



Condition and Durability Evaluation of Reinforced-Concrete Bridge Girders in the Surabaya Container Terminal Environment

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Abstract

An access bridge at PT Terminal Petikemas Surabaya has operated since 1992 in a chloride-bearing marine environment and carries repetitive heavy operational vehicles. The reinforced-concrete girders were repaired by epoxy injection and strengthened with carbon-fibre-reinforced polymer between 2010 and 2014; however, cracks were identified again in 2021. This study evaluates the existing condition and durability of the girders by integrating repair-history review, visual inspection and crack mapping, concrete core testing, rebound-hammer testing, reinforcement detection, half-cell potential measurements, and chloride-based service-life analysis. Seven 93-mm-diameter cores produced equivalent standard-cylinder compressive strengths of 51.33–59.31 MPa, with a mean of 54.74 MPa and a coefficient of variation of 5.20%, while the average rebound-hammer estimate was 62.02 MPa. A representative girder cover of 60 mm was adopted for durability modelling, although the raw scans contained local readings from 20.4 to 68.7 mm. Across ten girder half-cell survey areas, individual potentials ranged from –145 to +245 mV and area means from –88.0 to +206.2 mV; all readings were more positive than –200 mV, indicating a low conventional probability of active corrosion at the time of testing. The apparent chloride diffusion coefficient for the girders was $1.77 \times 10^{-8} \text{ cm}^2/\text{s}$, yielding a nominal corrosion-initiation time of 21.12 years under the adopted assumptions. Although the measured core strengths exceeded the conservative 50 MPa value adopted in structural checks, recurrent cracking remained evident. The integrated diagnosis therefore indicates that the observed cracking is more plausibly associated with continuing structural demand, previously active cracks, reduced cracked-section stiffness, or uncertain CFRP bond effectiveness than with generalised low concrete strength or widespread active corrosion. A risk-based maintenance framework is proposed, prioritising quantitative crack monitoring, verification of CFRP bond condition, repeated electrochemical assessment, and control of heavy vehicle operations.

Keywords: Bridge condition assessment; Chloride exposure; Concrete durability; Corrosion potential; Crack mapping; CFRP strengthening

1. Introduction

Reinforced-concrete bridges in port and coastal areas are exposed simultaneously to aggressive environmental actions and concentrated operational loads. Chloride ingress, high humidity, wetting–drying cycles and splash exposure can progressively depassivate reinforcing steel, while repeated axle loads may initiate and propagate flexural or shear cracks. Cracks then provide preferential pathways for water, oxygen and chloride ions, creating a coupled deterioration process between mechanical damage and durability loss [1]–[4].

Condition assessment of an existing bridge therefore requires more than a single material test. Visual inspection identifies the location and morphology of damage, core testing provides direct evidence of in-situ concrete strength, rebound-hammer testing supports spatial uniformity assessment, reinforcement detection verifies cover and bar arrangement, half-cell potential identifies areas with an increased probability of corrosion, and chloride analysis supports service-life evaluation [5]–[9]. Each method has limitations; consequently, a defensible diagnosis must integrate several sources of evidence rather than equating one test result with structural safety.

Carbon-fibre-reinforced polymer (CFRP) is widely used to strengthen damaged reinforced-concrete members because of its high tensile strength, low self-weight and corrosion resistance. Nevertheless, CFRP does not remove the cause of cracking. Its long-term effectiveness depends on substrate quality, bond integrity, anchorage, environmental exposure, existing strain at installation and the possibility of premature debonding [10]–[13]. Epoxy injection is similarly effective only for dormant cracks. Recurrent cracking after injection and strengthening is therefore an important warning that the underlying load or deterioration mechanism may remain active.



Figure 1. Access bridge at PT Terminal Petikemas Surabaya.

The access bridge investigated in this study was built in 1989 and has operated at PT Terminal Petikemas Surabaya since 1992. Between 2010 and 2014, cracks were repaired by epoxy injection and selected girders were strengthened with CFRP. Cracks were identified again in 2021 and were re-injected. The structure is located in a marine terminal and carries trucks, dollies and heavy container-handling equipment whose configurations have evolved since the original design. This history makes the bridge a relevant case for integrated condition and durability assessment.

