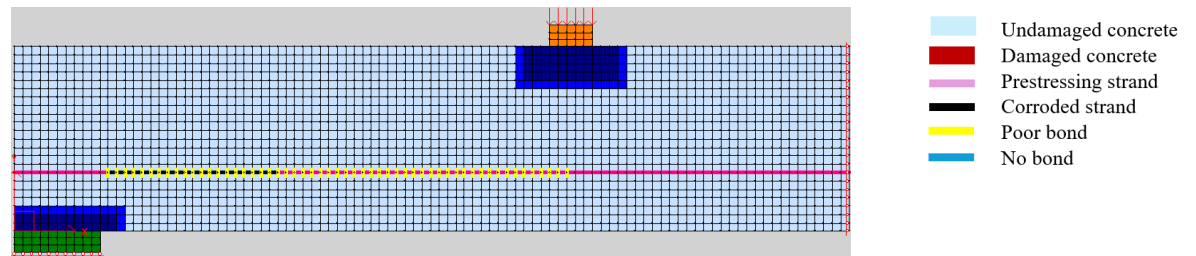
Although field measurements for S8A3-S8B3 typically fell between 2.2 and 3.5 km/s, suggesting a tensile strength between 2.16 and 2.23 MPa, a value of 2.30 MPa was ultimately adopted for the modified material elements. This value was selected due to the fact that corrosion-induced micro-cracking disproportionately degrades the concrete's static modulus and wave velocity, while the ultimate tensile capacity remains relatively more resilient (Coronelli & Gambarova, 2004; Zeng et al., 2022). Relying strictly on the lower tensile strength values derived from the 2.5 km/s UPV readings would likely result in an overestimation of damage and premature cracking in the numerical model that does not align with the observed global structural response of the slabs.

These modifications are critical because shear failure in members without transverse reinforcement is fundamentally a diagonal tension failure. The reduction in tensile capacity, combined with localized stiffness degradation, promotes the initiation and propagation of diagonal cracking at load levels lower than those required to activate ductile flexural mechanisms, thereby reproducing the experimentally observed shear-critical response.

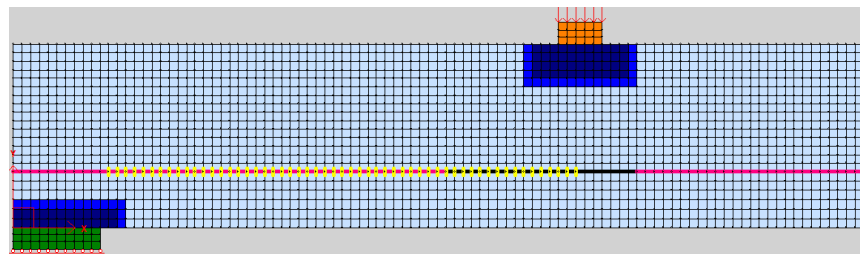
The adopted reductions are physically justified by the presence of corrosion-induced damage in the slabs extracted from the Champlain Bridge. Although the maximum average tendon mass loss in S8A3-S8B3 was limited (1.63%), corrosion is known to induce distributed microcracking, loss of aggregate interlock, and degradation of concrete tensile resistance in the surrounding concrete (Coronelli & Gambarova, 2004). These effects are not fully captured by mass-loss-based steel modelling alone. Such damage mechanisms primarily affect tensile and shear transfer behaviour rather than compressive strength. In addition, slab S8A3-S8B3 exhibited a longitudinal crack running along the tendon length on the soffit. This crack, combined with the crumbled and powdery grout observed during tendon extraction, as shown in Chapter 4, suggests that deterioration was not solely caused by corrosion but may also be associated with deficient or deteriorated grout conditions along the tendon. Furthermore, the reduction of the modulus of elasticity is also a representation of the poor concrete quality identified through UPV mapping, indicating that the slab condition was influenced by both corrosion effects and underlying material deficiencies.

The modelling of the unbonded region was limited to the flexural zone. Although corrosion was observed primarily near the supports, the assumed loss of bond was attributed mainly to deficient grouting conditions rather than corrosion, as poor grouting can reduce bond between the tendon

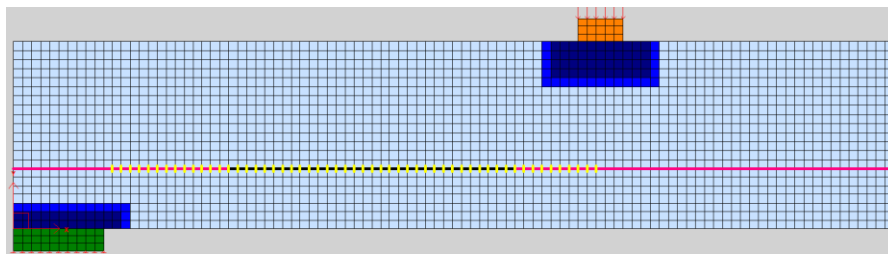
and surrounding concrete. In the flexural zone, the steel exhibited corrosion levels below 1%. However, inspection revealed poor-quality grout along the tendon in this zone, with multiple areas of crumbled or unbonded grout, as shown in Chapter 4. These conditions effectively created unbonded segments along the tendon in the moment zone.



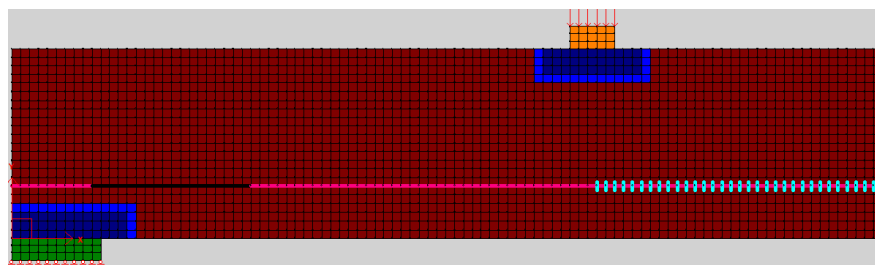
(a) S1A2-S1B2



(b) S3A2-S3B2



(c) S7A2-S7B2



(d) S8A3-S8B3

Figure 6-7 FE models of the analyzed slabs

## **6.5 Model validation and discussion**

### **6.5.1 Comparison of load-deflection response**

#### **6.5.1.1 Flexural-dominated slabs**

Figure 6-8 presents the load versus midspan deflection curves for the flexural slabs S1A2-S1B2, S3A2-S3B2, and S7A2-S7B2. Each figure compares the experimental response (black curve) with two distinct FE models: a “perfect bond (control)” model which assumes a perfect bond along the entire tendon length and does not account for corrosion, and an “imperfect bond (corroded)” model which incorporates corrosion effects and assumes an imperfect bond between the tendon and surrounding grout within the shear span. The collective results allow for a critical evaluation of the models' ability to capture the uncracked, post-cracked, and ultimate stages of the slabs' flexural behaviour. It is important to note that the results are considered conservative relative to real behaviour because the FEM does not include the initial prestressing force. Omitting the prestressing force in this analysis ensures a conservative baseline when modelling the behaviour of the slabs tested in the lab, as it evaluates the structural capacity without the beneficial effects of prestress.

Across all three specimens, a strong agreement is observed in the initial, pre-cracking stiffness between the experimental data and both numerical models. The slopes of the three curves are nearly identical up to the point of first cracking. This indicates that the material properties assigned to the uncracked concrete and the overall geometry of the models were accurately defined and representative of the physical specimens.

A key discrepancy arises in the post-cracking phase, where the numerical and experimental behaviours diverge. In the physical slabs, the loss of stiffness after initial cracking is a gradual process. As cracks form and widen, the member's flexural rigidity decreases progressively, with the concrete located between the cracks still contributing to the overall stiffness through tension stiffening. This results in the smooth, load-deflection path (post-capacity) seen in the experimental data. However, the imperfect bond model simulates this transition in an abrupt manner. The sharp drop in stiffness suggests that upon first cracking within the shear span, there is a sudden and widespread loss of composite action over that entire zone due to the tendon instantaneously transitioning from behaving as fully bonded reinforcement to a largely unbonded tendon over a significant length. This simulated, abrupt loss of strain compatibility and tension stiffening causes an exaggerated and premature reduction in the member's flexural rigidity, which is shown as the

sharp drop in the curve. Conversely, the control model indicates a higher post-cracking stiffness and acts as a stiff upper bound. This establishes the two models as theoretical bounds on the initial stages of the post-cracking behaviour.

The load-deflection response of the control models exhibit a distinct load 'spike' and subsequent drop at approximately 160 kN (for S1A2-S1B2 and S3A2-S3B2) and 180 kN (S8A3-S8B3). Analysis of the internal strain field using the Augustus post-processor (Bentz, 2013) confirms that this event is associated with the initiation of a longitudinal bond-splitting crack along the tendon axis. These spikes are attributed to the necessary simplification of modelling the 12-wire PT tendon as a single equivalent longitudinal bar. This concentrates the bond-transfer and radial bursting stresses onto a reduced interface surface, which then exceeds the splitting tensile capacity of the unconfined concrete.

The corroded model represents a flexible lower bound in this region due to its simulated premature loss of post-cracking stiffness, while the actual experimental behaviour lies between these two idealized extremes. This indicates that, at a global level, the corroded slabs retained some degree of composite action and did not experience the abrupt loss of stiffness predicted by the imperfect bond model immediately after cracking. Nevertheless, despite this difference in post-cracking behaviour, the imperfect bond model still provided a more accurate representation of the overall load-deflection response than the perfect bond model.

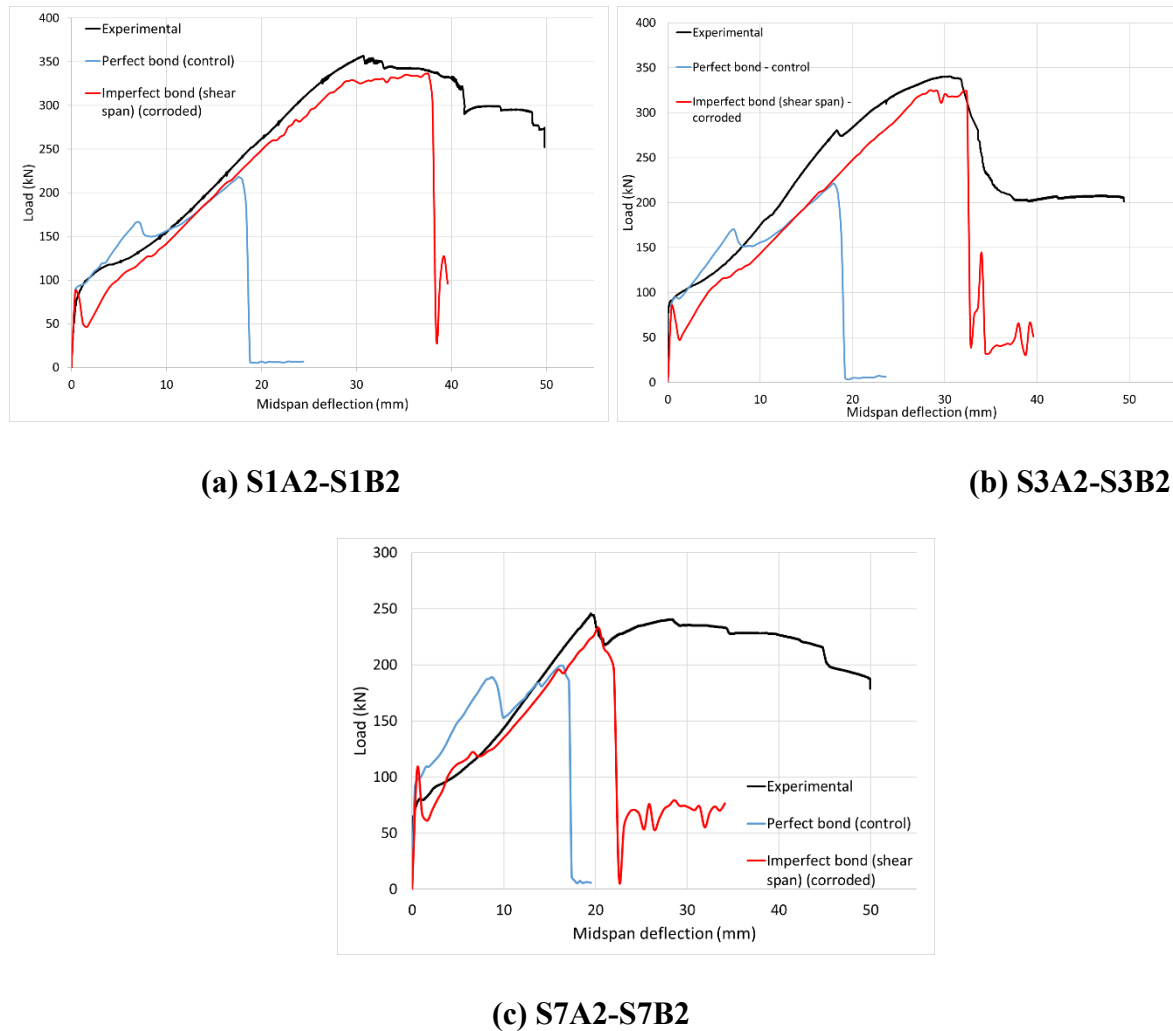


Figure 6-8 Comparison of experimental and numerical load-deflection curves for flexural-dominated slabs

The FE analysis for the undamaged (perfect bond) control slab exhibits a lower ultimate capacity compared to the experimental measurements of the corroded slabs. This discrepancy arises from the idealized assumptions in the control model, which considers perfect bond along the reinforcement. In contrast, the reduced bond in the corroded slabs allows stress to redistribute over a larger area, shifting the structural behaviour from beam action toward membrane or arch action. This redistribution mobilizes additional in-plane stiffness, thereby increasing the flexural ultimate load observed experimentally.

A critical aspect of the models' validity is the ability to predict the ultimate load and corresponding deflection. The corroded model demonstrated excellent predictive accuracy across all three slabs. For slab S1A2-S1B2, the model predicted an ultimate load of 336 kN at a failure deflection of 37.2

mm, which corresponds closely to the experimental results of 357 kN and 33 mm. Similarly, for slab S3A2-S3B2, the predicted values of 325 kN and 28.4 mm aligned well with the observed 341 kN and 30.6 mm. This accuracy was maintained for slab S7A2-S7B2, where the model calculated a capacity of 233 kN at 20.4 mm, compared to the experimental measurement of 246 kN at 19.5 mm (Table 6-5).

To validate the theoretical capacities from these models, a sectional analysis for the simply supported slabs was performed. The analysis assumed that plane sections remain plane at ultimate conditions. A concrete crushing strain of 0.0035, consistent with a rectangular stress block assumption, was adopted at ultimate conditions, and failure was assumed to occur by concrete crushing. No material strength reduction factors were applied. Initial prestressing forces in the tendons were neglected, as the tendons had been cut and any remaining residual force was considered negligible. As shown in Table 6-4, the results of this analysis align almost perfectly with the peak loads predicted by the control model, yielding ratios of calculated-to-predicted capacity near 1.0 for all slabs.

Table 6-4 Comparison of sectional analysis and perfect bond (control) model results

| Slab      | $P_{ult}$ (kN) |                    | Ratio |
|-----------|----------------|--------------------|-------|
|           | Control model  | Sectional analysis |       |
| S1A2-S1B2 | 218            | 208                | 1.05  |
| S3A2-S3B2 | 221            | 210                | 1.05  |
| S7A2-S7B2 | 199            | 217                | 0.92  |

#### 6.5.1.2 Shear-dominated slab

The modelling of slab S8A3-S8B3, which failed in shear, required a different approach than the flexure-dominated slabs. The model exhibited a brittle shear failure by reducing the concrete's tensile strength (14%) and elastic modulus (73%), reflecting the stiffness degradation associated with greater service-induced damage. These reductions led to the formation and growth of diagonal cracks at lower load levels, preventing the development of ductile flexural behaviour. Furthermore, the corrosion-induced bond degradation model was intentionally omitted based on the findings of preliminary sensitivity studies. These studies revealed that including the bond degradation model due to the confining effect of early-stage corrosion, as per the Feng et al. model (Feng et al., 2016), provided stiffer post-cracking behaviour. At low levels of corrosion, which is evident in slab S8A3-S8B3, an enhanced bond strength can occur (Habibi, 2017). This is because the initial build-up of

corrosion products applies radial pressure to the surrounding concrete, which adds to the confining action provided by the cover concrete and increases the frictional component of bond before significant cover cracking begins. This improved bond may result in a stiffer member. The inclusion of this model produced a stiffer post-cracking response that contradicted the experimental observations, and hence, was deemed unsuitable.

Figure 6-9 presents the load-deflection curves comparing the experimental result with the control and shear-calibrated models.

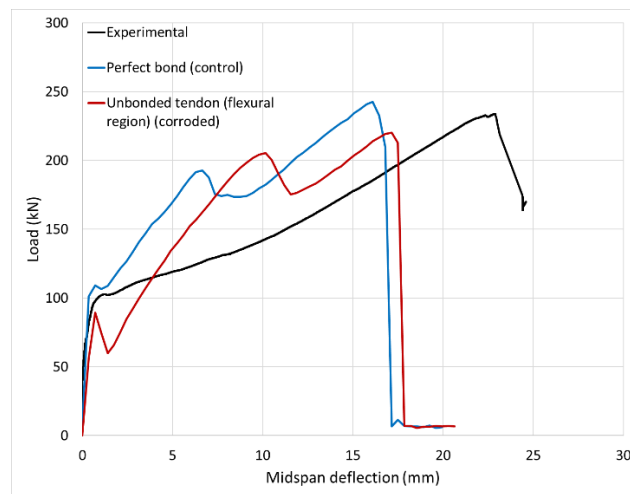


Figure 6-9 Comparison of experimental and numerical load-deflection curve for shear-dominant slab

Consistent with the findings from the flexural-dominant slabs, the control model accurately captured the initial pre-cracking stiffness of the experimental member and subsequently acted as a stiff upper bound to its response. A key initial observation from the control model is that the theoretical capacity of the slab was slightly higher than the load at which it actually failed experimentally.

The primary success of the shear-calibrated model was its ability to capture the correct failure mode of a sudden, brittle failure with limited deformation capacity. This demonstrates that the strategy of reducing the concrete's tensile strength and modulus of elasticity was effective in enabling a diagonal tension-mechanism failure before its full flexural capacity was reached.

The finite element model predicted an ultimate load of 220 kN compared to the experimentally measured 234 kN, corresponding to an underestimation of approximately 6%. The associated ultimate deflection was predicted as 17.2 mm, compared to 23.1 mm experimentally, representing



an underestimation of approximately 26%. Such discrepancies are commonly encountered in nonlinear FEM of concrete structures and stem primarily from modelling simplifications and uncertainties in key behavioural assumptions (Mathern & Yang, 2021). In addition, the discrepancy in the ultimate deflection is consistent with commonly reported trends in nonlinear FEA, where idealized material models, smeared cracking formulations, and simplified bond representations tend to produce a stiffer post-cracking response and reduced deformation capacity compared to experimental observations (Mathern & Yang, 2021).

The reduction in concrete tensile strength and elastic modulus follows a process based on the forensic evidence of S8A3-S8B3, as detailed in Chapter 3. The observed longitudinal crack parallel to the tendon and the presence of 'powdery' grout indicate a pre-existing state of bond-anchorage failure. By using 'effective' properties, the FEM accounts for the loss of bond at tendon-concrete interface and the reduction in stiffness and internal confinement caused by this damage.

In contrast to the flexural analyses, the shear-dominated model did not capture the post-cracking stiffness with the same degree of accuracy. As seen in Figure 6-9, the model exhibits a more complex and stiffer response path compared to the nearly linear and softer behaviour of the experimental slab after cracking. One primary reason could be the presence of unmodelled conditions in the experimental slab. For instance, it is plausible that poor grouting conditions or other construction imperfections created additional unbonded/imperfectly bonded zones in other locations along the tendon that were not accounted for in the model, which would alter the global stiffness. Impact Echo (IE) NDT testing, as detailed in Chapter 3 revealed multiple anomalous responses at depths corresponding to the tendon level along the tendon length, which may indicate the presence of grout defects or voids and supports the likelihood of poor grouting conditions. Although a splitting crack was observed along this slab's soffit, which would typically suggest higher corrosion levels, the measured corrosion was very low, making grouting issues a more likely explanation for the altered global stiffness.

#### **6.5.1.3 Reliability of the models**

The reliability of all models was confirmed by the predicted-to-experimental ratios. The ultimate loads and corresponding deflections from the experimental tests and the FE model are summarized in Table 6-5. For the three flexure-controlled slabs, the ultimate load ratios were consistently accurate and slightly conservative, ranging between 0.94 and 0.95, while deflection ratios at the

ultimate load also showed a strong correlation (ranging from 0.93 to 1.13). Furthermore, the flexure-controlled slab models also replicated the ductile failure mode but predicted a more abrupt drop in load after the peak, rather than the gradual post-peak response observed experimentally.

For the shear-controlled slab S8A3-S8B3, the predicted ultimate load ratio remained similarly conservative (0.94). However, the predicted deflection was lower than the experimental value, with a ratio of 0.74. This discrepancy is attributed to the brittle nature of shear-dominated failure, which limits deformation capacity and is highly sensitive to localized damage mechanisms. Despite the reduced deflection prediction, the model correctly captured the transition to shear-controlled behaviour and the loss of ductility observed experimentally, which is the critical outcome from a safety and failure-mode perspective.

Table 6-5 Comparison of experimental and predicted ultimate loads and deflections

| Slab      | Ultimate load, $P_{ult}$ (kN) |                               | Ratio of<br>predicted to<br>exp. | $\Delta_{ult}$ (mm)      |                                    | Ratio of<br>predicted to<br>exp. |
|-----------|-------------------------------|-------------------------------|----------------------------------|--------------------------|------------------------------------|----------------------------------|
|           | $P_{ult}$ (exp.)              | $P_{ult}$ (corroded<br>model) |                                  | $\Delta_{ult}$<br>(exp.) | $\Delta_{ult}$ (corroded<br>model) |                                  |
| S1A2-S1B2 | 357                           | 336                           | 0.94                             | 33.0                     | 37.2                               | 1.13                             |
| S3A2-S3B2 | 341                           | 325                           | 0.95                             | 30.6                     | 28.4                               | 0.93                             |
| S7A2-S7B2 | 246                           | 233                           | 0.95                             | 19.5                     | 20.4                               | 1.04                             |
| S8A3-S8B3 | 234                           | 220                           | 0.94                             | 23.1                     | 17.1                               | 0.74                             |

## 6.5.2 Comparison of failure modes

### 6.5.2.1 Flexure-dominated slabs

A visual comparison of the crack patterns provides strong validation of the FE models, confirming that these models accurately captured the physical failure mechanisms of the flexure-dominated slabs. Table 6-6 compares the crack patterns predicted by the FE models at ultimate load with photos of the tested slabs at failure, showing good agreement in the location and orientation of the damage. For all specimens, the models correctly predicted that failure would be initiated with vertical flexural cracks forming at the bottom tension face and propagating upwards.

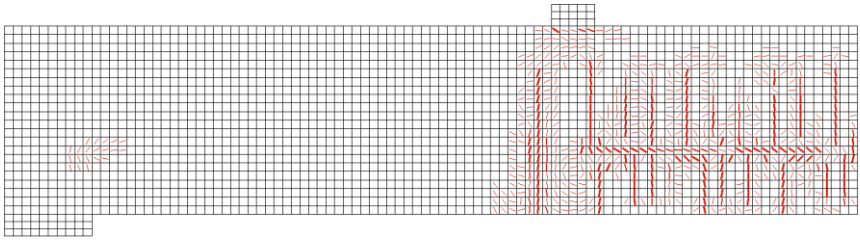
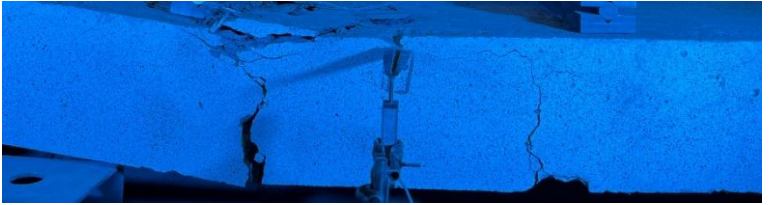
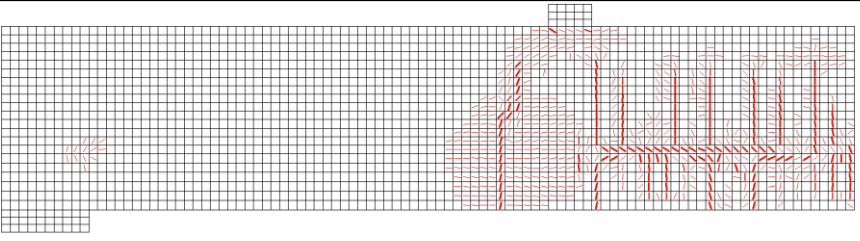
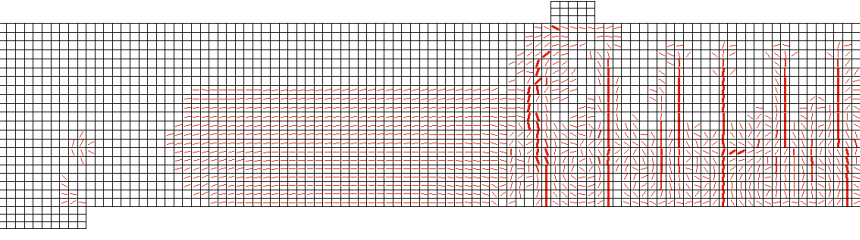
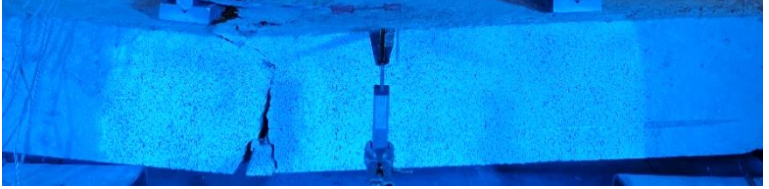
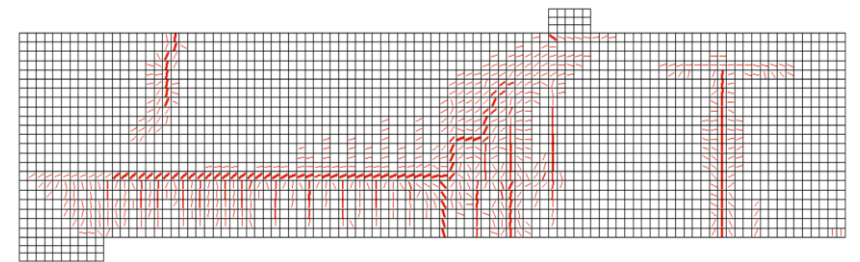
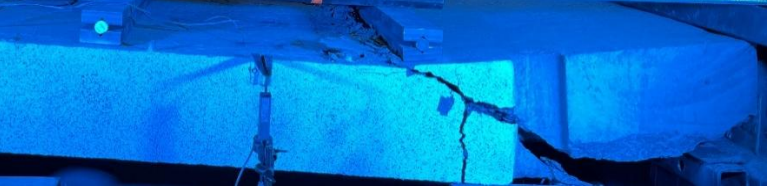
However, a notable difference in the crack distribution exists. The FE models show dense regions of vertical cracks, whereas the experiments typically resulted in the formation of a few major, wide cracks. The development of a single dominant flexural crack can be associated with compromised bond conditions (Mousa & Mousa, 2016). When the bond is poor, curvature and reinforcement

strains become concentrated at fewer crack locations. This leads to the localized crushing of the concrete in the compression zone and contributes to the stable, plateau-like post-peak response seen in the load-deflection curves.

The FE model did not fully capture this concentration of damage into a single crack because the imperfect bond was defined only in the shear spans, not in the constant moment region where these flexural cracks developed. For the flexure-controlled slabs, localized bond degradation in the shear span primarily promoted longitudinal strain redistribution along the tendon rather than strain localization at a single crack. In these cases, sufficient bond was maintained within the constant-moment region to preserve strain compatibility between the tendon and concrete, allowing the tendon to develop near-ultimate tensile stresses. As a result, flexural cracking remained distributed, and yielding governed the response rather than the formation of a single dominant crack.

This discrepancy suggests that the experimental slabs may have experienced localized areas of poor grouting conditions within the constant moment region that was not included in the model. Furthermore, the analysis confirmed that the tension reinforcement experienced yielding at ultimate load conditions, which is the defining characteristic of this ductile failure mode. Despite the differences in crack distribution, the modelling approach successfully captured the overall ductile failure mode and the critical damage locations at the ultimate load.

Table 6-6 Comparison of predicted and experimental crack patterns at ultimate load for flexural slabs

| Slab      | Cracking pattern from FE model at failure                                            | Cracking map at failure for test specimen                                             |
|-----------|--------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------|
| S1A2-S1B2 |    |    |
| S3A2-S3B2 |    |    |
| S7A2-S7B2 |   |   |
| S8A3-S8B3 |  |  |

### 6.5.2.2 Shear-dominated slabs

In contrast to the ductile flexural failures, the shear-dominated model for slab S8A3-S8B3 successfully captured the brittle shear failure observed in the experiment. This is confirmed by a strong agreement in both the physical crack pattern and the internal state of the reinforcement.

The experimental failure was sudden and governed by the formation of a single, catastrophic diagonal tension crack that propagated from the support region up towards the loading point, as shown in the test photograph (Table 6-6). The FE model accurately predicted this distinct failure mechanism. As seen in Table 6-6, the model shows a localized group of cracks forming along a nearly identical diagonal path. The excellent correlation in the location, angle, and nature of this critical shear crack provides strong validation for the modelling strategy. In addition, a defining characteristic of a brittle shear failure is that it occurs before the longitudinal reinforcement has a chance to yield. This was confirmed by the absence of a ductile response in both the experimental and FE load-deflection curves, as well as the maximum tendon steel stress value of 861 MPa in the tendon at ultimate conditions, which is below the yield stress.

## 6.6 Conclusions

This study successfully developed and validated nonlinear FE models against unique experimental data from full-scale, 57-year-old naturally corroded prestressed concrete slabs recovered from the Champlain Bridge. This work provides a unique benchmark for the numerical tools used to assess aging infrastructure, leveraging data that reflects natural, long-term deterioration processes rather than those from accelerated laboratory tests. The investigation yielded several key conclusions regarding the modelling of corrosion-induced damage and its influence on structural performance, specifically when investigating tendon-concrete bond conditions. The modelling approaches are summarized in Table 6-7.

- This observed load increase in the FE analysis appears to be associated with a decrease in bond strength, likely due to poor grouting conditions. The reduced bond allows stresses to redistribute over a larger area and shifts the structural response from beam action toward membrane/arch action, which can lead to a higher observed load capacity.
- In natural, long-term deterioration, the interaction between corrosion and bond behaviour can be complex. Experimental studies have shown that low levels of corrosion may

temporarily enhance bond at the steel-concrete interface due to surface roughening and radial pressure generated by corrosion products. At higher corrosion levels, however, the expansion of corrosion products can lead to cracking of the surrounding concrete and subsequent bond degradation. In the present study, the measured corrosion levels were low, and the observed reduction in bond is therefore associated with poor grouting conditions. The resulting decrease in bond allows stresses to redistribute over a larger area, shifting the response toward membrane/arch action and increasing the observed load capacity despite the presence of deterioration.

- For the three slabs that failed in a ductile flexural mode, the models demonstrated that the structural response was governed by the integrity of the strand-concrete bond. Accurate prediction of the ductile behaviour and ultimate capacity, with predicted-to-experimental load ratios between 0.94 and 0.95, was only possible when an imperfect bond, resulting poor grouting conditions, was explicitly modelled in the shear spans. This confirms that for flexure-critical members, corrosion-induced bond degradation is a primary mechanism controlling residual capacity and ductility.
- In contrast, the brittle shear failure of slab S8A3-S8B3 was not governed by the same bond-slip model. Instead, replicating the sudden diagonal tension failure required a 14% reduction in the concrete's tensile strength and treating the tendon as unbonded in the constant moment region. This indicates that for this shear-critical member, the dominant effect of deterioration was the compromised ability of the concrete matrix to resist diagonal tension, a failure mode that precedes significant bond slip or tendon yielding.
- A primary challenge in modelling corrosion in real structures compared to laboratory specimens is the high degree of variability in corrosion damage and bond conditions, as well as the nature of corrosion. While laboratory corrosion is often uniform and controlled, natural corrosion may be localized and may involve deficient or deteriorated grouting conditions that significantly affect the tendon-concrete bond. Ensuring good-quality grouting is critical to promote reliable bond performance, yet grout defects and voids are difficult to accurately detect and quantify, even with modern NDT techniques such as IE.

Table 6-7 Summary of modelling approaches based on failure mode

| Feature                                                  | Flexural-dominated slabs                                                                                              | Shear-dominated slab                                                                                                                 |
|----------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------|
| <b>Observed failure mode</b>                             | Ductile flexure, characterized by reinforcement yielding and a post-peak plateau.                                     | Brittle shear, characterized by a sudden diagonal tension crack and loss of capacity.                                                |
| <b>Critical calibration parameter</b>                    | Inclusion of the Feng et al. (20) empirical model for corrosion-induced bond degradation.                             | A 73% and 14% reduction in the concrete's elastic modulus and tensile strength.                                                      |
| <b>Modeled bond condition</b>                            | Imperfect bond in the shear span.                                                                                     | Unbonded in the flexural zone.                                                                                                       |
| <b>Model reliability (<math>P_{pred}/P_{exp}</math>)</b> | High (0.94 - 0.95).                                                                                                   | Conservative (0.76).                                                                                                                 |
| <b>Inferred governing mechanism</b>                      | Degradation of tendon-concrete bond integrity, leading to a loss of composite action and increased flexural capacity. | Loss of concrete's diagonal tension capacity, triggering a brittle failure before the member's flexural capacity could be mobilized. |

This research demonstrates that calibrated FE analysis can serve as a powerful forensic tool, allowing for an "inverse analysis" to diagnose hidden internal damage like bond degradation, which is otherwise challenging to quantify. A critical note for practicing engineers is that a single unified approach to modelling corrosion is not possible. The assessment must be tailored to the anticipated failure mode, as an analysis focused only on steel section loss can miss a dangerous shift from ductile to brittle failure.

In the absence of physical test specimens for direct validation, practicing engineers may adopt a multi-stage approach that considers the structural state using available field data. This involves utilizing NDT data, such as UPV and RN, to investigate concrete properties and implementing established bond-decay laws from the literature to represent the degraded steel-concrete interface. Instead of matching a single experimental curve, the assessment can focus on a parametric sensitivity analysis across a range of damage scenarios, such as varying levels of section loss and debonding, to identify critical thresholds of failure where the structure shifts from ductile flexural behaviour to brittle shear failure.

Furthermore, inverse analysis may still be performed to a limited extent by correlating the FE model's predicted global response with available data from Structural Health Monitoring (SHM). When the simulated global stiffness and deformation characteristics are consistent with measured in-service behaviour, confidence in the assumed local degradation parameters is increased. Ultimately, non-linear FE analysis remains a robust and valuable predictive tool for supporting the safe assessment and management of aging infrastructure.

Future research is required to develop advanced 3D models capable of more accurately representing bearing zones and out-of-plane effects. Further studies are also needed to clarify how corrosion level and the precise location of unbonded tendon regions due to poor grouting conditions influence the transition from ductile flexural behaviour to brittle shear failure.

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## Chapter 7: Conclusions and recommendations

### 7.1 Project summary, outcomes, contributions and novelty

This doctoral research was established to provide a definitive quantitative assessment of the residual structural performance of field-aged PC infrastructure. The decommissioning of the former Champlain Bridge in Montreal offered a unique and unprecedented opportunity to extract full-scale roadway infill slabs that had undergone nearly 60 years of natural environmental degradation and aggressive de-icing salt exposure. While the current body of literature is heavily populated by studies involving artificially accelerated corrosion on laboratory-scale specimens, this research bridges a critical gap by providing a comprehensive forensic and structural analysis of members that reached their true end-of-service life. Through a multi-scale experimental and numerical program, this study correlates material-scale deterioration derived from a large database of advanced NDT techniques and 975 individual prestressing wires with the global structural response of 15 full-scale slab panels.

A central outcome of this research is the discovery that global structural capacity in aged post-tensioned members is not governed by average material degradation, but by localized interface conditions. The experimental data revealed that the average mass loss calculated throughout the tendon length, which is a common metric in current bridge inspection protocols, is an insufficient and often misleading predictor of residual strength. Specifically, the significant variance in ultimate load observed at negligible corrosion levels confirms that the state of the tendon-concrete interface is a more sensitive predictor of structural reliability than distributed mass loss. Instead, the research identified that the transition from a desirable, ductile flexural failure mode to a brittle and catastrophic shear failure is primarily triggered by localized bond degradation and grout voids. This finding shifts the focus of structural health monitoring from purely volumetric steel loss to a more nuanced understanding of the tendon-concrete interface. Furthermore, the validation of nonlinear FE models in this study proved that accounting for bond-slip mechanisms is mandatory for accurately predicting the ultimate limit state, as the loss of bond fundamentally alters the internal stress distribution and triggers premature, brittle failure modes.

The test results, summarized in Figure 7-1, reveal two distinct structural performance trends that are independent of the average level of material degradation. The absence of a downward trendline indicates that average mass loss throughout the tendon length is an insufficient predictor of the

ultimate limit state. Specifically, the 'vertical capacity spread' observed at low mass-loss levels, where the ultimate load,  $P_{ult}$ , varies by over 200 kN, highlights the disproportionate influence of bond integrity. The superior performance of the solid-grout specimen (S5A2-S5B2) relative to specimens with comparable corrosion levels provides conclusive evidence that the state of the grout interface, rather than volumetric steel loss, dictates the failure threshold and residual strength of field-aged post-tensioned members.

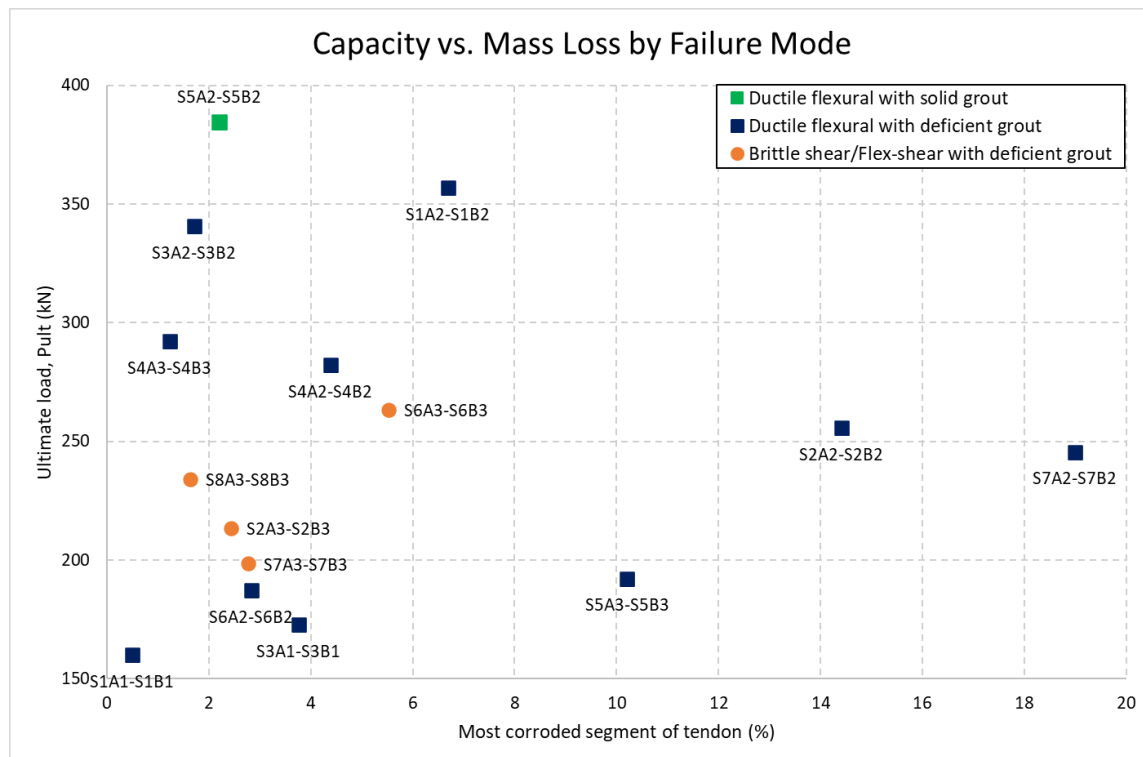


Figure 7-1 Residual capacity vs. average mass loss for all slabs

The key novelties and contributions are summarized as follows:

- A primary novelty of this work is the explicit integration of field-oriented condition assessment techniques with mechanical and structural performance evaluation. Unlike conventional approaches that treat inspection data and structural analysis as isolated workflows, this research establishes direct relationships between corrosion indicators and structural response, enabling a more performance-based assessment framework.
- This study utilizes a rare and valuable dataset derived from full-scale elements extracted from the Champlain Bridge. This captures the combined effects of 60 years of aging,

aggressive chloride exposure, and evolving service demands that cannot be replicated in a laboratory. Notably, these slabs represent an early generation of post-tensioning technology where durability considerations were not yet fully integrated into design practices.

- The research provides new insights into the behaviour of parallel-wire post-tensioning tendons, which are significantly less studied compared to conventional seven-wire strands. This work characterizes the unique mechanical degradation and failure mechanisms specific to these systems.
- A key scientific contribution is the identification of a transition in ductility degradation at approximately 3% mass loss. This threshold was consistently observed across multiple datasets, suggesting a robust and generalizable material behaviour that defines the onset of significant ductility loss in high-strength prestressing steel.
- This research demonstrates that pitting-induced wire failure governs structural performance even at low average mass-loss levels. While industry practitioners often rely on average mass loss due to the time and cost constraints of detailed analysis, this work proves that such simplified metrics can lead to unconservative safety estimates by failing to incorporate localized damage effects.
- The adoption of a logarithmic transformation of corrosion mass-loss data represents a methodological contribution that enables improved interpretation across wide corrosion ranges. This approach facilitates the identification of trends and thresholds that are obscured in conventional linear representations.
- Nonlinear finite element models were developed and validated to explicitly incorporate section loss, material and mechanical degradation, and bond-slip behaviour. This provides a verified tool for practitioners to conduct advanced structural evaluations that move beyond simple section-reduction assumptions.
- The research highlights that bond deterioration is the primary governor of structural behaviour. Critically, the research proves that bond degradation often acts as a more decisive factor in the ultimate limit state than the level of corrosion itself. While steel section loss remains a fundamental parameter for capacity assessment, the loss of bond alters the load-carrying mechanism, either by triggering a premature transition to brittle shear failure or by facilitating the activation of arching action. Consequently, the condition

of the interface was found to influence the ultimate load-carrying capacity and failure mode more significantly than volumetric steel loss alone, highlighting the limitations of traditional assessment models that focus exclusively on mass loss without considering shifts in the internal load-path.

- By combining experimental evidence, statistical analysis, and numerical modelling, this work contributes to a performance-based framework for evaluating aging bridges, moving beyond the simplified and conservative assumptions typically adopted in current design codes such as CSA S6:19.

By demonstrating that grout condition and bond integrity are the primary drivers of structural performance and failure mode, this research challenges current evaluation practices, including those in CSA S6:19, which rely heavily on average material degradation indicators. The findings highlight the need to explicitly incorporate bond-related deficiencies into evaluation methodologies, ensuring that the safety and reliability of aging bridge infrastructure are assessed based on the governing physical mechanisms rather than simplified assumptions.

## 7.2 Conclusions

The integrated experimental and numerical programs led to several fundamental conclusions regarding the assessment of aging post-tensioned infrastructure:

- **Effectiveness of multi-technique NDT for bond and corrosion assessment:** Non-destructive testing provides critical qualitative and semi-quantitative data, but its reliability is contingent upon a multi-technique approach. Surface-level methods, such as half-cell potential and surface resistivity, are effective for identifying active corrosion zones and regions of elevated risk. However, the use of IE proved most significant for structural evaluation. By successfully identifying internal grout voids and poor-quality infill, IE served as a direct indicator of compromised tendon-concrete bond, which was later verified during tendon inspections. This highlights the necessity of using internal imaging techniques to predict structural failure modes that surface corrosion NDTs cannot detect.
- **Mechanical degradation and ductility of prestressing steel:** The mechanical response of the tendons is characterized by an "asymmetric" degradation pattern. While ultimate strength is reduced proportionally to the cross-sectional area loss, the deformation capacity (ductility) is far more sensitive to corrosion. Statistical analysis revealed a consistent

transition in ductility deterioration near a 3% mass-loss threshold, beyond which ultimate strain reduction becomes statistically significant. This threshold remained robust across multiple independent datasets, suggesting that for high-strength steel, the loss of energy-dissipation capacity occurs much earlier than the critical loss of load-bearing strength.

- **Insufficiency of average mass loss as a performance predictor:** A critical finding of this research is that the average mass loss calculated throughout the tendon length is an insufficient predictor of residual capacity. The experimental data revealed a significant "capacity spread" at low maximum mass-loss levels ( $<2\%$ ), where specimens like S1A1-S1B1 and S5A2-S5B2 exhibited ultimate loads of 160.1 kN and 384.3 kN, respectively. This 224 kN disparity, occurring despite nearly identical average corrosion levels, proves that ultimate capacity is highly sensitive to local interface conditions rather than distributed steel area.
- **Methodological value of logarithmic data transformation:** The adoption of a logarithmic transformation for mass-loss data proved essential for interpreting field-extracted results. Given the wide range and stochastic nature of natural corrosion severities, the log-transformation enabled a consistent comparison across disparate datasets and facilitated the identification of the 3% ductility threshold. This methodological approach represents a refined tool for researchers handling the high variability inherent in "naturally" deteriorated infrastructure.
- **The governing role of grout integrity:** Grout integrity was identified as the primary variable governing the ultimate limit state. Specimen S5A2-S5B2, the only specimen with solid grout throughout its length, demonstrated the highest capacity and energy dissipation, serving as the benchmark for ideal bond performance. Conversely, specimens with crumbled or "powdery" grout exhibited a fundamental breakdown in the bond-slip mechanism. This loss of bond leads to:
  - Reduced composite action between the tendon and the concrete matrix.
  - Increased interface slip, preventing the development of full prestressing force across cracks.
  - Altered stress distributions that trigger a premature transition from ductile flexure to brittle shear failures, occurring at significantly lower deflection levels.



- **Numerical validation and the "Ductility Gap":** The nonlinear FE modelling successfully isolated the effects of bond loss. Accurate prediction of experimental load-deflection curves was only possible when imperfect bond and bond-slip mechanisms were explicitly modeled. The modelling confirmed a "ductility gap" in shear-dominated specimens like S8A3-S8B3, where bond deficiencies prevented the tendon from reaching yield strain in the flexure zone, leading to a sudden, shear-controlled structural collapse.

### 7.3 Recommendations for practice

The findings of this research suggest a necessary shift in the industry approach to evaluating aging post-tensioned infrastructure. To move toward a more reliable, performance-based assessment framework, the following recommendations are proposed:

- **Transition from average to localized assessment:** Industry practitioners should move away from only relying solely on average mass loss throughout the tendon length, as this metric can significantly mask localized damage. Assessment protocols should prioritize the identification of "pitting hotspots", specifically near construction joints, sloped deck edges, and drainage locations, where chloride-laden water accumulation facilitates localized section loss and bond degradation.
- **Prioritization of grout integrity as a Key Performance Indicator (KPI):** Grout condition should be treated as a primary variable in condition ratings. Forensic evaluations must recognize that a member with negligible corrosion but significant grout voids (as identified through IE testing) is structurally more vulnerable to brittle failure than a member with moderate, uniform corrosion and sound grout.
- **Implementation of bond-deterioration factors:** When performing residual capacity calculations under codes such as CSA S6:19, engineers should incorporate "bond-penalty factors." If NDT indicates internal grout deficiencies or "powdery" grout, a reduction factor should be applied to the assumed bond strength to account for the potential shift from ductile flexural behaviour to brittle shear failure.
- **A Multi-Technique NDT protocol:** Condition assessments should not rely on a single NDT method. A robust protocol should combine surface-level techniques (Half-cell potential and Surface Resistivity) to map active corrosion risk with internal imaging techniques (IE and UPV) to map the physical state of the grout-tendon interface.

- **Advanced numerical modelling with explicit bond-slip:** For critical or high-risk structures, structural evaluations should utilize nonlinear finite element analysis that explicitly models the bond-slip mechanism. Simplified modelling that assumes a "perfect bond" should be avoided for deteriorated members, as it fails to capture the "ductility gap" and overestimates the energy-dissipation capacity of the system.

#### 7.4 Recommendations for future research

While this research establishes a clear link between grout integrity and structural failure modes, several areas remain for further scientific exploration to refine the predictive capabilities of infrastructure management tools:

- **Validation for 7-Wire helical strands:** This study focused primarily on parallel-wire post-tensioning systems. Future research should expand the experimental database to include 7-wire helical strands, as their geometry may create different grout-penetration challenges and localized bond-stiffness characteristics.
- **Interaction of bond loss and dynamic/fatigue loading:** The current work assessed quasi-static capacity. However, the impact of poor grouting on the fatigue life of tendons is a critical area for study. Localized stress concentrations at bond-anchorage zones are likely magnified under cyclic traffic loading, which may significantly shorten the service life of rehabilitated members before they reach their static ultimate limit state.
- **Development of a standardized "Grout Quality Index":** To bridge the gap between NDT and structural modelling, research is needed to develop a quantitative "Grout Quality Index" derived from NDT signal processing. This index could be directly input into FEA software as a bond-deterioration scalar, allowing for more rapid and objective structural assessments.
- **Advanced discrete fracture modelling:** Future numerical studies should employ the discrete crack propagation tools to better simulate the fracture mechanics at the tendon-concrete interface. This would allow for a higher-resolution understanding of how localized pitting triggers the global transition to shear failure.
- **Real-Time monitoring via Distributed Fiber Optic Sensors (DFOS):** Investigations into the use of DFOS in new or repaired post-tensioned structures could provide a proactive means of monitoring strain redistribution. Research should focus on using DFOS to detect

early-stage debonding or "micro-slip" at the interface long before it manifests as visible surface cracking or efflorescence.

## Appendix A: Non-destructive tests for concrete and tendons

### A.1 Rebound Schmidt hammer

The rebound numbers were recorded at four locations along each of the three transverse strips located at 500 mm, 1,500 mm, and 2,500 mm intervals from the edge of all slab soffits. Table 7-1 to Table 7-8 display the rebound numbers obtained for each slab. A statistical analysis was also performed for all sets of rebound numbers.

