

Comparison of Ground Penetrating Radar Deck Surveys to
Conventional Level 2 Deck Testing Results

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Abstract

A 1500 MHz Ground Penetrating Radar (GPR) survey was conducted on the Beaver River Bridge, located near Dapp AB. The bridge deck had previously been identified as being partially testable with regards to Alberta Transportation Level 2 Copper Sulphate Electrode (CSE) half-cell potential testing due to problems with the western pre-cast prestressed RD box girder span. The deck also included an overhead through-truss (TH) span with a concrete deck that was constructed using galvanized reinforcing steel. After adjustment to account for higher chloride concentrations required for corrosion initiation of galvanized steel versus uncoated steel, areas of elevated GPR signal attenuation were thresholded correlated closely with areas of CSE values more negative than -400 mV in the TH span. Areas of elevated GPR signal attenuation levels, assumed to be caused by increased chloride content in the concrete cover layer, correlated closely with CSE values more negative than -400 mV on the TH deck span, and with areas where cracking and corrosion staining were observed in all spans. The GPR results were effective in describing areas of longitudinal cracking in the RD spans along the grout keys which appeared to be impacted by high levels of chloride ingress. The GPR also indicated low but elevated levels of chloride ingress along multiple transverse cracks observed in all of the TH span panels. Level 2 CSE testing was not able to identify either of these issues. Both the CSE and GPR results indicated that the majority of the deck appeared to have a low risk of corrosion in most areas that were not cracked. After adjustment to account for higher chloride concentrations required for corrosion initiation of galvanized steel versus uncoated steel, the GPR results indicated that conditions for corrosion were unlikely at two locations where reinforcement had been previously exposed for inspection due to CSE values that were indicative of corrosion activity. GPR survey results can provide transportation agencies with a rapid and accurate means to quickly gather important parameters related to monitoring the condition and service life prediction of their reinforced concrete deck infrastructure or for quality assurance purposes for new construction or rehabilitation.

Introduction

Alberta Transportation conducts inspection and testing of their bridge structures over frequent intervals as part of their management strategy to maximize the return on their infrastructure. Monitoring the bridge condition at regular intervals allows the Province to properly plan maintenance, rehabilitation and replacement activities that address specific problems and their root causes. Some routine maintenance actions are regularly scheduled, while other maintenance and repair actions are identified through the Department's Bridge Inspection and Maintenance (BIM) system. Rehabilitation and replacement strategies are typically determined through a bridge assessment that involves varying levels of analysis and engineering. All bridge management decisions require inventory and inspection data on the structure to identify needs and appropriate actions.

Most major bridges, standard bridges, and culverts can be adequately inspected by a certified bridge inspector on a routine basis (Level 1). However, certain major bridges or components of standard bridges and culverts require inspection with specialized knowledge, tools and equipment. Almost all bridges will require at least two specialized inspections (Level 2) during their life-span. Level 2 deck inspection and testing typically visual inspection and hammer sounding and/or chain drag and Copper Sulphate Electrode (CSE) half-cell potential testing of the wearing surface and curbs along with cover measurements using a pachometer. Rapid chloride testing of the deck concrete may also be conducted depending on the presence and condition of any waterproofing membranes applied to the deck top.

Alberta Transportation has found CSE testing to be effective for monitoring changes in the condition of the majority of their bridge infrastructure, but have also found that the results may be questionable for certain bridge types or conditions. Based on recent improvements reported in the methodology and accuracy achieved for bridge deck condition evaluation, Ground Penetrating Radar (GPR) was used in 2017 to survey the condition of the Pembina River Bridge, located near Dapp, AB.

A photo of the Pembina River Bridge is provided in Figure 1. The existing substructure remains from the original construction which occurred in 1935. The superstructure consists of three spans that were reconstructed/replaced in 1977. The main span is a 76.2 m long steel overhead through (TH) truss, while the approach spans are pre-stressed concrete Type RD box girders of 16.8 m and 18.3 m lengths. Each RD girder was approximately 1.22 m in width and is fabricated with a grout pocket on each interior web which is used to tie adjacent interior girder to one another, providing structural and electrical continuity through the deck. The cast-in-place concrete deck on the TH span consists of 12 panels, each 6.35 m in length, with each panel point located at a node in the bottom chord of the truss. The deck was constructed without a skew, and has a clear width of 7.3 m bounded by reinforced concrete curbs. The upper flanges of the RD girders comprise the reinforced concrete deck in the approach spans. The RD approach spans have a protective High Density concrete OverLay (HDOL) and the entire deck is chip seal coated. The TH span was constructed without a protection system. Review of historical data indicated that the deck has never been rehabilitated. The uppermost reinforcement in the RD approach span girders are 13 mm diameter steel reinforcing bars with a specified concrete cover of 45 mm, plus 50 mm of HDOL. The uppermost reinforcement in the TH deck consists of 20M galvanized steel reinforcing bars with a specified concrete cover of 57 mm.

The objective of the study was to assess the capability of GPR for simultaneously assessing the concrete cover thickness and for mapping and estimating areas of potential corrosion in the deck and curbs. The GPR results were compared to prior recent Level deck testing and inspection results, which included CSE and chain drag testing along with cover measurements, for

consideration as an additional useful tool which could sometimes supplement Level 2 deck testing.

Level 2 CSE Testing

CSE tests measure the electrical potential of the steel reinforcement within the bridge deck. The half-cell reference electrode acts as a reference point from which electrical potential measurements are made. The more negative the potential, the higher the probability that the steel is experiencing corrosion. By combining CSE testing experience with field experience in repairing bridge decks, Alberta Transportation has developed a data interpretation scale:

1. 0.000 V to -0.300 V = Inactive (very low probability of active corrosion)
2. -0.301 V to -0.400 V = Transition (good probability that corrosion is initiating)
3. -0.401 V to -0.800 V = Active (very high probability of active corrosion)

ASTM C876 also describes CSE testing and discusses the probability of corrosion relative to the CSE readings and defines inactive, uncertain, and active ranges in CSE test results. The ranges suggested in ASTM C876 are slightly different than those used by Alberta Transportation.

Since 1977, Alberta Transportation has been using CSE testing to help evaluate the condition of concrete bridge decks. Tests are usually conducted on a 4 or 5 year rotation and are compared to assess the advancement of corrosion and to predict the future deck condition. Prediction models based on the CSE data are used to help determine the ideal time to rehabilitate a deck, to use preventative maintenance (prior to any visible damage), or to help evaluate the condition of a deck that is not visible, such as a deck that is covered with asphalt concrete pavement or another protection system.