Previous investigations of marine reinforced-concrete infrastructure have demonstrated that chloride concentration alone does not fully determine corrosion activity and that field exposure, concrete cover, moisture conditions, and crack characteristics should be considered simultaneously [14]–[18]. However, many published assessments emphasise individual deterioration indicators, such as concrete strength, chloride penetration, corrosion potential, or crack condition, without integrating them into a unified diagnosis for a previously repaired and CFRP-strengthened bridge.

Unlike assessments based on a single test method, this study integrates repair history, visual inspection, crack mapping, direct and indirect concrete-strength testing, reinforcement detection, half-cell potential measurements, and chloride-based service-life prediction. The objective is to evaluate the present condition and durability of the bridge girders, identify the most plausible causes of recurrent cracking, and establish a practical framework for maintenance prioritisation in a marine terminal environment.



Figure 2. Location of Access bridge at PT Terminal Petikemas Surabaya.

2. Method

2.1. Study object and assessment history

The assessment focused on reinforced-concrete T-girders of the access bridge. The bridge has been in service for more than three decades and has undergone repeated repair interventions. Historical information, drawings, inspection records and available test reports were reviewed to establish the chronology of construction, operational changes, cracking, epoxy injection and CFRP strengthening. This chronology was used to distinguish original defects, repaired cracks and recurrent cracks.

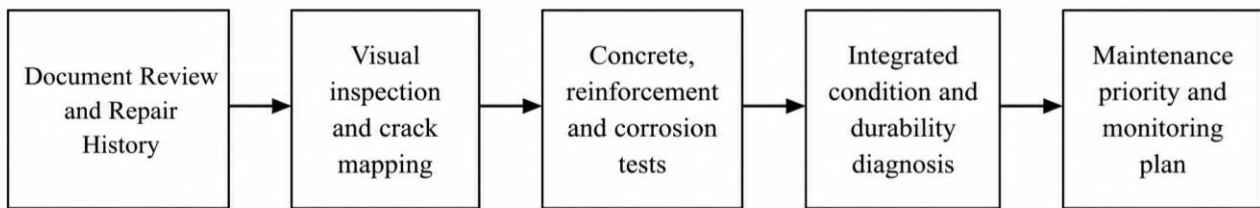
Table 1. Bridge history and its relevance to the assessment.

Period	Recorded event	Assessment implication
1989–1992	Construction and commencement of operation	Baseline geometry, materials and original loading assumptions
Before 2010	Change in terminal vehicle operations, including dolly use	Potential increase or redistribution of axle effects
2010–2014	Epoxy injection and CFRP strengthening	Prior repair and strengthening condition
2021	Cracks identified again and re-injected	Evidence of recurrent or active damage mechanism
Current study	Integrated field and laboratory assessment	Diagnosis and maintenance prioritisation

2.2. Integrated condition-assessment procedure

The assessment procedure is shown in Figure 3. The investigation combined document review, field observations, material testing, electrochemical assessment, and chloride-based modelling. Historical records were first reviewed to establish the chronology of construction, operational changes, cracking, epoxy injection, and CFRP strengthening. Visual inspection and crack mapping were then used to identify the location, morphology, and recurrence of damage. Concrete and reinforcement tests were used to characterise material condition and cover, while half-cell potential and chloride data were used to evaluate corrosion probability and nominal corrosion-initiation time.

All sources of evidence were interpreted jointly. Concrete strength was not used as the sole indicator of structural condition, half-cell potentials were interpreted as corrosion probability rather than corrosion rate, and chloride-based service-life calculations were treated as predictions conditional on the adopted transport model, critical threshold, and



monitoring dataset was recognised as a limitation. Accordingly, the present paper uses crack morphology and recurrence as diagnostic evidence rather than as a direct code-compliance check.