Table 7-1. Schmidt hammer results – S1

| Strip    | Zone | Point |    |    |    |    |    |                   |    |    |    | Average      | Average of<br>4 zones | Standard<br>deviation | Coefficient<br>of variation<br>(%) |
|----------|------|-------|----|----|----|----|----|-------------------|----|----|----|--------------|-----------------------|-----------------------|------------------------------------|
|          |      | 1     | 2  | 3  | 4  | 5  | 6  | 7                 | 8  | 9  | 10 |              |                       |                       |                                    |
| <b>1</b> | A    | 56    | 52 | 55 | 57 | 56 | 53 | 56                | 58 | 59 | 53 | 55.5         | 55.45                 | 1.75                  | 3.15                               |
|          | B    | 60    | 58 | 58 | 56 | 58 | 56 | 57                | 55 | 57 | 58 | 57.3         |                       |                       |                                    |
|          | C    | 50    | 56 | 57 | 57 | 54 | 60 | 59                | 57 | 54 | 55 | 55.9         |                       |                       |                                    |
|          | D    | 52    | 53 | 51 | 54 | 53 | 52 | 54                | 55 | 52 | 55 | 53.1         |                       |                       |                                    |
| <b>2</b> | A    | 55    | 58 | 55 | 52 | 55 | 49 | 54                | 58 | 54 | 51 | 54.1         | 54.9                  | 0.88                  | 1.59                               |
|          | B    | 50    | 57 | 51 | 52 | 58 | 54 | 58                | 54 | 56 | 53 | 54.3         |                       |                       |                                    |
|          | C    | 55    | 58 | 57 | 58 | 58 | 52 | 52                | 56 | 60 | 54 | 56           |                       |                       |                                    |
|          | D    | 56    | 57 | 57 | 50 | 56 | 52 | 56                | 58 | 56 | 54 | 55.2         |                       |                       |                                    |
| <b>3</b> | A    | 54    | 56 | 53 | 52 | 52 | 56 | 52                | 56 | 54 | 52 | 53.7         | 54.18                 | 0.59                  | 1.09                               |
|          | B    | 56    | 58 | 53 | 56 | 56 | 56 | 51                | 51 | 50 | 55 | 54.2         |                       |                       |                                    |
|          | C    | 50    | 53 | 56 | 57 | 53 | 56 | 52                | 56 | 55 | 50 | 53.8         |                       |                       |                                    |
|          | D    | 54    | 53 | 58 | 57 | 54 | 55 | 52                | 60 | 53 | 54 | 55           |                       |                       |                                    |
|          |      |       |    |    |    |    |    | <b>All values</b> |    |    |    | <b>54.84</b> |                       | <b>1.20</b>           | <b>2.18</b>                        |

Table 7-2. Schmidt hammer results – S2

| Strip | Zone | Point |    |    |    |    |    |    |            |    |    | Average | Average of 4 zones | Standard deviation | Coefficient of variation (%) |
|-------|------|-------|----|----|----|----|----|----|------------|----|----|---------|--------------------|--------------------|------------------------------|
|       |      | 1     | 2  | 3  | 4  | 5  | 6  | 7  | 8          | 9  | 10 |         |                    |                    |                              |
| 1     | A    | 58    | 59 | 58 | 54 | 56 | 56 | 59 | 54         | 56 | 55 | 56.5    | 55.98              | 0.57               | 1.02                         |
|       | B    | 58    | 52 | 58 | 60 | 52 | 54 | 56 | 58         | 60 | 55 | 56.3    |                    |                    |                              |
|       | C    | 60    | 60 | 56 | 52 | 50 | 56 | 54 | 52         | 58 | 54 | 55.2    |                    |                    |                              |
|       | D    | 54    | 55 | 58 | 57 | 57 | 58 | 55 | 57         | 56 | 52 | 55.9    |                    |                    |                              |
| 2     | A    | 53    | 50 | 53 | 57 | 58 | 54 | 55 | 58         | 60 | 58 | 55.6    | 55.23              | 0.52               | 0.94                         |
|       | B    | 53    | 54 | 58 | 58 | 53 | 60 | 52 | 56         | 52 | 60 | 55.6    |                    |                    |                              |
|       | C    | 58    | 52 | 52 | 56 | 52 | 56 | 52 | 56         | 60 | 58 | 55.2    |                    |                    |                              |
|       | D    | 56    | 52 | 57 | 56 | 52 | 55 | 51 | 60         | 52 | 54 | 54.5    |                    |                    |                              |
| 3     | A    | 50    | 50 | 50 | 56 | 56 | 55 | 58 | 55         | 55 | 53 | 53.8    | 53.13              | 0.99               | 1.87                         |
|       | B    | 51    | 52 | 48 | 54 | 54 | 49 | 52 | 47         | 54 | 56 | 51.7    |                    |                    |                              |
|       | C    | 55    | 55 | 56 | 50 | 54 | 54 | 54 | 54         | 52 | 54 | 53.8    |                    |                    |                              |
|       | D    | 52    | 50 | 54 | 54 | 54 | 54 | 57 | 50         | 52 | 55 | 53.2    |                    |                    |                              |
|       |      |       |    |    |    |    |    |    | All values |    |    | 54.78   |                    | 1.42               | 2.59                         |

Table 7-3. Schmidt hammer results – S3

| Strip | Zone | Point |    |    |    |    |    |    |            |    |    | Average | Average of 4 zones | Standard deviation | Coefficient of variation (%) |
|-------|------|-------|----|----|----|----|----|----|------------|----|----|---------|--------------------|--------------------|------------------------------|
|       |      | 1     | 2  | 3  | 4  | 5  | 6  | 7  | 8          | 9  | 10 |         |                    |                    |                              |
| 1     | A    | 54    | 52 | 50 | 56 | 51 | 52 | 55 | 50         | 56 | 52 | 52.8    | 53.3               | 0.37               | 0.70                         |
|       | B    | 53    | 54 | 52 | 53 | 52 | 56 | 56 | 53         | 50 | 54 | 53.3    |                    |                    |                              |
|       | C    | 54    | 52 | 54 | 57 | 50 | 54 | 54 | 55         | 54 | 50 | 53.4    |                    |                    |                              |
|       | D    | 51    | 56 | 49 | 53 | 58 | 54 | 54 | 54         | 52 | 56 | 53.7    |                    |                    |                              |
| 2     | A    | 54    | 54 | 56 | 58 | 57 | 55 | 58 | 58         | 55 | 55 | 56      | 54.73              | 0.89               | 1.62                         |
|       | B    | 58    | 55 | 53 | 54 | 56 | 55 | 53 | 54         | 54 | 54 | 54.6    |                    |                    |                              |
|       | C    | 54    | 55 | 52 | 53 | 54 | 52 | 56 | 54         | 54 | 56 | 54      |                    |                    |                              |
|       | D    | 58    | 54 | 50 | 54 | 54 | 53 | 58 | 52         | 54 | 56 | 54.3    |                    |                    |                              |
| 3     | A    | 50    | 57 | 55 | 58 | 54 | 56 | 54 | 58         | 58 | 52 | 55.2    | 55.00              | 1.33               | 2.42                         |
|       | B    | 52    | 56 | 55 | 55 | 56 | 54 | 52 | 52         | 57 | 50 | 53.9    |                    |                    |                              |
|       | C    | 50    | 58 | 53 | 54 | 53 | 54 | 55 | 54         | 58 | 52 | 54.1    |                    |                    |                              |
|       | D    | 57    | 55 | 56 | 58 | 57 | 58 | 59 | 58         | 56 | 54 | 56.8    |                    |                    |                              |
|       |      |       |    |    |    |    |    |    | All values |    |    | 54.34   |                    | 1.16               | 2.13                         |

Table 7-4. Schmidt hammer results – S4

| Strip | Zone | Point |    |    |    |    |    |    |    |    |                   | Average      | Average<br>of 4<br>zones | Standard<br>deviation | Coefficient<br>of variation<br>(%) |
|-------|------|-------|----|----|----|----|----|----|----|----|-------------------|--------------|--------------------------|-----------------------|------------------------------------|
|       |      | 1     | 2  | 3  | 4  | 5  | 6  | 7  | 8  | 9  | 10                |              |                          |                       |                                    |
| 1     | A    | 57    | 55 | 53 | 56 | 54 | 53 | 56 | 56 | 59 | 52                | 55.1         | 55.08                    | 1.97                  | 3.57                               |
|       | B    | 54    | 53 | 49 | 57 | 54 | 53 | 53 | 51 | 48 | 51                | 52.3         |                          |                       |                                    |
|       | C    | 58    | 58 | 56 | 57 | 59 | 56 | 54 | 59 | 54 | 56                | 56.7         |                          |                       |                                    |
|       | D    | 56    | 57 | 56 | 55 | 54 | 57 | 58 | 58 | 55 | 56                | 56.2         |                          |                       |                                    |
| 2     | A    | 49    | 55 | 54 | 55 | 59 | 53 | 50 | 54 | 54 | 50                | 53.3         | 54.55                    | 2.33                  | 4.26                               |
|       | B    | 51    | 52 | 52 | 50 | 50 | 55 | 54 | 48 | 54 | 53                | 51.9         |                          |                       |                                    |
|       | C    | 59    | 55 | 57 | 55 | 58 | 52 | 58 | 56 | 58 | 56                | 56.4         |                          |                       |                                    |
|       | D    | 57    | 60 | 55 | 55 | 59 | 56 | 59 | 54 | 57 | 54                | 56.6         |                          |                       |                                    |
| 3     | A    | 58    | 54 | 54 | 55 | 54 | 54 | 55 | 53 | 49 | 56                | 54.2         | 56.13                    | 1.75                  | 3.12                               |
|       | B    | 54    | 53 | 58 | 56 | 57 | 59 | 60 | 61 | 55 | 57                | 57           |                          |                       |                                    |
|       | C    | 56    | 50 | 54 | 58 | 54 | 56 | 57 | 53 | 58 | 56                | 55.2         |                          |                       |                                    |
|       | D    | 60    | 60 | 62 | 54 | 56 | 61 | 54 | 58 | 57 | 59                | 58.1         |                          |                       |                                    |
|       |      |       |    |    |    |    |    |    |    |    | <b>All values</b> | <b>55.25</b> |                          | <b>1.96</b>           | <b>3.55</b>                        |

Table 7-5. Schmidt hammer results – S5

| Strip | Zone | Point |    |    |    |    |    |    |    |    |                   | Average      | Average<br>of 4<br>zones | Standard<br>deviation | Coefficient<br>of variation<br>(%) |
|-------|------|-------|----|----|----|----|----|----|----|----|-------------------|--------------|--------------------------|-----------------------|------------------------------------|
|       |      | 1     | 2  | 3  | 4  | 5  | 6  | 7  | 8  | 9  | 10                |              |                          |                       |                                    |
| 1     | A    | 46    | 55 | 48 | 53 | 53 | 53 | 46 | 47 | 50 | 51                | 50.2         | 52.8                     | 2.18                  | 4.13%                              |
|       | B    | 50    | 54 | 56 | 56 | 58 | 55 | 54 | 56 | 58 | 54                | 55.1         |                          |                       |                                    |
|       | C    | 60    | 48 | 49 | 56 | 52 | 50 | 55 | 58 | 54 | 58                | 54           |                          |                       |                                    |
|       | D    | 54    | 50 | 55 | 53 | 47 | 46 | 54 | 56 | 50 | 54                | 51.9         |                          |                       |                                    |
| 2     | A    | 56    | 58 | 55 | 57 | 56 | 55 | 59 | 58 | 53 | 57                | 56.4         | 55.45                    | 0.68                  | 1.23%                              |
|       | B    | 55    | 57 | 54 | 58 | 56 | 50 | 56 | 54 | 58 | 54                | 55.2         |                          |                       |                                    |
|       | C    | 56    | 56 | 57 | 56 | 58 | 58 | 56 | 52 | 56 | 50                | 55.5         |                          |                       |                                    |
|       | D    | 54    | 56 | 52 | 50 | 53 | 54 | 57 | 58 | 59 | 55                | 54.8         |                          |                       |                                    |
| 3     | A    | 59    | 55 | 62 | 54 | 54 | 60 | 58 | 53 | 59 | 56                | 57           | 56.65                    | 1.55                  | 2.74%                              |
|       | B    | 60    | 58 | 56 | 60 | 58 | 60 | 60 | 60 | 55 | 55                | 58.2         |                          |                       |                                    |
|       | C    | 59    | 56 | 54 | 55 | 56 | 58 | 55 | 60 | 57 | 59                | 56.9         |                          |                       |                                    |
|       | D    | 56    | 57 | 55 | 55 | 52 | 53 | 52 | 51 | 58 | 56                | 54.5         |                          |                       |                                    |
|       |      |       |    |    |    |    |    |    |    |    | <b>All values</b> | <b>54.98</b> |                          | <b>2.22</b>           | <b>4.03</b>                        |

Table 7-6. Schmidt hammer results – S6

| Strip    | Zone | Point |    |    |    |    |    |    |    |                   |    | Average      | Average<br>of 4<br>zones | Standard<br>deviation | Coefficient<br>of variation<br>(%) |
|----------|------|-------|----|----|----|----|----|----|----|-------------------|----|--------------|--------------------------|-----------------------|------------------------------------|
|          |      | 1     | 2  | 3  | 4  | 5  | 6  | 7  | 8  | 9                 | 10 |              |                          |                       |                                    |
| <b>1</b> | A    | 58    | 53 | 57 | 56 | 54 | 58 | 55 | 57 | 56                | 53 | 55.7         | 57.55                    | 1.26                  | 2.19                               |
|          | B    | 56    | 58 | 59 | 57 | 60 | 60 | 59 | 61 | 58                | 57 | 58.5         |                          |                       |                                    |
|          | C    | 58    | 57 | 56 | 57 | 60 | 61 | 60 | 58 | 60                | 54 | 58.1         |                          |                       |                                    |
|          | D    | 60    | 55 | 58 | 61 | 58 | 56 | 56 | 55 | 59                | 61 | 57.9         |                          |                       |                                    |
| <b>2</b> | A    | 60    | 56 | 57 | 60 | 56 | 59 | 58 | 60 | 58                | 57 | 58.1         | 58.05                    | 0.39                  | 0.67                               |
|          | B    | 56    | 60 | 59 | 57 | 59 | 53 | 56 | 61 | 56                | 58 | 57.5         |                          |                       |                                    |
|          | C    | 61    | 59 | 57 | 58 | 61 | 58 | 57 | 56 | 59                | 58 | 58.4         |                          |                       |                                    |
|          | D    | 56    | 61 | 58 | 57 | 60 | 60 | 58 | 58 | 58                | 56 | 58.2         |                          |                       |                                    |
| <b>3</b> | A    | 56    | 60 | 55 | 56 | 59 | 55 | 59 | 56 | 54                | 55 | 56.5         | 55.9                     | 0.83                  | 1.48                               |
|          | B    | 60    | 61 | 52 | 52 | 58 | 50 | 54 | 53 | 57                | 57 | 55.4         |                          |                       |                                    |
|          | C    | 59    | 57 | 57 | 59 | 58 | 56 | 52 | 56 | 57                | 56 | 56.7         |                          |                       |                                    |
|          | D    | 53    | 54 | 50 | 57 | 57 | 56 | 59 | 54 | 54                | 56 | 55           |                          |                       |                                    |
|          |      |       |    |    |    |    |    |    |    | <b>All values</b> |    | <b>57.17</b> |                          | <b>1.26</b>           | <b>2.20</b>                        |

Table 7-7. Schmidt hammer results – S7

| Strip    | Zone | Point |    |    |    |    |    |    |    |                   |    | Average      | Average<br>of 4<br>zones | Standard<br>deviation | Coefficient<br>of variation<br>(%) |
|----------|------|-------|----|----|----|----|----|----|----|-------------------|----|--------------|--------------------------|-----------------------|------------------------------------|
|          |      | 1     | 2  | 3  | 4  | 5  | 6  | 7  | 8  | 9                 | 10 |              |                          |                       |                                    |
| <b>1</b> | A    | 49    | 54 | 57 | 50 | 54 | 56 | 54 | 59 | 58                | 53 | 54.4         | 55.83                    | 2.32                  | 4.16                               |
|          | B    | 54    | 55 | 52 | 53 | 53 | 59 | 50 | 55 | 51                | 51 | 53.3         |                          |                       |                                    |
|          | C    | 59    | 58 | 57 | 58 | 59 | 57 | 56 | 58 | 58                | 58 | 57.8         |                          |                       |                                    |
|          | D    | 59    | 58 | 57 | 58 | 59 | 57 | 56 | 58 | 58                | 58 | 57.8         |                          |                       |                                    |
| <b>2</b> | A    | 56    | 61 | 56 | 55 | 57 | 58 | 60 | 56 | 56                | 56 | 57.1         | 54.45                    | 2.64                  | 4.84                               |
|          | B    | 57    | 52 | 57 | 49 | 58 | 56 | 50 | 56 | 58                | 57 | 55           |                          |                       |                                    |
|          | C    | 56    | 53 | 55 | 55 | 51 | 54 | 57 | 56 | 56                | 56 | 54.9         |                          |                       |                                    |
|          | D    | 45    | 53 | 56 | 48 | 50 | 53 | 56 | 47 | 53                | 47 | 50.8         |                          |                       |                                    |
| <b>3</b> | A    | 55    | 49 | 55 | 57 | 55 | 53 | 56 | 56 | 48                | 54 | 53.8         | 55.90                    | 1.69                  | 3.02                               |
|          | B    | 56    | 57 | 56 | 58 | 58 | 52 | 57 | 50 | 56                | 55 | 55.5         |                          |                       |                                    |
|          | C    | 60    | 58 | 60 | 59 | 59 | 56 | 57 | 54 | 57                | 58 | 57.8         |                          |                       |                                    |
|          | D    | 59    | 54 | 58 | 54 | 52 | 58 | 62 | 57 | 57                | 54 | 56.5         |                          |                       |                                    |
|          |      |       |    |    |    |    |    |    |    | <b>All values</b> |    | <b>55.39</b> |                          | <b>2.15</b>           | <b>3.89</b>                        |

Table 7-8. Schmidt hammer results – S8

| Strip | Zone | Point |    |    |    |    |    |    |            |    |    | Average | Average of 4 zones | Standard deviation | Coefficient of variation (%) |
|-------|------|-------|----|----|----|----|----|----|------------|----|----|---------|--------------------|--------------------|------------------------------|
|       |      | 1     | 2  | 3  | 4  | 5  | 6  | 7  | 8          | 9  | 10 |         |                    |                    |                              |
| 1     | A    | 60    | 56 | 59 | 54 | 55 | 54 | 56 | 54         | 54 | 62 | 56.4    | 54.83              | 1.36               | 2.48                         |
|       | B    | 54    | 59 | 51 | 55 | 53 | 50 | 55 | 53         | 54 | 58 | 54.2    |                    |                    |                              |
|       | C    | 54    | 53 | 55 | 50 | 50 | 57 | 52 | 53         | 54 | 55 | 53.3    |                    |                    |                              |
|       | D    | 58    | 58 | 60 | 54 | 52 | 53 | 56 | 59         | 51 | 53 | 55.4    |                    |                    |                              |
| 2     | A    | 53    | 55 | 52 | 58 | 60 | 52 | 56 | 52         | 54 | 60 | 55.2    | 56.23              | 1.08               | 1.92                         |
|       | B    | 53    | 53 | 53 | 54 | 56 | 56 | 57 | 58         | 59 | 55 | 55.4    |                    |                    |                              |
|       | C    | 55    | 58 | 57 | 60 | 55 | 55 | 60 | 56         | 58 | 56 | 57      |                    |                    |                              |
|       | D    | 61    | 56 | 56 | 60 | 53 | 57 | 59 | 57         | 55 | 59 | 57.3    |                    |                    |                              |
| 3     | A    | 54    | 53 | 55 | 56 | 52 | 50 | 55 | 56         | 56 | 55 | 54.2    | 56.73              | 1.79               | 3.16                         |
|       | B    | 60    | 58 | 57 | 57 | 58 | 55 | 56 | 58         | 59 | 58 | 57.6    |                    |                    |                              |
|       | C    | 60    | 59 | 61 | 58 | 61 | 57 | 54 | 58         | 55 | 60 | 58.3    |                    |                    |                              |
|       | D    | 54    | 58 | 55 | 59 | 56 | 57 | 56 | 60         | 56 | 57 | 56.8    |                    |                    |                              |
|       |      |       |    |    |    |    |    |    | All values |    |    | 55.93   |                    | 1.55               | 2.77                         |

## A.2 Ultrasonic pulse velocity

Ultrasonic pulse velocity measurements were recorded along 15-800 mm longitudinal strips that are bounded by the steel reinforcement and transverse prestressing tendons. Figure 7-2 to Figure 7-9 present contour maps for the ultrasonic pulse velocities of all slabs.

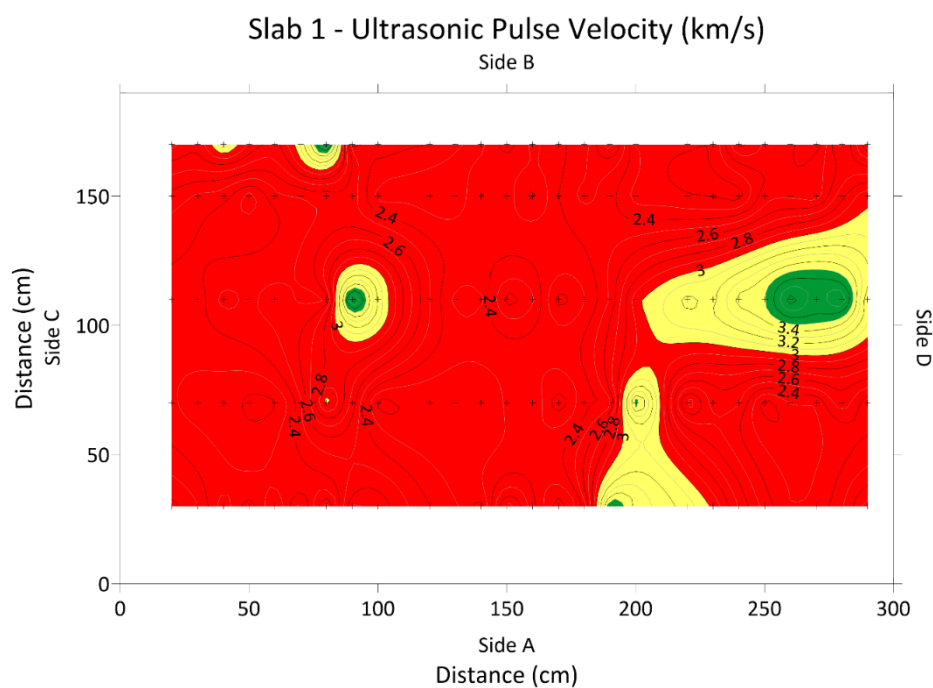




Figure 7-2. UPV mapping for slab 1 (47.6)

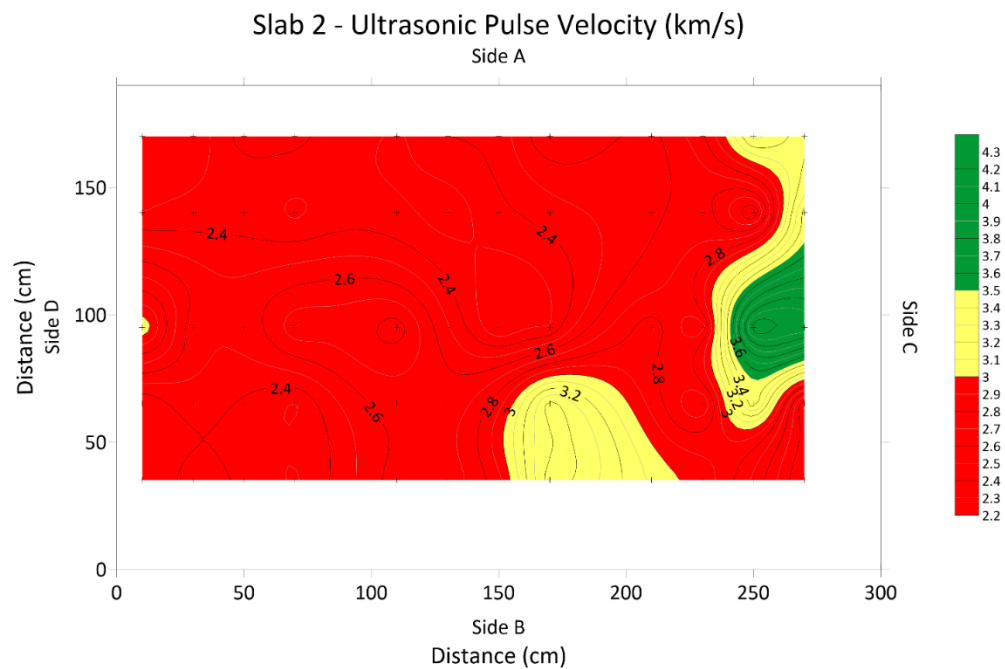


Figure 7-3. UPV mapping for slab 2 (47.2)

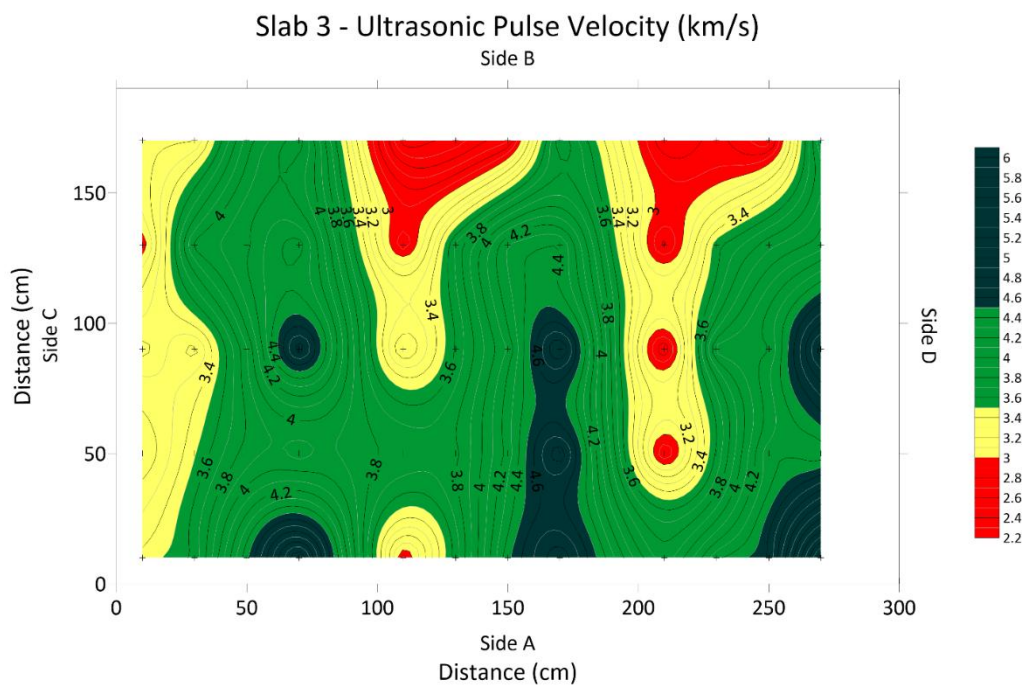


Figure 7-4. UPV mapping for slab 3 (47.5)

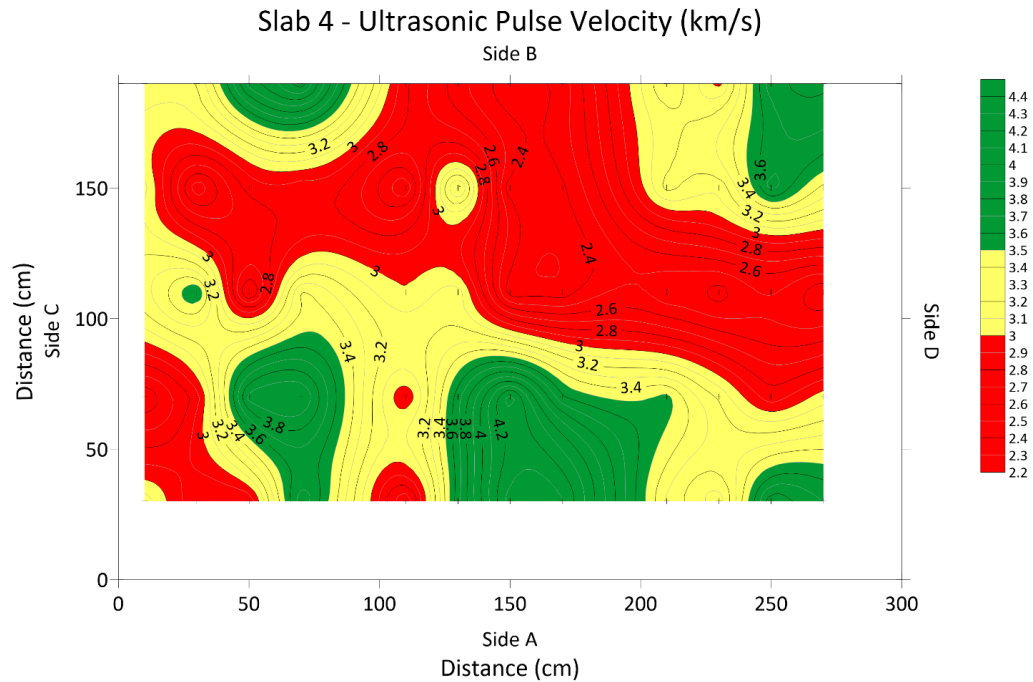


Figure 7-5. UPV mapping for slab 4 (47.1)

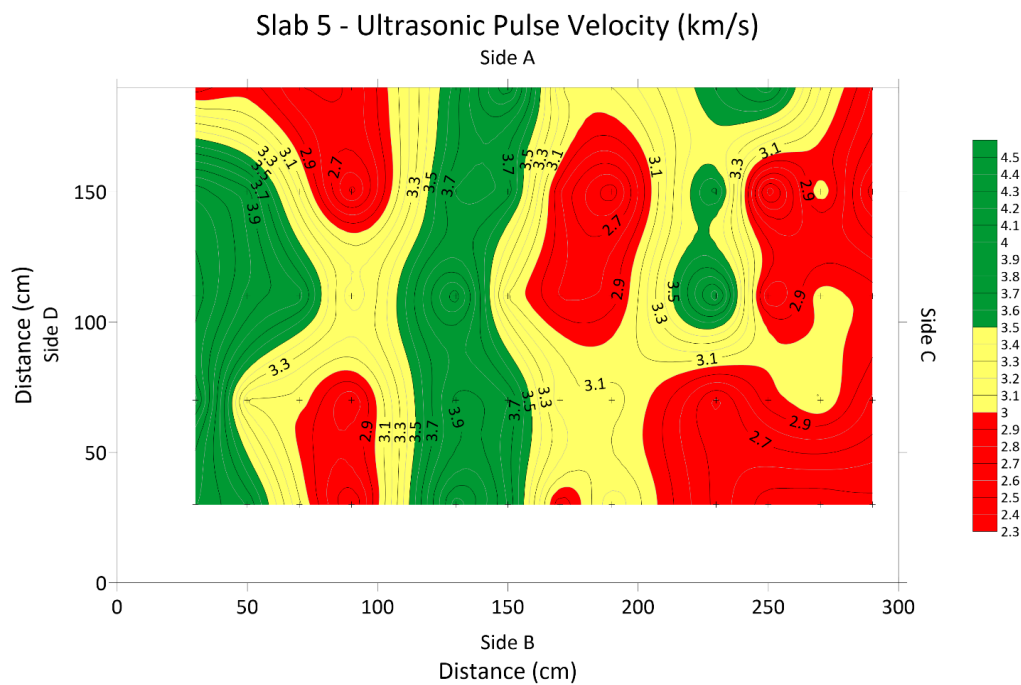


Figure 7-6. UPV mapping for slab 5 (47.3)

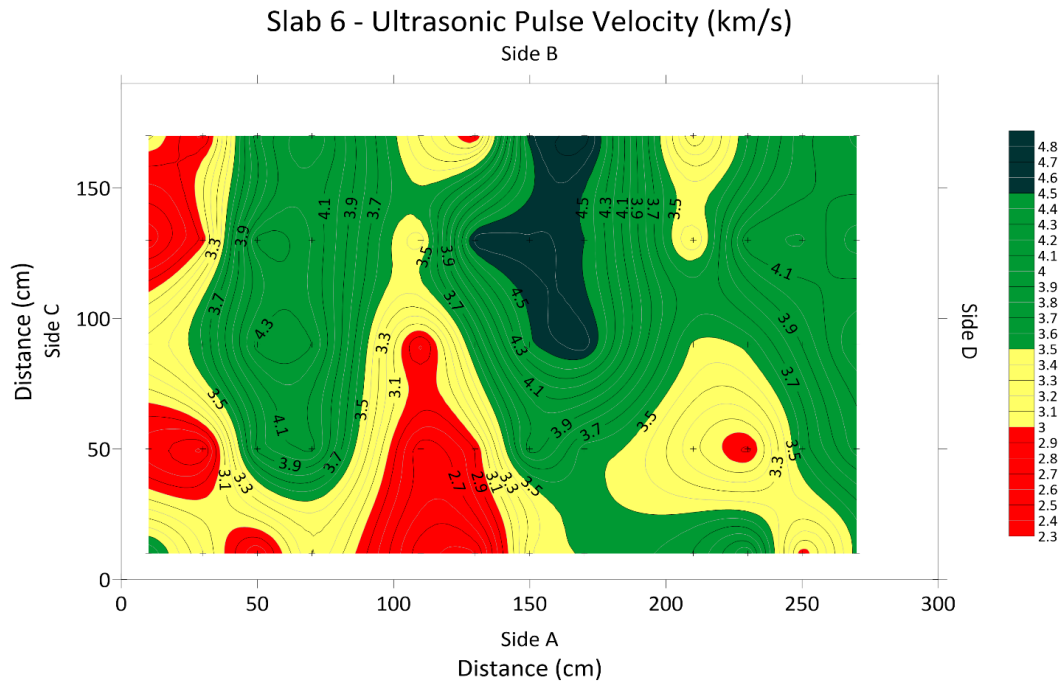


Figure 7-7. UPV mapping for slab 6 (47.4)

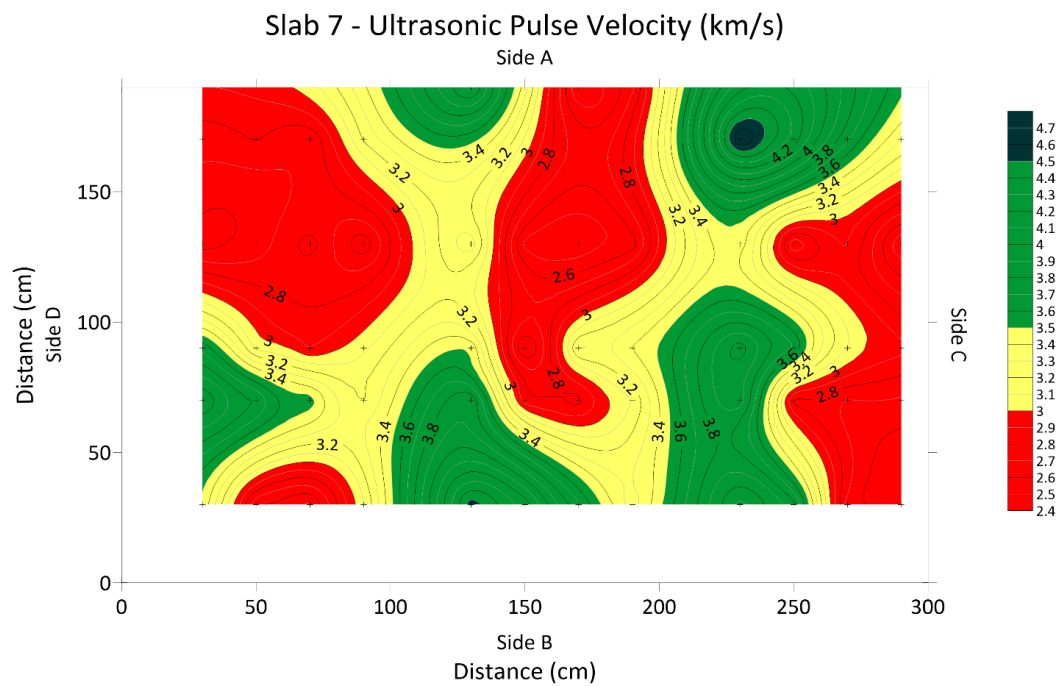


Figure 7-8. UPV mapping for slab 7 (47.7)

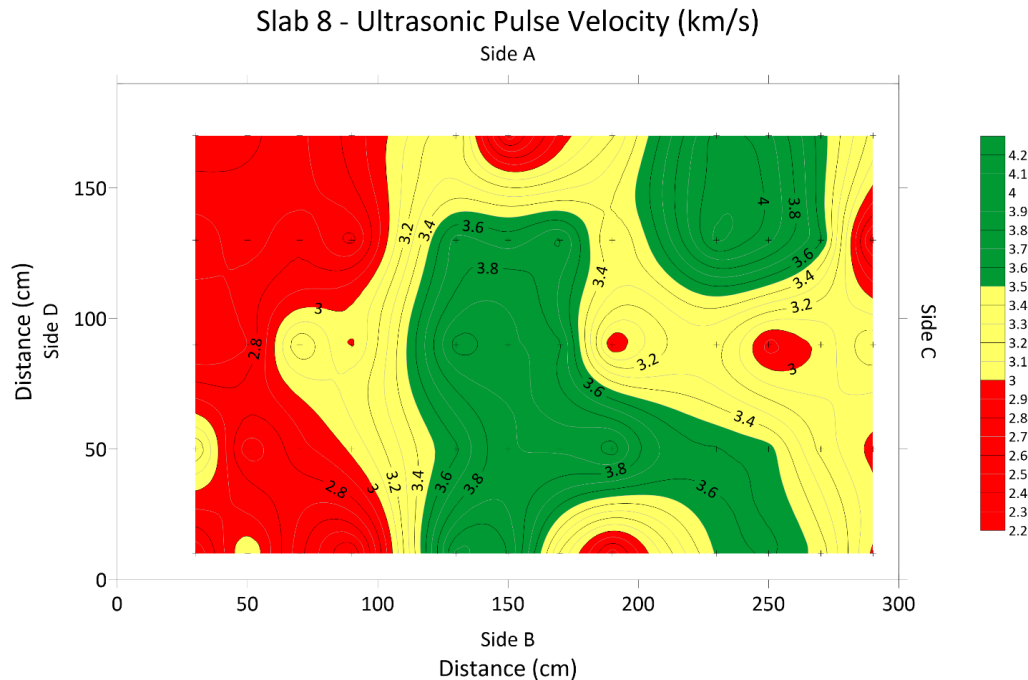


Figure 7-9. UPV mapping for slab 8 (47.8)

### A.3 Combined sonic rebound method

Concrete compressive strength values for each slab were estimated using 17 formulations using the SonReb method. Average rebound numbers for each slab were used as input, as the coefficient of variation for the obtained rebound numbers were relatively low, as reported in section A.1 of this report. Localized values for UPV measurements were also used as input parameters to obtain localized estimates of concrete compressive strengths. The reported values in Table 7-9 and Table 7-10 highlight the minimum, maximum, median and average compressive strength estimates for each slab by implementing each of the provided formulations.

Table 7-9. Estimated concrete compressive strengths (MPa) using the SonReb method

| Slab number | Value   | Formulations by authors (Year) |                         |                               |                         |                      |                 |              |                        |
|-------------|---------|--------------------------------|-------------------------|-------------------------------|-------------------------|----------------------|-----------------|--------------|------------------------|
|             |         | Bellander (1979)               | Meynink, Samarin (1979) | Giacchetti, Lacquaniti (1980) | Bocca, Cianfrone (1983) | Samarin, Dhir (1984) | Gasparik (1992) | RILEM (1993) | Di Leo, Pascale (1994) |
| 1 (47.6)    | Minimum | 156.50                         | 44.15                   | 10.72                         | 11.20                   | 31.46                | 19.87           | 12.92        | 13.13                  |
|             | Maximum | 182.26                         | 44.80                   | 57.66                         | 55.96                   | 53.33                | 65.78           | 69.46        | 63.93                  |

|          |                |               |              |              |              |              |              |              |              |
|----------|----------------|---------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|
|          | <i>Median</i>  | <i>162.15</i> | <i>44.26</i> | <i>17.21</i> | <i>17.60</i> | <i>33.36</i> | <i>27.82</i> | <i>20.73</i> | <i>20.49</i> |
|          | <i>Average</i> | <i>163.78</i> | <i>44.30</i> | <i>20.57</i> | <i>20.80</i> | <i>34.97</i> | <i>30.98</i> | <i>24.79</i> | <i>24.11</i> |
| 2 (47.2) | Minimum        | 157.99        | 44.10        | 12.90        | 13.36        | 32.00        | 22.66        | 15.54        | 15.64        |
|          | Maximum        | 186.94        | 44.90        | 73.39        | 70.49        | 64.00        | 78.08        | 88.41        | 80.24        |
|          | <i>Median</i>  | <i>162.19</i> | <i>44.19</i> | <i>18.05</i> | <i>18.43</i> | <i>33.60</i> | <i>28.78</i> | <i>21.75</i> | <i>21.44</i> |
|          | <i>Average</i> | <i>164.15</i> | <i>44.24</i> | <i>22.20</i> | <i>22.35</i> | <i>35.67</i> | <i>32.61</i> | <i>26.74</i> | <i>25.89</i> |
| 3 (47.5) | Minimum        | 155.90        | 43.60        | 14.90        | 15.34        | 32.31        | 25.05        | 17.95        | 17.40        |
|          | Maximum        | 204.95        | 45.30        | 164.61       | 152.68       | 150.45       | 138.45       | 198.31       | 171.91       |
|          | <i>Median</i>  | <i>176.71</i> | <i>44.14</i> | <i>54.33</i> | <i>52.87</i> | <i>51.31</i> | <i>62.91</i> | <i>65.44</i> | <i>60.59</i> |
|          | <i>Average</i> | <i>175.99</i> | <i>44.16</i> | <i>56.92</i> | <i>54.99</i> | <i>55.47</i> | <i>63.31</i> | <i>68.57</i> | <i>62.85</i> |
| 4 (47.1) | Minimum        | 163.73        | 44.72        | 14.58        | 15.01        | 32.80        | 24.77        | 17.56        | 17.50        |
|          | Maximum        | 192.01        | 45.51        | 76.40        | 73.24        | 65.88        | 80.52        | 92.04        | 83.15        |
|          | <i>Median</i>  | <i>175.20</i> | <i>44.98</i> | <i>32.52</i> | <i>32.35</i> | <i>39.63</i> | <i>43.84</i> | <i>39.17</i> | <i>37.22</i> |
|          | <i>Average</i> | <i>175.27</i> | <i>45.00</i> | <i>35.08</i> | <i>34.62</i> | <i>41.87</i> | <i>45.19</i> | <i>42.26</i> | <i>39.73</i> |
| 5 (47.3) | Minimum        | 161.23        | 44.38        | 14.56        | 15.00        | 32.61        | 24.72        | 17.54        | 17.50        |
|          | Maximum        | 190.84        | 45.22        | 80.57        | 77.06        | 69.15        | 83.52        | 97.06        | 87.52        |
|          | <i>Median</i>  | <i>172.19</i> | <i>44.63</i> | <i>31.44</i> | <i>31.33</i> | <i>39.04</i> | <i>42.75</i> | <i>37.87</i> | <i>36.11</i> |
|          | <i>Average</i> | <i>174.06</i> | <i>44.70</i> | <i>37.61</i> | <i>37.01</i> | <i>43.09</i> | <i>47.48</i> | <i>45.31</i> | <i>42.48</i> |
| 6 (47.4) | Minimum        | 183.45        | 47.12        | 16.70        | 17.08        | 34.66        | 27.52        | 20.12        | 19.71        |
|          | Maximum        | 216.43        | 48.11        | 101.87       | 96.34        | 83.36        | 99.65        | 122.72       | 108.03       |
|          | <i>Median</i>  | <i>200.33</i> | <i>47.54</i> | <i>49.13</i> | <i>47.96</i> | <i>48.35</i> | <i>59.31</i> | <i>59.18</i> | <i>54.40</i> |
|          | <i>Average</i> | <i>200.41</i> | <i>47.57</i> | <i>52.70</i> | <i>51.07</i> | <i>51.83</i> | <i>61.03</i> | <i>63.49</i> | <i>57.78</i> |
| 7 (47.7) | Minimum        | 166.16        | 44.91        | 15.98        | 16.39        | 33.31        | 26.46        | 19.25        | 19.06        |
|          | Maximum        | 198.36        | 45.88        | 94.51        | 89.75        | 79.60        | 93.73        | 113.85       | 101.49       |
|          | <i>Median</i>  | <i>176.08</i> | <i>45.14</i> | <i>31.75</i> | <i>31.62</i> | <i>39.34</i> | <i>43.13</i> | <i>38.25</i> | <i>36.37</i> |
|          | <i>Average</i> | <i>177.56</i> | <i>45.20</i> | <i>37.38</i> | <i>36.77</i> | <i>43.22</i> | <i>47.25</i> | <i>45.03</i> | <i>42.13</i> |
| 8 (47.8) | Minimum        | 169.83        | 45.55        | 14.45        | 14.89        | 33.20        | 24.70        | 17.41        | 17.31        |
|          | Maximum        | 195.38        | 46.24        | 67.90        | 65.39        | 59.66        | 74.26        | 81.79        | 74.17        |
|          | <i>Median</i>  | <i>182.96</i> | <i>45.85</i> | <i>35.81</i> | <i>35.47</i> | <i>41.40</i> | <i>47.11</i> | <i>43.14</i> | <i>40.64</i> |
|          | <i>Average</i> | <i>183.28</i> | <i>45.88</i> | <i>38.46</i> | <i>37.84</i> | <i>43.49</i> | <i>48.71</i> | <i>46.33</i> | <i>43.25</i> |

Table 7-10. Estimated concrete compressive strengths (MPa) using the SonReb method (continued)

| Slab number | Value   | Formulations by authors (Year) |                   |               |                            |                      |                         |                        |                      |                      |
|-------------|---------|--------------------------------|-------------------|---------------|----------------------------|----------------------|-------------------------|------------------------|----------------------|----------------------|
|             |         | Arioglu, Koyluglu (1996)       | Ramyar Kol (1996) | Kheder (1996) | Benoncini, Formichi (2003) | Caiaro et al. (2003) | Del Monte et al. (2004) | Menditto et al. (2004) | Faella et al. (2009) | Faella et al. (2009) |
| 1 (47.6)    | Minimum | 17.10                          | 54.84             | 44.41         | 8.54                       | 10.91                | 19.89                   | 44.55                  | 10.53                | 10.58                |
|             | Maximum | 83.14                          | 64.30             | 58.48         | 41.08                      | 50.21                | 59.38                   | 75.17                  | 46.26                | 42.92                |
|             | Median  | 26.68                          | 56.91             | 47.98         | 11.37                      | 16.75                | 27.06                   | 51.61                  | 15.96                | 17.67                |
|             | Average | 31.38                          | 57.51             | 48.81         | 13.76                      | 19.54                | 29.74                   | 53.51                  | 18.49                | 19.72                |
| 2 (47.2)    | Minimum | 20.34                          | 55.51             | 45.72         | 9.42                       | 12.92                | 22.43                   | 47.11                  | 12.40                | 13.2                 |
|             | Maximum | 104.24                         | 66.14             | 60.77         | 56.96                      | 62.61                | 69.45                   | 80.88                  | 57.25                | 49.55                |
|             | Median  | 27.89                          | 57.05             | 48.31         | 11.80                      | 17.53                | 27.90                   | 52.29                  | 16.66                | 18.47                |
|             | Average | 33.66                          | 57.77             | 49.29         | 14.88                      | 20.95                | 31.15                   | 54.56                  | 19.75                | 20.93                |
| 3 (47.5)    | Minimum | 23.19                          | 55.53             | 46.48         | 10.33                      | 14.88                | 24.59                   | 48.70                  | 14.15                | 15.41                |
|             | Maximum | 221.82                         | 73.54             | 65.86         | 184.44                     | 131.73               | 117.21                  | 102.78                 | 117.17               | 77.00                |
|             | Median  | 78.24                          | 63.18             | 57.44         | 38.34                      | 48.16                | 57.01                   | 72.82                  | 44.18                | 41.55                |
|             | Average | 81.16                          | 62.91             | 56.82         | 44.47                      | 49.70                | 57.04                   | 71.78                  | 45.40                | 40.64                |
| 4 (47.1)    | Minimum | 22.91                          | 56.72             | 47.00         | 10.10                      | 14.27                | 24.33                   | 49.54                  | 13.72                | 15.01                |
|             | Maximum | 108.74                         | 67.10             | 61.64         | 59.68                      | 64.19                | 71.42                   | 82.93                  | 58.95                | 50.52                |
|             | Median  | 48.71                          | 60.93             | 53.60         | 20.35                      | 29.56                | 40.99                   | 63.58                  | 27.80                | 29.41                |
|             | Average | 51.98                          | 60.96             | 53.40         | 23.70                      | 31.37                | 41.91                   | 63.49                  | 29.36                | 29.50                |
| 5 (47.3)    | Minimum | 22.83                          | 56.33             | 46.79         | 10.12                      | 14.35                | 24.29                   | 49.17                  | 13.76                | 15.01                |
|             | Maximum | 114.01                         | 67.20             | 61.90         | 64.59                      | 67.81                | 73.85                   | 83.70                  | 61.99                | 52.19                |
|             | Median  | 47.07                          | 60.35             | 53.07         | 19.70                      | 28.86                | 40.05                   | 62.47                  | 27.08                | 28.77                |
|             | Average | 55.37                          | 61.04             | 53.82         | 25.74                      | 33.65                | 43.89                   | 64.50                  | 31.34                | 31.13                |
| 6 (47.4)    | Minimum | 26.50                          | 60.06             | 49.54         | 10.88                      | 15.42                | 26.77                   | 54.30                  | 15.10                | 17.02                |
|             | Maximum | 145.03                         | 72.17             | 66.59         | 86.38                      | 79.62                | 86.75                   | 95.27                  | 74.14                | 58.44                |
|             | Median  | 73.07                          | 66.26             | 59.10         | 32.11                      | 41.07                | 53.99                   | 75.94                  | 39.03                | 38.22                |
|             | Average | 77.61                          | 66.29             | 58.89         | 37.50                      | 43.40                | 55.18                   | 75.83                  | 41.07                | 38.32                |
| 7 (47.7)    | Minimum | 25.01                          | 57.34             | 47.82         | 10.73                      | 47.82                | 15.46                   | 25.84                  | 51.17                | 14.85                |
|             | Maximum | 132.98                         | 69.16             | 63.97         | 80.27                      | 77.59                | 82.05                   | 88.92                  | 70.95                | 56.87                |
|             | Median  | 47.70                          | 60.98             | 53.51         | 19.79                      | 28.83                | 40.38                   | 63.35                  | 27.17                | 28.89                |
|             | Average | 55.23                          | 61.53             | 54.05         | 25.61                      | 33.09                | 43.67                   | 64.93                  | 30.97                | 30.75                |
| 8 (47.8)    | Minimum | 22.88                          | 57.64             | 47.45         | 9.99                       | 13.93                | 24.26                   | 50.29                  | 13.51                | 14.85                |
|             | Maximum | 97.93                          | 67.02             | 61.11         | 50.12                      | 56.74                | 66.32                   | 81.35                  | 56.69                | 46.93                |
|             | Median  | 53.68                          | 62.46             | 55.04         | 22.43                      | 31.75                | 43.76                   | 66.68                  | 30.01                | 31.33                |
|             | Average | 57.13                          | 62.58             | 55.03         | 25.60                      | 33.63                | 44.96                   | 66.96                  | 31.67                | 31.73                |

The empirical formulation developed by Meynink and Samarin (1979) was found to yield concrete compressive strength values that fall within an acceptable range (9 – 21%) of the reported concrete design strengths of 40 MPa. Figure 7-10 to Figure 7-17 display contour maps showing the

estimated compressive strength values obtained using the SonReb method and by implementing the formulations developed by Meynink and Samarin (1979).