In order for a deck to be testable using CSE, an electrically interconnected mat of reinforcing steel in the concrete and a means to connect to this mat are required. The electrical continuity of a deck is first checked by measuring the resistance using convenient ground locations such as steel deck joints, bridge rail anchor bolts, deck drains, or exposed reinforcing steel. Testing directly to the deck reinforcing steel is considered a last resort, and is used when other locations have failed. These ground locations need to be thoroughly cleaned to expose bright steel and should be located at opposite corners of a given deck span. A deck is considered to be testable if the resistance measured between ground locations across the deck span is below 3 to 4 Ohms. Without establishing a proper ground connection, testing will not produce accurate results, if any. This method is based on the assumption that the external ground locations are electrically interconnected with the deck reinforcement.

Decks with asphalt, chip seal, and polymer (epoxy) wearing surfaces are deemed to be testable after the wearing surface has been sufficiently pre-wetted. Polymer overlays have been designed to be breathable and are therefore sufficiently permeable to be tested with CSE. However, certain bridge or wearing surface types do not intuitively appear to meet the requirement for electrical interconnectivity between external ground points or even within the deck itself:

4. Decks with impermeable waterproofing membranes or epoxy coated steel will not be testable due to the barrier between the electrode and the reinforcing steel. These decks have been shown to become testable over time as these barriers deteriorate, and a change in the measured CSE potentials provides an indication of this transition.
5. Many bridges where the deck consists of the top flanges of pre-cast girders are not testable since the reinforcement is not continuous between the multiple girders. In some cases, contact between reinforcement hooks that extend into grouted key connections between adjacent girders can achieve continuity. Precast girders that are electrically

connected through deck joints that are connected to the reinforcing steel in the girders can also be testable.

6. Bridges with galvanized reinforcing steel are testable but must be interpreted differently due to different electrical potentials resulting from the presence of the zinc.

GPR Testing

GPR has become a more commonly accepted approach for bridge deck condition assessment by many transportation agencies across North America. GPR transmits a brief pulse of very low power microwave energy into the deck and records a waveform consisting of a series of reflections of this pulse over time that occur from different material interfaces at different depths. A series of waveforms is recorded over some distance to produce a profile image of the interior structure of the deck. Ground-coupled GPR surveys using antenna centre frequencies between 1500 to 2600 MHz provide highly detailed data that image individual reinforcing bars versus scan distance. Such surveys can be completed relatively quickly within closed lanes on bridge decks at walking speed.

Typically, GPR data are recorded at a spatial resolution of 150 to 200 scans per metre and at a spacing of approximately 0.5 metres between adjacent scans. The data must be recorded in a direction approximately perpendicular to the orientation of the shallowest reinforcing bars to ensure that a peak reflection amplitude is obtained as the antenna crosses over the centreline of the bar. This approach provides consistency in the quality of the data and in the amplitude of the reinforcing bar reflections at a given depth, provided that conditions at the surface and within the concrete remain uniform.

The first reflection recorded in the data results from the direct transmission from the transmitter to the receiver within the GPR antenna, which is called the direct coupling reflection. This reflection is typically used as a reference from which the two-way travel time from the antenna to a layer interface and back is measured and used to calculate its depth, depending on the velocity of the GPR signal through the upper layer. As shown in Equation 1, this velocity is largely affected by the relative dielectric constant of the layer, ϵ_r , which in turn depends on the layer material and moisture content.

$$v = 300/\sqrt{\epsilon_r} \text{ (mm/ns)} \quad (1)$$

The magnitude and phase of the layer reflection depends on the level of contrast between the dielectric constants of the layers meeting at the interface, the distance from the antenna, and the rate of signal loss through the upper layer. The reflection coefficient, R , as provided in Equation 2, describes proportion of the transmitted wave that is reflected from a layer interface (Davis and Annan, 1989).

$$R = (\sqrt{\epsilon_{r1}} - \sqrt{\epsilon_{r2}})/(\sqrt{\epsilon_{r1}} + \sqrt{\epsilon_{r2}}) \quad (2)$$

Variations in the amplitude of reflections in the GPR data provide an indication of changes in the dielectric properties of the layers at the interface, or in the amount of energy lost as the GPR pulse travels between the layer interface and the antenna. The rate of energy loss, or attenuation, of the GPR signal as it travels through the concrete is mostly influenced its conductivity, and hence the chloride content of the concrete after it has matured beyond early ages. Accordingly, changes in the attenuation levels measured within the GPR data across the deck provide a valuable measure of the variations in the chloride content within the concrete cover layer over the deck surface. These areas of elevated chloride content as identified from the GPR data are correlated

to more negative CSE potentials that result once corrosion has initiated, and can be used to identify locations and estimate quantities of area where there is a high risk of corrosion induced damage (Romero et al., 2015)(Barnes et al., 2007).

Pembina River Bridge Level 2 Test Results

Level 2 inspection and testing completed on the Pembina River Bridge in 2015 indicated that 27.5% of the deck exhibited CSE values in the Transition level between -300 mV and -400 mV, while 20.1% of the deck exhibited Active corrosion levels more negative than -400 mV. It was noted that while adequate resistance was measured between the deck joint at the east pier and various deck drains down the bridge along both curbs, but a suitable connection could not be found at the northeast corner of the deck. As a result the deck was considered to be partially CSE testable. CSE testing was completed at 1.2 m grid spacing in both the longitudinal and transverse directions along the bridge, yielding 644 and 184 potentials measured from the deck and curbs, respectively.

Review of the current and prior CSE data suggested that the deck's corrosion condition improved after the 2011 chip coat was applied, possibly due to the many transverse cracks being sealed. Recent CSE increases appeared to be mostly on the main TH span, where the deck had transverse cracks at each of the 11 panel points over the floor beams.

The map of the 2015 CSE and chain drag results shown in Figure 2 followed the typical expected trends with high readings along the curbs, the deck joints, and the transverse crack locations, especially at the 11 intermediate TH span panel points. Figure 2 also indicates the locations of the 32 m² (3.9%) of lost chip coat (light blue) and the 5 m² (0.9%) of delaminated concrete (fuchsia) on the main span. All of the delaminated areas appeared to coincide with high CSE readings. It appeared that the floor beam cracks heavily influenced the CSE readings on the main span. The CSE contours on the west RD span were very high, exhibited little topography, and did not agree with the contours on the east RD span. It was stated in the Level 2 testing report that the west span CSE readings were believed to be incorrect.