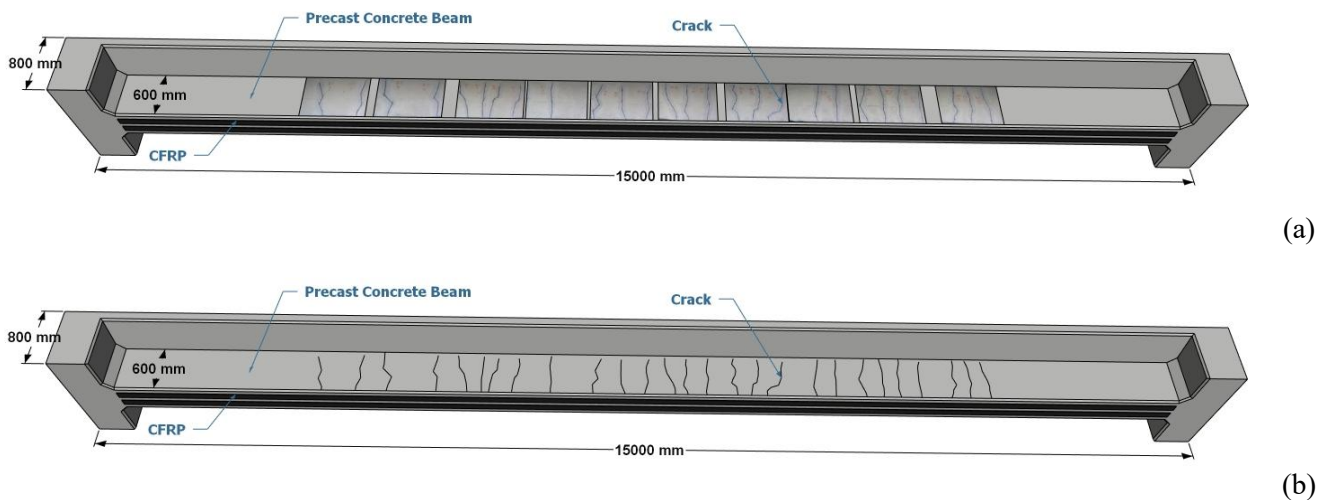


Figure 6. Crack mapping of Girder B at Span 53, east side: (a) compilation of field photographs plotted on the girder side elevation; (b) digitised crack pattern derived from the photographic documentation.

Figure 4 illustrates the documentation method used for Girder B at Span 53. Individual photograph segments were positioned according to their longitudinal locations along the 15 m girder, after which the visible crack traces were digitised to remove the photographic background and enable consistent comparison of crack orientation, spacing, and distribution. The map shows clusters of predominantly near-vertical cracks over the central region, with fewer cracks toward the girder ends. Because complete crack-width and temporal data were unavailable, the digitised map was used for morphological interpretation rather than quantitative crack-growth evaluation.

2.4. Concrete and reinforcement testing

Seven concrete cores were extracted from representative locations across several spans and girders. Each core had a nominal diameter of 93 mm and a height of 186 mm, giving a length-to-diameter ratio of 2.0. The samples were tested in compression on 23 November 2021. The reported core stresses were corrected for the presence of embedded transverse reinforcement where applicable and converted to equivalent 150×300 mm cylinder strengths using the conversion basis adopted in the assessment report. The resulting equivalent cylinder strengths were used in the present analysis. Rebound-hammer measurements were used to examine surface uniformity and provide supporting estimates rather than substitute for core strength. Reinforcement detection was performed at 30 locations across slabs, girders, and pier heads to verify bar arrangement, avoid bars during coring, and determine concrete cover. The girder subset included side and soffit scans at Spans 55, 60, 63, 75, 80, 90, and 100. The girder section was 1000 mm deep with a 450 mm web, a 1570 mm effective flange width, and a 400 mm flange thickness. The detected tensile reinforcement comprised 6D36, with D16 three-legged stirrups at 200 mm spacing. The raw girder cover readings ranged from 20.4 to 68.7 mm, while 60 mm was retained as the representative cover for durability modelling.

2.5. Half-cell potential and chloride-based durability assessment

Half-cell potential testing comprised 30 survey areas across slabs, girders, and pier heads. The girder subset comprised ten gridded survey areas at Spans 55, 57, 60, 63, 70, 75, 80, 85, 90, and 100, with multiple readings within each area. The results were interpreted in accordance with ASTM C876. Potentials more positive than -200 mV indicate a low probability of active corrosion under the standard's conventional interpretation, provided that electrical continuity, surface moisture, concrete resistivity, and the reference electrode are appropriately controlled [7]. The test does not measure corrosion rate or steel loss directly.

The chloride data were used to estimate an apparent diffusion coefficient and the time required for chloride concentration at reinforcement depth to reach the adopted critical level. The analysis used a one-dimensional Fickian diffusion formulation:

$$C(x, t) = C_s - (C_s - C_i) \operatorname{erf}\left(\frac{x}{2\sqrt{D_{app}t}}\right) \quad (1)$$

where $C(x, t)$ is the chloride concentration at depth x and time t , C_s is the surface chloride concentration, C_0 is the initial chloride concentration and D_{app} is the apparent diffusion coefficient. A critical chloride content of 0.4% by weight of cement was adopted following the service-life assessment basis used in the thesis. Because cracking and time-dependent diffusion can accelerate transport, the resulting initiation time is interpreted as a nominal prediction rather than an exact remaining-life value.