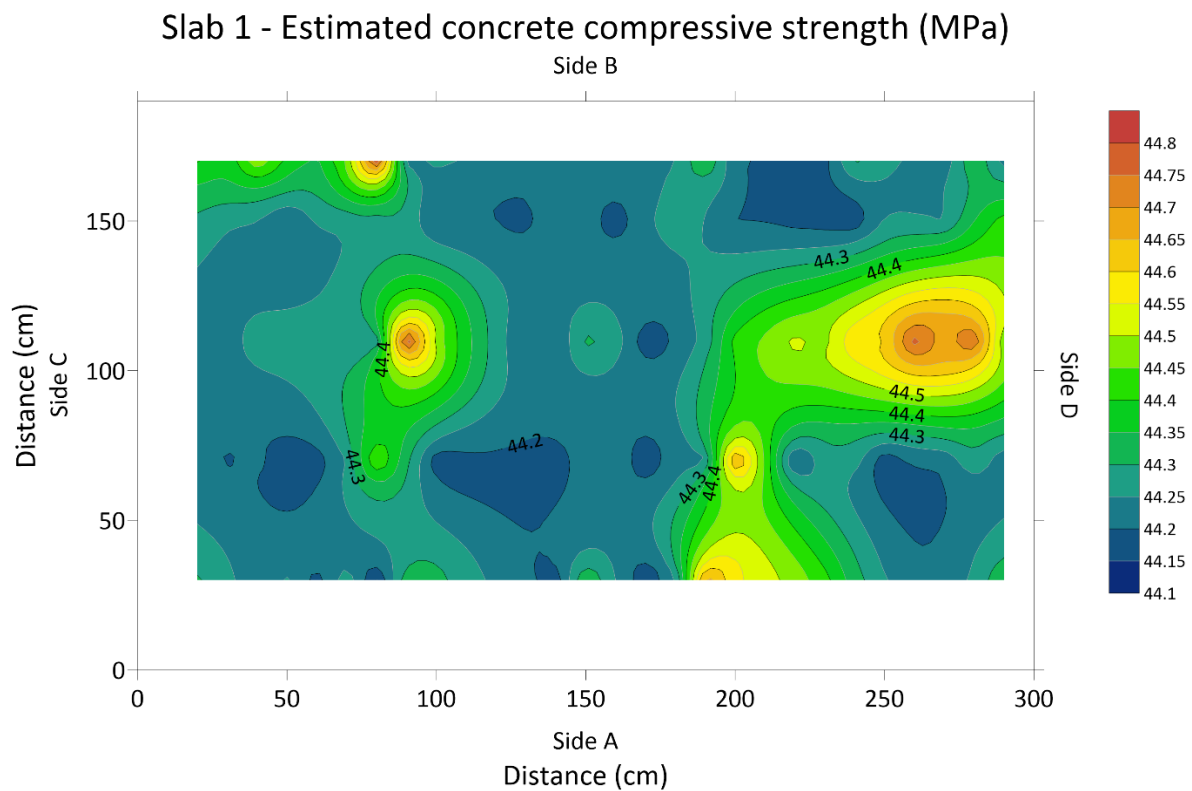


Figure 7-10. Estimated concrete compressive strength for slab 1 (47.6) using SonReb Method

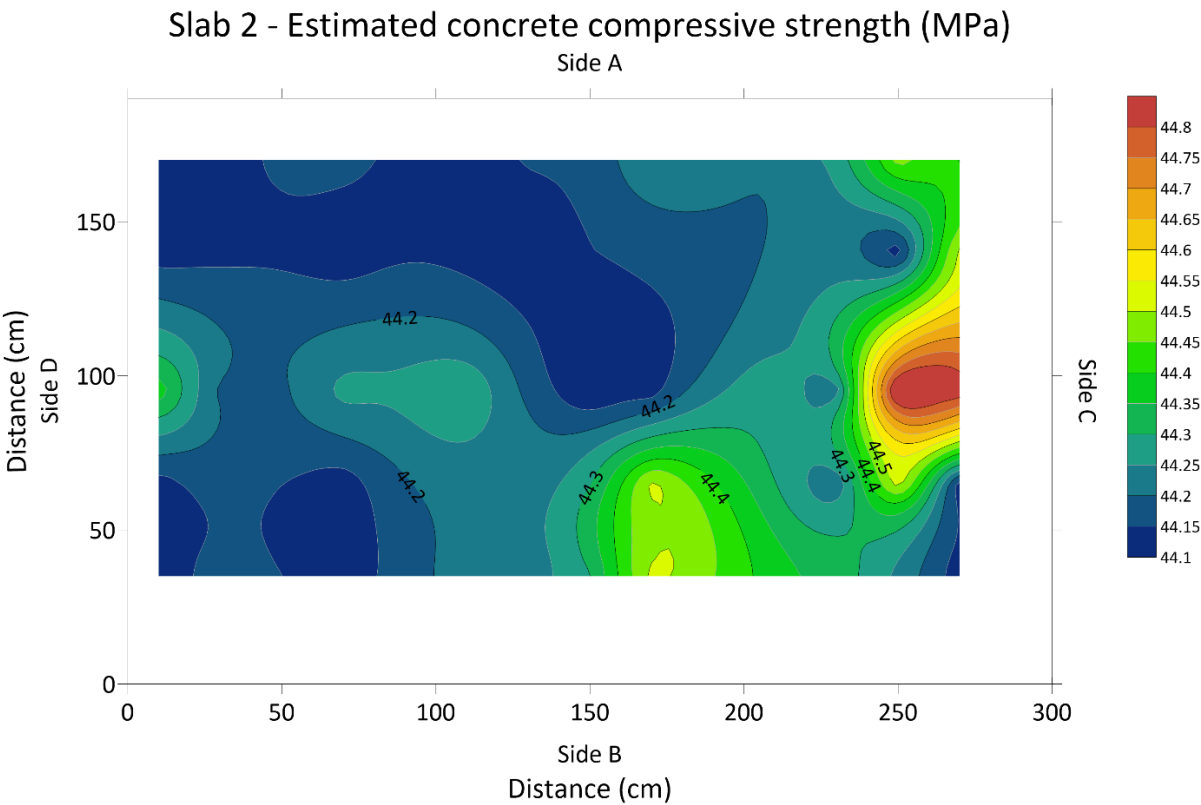


Figure 7-11. Estimated concrete compressive strength for slab 2 (47.2) using SonReb Method

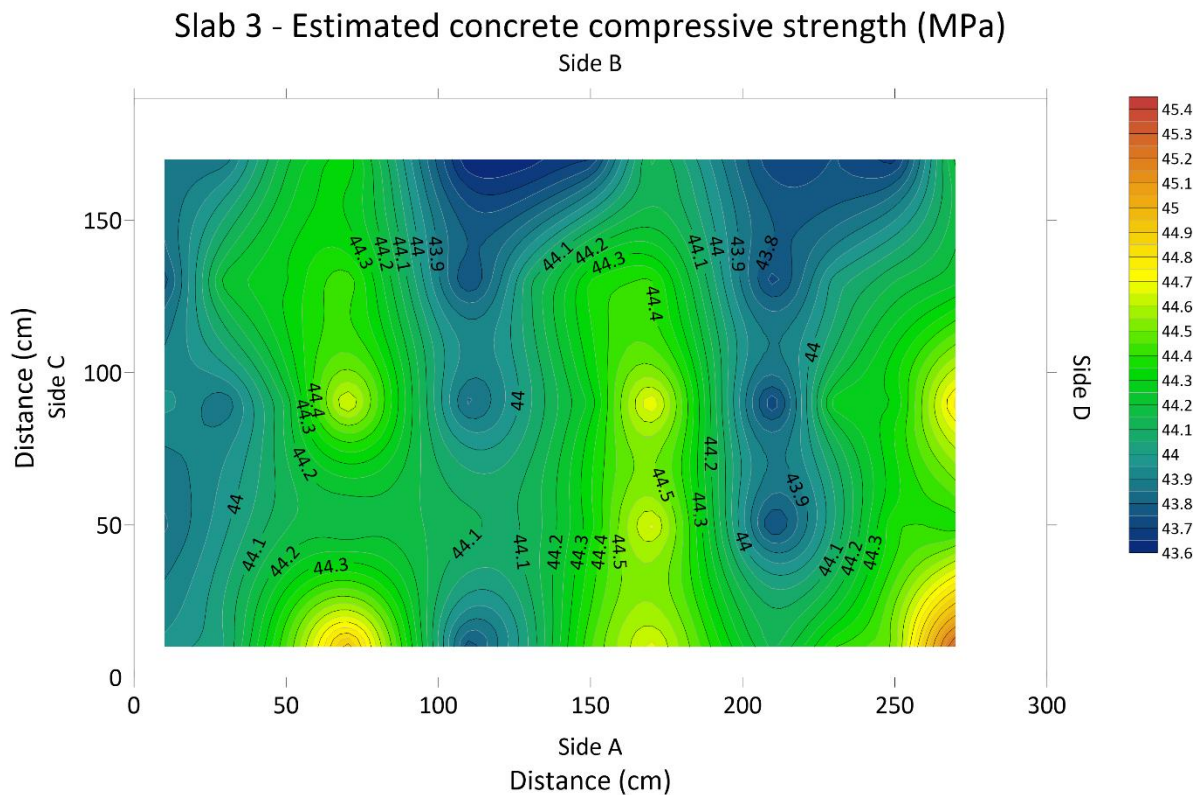




Figure 7-12. Estimated concrete compressive strength for slab 3 (47.5) using SonReb Method

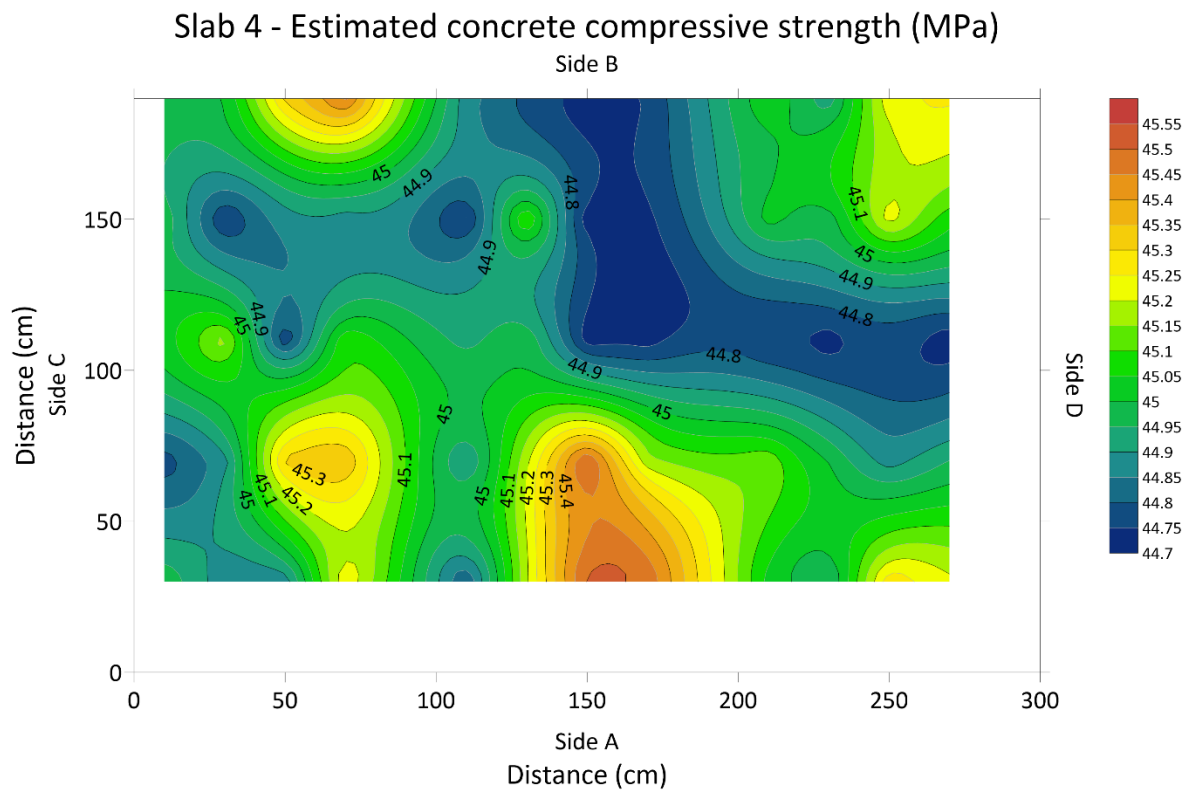


Figure 7-13. Estimated concrete compressive strength for slab 4 (47.1) using SonReb Method

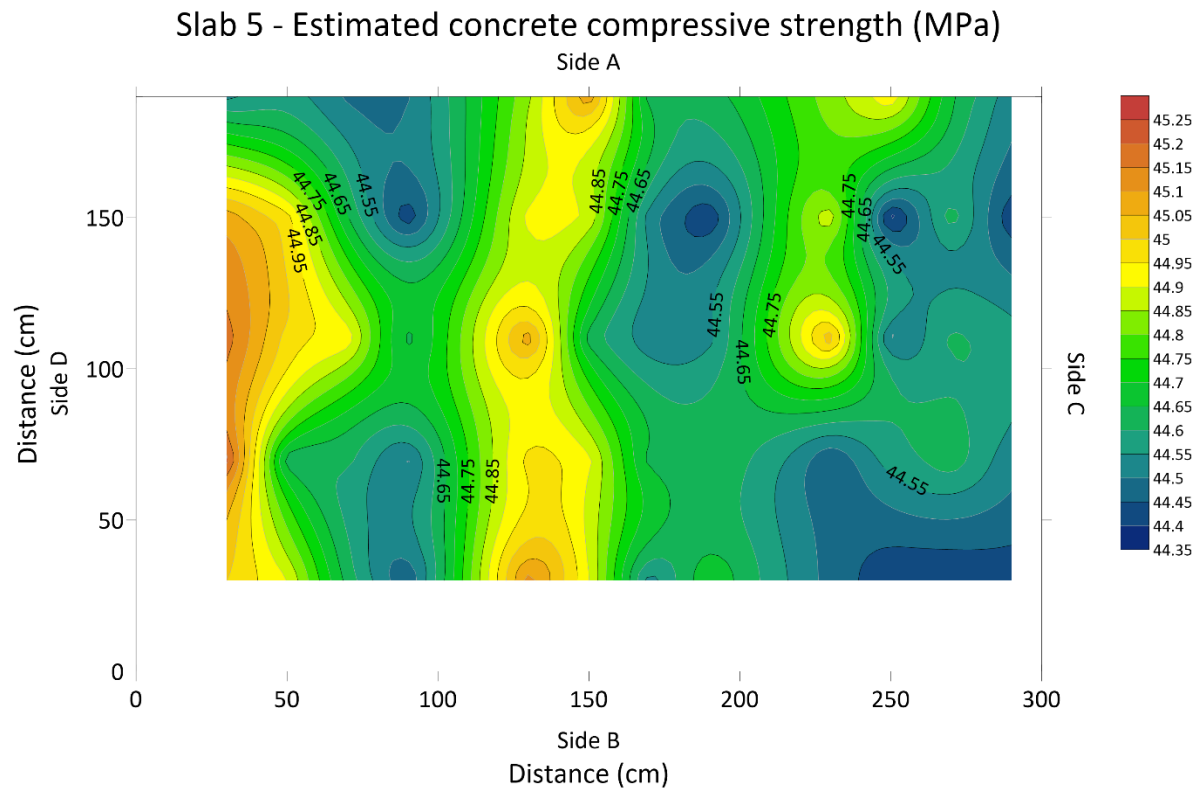


Figure 7-14. Estimated concrete compressive strength for slab 5 (47.3) using SonReb Method

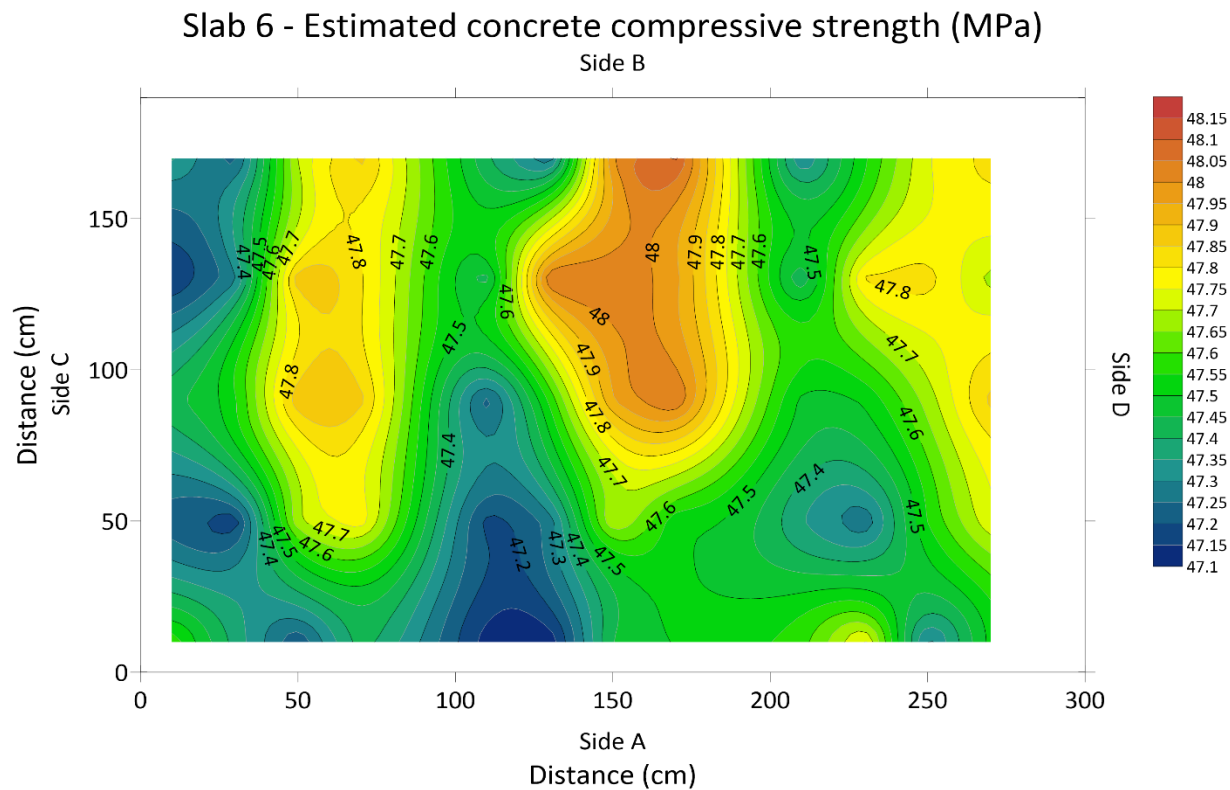


Figure 7-15. Estimated concrete compressive strength for slab 6 (47.4) using SonReb Method

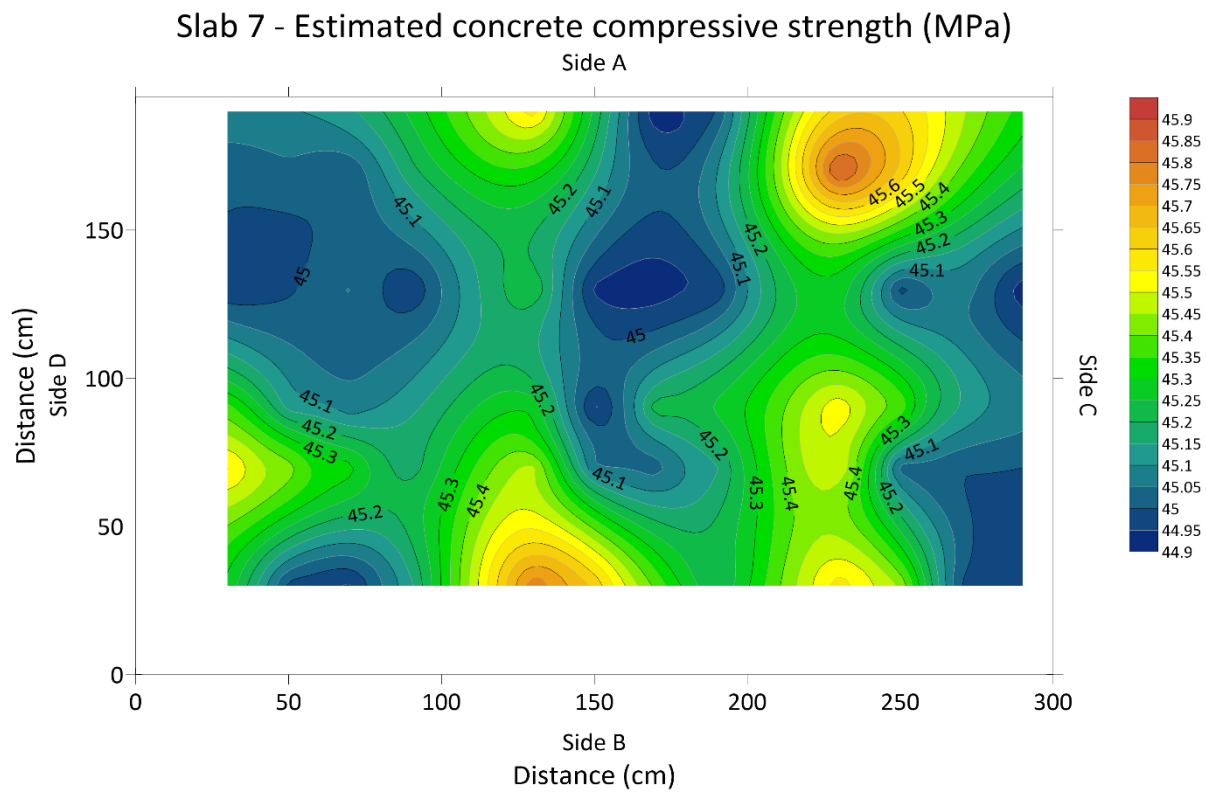


Figure 7-16. Estimated concrete compressive strength for slab 7 (47.7) using SonReb Method

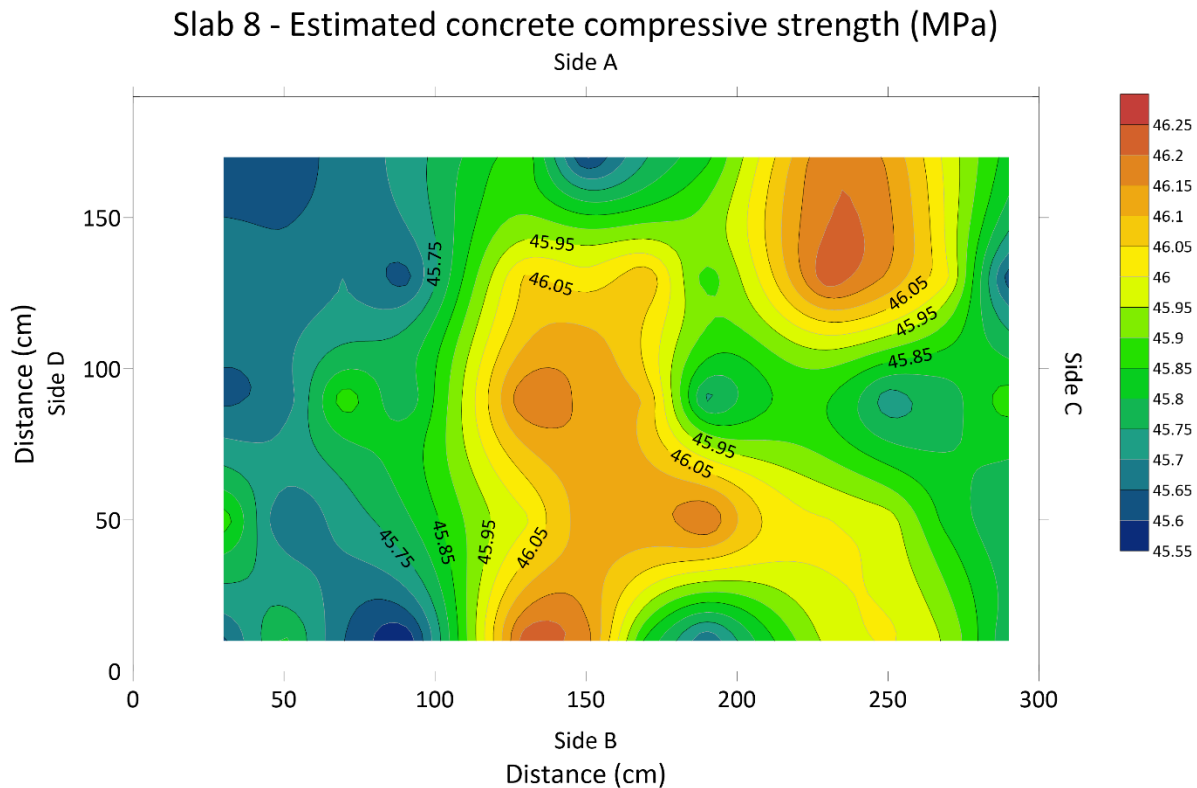


Figure 7-17. Estimated concrete compressive strength for slab 8 (47.8) using SonReb Method

#### A.4 Corrosion measurements with Giatec iCOR device

Corrosion data was obtained for the reinforcing steel embedded within each slab at 500 mm intervals along the length of the bars. As for the prestressing tendons, measurements were taken halfway in-between consecutive reinforcing steel bars. Contour maps highlighting the corrosion data for each type of reinforcement embedded within each slab are presented in the following sections.

##### A.4.1 Half-cell potential

###### A.4.1.1 Prestressing tendons

Figure 7-18 to Figure 7-25 display the contour maps of the half-cell potential measurements obtained along the length of the transverse prestressing tendons for each slab. Note that green areas correspond to potential measurements greater than -200 mV CSE, indicating a 10% likelihood of active corrosion. Yellow areas correspond to measurements ranging between -200 mV to -350 mV

CSE, where the likelihood of corrosion is unknown. Finally, red areas correspond to measurements less than -350 mV CSE, indicating a 90% likelihood of active corrosion (ASTM C876, 2015).

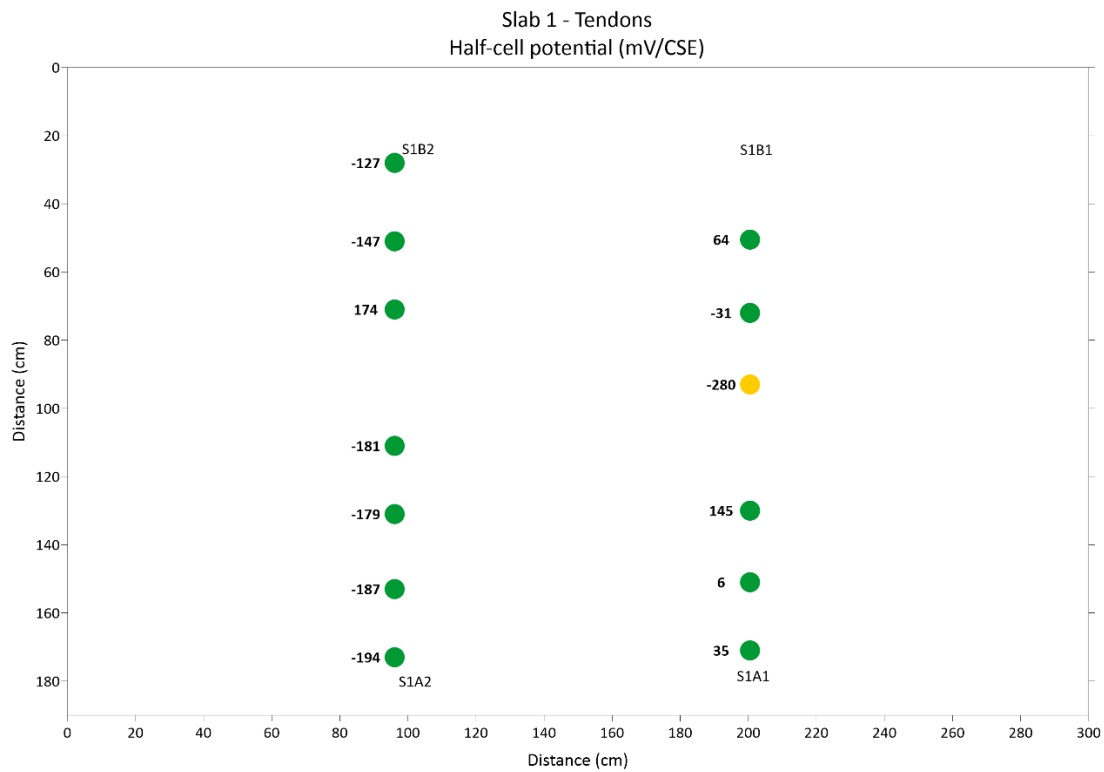


Figure 7-18. Half-cell potential mapping of prestressing tendons for slab 1 (47.6)

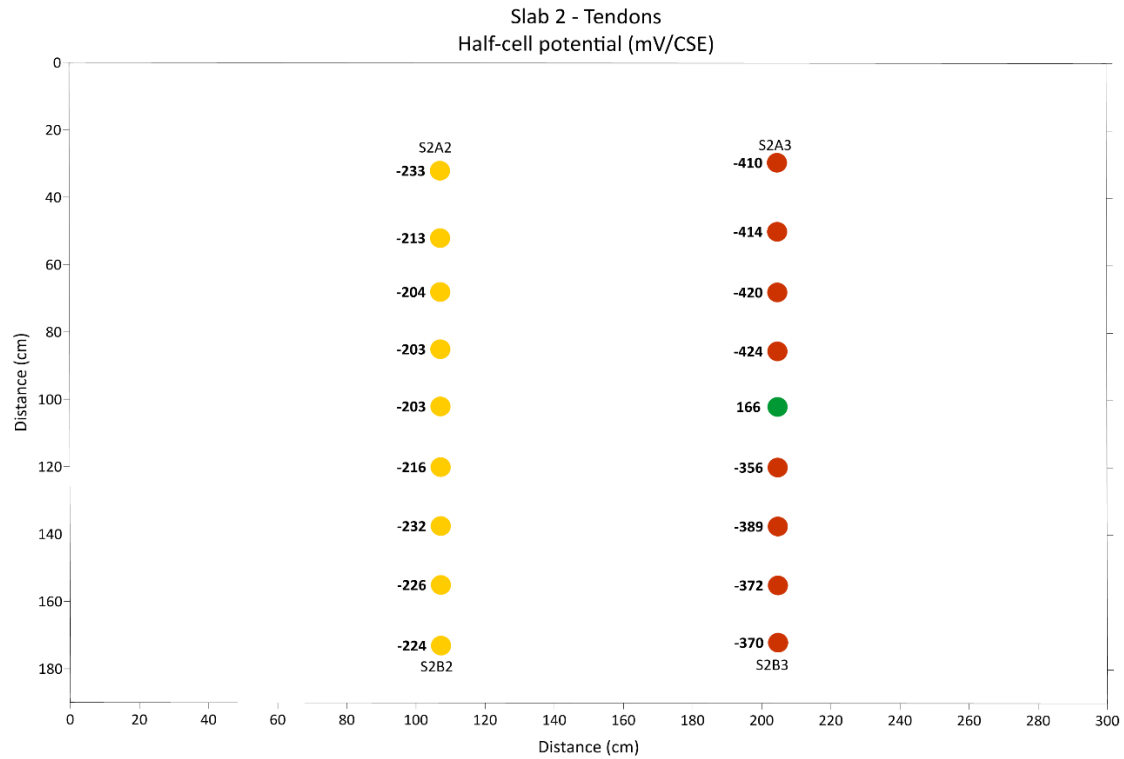


Figure 7-19. Half-cell potential mapping of prestressing tendons for slab 2 (47.2)

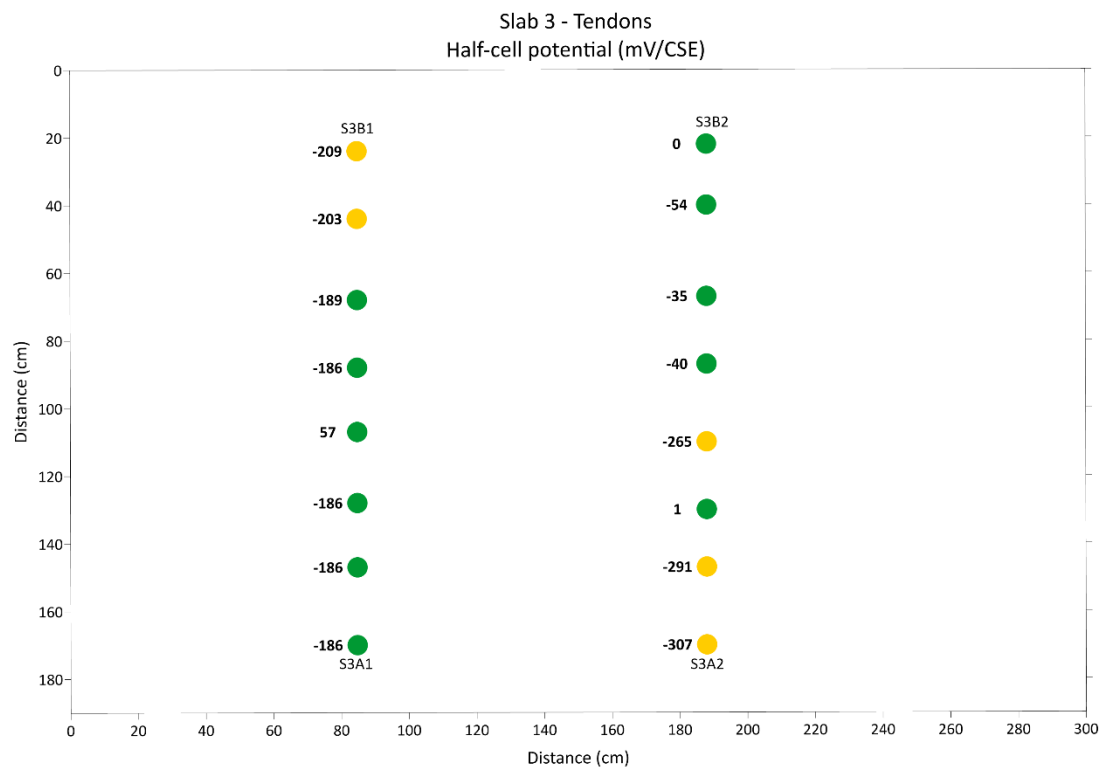


Figure 7-20. Half-cell potential mapping of prestressing tendons for slab 3 (47.5)

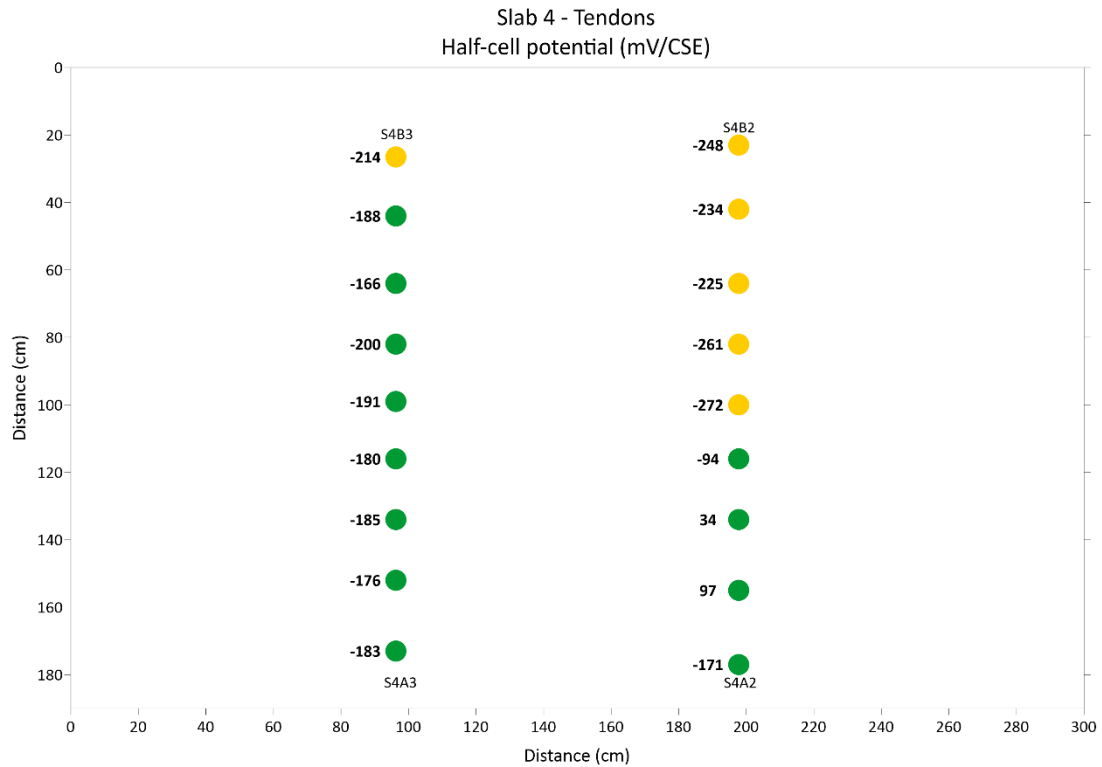


Figure 7-21. Half-cell potential mapping of prestressing tendons for slab 4 (47.1)

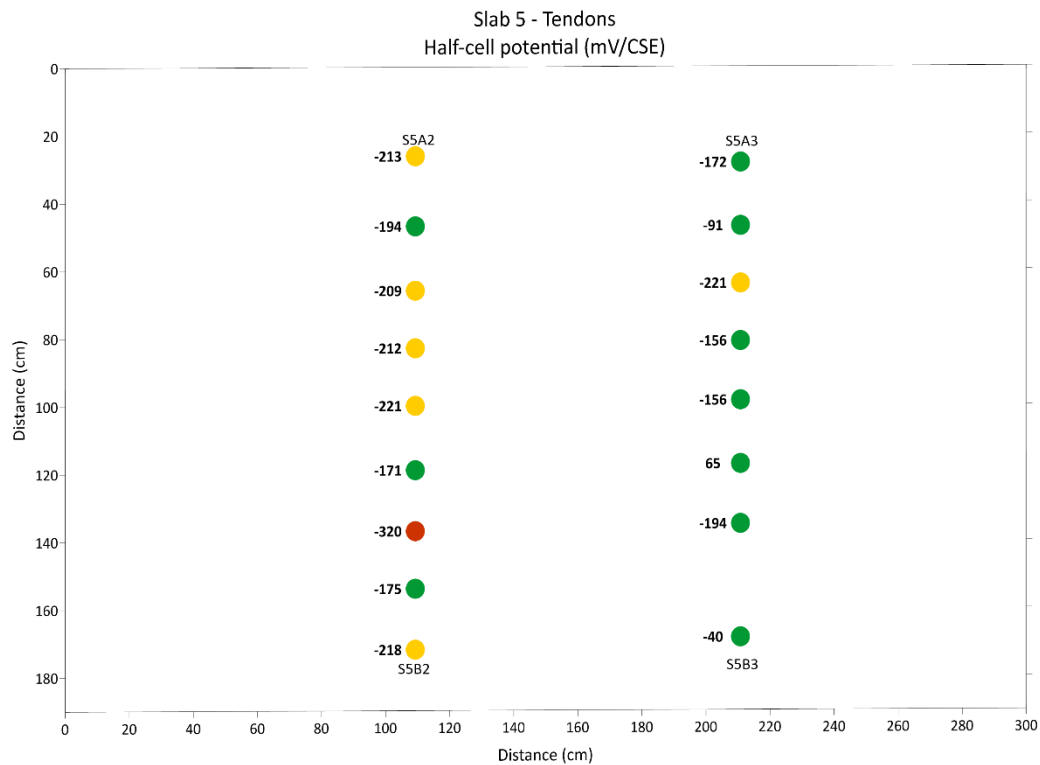


Figure 7-22. Half-cell potential mapping of prestressing tendons for slab 5 (47.3)

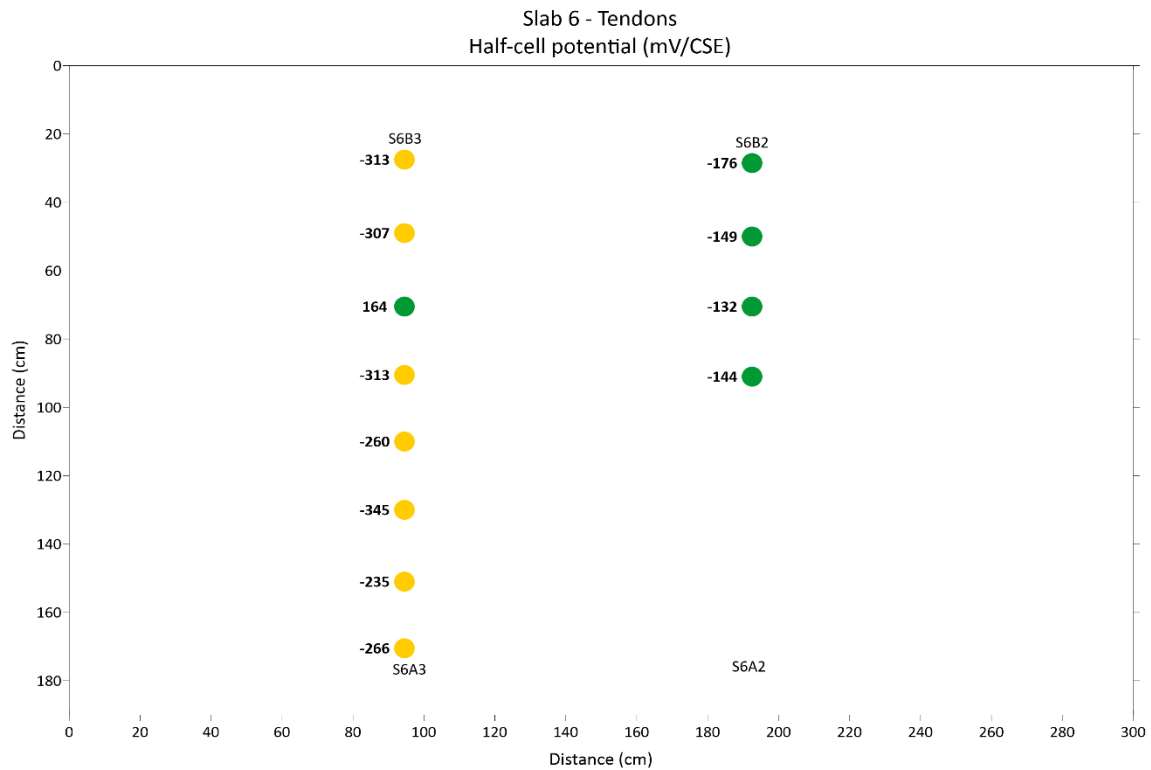


Figure 7-23. Half-cell potential mapping of prestressing tendons for slab 6 (47.4)

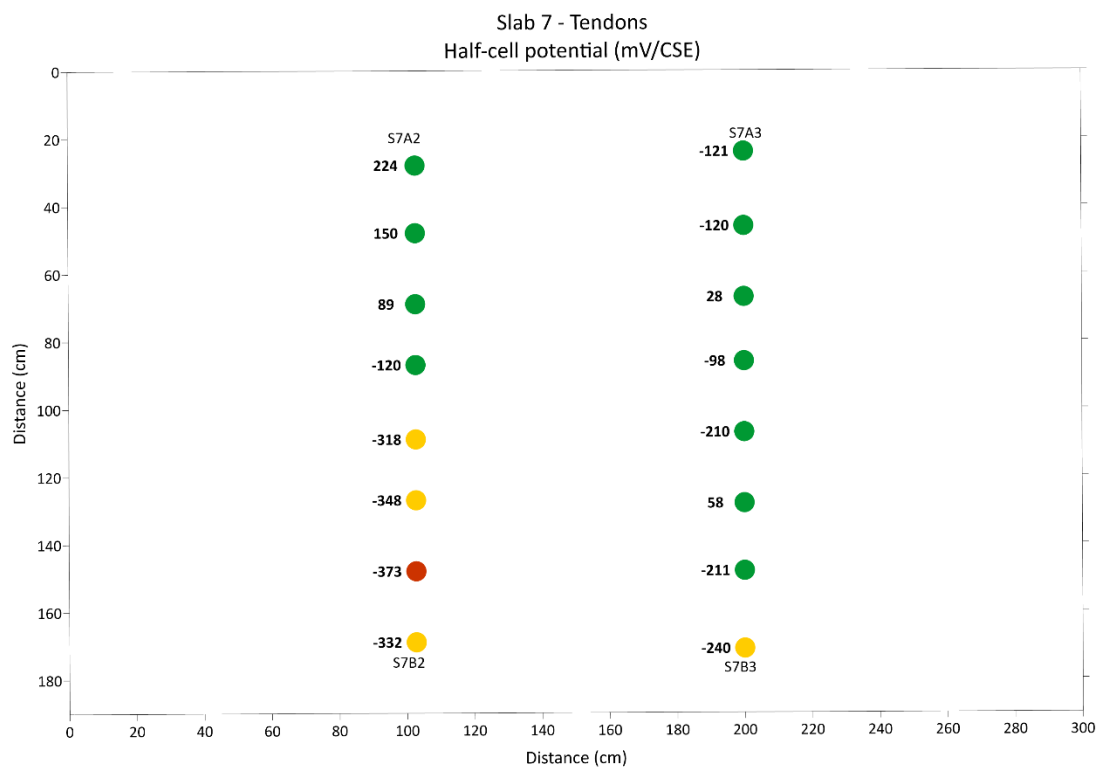


Figure 7-24. Half-cell potential mapping of prestressing tendons for slab 7 (47.7)



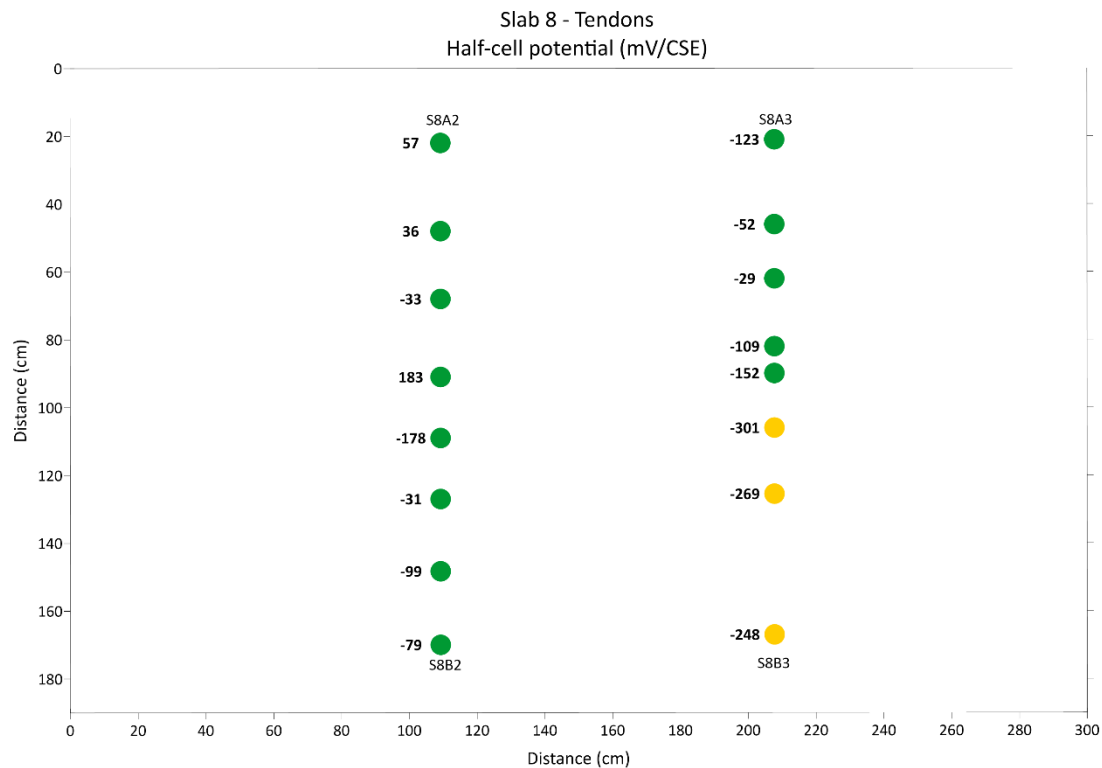


Figure 7-25. Half-cell potential mapping of prestressing tendons for slab 8 (47.8)

#### A.4.1.2 Temperature reinforcement

Figure 7-26 to Figure 7-29 display the contour maps of the half-cell potential measurements obtained at the locations identified with a '+' along the temperature reinforcement on slabs 1 to 8. Note that contour colours follow the guidelines in ASTM C876. Green corresponds to potential measurements  $> -200$  mV CSE (indicating a 10% likelihood of active corrosion). Yellow corresponds to the range  $-200$  mV to  $-350$  mV CSE (unknown if corrosion is occurring). Red corresponds to  $< -350$  mV CSE (indicating a 90% likelihood of active corrosion) (ASTM C876, 2015).

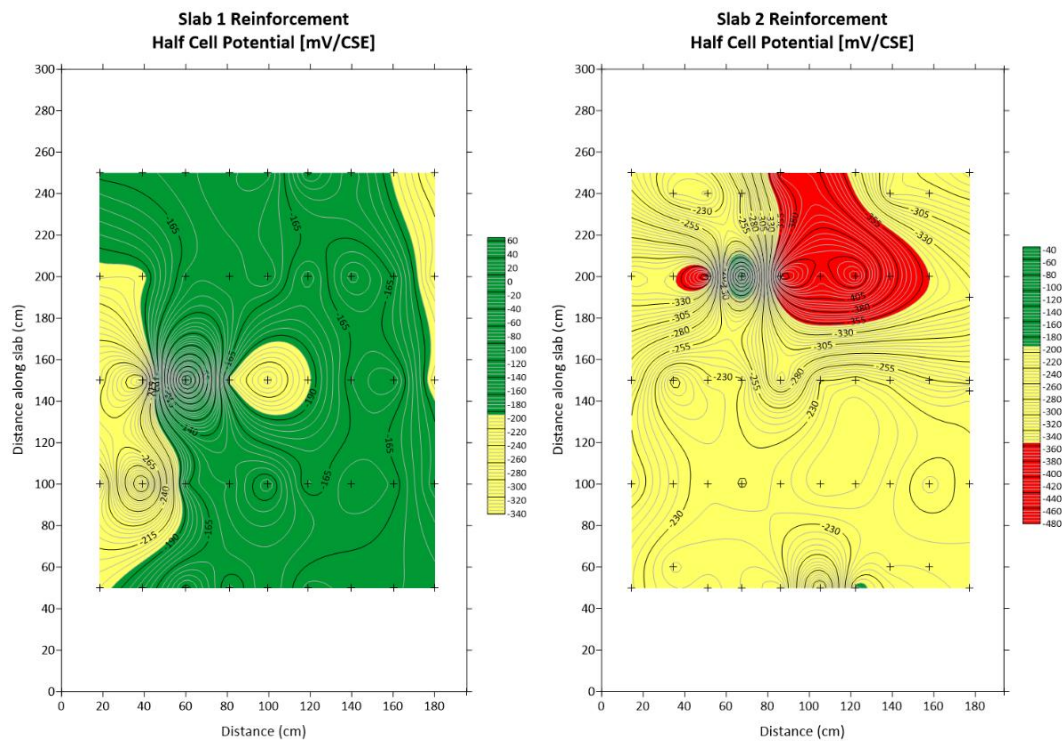


Figure 7-26. Half cell potential mapping for slabs 1 (47.6) and 2 (47.2)

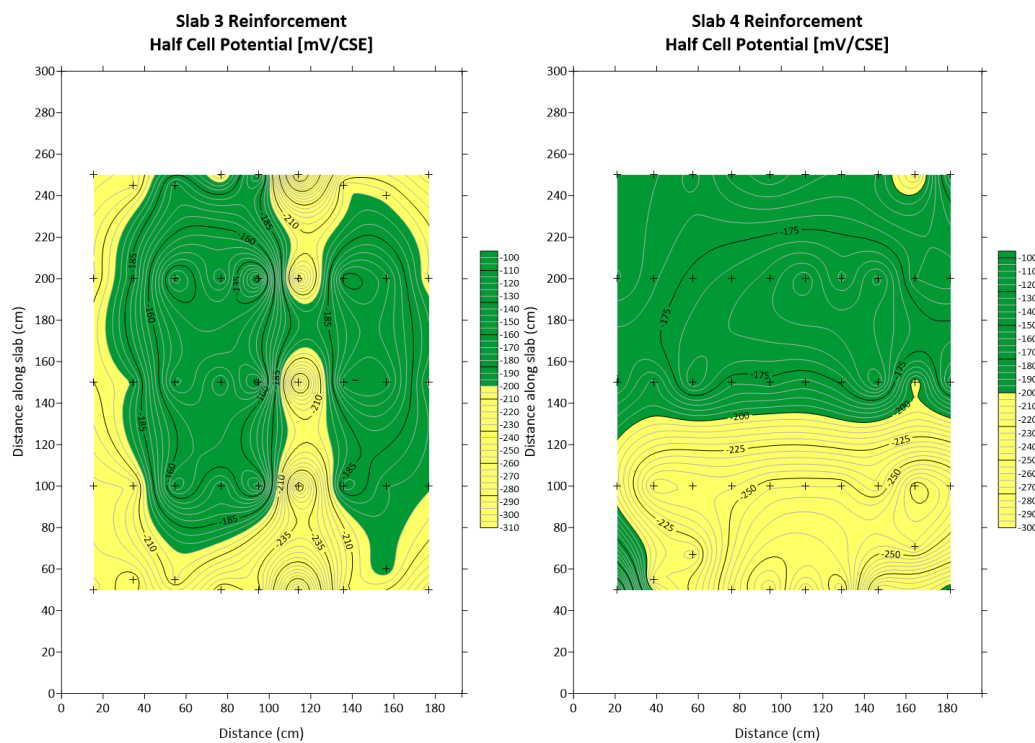


Figure 7-27. Half cell potential mapping for slabs 3 (47.5) and 4 (47.1)

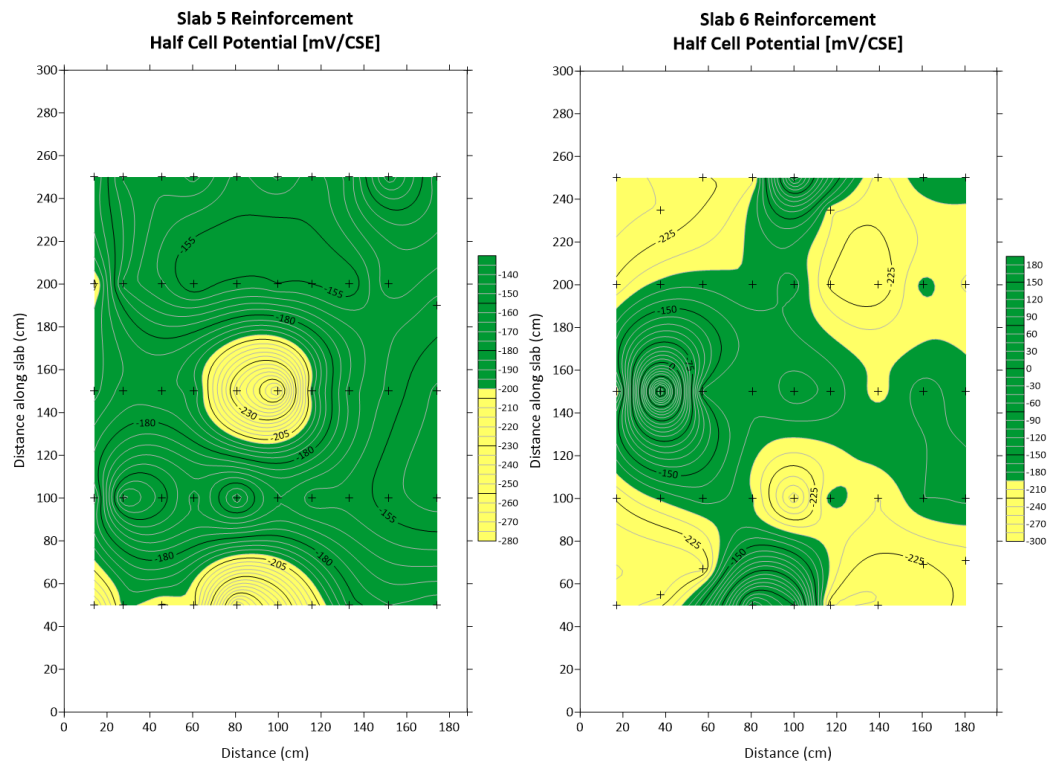


Figure 7-28. Half cell potential mapping for slabs 5 (47.3) and 6 (47.4)

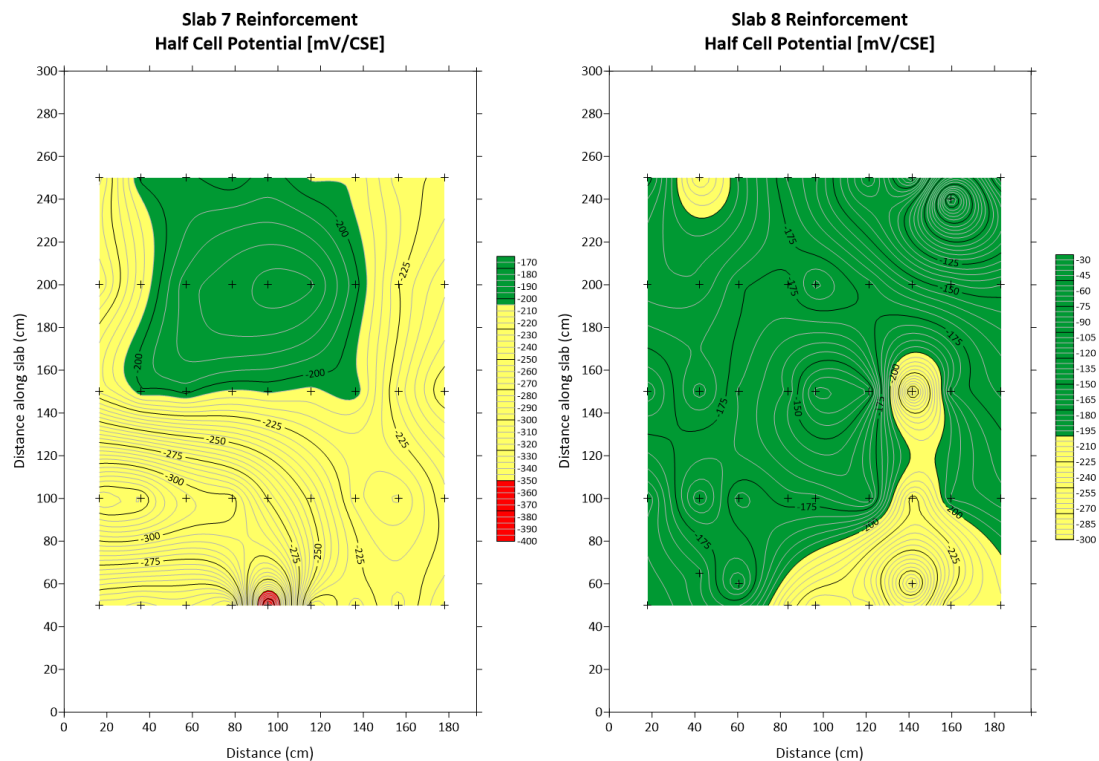


Figure 7-29. Half cell potential mapping for slabs 7 (47.7) and 8 (47.8)

## A.4.2 Corrosion rate

### A.4.2.1 Prestressing tendons

Figure 7-30 to Figure 7-37 display the corrosion-rate measurements obtained along the length of the transverse prestressing tendons for each slab. Note that green areas correspond to corrosion rate measurements less than  $0.5 \mu\text{A}/\text{cm}^2$  (indicating low corrosion levels), yellow areas correspond to values ranging between  $0.5$  to  $1.0 \mu\text{A}/\text{cm}^2$  (indicating moderate corrosion levels), and red areas correspond to measurements exceeding  $1.0 \mu\text{A}/\text{cm}^2$  (indicating high corrosion levels) (Andrade & Alonso, 1996).