Rapid chloride test results previously taken in 2004 and 2010 in areas of un-cracked concrete indicated low levels of chloride content at a depth of 50 mm, varying between 0.001% to 0.008% by mass of concrete, which is well below values of 0.040% to 0.050% that are often assumed to represent critical corrosion initiation thresholds for uncoated carbon steel reinforcement.

The average deck cover was measured by pachometer to be 105 mm in the RD girder approach spans with a standard deviation of 10 mm, and 70 mm for the TH overhead truss span with a standard deviation of 8 mm. The average cover measured by pachometer on the curbs was 56 mm in the RD spans with a standard deviation of 9 mm, and 82 mm in the TH span with a standard deviation of 14 mm. The Level 2 protocol for measuring cover specifies 2 curb and 5 deck measurements at three different locations per span, which resulted in 45 and 18 cover thicknesses measured on the deck and curbs, respectively.

The 2015 Level 2 testing crew had used a jackhammer to expose rebar at two locations on the centre span exhibiting highly negative CSE readings in order to check for corrosion. However, the rebar was found to be in good condition at both locations.

Pembina River Bridge GPR Survey Deck Condition Results

The Pembina River Bridge deck was surveyed using a 1500 MHz ground coupled GPR at a resolution of 150 scans per metre. A series of 14 survey lines, spaced at 0.3 m from curb and subsequent 0.6 m transverse intervals in each direction of traffic flow were marked on the deck for the GPR survey. One line of GPR data was recorded along each curb positioned between the

vertical curb face and the bridge rail posts. Figure 3 shows an example of the rebar reflections recorded in the RD span and TH spans adjacent to Pier 1. A difference in intensity of the reinforcing bar reflections between the two deck spans is due to the different size reinforcing bars used in each. The deck condition and cover survey was based on 7,292 and 1,022 rebar reflections measured from the deck and curbs, respectively. This represents over 11 times more data on the deck and over 5 times more on the curbs compared to the CSE testing, and over 162 times more on the deck and over 56 times more data on the curbs compared to pachometer based cover measurements. GPR surveys provide complete contiguous surveys of the bridge deck condition and as-built cover distribution that are statistically robust compared to conventional techniques, yet require less time and effort in the field to complete.

Figure 5 a) shows a plan view contour map of the GPR signal attenuation over the entire deck surface of the bridge using a consistent corrosion prediction threshold that was developed for reinforced concrete decks incorporating uncoated carbon steel bars. The TH span exhibits significantly higher levels of attenuation at the nodal points of the truss between the deck panels and also within the westernmost panel, which is consistent with the more negative CSE potentials measured in 2015. Many transverse lines of elevated attenuation levels, which are consistent with chloride levels approaching corrosion initiation, can be observed within the TH panels. However, no symptoms of corrosion were evident in these areas, nor did the CSE potentials indicate elevated risk of corrosion in these locations. Transverse cracks were observed during the GPR survey within the TH deck panels at locations where the chip coat had been lost, as shown in Figure 4.

Galvanized reinforcing steel was used in constructing the TH span of the bridge deck. In a Kansas State University study comparing the corrosion behaviour of various reinforcement types, Darwin et al. (2007) reported that the average critical concentration to initiate corrosion on galvanized steel bars was 1.57 times that required for conventional carbon steel. Since GPR signal attenuation is influenced by the chloride content of the materials between the antenna and the reinforcement being measured, it follows that the typical GPR signal loss threshold which has been used to indicate active CSE values of approximately -400 mV could be adjusted to by a similar factor to estimate the probable existence of steel corrosion in galvanized reinforcement.

Figure 5 b) provides a plan view map of the variation in GPR signal attenuation over the deck surface, but utilizes these different threshold values for uncoated and galvanized carbon steel bars for the RD and TH spans, respectively. It is noted that the two locations where the reinforcement was exposed and inspected in 2015 do not exhibit signal loss levels in Figure 5 b) that would be associated with reinforcement corrosion when the threshold loss levels were adjusted to account for the presence of galvanized bars. It is unknown if the locations selected for inspecting the reinforcing steel were evaluated with consideration of any differences in the threshold values describing Inactive, Transition, or Active ranges that would occur due to presence of the galvanization on the bars. The majority of the deck surface in the TH span shown in Figure 5 b) exhibits GPR signal losses that would correspond to inactive CSE values, which is in agreement with the lack of corrosion identified via the two locations where the reinforcement was exposed and with the general conclusions made in the 2015 Level 2 deck testing report.

The areas of elevated GPR signal losses in Figure 5 b) generally appear to correlate well with areas of highly negative CSE values associated with active corrosion that were determined via the 2015 CSE testing results. High levels of chloride ingress are evident in transverse bands across the deck that are located at the TH truss nodal points in Figure 5 b), and the GPR losses levels suggested that sufficient chloride ingress to result in corrosion existed at many of these locations, even for galvanized bars. Corrosion staining, shown in Figure 6, was observed at a cracked panel point location approximately 23 m along the deck length in the westbound lane which exhibited these high levels of GPR signal attenuation. Relatively lower levels of GPR signal

attenuation and more positive CSE values are observed at cracked nodal points in the TH span at distances of 42 m and 66 m along the deck length, compared to the remaining locations. Additionally, high levels of GPR signal loss (chloride ingress) are observed near the middle of the westernmost TH deck panel in the eastbound lane, where both CSE values more negative than -500 mV and concrete deck delaminations were reported in the 2015 Level 2 testing report.