3. Results and Discussion

3.1. Existing crack condition

Cracking was distributed along multiple girders, with numerous approximately vertical cracks on the longitudinal faces. The digitised map of Girder B at Span 53 in Figure 4 shows that the cracks are concentrated mainly within the central portion of the span and are predominantly near-vertical, a morphology compatible with flexural action in positive-moment regions. Some cracks occurred at or near previously repaired areas, indicating that injection restored local continuity but did not eliminate the action generating tensile demand. The absence of widespread longitudinal splitting, rust staining, and spalling on the inspected girder surfaces is consistent with the low half-cell corrosion probability measured at the time of testing. Nevertheless, the visual absence of corrosion products cannot exclude localised pitting in a chloride environment.

The recurrence of cracks after epoxy injection has two practical implications. First, the cracks should be treated as potentially active until monitoring demonstrates otherwise. Second, re-injection alone is unlikely to provide a durable solution if axle loading, differential stiffness, CFRP debonding or support movement remains unaddressed. Recent studies similarly emphasise that crack activity and substrate condition strongly influence the effectiveness of repair and strengthening systems [11], [12], [19].

3.2. Concrete strength and uniformity

Table 2. Equivalent cylinder compressive strengths of the seven core samples.

Sample	Location	Equivalent cylinder strength, f'_c (Mpa)
CD-4	Span 55, Girder H flange, outer side	51.33
CD-5	Span 60, Girder B flange, west side	59.31
CD-6	Span 63, Girder C flange, west side	53.80
CD-7	Span 75, Girder B flange, west side	58.86
CD-8	Span 80, Girder B flange, west side	53.23
CD-9	Span 90, Girder B flange, west side	53.09
CD-10	Span 100, Girder C flange, west side	53.59

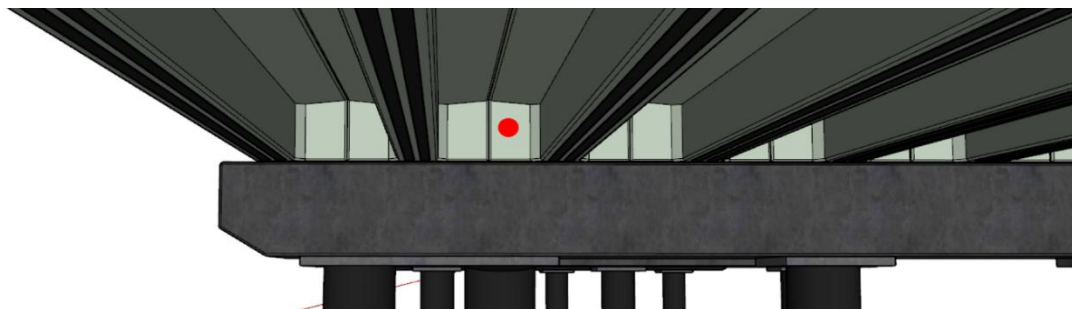


Figure 7. Core Drill Test Specimen Sampling Point

The equivalent standard-cylinder strengths ranged from 51.33 to 59.31 MPa, with a mean of 54.74 MPa, a population standard deviation of 2.85 MPa, and a coefficient of variation of 5.20%. The relatively low coefficient of variation indicates consistent concrete quality among the seven sampled locations. All core values exceeded the conservative $f'_c = 50$ MPa adopted in the structural checks, although the minimum value was only moderately above this basis. The rebound-hammer mean of 62.02 MPa was higher than the core mean and slightly above the core range. This difference should not be interpreted as a direct calibration because rebound response is affected by surface hardness, moisture, carbonation, aggregate characteristics, and testing direction [5], [6].

The diagnostic implication is that recurrent cracking cannot reasonably be attributed to generalised low compressive strength. However, compressive strength alone is not sufficient to represent structural performance. Crack development in reinforced-concrete girders is governed by tensile stress, stiffness degradation after cracking, load repetition, support behaviour, and the effectiveness of previous strengthening measures. Hence, the material results should be considered together with crack distribution, operational load history, structural demand, and the in-situ condition of the CFRP system. This distinction separates material adequacy from structural and durability risk.