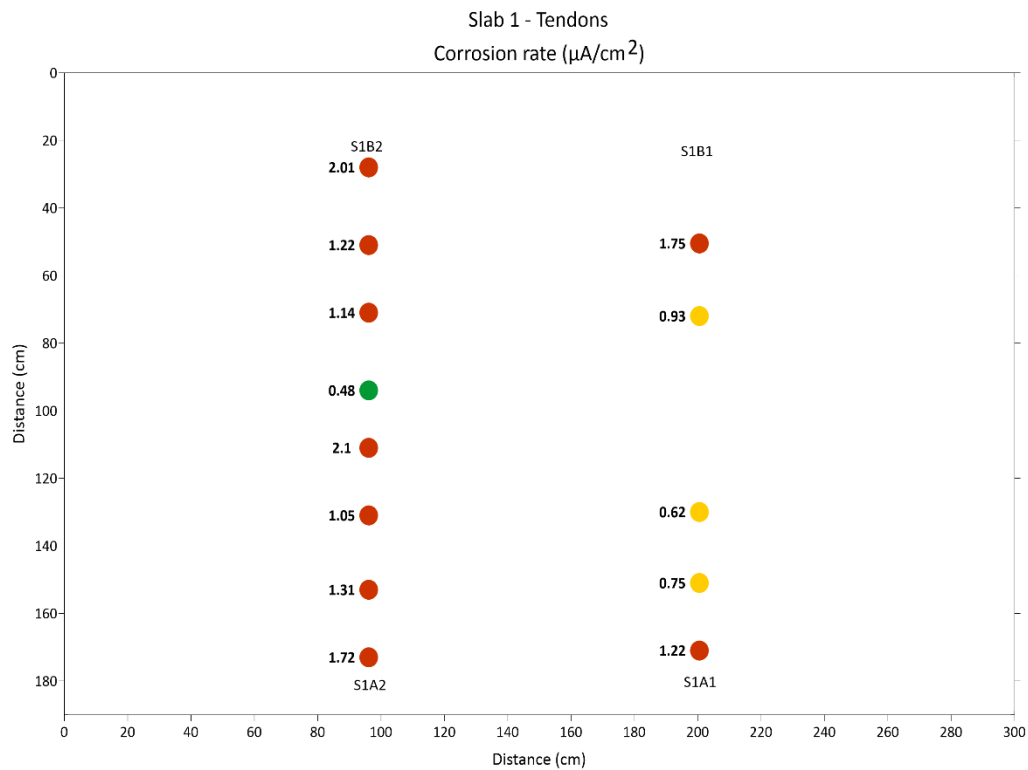


Figure 7-30. Corrosion rate mapping of prestressing tendons for slab 1 (47.6)

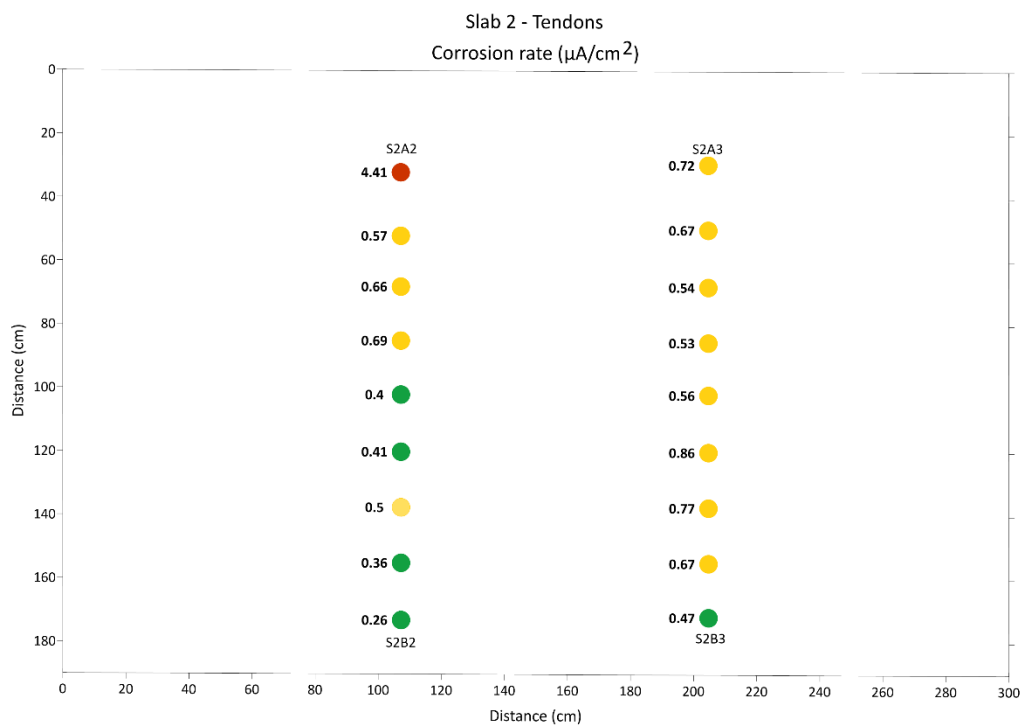


Figure 7-31. Corrosion rate mapping of prestressing tendons for slab 2 (47.2)

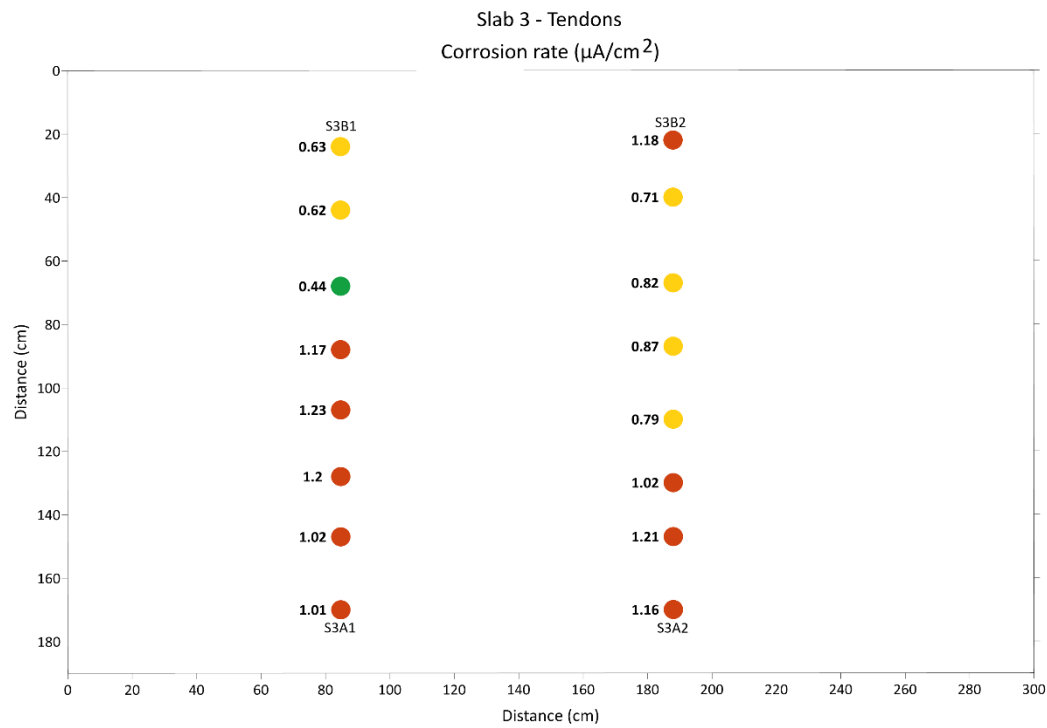


Figure 7-32. Corrosion rate mapping of prestressing tendons for slab 3 (47.5)

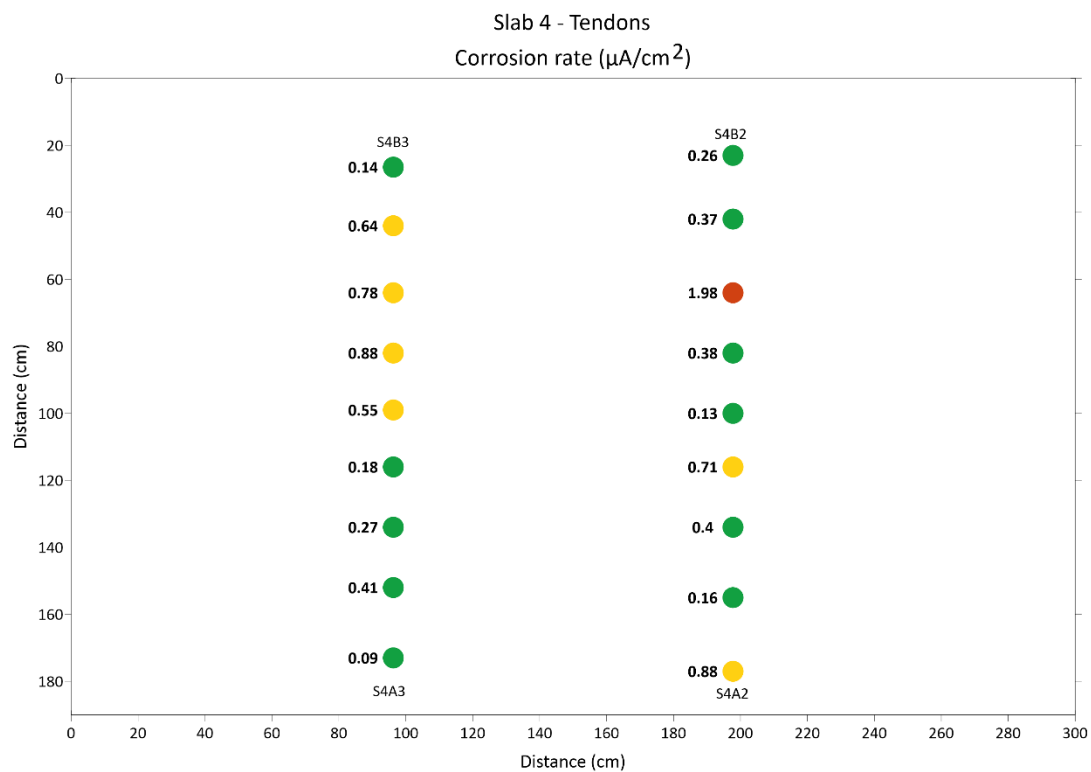


Figure 7-33. Corrosion rate mapping of prestressing tendons for slab 4 (47.1)

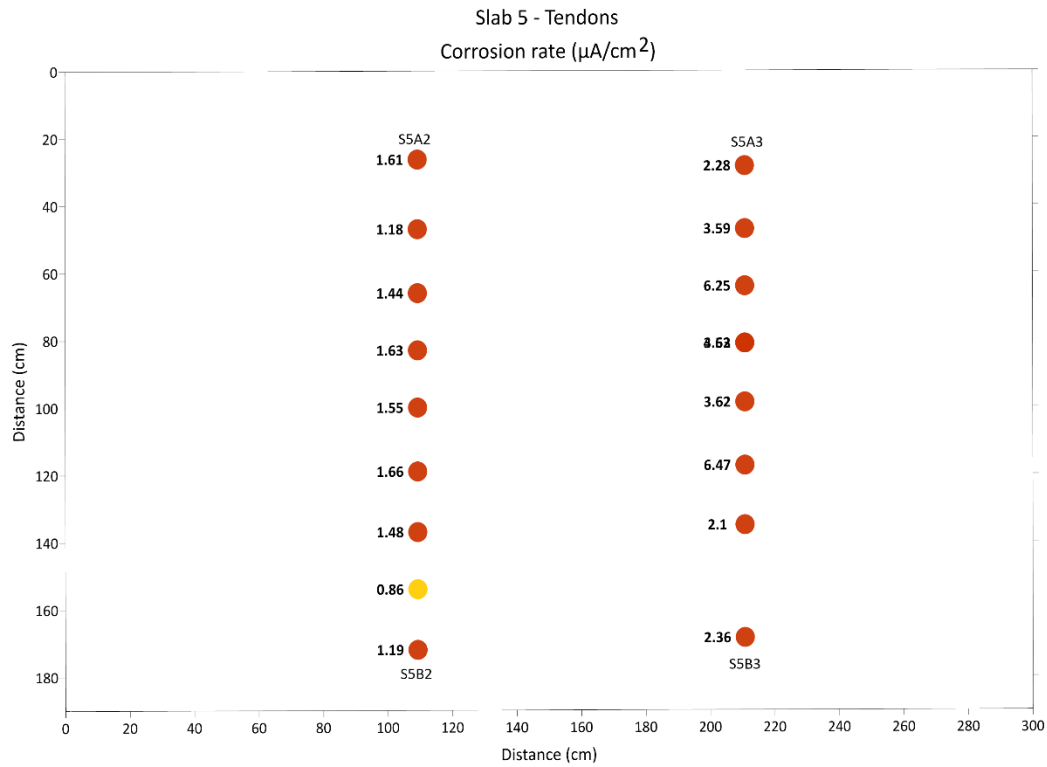


Figure 7-34. Corrosion rate mapping of prestressing tendons for slab 5 (47.3)

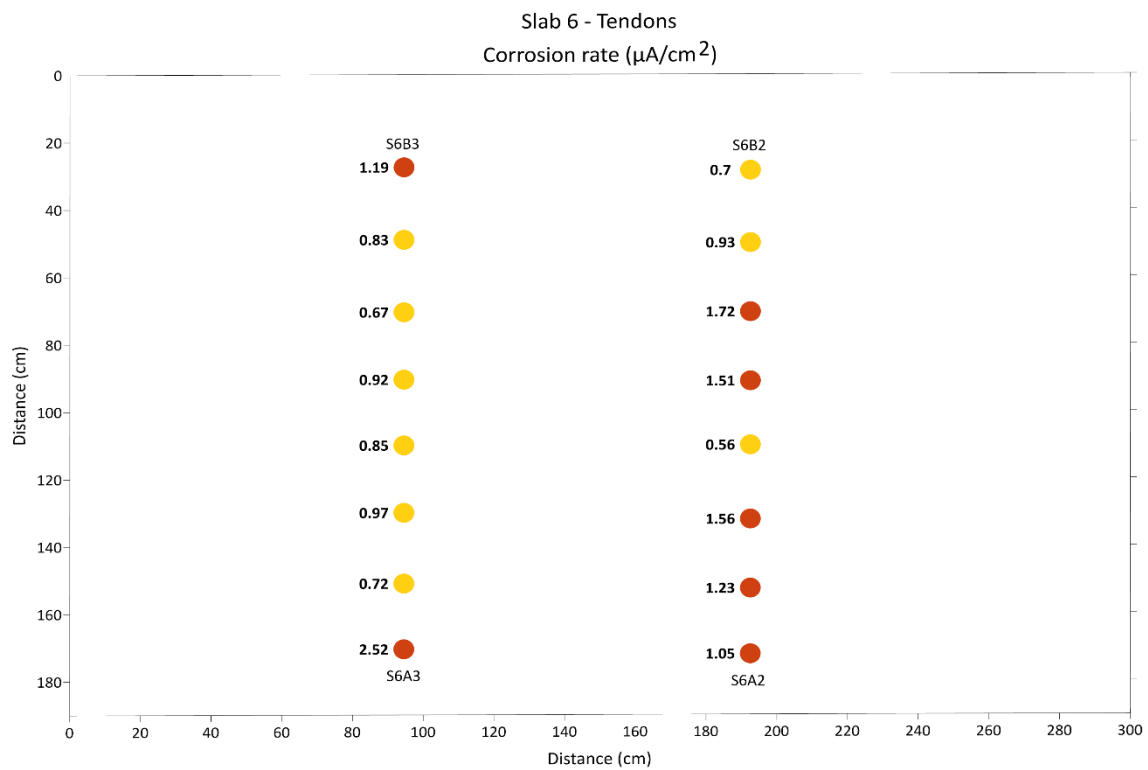


Figure 7-35. Corrosion rate mapping of prestressing tendons for slab 6 (47.4)

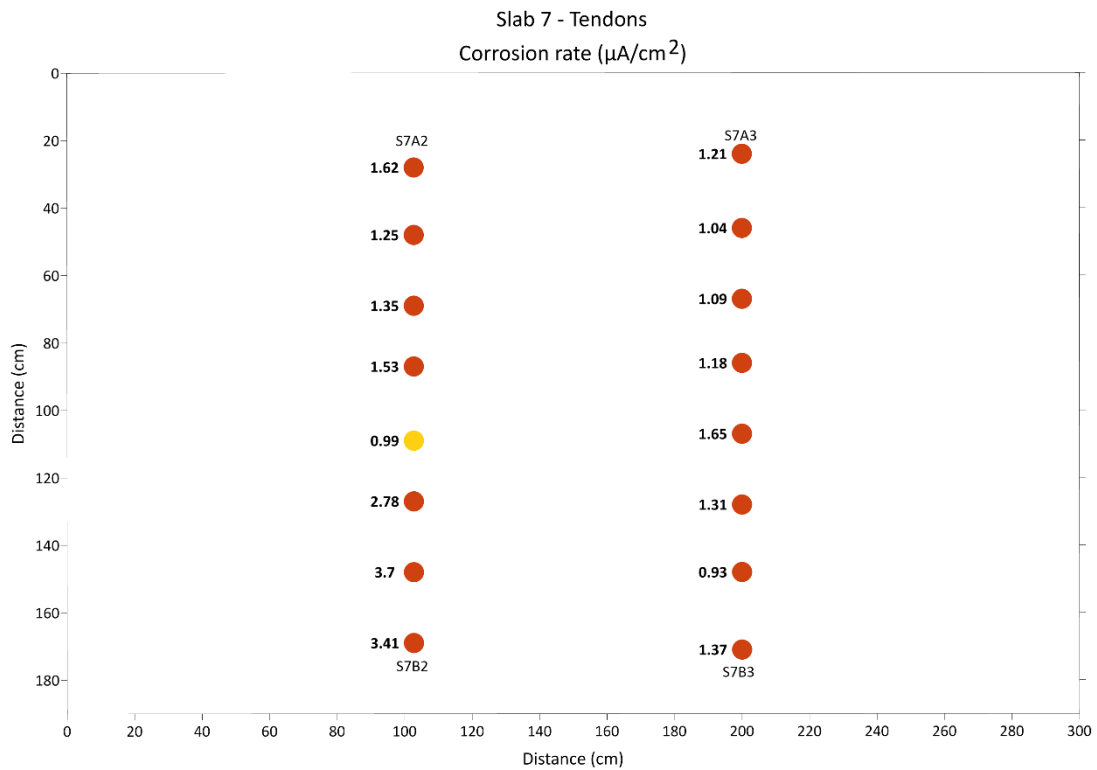


Figure 7-36. Corrosion rate mapping of prestressing tendons for slab 7 (47.7)

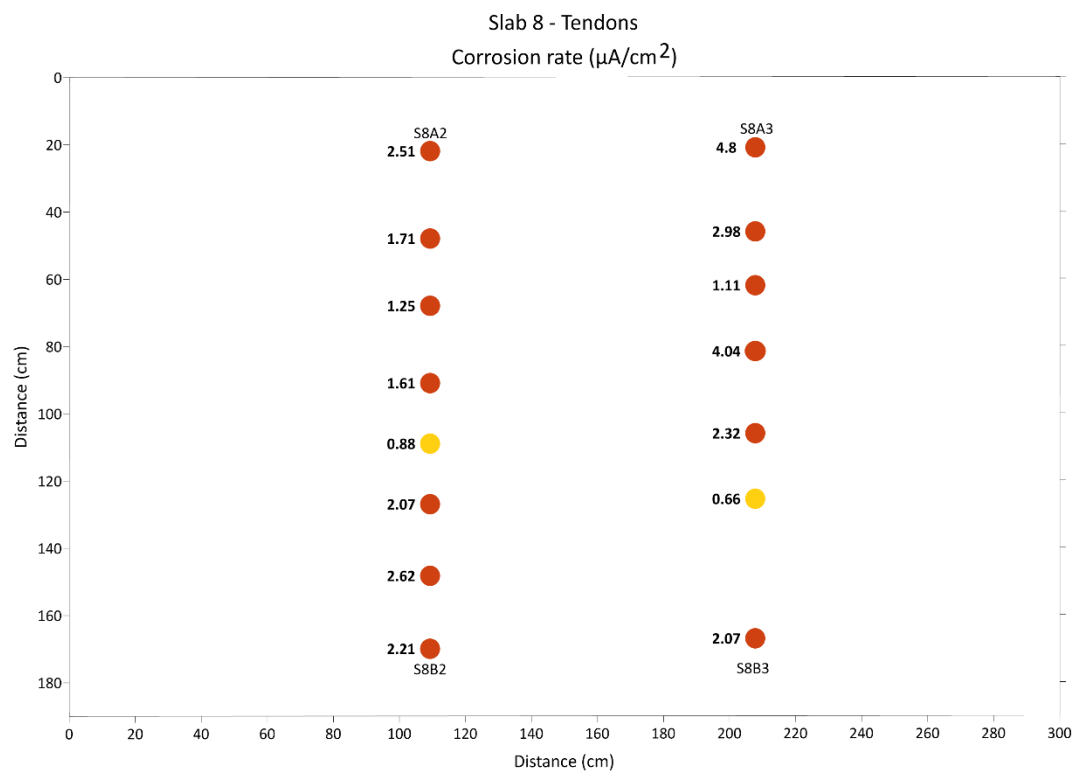


Figure 7-37. Corrosion rate mapping of prestressing tendons for slab 8 (47.8)



### A.4.2.2 Temperature reinforcement

Figure 7-38 and Figure 7-41 display the contour maps of the corrosion rate measurements obtained at the locations identified with a '+' along the temperature reinforcement on slabs 1 to 8. Note that green corresponds to corrosion rate measurements  $< 0.5 \mu\text{A}/\text{cm}^2$  (indicating low corrosion levels). Yellow corresponds to the range  $0.5$  to  $1.0 \mu\text{A}/\text{cm}^2$  (indicating moderate corrosion levels). Red corresponds to  $> 1.0 \mu\text{A}/\text{cm}^2$  (indicating high corrosion levels) (Andrade & Alonso, 1996).

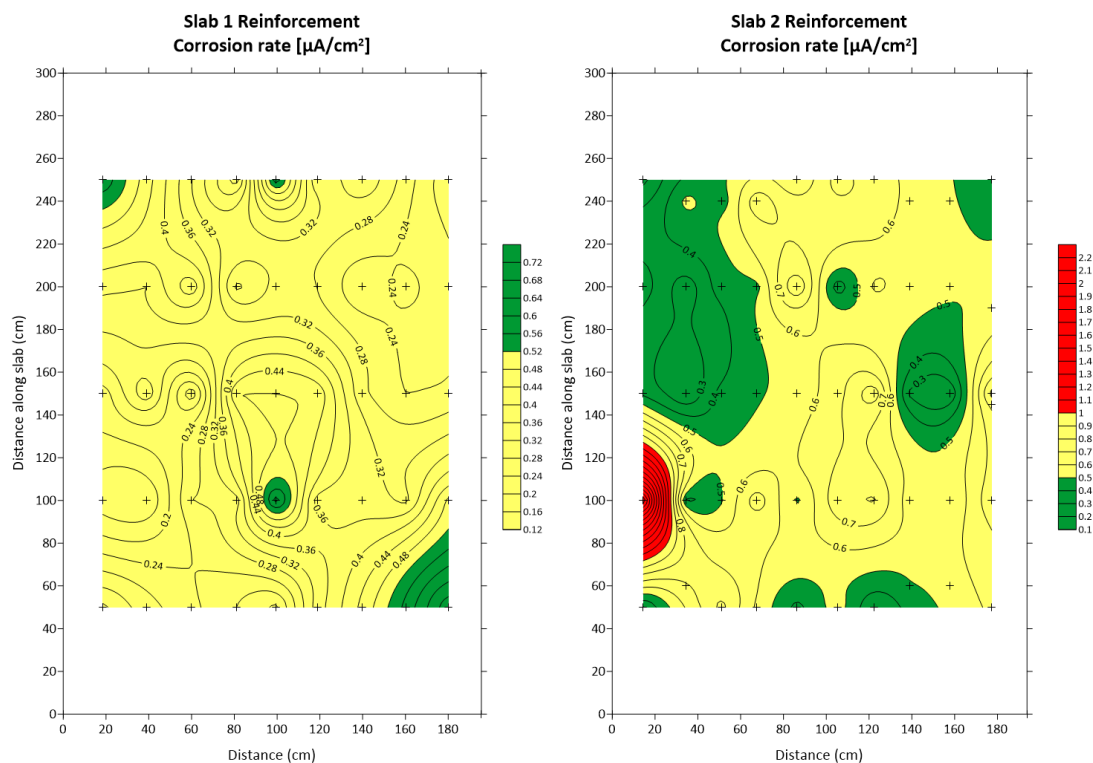


Figure 7-38. Corrosion rate mapping for slabs 1 (47.6) and 2 (47.2)

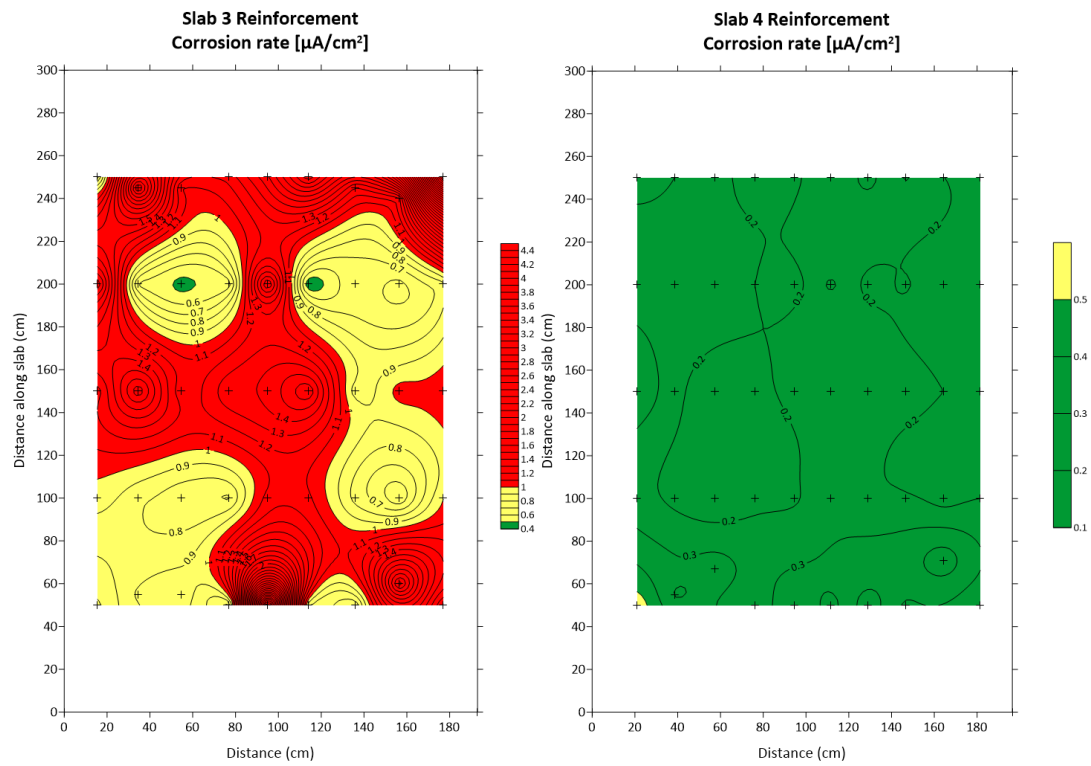


Figure 7-39. Corrosion Current mapping for slabs 3 (47.5) and 4 (47.1)

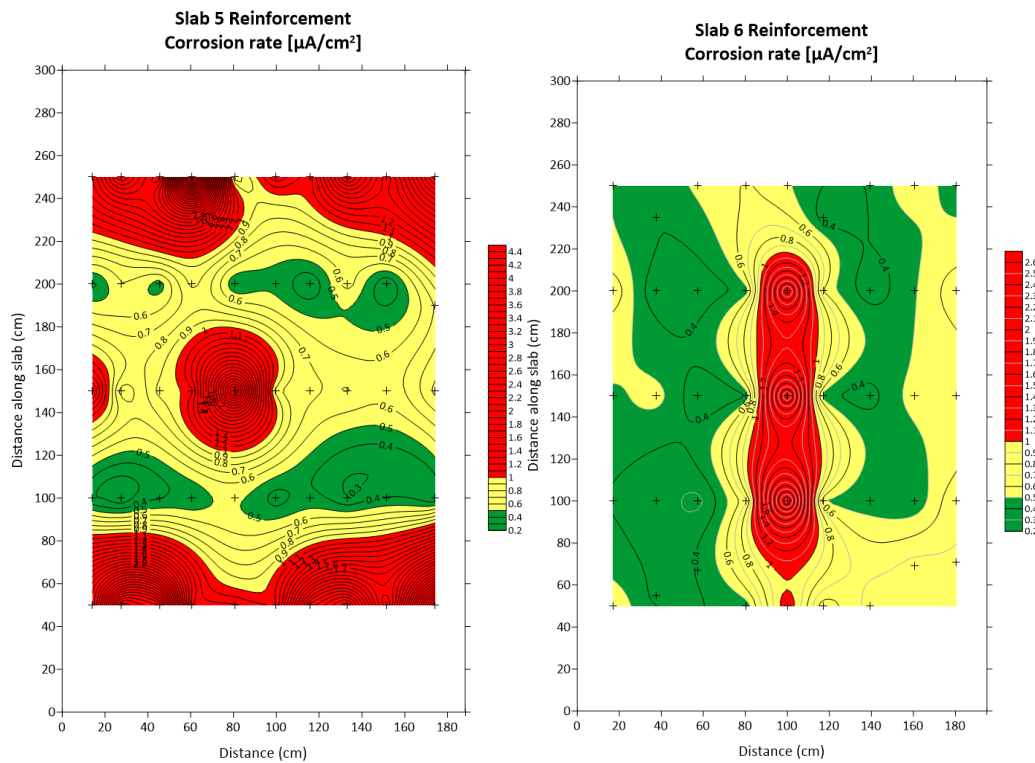


Figure 7-40. Corrosion Current mapping for slabs 5 (47.3) and 6 (47.4)

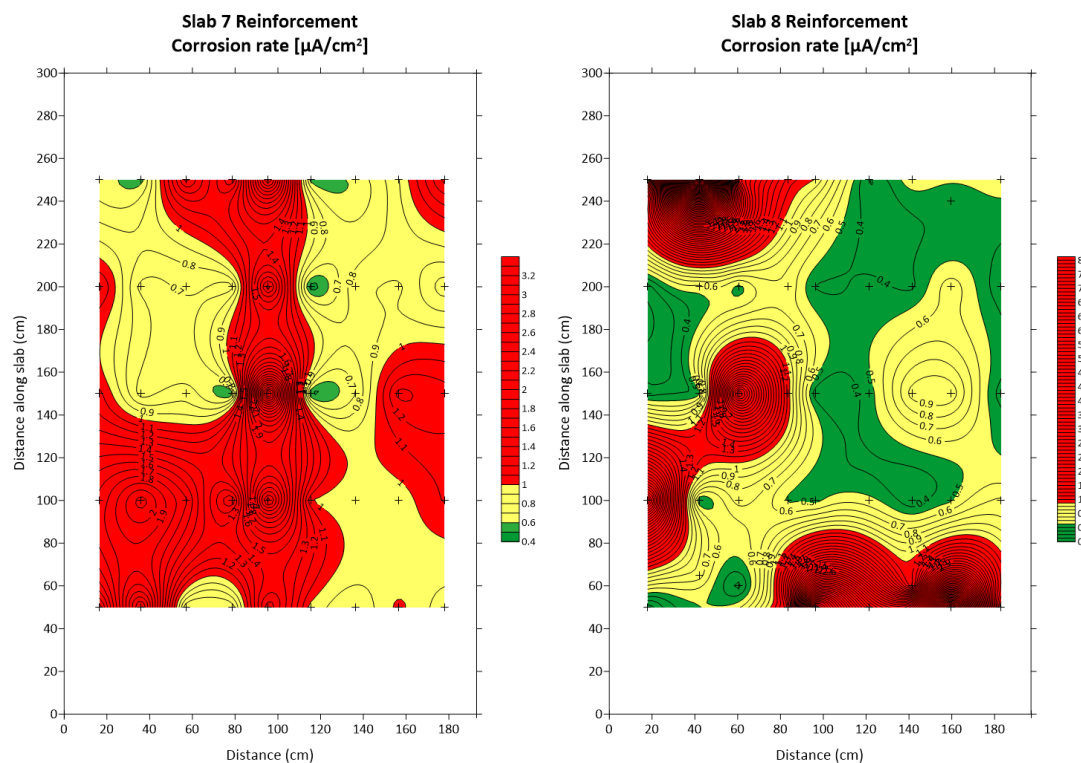


Figure 7-41. Corrosion Current mapping for slabs 7 (47.7) and 8 (47.8)

### A.4.3 Concrete surface resistivity

#### A.4.3.1 Prestressing tendons

Figure 7-42 to Figure 7-49 display the contour maps of the concrete surface resistivity measurements obtained along the length of the transverse prestressing tendons for each slab.

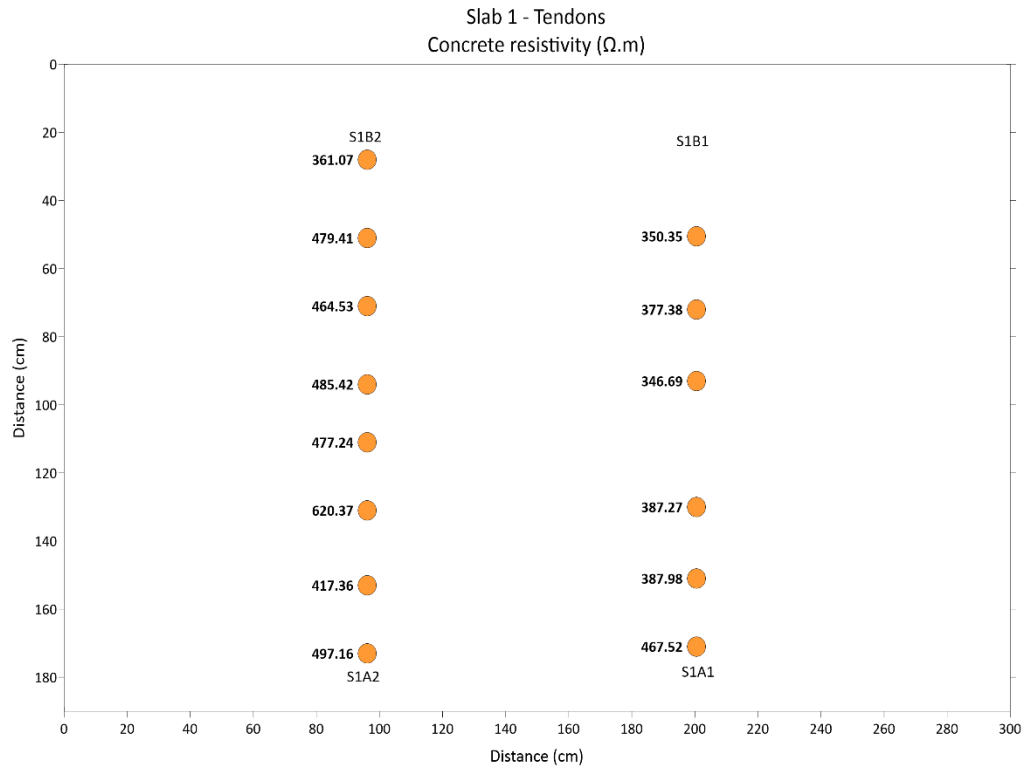


Figure 7-42. Surface resistivity mapping of concrete along prestressing tendons for slab 1 (47.6)

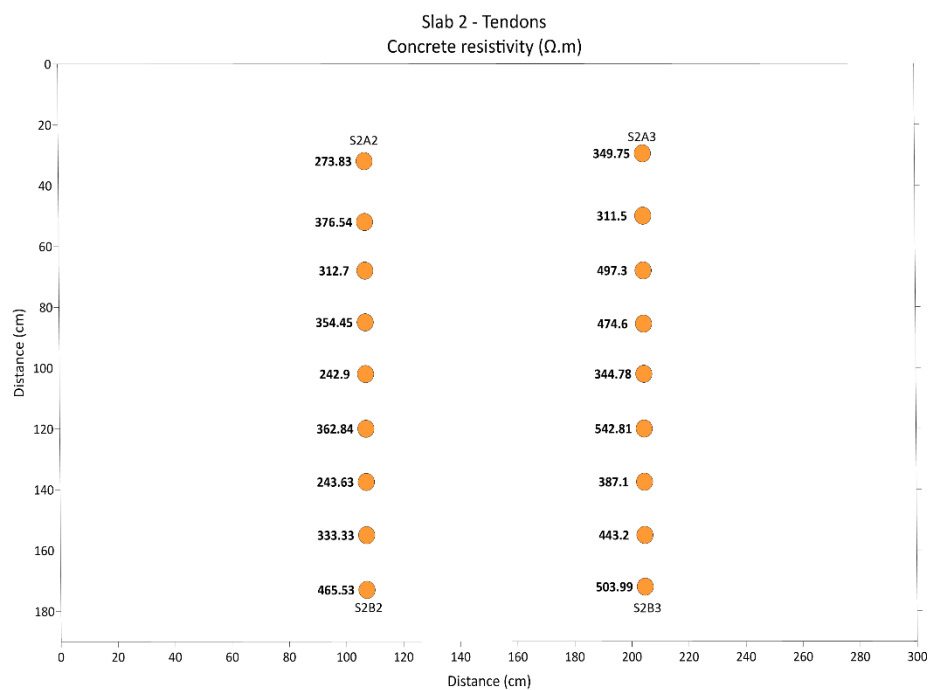


Figure 7-43. Surface resistivity mapping of concrete along prestressing tendons for slab 2 (47.2)

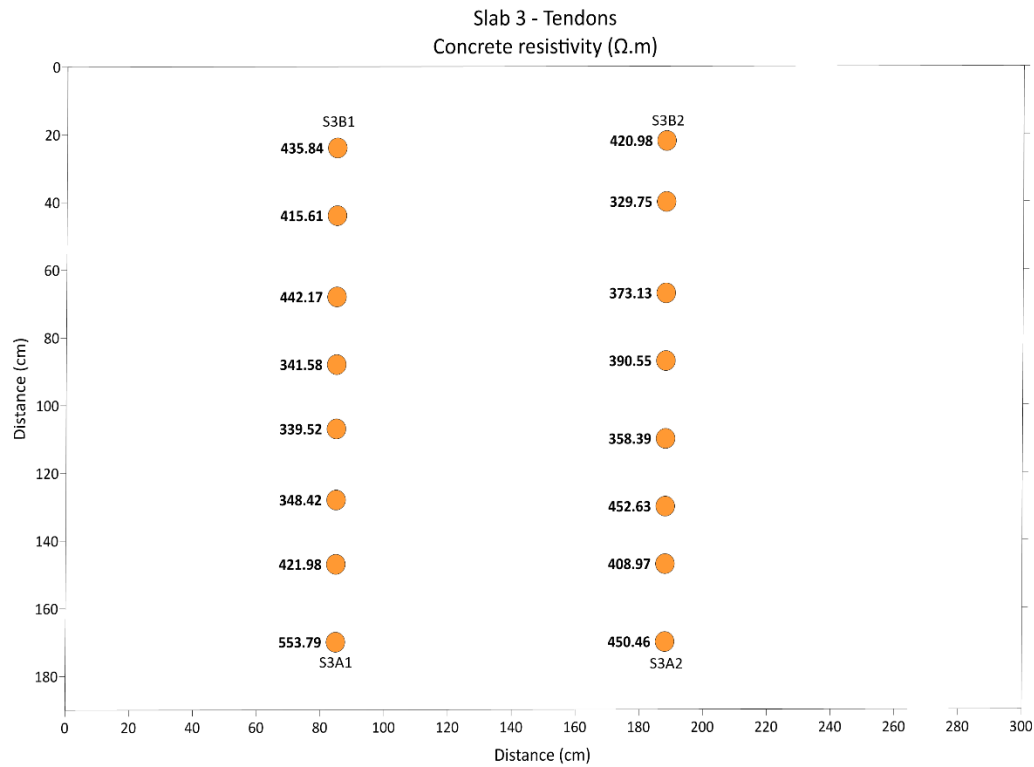


Figure 7-44. Surface resistivity mapping of concrete along prestressing tendons for slab 3 (47.5)

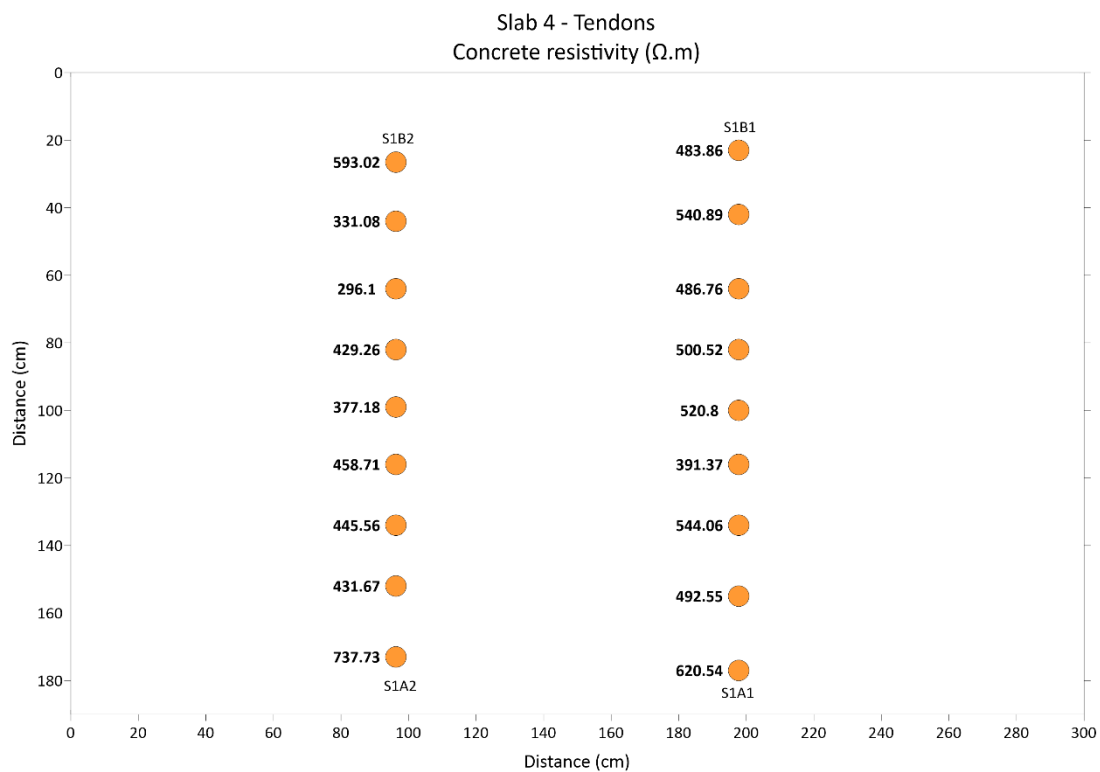


Figure 7-45. Surface resistivity mapping of concrete along prestressing tendons for slab 4 (47.1)

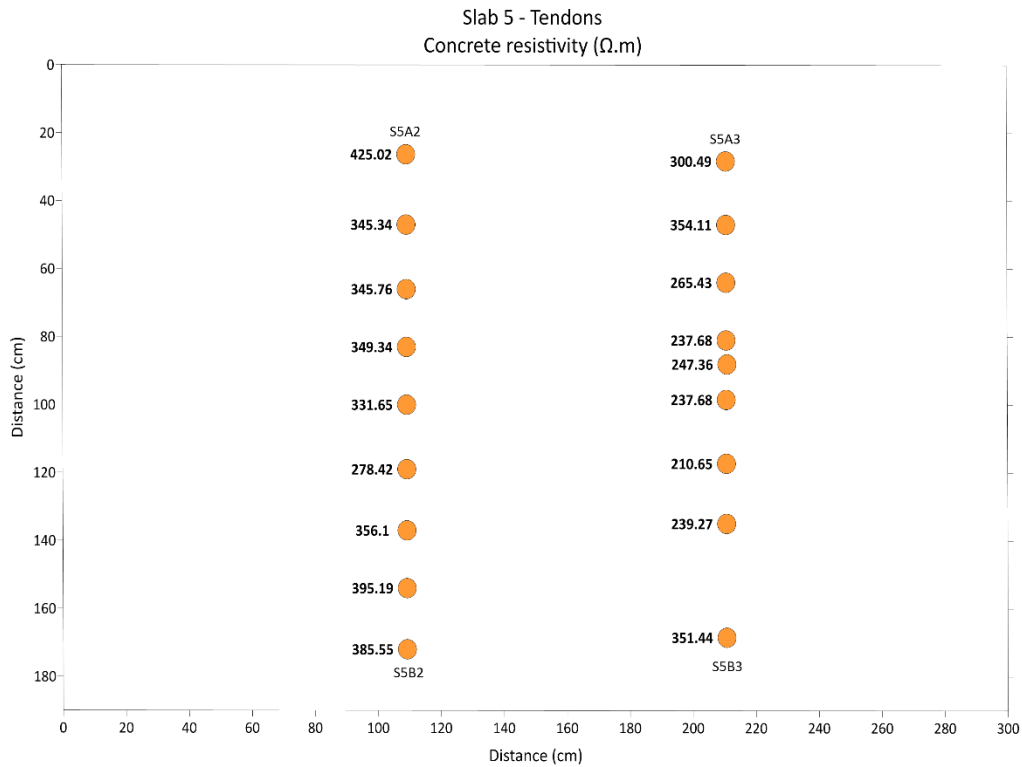


Figure 7-46. Surface resistivity mapping of concrete along prestressing tendons for slab 5 (47.3)

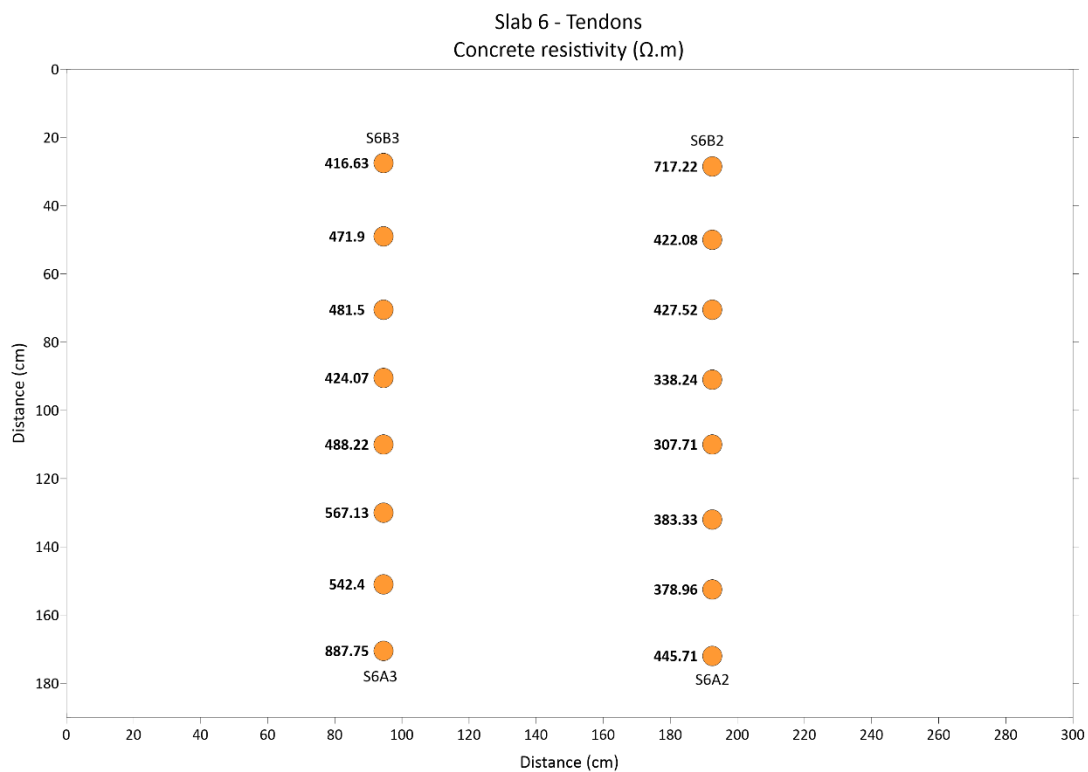


Figure 7-47. Surface resistivity mapping of concrete along prestressing tendons for slab 6 (47.4)

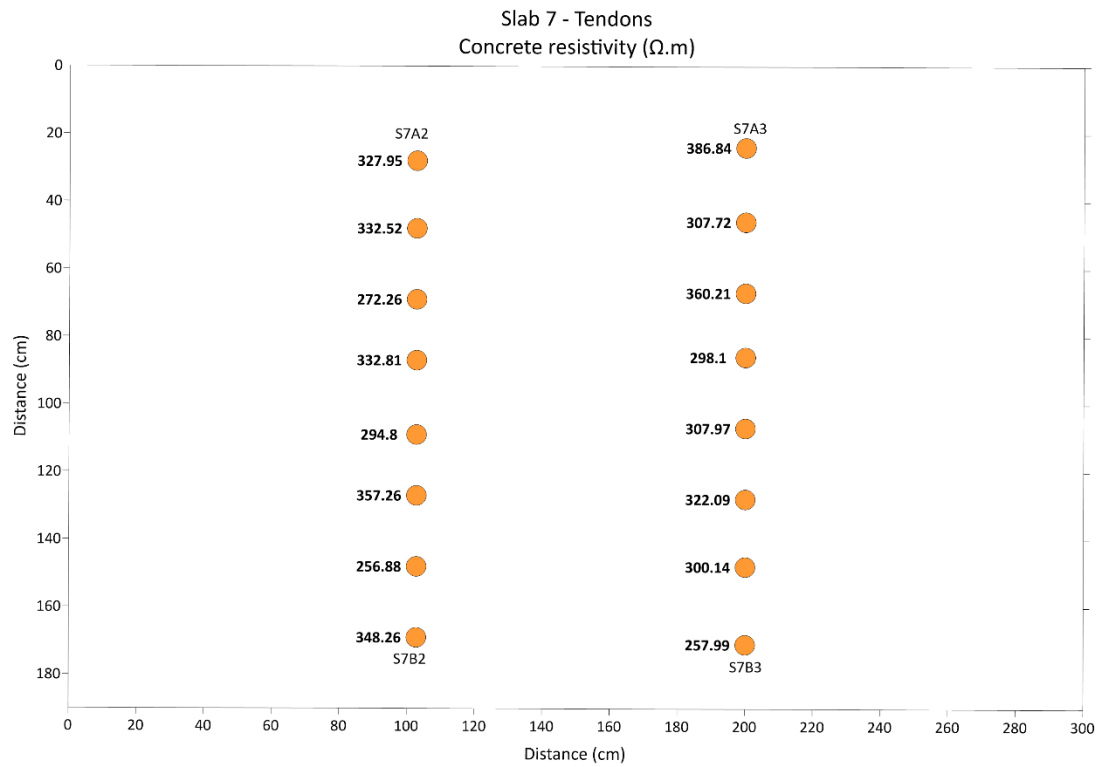


Figure 7-48. Surface resistivity mapping of concrete along prestressing tendons for slab 7 (47.7)

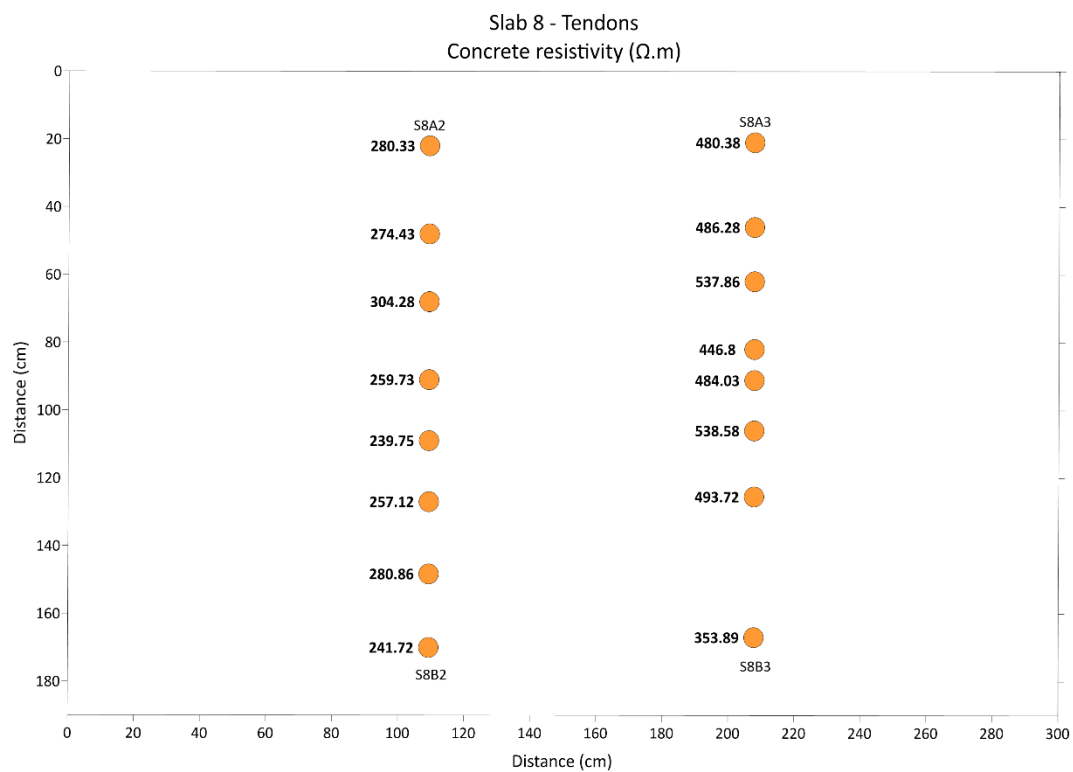


Figure 7-49. Surface resistivity mapping of concrete along prestressing tendons for slab 8 (47.8)

### A.4.3.2 Temperature reinforcement

Figure 7-50 to Figure 7-57 display the contour maps of the concrete surface resistivity measurements obtained at the locations identified with a '+' along the temperature reinforcement of slabs 1 to 8.