Also of note in Figure 5 a) and b) are several of higher GPR signal losses that occur longitudinally within the RD spans. Four areas can be observed in the eastern span and three within the western span. Initially, it was thought that perhaps the GPR data profile might have been positioned over bent bars leading from the upper flange to the webs of the girders, which would cause the GPR signal to reflect off the bars at an angle and reduce the amplitude of the measured response. However, signal attenuation losses could be observed throughout the full time range of the waveforms in these areas, indicating that the reduced amplitude was likely due to higher chloride levels. Two longitudinal cracks observed in the HDOL on the eastern span is shown in Figure 7, which correspond to the longitudinal areas of GPR signal loss at transverse distances of 4.0 m and 5.2 m across the deck width in Figure 5 a) and b). Subsequent review of the bridge inspection data forms for the bridge revealed that in 2011 inspectors had noted four longitudinal cracks in the eastern span and two in the western span, likely originating from failure of the grout keys connecting adjacent RD girders. Chloride ingress through all six of these cracks can be observed in the GPR results shown in Figure 5 a) and b), and based on the GPR results it appears that one additional crack exists in the western RD span. It is noted that the GPR profiles were not completely coincident with these cracks, but were offset by approximately 15 cm as indicated by the chalk marks observed in Figure 7. This provides some indication of the lateral extent that chloride ingress from these cracks into the deck has progressed. It is also noted that the CSE testing was not effective in identifying these areas of suspected chloride ingress, possibly due to a lack of electrical continuity in the RD spans.

The GPR results indicated that 34.2 m² (11.5%) of the deck surface on the RD spans exhibited signal attenuation levels that are consistent with chloride levels in areas of active corrosion and corrosion damage of uncoated carbon steel, while 47.1 m² (15.8%) of the RD deck spans exhibited signal losses corresponding to chloride levels approaching the corrosion initiation threshold. 216.2 m² (71.7%) of the RD deck spans exhibited low levels of GPR signal attenuation consistent with low levels of chloride content and corrosion risk.

The GPR results also indicated that 17.8 m² (2.8%) of the TH deck spans exhibited signal attenuation levels consistent with chloride levels found in areas of active corrosion and corrosion damage for galvanized steel rebar, while 17.6 m² (2.7%) of the RD deck spans exhibited signal losses corresponding to chloride levels approaching the corrosion initiation threshold for galvanized steel. 611.5 m² (94.5%) of the RD deck spans exhibited low levels of GPR signal attenuation consistent with low levels of chloride content and corrosion risk.

In general the curbs exhibited GPR signal loss levels that were consistent with low levels of chloride ingress, except for several locations along the north curb of the eastern RD span, and along both curbs of the TH span. Review of the 2015 Level 2 deck testing report indicated the curbs were sealed in 2003, 2008, and 2015 using an acrylic sealer. The relatively low levels of chloride ingress indicated by the GPR results demonstrates the effectiveness of the curb sealing program.

Pembina River Bridge GPR Survey Cover Thickness Results

The cover measurements obtained from the GPR survey were calibrated using several pachometer measurements over marked rebar locations. The average GPR velocity was

determined based on these measured cover depths and the two-way travel time delay measured between the direct coupling reflection and the rebar reflection in the GPR data.

Figure 8 provides a plan view contour map of the estimated as-built cover thickness over the entire bridge deck surface. The difference in cover thickness between the RD and TH spans is highly evident in Figure 8. In general, the cover appeared to be slightly thinner in the eastbound lane, compared to the westbound lane. Thicker cover values ranging from 90-110 mm were estimated at the 11 nodal points in the TH span compared to the adjacent cover in the deck panels which generally ranged from 70-90 mm. This apparent increase in thickness may have resulted from higher moisture content due to the cracking at these locations which would increase the dielectric value and decrease the actual GPR wave speed, thus increasing the two-way travel time of the reinforcing bar reflections. Areas of shallow cover are noted adjacent to the deck joints located over the piers. The deck cover also appeared to be shallower in the western lane compared to the eastern lane.

It is noted that some cover measurements appear to be excessive in the RD spans within certain strips of GPR data near the deck centreline and curbs and laterally across the deck width adjacent to the abutment and pier joints. It is likely that the GPR data files exhibiting thick cover near the centre of the deck width were recorded over the section of the girder where the top layer has been bent downwards at an angle to extend down below the bottom layer into the underlying girder web. The thickening around the perimeter of the RD spans may be due to poor control of the HDOL layer thickness during its placement. Similarly, a relatively large region of shallow cover within the eastern RD span can also be observed.

The mean total cover in the RD deck spans was estimated to be 115.1 mm, with a standard deviation of 23.4 mm, and range from a minimum of 65.6 mm to a maximum of 203.3 mm. The mean total cover in the TH deck span was estimated to be 82.7 mm, with a standard deviation of 10.3 mm, and ranged from a minimum of 11.7 mm to a maximum of 165.3 mm.

The mean clear cover on the curbs of the RD spans was estimated to be 50.8 mm with a standard deviation of 11.7 mm, and ranged from a minimum of 22.7 mm to 83.8 mm. The mean clear cover on the curbs of the TH span was estimated to be 77.2 mm with a standard deviation of 23.4 mm, and ranged from a minimum of 26.9 mm to 165.3 mm.

The variability in the as-built cover thickness appears to be relatively high within the RD spans and relatively low in the TH span according to Figure 8. These differences are also observed via the width of the as-built cover thickness histograms for the RD and TH spans, as provided in Figure 9 and Figure 10, respectively. Figure 11 and Figure 12 also provide histograms of the curb cover for the RD and TH spans, respectively. Noting the relatively low minimum cover thickness values and a high degree of variability in the as-built curb cover, a correlation between shallow cover and areas of elevated GPR signal loss or chloride along the curbs does not appear to exist when comparing Figure 5 b) and Figure 8. This lack of correlation also appears to demonstrate the effectiveness of the curb sealing program in protecting the curbs from corrosion, particularly where shallow cover exists. The areas of elevated corrosion risk observed along the curbs in Figure 5 b) likely coincide with transverse cracks in the curbs.

While these results are reasonably consistent with those provided using the pachometer, the differences between the mean values is expected since the GPR provided cover data from all of the upper transverse reinforcement intersected by each data profile, while relatively few measurements comprised the data set obtained using the pachometer.