3.3. Reinforcement cover and corrosion probability

Table 3. Reinforcement and electrochemical condition indicators.

Indicator	Measured/adopted value	Condition interpretation
Girder concrete cover	60 mm adopted; local readings 20.4–68.7 mm; survey-area means 42.88–64.50 mm	Moderate representative cover, but local thin-cover zones are present
Girder half-cell potential	10 survey areas; individual readings –145 to +245 mV; area means –88.0 to +206.2 mV	All readings > -200 mV; low conventional probability at the time of testing
Conventional corrosion probability	Approximately $\leq 10\%$	Area-level screening result, not a corrosion-rate measurement
Apparent chloride diffusion coefficient	1.77×10^{-8} cm ² /s	Input to the nominal initiation-time prediction
Predicted corrosion initiation	21.12 years	Conditional prediction under the adopted assumptions

Half-cell mapping covered ten girder survey areas. Individual grid readings ranged from –145 mV at Span 63 to +245 mV at Span 90, while the area-mean potentials ranged from –88.0 mV at Span 75 to +206.2 mV at Span 90. Neither the individual readings nor the area means crossed the –200 mV threshold. Under the conventional ASTM C876 interpretation, the surveyed girder areas therefore had a low probability of active corrosion at the time of testing. This result is consistent with the absence of extensive spalling, rust staining, or corrosion-induced longitudinal splitting. Nevertheless, half-cell potential identifies corrosion probability rather than corrosion rate or section loss, and localised pitting cannot be excluded by a single survey campaign. The bridge also remains exposed to airborne salt, high humidity, and possible splash or leakage; repeat surveys at fixed grid points are therefore required to identify meaningful temporal shifts [7], [14], [20].

Reinforcement detection supports, but also qualifies, the favourable half-cell interpretation. Although 60 mm was adopted as the representative girder cover, the survey-area means ranged from 42.88 to 64.50 mm and individual readings ranged from 20.4 to 68.7 mm. The lowest readings occurred mainly in selected side scans, demonstrating that a single average can conceal local thin-cover zones. Such zones have a shorter chloride transport path and may reach the critical concentration earlier, particularly where flexural cracks provide preferential moisture and chloride pathways. Future surveys should therefore report the minimum, mean, and coefficient of variation of cover and overlay the results with half-cell contours and crack maps. This spatial integration is more informative than independent averages because chloride-induced deterioration is strongly localised.

3.4. Chloride transport and service-life interpretation

Using the representative 60 mm girder cover, the apparent diffusion coefficient obtained for the girders was $1.77 \times 10^{-8} \text{ cm}^2/\text{s}$ and the corresponding nominal corrosion-initiation time was 21.12 years. The bridge age already exceeds this predicted initiation time, whereas all measured girder half-cell potentials remained more positive than -200 mV . This difference does not invalidate either result. The chloride model predicts when an adopted chemical threshold may be reached under idealised transport and boundary assumptions, whereas half-cell mapping screens the electrochemical probability of corrosion during a specific field campaign. Their combined interpretation indicates that predicted chloride arrival does not automatically imply widespread active corrosion. Local cover variability (20.4–68.7 mm), moisture, oxygen availability, binder chemistry, chloride binding, and the spatial distribution of cracks can delay or accelerate the transition from chloride presence to active corrosion [14]–[18], [21]. The assessment report also estimated a beam corrosion current density of $2.48 \mu\text{A}/\text{cm}^2$ using an empirical propagation model; because this value was calculated rather than directly measured, it is treated here as a scenario parameter and not as evidence that active corrosion was occurring throughout the girders.

The service-life result should therefore be used as a trigger for enhanced inspection rather than as a deterministic failure date. A sound management decision is to combine periodic half-cell mapping with electrical resistivity or linear-polarisation measurements, chloride profiles at several depths and crack-width monitoring. This approach can distinguish chloride presence from active electrochemical deterioration and can improve future calibration of the diffusion model.

3.5. Integrated condition diagnosis

Table 4. Integrated diagnosis and maintenance priority.