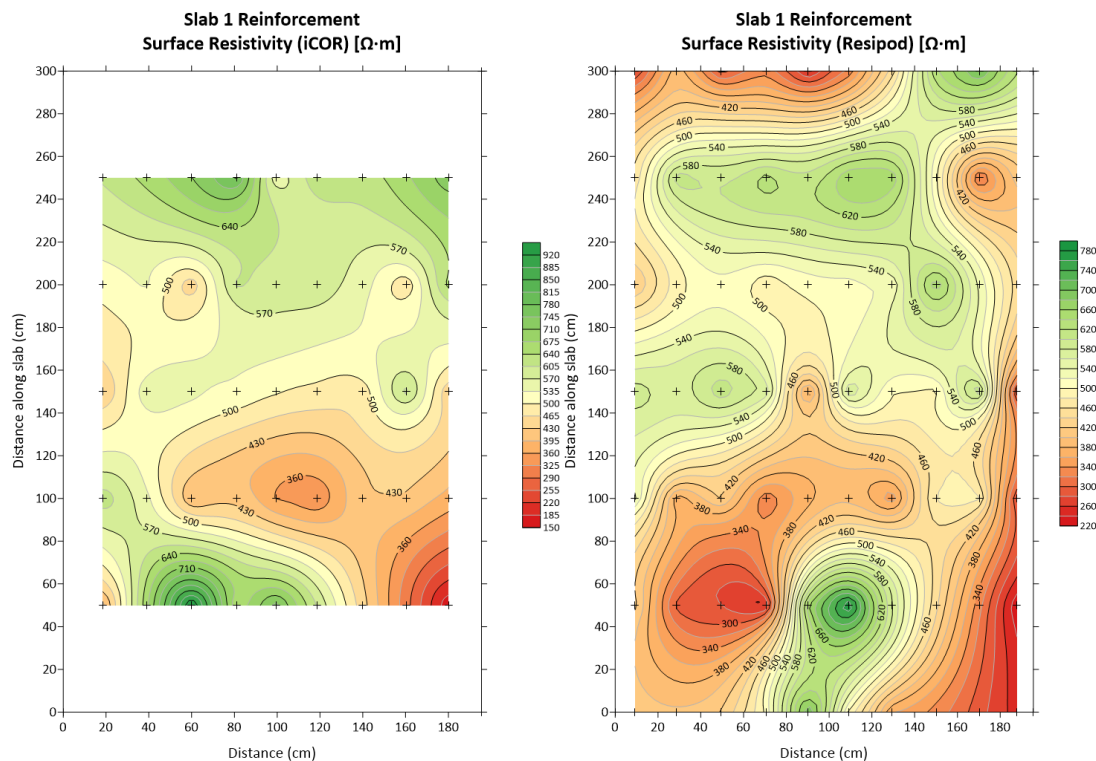


Figure 7-50. Surface resistivity mapping for slab 1 (47.6) from the iCOR device (L) and Resipod Wenner probe (R)



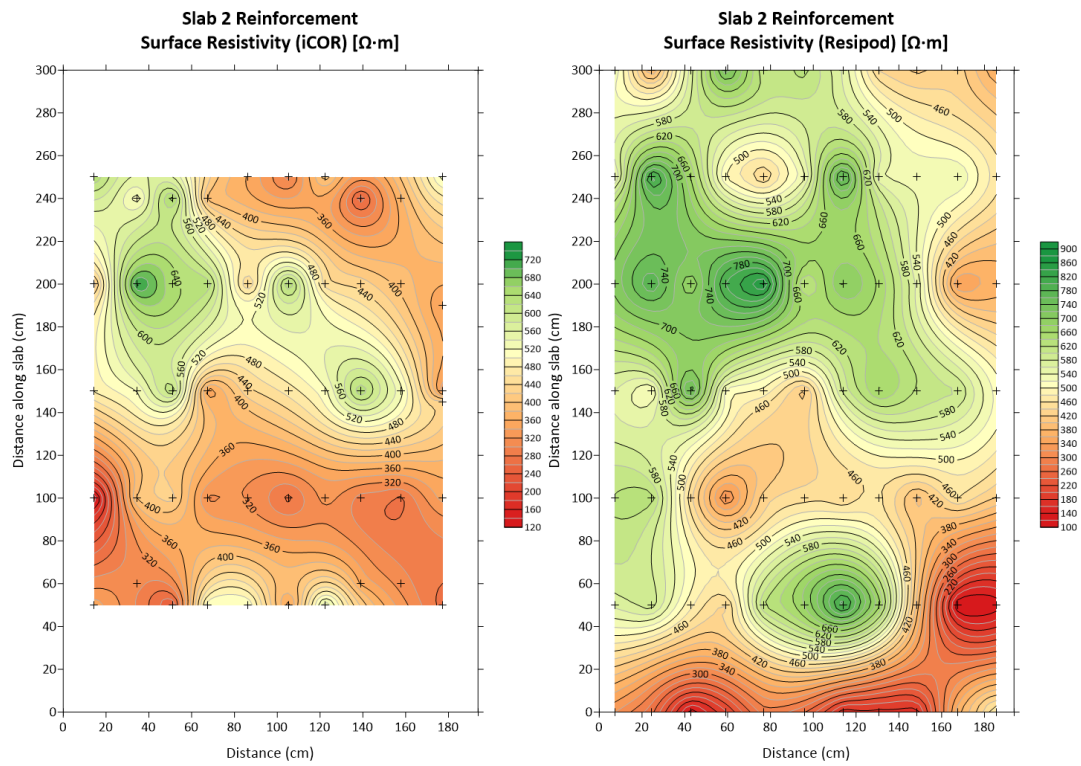


Figure 7-51. Surface resistivity mapping for slab 2 (47.2) from the iCOR device (L) and Resipod Wenner probe (R)

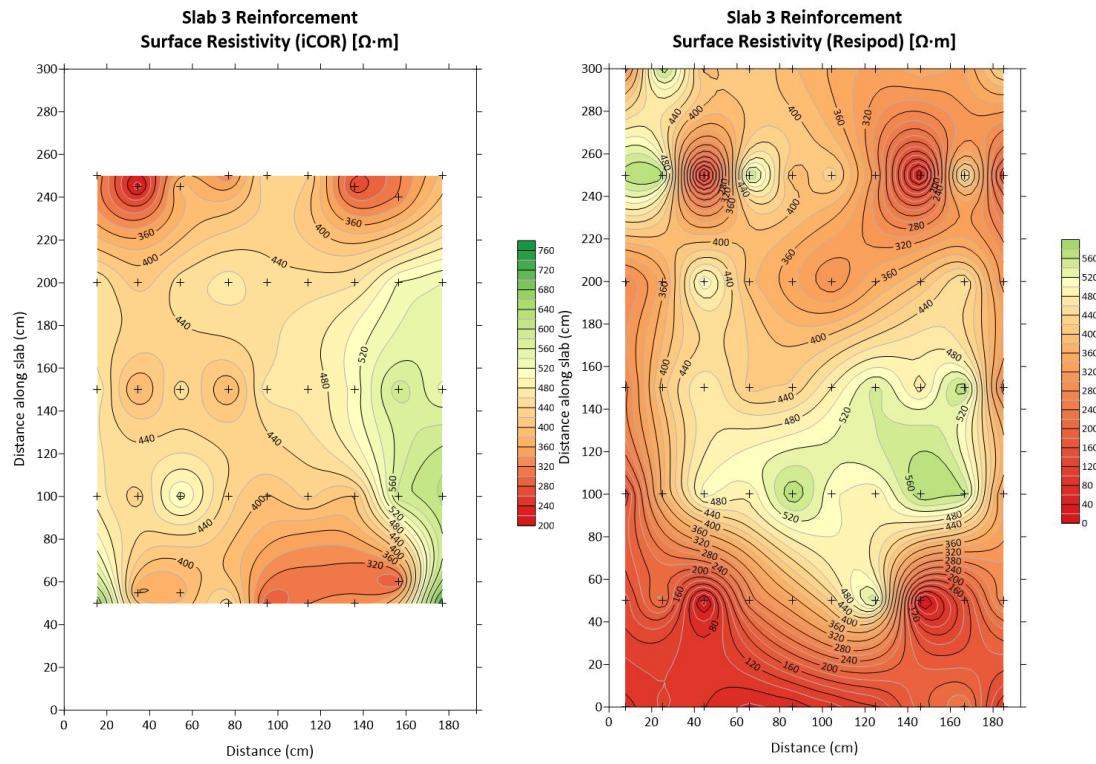


Figure 7-52. Surface resistivity mapping for slab 3 (47.5) from the iCOR device (L) and Resipod Wenner probe (R)

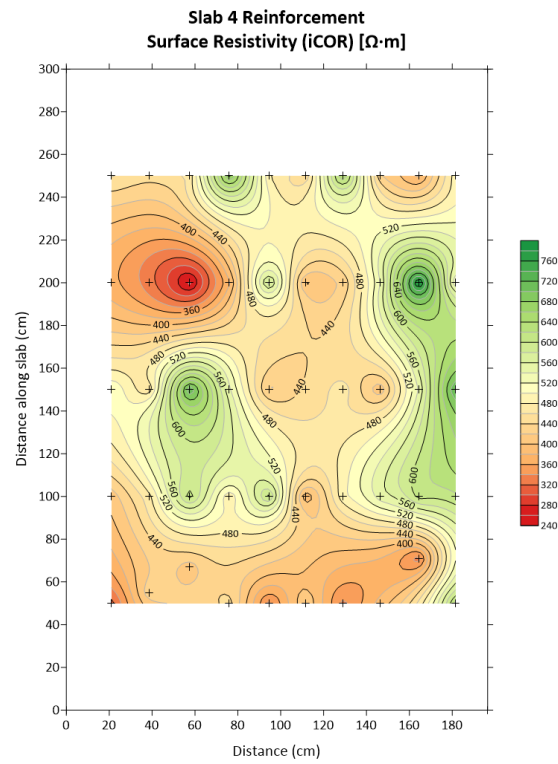


Figure 7-53. Surface resistivity mapping for slab 4 (47.1) from the iCOR device

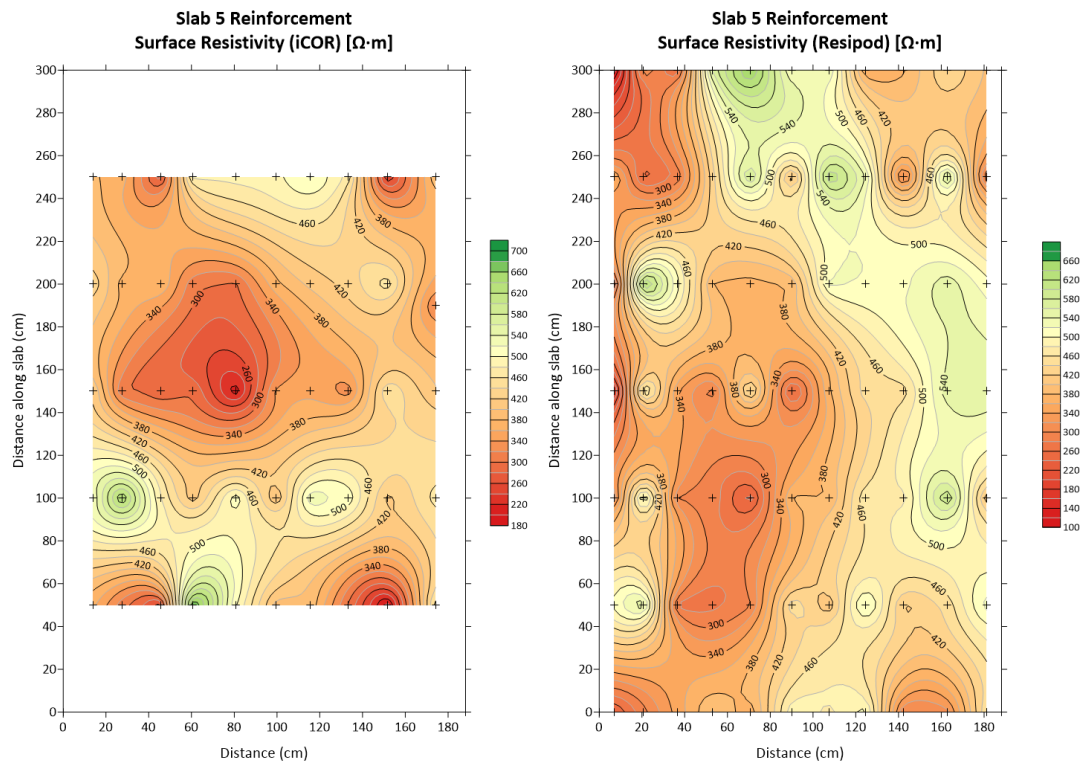


Figure 7-54. Surface resistivity mapping for slab 5 (47.3) from the iCOR device (L) and Resipod Wenner probe (R)

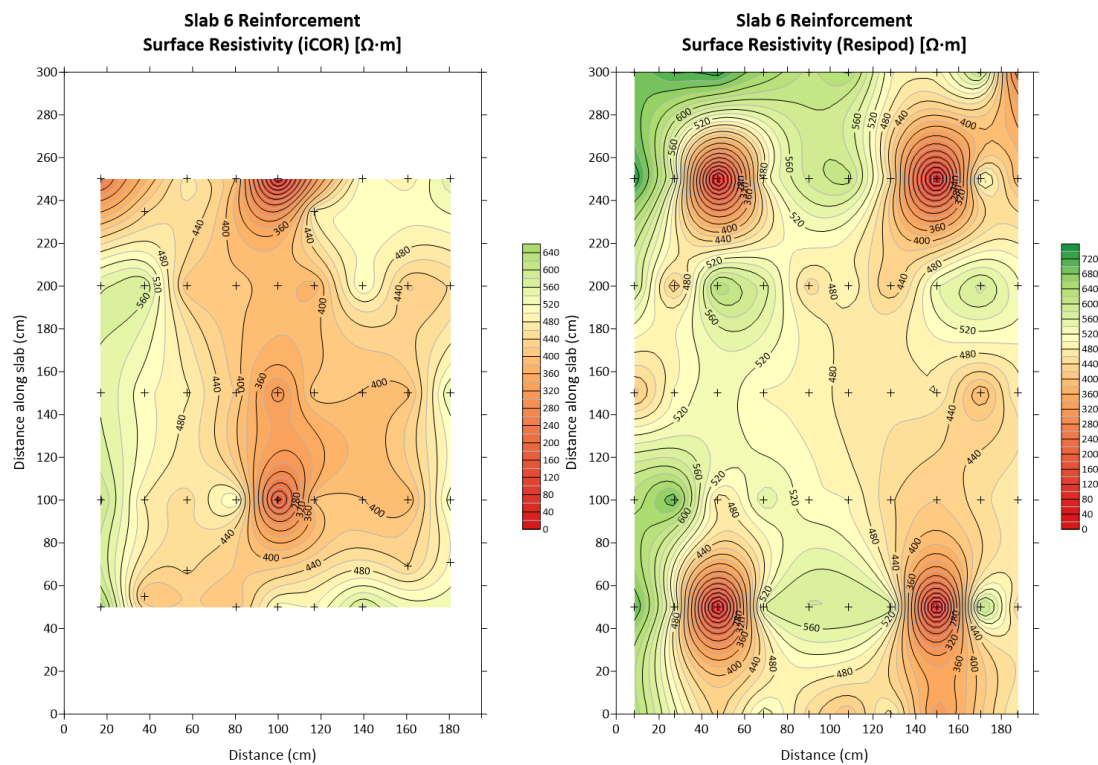


Figure 7-55. Surface resistivity mapping for slab 6 (47.4) from the iCOR device (L) and Resipod Wenner probe ®

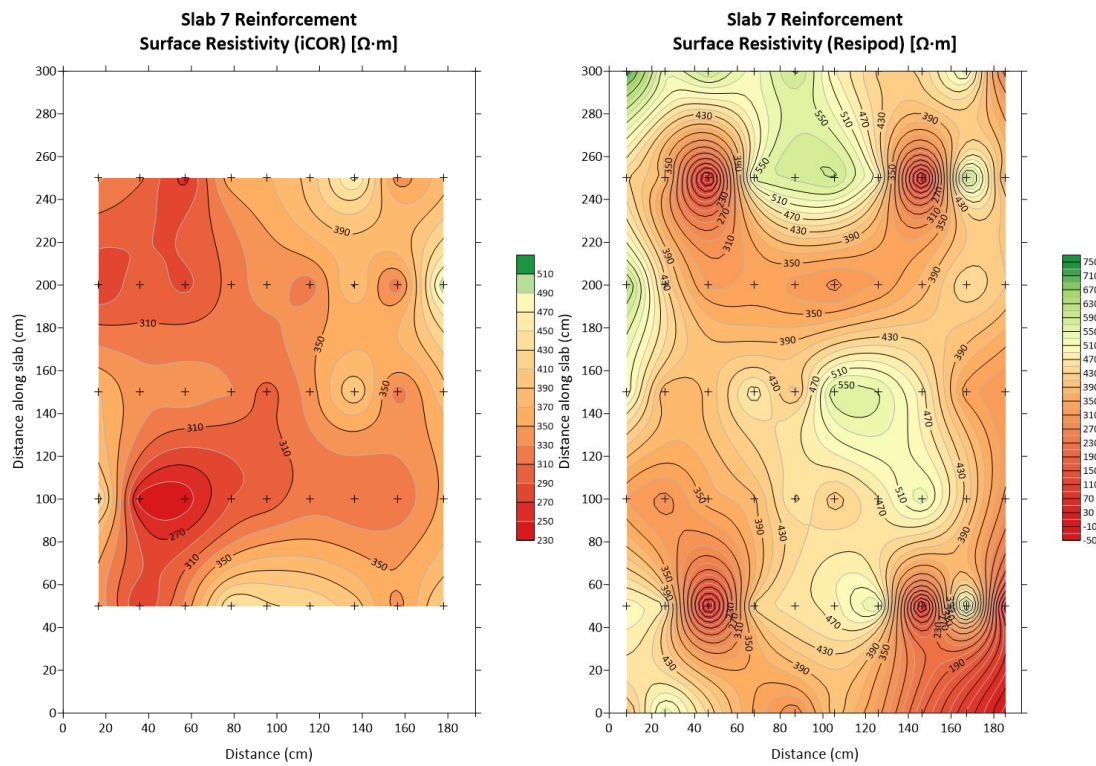


Figure 7-56. Surface resistivity mapping for slab 7 (47.7) from the iCOR device (L) and Resipod Wenner probe (R)



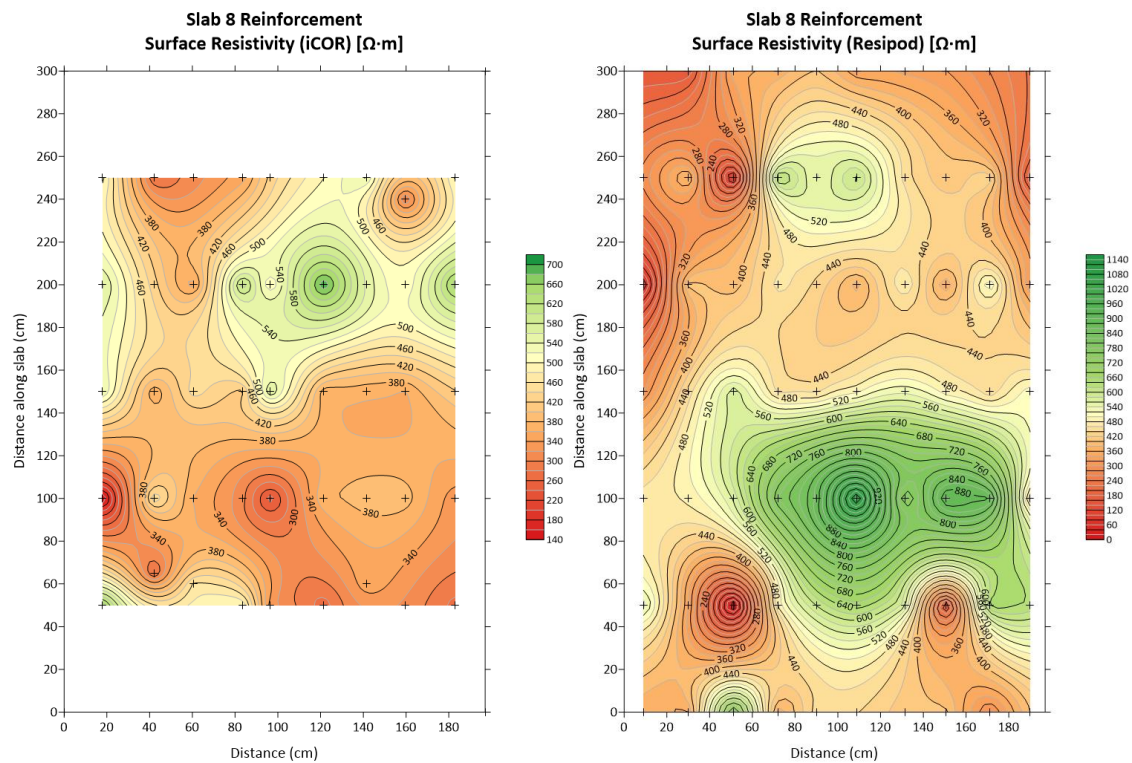


Figure 7-57. Surface resistivity mapping for slab 8 (47.8) from the iCOR device (L) and Resipod Wenner probe (R)

The average surface resistivity values along slab surfaces using the Resipod Wenner probe are shown in

Table 7-11. Although some variation can be observed in the maximum and minimum values due to variability in moisture after ponding, the average resistivity values were relatively constant for all slabs. Additionally, due to time restraints, slab 4 could not be tested with the Resipod after ponding.

Table 7-11. Average, maximum, and minimum surface resistivity readings [ $K\Omega \cdot cm$ ] for slabs using Resipod device

|                                                |      | S1<br>(47.6) | S2<br>(47.2) | S3<br>(47.5) | S4<br>(47.1) | S5<br>(47.3) | S6<br>(47.4) | S7<br>(47.7) | S8<br>(47.8) |
|------------------------------------------------|------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|
| <i>Measured<br/>parallel to rebar</i>          | Avg. | 47.0         | 50.0         | 35.6         | NA           | 41.5         | 52.5         | 43.8         | 46.1         |
|                                                | Max. | 77.9         | 86.3         | 58.6         | NA           | 66.4         | 74.2         | 72.9         | 111.7        |
|                                                | Min. | 22.5         | 11.9         | 5.5          | NA           | 11           | 25.0         | 8.7          | 5.6          |
| <i>Measured<br/>perpendicular to<br/>rebar</i> | Avg. | 44.7         | 50.3         | 38.2         | NA           | 38.7         | NA           | 39.0         | 43.6         |
|                                                | Max. | 95.4         | 91.6         | 69.9         | NA           | 74.3         | NA           | 55.2         | 109.4        |
|                                                | Min. | 14.6         | 9.7          | 4.1          | NA           | 7.6          | NA           | 11.8         | 9.7          |

#### A.4.4 Integrated corrosion profile of temperature reinforcement

The following figures provide a consolidated spatial mapping of the corrosion results for the temperature reinforcement. Figure 7-58 and Figure 7-59 present these data points within a reconstructed plan view, aligning the individual slabs according to their original relative positions within the bridge deck to illustrate longitudinal corrosion trends.



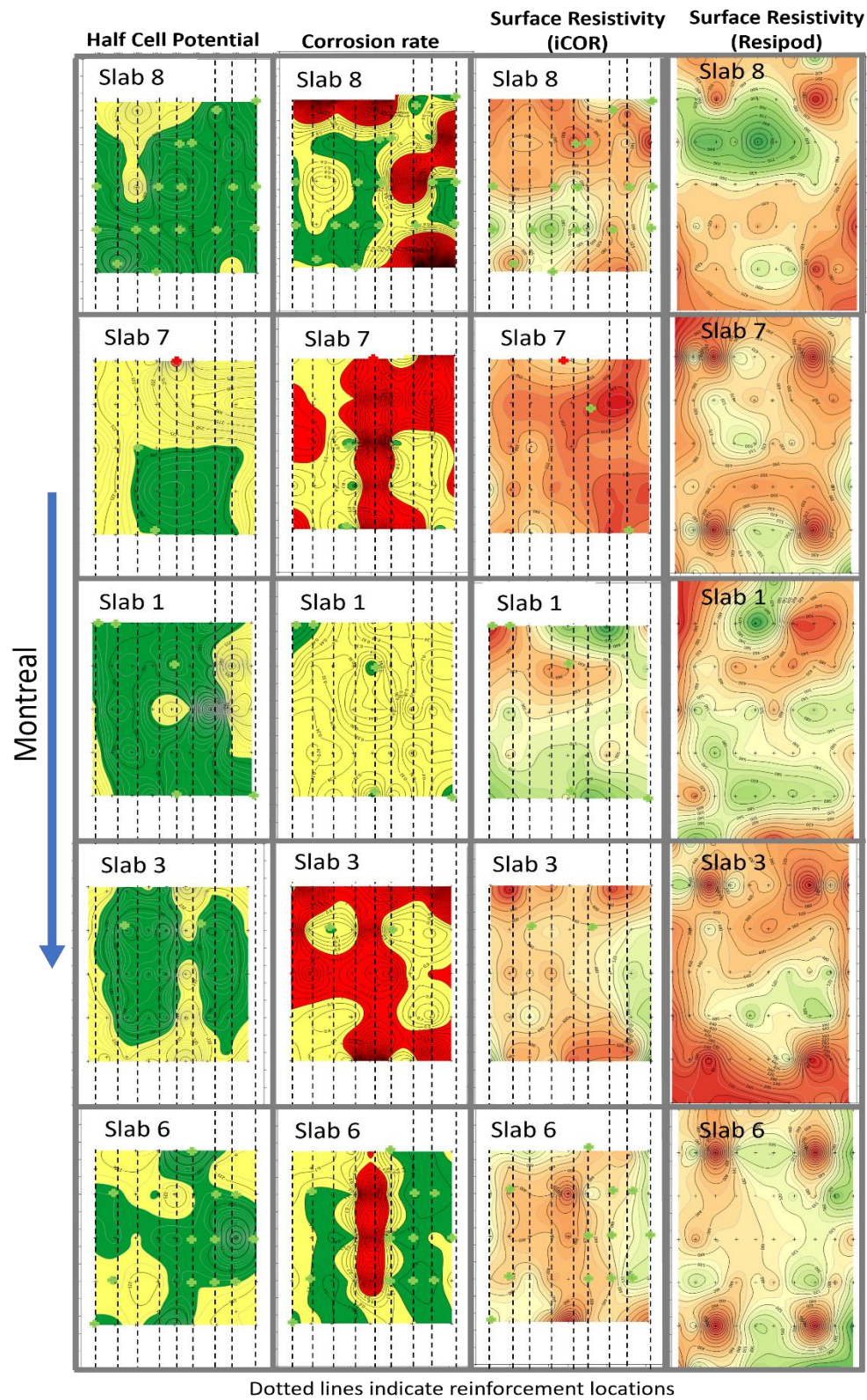


Figure 7-58. Corrosion and resistivity mapping for connected slabs (approximate reinforcement locations indicated by black lines)

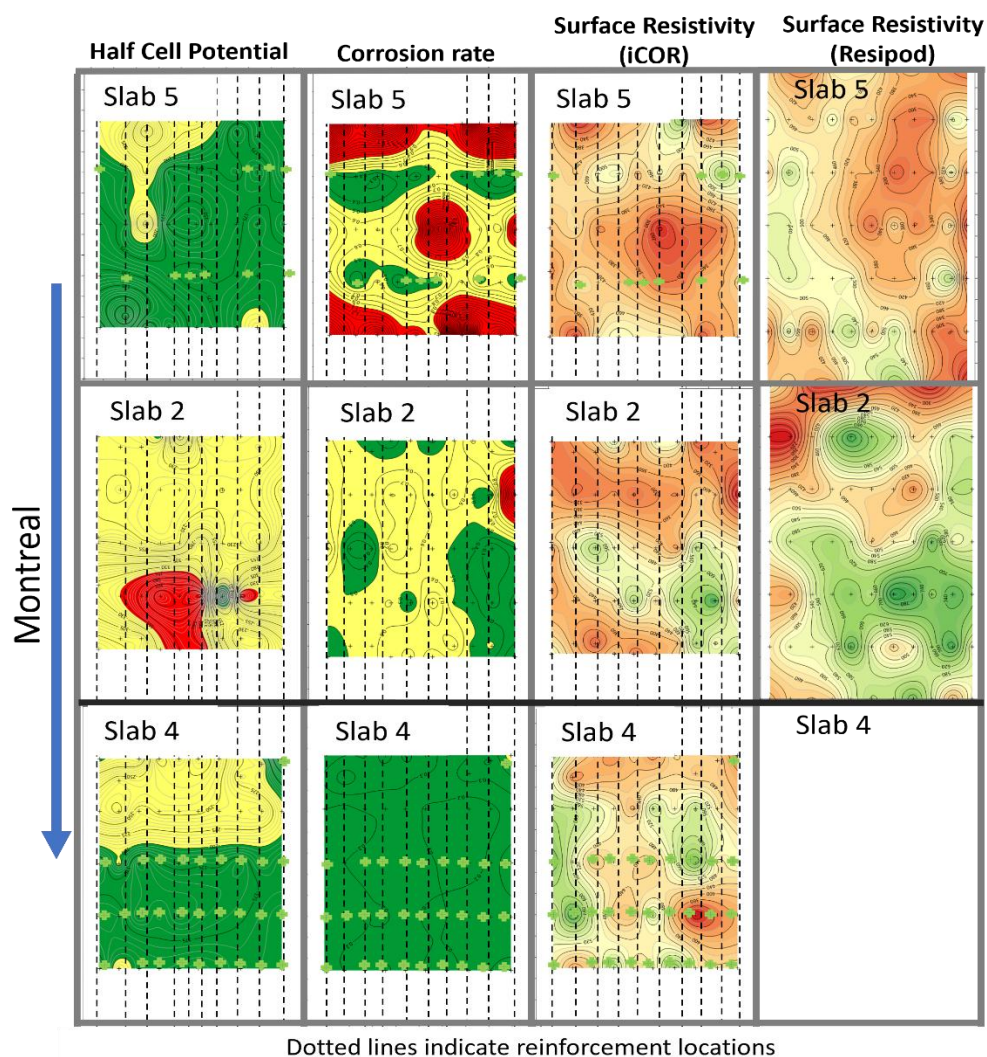
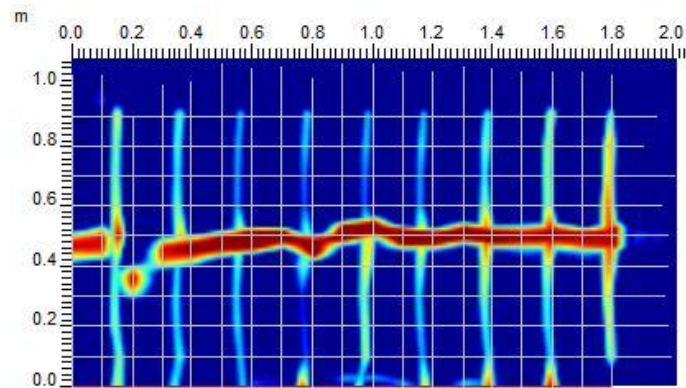


Figure 7-59. Corrosion and resistivity mapping for slabs 5 and 2 (connected) and slab 4 (approximate reinforcement locations indicated by dotted lines)

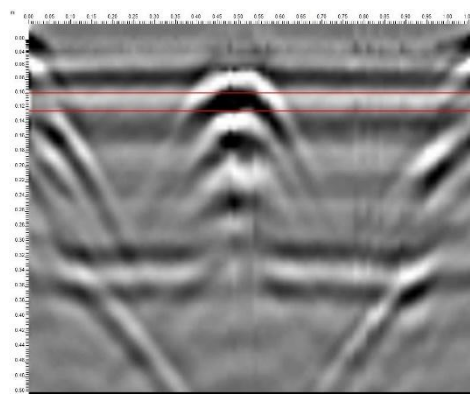
### A.5 Gound penetrating radar

The resulting GPR soffit grid scans for all slabs, conducted at a certain slice depth, are depicted in Figure 7-60 to Figure 7-75. Strong GPR signal responses are depicted by red areas, whereas intermediate and weak signal responses are represented by yellow and blue, respectively. Red signals typically denote high-intensity reflections, indicating a strong contrast in di-electric properties at an interface between two media, such as embedded reinforcement within the concrete. Yellow signals typically represent intermediate reflections, suggesting moderate contrasts in di-electric properties that point to less distinct features. Blue signals generally indicate low-intensity reflections, associated with more uniform concrete and less reflective areas.



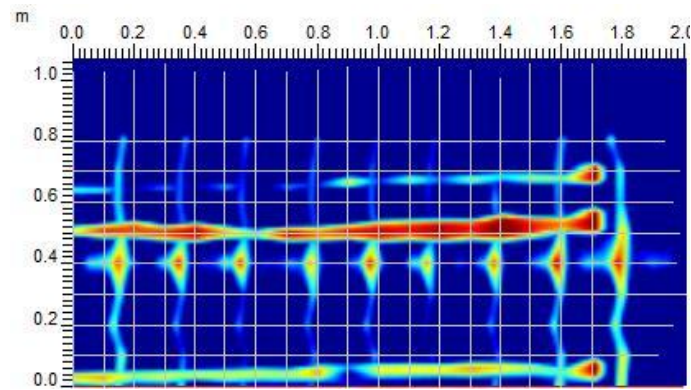


(a)

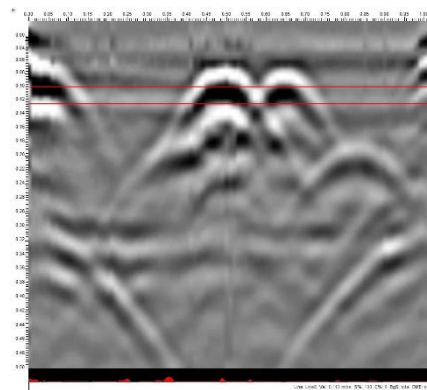


(b)

Figure 7-60 S1A1-S1B1: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 0.7 m slice view from edge A – Slab section view

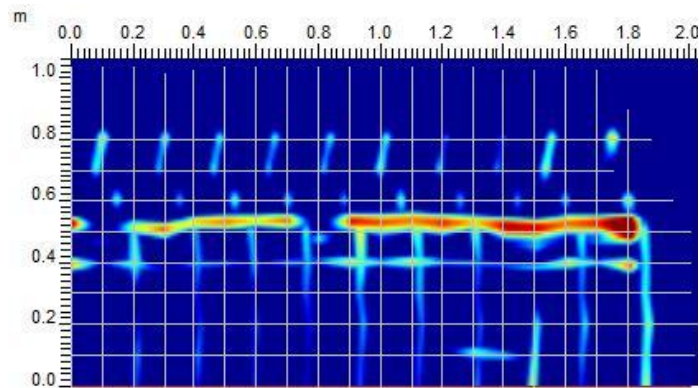


(a)

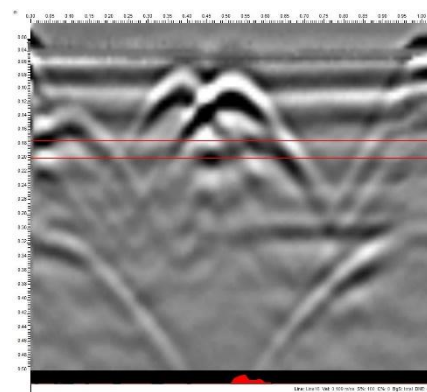


(b)

Figure 7-61 S1A2-S1B2: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 0.7 m slice view from edge A – Slab section view

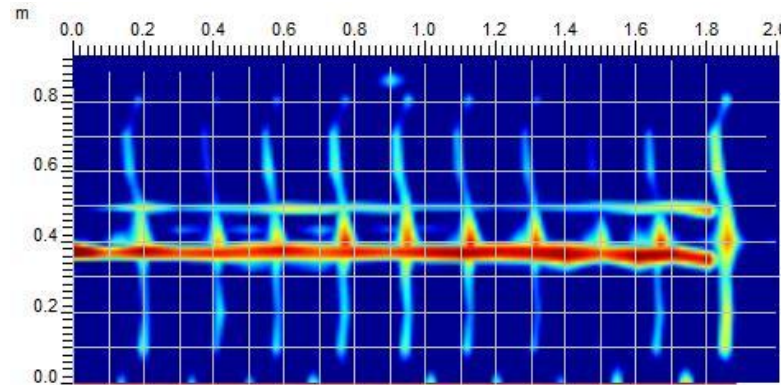


(a)

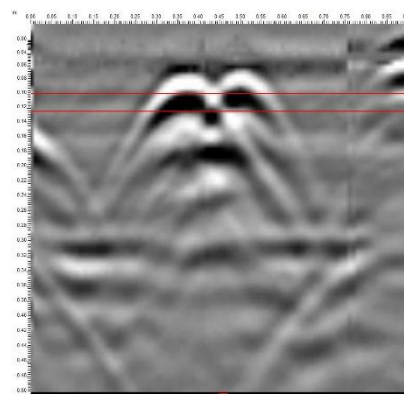


(b)

Figure 7-62 S2A2-S2B2: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 1.5 m slice view from edge A – Slab section view

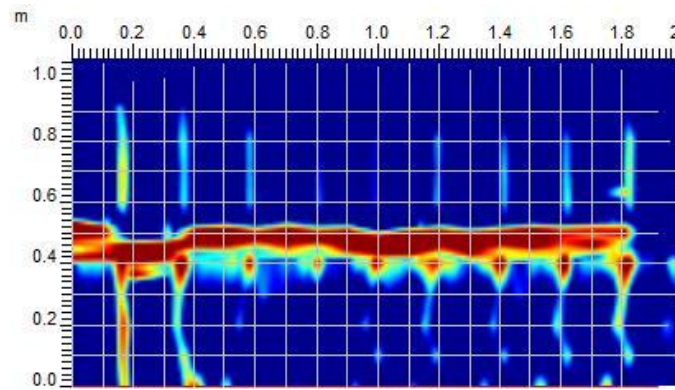


(a)

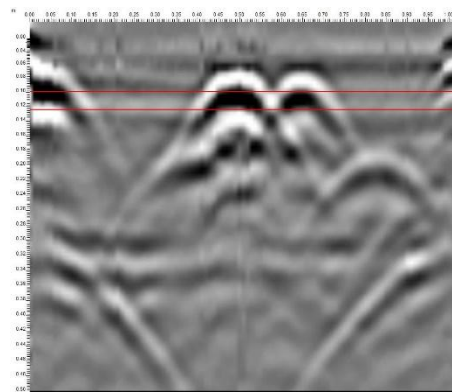


(b)

Figure 7-63 S2A3-S2B3: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and  
(b) 1.5 m slice view from edge A – Slab section view

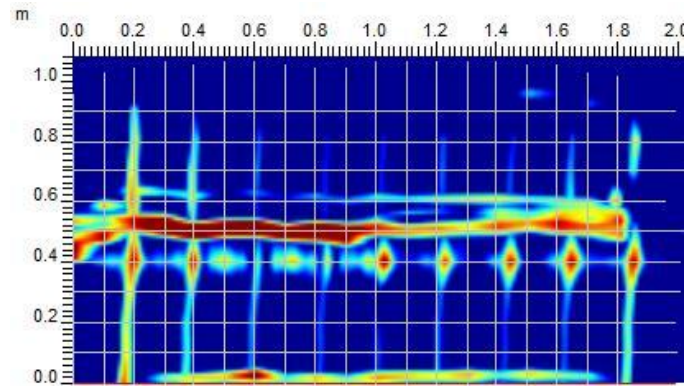


(a)

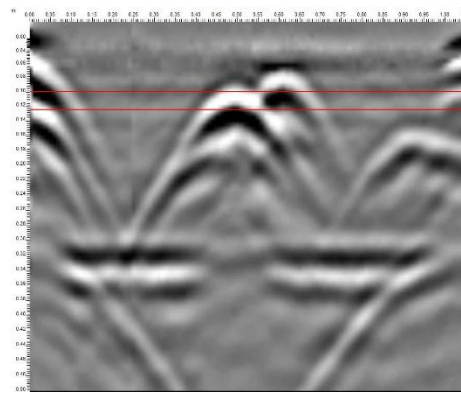


(b)

Figure 7-64 S3A1-S3B1: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 0.7 m slice view from edge A – Slab section view



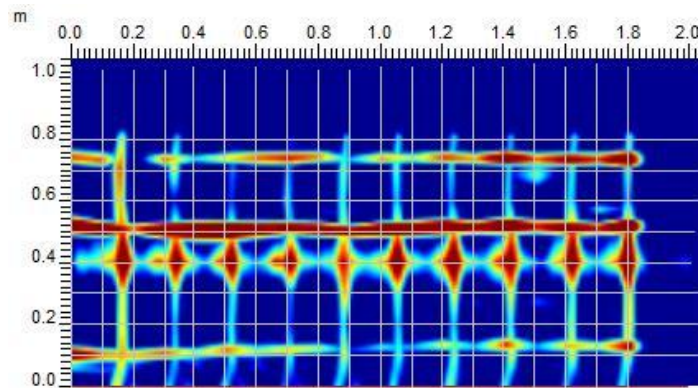
(a)



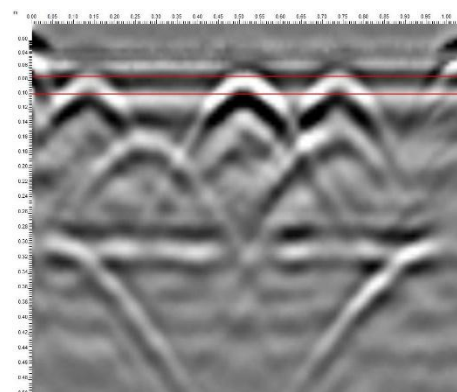
(b)

Figure 7-65 S3A2-S3B2: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 0.7 m slice view from edge A – Slab section view



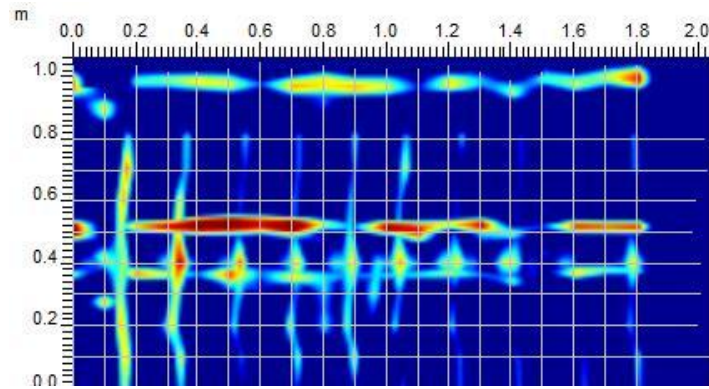


(a)

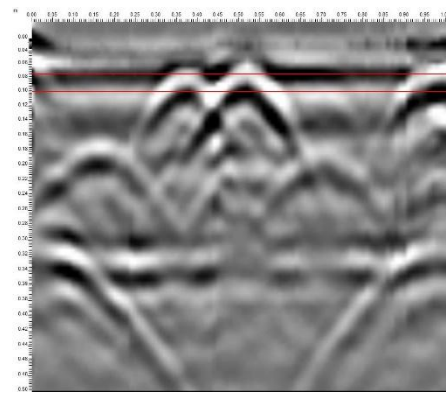


(b)

Figure 7-66 S4A2-S4B2: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 1.5 m slice view from edge A – Slab section view

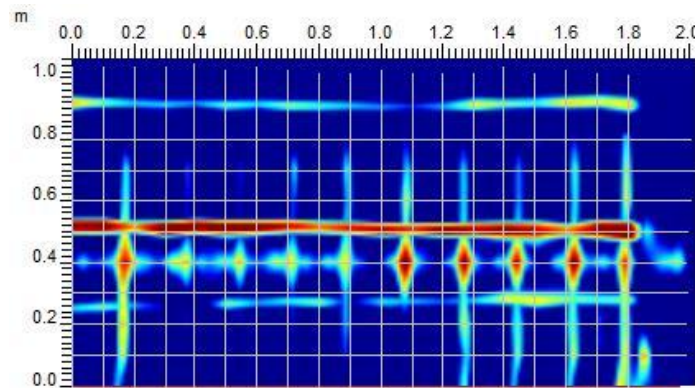


(a)

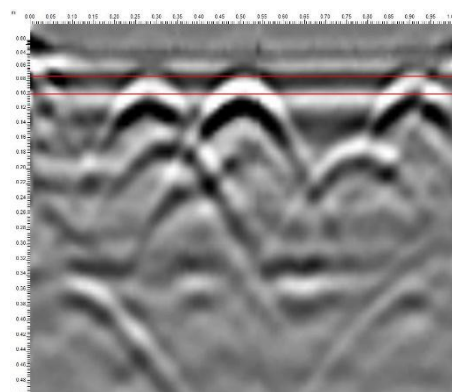


(b)

Figure 7-67 S4A3-S4B3: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 1.5 m slice view from edge A – Slab section view

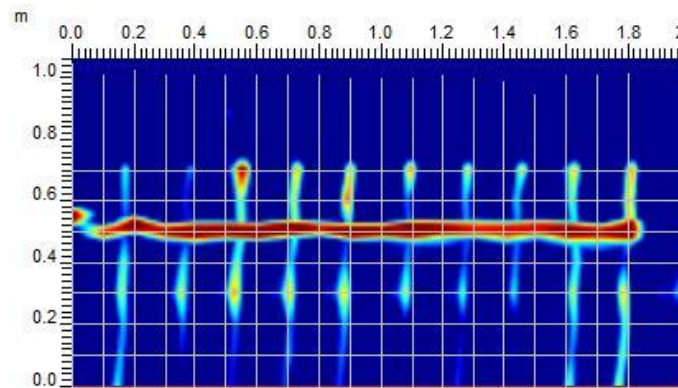


(a)

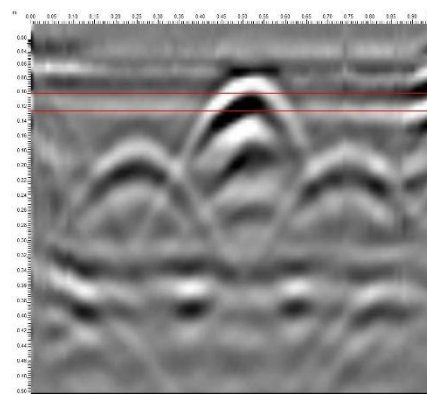


(b)

Figure 7-68 S5A2-S5B2: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 1.5 m slice view from edge A – Slab section view

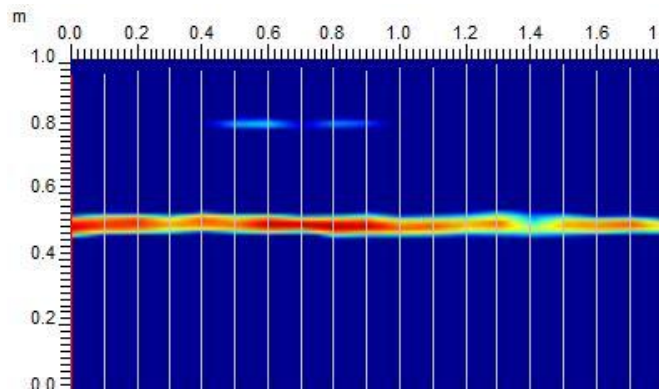


(a)

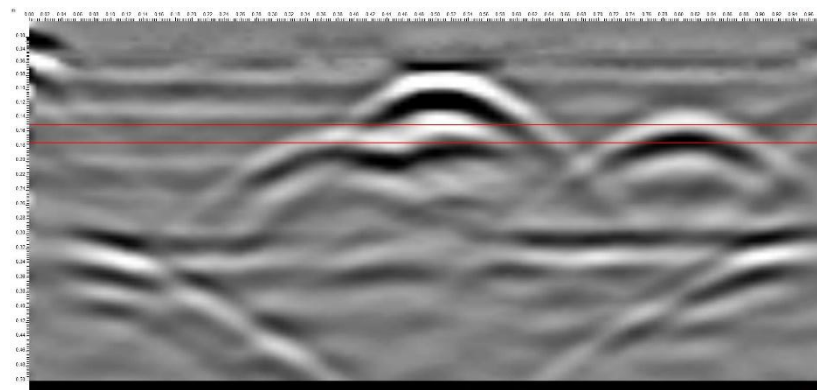


(b)

Figure 7-69 S5A3-S5B3: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 1.5 m slice view from edge A – Slab section view

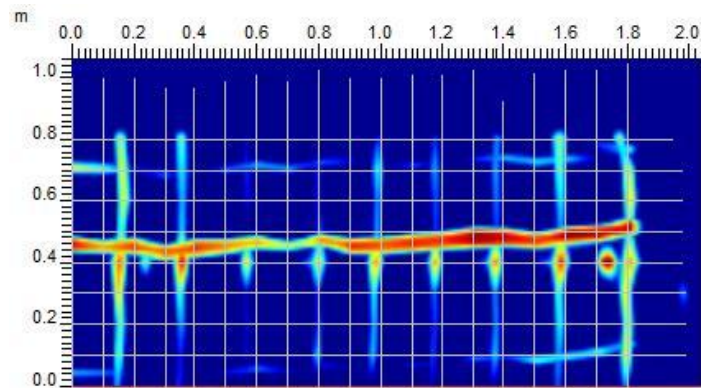


(a)

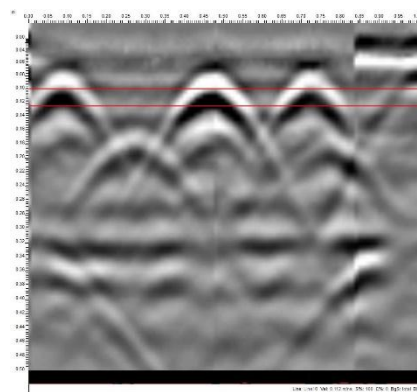


(b)

Figure 7-70 S6A2-S6B2: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 0.5 m slice view from edge A – Slab section view

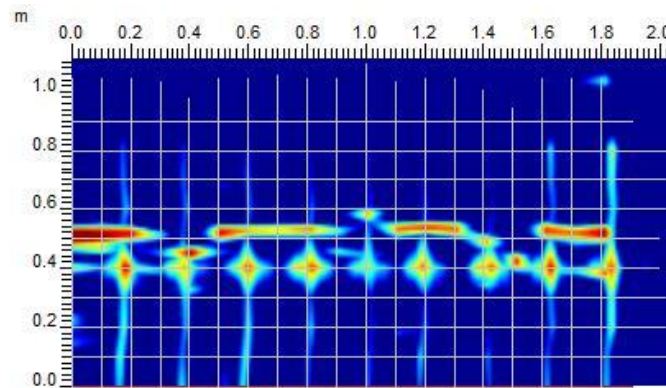


(a)

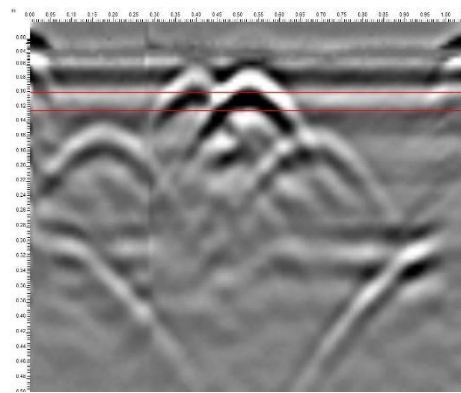


(b)

Figure 7-71 S6A3-S6B3: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 1.5 m slice view from edge A – Slab section view

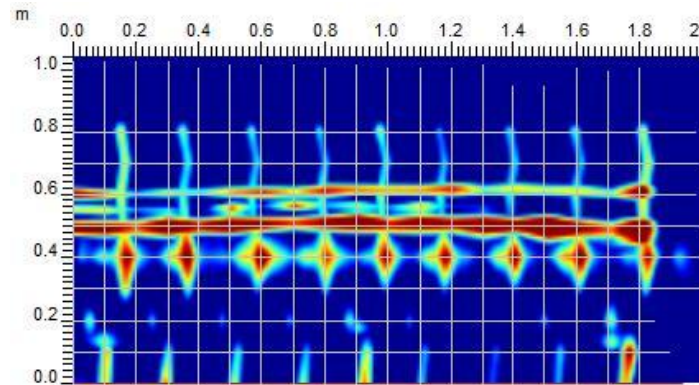


(a)

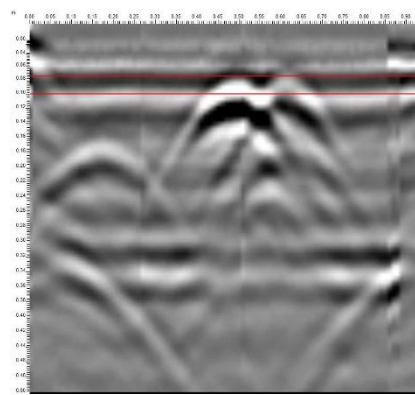


(b)

Figure 7-72 S7A2-S7B2: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 1.5 m slice view from edge A – Slab section view



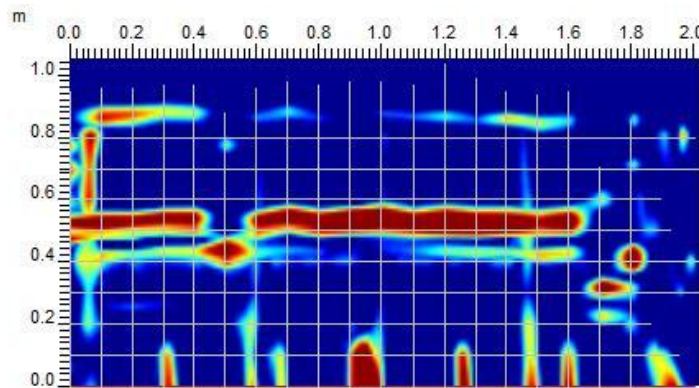
(a)



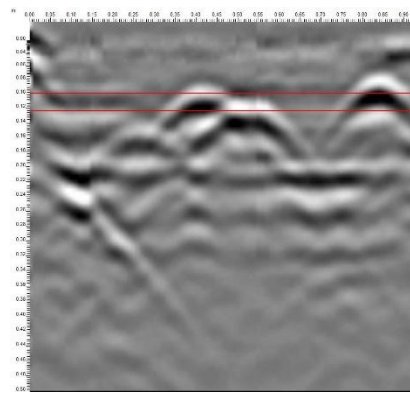
(b)

Figure 7-73 S7A3-S7B3: Soffit GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 1.5 m slice view from edge A – Slab section view



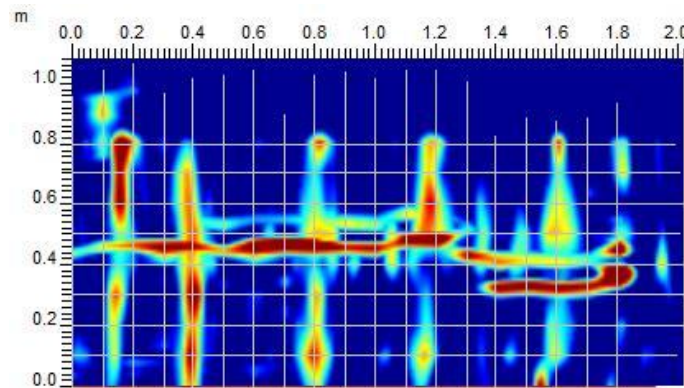


(a)

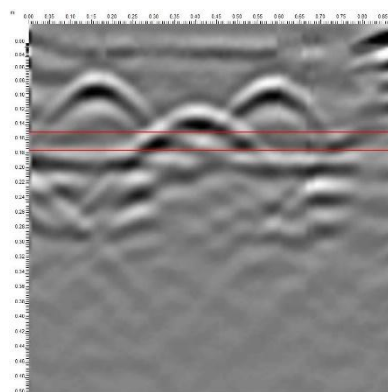


(b)

Figure 7-74 S8A2-S8B2: Top surface GPR grid view at (a) 0.125 – 0.150 m slice depth (slab plan view); and (b) 1.5 m slice view from edge A – Slab section view



(a)



(b)

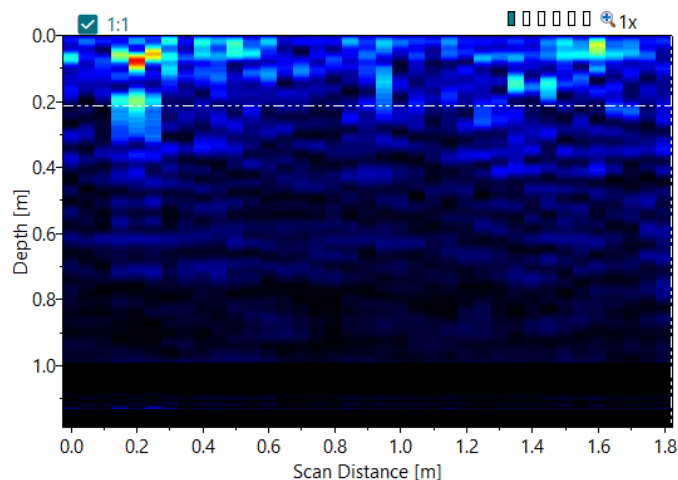
Figure 7-75 S8A3-S8B3: Top surface GPR grid view at (a) 0.150 – 0.175 m slice depth (slab plan view); and (b) 1.5 m slice view from edge A – Slab section view

## A.6 Ultrasonic pulse echo

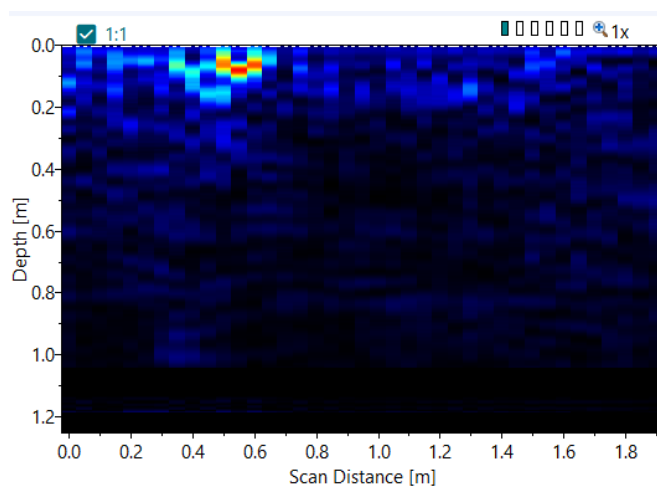
Figure 7-76 to Figure 7-91 provide a sectional view of the slabs, where UPE strength signals along the tendons are displayed in colour format. Both sets of measurements were acquired along the centerline of the tendons and at 5 cm intervals, starting from edge A and progressing to edge B.