Conclusions

A 1500 MHz ground coupled GPR survey was conducted on a reinforced concrete deck which has exhibited questionable prior CSE results and also contained galvanized steel reinforcement in one span. The GPR survey provided 11 times more data on the deck and over 5 times more on the curbs compared to the CSE testing, and over 162 times more on the deck and over 56 times more data on the curbs compared to pachometer based cover measurements. The following conclusions could be made based on this study:

1. Areas of elevated GPR signal attenuation levels, being based on increased conductivity assumed to be caused by increased chloride content in the concrete cover layer, correlated closely with CSE values more negative than -400 mV on the TH deck span, and with areas where cracking and corrosion staining were observed in all spans.
2. The GPR signal attenuation results were effective in describing areas of longitudinal cracking in the RD spans along the grout keys which appear to be impacted by high levels of chloride ingress. The GPR also indicated low but elevated levels of chloride ingress along multiple transverse cracks observed in all of the TH span panels. Level 2 CSE testing was not able to identify either of these issues, which can be addressed through repair of the grout keys and re-application of a chip seal coat to the deck surface to extend the service life of the bridge. Both the CSE and GPR results indicated that the majority of the deck appeared to have a low risk of corrosion in most areas that were not cracked.
3. After adjustment to account for higher chloride concentrations that are required for corrosion initiation of galvanized steel versus uncoated steel, it was shown that conditions for corrosion were unlikely at two locations where reinforcement had been previously exposed for inspection due to CSE values that were indicative of corrosion activity.
4. With over 8,300 cover measurements obtained over the deck and curbs, the GPR survey provided a much more statistically robust measure and map of the as-built cover thickness than could be achieved by Level 2 testing which yielded 63 cover measurements using a pachometer. The wide range of cover identified and mapped using GPR may aid in better identifying structures which should be selected for frequent sealer applications as preventative maintenance.
5. Little correlation was observed between elevated GPR signal attenuation, indicating elevated chloride content and corrosion risk with as-built curb cover. This appeared to indicate the effectiveness of the 5-7 year interval used in the curb sealing program for this bridge.
6. GPR survey results provide Alberta Transportation agencies with a rapid and accurate means to quickly gather important parameters related to monitoring the condition and service life prediction of their reinforced concrete deck infrastructure or for quality assurance purposes for new construction or rehabilitation. Measurement and mapping of the GPR signal attenuation to indicate relative variations chloride levels throughout the deck and identifying areas with higher risk of corrosion and corrosion induced damage can assist interpretation of CSE test results obtained on decks that may be partially or non-testable and assist in identifying preventative or corrective maintenance requirements.

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Figure 1. BF 13166-1 Pembina River Bridge.

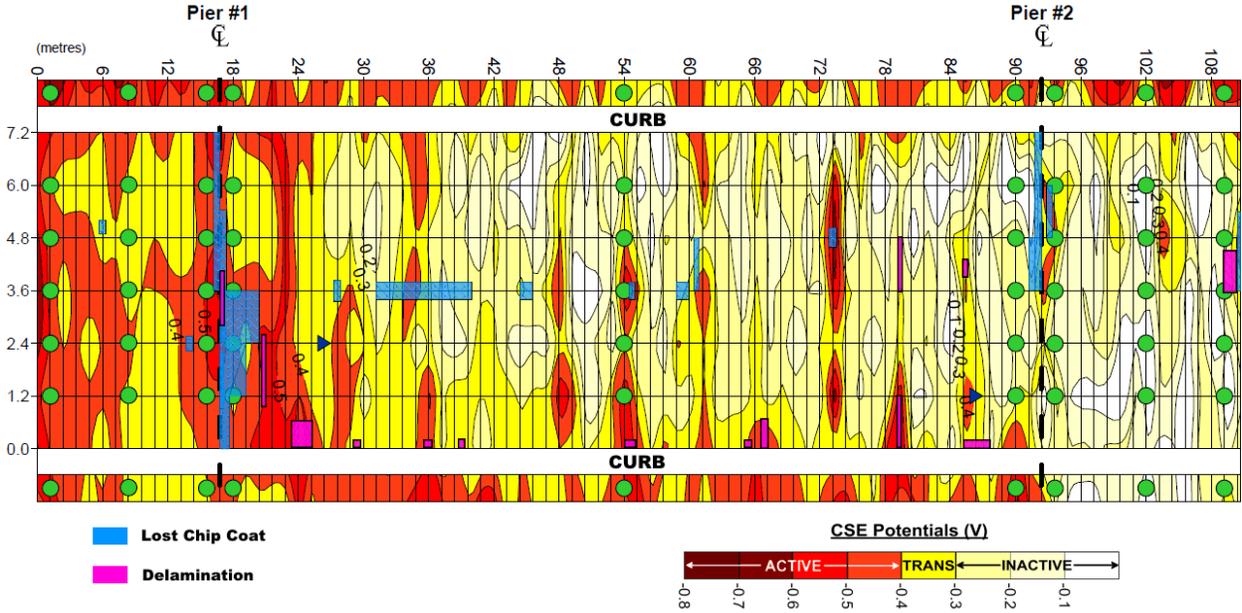


Figure 2. Plan view map of 2015 CSE testing and chain drag results for BF 13166-1 Pembina River Bridge.

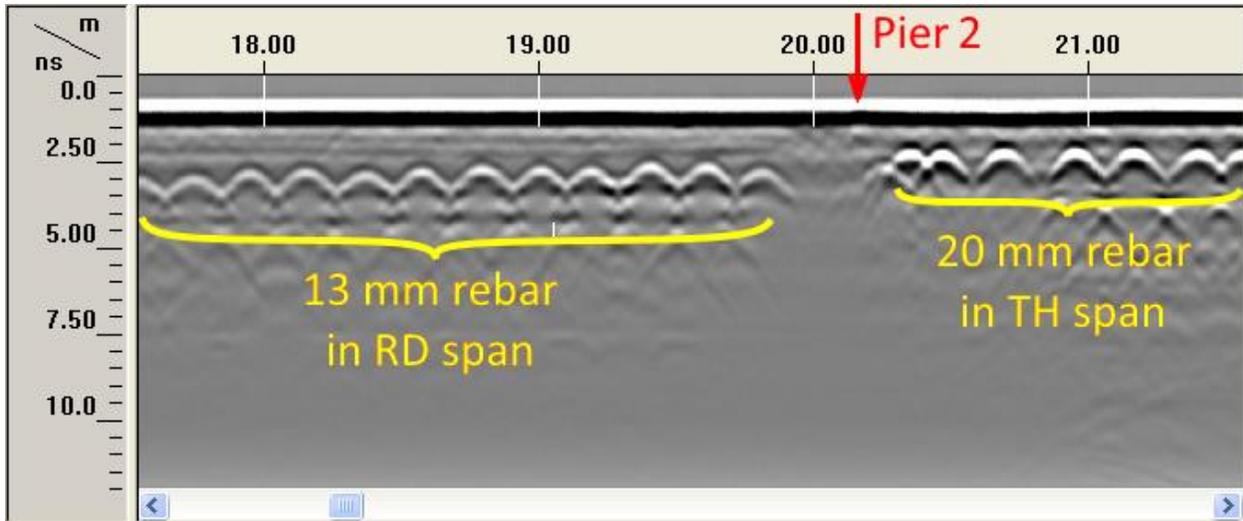


Figure 3. GPR data rebar reflections in RD and TH spans near Pier 1 in Pembina River Bridge.