Assessment aspect	Evidence	Condition level	Recommended action
Concrete compressive strength	Core mean 54.74 MPa; COV 5.20%; all cores > 50 MPa assessment basis	Adequate relative to the conservative structural-check basis	Retain the measured-strength database and use it with structural-demand checks
Surface uniformity	Hammer mean 62.02 MPa	Supporting indicator	Use for comparative mapping and interpret with core results
Cracking	Multiple recurrent, predominantly vertical cracks	Moderate–high concern	Measure crack widths and install gauges at critical girders
Reinforcement cover	60 mm adopted; area means 42.88–64.50 mm; local minimum 20.4 mm	Moderate overall protection with local variability	Map local minima and correlate them with cracks and chloride data
Current corrosion probability	10 girder areas; -145 to $+245 \text{ mV}$; all readings > -200 mV	Low at the time of testing	Repeat contour surveys and add resistivity/corrosion-rate testing
Chloride durability	$D_{app} = 1.77 \times 10^{-8} \text{ cm}^2/\text{s}$; nominal initiation = 21.12 years	Potential long-term concern	Obtain depth profiles and recalibrate the diffusion model
CFRP strengthening	Historical installation; present bond condition not fully verified	Uncertain	Conduct tap testing/thermography and selected pull-off testing
Operational loading	Heavy and evolving terminal vehicles	High management relevance	Control vehicle mass, axle configuration, route, and speed

Table 4 demonstrates that no individual test is sufficient to explain the observed deterioration. The core results were above the conservative $f'_c = 50$ MPa assessment basis and had low variability, while the rebound-hammer results supported the absence of generalised low-strength concrete. The ten girder half-cell maps likewise indicated a low conventional probability of active corrosion during the survey because all individual readings were more positive than -200 mV. However, the representative 60 mm cover concealed local readings as low as 20.4 mm, and the chloride model produced a nominal initiation time shorter than the bridge age. The combined evidence therefore supports a mechanically driven recurrent-crack mechanism acting within a chloride-exposed and spatially variable durability environment. Continuing structural demand, reduced cracked-section stiffness, previously active cracks, support behaviour, and uncertain CFRP bond effectiveness are more plausible immediate causes of the observed cracking than generalised low concrete strength or widespread active corrosion. The chloride exposure and local thin-cover zones nevertheless remain important because they can accelerate future deterioration along existing crack paths. Maintenance should consequently prioritise quantitative crack monitoring, direct verification of CFRP bond integrity, repeated electrochemical mapping at fixed grids, targeted chloride-depth profiles, and operational control of heavy vehicles.

3.6. *Proposed maintenance and monitoring programme*

A tiered maintenance programme is recommended. The first tier comprises immediate documentation of crack width, crack activity, CFRP surface defects and approved vehicle configurations. The second tier comprises six- to twelve-month crack and CFRP inspections, with shorter intervals for girders showing measurable crack growth. The third tier comprises periodic electrochemical and chloride reassessment, preferably at consistent locations so that trends can be distinguished from spatial variability. Intervention thresholds should include increasing crack width, new diagonal cracking, evidence of CFRP debonding, potentials shifting into a more negative range over contiguous areas, increased corrosion current density, delamination or spalling.

Repair selection should follow diagnosis. Dormant fine cracks may be re-injected after confirming adequate structural capacity, whereas active cracks require control of load demand, stiffness or support conditions. Local CFRP debonding requires substrate and anchorage evaluation rather than cosmetic patching. Where chloride levels at reinforcement depth are elevated but corrosion remains limited, surface protection and moisture control may be effective; active corrosion may require concrete removal, reinforcement treatment, sectional restoration or electrochemical protection. This condition-based sequence avoids repeated local repairs that do not address the governing mechanism.

3.7. *Study limitations*

The available dataset has limitations that should be addressed in subsequent monitoring. Complete crack-width statistics and time-series measurements were not available; half-cell potential was not accompanied by direct corrosion-rate measurement; the reported $2.48 \mu\text{A}/\text{cm}^2$ beam corrosion current density was model-derived rather than measured; the chloride profile and model parameters require fuller reporting; and the in-situ CFRP bond condition was not quantified. These limitations do not negate the present diagnosis, but they constrain the precision of remaining-life estimates and prevent the CFRP system from being assumed fully effective without verification.

3.8. *Decision criteria for future inspections*

A practical inspection programme requires measurable decision criteria. Crack monitoring should record the same locations, illumination, scale and loading condition at each campaign. For critical positive-moment regions, the database should include crack identification number, width, length, orientation, distance from the nearest support, relationship to CFRP edges and whether the crack crosses a previous injection line. A change in width that exceeds the measurement repeatability should trigger a structural review rather than automatic re-injection. Similarly, CFRP inspection should distinguish superficial coating defects from bond-related anomalies. Hollow sound during tap testing, thermal anomalies, edge lifting and substrate cracking should be mapped and quantified as a percentage of the bonded area.