In UPE scans, colours are used to represent different signal strengths: red areas indicate strong UPE signal responses, yellow areas show intermediate responses, and blue areas represent weak responses. Red signals can either denote distinct and well-defined reflections from the back wall, where the signal reflects off the opposite concrete surface, suggesting good-quality concrete that is free from defects or internal flaws, or indicate scattered strong reflections from the internal flaws themselves, indicating lower concrete quality. Such concrete sections will lack a clear and well-

defined back-wall reflection as a result of these internal reflectors (flaws) or surface degradation. Yellow signals suggest that the concrete in those regions may exhibit some level of deterioration or inconsistency, while blue signals suggest that the concrete is free from significant defects.

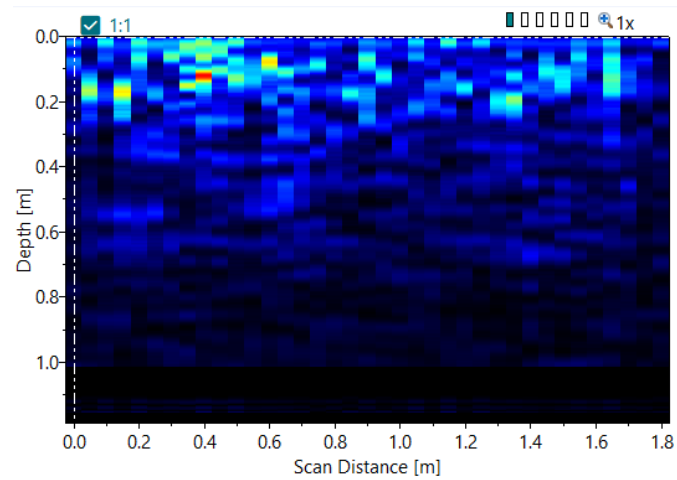


(a)

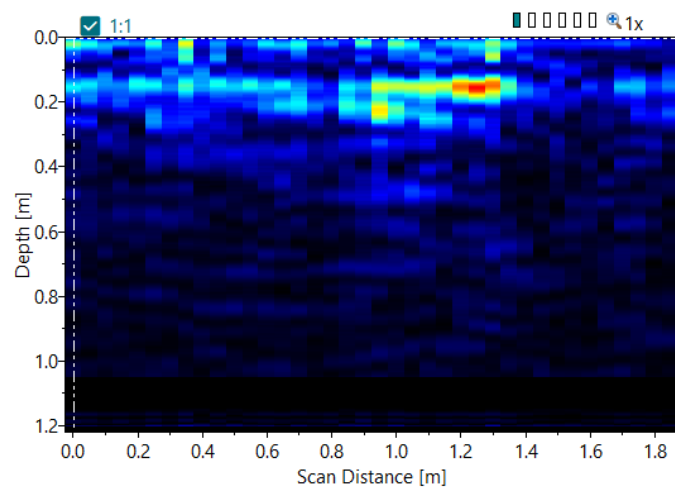


(b)

Figure 7-76 S1A1-S1B1: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface

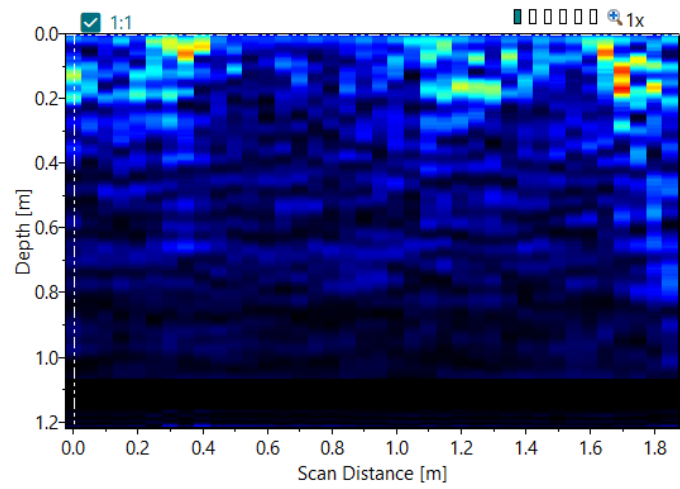


(a)

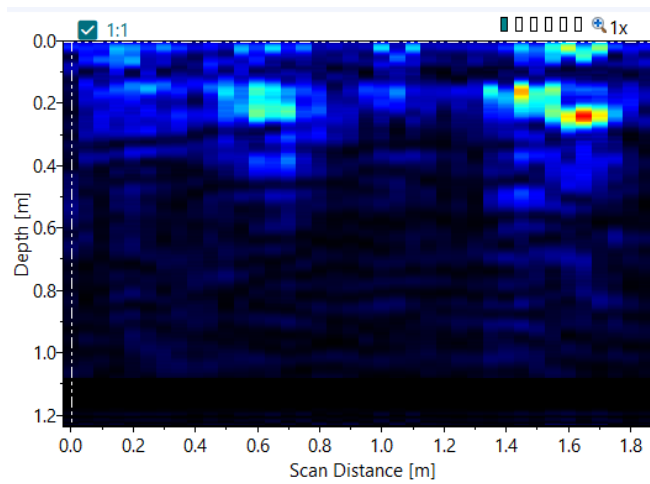


(b)

Figure 7-77 S1A2-S1B2: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface

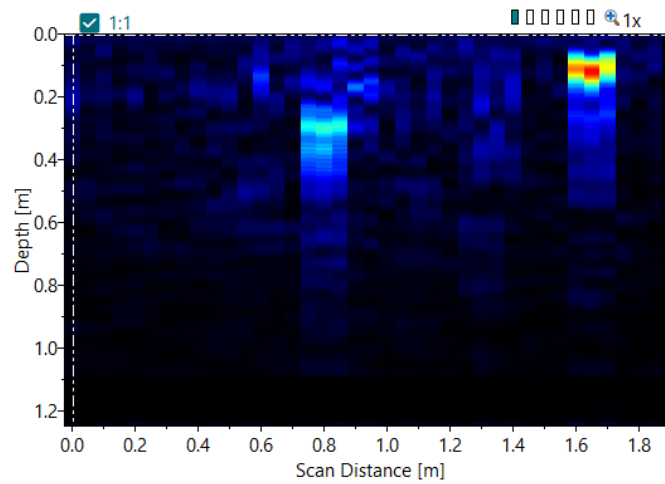


(a)

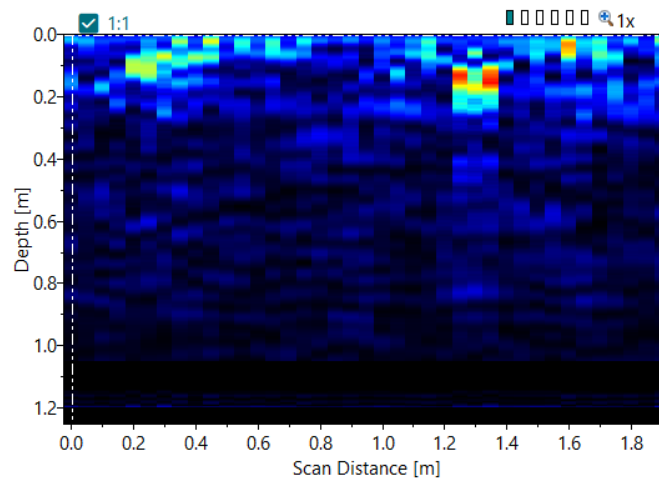


(b)

Figure 7-78 S2A2-S2B2: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface

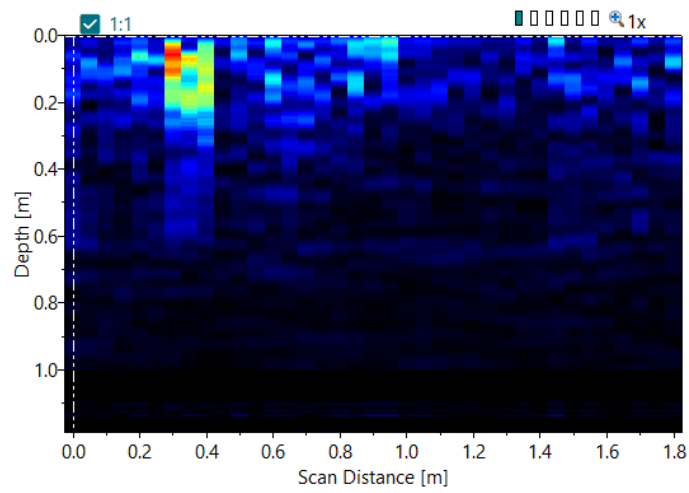


(a)

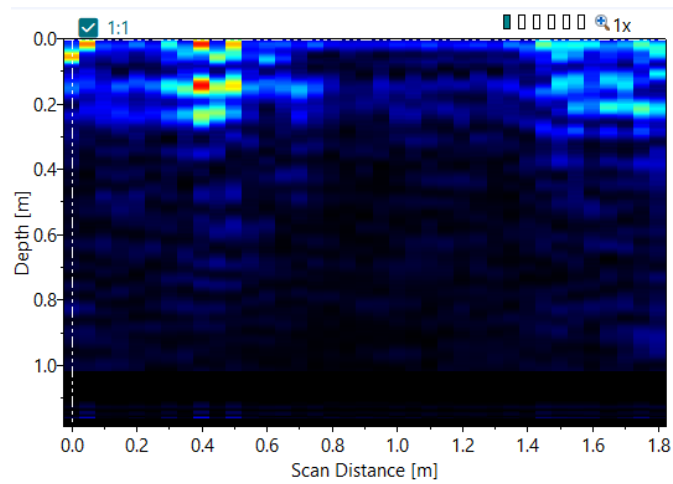


(b)

Figure 7-79 S2A3-S2B3: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface

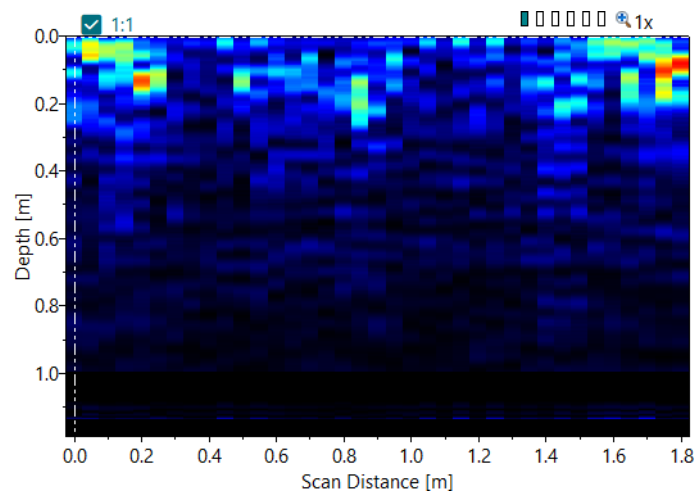


(a)

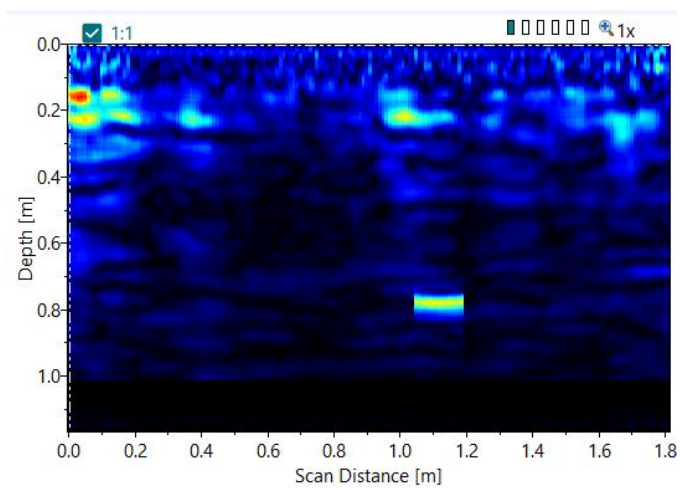


(b)

Figure 7-80 S3A1-S3B1: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface



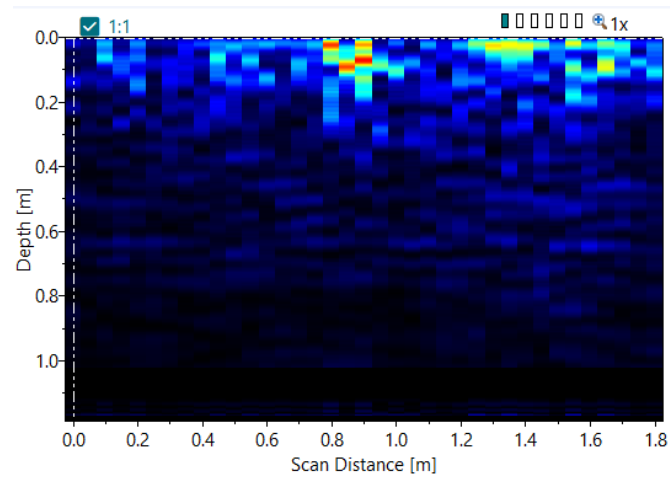
(a)



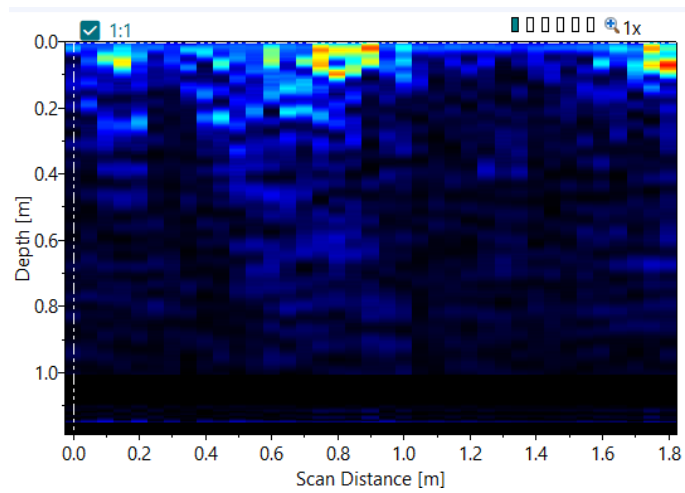
(b)

Figure 7-81 S3A2-S3B2: UPE data measured at 5 cm intervals along the (a) soffit; and 1 cm intervals along the (b) top concrete surface



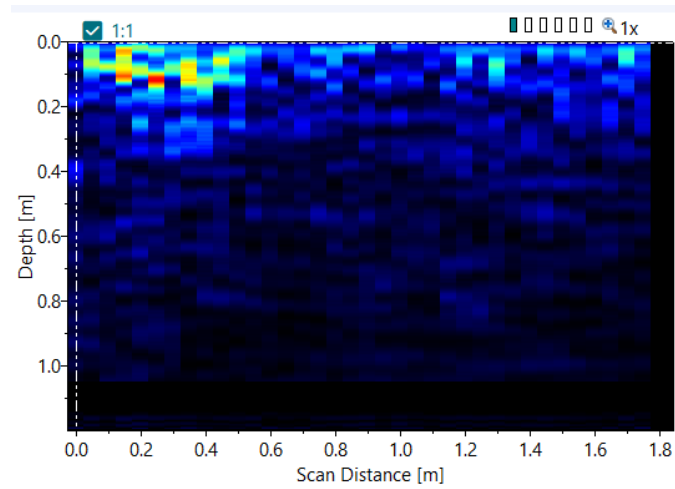


(a)

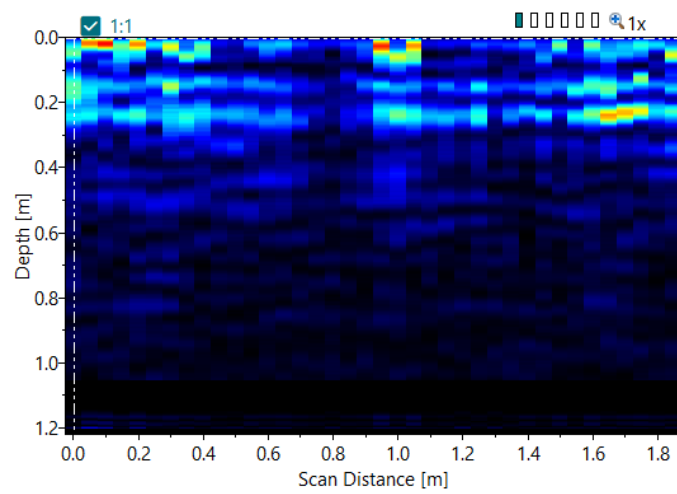


(b)

Figure 7-82 S4A2-S4B2: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface

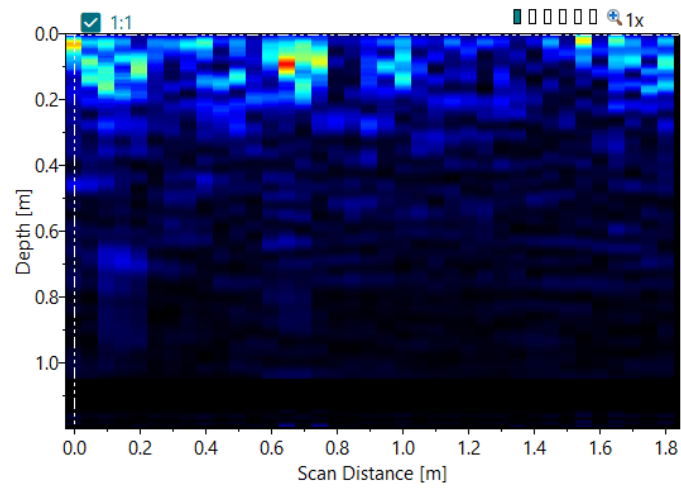


(a)

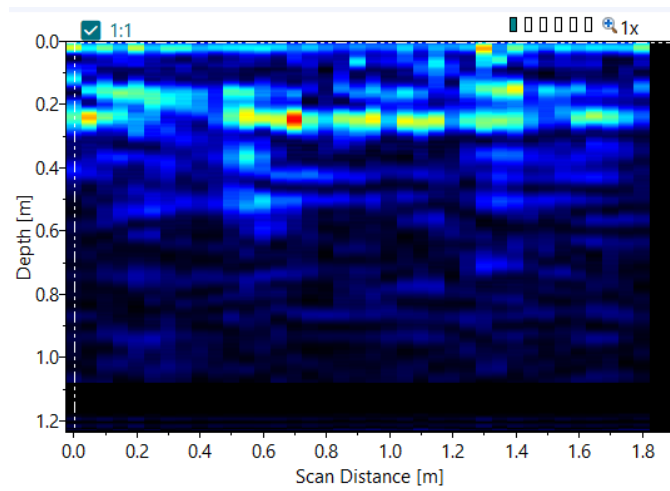


(b)

Figure 7-83 S4A3-S4B3: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface

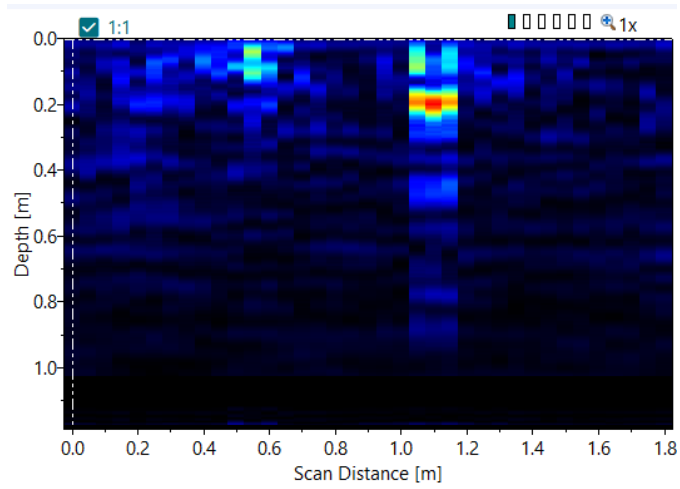


(a)

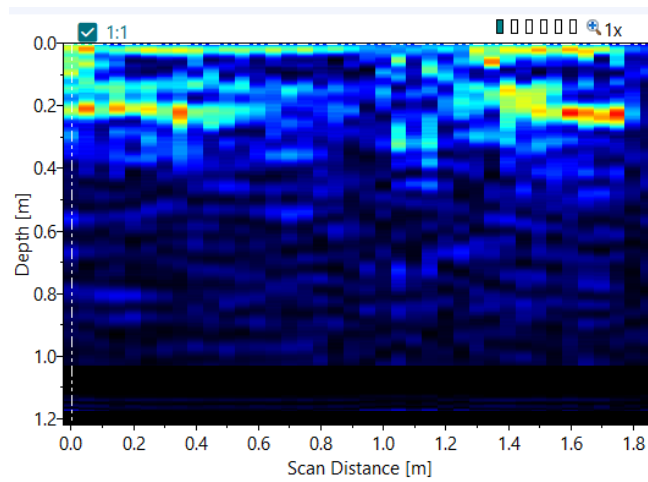


(b)

Figure 7-84 S5A2-S5B2: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface

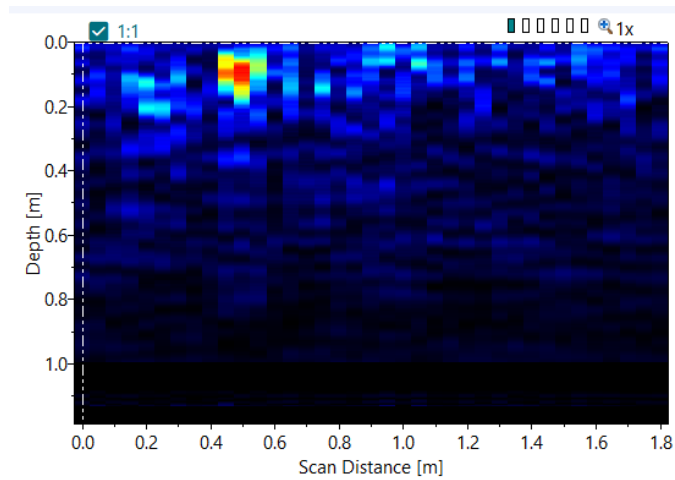


(a)

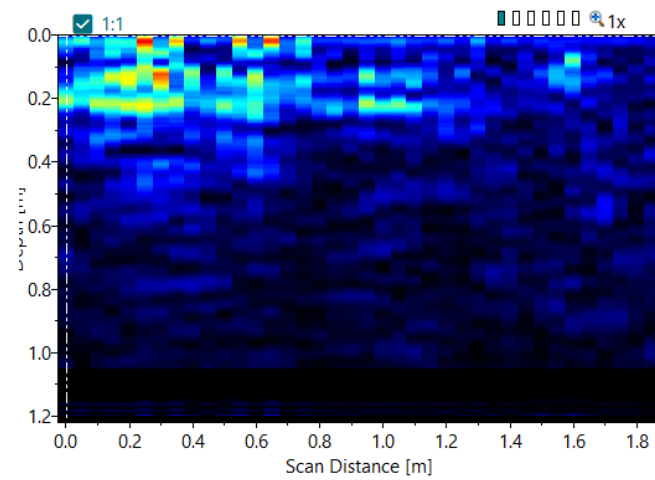


(b)

Figure 7-85 S5A3-S5B3: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface

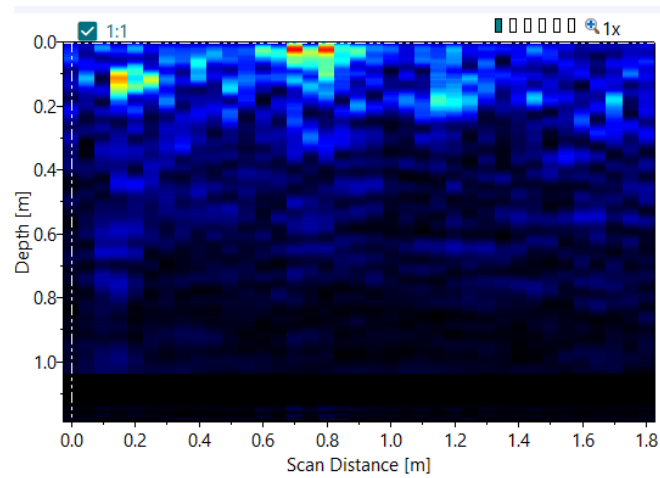


(a)

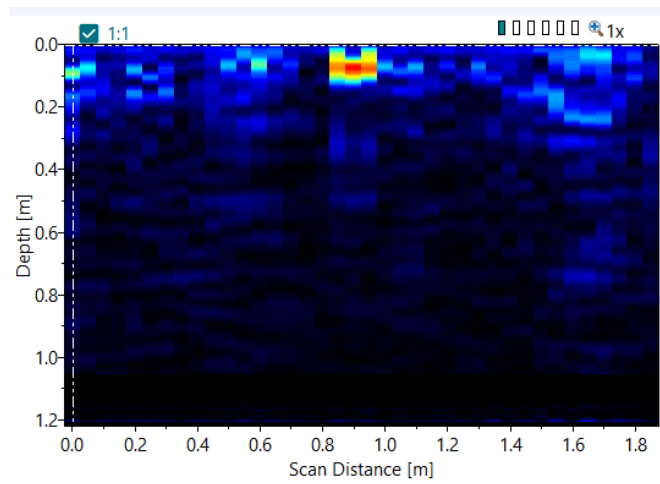


(b)

Figure 7-86 S6A2-S6B2: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface

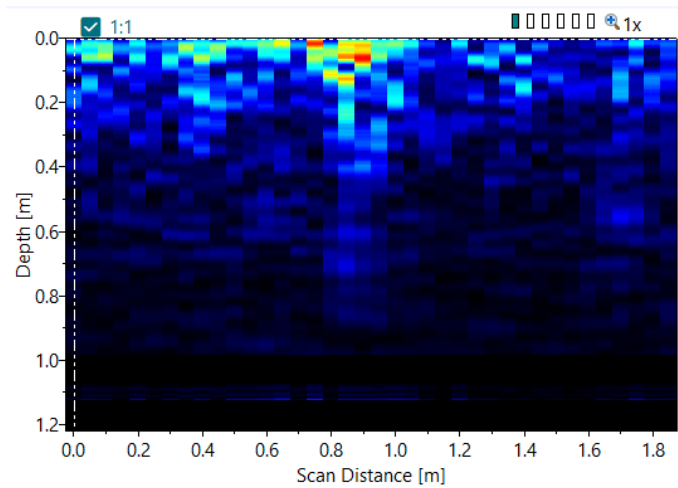


(a)

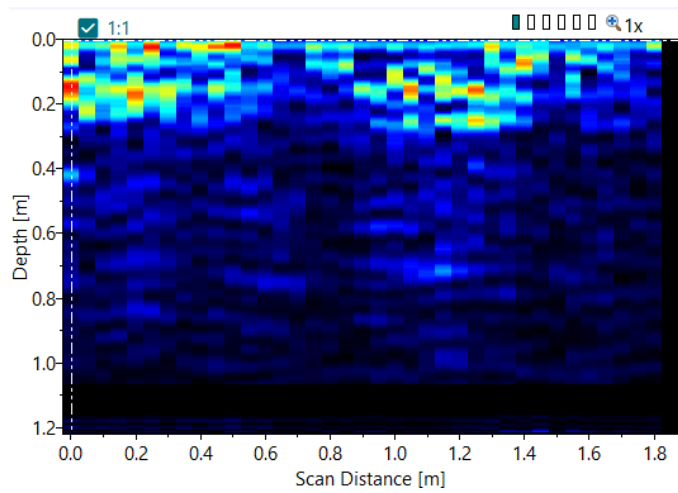


(b)

Figure 7-87 S6A3-S6B3: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface

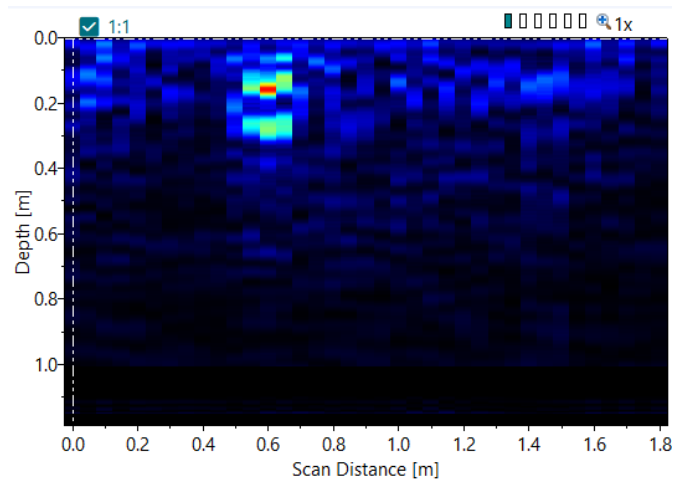


(a)

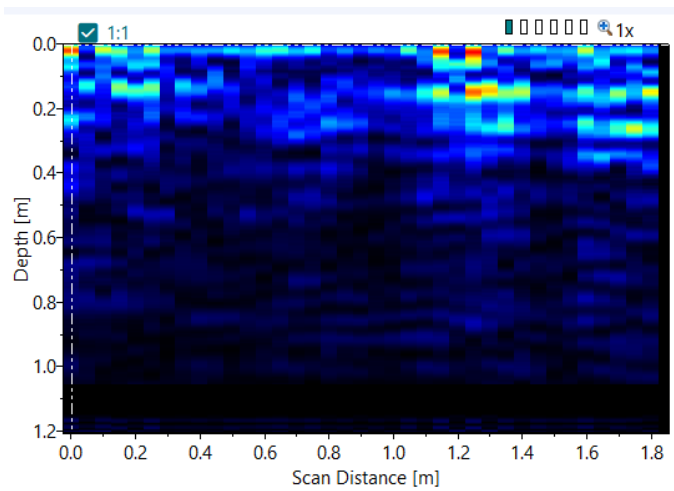


(b)

Figure 7-88 S7A2-S7B2 UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface



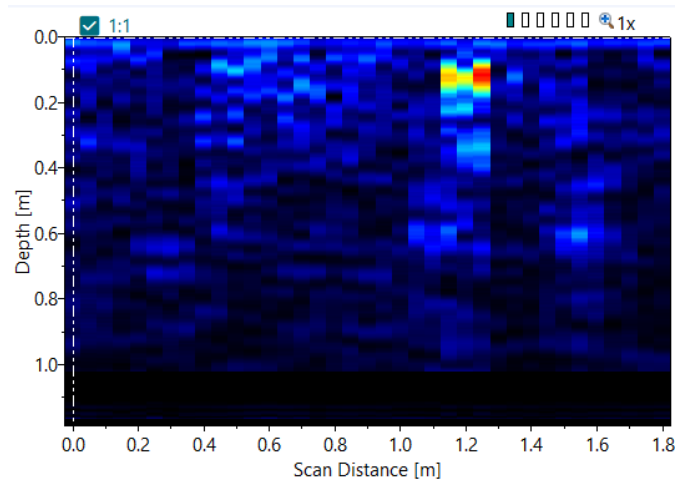
(a)



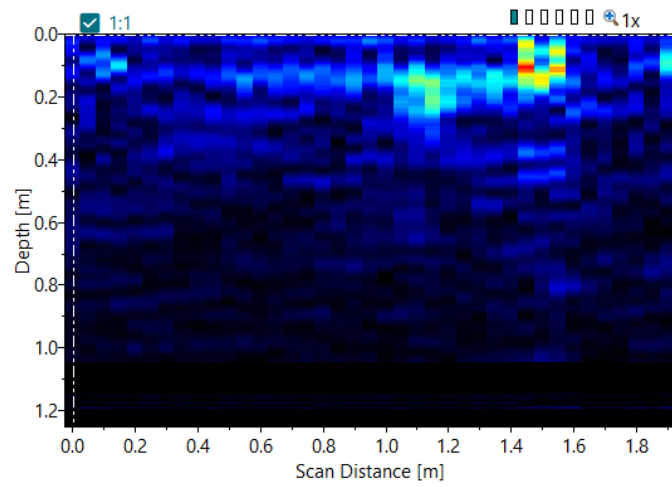
(b)

Figure 7-89 S7A3-S7B3: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface



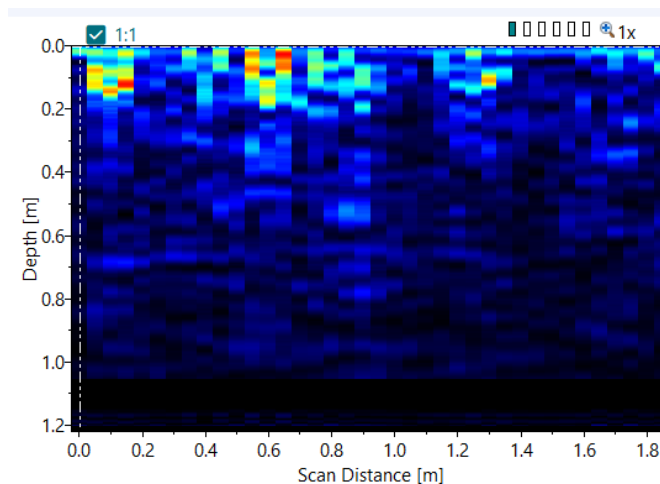


(a)

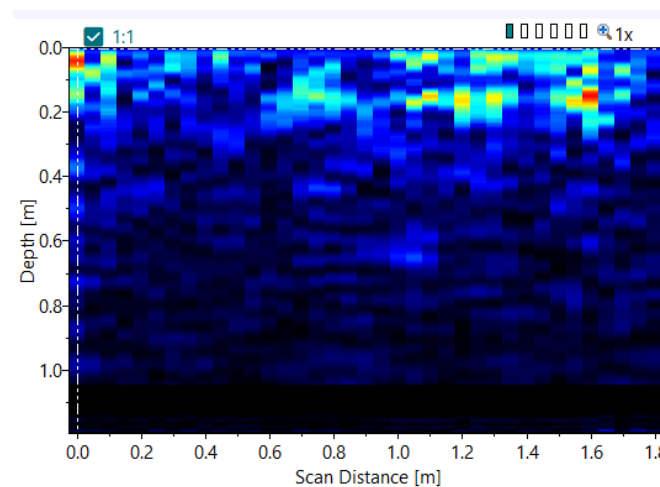


(b)

Figure 7-90 S8A2-S8B2: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface



(a)



(b)

Figure 7-91 S8A3-S8B3: UPE data measured at 5 cm intervals along the (a) soffit; and (b) top concrete surface

## A.7 Impact echo

The graphs in Figure 20 and Figure 21 display the void locations along S1A1-S1B1 and S1A2-S1B2 detected during testing, represented as points along the distance axis. Each point on the distance axis represents a void identified through post-processing analysis of the device's data. Waves produced by the stress pulse reflect off internal or external interferences and travel back to the surface impact point. Post-processing techniques are then applied to determine the depth of discontinuities or establish the boundary of the member. Similarly to UPE scans, regions

containing internal flaws exhibit either the absence of a back-wall reflection due to internal reflectors, surface degradation, or both, or signal reflections indicating the depth of the flaw.

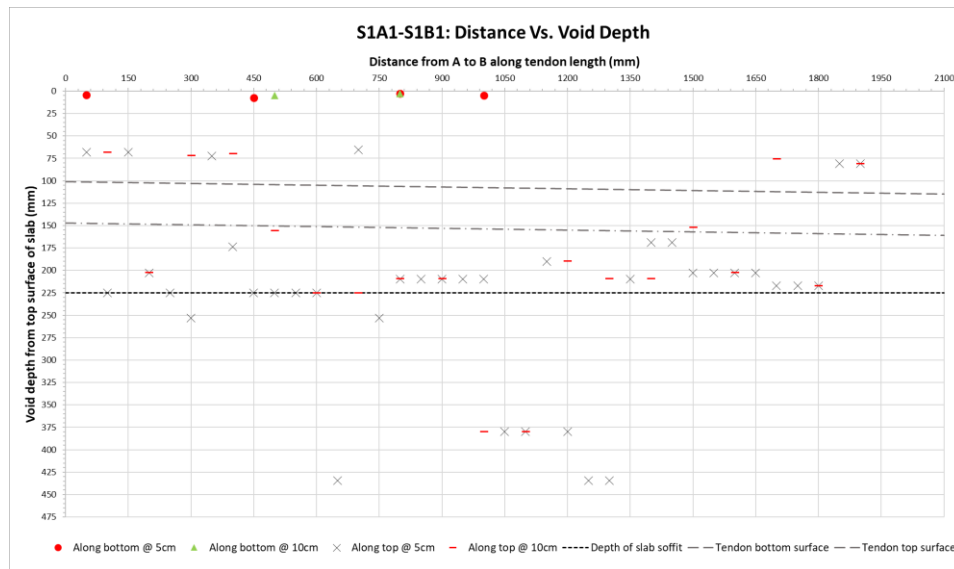


Figure 7-92 S1A1-S1B1: IE data at 5 cm intervals from both slab surfaces

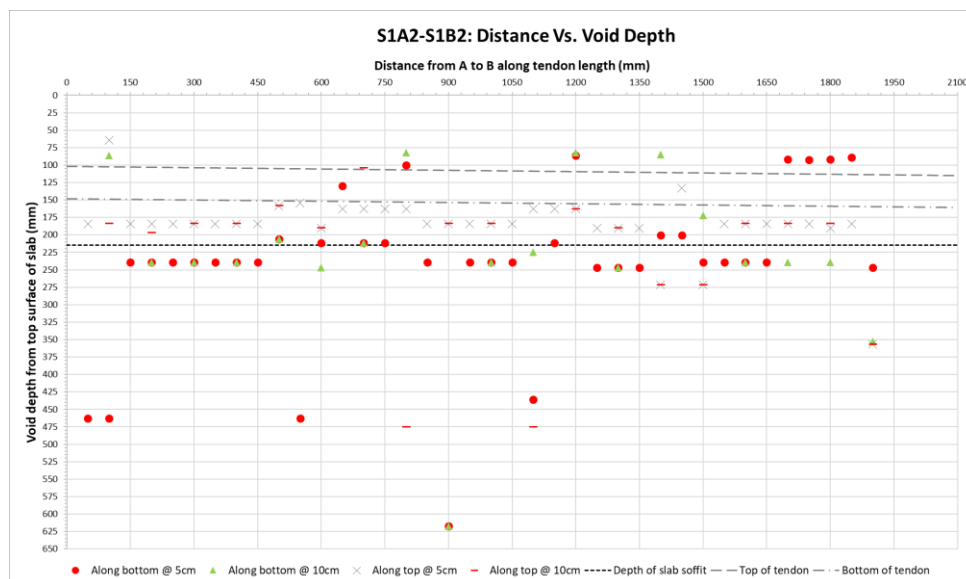


Figure 7-93 S1A2-S1B2: IE data at 5 cm intervals from both slab surfaces

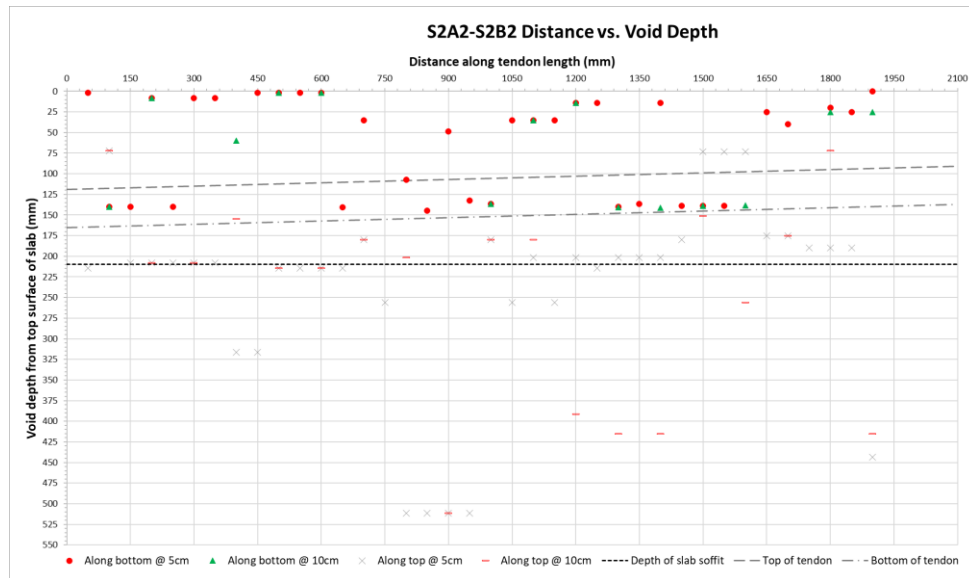


Figure 7-94 S2A2-S2B2: IE data at 5 cm and 10 cm intervals from both slab surfaces

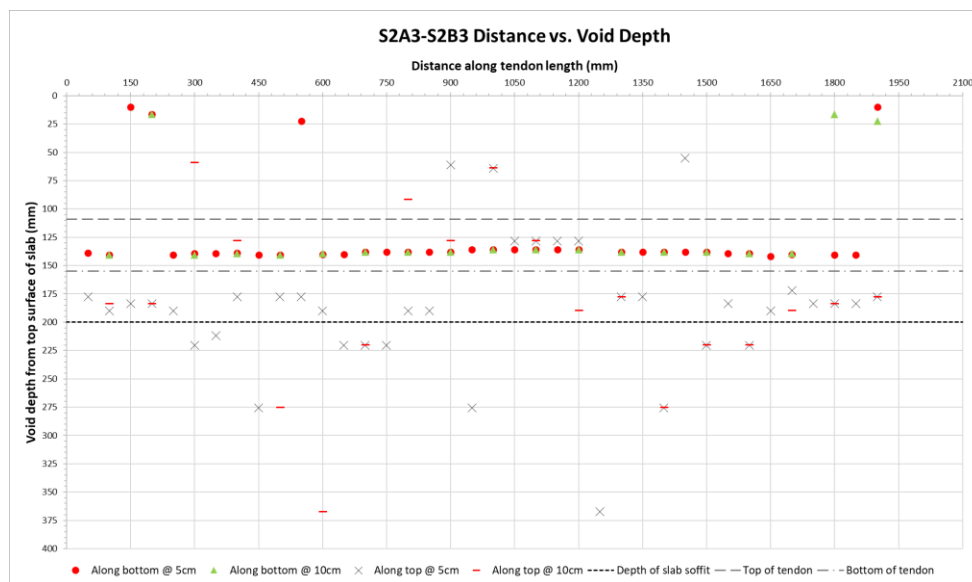


Figure 7-95 S2A3-S2B3: IE data at 5 cm and 10 cm intervals from both slab surfaces

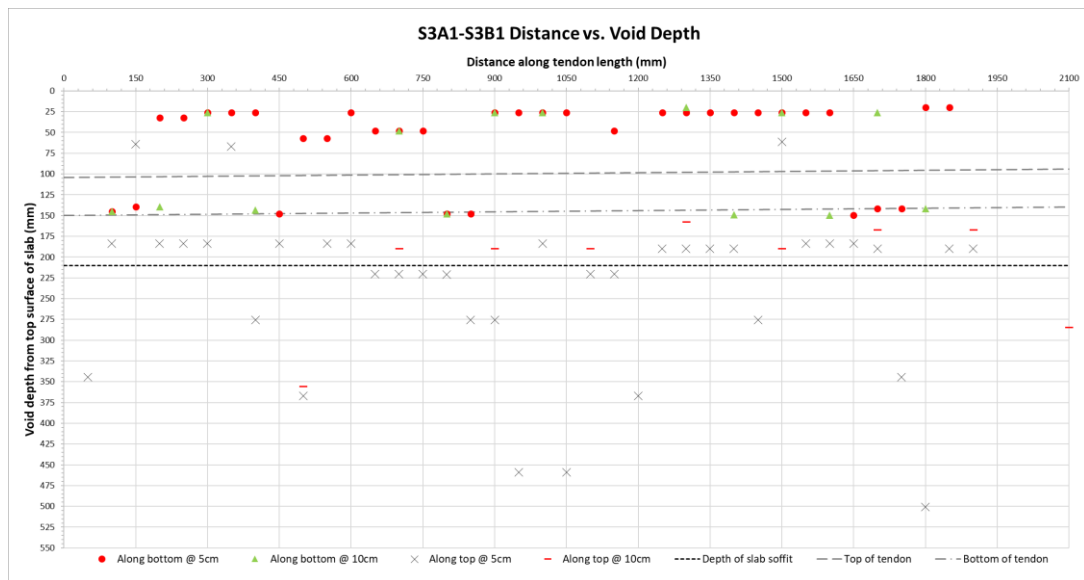


Figure 7-96 S3A1-S3B1: IE data at 5 cm and 10 cm intervals from both slab surfaces

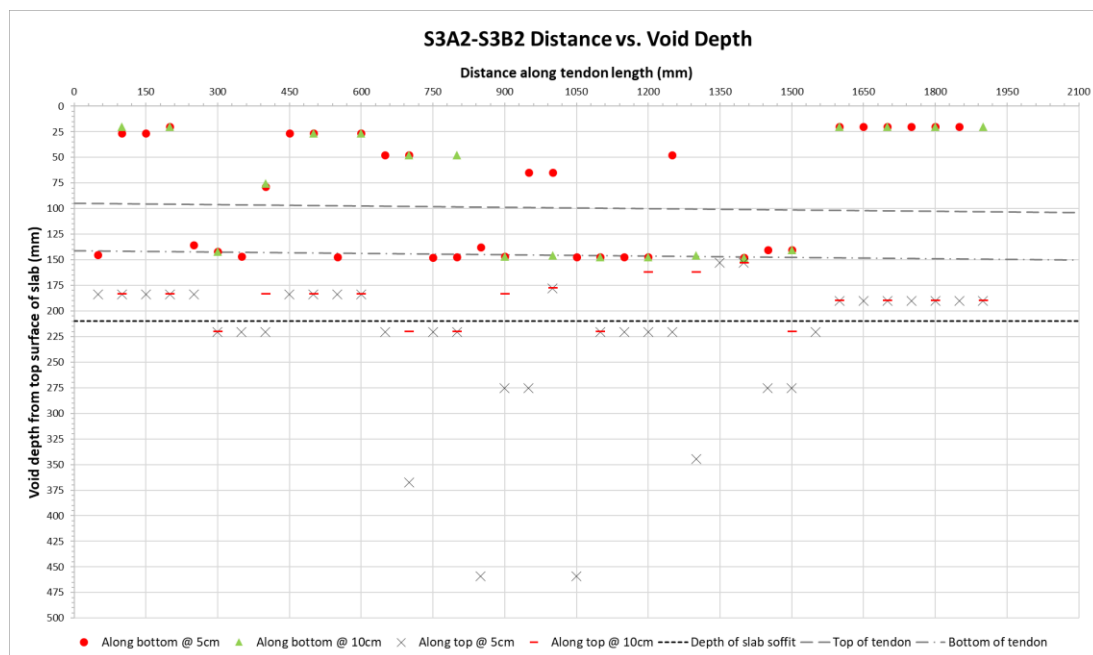


Figure 7-97 S3A2-S3B2: IE data at 5 cm and 10 cm intervals from both slab surfaces

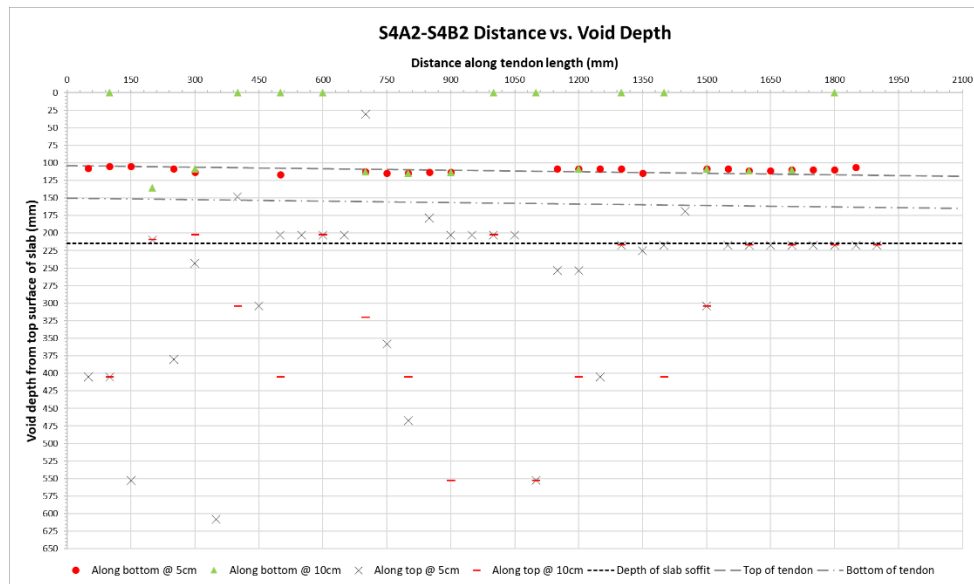


Figure 7-98 S4A2-S4B2: IE data at 5 cm and 10 cm intervals from both slab surfaces