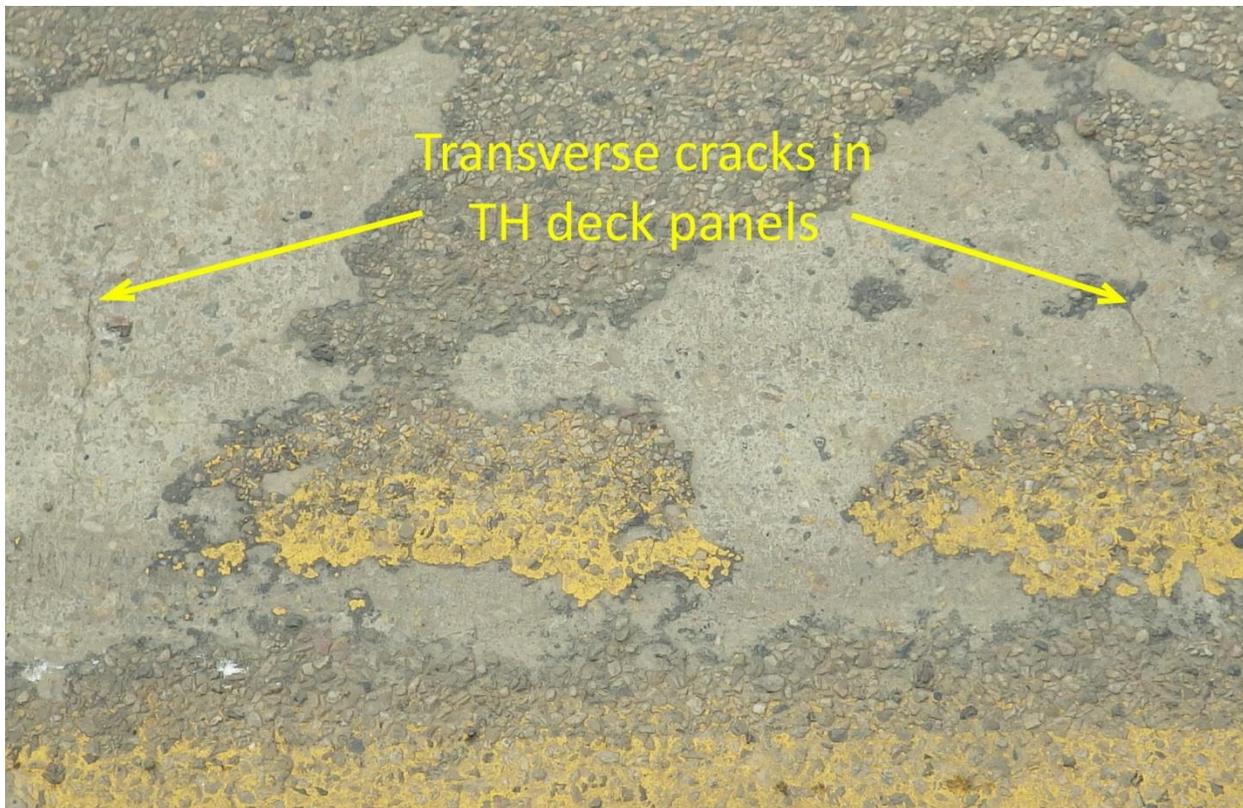
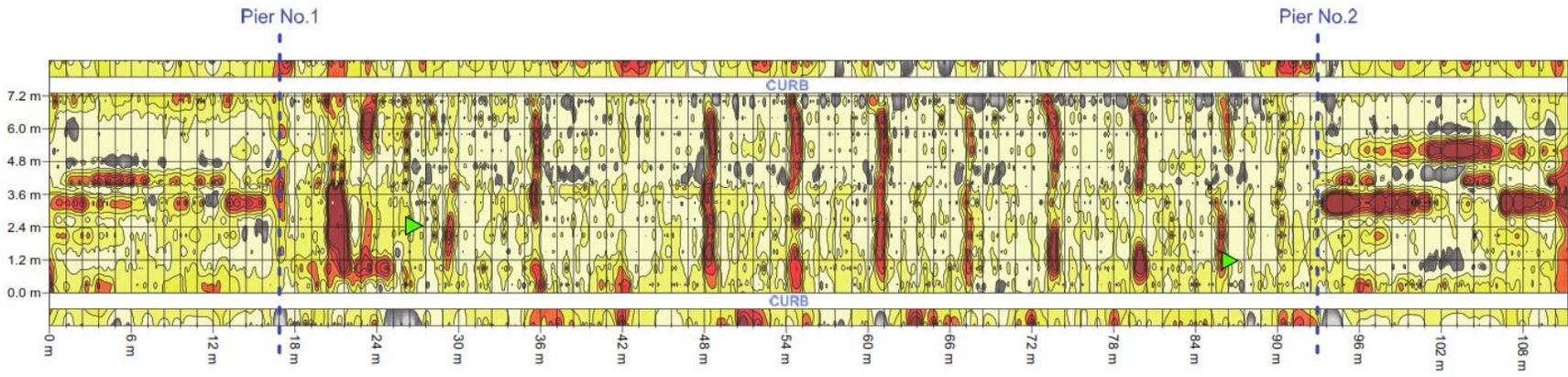
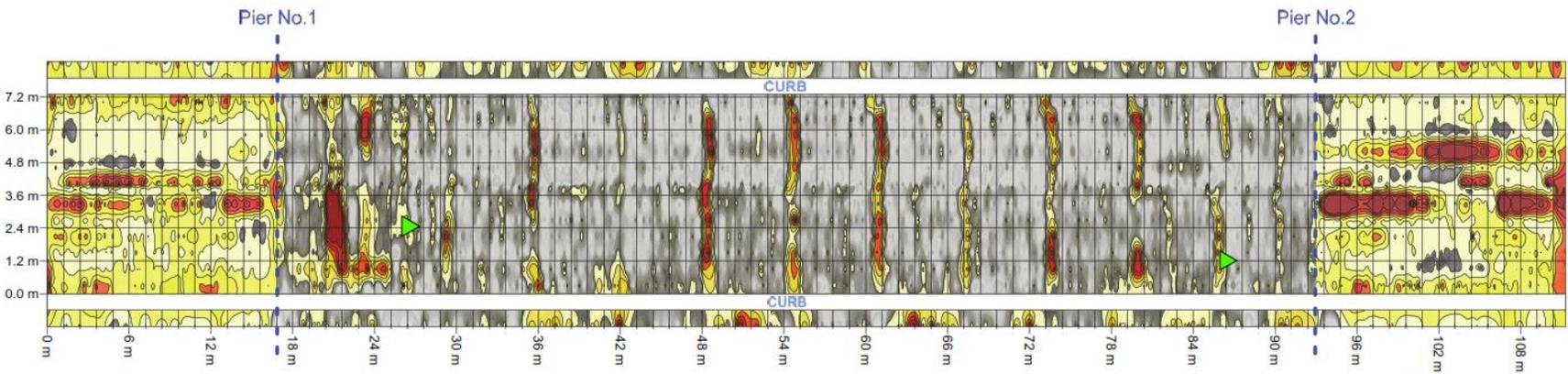