Electrochemical results should be interpreted as trends. A shift to more negative half-cell potentials over a contiguous area is more meaningful than an isolated reading, particularly when accompanied by low concrete resistivity, increasing chloride concentration or visible moisture paths. The monitoring network should therefore use permanent reference points so that spatial changes can be compared over time. Where the probability of corrosion

increases, linear-polarisation resistance or galvanostatic pulse testing should be used to estimate corrosion current density, and local opening-up may be justified to verify reinforcement condition.

Table 5. Proposed inspection indicators and escalation criteria.

Indicator	Routine record	Escalation trigger	Follow-up action
Crack behaviour	Width, length, orientation and photograph at fixed points	Measurable width growth, new diagonal cracks or crack recurrence after repair	Load review, detailed structural assessment and active-crack treatment
CFRP condition	Tap-test response, edge condition and surface defects	Hollow area, blister, edge lifting or substrate cracking	Thermography/pull-off test and strengthening-system review
Half-cell potential	Contour map at permanent grid points	Progressive negative shift over a connected area	Add resistivity and corrosion-rate measurement
Chloride exposure	Depth profile and surface condition	Increasing chloride at reinforcement depth	Protective treatment or corrosion-control design
Concrete cover	Minimum, mean and spatial variability	Local cover substantially below design expectation	Targeted durability assessment and repair
Vehicle operation	Gross mass, axle spacing, route and speed	Unapproved axle configuration or repeated overload	Immediate operational restriction and load-rating review

The proposed criteria convert condition assessment into an auditable maintenance process. They also reduce the risk that high concrete strength or a single favourable half-cell survey is interpreted as proof of long-term durability. For a working terminal, the monitoring plan should be coordinated with operational records so that crack or electrochemical changes can be related to vehicle frequency, exceptional lifts, collisions, drainage failures or periods of intense marine exposure.

3.9. Practical engineering implications

The integrated assessment framework proposed in this study may be adopted for routine inspection of reinforced-concrete bridges operating in aggressive marine environments. Rather than relying solely on compressive strength or corrosion-potential measurements, maintenance decisions should consider the interaction among crack behaviour, concrete quality, chloride exposure, strengthening-system condition, and operational loading. The framework provides a practical basis for prioritising repair works, defining inspection intervals, coordinating structural monitoring with vehicle records, and directing limited maintenance resources toward the most consequential risks.

4. Conclusions

The integrated assessment indicates that the reinforced-concrete bridge girders retain relatively consistent in-situ compressive strength above the conservative assessment basis. Seven equivalent standard-cylinder core strengths ranged from 51.33 to 59.31 MPa, with a mean of 54.74 MPa and a coefficient of variation of 5.20%; all values exceeded the $f'_c = 50$ MPa value adopted in the structural checks. The rebound-hammer mean of 62.02 MPa supports the absence of generalised low-strength concrete but is treated only as supplementary evidence. Reinforcement detection produced a representative girder cover of 60 mm, although survey-area means ranged from 42.88 to 64.50 mm and individual readings from 20.4 to 68.7 mm. Across ten girder half-cell survey areas, individual potentials ranged from -145 to $+245$ mV and area means from -88.0 to $+206.2$ mV; all were more positive than -200 mV, indicating a low conventional probability of active corrosion at the time of testing. Nevertheless, the chloride-exposed terminal environment remains a long-term durability concern: the apparent diffusion coefficient of 1.77×10^{-8} cm²/s produced a nominal corrosion-initiation time of 21.12 years, which should be interpreted as a monitoring trigger rather than a deterministic failure date. Recurrent predominantly vertical cracks after epoxy injection and CFRP strengthening are therefore more plausibly associated with continuing structural demand, reduced cracked-section stiffness, previously

active cracks, support behaviour, or uncertain CFRP bond effectiveness than with generalised low concrete strength or widespread active corrosion. The bridge should be managed through quantitative crack monitoring, direct verification of CFRP bond condition, repeated electrochemical mapping, targeted chloride-depth testing at locally thin-cover zones, and operational control of heavy vehicles. This study demonstrates that integrating historical, visual, material, electrochemical, and durability evidence provides a more defensible diagnosis than interpreting individual tests separately and offers a practical basis for evidence-based maintenance planning.

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