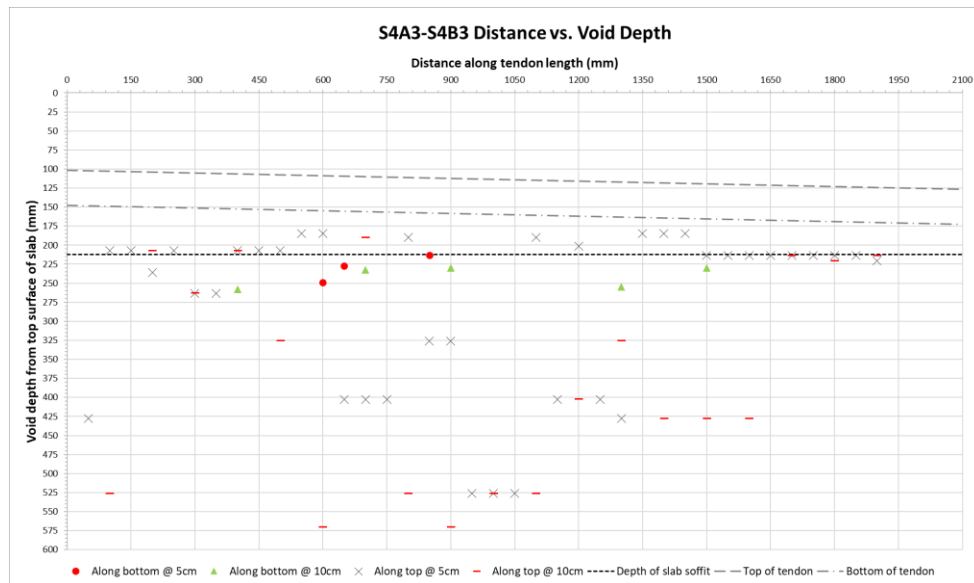


Figure 7-99 S4A3-S4B3: IE data at 5 cm and 10 cm intervals from both slab surfaces

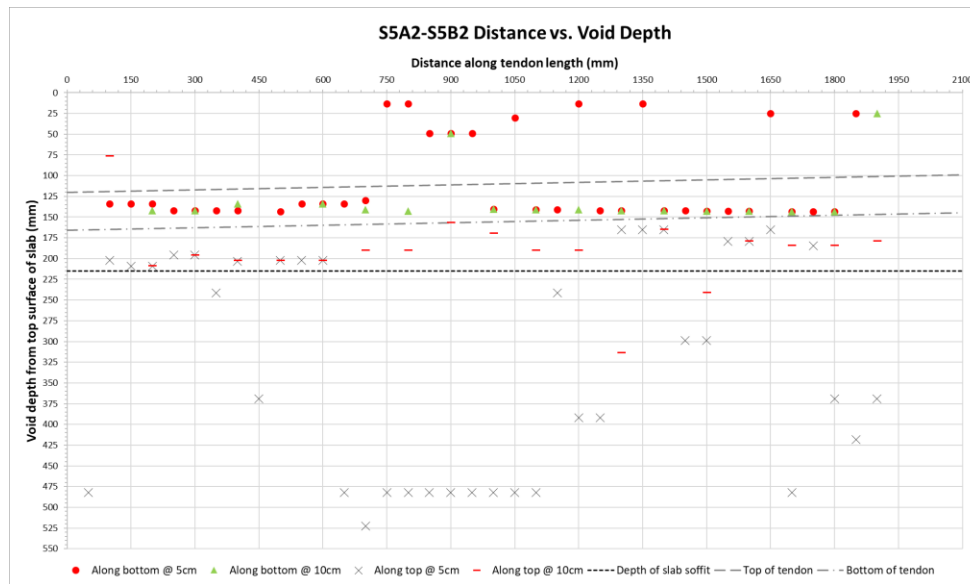


Figure 7-100 S5A2-S5B2: IE data at 5 cm and 10 cm intervals from both slab surfaces

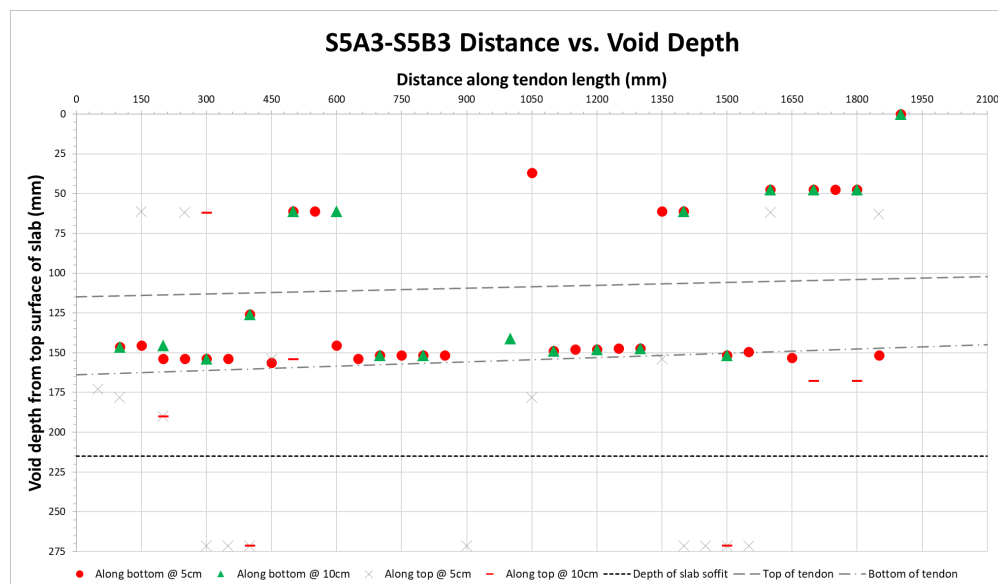


Figure 7-101 S5A3-S5B3: IE data at 5 cm and 10 cm intervals from both slab surfaces

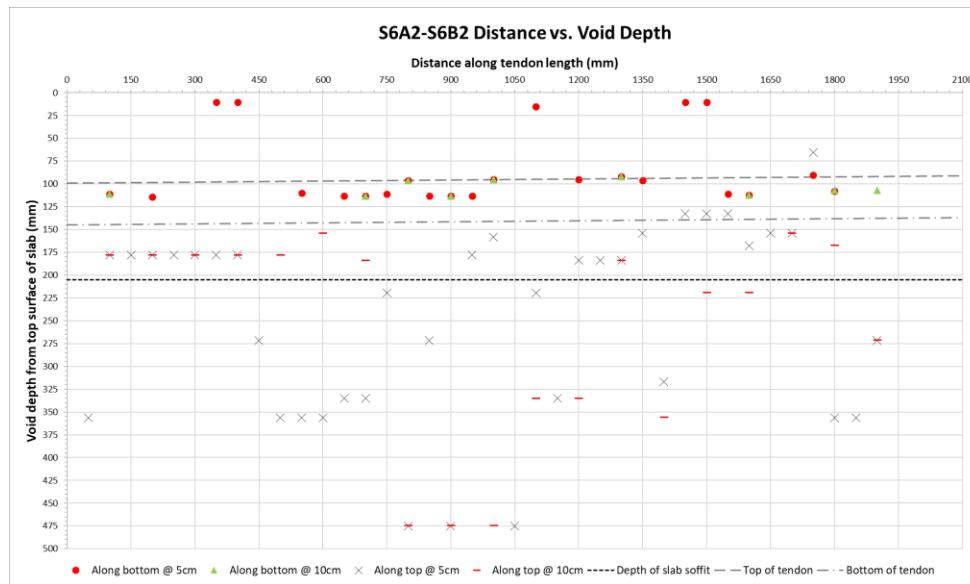


Figure 7-102 S6A2-S6B2: IE data at 5 cm and 10 cm intervals from both slab surfaces

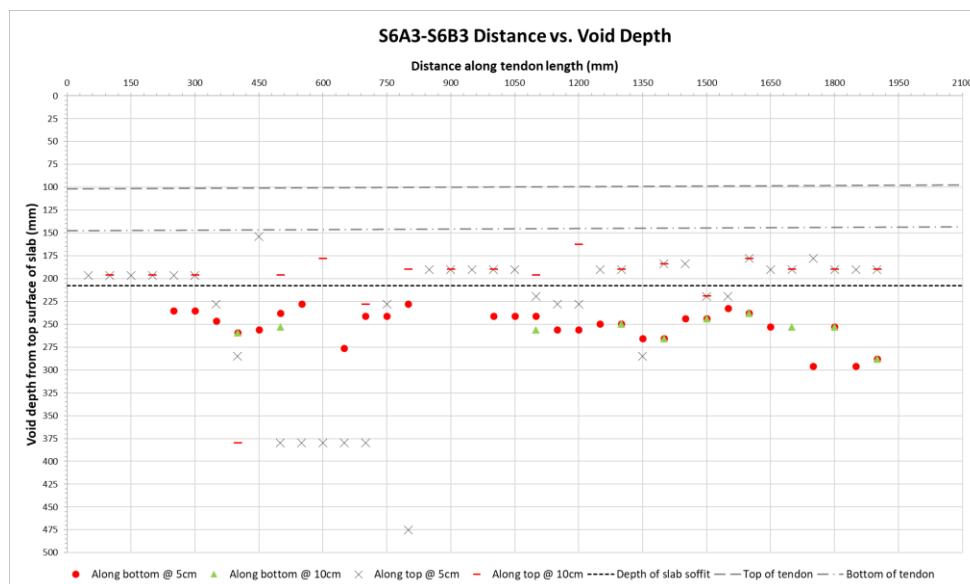


Figure 7-103 S6A3-S6B3: IE data at 5 cm and 10 cm intervals from both slab surfaces



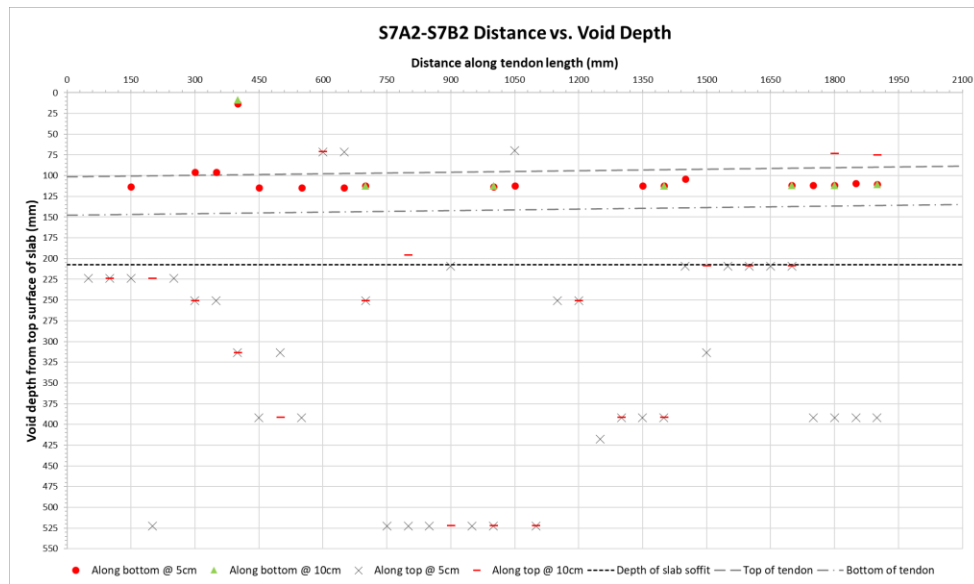


Figure 7-104 S7A2-S7B2 IE data at 5 cm and 10 cm intervals from both slab surfaces

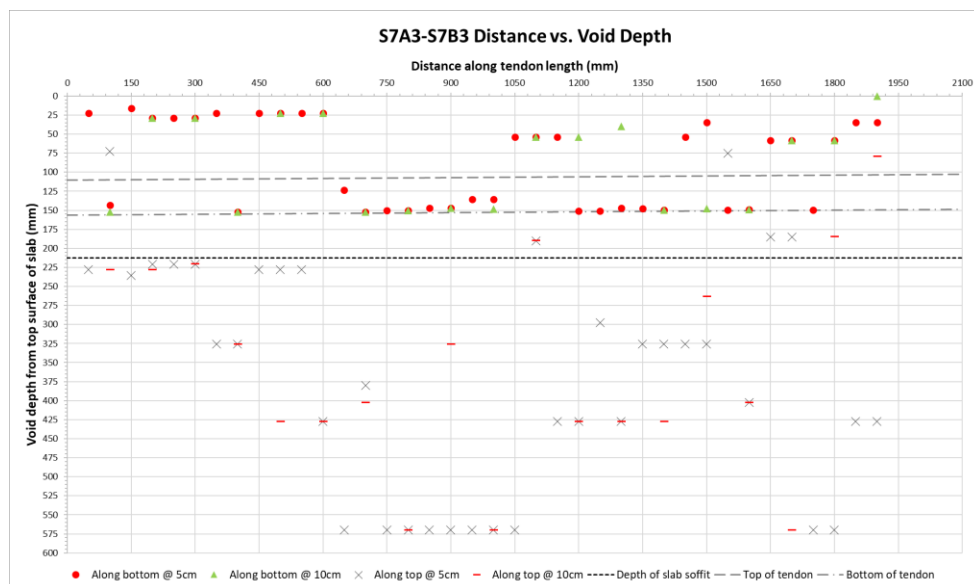


Figure 7-105 S7A3-S7B3: IE data at 5 cm and 10 cm intervals from both slab surfaces

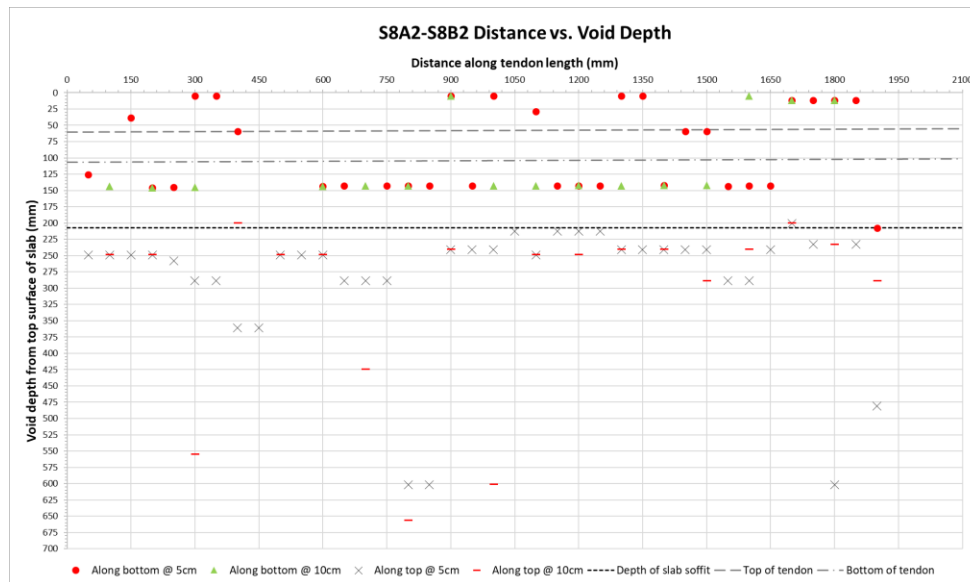


Figure 7-106 S8A2-S8B2: IE data at 5 cm and 10 cm intervals from both slab surfaces

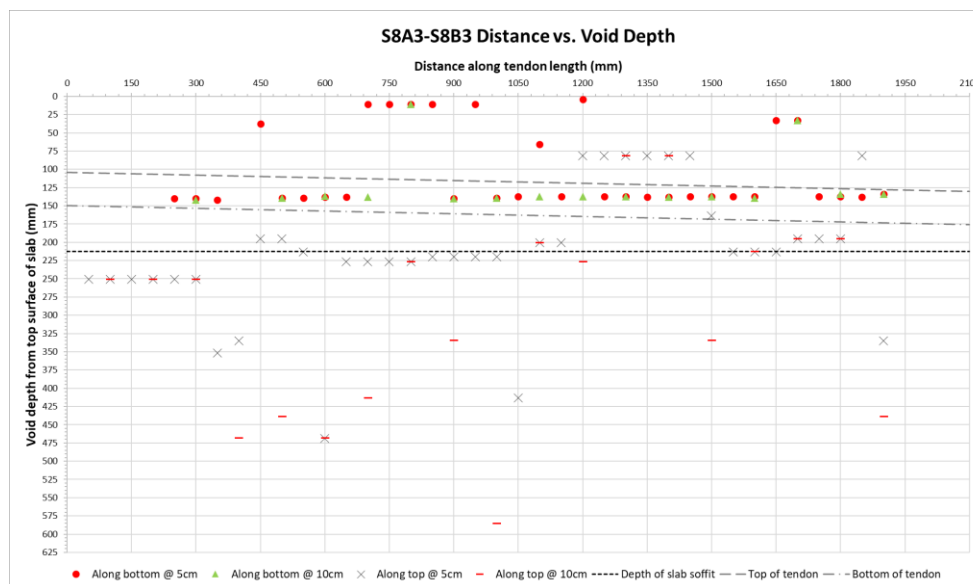


Figure 7-107 S8A3-S8B3: IE data at 5 cm and 10 cm intervals from both slab surfaces

## Appendix B: Gravimetric mass loss and tensile test results

The following tables present the mass loss and ultimate strain of the twelve wires for each prestressing strand, based on the 15 collected tendons. The wires were analyzed in segments to assess the variations in mechanical properties along their length.

Table 7-12. Set number 1 (S1A1-S1B1)

| Sample ID                | Mass<br>loss (%) | $\epsilon_u$ |
|--------------------------|------------------|--------------|
| S1A1-S1B1 - W1 (0-40)    | 0.03             | 0.05407      |
| S1A1-S1B1 - W1 (40-85)   | 0.23             | 0.07472      |
| S1A1-S1B1 - W1 (85-125)  | 0.01             | 0.07492      |
| S1A1-S1B1 - W1 (125-160) | 0.09             | 0.07172      |
| S1A1-S1B1 - W1 (160-195) | 0.11             | 0.06648      |
| S1A1-S1B1 - W2 (0-30)    | 0.16             | 0.06307      |
| S1A1-S1B1 - W2 (30-60)   | 0.19             | 0.07827      |
| S1A1-S1B1 - W2 (60-110)  | 0.03             | 0.07442      |
| S1A1-S1B1 - W2 (110-150) | 0.15             | 0.07214      |
| S1A1-S1B1 - W2 (150-195) | 0.07             | 0.08267      |
| S1A1-S1B1 - W3 (0-25)    | 0.12             | 0.09311      |
| S1A1-S1B1 - W3 (25-55)   | 0.04             | 0.09348      |
| S1A1-S1B1 - W3 (55-95)   | 0.15             | 0.10314      |
| S1A1-S1B1 - W3 (95-140)  | 0.11             | 0.09429      |
| S1A1-S1B1 - W3 (140-195) | 0.07             | 0.07417      |
| S1A1-S1B1 - W4 (0-30)    | 0.11             | 0.06594      |
| S1A1-S1B1 - W4 (30-70)   | 0.04             | 0.06494      |
| S1A1-S1B1 - W4 (70-115)  | 0.18             | 0.06168      |
| S1A1-S1B1 - W4 (115-150) | 0.19             | 0.06817      |
| S1A1-S1B1 - W4 (150-195) | 0.20             | 0.06861      |
| S1A1-S1B1 - W5 (0-40)    | 0.22             | 0.04688      |
| S1A1-S1B1 - W5 (40-80)   | 0.17             | 0.06715      |
| S1A1-S1B1 - W5 (80-105)  | 0.42             | 0.06170      |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S1A1-S1B1 - W5 (105-145)</b>  | 0.24 | 0.06295 |
| <b>S1A1-S1B1 - W5 (145-195)</b>  | 0.15 | 0.06131 |
| <b>S1A1-S1B1 - W6 (0-40)</b>     | 0.13 | 0.06272 |
| <b>S1A1-S1B1 - W6 (40-80)</b>    | 0.14 | 0.06624 |
| <b>S1A1-S1B1 - W6 (80-120)</b>   | 0.22 | 0.06267 |
| <b>S1A1-S1B1 - W6 (120-165)</b>  | 0.03 | 0.06609 |
| <b>S1A1-S1B1 - W6 (165-195)</b>  | 0.29 | 0.06177 |
| <b>S1A1-S1B1 - W7 (0-40)</b>     | 0.26 | 0.02640 |
| <b>S1A1-S1B1 - W7 (40-80)</b>    | 0.23 | 0.04988 |
| <b>S1A1-S1B1 - W7 (80-125)</b>   | 0.06 | 0.04943 |
| <b>S1A1-S1B1 - W7 (125-160)</b>  | 0.03 | 0.06037 |
| <b>S1A1-S1B1 - W7 (160-195)</b>  | 0.37 | 0.05723 |
| <b>S1A1-S1B1 - W8 (0-50)</b>     | 0.07 | 0.06862 |
| <b>S1A1-S1B1 - W8 (50-90)</b>    | 0.07 | 0.06054 |
| <b>S1A1-S1B1 - W8 (90-120)</b>   | 0.31 | 0.06925 |
| <b>S1A1-S1B1 - W8 (120-150)</b>  | 0.15 | 0.06458 |
| <b>S1A1-S1B1 - W8 (150-195)</b>  | 0.05 | 0.06258 |
| <b>S1A1-S1B1 - W9 (0-30)</b>     | 0.11 | 0.04275 |
| <b>S1A1-S1B1 - W9 (30-60)</b>    | 0.40 | 0.06735 |
| <b>S1A1-S1B1 - W9 (60-90)</b>    | 0.40 | 0.06235 |
| <b>S1A1-S1B1 - W9 (90-120)</b>   | 0.27 | 0.05975 |
| <b>S1A1-S1B1 - W9 (120-150)</b>  | 0.21 | 0.06014 |
| <b>S1A1-S1B1 - W9 (150-195)</b>  | 0.05 | 0.04611 |
| <b>S1A1-S1B1 - W10 (0-30)</b>    | N/A  |         |
| <b>S1A1-S1B1 - W10 (30-70)</b>   | 0.22 | 0.06741 |
| <b>S1A1-S1B1 - W10 (70-100)</b>  | 0.27 | 0.06446 |
| <b>S1A1-S1B1 - W10 (100-135)</b> | 0.11 | 0.06290 |
| <b>S1A1-S1B1 - W10 (135-175)</b> | 0.07 | 0.06185 |
| <b>S1A1-S1B1 - W10 (175-195)</b> | 0.20 | 0.06410 |
| <b>S1A1-S1B1 - W11 (0-40)</b>    | 0.01 | 0.04117 |
| <b>S1A1-S1B1 - W11 (40-80)</b>   | 0.15 | 0.05719 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S1A1-S1B1 - W11 (80-115)</b>  | 0.23 | 0.08032 |
| <b>S1A1-S1B1 - W11 (115-140)</b> | 0.27 | 0.06394 |
| <b>S1A1-S1B1 - W11 (140-165)</b> | 0.41 | 0.06565 |
| <b>S1A1-S1B1 - W11 (165-195)</b> | 0.09 | 0.05221 |
| <hr/>                            |      |         |
| <b>S1A1-S1B1 - W12 (0-40)</b>    | 0.26 | 0.06470 |
| <b>S1A1-S1B1 - W12 (40-70)</b>   | 0.17 | 0.06214 |
| <b>S1A1-S1B1 - W12 (70-100)</b>  | 0.05 | 0.06688 |
| <b>S1A1-S1B1 - W12 (100-130)</b> | 0.10 | 0.06416 |
| <b>S1A1-S1B1 - W12 (130-160)</b> | 0.07 | 0.06383 |
| <b>S1A1-S1B1 - W12 (160-195)</b> | 0.16 | 0.06678 |

Table 7-13. Set number 2 (S1A2-S1B2)

| <b>Sample ID</b>                | <b>Mass<br/>loss (%)</b> | <b><math>\epsilon_u</math></b> |
|---------------------------------|--------------------------|--------------------------------|
| <b>S1A2-S1B2 - W1 (0-30)</b>    | 0.10                     | 0.05118                        |
| <b>S1A2-S1B2 - W1 (30-70)</b>   | 0.03                     | 0.08160                        |
| <b>S1A2-S1B2 - W1 (70-125)</b>  | 0.08                     | 0.06080                        |
| <b>S1A2-S1B2 - W1 (125-165)</b> | 0.18                     | 0.06724                        |
| <b>S1A2-S1B2 - W1 (165-195)</b> | 0.20                     | 0.05859                        |
| <b>S1A2-S1B2 - W2 (0-30)</b>    | 0.28                     | 0.05815                        |
| <b>S1A2-S1B2 - W2 (30-70)</b>   | 0.04                     | 0.06159                        |
| <b>S1A2-S1B2 - W2 (70-125)</b>  | 0.03                     | 0.07575                        |
| <b>S1A2-S1B2 - W2 (125-165)</b> | 0.11                     | 0.08156                        |
| <b>S1A2-S1B2 - W2 (165-195)</b> | 0.14                     | 0.06554                        |
| <b>S1A2-S1B2 - W3 (0-30)</b>    | 0.11                     | 0.06027                        |
| <b>S1A2-S1B2 - W3 (30-70)</b>   | 0.17                     | 0.07252                        |
| <b>S1A2-S1B2 - W3 (70-125)</b>  | 0.02                     | 0.05094                        |
| <b>S1A2-S1B2 - W3 (125-165)</b> | 0.03                     | 0.06784                        |
| <b>S1A2-S1B2 - W3 (165-195)</b> | 0.22                     | 0.07096                        |
| <b>S1A2-S1B2 - W4 (0-30)</b>    | 0.17                     | 0.05631                        |
| <b>S1A2-S1B2 - W4 (30-70)</b>   | 3.58                     | N/A                            |
| <b>S1A2-S1B2 - W4 (70-125)</b>  | 0.05                     | 0.04736                        |
| <b>S1A2-S1B2 - W4 (125-165)</b> | 0.02                     | 0.06799                        |
| <b>S1A2-S1B2 - W4 (165-195)</b> | 0.02                     | 0.07623                        |
| <b>S1A2-S1B2 - W5 (0-30)</b>    | 0.22                     | 0.07482                        |
| <b>S1A2-S1B2 - W5 (30-70)</b>   | 0.14                     | 0.09486                        |
| <b>S1A2-S1B2 - W5 (70-125)</b>  | 0.18                     | 0.05656                        |
| <b>S1A2-S1B2 - W5 (125-165)</b> | 0.19                     | 0.06313                        |
| <b>S1A2-S1B2 - W5 (165-195)</b> | 0.01                     | 0.05815                        |
| <b>S1A2-S1B2 - W6 (0-30)</b>    | 0.04                     | 0.05544                        |
| <b>S1A2-S1B2 - W6 (30-70)</b>   | 0.17                     | 0.07553                        |
| <b>S1A2-S1B2 - W6 (70-125)</b>  | 0.10                     | 0.05997                        |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S1A2-S1B2 - W6 (125-165)</b>  | 0.16 | 0.07734 |
| <b>S1A2-S1B2 - W6 (165-195)</b>  | 0.13 | 0.05545 |
| <b>S1A2-S1B2 - W7 (0-30)</b>     | 0.26 | 0.07235 |
| <b>S1A2-S1B2 - W7 (30-70)</b>    | 0.12 | 0.08120 |
| <b>S1A2-S1B2 - W7 (70-125)</b>   | 0.08 | 0.08518 |
| <b>S1A2-S1B2 - W7 (125-165)</b>  | 0.11 | 0.07949 |
| <b>S1A2-S1B2 - W7 (165-195)</b>  | 0.00 | 0.03608 |
| <b>S1A2-S1B2 - W8 (0-30)</b>     | 6.70 | 0.06111 |
| <b>S1A2-S1B2 - W8 (30-70)</b>    | 0.17 | 0.06693 |
| <b>S1A2-S1B2 - W8 (70-125)</b>   | 0.00 | 0.06907 |
| <b>S1A2-S1B2 - W8 (125-165)</b>  | 0.15 | 0.07034 |
| <b>S1A2-S1B2 - W8 (165-195)</b>  | 0.10 | 0.05164 |
| <b>S1A2-S1B2 - W9 (0-30)</b>     | 0.21 | 0.06714 |
| <b>S1A2-S1B2 - W9 (30-70)</b>    | 0.23 | 0.07306 |
| <b>S1A2-S1B2 - W9 (70-125)</b>   | 0.04 | 0.06472 |
| <b>S1A2-S1B2 - W9 (125-165)</b>  | 0.07 | 0.06875 |
| <b>S1A2-S1B2 - W9 (165-195)</b>  | 0.26 | 0.06772 |
| <b>S1A2-S1B2 - W10 (0-30)</b>    | 0.33 | 0.04875 |
| <b>S1A2-S1B2 - W10 (30-70)</b>   | N/A  | 0.07559 |
| <b>S1A2-S1B2 - W10 (70-125)</b>  | 0.10 | 0.06699 |
| <b>S1A2-S1B2 - W10 (125-165)</b> | 0.11 | 0.07149 |
| <b>S1A2-S1B2 - W10 (165-195)</b> | 0.32 | 0.05458 |
| <b>S1A2-S1B2 - W11 (0-30)</b>    | 0.03 | 0.05622 |
| <b>S1A2-S1B2 - W11 (30-70)</b>   | 0.12 | 0.06061 |
| <b>S1A2-S1B2 - W11 (70-125)</b>  | 0.18 | 0.05927 |
| <b>S1A2-S1B2 - W11 (125-165)</b> | 0.16 | 0.04862 |
| <b>S1A2-S1B2 - W11 (165-195)</b> | 0.13 | 0.06537 |
| <b>S1A2-S1B2 - W12 (0-30)</b>    | 0.21 | 0.05729 |
| <b>S1A2-S1B2 - W12 (30-70)</b>   | 0.10 | 0.05677 |
| <b>S1A2-S1B2 - W12 (70-125)</b>  | 0.04 | 0.07669 |
| <b>S1A2-S1B2 - W12 (125-165)</b> | 0.22 | 0.07003 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S1A2-S1B2 - W12 (165-195)</b> | 0.67 | 0.04325 |
|----------------------------------|------|---------|



Table 7-14. Set number 3 (S2A2-S2B2)

| Sample ID                | Mass loss<br>(%) | $\xi_u$ |
|--------------------------|------------------|---------|
| S2A2-S2B2 - W1 (0-30)    | 0.30             | 0.05525 |
| S2A2-S2B2 - W1 (30-70)   | 0.13             | 0.04654 |
| S2A2-S2B2 - W1 (70-110)  | 0.25             | 0.04423 |
| S2A2-S2B2 - W1 (110-135) | 14.42            | 0.04600 |
| S2A2-S2B2 - W1 (135-165) | 0.25             | 0.05446 |
| S2A2-S2B2 - W1 (165-195) | 0.14             | 0.06077 |
| S2A2-S2B2 - W2 (0-30)    | 0.26             | 0.06032 |
| S2A2-S2B2 - W2 (30-70)   | 0.17             | 0.06051 |
| S2A2-S2B2 - W2 (70-100)  | 0.32             | 0.06711 |
| S2A2-S2B2 - W2 (100-130) | 0.79             | 0.06231 |
| S2A2-S2B2 - W2 (130-165) | 0.17             | 0.05509 |
| S2A2-S2B2 - W2 (165-195) | 0.02             | 0.05591 |
| S2A2-S2B2 - W3 (0-30)    | 0.28             | 0.06104 |
| S2A2-S2B2 - W3 (30-70)   | 0.06             | 0.07027 |
| S2A2-S2B2 - W3 (70-110)  | 0.01             | 0.06250 |
| S2A2-S2B2 - W3 (110-150) | 0.09             | 0.06596 |
| S2A2-S2B2 - W3 (150-195) | 0.03             | 0.05163 |
| S2A2-S2B2 - W4 (0-30)    | 0.07             | 0.06137 |
| S2A2-S2B2 - W4 (30-70)   | 0.06             | 0.06939 |
| S2A2-S2B2 - W4 (70-110)  | 0.16             | 0.05266 |
| S2A2-S2B2 - W4 (110-135) | 0.17             | 0.06969 |
| S2A2-S2B2 - W4 (135-165) | 0.16             | 0.07309 |
| S2A2-S2B2 - W4 (165-195) | 0.13             | 0.06036 |
| S2A2-S2B2 - W5 (0-30)    | 0.11             | 0.06253 |
| S2A2-S2B2 - W5 (30-70)   | 0.02             | 0.06247 |
| S2A2-S2B2 - W5 (70-110)  | 0.16             | 0.05801 |
| S2A2-S2B2 - W5 (110-140) | 3.19             | 0.06372 |
| S2A2-S2B2 - W5 (140-165) | 0.38             | 0.06767 |
| S2A2-S2B2 - W5 (165-195) | 0.17             | 0.06174 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S2A2-S2B2 - W6 (0-30)</b>     | 0.01 | 0.06619 |
| <b>S2A2-S2B2 - W6 (30-70)</b>    | 0.20 | 0.06145 |
| <b>S2A2-S2B2 - W6 (70-110)</b>   | 0.05 | 0.06218 |
| <b>S2A2-S2B2 - W6 (110-135)</b>  | 0.04 | 0.06996 |
| <b>S2A2-S2B2 - W6 (135-165)</b>  | 0.20 | 0.06494 |
| <b>S2A2-S2B2 - W6 (165-195)</b>  | 0.24 | 0.05852 |
| <b>S2A2-S2B2 - W7 (0-30)</b>     | 2.37 | 0.05818 |
| <b>S2A2-S2B2 - W7 (30-70)</b>    | 0.06 | 0.07662 |
| <b>S2A2-S2B2 - W7 (70-110)</b>   | 0.03 | 0.05496 |
| <b>S2A2-S2B2 - W7 (110-135)</b>  | 0.20 | 0.07609 |
| <b>S2A2-S2B2 - W7 (135-165)</b>  | 0.17 | 0.07755 |
| <b>S2A2-S2B2 - W7 (165-195)</b>  | 0.05 | 0.07001 |
| <b>S2A2-S2B2 - W8 (0-30)</b>     | 0.34 | 0.05394 |
| <b>S2A2-S2B2 - W8 (30-70)</b>    | 0.20 | 0.07108 |
| <b>S2A2-S2B2 - W8 (70-110)</b>   | 1.16 | 0.06989 |
| <b>S2A2-S2B2 - W8 (110-140)</b>  | 0.16 | 0.06357 |
| <b>S2A2-S2B2 - W8 (140-165)</b>  | 0.15 | 0.07079 |
| <b>S2A2-S2B2 - W8 (165-195)</b>  | 0.00 | 0.06458 |
| <b>S2A2-S2B2 - W9 (0-30)</b>     | 0.18 | 0.05942 |
| <b>S2A2-S2B2 - W9 (30-60)</b>    | 0.74 | 0.07253 |
| <b>S2A2-S2B2 - W9 (60-100)</b>   | 0.08 | 0.07276 |
| <b>S2A2-S2B2 - W9 (100-135)</b>  | 0.14 | 0.06574 |
| <b>S2A2-S2B2 - W9 (135-165)</b>  | 0.21 | 0.06467 |
| <b>S2A2-S2B2 - W9 (165-195)</b>  | 2.03 | 0.05230 |
| <b>S2A2-S2B2 - W10 (0-30)</b>    | 0.27 | 0.06116 |
| <b>S2A2-S2B2 - W10 (30-70)</b>   | N/A  | 0.06567 |
| <b>S2A2-S2B2 - W10 (70-110)</b>  | 0.15 | 0.06562 |
| <b>S2A2-S2B2 - W10 (110-155)</b> | 0.06 | 0.06335 |
| <b>S2A2-S2B2 - W10 (155-195)</b> | 0.24 | 0.07560 |
| <b>S2A2-S2B2 - W11 (0-30)</b>    | 0.09 | 0.05155 |
| <b>S2A2-S2B2 - W11 (30-70)</b>   | 0.21 | 0.06834 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S2A2-S2B2 - W11 (70-110)</b>  | 0.22 | 0.06513 |
| <b>S2A2-S2B2 - W11 (110-155)</b> | 0.15 | 0.05714 |
| <b>S2A2-S2B2 - W11 (155-195)</b> | 0.07 | 0.06303 |
| <b>S2A2-S2B2 - W12 (0-40)</b>    | 0.24 | 0.06494 |
| <b>S2A2-S2B2 - W12 (40-80)</b>   | 0.20 | 0.06961 |
| <b>S2A2-S2B2 - W12 (80-120)</b>  | 0.15 | 0.06406 |
| <b>S2A2-S2B2 - W12 (120-145)</b> | 0.34 | 0.06576 |
| <b>S2A2-S2B2 - W12 (145-165)</b> | 0.27 | 0.07456 |
| <b>S2A2-S2B2 - W12 (165-195)</b> | 0.20 | 0.05627 |

Table 7-15. Set number 4 (S2A3-S2B3)

| Sample ID                | Mass loss<br>(%) | $\xi_u$ |
|--------------------------|------------------|---------|
| S2A3-S2B3 - W1 (0-30)    | 0.12             | 0.06830 |
| S2A3-S2B3 - W1 (30-60)   | 0.19             | 0.05802 |
| S2A3-S2B3 - W1 (60-90)   | 0.30             | 0.06772 |
| S2A3-S2B3 - W1 (90-130)  | 0.24             | 0.05079 |
| S2A3-S2B3 - W1 (130-160) | 0.17             | 0.06135 |
| S2A3-S2B3 - W1 (160-195) | 0.15             | 0.05804 |
| S2A3-S2B3 - W2 (0-30)    | 0.20             | 0.06855 |
| S2A3-S2B3 - W2 (30-70)   | 2.43             | 0.05058 |
| S2A3-S2B3 - W2 (70-100)  | 0.06             | 0.06800 |
| S2A3-S2B3 - W2 (100-130) | 0.24             | 0.06744 |
| S2A3-S2B3 - W2 (130-160) | 0.17             | 0.06602 |
| S2A3-S2B3 - W2 (160-195) | 0.00             |         |
| S2A3-S2B3 - W3 (0-30)    | 0.07             | 0.06678 |
| S2A3-S2B3 - W3 (30-70)   | 0.03             | 0.06377 |
| S2A3-S2B3 - W3 (70-110)  | 0.07             | 0.06636 |
| S2A3-S2B3 - W3 (110-140) | 0.03             | 0.06310 |
| S2A3-S2B3 - W3 (140-165) | 0.20             | 0.07925 |
| S2A3-S2B3 - W3 (165-195) | 0.28             | 0.06130 |
| S2A3-S2B3 - W4 (0-30)    | 0.03             | 0.06278 |
| S2A3-S2B3 - W4 (30-70)   | 0.01             | 0.04693 |
| S2A3-S2B3 - W4 (70-110)  | 0.08             | 0.06513 |
| S2A3-S2B3 - W4 (110-150) | 0.00             | 0.05571 |
| S2A3-S2B3 - W4 (150-195) | 1.21             | 0.04090 |
| S2A3-S2B3 - W5 (0-30)    | 0.11             | 0.06170 |
| S2A3-S2B3 - W5 (30-70)   | 0.66             | 0.04833 |
| S2A3-S2B3 - W5 (70-100)  | 0.09             | 0.06662 |
| S2A3-S2B3 - W5 (100-130) | 0.01             | 0.06298 |
| S2A3-S2B3 - W5 (130-160) | 0.07             | 0.06431 |
| S2A3-S2B3 - W5 (160-195) | 0.15             | 0.05192 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S2A3-S2B3 - W6 (0-40)</b>     | 0.11 | 0.06518 |
| <b>S2A3-S2B3 - W6 (40-80)</b>    | 0.07 | 0.06785 |
| <b>S2A3-S2B3 - W6 (80-120)</b>   | 0.23 | 0.10680 |
| <b>S2A3-S2B3 - W6 (120-160)</b>  | 0.06 | 0.08522 |
| <b>S2A3-S2B3 - W6 (160-195)</b>  | 0.95 | 0.05900 |
| <b>S2A3-S2B3 - W7 (0-40)</b>     | 0.17 | 0.05484 |
| <b>S2A3-S2B3 - W7 (40-80)</b>    | 0.00 | 0.05582 |
| <b>S2A3-S2B3 - W7 (80-120)</b>   | 0.15 | 0.05567 |
| <b>S2A3-S2B3 - W7 (120-160)</b>  | 0.17 | 0.05980 |
| <b>S2A3-S2B3 - W7 (160-195)</b>  | 0.18 | 0.08381 |
| <b>S2A3-S2B3 - W8 (0-40)</b>     | 0.16 | 0.06687 |
| <b>S2A3-S2B3 - W8 (40-80)</b>    | 0.23 | 0.07130 |
| <b>S2A3-S2B3 - W8 (80-120)</b>   | 0.24 | 0.06846 |
| <b>S2A3-S2B3 - W8 (120-160)</b>  | 0.07 | 0.06716 |
| <b>S2A3-S2B3 - W8 (160-195)</b>  | 0.01 | 0.06668 |
| <b>S2A3-S2B3 - W9 (0-40)</b>     | 0.08 | 0.06626 |
| <b>S2A3-S2B3 - W9 (40-80)</b>    | 0.18 | 0.07177 |
| <b>S2A3-S2B3 - W9 (80-115)</b>   | 0.08 | 0.06928 |
| <b>S2A3-S2B3 - W9 (115-145)</b>  | 0.32 | 0.06811 |
| <b>S2A3-S2B3 - W9 (145-195)</b>  | 0.14 | 0.06913 |
| <b>S2A3-S2B3 - W10 (0-40)</b>    | 0.16 | 0.05981 |
| <b>S2A3-S2B3 - W10 (40-80)</b>   | N/A  | 0.07236 |
| <b>S2A3-S2B3 - W10 (80-120)</b>  | 0.23 | 0.06851 |
| <b>S2A3-S2B3 - W10 (120-160)</b> | 0.07 | 0.06808 |
| <b>S2A3-S2B3 - W10 (160-195)</b> | 0.12 | 0.05508 |
| <b>S2A3-S2B3 - W11 (0-40)</b>    | 0.12 | 0.06532 |
| <b>S2A3-S2B3 - W11 (40-80)</b>   | 0.17 | 0.06357 |
| <b>S2A3-S2B3 - W11 (80-120)</b>  | 0.05 | 0.08453 |
| <b>S2A3-S2B3 - W11 (120-160)</b> | 1.60 | 0.06697 |
| <b>S2A3-S2B3 - W11 (160-195)</b> | 0.25 | 0.06882 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S2A3-S2B3 - W12 (0-40)</b>    | 0.02 | 0.06498 |
| <b>S2A3-S2B3 - W12 (40-80)</b>   | 0.18 | 0.05880 |
| <b>S2A3-S2B3 - W12 (80-120)</b>  | 0.03 | 0.06068 |
| <b>S2A3-S2B3 - W12 (120-160)</b> | 0.07 | 0.06938 |
| <b>S2A3-S2B3 - W12 (160-195)</b> | 0.76 | 0.06489 |

Table 7-16. Set number 5 (S3A1-S3B1)

| Sample ID                | Mass loss<br>(%) | $\xi_u$ |
|--------------------------|------------------|---------|
| S3A1-S3B1 - W1 (0-30)    | 0.13             | 0.05369 |
| S3A1-S3B1 - W1 (30-70)   | 0.02             | 0.05792 |
| S3A1-S3B1 - W1 (70-125)  | 0.18             | 0.05717 |
| S3A1-S3B1 - W1 (125-165) | 0.02             | 0.05235 |
| S3A1-S3B1 - W1 (165-195) | 0.14             | 0.05126 |
| S3A1-S3B1 - W2 (0-30)    | 0.30             | 0.05556 |
| S3A1-S3B1 - W2 (30-70)   | 0.08             | 0.07049 |
| S3A1-S3B1 - W2 (70-125)  | 0.16             | 0.07281 |
| S3A1-S3B1 - W2 (125-165) | 0.25             | 0.06109 |
| S3A1-S3B1 - W2 (165-195) | 0.26             | 0.06014 |
| S3A1-S3B1 - W3 (0-30)    | 0.14             | 0.05761 |
| S3A1-S3B1 - W3 (30-70)   | 0.15             | 0.06047 |
| S3A1-S3B1 - W3 (70-125)  | 0.11             | 0.05327 |
| S3A1-S3B1 - W3 (125-165) | 0.23             | 0.05532 |
| S3A1-S3B1 - W3 (165-195) | 2.24             | 0.05512 |
| S3A1-S3B1 - W4 (0-30)    | 0.11             | 0.05776 |
| S3A1-S3B1 - W4 (30-70)   | 0.00             | N/A     |
| S3A1-S3B1 - W4 (70-125)  | 0.04             | 0.05048 |
| S3A1-S3B1 - W4 (125-165) | 0.12             | 0.05035 |
| S3A1-S3B1 - W4 (165-195) | 0.04             | 0.04781 |
| S3A1-S3B1 - W5 (0-30)    | 0.17             | 0.05956 |
| S3A1-S3B1 - W5 (30-70)   | 0.05             | 0.05580 |
| S3A1-S3B1 - W5 (70-125)  | 0.09             | 0.05779 |
| S3A1-S3B1 - W5 (125-165) | 0.24             | 0.07115 |
| S3A1-S3B1 - W5 (165-195) | 0.29             | 0.04986 |

|                           |      |         |
|---------------------------|------|---------|
| S3A1-S3B1 - W6 (0-30)     | 0.16 | 0.08608 |
| S3A1-S3B1 - W6 (30-70)    | 0.24 | 0.06083 |
| S3A1-S3B1 - W6 (70-125)   | 0.15 | 0.06015 |
| S3A1-S3B1 - W6 (125-165)  | 0.09 | 0.06608 |
| S3A1-S3B1 - W6 (165-195)  | 0.29 | 0.05110 |
| S3A1-S3B1 - W7 (0-30)     | 0.03 | 0.06095 |
| S3A1-S3B1 - W7 (30-70)    | 0.12 | 0.05590 |
| S3A1-S3B1 - W7 (70-125)   | 0.07 | 0.05721 |
| S3A1-S3B1 - W7 (125-165)  | 0.24 | 0.07202 |
| S3A1-S3B1 - W7 (165-195)  | 0.10 | 0.05599 |
| S3A1-S3B1 - W8 (0-30)     | 0.25 | 0.05726 |
| S3A1-S3B1 - W8 (30-70)    | 0.11 | 0.06044 |
| S3A1-S3B1 - W8 (70-125)   | 0.10 | 0.06604 |
| S3A1-S3B1 - W8 (125-165)  | 0.20 | 0.06890 |
| S3A1-S3B1 - W8 (165-195)  | 1.26 | 0.08709 |
| S3A1-S3B1 - W9 (0-30)     | 0.07 | 0.05801 |
| S3A1-S3B1 - W9 (30-70)    | 0.05 | 0.06353 |
| S3A1-S3B1 - W9 (70-125)   | 0.17 | 0.05587 |
| S3A1-S3B1 - W9 (125-165)  | 0.16 | 0.05567 |
| S3A1-S3B1 - W9 (165-195)  | 0.10 | 0.06224 |
| S3A1-S3B1 - W10 (0-30)    | 0.25 | 0.05434 |
| S3A1-S3B1 - W10 (30-70)   | N/A  | 0.05936 |
| S3A1-S3B1 - W10 (70-125)  | 0.13 | 0.06252 |
| S3A1-S3B1 - W10 (125-165) | 0.22 | 0.06445 |
| S3A1-S3B1 - W10 (165-195) | 0.69 | 0.07017 |
| S3A1-S3B1 - W11 (0-30)    | 0.08 | 0.06357 |
| S3A1-S3B1 - W11 (30-70)   | 0.18 | 0.07134 |
| S3A1-S3B1 - W11 (70-125)  | 0.53 | 0.06176 |
| S3A1-S3B1 - W11 (125-165) | 3.77 | 0.03597 |
| S3A1-S3B1 - W11 (165-195) | 2.80 | 0.07200 |
| S3A1-S3B1 - W12 (0-30)    | 0.95 | 0.04273 |



|                                  |      |         |
|----------------------------------|------|---------|
| <b>S3A1-S3B1 - W12 (30-70)</b>   | 0.21 | 0.06508 |
| <b>S3A1-S3B1 - W12 (70-125)</b>  | 0.15 | 0.07000 |
| <b>S3A1-S3B1 - W12 (125-165)</b> | 0.22 | 0.06208 |
| <b>S3A1-S3B1 - W12 (165-195)</b> | 0.31 | 0.06915 |

Table 7-17. Set number 6 (S3A2-S3B2)

| <b>Sample ID</b>                | <b>Mass loss<br/>(%)</b> | <b><math>\xi_u</math></b> |
|---------------------------------|--------------------------|---------------------------|
| <b>S3A2-S3B2 - W1 (0-30)</b>    | 0.10                     | 0.06907                   |
| <b>S3A2-S3B2 - W1 (30-50)</b>   | 0.08                     | 0.06240                   |
| <b>S3A2-S3B2 - W1 (50-80)</b>   | 1.71                     | 0.05728                   |
| <b>S3A2-S3B2 - W1 (80-110)</b>  | 0.20                     | 0.05827                   |
| <b>S3A2-S3B2 - W1 (110-140)</b> | 0.11                     | 0.05998                   |
| <b>S3A2-S3B2 - W1 (140-160)</b> | 0.29                     | 0.05975                   |
| <b>S3A2-S3B2 - W1 (160-195)</b> | 0.29                     | 0.05394                   |
| <b>S3A2-S3B2 - W2 (0-30)</b>    | 0.19                     | 0.05710                   |
| <b>S3A2-S3B2 - W2 (30-60)</b>   | 0.09                     | 0.05594                   |
| <b>S3A2-S3B2 - W2 (60-95)</b>   | 0.16                     | 0.05178                   |
| <b>S3A2-S3B2 - W2 (95-125)</b>  | 1.38                     | 0.05615                   |
| <b>S3A2-S3B2 - W2 (125-155)</b> | 0.10                     | 0.05242                   |
| <b>S3A2-S3B2 - W2 (155-195)</b> | 0.16                     | 0.05821                   |
| <b>S3A2-S3B2 - W3 (0-30)</b>    | 0.11                     | 0.06515                   |
| <b>S3A2-S3B2 - W3 (30-70)</b>   | 0.09                     | 0.06175                   |
| <b>S3A2-S3B2 - W3 (70-110)</b>  | 0.20                     | 0.06577                   |
| <b>S3A2-S3B2 - W3 (110-150)</b> | 0.06                     | 0.06711                   |
| <b>S3A2-S3B2 - W3 (150-195)</b> | 0.24                     | 0.06735                   |
| <b>S3A2-S3B2 - W4 (0-30)</b>    | 0.08                     | 0.06275                   |
| <b>S3A2-S3B2 - W4 (30-65)</b>   | 0.10                     | 0.05412                   |
| <b>S3A2-S3B2 - W4 (65-100)</b>  | 0.02                     | 0.05261                   |
| <b>S3A2-S3B2 - W4 (100-140)</b> | 0.01                     | 0.05645                   |
| <b>S3A2-S3B2 - W4 (140-160)</b> | 0.04                     | 0.07063                   |
| <b>S3A2-S3B2 - W4 (160-195)</b> | 0.01                     | 0.06586                   |
| <b>S3A2-S3B2 - W5 (0-30)</b>    | 0.18                     | 0.06315                   |
| <b>S3A2-S3B2 - W5 (30-70)</b>   | 0.02                     | 0.04885                   |
| <b>S3A2-S3B2 - W5 (70-120)</b>  | 0.05                     | 0.06007                   |
| <b>S3A2-S3B2 - W5 (120-160)</b> | 0.08                     | 0.07486                   |
| <b>S3A2-S3B2 - W5 (160-195)</b> | 0.15                     | 0.05195                   |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S3A2-S3B2 - W6 (0-40)</b>     | 0.11 | 0.04427 |
| <b>S3A2-S3B2 - W6 (40-70)</b>    | 0.22 | 0.05554 |
| <b>S3A2-S3B2 - W6 (70-110)</b>   | 0.03 | 0.05425 |
| <b>S3A2-S3B2 - W6 (110-150)</b>  | 0.06 | 0.05380 |
| <b>S3A2-S3B2 - W6 (150-195)</b>  | 0.04 | 0.06170 |
| <b>S3A2-S3B2 - W7 (0-30)</b>     | 0.25 | 0.06213 |
| <b>S3A2-S3B2 - W7 (30-65)</b>    | 0.62 | 0.06390 |
| <b>S3A2-S3B2 - W7 (65-105)</b>   | 0.07 | 0.06381 |
| <b>S3A2-S3B2 - W7 (105-150)</b>  | 0.14 | 0.06785 |
| <b>S3A2-S3B2 - W7 (150-195)</b>  | 0.15 | 0.05834 |
| <b>S3A2-S3B2 - W8 (0-30)</b>     | 0.09 | 0.04694 |
| <b>S3A2-S3B2 - W8 (30-80)</b>    | 0.08 | 0.05646 |
| <b>S3A2-S3B2 - W8 (80-120)</b>   | 0.18 | 0.05666 |
| <b>S3A2-S3B2 - W8 (120-150)</b>  | 0.27 | 0.05505 |
| <b>S3A2-S3B2 - W8 (150-195)</b>  | 0.55 | 0.08199 |
| <b>S3A2-S3B2 - W9 (0-30)</b>     | 0.28 | 0.07275 |
| <b>S3A2-S3B2 - W9 (30-80)</b>    | 0.08 | 0.06162 |
| <b>S3A2-S3B2 - W9 (80-130)</b>   | 0.10 | 0.06726 |
| <b>S3A2-S3B2 - W9 (130-165)</b>  | 0.05 | 0.06362 |
| <b>S3A2-S3B2 - W9 (165-195)</b>  | 0.26 | 0.06352 |
| <b>S3A2-S3B2 - W10 (0-40)</b>    | 0.03 | 0.06021 |
| <b>S3A2-S3B2 - W10 (40-80)</b>   | N/A  | 0.06347 |
| <b>S3A2-S3B2 - W10 (80-120)</b>  | 0.16 | 0.05822 |
| <b>S3A2-S3B2 - W10 (120-165)</b> | 0.19 | 0.09078 |
| <b>S3A2-S3B2 - W10 (165-195)</b> | 0.13 | 0.05945 |
| <b>S3A2-S3B2 - W11 (0-30)</b>    | 0.00 | N/A     |
| <b>S3A2-S3B2 - W11 (30-60)</b>   | 0.00 | N/A     |
| <b>S3A2-S3B2 - W11 (60-100)</b>  | 0.00 | N/A     |
| <b>S3A2-S3B2 - W11 (100-130)</b> | 0.00 | N/A     |
| <b>S3A2-S3B2 - W11 (130-160)</b> | 0.00 | N/A     |
| <b>S3A2-S3B2 - W11 (160-195)</b> | 0.00 | N/A     |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S3A2-S3B2 - W12 (0-30)</b>    | 0.08 | 0.05814 |
| <b>S3A2-S3B2 - W12 (30-65)</b>   | 0.19 | 0.05138 |
| <b>S3A2-S3B2 - W12 (65-95)</b>   | 0.01 | 0.05766 |
| <b>S3A2-S3B2 - W12 (95-125)</b>  | 0.29 | 0.05563 |
| <b>S3A2-S3B2 - W12 (125-150)</b> | 0.37 | 0.04573 |
| <b>S3A2-S3B2 - W12 (150-170)</b> | 0.29 | 0.04858 |
| <b>S3A2-S3B2 - W12 (170-195)</b> | 3.39 | N/A     |