Figure 4. Transverse cracks observed in TH deck panels.



a) GPR results in all spans using corrosion threshold for uncoated carbon steel.



b) GPR results in TH span adjusted using corrosion threshold for galvanized reinforcing steel.



Figure 5. BF 13166-1 Pembina River Bridge GPR Deck Condition Results.



Figure 6. Corrosion staining in transverse deck crack at TH span nodal point located 23 m along the bridge length.



Figure 7. Two longitudinal cracks by Pier 2 joint in the eastern RD span.

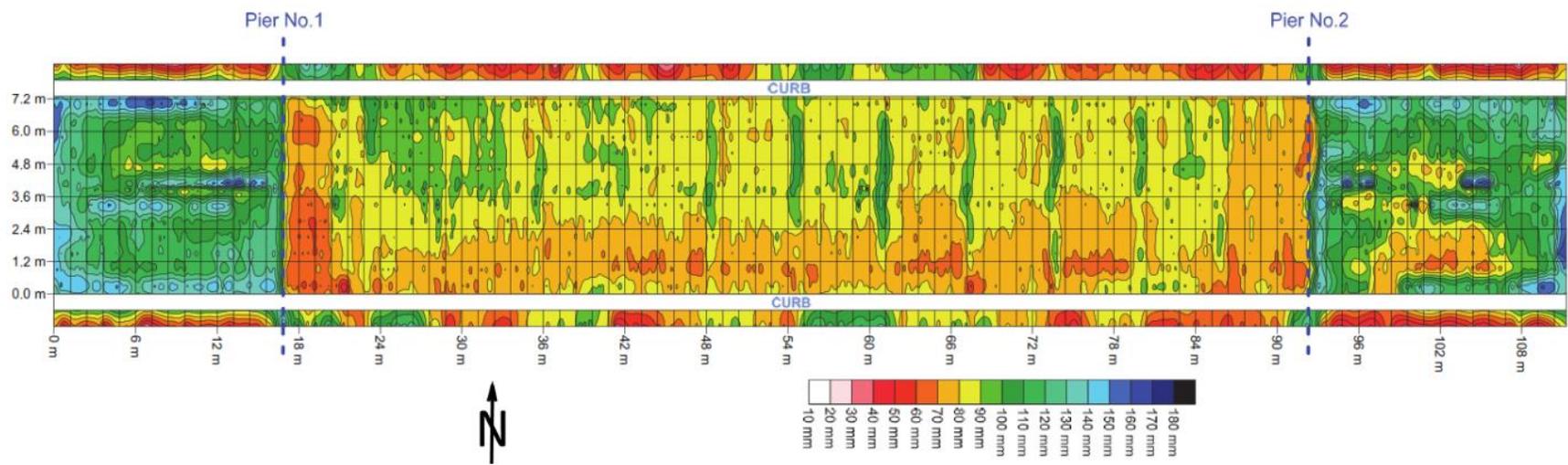


Figure 8. Contour map of estimated as-built cover thickness.

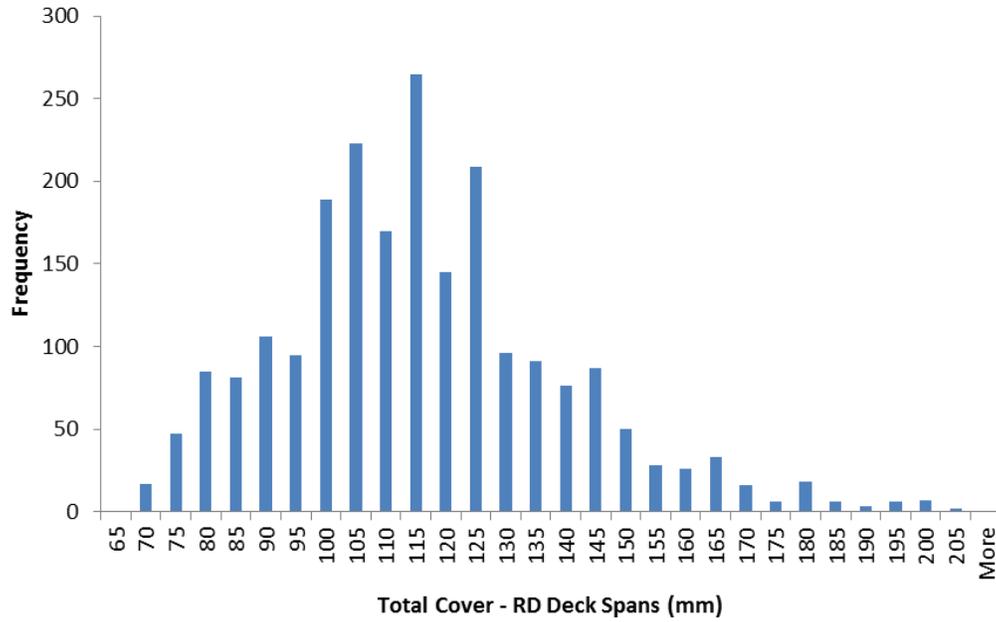


Figure 9. Histogram of as-built total cover thickness estimated for RD deck spans.

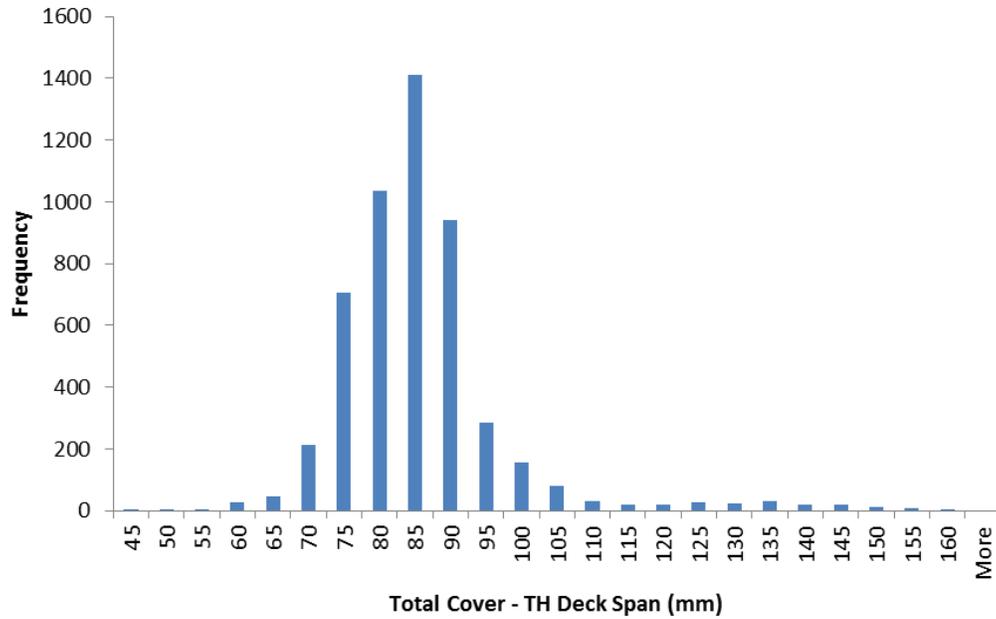


Figure 10. Histogram of as-built total cover thickness estimated for TH deck span.

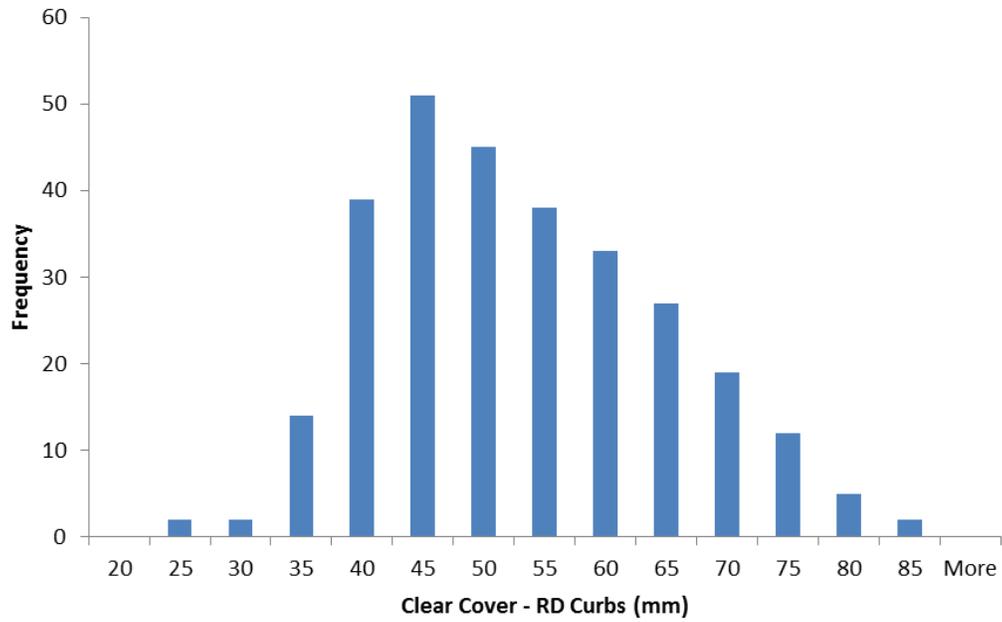


Figure 11. Histogram of as-built total cover thickness estimated for RD deck spans.

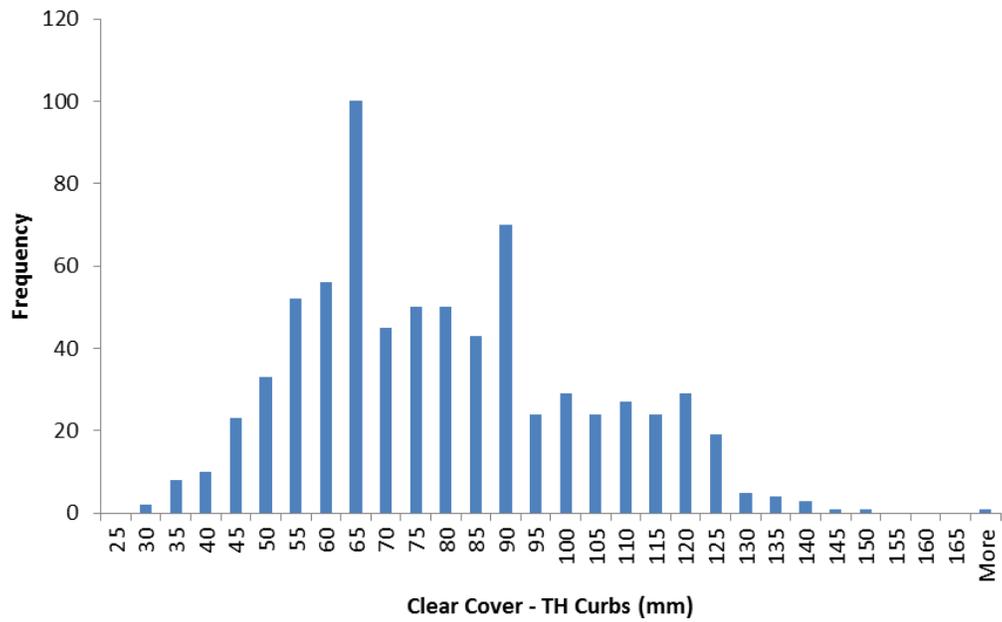


Figure 12. Histogram of as-built total cover thickness estimated for TH deck span.