Table 7-18. Set number 7 (S4A2-S4B2)

| Sample ID                | Mass loss<br>(%) | $\xi_u$ |
|--------------------------|------------------|---------|
| S4A2-S4B2 - W1 (0-50)    | 3.07             | 0.03714 |
| S4A2-S4B2 - W1 (50-100)  | 0.15             | 0.09052 |
| S4A2-S4B2 - W1 (100-150) | 1.02             | 0.07374 |
| S4A2-S4B2 - W1 (150-195) | 4.39             | 0.03047 |
| S4A2-S4B2 - W2 (0-50)    | 2.29             | 0.03717 |
| S4A2-S4B2 - W2 (50-90)   | 0.70             | 0.04921 |
| S4A2-S4B2 - W2 (90-135)  | 0.11             | 0.05695 |
| S4A2-S4B2 - W2 (135-195) | 0.58             | 0.06308 |
| S4A2-S4B2 - W3 (0-50)    | 0.20             | 0.05576 |
| S4A2-S4B2 - W3 (50-90)   | 0.59             | 0.04818 |
| S4A2-S4B2 - W3 (90-140)  | 0.15             | 0.05386 |
| S4A2-S4B2 - W3 (140-195) | 0.10             | 0.05744 |
| S4A2-S4B2 - W4 (0-50)    | 0.10             | 0.06453 |
| S4A2-S4B2 - W4 (50-90)   | 0.11             | 0.05866 |
| S4A2-S4B2 - W4 (90-145)  | 0.05             | 0.06510 |
| S4A2-S4B2 - W4 (145-195) | 0.11             | 0.05453 |
| S4A2-S4B2 - W5 (0-50)    | 0.21             | 0.06453 |
| S4A2-S4B2 - W5 (50-100)  | 0.05             | 0.04693 |
| S4A2-S4B2 - W5 (100-140) | 0.12             | 0.08952 |
| S4A2-S4B2 - W5 (140-195) | 0.32             | 0.04957 |
| S4A2-S4B2 - W6 (0-50)    | 0.17             | 0.04685 |
| S4A2-S4B2 - W6 (50-110)  | 1.50             | 0.03189 |
| S4A2-S4B2 - W6 (110-160) | 0.15             | 0.07161 |
| S4A2-S4B2 - W6 (160-195) | 0.09             | 0.07790 |
| S4A2-S4B2 - W7 (0-55)    | 0.01             | 0.05087 |
| S4A2-S4B2 - W7 (55-105)  | 0.31             | 0.04675 |
| S4A2-S4B2 - W7 (105-160) | 0.15             | 0.06135 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S4A2-S4B2 - W7 (160-195)</b>  | 0.03 | 0.06348 |
| <b>S4A2-S4B2 - W8 (0-45)</b>     | 0.09 | 0.05993 |
| <b>S4A2-S4B2 - W8 (45-95)</b>    | 0.08 | 0.06229 |
| <b>S4A2-S4B2 - W8 (95-145)</b>   | 0.10 | 0.04648 |
| <b>S4A2-S4B2 - W8 (145-195)</b>  | 0.08 | 0.05803 |
| <b>S4A2-S4B2 - W9 (0-45)</b>     | 0.13 | 0.06604 |
| <b>S4A2-S4B2 - W9 (45-100)</b>   | 0.14 | 0.05776 |
| <b>S4A2-S4B2 - W9 (100-160)</b>  | 0.17 | 0.06611 |
| <b>S4A2-S4B2 - W9 (160-195)</b>  | 0.27 | 0.06611 |
| <b>S4A2-S4B2 - W10 (0-50)</b>    | 0.13 | 0.06140 |
| <b>S4A2-S4B2 - W10 (50-100)</b>  | N/A  | 0.07152 |
| <b>S4A2-S4B2 - W10 (100-140)</b> | 0.06 | 0.08664 |
| <b>S4A2-S4B2 - W10 (140-195)</b> | 0.16 | 0.05986 |
| <b>S4A2-S4B2 - W11 (0-60)</b>    | 0.17 | 0.06670 |
| <b>S4A2-S4B2 - W11 (60-100)</b>  | 0.05 | 0.08697 |
| <b>S4A2-S4B2 - W11 (100-140)</b> | 0.05 | 0.07691 |
| <b>S4A2-S4B2 - W11 (140-195)</b> | 0.13 | 0.07509 |
| <b>S4A2-S4B2 - W12 (0-50)</b>    | 0.07 | 0.07497 |
| <b>S4A2-S4B2 - W12 (50-100)</b>  | 0.11 | 0.08371 |
| <b>S4A2-S4B2 - W12 (100-140)</b> | 0.02 | 0.11648 |
| <b>S4A2-S4B2 - W12 (140-195)</b> | 0.89 | 0.04810 |

Table 7-19. Set number 8 (S4A3-S4B3)

| Sample ID                | Mass<br>loss (%) | $\epsilon_u$ |
|--------------------------|------------------|--------------|
| S4A3-S4B3 - W1 (0-30)    | 0.23             | 0.06529      |
| S4A3-S4B3 - W1 (30-60)   | 0.05             | 0.06776      |
| S4A3-S4B3 - W1 (60-90)   | 0.14             | 0.06118      |
| S4A3-S4B3 - W1 (90-120)  | 0.26             | 0.05245      |
| S4A3-S4B3 - W1 (120-145) | 0.33             | 0.05071      |
| S4A3-S4B3 - W1 (145-165) | 0.09             | 0.06919      |
| S4A3-S4B3 - W1 (165-195) | 1.24             | 0.04979      |
| S4A3-S4B3 - W2 (0-25)    | 0.15             | 0.04987      |
| S4A3-S4B3 - W2 (25-50)   | 0.35             | 0.05734      |
| S4A3-S4B3 - W2 (50-75)   | 0.28             | 0.05254      |
| S4A3-S4B3 - W2 (75-105)  | 0.10             | 0.05207      |
| S4A3-S4B3 - W2 (105-135) | 0.11             | 0.05417      |
| S4A3-S4B3 - W2 (135-165) | 0.00             | 0.05307      |
| S4A3-S4B3 - W2 (165-195) | 0.07             | 0.04860      |
| S4A3-S4B3 - W3 (0-30)    | 0.03             | 0.06152      |
| S4A3-S4B3 - W3 (30-60)   | 0.30             | 0.06108      |
| S4A3-S4B3 - W3 (60-80)   | 0.04             | 0.06840      |
| S4A3-S4B3 - W3 (105-80)  | 0.26             | 0.06320      |
| S4A3-S4B3 - W3 (105-130) | 0.42             | 0.06377      |
| S4A3-S4B3 - W3 (130-160) | 0.07             | 0.05100      |
| S4A3-S4B3 - W3 (160-195) | 0.25             | 0.06546      |
| S4A3-S4B3 - W4 (0-30)    | 0.30             | 0.06142      |
| S4A3-S4B3 - W4 (30-60)   | 0.05             | 0.05184      |
| S4A3-S4B3 - W4 (60-90)   | 0.22             | 0.06472      |
| S4A3-S4B3 - W4 (90-125)  | 0.12             | 0.05769      |
| S4A3-S4B3 - W4 (125-155) | 0.13             | 0.06546      |
| S4A3-S4B3 - W4 (155-195) | 0.12             | 0.05592      |
| S4A3-S4B3 - W5 (0-30)    | 0.09             | 0.06588      |

|                          |      |         |
|--------------------------|------|---------|
| S4A3-S4B3 - W5 (30-70)   | 0.21 | 0.05889 |
| S4A3-S4B3 - W5 (70-110)  | 0.22 | 0.05308 |
| S4A3-S4B3 - W5 (110-135) | 0.17 | 0.05625 |
| S4A3-S4B3 - W5 (135-165) | 0.04 | 0.05822 |
| S4A3-S4B3 - W5 (165-195) | 0.09 | 0.04262 |
| <hr/>                    |      |         |
| S4A3-S4B3 - W6 (0-30)    | 0.16 | 0.05869 |
| S4A3-S4B3 - W6 (30-70)   | 0.05 | 0.06007 |
| S4A3-S4B3 - W6 (70-110)  | 0.23 | 0.06467 |
| S4A3-S4B3 - W6 (110-140) | 0.13 | 0.05958 |
| S4A3-S4B3 - W6 (140-165) | 0.09 | 0.06559 |
| S4A3-S4B3 - W6 (165-195) | 0.11 | 0.11186 |
| <hr/>                    |      |         |
| S4A3-S4B3 - W7 (0-30)    | 0.25 | 0.08890 |
| S4A3-S4B3 - W7 (30-70)   | 0.16 | 0.05658 |
| S4A3-S4B3 - W7 (70-110)  | 0.16 | 0.06923 |
| S4A3-S4B3 - W7 (110-145) | 0.04 | 0.07639 |
| S4A3-S4B3 - W7 (145-165) | 0.39 | 0.05091 |
| S4A3-S4B3 - W7 (165-195) | 0.15 | 0.05244 |
| <hr/>                    |      |         |
| S4A3-S4B3 - W8 (0-30)    | 0.30 | 0.06630 |
| S4A3-S4B3 - W8 (30-70)   | 0.09 | 0.06346 |
| S4A3-S4B3 - W8 (70-110)  | 0.12 | 0.07674 |
| S4A3-S4B3 - W8 (110-145) | 0.05 | 0.05361 |
| S4A3-S4B3 - W8 (145-170) | 0.21 | 0.06873 |
| S4A3-S4B3 - W8 (170-195) | 0.78 | 0.03288 |
| <hr/>                    |      |         |
| S4A3-S4B3 - W9 (0-30)    | 0.22 | 0.09117 |
| S4A3-S4B3 - W9 (30-70)   | 0.24 | 0.06899 |
| S4A3-S4B3 - W9 (70-110)  | 0.02 | 0.05620 |
| S4A3-S4B3 - W9 (110-165) | 0.16 | 0.05959 |
| S4A3-S4B3 - W9 (165-195) | 0.07 | 0.04495 |
| <hr/>                    |      |         |
| S4A3-S4B3 - W10 (0-30)   | 0.05 | 0.12583 |



|                           |      |         |
|---------------------------|------|---------|
| S4A3-S4B3 - W10 (30-70)   | 0.20 | 0.06787 |
| S4A3-S4B3 - W10 (70-115)  | 0.02 | 0.06571 |
| S4A3-S4B3 - W10 (115-165) | 0.13 | 0.07142 |
| S4A3-S4B3 - W10 (165-195) | 0.11 | 0.05966 |
| S4A3-S4B3 - W11 (0-30)    | 0.08 | 0.07801 |
| S4A3-S4B3 - W11 (30-70)   | 0.01 | 0.06479 |
| S4A3-S4B3 - W11 (70-110)  | 0.20 | 0.06085 |
| S4A3-S4B3 - W11 (110-140) | 0.22 | 0.07029 |
| S4A3-S4B3 - W11 (140-165) | 0.26 | 0.07541 |
| S4A3-S4B3 - W11 (165-195) | 0.14 | 0.06537 |
| S4A3-S4B3 - W12 (0-40)    | 0.05 | 0.05043 |
| S4A3-S4B3 - W12 (40-70)   | 0.13 | 0.05760 |
| S4A3-S4B3 - W12 (70-110)  | 0.06 | 0.05807 |
| S4A3-S4B3 - W12 (110-155) | 0.06 | 0.05986 |
| S4A3-S4B3 - W12 (155-195) | 0.19 | 0.04394 |

Table 7-20. Set number 9 (S5A2-S5B2)

| Sample ID                | Mass<br>loss (%) | $\epsilon_u$ |
|--------------------------|------------------|--------------|
| S5A2-S5B2 - W1 (0-30)    | 0.31             | 0.06940      |
| S5A2-S5B2 - W1 (30-70)   | 0.17             | 0.06075      |
| S5A2-S5B2 - W1 (70-125)  | 0.17             | 0.06885      |
| S5A2-S5B2 - W1 (125-165) | 2.04             | 0.06282      |
| S5A2-S5B2 - W1 (165-195) | 0.79             | 0.02782      |
| S5A2-S5B2 - W2 (0-30)    | 0.20             | 0.06830      |
| S5A2-S5B2 - W2 (30-70)   | 0.25             | 0.06830      |
| S5A2-S5B2 - W2 (70-125)  | 0.17             | 0.05919      |
| S5A2-S5B2 - W2 (125-165) | 0.50             | 0.06896      |
| S5A2-S5B2 - W2 (165-195) | 0.17             | 0.09670      |
| S5A2-S5B2 - W3 (0-30)    | 0.31             | 0.04714      |
| S5A2-S5B2 - W3 (30-70)   | 0.17             | 0.07115      |
| S5A2-S5B2 - W3 (70-125)  | 0.23             | 0.06734      |
| S5A2-S5B2 - W3 (125-165) | 2.21             | 0.06829      |
| S5A2-S5B2 - W3 (165-195) | 0.30             | 0.08424      |
| S5A2-S5B2 - W4 (0-30)    | 0.42             | 0.04980      |
| S5A2-S5B2 - W4 (30-70)   | 0.25             | 0.05923      |
| S5A2-S5B2 - W4 (70-125)  | 0.05             | 0.06397      |
| S5A2-S5B2 - W4 (125-165) | 0.17             | 0.07171      |
| S5A2-S5B2 - W4 (165-195) | 0.30             | 0.06601      |
| S5A2-S5B2 - W5 (0-30)    | 0.20             | 0.03382      |
| S5A2-S5B2 - W5 (30-70)   | 0.75             | 0.06582      |
| S5A2-S5B2 - W5 (70-125)  | 0.24             | 0.06007      |
| S5A2-S5B2 - W5 (125-165) | 0.33             | 0.07114      |
| S5A2-S5B2 - W5 (165-195) | 0.04             | 0.04869      |
| S5A2-S5B2 - W6 (0-30)    | 0.53             | 0.06111      |
| S5A2-S5B2 - W6 (30-70)   | 0.08             | 0.06021      |
| S5A2-S5B2 - W6 (70-125)  | 0.01             | 0.06227      |

|                           |      |         |
|---------------------------|------|---------|
| S5A2-S5B2 - W6 (125-165)  | 0.17 | 0.06040 |
| S5A2-S5B2 - W6 (165-195)  | 0.66 | 0.05349 |
| S5A2-S5B2 - W7 (0-30)     | 0.64 | 0.06145 |
| S5A2-S5B2 - W7 (30-70)    | 0.58 | 0.06459 |
| S5A2-S5B2 - W7 (70-125)   | 0.30 | 0.07123 |
| S5A2-S5B2 - W7 (125-165)  | 0.50 | 0.06485 |
| S5A2-S5B2 - W7 (165-195)  | 0.54 | 0.05564 |
| S5A2-S5B2 - W8 (0-30)     | 0.53 | 0.04641 |
| S5A2-S5B2 - W8 (30-70)    | 0.09 | 0.06732 |
| S5A2-S5B2 - W8 (70-125)   | 0.17 | 0.07126 |
| S5A2-S5B2 - W8 (125-165)  | 0.25 | 0.06697 |
| S5A2-S5B2 - W8 (165-195)  | 1.01 | 0.07038 |
| S5A2-S5B2 - W9 (0-30)     | 0.31 | 0.05679 |
| S5A2-S5B2 - W9 (30-70)    | 0.17 | 0.05099 |
| S5A2-S5B2 - W9 (70-125)   | 0.18 | 0.06478 |
| S5A2-S5B2 - W9 (125-165)  | 0.00 | 0.06621 |
| S5A2-S5B2 - W9 (165-195)  | 0.42 | 0.06575 |
| S5A2-S5B2 - W10 (0-30)    | 0.09 | 0.06774 |
| S5A2-S5B2 - W10 (30-70)   | 0.25 | 0.07824 |
| S5A2-S5B2 - W10 (70-125)  | 0.24 | 0.06903 |
| S5A2-S5B2 - W10 (125-165) | 0.50 | 0.06415 |
| S5A2-S5B2 - W10 (165-195) | 0.17 | 0.09873 |
| S5A2-S5B2 - W11 (0-30)    | 0.09 | 0.06836 |
| S5A2-S5B2 - W11 (30-70)   | 0.42 | 0.06809 |
| S5A2-S5B2 - W11 (70-125)  | 0.42 | 0.06595 |
| S5A2-S5B2 - W11 (125-165) | 0.42 | 0.04546 |
| S5A2-S5B2 - W11 (165-195) | 0.29 | 0.05943 |
| S5A2-S5B2 - W12 (0-30)    | 0.31 | 0.03978 |
| S5A2-S5B2 - W12 (30-70)   | 0.25 | 0.06332 |
| S5A2-S5B2 - W12 (70-125)  | 0.48 | 0.05848 |
| S5A2-S5B2 - W12 (125-165) | 0.83 | 0.07160 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S5A2-S5B2 - W12 (165-195)</b> | 0.54 | 0.07304 |
|----------------------------------|------|---------|

Table 7-21. Set number 10 (S5A3-S5B3)

| Sample ID                | Mass<br>loss (%) | $\epsilon_u$ |
|--------------------------|------------------|--------------|
| S5A3-S5B3 - W1 (0-30)    | 0.16             | 0.06496      |
| S5A3-S5B3 - W1 (30-70)   | 0.08             | 0.06833      |
| S5A3-S5B3 - W1 (70-125)  | 0.65             | 0.04076      |
| S5A3-S5B3 - W1 (125-165) | 1.71             | 0.07353      |
| S5A3-S5B3 - W1 (165-195) | 0.26             | 0.05523      |
| S5A3-S5B3 - W2 (0-30)    | 0.63             | 0.05925      |
| S5A3-S5B3 - W2 (30-70)   | 0.01             | 0.06485      |
| S5A3-S5B3 - W2 (70-125)  | 0.91             | 0.04521      |
| S5A3-S5B3 - W2 (125-165) | 0.14             | 0.07560      |
| S5A3-S5B3 - W2 (165-195) | 0.01             | 0.09761      |
| S5A3-S5B3 - W3 (0-30)    | 0.12             | 0.06819      |
| S5A3-S5B3 - W3 (30-70)   | 0.15             | 0.07751      |
| S5A3-S5B3 - W3 (70-125)  | 0.08             | 0.06388      |
| S5A3-S5B3 - W3 (125-165) | 0.15             | 0.06624      |
| S5A3-S5B3 - W3 (165-195) | 0.24             | 0.06462      |
| S5A3-S5B3 - W4 (0-30)    | 1.85             | 0.05772      |
| S5A3-S5B3 - W4 (30-70)   | 0.21             | 0.08051      |
| S5A3-S5B3 - W4 (70-125)  | 0.13             | 0.06870      |
| S5A3-S5B3 - W4 (125-165) | 0.54             | 0.06935      |
| S5A3-S5B3 - W4 (165-195) | 0.26             | 0.06450      |
| S5A3-S5B3 - W5 (0-30)    | 0.30             | 0.08669      |
| S5A3-S5B3 - W5 (30-70)   | 0.21             | 0.09068      |
| S5A3-S5B3 - W5 (70-125)  | 1.46             | 0.06595      |
| S5A3-S5B3 - W5 (125-165) | 0.15             | 0.07446      |
| S5A3-S5B3 - W5 (165-195) | 0.81             | 0.06047      |
| S5A3-S5B3 - W6 (0-30)    | 0.09             | 0.06123      |
| S5A3-S5B3 - W6 (30-70)   | 0.24             | 0.06687      |
| S5A3-S5B3 - W6 (70-125)  | 0.06             | 0.06142      |

|                           |       |         |
|---------------------------|-------|---------|
| S5A3-S5B3 - W6 (125-165)  | 0.63  | 0.05403 |
| S5A3-S5B3 - W6 (165-195)  | 0.17  | 0.06543 |
| S5A3-S5B3 - W7 (0-30)     | 0.02  | 0.06239 |
| S5A3-S5B3 - W7 (30-70)    | 0.89  | 0.06629 |
| S5A3-S5B3 - W7 (70-125)   | 0.17  | 0.06767 |
| S5A3-S5B3 - W7 (125-165)  | 0.02  | 0.07012 |
| S5A3-S5B3 - W7 (165-195)  | 10.21 | 0.05809 |
| S5A3-S5B3 - W8 (0-30)     | 2.11  | 0.05726 |
| S5A3-S5B3 - W8 (30-70)    | 0.27  | 0.06044 |
| S5A3-S5B3 - W8 (70-125)   | 0.01  | 0.06604 |
| S5A3-S5B3 - W8 (125-165)  | 1.11  | 0.06890 |
| S5A3-S5B3 - W8 (165-195)  | 0.15  | 0.07424 |
| S5A3-S5B3 - W9 (0-30)     | 0.12  | 0.05194 |
| S5A3-S5B3 - W9 (30-70)    | 1.09  | 0.06899 |
| S5A3-S5B3 - W9 (70-125)   | 0.99  | 0.06320 |
| S5A3-S5B3 - W9 (125-165)  | 0.09  | 0.06230 |
| S5A3-S5B3 - W9 (165-195)  | 0.30  | 0.08329 |
| S5A3-S5B3 - W10 (0-30)    | 0.12  | 0.04413 |
| S5A3-S5B3 - W10 (30-70)   | 0.22  | 0.04225 |
| S5A3-S5B3 - W10 (70-125)  | 0.03  | 0.04284 |
| S5A3-S5B3 - W10 (125-165) | 0.20  | 0.05528 |
| S5A3-S5B3 - W10 (165-195) | 0.06  | 0.05469 |
| S5A3-S5B3 - W11 (0-30)    | 0.26  | 0.06936 |
| S5A3-S5B3 - W11 (30-70)   | 0.10  | 0.06665 |
| S5A3-S5B3 - W11 (70-125)  | 0.09  | 0.06792 |
| S5A3-S5B3 - W11 (125-165) | 0.12  | 0.05551 |
| S5A3-S5B3 - W11 (165-195) | 0.11  | 0.07058 |
| S5A3-S5B3 - W12 (0-30)    | 0.17  | 0.05727 |
| S5A3-S5B3 - W12 (30-70)   | 0.11  | 0.07110 |
| S5A3-S5B3 - W12 (70-125)  | 0.10  | 0.06315 |
| S5A3-S5B3 - W12 (125-165) | 0.21  | 0.06412 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S5A3-S5B3 - W12 (165-195)</b> | 3.80 | 0.05896 |
|----------------------------------|------|---------|

Table 7-22. Set number 11 (S6A2-S6B2)

| Sample ID                | Mass<br>loss (%) | $\epsilon_u$ |
|--------------------------|------------------|--------------|
| S6A2-S6B2 - W1 (0-30)    | 0.19             | 0.05887      |
| S6A2-S6B2 - W1 (30-70)   | 0.07             | 0.06351      |
| S6A2-S6B2 - W1 (70-125)  | 0.07             | 0.06314      |
| S6A2-S6B2 - W1 (125-165) | 1.06             | 0.05690      |
| S6A2-S6B2 - W1 (165-195) | 1.13             | 0.02782      |
| S6A2-S6B2 - W2 (0-30)    | 0.32             | 0.06205      |
| S6A2-S6B2 - W2 (30-70)   | 1.50             | 0.06306      |
| S6A2-S6B2 - W2 (70-125)  | 0.02             | 0.05559      |
| S6A2-S6B2 - W2 (125-165) | 0.08             | 0.06176      |
| S6A2-S6B2 - W2 (165-195) | 0.11             | 0.04380      |
| S6A2-S6B2 - W3 (0-30)    | 0.06             | 0.06899      |
| S6A2-S6B2 - W3 (30-70)   | 0.16             | 0.06543      |
| S6A2-S6B2 - W3 (70-125)  | 0.04             | 0.06124      |
| S6A2-S6B2 - W3 (125-165) | 0.03             | 0.06164      |
| S6A2-S6B2 - W3 (165-195) | 0.07             | 0.05761      |
| S6A2-S6B2 - W4 (0-30)    | 0.26             | 0.06822      |
| S6A2-S6B2 - W4 (30-70)   | 0.23             | 0.07301      |
| S6A2-S6B2 - W4 (70-125)  | 0.04             | 0.07461      |
| S6A2-S6B2 - W4 (125-165) | 0.09             | 0.07111      |
| S6A2-S6B2 - W4 (165-195) | 0.26             | 0.05159      |
| S6A2-S6B2 - W5 (0-30)    | 0.46             | 0.06264      |
| S6A2-S6B2 - W5 (30-70)   | 0.12             | 0.05728      |
| S6A2-S6B2 - W5 (70-125)  | 0.14             | 0.05780      |
| S6A2-S6B2 - W5 (125-165) | 0.18             | 0.06243      |
| S6A2-S6B2 - W5 (165-195) | 0.08             | 0.05023      |
| S6A2-S6B2 - W6 (0-30)    | 0.06             | 0.06628      |
| S6A2-S6B2 - W6 (30-70)   | 0.24             | 0.06064      |
| S6A2-S6B2 - W6 (70-125)  | 0.15             | 0.05177      |



|                           |      |         |
|---------------------------|------|---------|
| S6A2-S6B2 - W6 (125-165)  | 0.18 | 0.08018 |
| S6A2-S6B2 - W6 (165-195)  | 0.30 | 0.04385 |
| S6A2-S6B2 - W7 (0-30)     | 1.06 | 0.05964 |
| S6A2-S6B2 - W7 (30-70)    | 0.12 | 0.05606 |
| S6A2-S6B2 - W7 (70-125)   | 0.57 | 0.05393 |
| S6A2-S6B2 - W7 (125-165)  | 0.82 | 0.06546 |
| S6A2-S6B2 - W7 (165-195)  | 2.84 | 0.05286 |
| S6A2-S6B2 - W8 (0-30)     | 0.35 | 0.06345 |
| S6A2-S6B2 - W8 (30-70)    | 0.16 | 0.05694 |
| S6A2-S6B2 - W8 (70-125)   | 0.01 | 0.06003 |
| S6A2-S6B2 - W8 (125-165)  | 0.22 | 0.06279 |
| S6A2-S6B2 - W8 (165-195)  | 1.09 | 0.04581 |
| S6A2-S6B2 - W9 (0-30)     | 2.37 | 0.06827 |
| S6A2-S6B2 - W9 (30-70)    | 0.10 | 0.10708 |
| S6A2-S6B2 - W9 (70-125)   | 0.10 | 0.05899 |
| S6A2-S6B2 - W9 (125-165)  | 0.08 | 0.06415 |
| S6A2-S6B2 - W9 (165-195)  | 0.27 | 0.05066 |
| S6A2-S6B2 - W10 (0-30)    | 0.15 | 0.06123 |
| S6A2-S6B2 - W10 (30-70)   | 0.86 | 0.06553 |
| S6A2-S6B2 - W10 (70-125)  | 0.07 | 0.05746 |
| S6A2-S6B2 - W10 (125-165) | 0.88 | 0.05848 |
| S6A2-S6B2 - W10 (165-195) | 0.02 | 0.05071 |
| S6A2-S6B2 - W11 (0-30)    | N/A  | N/A     |
| S6A2-S6B2 - W11 (30-70)   | 0.22 | 0.06094 |
| S6A2-S6B2 - W11 (70-125)  | 0.24 | 0.06588 |
| S6A2-S6B2 - W11 (125-165) | 0.07 | 0.05045 |
| S6A2-S6B2 - W11 (165-195) | 0.20 | 0.06123 |
| S6A2-S6B2 - W12 (0-30)    | 0.21 | 0.06067 |
| S6A2-S6B2 - W12 (30-70)   | 0.17 | 0.06674 |
| S6A2-S6B2 - W12 (70-125)  | 0.02 | 0.05638 |
| S6A2-S6B2 - W12 (125-165) | 0.12 | 0.06380 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S6A2-S6B2 - W12 (165-195)</b> | 0.79 | 0.03976 |
|----------------------------------|------|---------|

Table 7-23. Set number 12 (S6A3-S6B3)

| Sample ID                | Mass<br>loss (%) | $\epsilon_u$ |
|--------------------------|------------------|--------------|
| S6A3-S6B3 - W1 (0-30)    | 0.23             | 0.06204      |
| S6A3-S6B3 - W1 (30-70)   | 0.23             | 0.06018      |
| S6A3-S6B3 - W1 (70-125)  | 0.06             | 0.06936      |
| S6A3-S6B3 - W1 (125-165) | 0.16             | 0.05876      |
| S6A3-S6B3 - W1 (165-195) | 0.27             | 0.05399      |
| S6A3-S6B3 - W2 (0-30)    | 0.30             | 0.07788      |
| S6A3-S6B3 - W2 (30-70)   | 0.23             | 0.06143      |
| S6A3-S6B3 - W2 (70-125)  | 0.04             | 0.06053      |
| S6A3-S6B3 - W2 (125-165) | 0.18             | 0.06331      |
| S6A3-S6B3 - W2 (165-195) | 0.31             | 0.10796      |
| S6A3-S6B3 - W3 (0-30)    | 0.14             | 0.07288      |
| S6A3-S6B3 - W3 (30-70)   | 0.07             | 0.06205      |
| S6A3-S6B3 - W3 (70-125)  | 0.06             | 0.05970      |
| S6A3-S6B3 - W3 (125-165) | 0.04             | 0.05336      |
| S6A3-S6B3 - W3 (165-195) | 2.03             | 0.05413      |
| S6A3-S6B3 - W4 (0-30)    | 0.65             | 0.05738      |
| S6A3-S6B3 - W4 (30-70)   | 0.06             | 0.05831      |
| S6A3-S6B3 - W4 (70-125)  | 0.04             | 0.07461      |
| S6A3-S6B3 - W4 (125-165) | 0.03             | 0.06289      |
| S6A3-S6B3 - W4 (165-195) | 0.54             | 0.05146      |
| S6A3-S6B3 - W5 (0-30)    | 0.23             | 0.08738      |
| S6A3-S6B3 - W5 (30-70)   | 0.24             | 0.10816      |
| S6A3-S6B3 - W5 (70-125)  | 5.52             | 0.06368      |
| S6A3-S6B3 - W5 (125-165) | 0.19             | 0.10513      |
| S6A3-S6B3 - W5 (165-195) | 0.06             | 0.05332      |
| S6A3-S6B3 - W6 (0-30)    | 0.29             | 0.06249      |
| S6A3-S6B3 - W6 (30-70)   | 0.24             | 0.05599      |
| S6A3-S6B3 - W6 (70-125)  | 0.05             | 0.05589      |

|                           |      |         |
|---------------------------|------|---------|
| S6A3-S6B3 - W6 (125-165)  | 0.17 | 0.07373 |
| S6A3-S6B3 - W6 (165-195)  | 0.25 | 0.05651 |
| S6A3-S6B3 - W7 (0-30)     | 0.69 | 0.05582 |
| S6A3-S6B3 - W7 (30-70)    | 0.17 | 0.04551 |
| S6A3-S6B3 - W7 (70-125)   | 0.10 | 0.06157 |
| S6A3-S6B3 - W7 (125-165)  | 0.06 | 0.05655 |
| S6A3-S6B3 - W7 (165-195)  | 0.28 | 0.06629 |
| S6A3-S6B3 - W8 (0-30)     | 0.22 | 0.08624 |
| S6A3-S6B3 - W8 (30-70)    | 0.07 | 0.06067 |
| S6A3-S6B3 - W8 (70-125)   | 0.02 | 0.05293 |
| S6A3-S6B3 - W8 (125-165)  | 0.16 | 0.05709 |
| S6A3-S6B3 - W8 (165-195)  | 3.18 | 0.05564 |
| S6A3-S6B3 - W9 (0-30)     | 0.26 | 0.07075 |
| S6A3-S6B3 - W9 (30-70)    | 0.10 | 0.06469 |
| S6A3-S6B3 - W9 (70-125)   | 0.15 | 0.05818 |
| S6A3-S6B3 - W9 (125-165)  | 0.02 | 0.04824 |
| S6A3-S6B3 - W9 (165-195)  | 0.06 | 0.06014 |
| S6A3-S6B3 - W10 (0-30)    | 0.12 | 0.09550 |
| S6A3-S6B3 - W10 (30-70)   | 0.14 | 0.06237 |
| S6A3-S6B3 - W10 (70-125)  | 0.08 | 0.08700 |
| S6A3-S6B3 - W10 (125-165) | 4.07 | 0.06003 |
| S6A3-S6B3 - W10 (165-195) | 0.31 | 0.06478 |
| S6A3-S6B3 - W11 (0-30)    | 0.06 | 0.08086 |
| S6A3-S6B3 - W11 (30-70)   | 0.21 | 0.05839 |
| S6A3-S6B3 - W11 (70-125)  | 0.13 | 0.08949 |
| S6A3-S6B3 - W11 (125-165) | 0.96 | 0.04235 |
| S6A3-S6B3 - W11 (165-195) | 0.86 | 0.04059 |
| S6A3-S6B3 - W12 (0-30)    | 0.02 | 0.07547 |
| S6A3-S6B3 - W12 (30-70)   | 0.01 | 0.04310 |
| S6A3-S6B3 - W12 (70-125)  | 0.16 | 0.06408 |
| S6A3-S6B3 - W12 (125-165) | 0.19 | 0.08528 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S6A3-S6B3 - W12 (165-195)</b> | 0.18 | 0.05450 |
|----------------------------------|------|---------|

Table 7-24. Set number 13 (S7A2-S7B2)

| Sample ID                | Mass<br>loss (%) | $\epsilon_u$ |
|--------------------------|------------------|--------------|
| S7A2-S7B2 - W1 (0-30)    | 0.33             | 0.05945      |
| S7A2-S7B2 - W1 (30-60)   | 0.02             | 0.05793      |
| S7A2-S7B2 - W1 (60-90)   | 0.16             | 0.06279      |
| S7A2-S7B2 - W1 (90-130)  | 0.12             | 0.05562      |
| S7A2-S7B2 - W1 (130-160) | 0.11             | 0.04809      |
| S7A2-S7B2 - W1 (160-195) | 0.03             | 0.06556      |
| S7A2-S7B2 - W2 (0-30)    | 0.38             | 0.05895      |
| S7A2-S7B2 - W2 (30-60)   | 0.09             | 0.07035      |
| S7A2-S7B2 - W2 (60-90)   | 0.02             | 0.06700      |
| S7A2-S7B2 - W2 (90-125)  | 0.05             | 0.06675      |
| S7A2-S7B2 - W2 (125-165) | 0.12             | 0.05855      |
| S7A2-S7B2 - W2 (165-195) | 0.16             | 0.06796      |
| S7A2-S7B2 - W3 (0-30)    | 1.15             | 0.03751      |
| S7A2-S7B2 - W3 (30-55)   | 0.75             | 0.06524      |
| S7A2-S7B2 - W3 (55-80)   | 0.08             | 0.06850      |
| S7A2-S7B2 - W3 (80-110)  | 0.01             | 0.06476      |
| S7A2-S7B2 - W3 (110-140) | 0.16             | 0.06244      |
| S7A2-S7B2 - W3 (140-165) | 0.12             | 0.07217      |
| S7A2-S7B2 - W3 (165-195) | 0.79             | 0.05672      |
| S7A2-S7B2 - W4 (0-30)    | 4.86             | 0.03546      |
| S7A2-S7B2 - W4 (30-60)   | 0.73             | 0.06481      |
| S7A2-S7B2 - W4 (60-95)   | 2.38             | 0.06601      |
| S7A2-S7B2 - W4 (95-130)  | 0.05             | 0.06713      |
| S7A2-S7B2 - W4 (130-165) | 0.30             | 0.06698      |
| S7A2-S7B2 - W4 (165-195) | 0.10             | 0.04529      |
| S7A2-S7B2 - W5 (0-30)    | 0.29             | 0.06003      |
| S7A2-S7B2 - W5 (30-60)   | 19.00            | 0.06269      |
| S7A2-S7B2 - W5 (60-100)  | 0.15             | 0.06786      |
| S7A2-S7B2 - W5 (100-130) | 0.13             | 0.05883      |

|                          |      |         |
|--------------------------|------|---------|
| S7A2-S7B2 - W5 (130-165) | 0.26 | 0.05946 |
| S7A2-S7B2 - W5 (165-195) | 0.32 | 0.07799 |
| S7A2-S7B2 - W6 (0-30)    | 0.06 | 0.06835 |
| S7A2-S7B2 - W6 (30-60)   | 0.02 | 0.06920 |
| S7A2-S7B2 - W6 (60-100)  | 1.19 | 0.05859 |
| S7A2-S7B2 - W6 (100-130) | 0.07 | 0.05830 |
| S7A2-S7B2 - W6 (130-160) | 6.06 | 0.05589 |
| S7A2-S7B2 - W6 (160-195) | 0.13 | 0.06468 |
| S7A2-S7B2 - W7 (0-30)    | 0.88 | 0.04872 |
| S7A2-S7B2 - W7 (30-60)   | 0.20 | 0.05851 |
| S7A2-S7B2 - W7 (60-90)   | 0.13 | 0.05787 |
| S7A2-S7B2 - W7 (90-115)  | 0.34 | 0.05999 |
| S7A2-S7B2 - W7 (115-140) | 0.32 | 0.04807 |
| S7A2-S7B2 - W7 (140-165) | 0.08 | 0.06964 |
| S7A2-S7B2 - W7 (165-195) | 0.14 | 0.06574 |
| S7A2-S7B2 - W8 (0-30)    | 0.98 | 0.04264 |
| S7A2-S7B2 - W8 (30-60)   | N/A  | N/A     |
| S7A2-S7B2 - W8 (60-85)   | 0.34 | 0.06510 |
| S7A2-S7B2 - W8 (85-110)  | 0.82 | 0.05784 |
| S7A2-S7B2 - W8 (110-140) | 0.16 | 0.05219 |
| S7A2-S7B2 - W8 (140-165) | 0.03 | 0.07265 |
| S7A2-S7B2 - W8 (165-195) | 0.09 | 0.04198 |
| S7A2-S7B2 - W9 (0-30)    | 0.05 | 0.04646 |
| S7A2-S7B2 - W9 (30-60)   | 0.16 | 0.06614 |
| S7A2-S7B2 - W9 (60-100)  | 2.32 | 0.06366 |
| S7A2-S7B2 - W9 (100-135) | 0.37 | 0.02140 |
| S7A2-S7B2 - W9 (135-165) | 0.12 | 0.06606 |
| S7A2-S7B2 - W9 (165-195) | 0.27 | 0.06214 |
| S7A2-S7B2 - W10 (0-30)   | 0.14 | 0.05512 |
| S7A2-S7B2 - W10 (30-60)  | 5.33 | 0.05849 |
| S7A2-S7B2 - W10 (60-90)  | 0.12 | 0.05915 |
| S7A2-S7B2 - W10 (90-120) | 0.10 | 0.06060 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S7A2-S7B2 - W10 (120-150)</b> | 0.25 | 0.06561 |
| <b>S7A2-S7B2 - W10 (150-170)</b> | 0.04 | 0.06856 |
| <b>S7A2-S7B2 - W10 (170-195)</b> | 0.34 | 0.06430 |
| <b>S7A2-S7B2 - W11 (0-30)</b>    | 0.73 | 0.05491 |
| <b>S7A2-S7B2 - W11 (30-60)</b>   | 0.29 | 0.06829 |
| <b>S7A2-S7B2 - W11 (60-100)</b>  | 1.31 | 0.05881 |
| <b>S7A2-S7B2 - W11 (100-140)</b> | 0.11 | 0.08506 |
| <b>S7A2-S7B2 - W11 (140-165)</b> | 0.13 | 0.09336 |
| <b>S7A2-S7B2 - W11 (165-195)</b> | 0.09 | 0.05268 |
| <b>S7A2-S7B2 - W12 (0-30)</b>    | 0.15 | 0.04959 |
| <b>S7A2-S7B2 - W12 (30-60)</b>   | 0.03 | 0.06152 |
| <b>S7A2-S7B2 - W12 (60-90)</b>   | 0.08 | 0.06339 |
| <b>S7A2-S7B2 - W12 (90-115)</b>  | 0.14 | 0.06882 |
| <b>S7A2-S7B2 - W12 (115-145)</b> | 0.08 | 0.04669 |
| <b>S7A2-S7B2 - W12 (145-165)</b> | 0.40 | 0.06229 |
| <b>S7A2-S7B2 - W12 (165-195)</b> | 0.05 | 0.06694 |



Table 7-25. Set number 14 (S7A3-S7B3)

| Sample ID                | Mass<br>loss (%) | $\epsilon_u$ |
|--------------------------|------------------|--------------|
| S7A3-S7B3 - W1 (0-30)    | 0.15             | 0.06208      |
| S7A3-S7B3 - W1 (30-55)   | 0.21             | 0.06288      |
| S7A3-S7B3 - W1 (55-85)   | 0.15             | 0.06770      |
| S7A3-S7B3 - W1 (85-115)  | 0.02             | 0.05793      |
| S7A3-S7B3 - W1 (115-145) | 0.04             | 0.06323      |
| S7A3-S7B3 - W1 (145-165) | 0.27             | 0.06314      |
| S7A3-S7B3 - W1 (165-195) | 0.29             | 0.06771      |
| S7A3-S7B3 - W2 (0-40)    | 0.20             | 0.05191      |
| S7A3-S7B3 - W2 (40-65)   | 0.30             | 0.06010      |
| S7A3-S7B3 - W2 (65-95)   | 0.23             | 0.05791      |
| S7A3-S7B3 - W2 (95-125)  | 0.03             | 0.06745      |
| S7A3-S7B3 - W2 (125-155) | 0.22             | 0.06405      |
| S7A3-S7B3 - W2 (155-175) | 0.13             | 0.06686      |
| S7A3-S7B3 - W2 (175-195) | 0.21             | 0.05804      |
| S7A3-S7B3 - W3 (0-25)    | 0.29             | 0.06567      |
| S7A3-S7B3 - W3 (25-50)   | 0.37             | 0.06225      |
| S7A3-S7B3 - W3 (50-80)   | 0.03             | 0.06868      |
| S7A3-S7B3 - W3 (80-120)  | 0.17             | 0.06610      |
| S7A3-S7B3 - W3 (120-150) | 0.29             | 0.06539      |
| S7A3-S7B3 - W3 (150-165) | 0.19             | 0.06627      |
| S7A3-S7B3 - W3 (165-195) | 0.07             | 0.05579      |
| S7A3-S7B3 - W4 (0-40)    | 0.11             | 0.06144      |
| S7A3-S7B3 - W4 (40-70)   | 0.20             | 0.06466      |
| S7A3-S7B3 - W4 (70-100)  | 0.08             | 0.06473      |
| S7A3-S7B3 - W4 (100-130) | 0.29             | 0.06294      |
| S7A3-S7B3 - W4 (130-165) | 0.01             | 0.06460      |
| S7A3-S7B3 - W4 (165-195) | 0.08             | 0.06684      |
| S7A3-S7B3 - W5 (0-30)    | 0.04             | 0.06500      |
| S7A3-S7B3 - W5 (30-60)   | 0.28             | 0.06367      |

|                                 |      |         |
|---------------------------------|------|---------|
| <b>S7A3-S7B3 - W5 (60-95)</b>   | 0.10 | 0.07117 |
| <b>S7A3-S7B3 - W5 (95-130)</b>  | 0.27 | 0.06531 |
| <b>S7A3-S7B3 - W5 (130-170)</b> | 0.01 | 0.07167 |
| <b>S7A3-S7B3 - W5 (170-195)</b> | 0.34 | 0.06512 |
| <b>S7A3-S7B3 - W6 (0-30)</b>    | 0.22 | 0.08695 |
| <b>S7A3-S7B3 - W6 (30-50)</b>   | 0.28 | 0.06262 |
| <b>S7A3-S7B3 - W6 (50-85)</b>   | 0.10 | 0.05934 |
| <b>S7A3-S7B3 - W6 (85-105)</b>  | 0.28 | 0.05927 |
| <b>S7A3-S7B3 - W6 (105-125)</b> | 0.47 | 0.05766 |
| <b>S7A3-S7B3 - W6 (125-155)</b> | 0.10 | 0.06397 |
| <b>S7A3-S7B3 - W6 (155-175)</b> | 0.31 | 0.06304 |
| <b>S7A3-S7B3 - W6 (175-195)</b> | 2.88 | 0.05873 |
| <b>S7A3-S7B3 - W7 (0-30)</b>    | 0.19 | 0.07064 |
| <b>S7A3-S7B3 - W7 (30-55)</b>   | 0.32 | 0.07110 |
| <b>S7A3-S7B3 - W7 (55-85)</b>   | 0.05 | 0.06638 |
| <b>S7A3-S7B3 - W7 (85-115)</b>  | 0.22 | 0.08355 |
| <b>S7A3-S7B3 - W7 (115-145)</b> | 0.09 | 0.07284 |
| <b>S7A3-S7B3 - W7 (145-170)</b> | 0.04 | 0.06545 |
| <b>S7A3-S7B3 - W7 (170-195)</b> | 0.14 | 0.06550 |
| <b>S7A3-S7B3 - W8 (0-40)</b>    | 0.11 | 0.06724 |
| <b>S7A3-S7B3 - W8 (40-70)</b>   | 0.15 | 0.05932 |
| <b>S7A3-S7B3 - W8 (70-100)</b>  | 0.20 | 0.06106 |
| <b>S7A3-S7B3 - W8 (100-140)</b> | 0.19 | 0.06103 |
| <b>S7A3-S7B3 - W8 (140-165)</b> | 0.36 | 0.06358 |
| <b>S7A3-S7B3 - W8 (165-195)</b> | 0.31 | 0.06556 |
| <b>S7A3-S7B3 - W9 (0-40)</b>    | 0.24 | 0.06846 |
| <b>S7A3-S7B3 - W9 (40-75)</b>   | 1.24 | 0.06308 |
| <b>S7A3-S7B3 - W9 (75-105)</b>  | 0.01 | 0.07032 |
| <b>S7A3-S7B3 - W9 (105-135)</b> | 2.77 | 0.06166 |
| <b>S7A3-S7B3 - W9 (135-165)</b> | 0.24 | 0.06784 |
| <b>S7A3-S7B3 - W9 (165-195)</b> | 0.18 | 0.06463 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S7A3-S7B3 - W10 (0-30)</b>    | 0.28 | 0.05130 |
| <b>S7A3-S7B3 - W10 (30-75)</b>   | 0.02 | 0.07503 |
| <b>S7A3-S7B3 - W10 (75-110)</b>  | 0.08 | 0.06333 |
| <b>S7A3-S7B3 - W10 (110-140)</b> | 0.02 | 0.06521 |
| <b>S7A3-S7B3 - W10 (140-165)</b> | 0.36 | 0.06213 |
| <b>S7A3-S7B3 - W10 (165-195)</b> | 0.28 | 0.05554 |
| <b>S7A3-S7B3 - W11 (0-40)</b>    | 0.05 | 0.06437 |
| <b>S7A3-S7B3 - W11 (40-80)</b>   | 0.23 | 0.05620 |
| <b>S7A3-S7B3 - W11 (80-110)</b>  | 0.23 | 0.05678 |
| <b>S7A3-S7B3 - W11 (110-140)</b> | 0.23 | 0.06009 |
| <b>S7A3-S7B3 - W11 (140-165)</b> | 0.13 | 0.06010 |
| <b>S7A3-S7B3 - W11 (165-195)</b> | 0.21 | 0.06005 |
| <b>S7A3-S7B3 - W12 (0-40)</b>    | 0.21 | 0.06972 |
| <b>S7A3-S7B3 - W12 (40-80)</b>   | 0.04 | 0.06562 |
| <b>S7A3-S7B3 - W12 (80-100)</b>  | 0.01 | 0.06544 |
| <b>S7A3-S7B3 - W12 (100-130)</b> | 0.17 | 0.06385 |
| <b>S7A3-S7B3 - W12 (130-165)</b> | 0.06 | 0.06626 |
| <b>S7A3-S7B3 - W12 (165-195)</b> | 0.26 | 0.06060 |

Table 7-26. Set number 15 (S8A3-S8B3)

| Sample ID                | Mass<br>loss (%) | $\epsilon_u$ |
|--------------------------|------------------|--------------|
| S8A3-S8B3 - W1 (0-40)    | 0.13             | 0.06555      |
| S8A3-S8B3 - W1 (40-70)   | 0.13             | 0.07039      |
| S8A3-S8B3 - W1 (70-100)  | 0.25             | 0.06460      |
| S8A3-S8B3 - W1 (100-135) | 0.04             | 0.07687      |
| S8A3-S8B3 - W1 (135-165) | 0.26             | 0.06911      |
| S8A3-S8B3 - W1 (165-195) | 0.04             | 0.07386      |
| S8A3-S8B3 - W2 (0-40)    | 0.13             | 0.05909      |
| S8A3-S8B3 - W2 (40-80)   | 0.07             | 0.05726      |
| S8A3-S8B3 - W2 (80-100)  | 0.44             | 0.06556      |
| S8A3-S8B3 - W2 (100-130) | 0.14             | 0.06017      |
| S8A3-S8B3 - W2 (130-160) | 0.29             | 0.06196      |
| S8A3-S8B3 - W2 (160-195) | 0.94             | 0.03218      |
| S8A3-S8B3 - W3 (0-40)    | 0.13             | 0.06668      |
| S8A3-S8B3 - W3 (40-80)   | 0.21             | 0.06417      |
| S8A3-S8B3 - W3 (80-120)  | 0.09             | 0.06544      |
| S8A3-S8B3 - W3 (120-160) | 0.05             | 0.06509      |
| S8A3-S8B3 - W3 (160-195) | 0.06             | 0.05701      |
| S8A3-S8B3 - W4 (0-40)    | 0.03             | 0.06563      |
| S8A3-S8B3 - W4 (40-80)   | 0.01             | 0.06291      |
| S8A3-S8B3 - W4 (80-120)  | 0.13             | 0.06809      |
| S8A3-S8B3 - W4 (120-160) | 0.11             | 0.13400      |
| S8A3-S8B3 - W4 (160-195) | 0.11             | 0.06440      |
| S8A3-S8B3 - W5 (0-40)    | 0.22             | 0.06214      |
| S8A3-S8B3 - W5 (40-80)   | 0.16             | 0.05950      |
| S8A3-S8B3 - W5 (80-120)  | 0.19             | 0.06698      |
| S8A3-S8B3 - W5 (120-160) | 0.18             | 0.06358      |
| S8A3-S8B3 - W5 (160-195) | 0.24             | 0.05327      |
| S8A3-S8B3 - W6 (0-40)    | 0.16             | 0.06632      |

|                           |      |         |
|---------------------------|------|---------|
| S8A3-S8B3 - W6 (40-80)    | 0.13 | 0.05822 |
| S8A3-S8B3 - W6 (80-120)   | 0.15 | 0.06339 |
| S8A3-S8B3 - W6 (120-160)  | 0.15 | 0.05686 |
| S8A3-S8B3 - W6 (160-195)  | 0.11 | 0.06571 |
| S8A3-S8B3 - W7 (0-40)     | 0.23 | 0.06652 |
| S8A3-S8B3 - W7 (40-80)    | 0.15 | 0.06195 |
| S8A3-S8B3 - W7 (80-120)   | 0.06 | 0.06580 |
| S8A3-S8B3 - W7 (120-160)  | 0.25 | 0.06302 |
| S8A3-S8B3 - W7 (160-195)  | 0.12 | 0.06979 |
| S8A3-S8B3 - W8 (0-40)     | 0.24 | 0.06607 |
| S8A3-S8B3 - W8 (40-80)    | 0.24 | 0.06470 |
| S8A3-S8B3 - W8 (80-110)   | 0.28 | 0.06455 |
| S8A3-S8B3 - W8 (110-135)  | 0.19 | 0.06849 |
| S8A3-S8B3 - W8 (135-165)  | 0.62 | 0.06614 |
| S8A3-S8B3 - W8 (165-195)  | 0.27 | 0.07241 |
| S8A3-S8B3 - W9 (0-40)     | 0.16 | 0.06718 |
| S8A3-S8B3 - W9 (40-80)    | 0.03 | 0.06660 |
| S8A3-S8B3 - W9 (80-120)   | 0.12 | 0.06961 |
| S8A3-S8B3 - W9 (120-160)  | 0.23 | 0.06615 |
| S8A3-S8B3 - W9 (160-195)  | 0.54 | 0.05580 |
| S8A3-S8B3 - W10 (0-30)    | 0.28 | 0.06274 |
| S8A3-S8B3 - W10 (30-70)   | 0.23 | 0.05848 |
| S8A3-S8B3 - W10 (70-100)  | 0.21 | 0.06326 |
| S8A3-S8B3 - W10 (100-140) | 0.05 | 0.06769 |
| S8A3-S8B3 - W10 (140-165) | 0.14 | 0.06771 |
| S8A3-S8B3 - W10 (165-195) | 0.04 | 0.04926 |
| S8A3-S8B3 - W11 (0-30)    | 0.33 | 0.06254 |
| S8A3-S8B3 - W11 (30-70)   | 0.20 | 0.07470 |
| S8A3-S8B3 - W11 (70-110)  | 0.17 | 0.07026 |
| S8A3-S8B3 - W11 (110-135) | 0.30 | 0.07118 |
| S8A3-S8B3 - W11 (135-165) | 0.25 | 0.07115 |
| S8A3-S8B3 - W11 (165-195) | 0.23 | 0.06254 |

|                                  |      |         |
|----------------------------------|------|---------|
| <b>S8A3-S8B3 - W12 (0-30)</b>    | 0.02 | 0.09296 |
| <b>S8A3-S8B3 - W12 (30-70)</b>   | 0.15 | 0.06535 |
| <b>S8A3-S8B3 - W12 (70-100)</b>  | 0.09 | 0.06660 |
| <b>S8A3-S8B3 - W12 (100-130)</b> | 0.17 | 0.07070 |
| <b>S8A3-S8B3 - W12 (130-165)</b> | 0.10 | 0.06587 |
| <b>S8A3-S8B3 - W12 (165-195)</b> | 1.63 | 0.06060 |